MEASURE AND USE OF THE INSITU LATERAL STRESS

by

John H. Schmertmann[1]

SYNOPSIS -- Twenty examples show that a wide and often unexpected range in soil lateral stress (K) conditions can occur insitu. At least 8 common geotechnical problems have the insitu K condition as a major variable, and at least 17 methods exist for measuring and profiling K. Conservatism, economy, satisfaction, and sometimes safety all suggest the routine measurement and use of insitu K data.

INTRODUCTION

Professor Osterberg occasionally taught his students, including the writer, by telling entertaining stories about his geotechnical experiences. These stories often included the message that many geotechnical failures result not from some misunderstood or miscalculated detail, but rather from overlooking or not understanding a major factor in the site investigation, design or construction. This former student believes that engineers may often and needlessly run the risk of overlooking a possible major factor -- an unexpected low or high or variable lateral stress condition.

This paper tells or retells a simple story: lateral stress conditions can vary unpredictably over a wide range, they represent a key site condition variable which engineers should consider in their investigations and analyses, we now have practical and adequate methods for measuring lateral stresses, and therefore we ought to measure and use them routinely. Failure to measure and use the insitu lateral stress can easily result in unconservative or uneconomical design.

A study of the list of over 60 references cited herein, plus their references, etc., will soon demonstrate to anyone interested that the geotechnical literature contains a huge amount of scattered information about lateral stress effects. A sense of order and purpose needs to be established. Its importance in geotechnical engineering practice needs to be focused. But why now? Because we can now do something practical about measuring and using lateral stresses. This paper presents some new information, but mostly compiles some of the existing information on lateral stress to focus on this one part of geotechnical site investigation and design.

Because engineers can ordinarily calculate the vertical stress with relatively small error, the writer uses the common convenience of normalizing

---

1 Principal, Schmertmann & Crapps, Inc., Consulting Geotechnical Engineers, Gainesville, FL, USA

Reprinted from:
THE PRACTICE OF FOUNDATION ENGINEERING
- a Volume honoring Jorj O. Osterberg
Published by: The Department of Civil Engineering, Northwestern University, 1985.

the lateral stress condition by using a K-factor by which to multiply the vertical effective stress to get the horizontal effective stress. The commonly used K_o designation means, in this paper, the K condition for the special case of normally consolidated or simply overconsolidated and at rest (undisturbed by man-induced deformation) condition in a large soil mass with a horizontal and unloaded surface. Sometimes K will equal K_o, but generally not. Also the writer uses "lateral stress" and "horizontal stress" synonymously and sidesteps the occasional complication of a sloping ground surface. Symbols and abbreviations are defined where first used.

K VARIABILITY

This section has the objective of demonstrating that attempts to guess K conditions at a site can lead to very poor results. For subsequent reference, Table 1 contains 20 items which document a 0.2 to 6.4 range of values for K as found insitu either by personal measurement or as reported in the literature.

The NC and OC K_o Condition

Normally consolidated (NC) soils, both sands and clays, have a rather narrow range for K_o. The well known Jaky equation of $K_o = 1 - \sin \phi'$ predicts a factor of 2 range of about 0.35-0.7 for the range $\phi' = 17\text{-}40^o$. Items 1 to 4 in Table 1 present examples of measured NC (or nearly NC) K_o values in clays as do items 15-18 for sands, and they do indeed vary approximately over this range.

Simple overconsolidation (OC) by the removal of overburden can greatly increase K_o. A simple expression that includes the overconsolidation ratio (OCR), such as $K_{oOC}/K_{oNC} = OCR^x$, has been proposed by a variety of engineers with the suggested exponent varying from about 0.4 to 0.5 in all soils. Several researchers have reported high K_o values in London clay which they believe resulted from simple erosion unloading. This erosion leads to high OCR and K_o values near the clay surface, which then diminish with depth. Items 8 to 10 in Table 1 give examples and show the reported K_o values in London clay as high as 4.4.

Mayne and Kulhawy (1982) have proposed an empirical formula to account for OCR effects on K_o after one cycle of unloading and partial reloading. In general the more complicated the loading and unloading history of deposit the more difficult for the engineer to predict K_o.

Active and Passive Lateral Stresses

The active and passive states of stress represent the lateral stress conditions for the stability limit when a vertical face moves away from and towards the soil, respectively. The classical Rankine equations give a range of K_a to K_p of 0.22 to 4.6 for a 40^o friction angle. For soils with true bond cohesion, or cementation, K_a can decrease to zero and K_p can reach very high values with increasing strength of the bonds. Theoretically at least, the ratio of maximum to minimum K conditions possible in the field now goes to infinity.

Soil Swelling Effect

Some active clay minerals have a strong tendency to expand or contract when water equilibrium conditions change. The literature now contains a number of examples of high K conditions believed due to the swelling of clay minerals, with pressures sometimes approaching passive values. Items 5, 6, 12 and 13 in Table 1 present examples of shrink-swell effects on K, with values as high as 6.4 at a depth of 20-40 ft. Some of the authors attribute the high K conditions to the sequence of dessication, clay cracking, the infilling of cracks with other materials during the time they remain open, and then subsequent inundation and the reswelling of the clay. The clay can reswell freely in a vertical direction but the filled cracks now restrain lateral swelling. High K conditions apparently can result from the thwarted swelling.

High K from Rock Weathering

Those engineers who work with rock mechanics know well that high K conditions can, and often do exist in rock masses. For example, Haimson (1973) reported K values at three sites that varied from 3.5 to 1.7 at depths of 325 to 2650 ft. High K conditions in rock can result from the erosion of overburden, the diagenetic swelling of minerals in the rock, tectonic action, and probably a variety of other causes. Bjerrum (1967) explained how the diagenetic weathering of rocks, especially clay-shales, breaks the bonds within the rock and releases the strain energy stored in the bending of the clay particles in the rock. The weathering release of this stored bending energy creates high K conditions if lateral movement is restrained. If movement can occur, then there may be a full or partial release of the high K condition until further weathering causes the cycle to repeat.

The above may also occur in relatively soft rocks that engineers may still consider "soil". In residual soil areas, and especially in the zone of transition between the residual soil and the weathering parent rock, K conditions may be very variable and range from low in the completely decomposed rock after the release of any stored strain energy to high K in only partly decomposed rock if the rock itself had a high-K condition prior to weathering. The data in Figure 1, especially from the MPMT, illustrate the probable increase in K in a micaceous, sandy silt residual soil profile when approaching the intact schist parent rock. This figure also shows the induced K-increase near the surface from excavation and compaction by loaded pans and other construction equipment rolling across the site.

K may also increase due to weathering, but without the release of stored strain energy. For example, Kane (1973) showed in lab samples how the destruction of cement bonds by shear in a loose-structured, cemented loess caused K to increase from 0.23 to 0.54. Weathering, or some other action (such as wetting in a loess), could reduce shear strength to the point of an internal shear failure with a resulting increase in K.

K Anisotropy

Although commonly assumed, engineers have little or no justification for taking the K condition in the field as isotropic. More likely, natural or man-made events will make them anisotropic. Haimson (1973) reported anisotropic K_x/K_y ratios of 1.5 to 3.2 and appears to measure and report horizontal stress anisotropy in rock as a routine matter. Dalton and Hawkins (1982) (also item 11 in Table 1) reported measuring an anisotropic K ratio of 1.8 using self-boring pressuremeter testing (SBP). To provoke thought -- perhaps this happened here or elsewhere due to the movement direction of the glaciers that overrode the site?

The writer can imagine other causes for anisotropic K. For example, a cut or adjacent erosion will reduce the K condition on vertical planes facing the cut or line of erosion but will have much less of an influence on perpendicular planes. Seepage should produce anisotropic K, as does directional roller compaction (see Table 1, items 7 and 19).

A recent advertisement by the Cambridge Insitu Company (1984) featured the separate measurement of the x and y-components of K, which demonstrates their support for the idea of anisotropic K. When possibly important, we should use measurement methods that give directional K results.

Misc. Natural Effects

One can imagine various natural events in addition to those already described which might have an effect on the local K-condition. For example, one might expect that the formation of cavities in the overburden, such as associated with the solution of limestone and the formation of sinkholes, will cause a reduction in the K-condition adjacent to such cavities or adjacent to the ravelled infill that eventually fills such cavities. Item 14 in Table 1 notes a low-K condition around infilled cavities in a very sinkhole-prone area near Orlando, FL. Dessication, as by tree roots in clay, would presumably be another cause of local low-K conditions. The soil adjacent to tension cracks, caused either by dessication or slope deformations, would obviously produce adjacent relatively low K conditions. Item 5 in Table 1 gives an example. At the bottom of a slope one would expect a higher than normal K-condition on planes perpendicular to the slope, while the lower than normal could occur near the top of the slope.

The above misc. natural K-effects have either been experienced by the writer or he has imagined their possible occurrence. The reader can probably add to the list. The important point is that a large variety of natural events can subtly or dramatically alter an actual site K-profile from any simple condition an engineer might assume.

Man Induced Effects

Of course, man can sometimes dramatically change the K conditions at a site. Surcharging will likely produce irreversible increases in K after the removal of the surcharge. An excavation will obviously reduce K along its sides. It will also increase K below the excavation -- as in Table 1, item 17. Leaking pipes may produce artificial cavity and sinkhole conditions with their associated K effects. The movements of walls will effect the K conditions adjacent to them. Compaction activities also affect K. Items 18 and 19 in Table 1 present documented examples of K increasing in sand after the passage of a vibratory roller -- item 19 with an anisotropic effect. Table 2 (see section on Evaluating Ground Treatment) presents some flat dilatometer test (DMT) results from a site treated with dynamic compaction.

They demonstrate an increase in K to an average limit of about 1.3. The driving of displacement piles will also increase K in their vicinity, sometimes to the level that it becomes difficult or impossible to drive additional piles within a group. Compaction grouting has as one of its primary objectives the increase in K to enhance the stability of the site. The reader can no doubt add to these examples.

K Can Change With Time

To complicate the problem of predicting a site K-condition even further, the reader will recognize that most of the aforementioned factors that influence K can and will change with time. Changing moisture conditions will cause clays to shrink or swell and vegetation to grow or die. Continuing erosion on the surface can induce slope movements and both the erosion and associated deposition will have local K effects. Subsurface erosion eventually causes sinkholes with their K-effects. The weathering process continues relentlessly and it changes K. Buried organic material, whether natural (such as trees) or man-made (such as wooden piles) will eventually decay or rot and influence the local K environment. Even the natural process of soil aging probably changes K. Once again, the reader can probably add to the list.

The Need to Measure K

Hopefully the above adequately documents the great variability of insitu K which can result from a multitude of possible causes and effects. Furthermore, these will seldom act singularly but rather with complicated interactions. All of this argues that the K conditions at any site may be essentially unpredictable and to determine them requires site specific measurements. As an example, consider the well documented profile of horizontal stress presented in Figure 2 by Handy et. al. at a site near Houston. The initial SBP data outlined the variability, and the subsequent K_o stepped blade data filled in the details. Could any engineer have confidently guessed such a variable profile? Probably no one suspected the true variability of horizontal stress and K at this site.

K AND ENGINEERING PERFORMANCE

The aforementioned variability and unpredictability of K at any given site might be interesting, but of minor consequence, if the K condition has only a minor influence on engineering design and performance. Unfortunately, the K condition has a very important and sometimes even critical effect as the following subsections will review:

Bearing Capacity

Most engineers know of cases where adjacent trench excavations have "undermined" footings and produced a bearing capacity failure. But, such undermining may only reduce K and thus trigger the failure. One can easily reason that the K condition will have a major influence on surface bearing capacity in non-cemented soils. A bearing capacity failure usually involves some lateral displacement of the soil. If the K condition approaches K_a the soil will offer very little resistance to lateral movement and therefore a low bearing capacity. On the other hand, a K condition approaching K_p will offer a large resistance to lateral movement and thus a much higher bearing capacity. Figure 3 shows the results from field bearing capacity experiments in sand using a 1.0 ft^2 plate surrounded by air bags in vertical trenches to provide the controlled horizontal stress condition. These results clearly show the large difference in bearing capacity, and also deformation modulus, that results from changing the surrounding K condition.

The bearing capacity theory by Durgunoglu and Mitchell (1975) includes the effects of the K condition for the first time in a theory that has enjoyed broad recognition. Schmertmann (1982) and Schmertmann and Crapps (S&C) (1983) have used this new theory to interpret DMTs and static cone penetration tests (CPTs) in sand, as have others.

The K condition in uncemented sands essentially controls the results obtained from such insitu tests as the static cone penetration test (CPT) and the standard penetration test (SPT). All the researchers working with the CPT in large, controlled stress, calibration chambers have found the K condition virtually determines the CPT bearing pressure q_c. As one example, Figure 4 shows chamber test data from Veismanis (1974) illustrating how K dominates q_c. Schmertmann (1979) also argued that SPT N-values depend significantly on K. The test results from the surface 10 ft in Fig. 1 illustrate this dependence. It seems very likely that the results from any dynamic or static insitu bearing penetration test will depend in an important way on the K condition existing at the time of the tests.

The reverse bearing capacity, as experienced in anchor pullout tests, also appears largely controlled by lateral stress conditions. For example, Clemence and Pepe (1984) found that the initial K condition, plus the increase in local K surrounding an anchor as a result of the uplift pull on it, have a dominant effect on its pullout resistance in sand.

From the above review it seems clear that K has a major effect on a soil's drained bearing capacity when its strength depends on effective stress.

Slope Stability

A number of engineers have found, particularly when dealing with cuts, that the K condition can have a dominant effect on not only the limit equilibrium factor of safety but also on the mechanism of deformation and the geometry of the failure of a slope. Bjerrum (1967) showed that the weathering destruction of bonds in hard clays and soft rocks released the recoverable strain energy in bent particles. This release can cause progressive movements, which in turn can result in reaching residual strength conditions and a possible failure. He argued that high K conditions accelerated this progressive action and also any creep effects. Gould (1970) presented a case history that involved work by Shannon and Wilson, Inc. and R. B. Peck wherein a high insitu K condition controlled the choice of the type of slope retaining structure used to protect a highway cut.

Duncan and Dunlop (1969) in a series of early finite element method (FEM) studies of cut slope behavior presented examples that showed that doubling K produced a doubling of the maximum shear stress in the cut. In a further study Dunlop and Duncan (1970) showed how a high K condition causes the progressive failure of a cut to begin at the toe and progress backwards, while with a low K condition the failure starts at the crest and progresses downward to the toe. Brown and King (1966) showed how the high K condition causes the potential failure surface to move much deeper into the slope than with a low K condition. All of the above investigations assumed ideal elastic-plastic, frictionless soil material. Lo and Lee (1973) used the FEM to model strain softening in frictionless soil and found in one example analysis that increasing K from 1 to 2 decreased the factor of safety from 1.45 to 1.16.

It seems clear that for slope stability problems, at least for cuts, the methods of analysis need to include the K condition as one of the key input parameters.

Hydraulic Fracturing in Dams

Deliberate hydraulic fracturing is one of the methods used for measuring insitu K. As mentioned previously, Haimson (1973) used this technique in rocks. The subsequent section on METHODS FOR MEASURING K gives additional references for the use of this technique for soils both in the laboratory and insitu. The lower K, the easier for hydraulic fracturing to occur. A potentially serious problem arises when an embankment dam deforms, say by differential settlement of its foundation, and arches in such a way as to produce a low-K condition within the embankment. Then, subsequent hydraulic pressures and seepage forces might produce hydraulic fracturing within this portion of the dam and lead to serious problems and even failure. Sherard et. al. (1972) discuss actual failures that occurred by this mechanism. According to Arthur (1976), at least one of the hypotheses for the failure of the Teton Dam has hydraulic fracturing as a principle cause of the failure.

From the above it would seem that K-profile measurements might be of particular value when determining the factor of safety against hydraulic fracturing within embankment dams.

Pressures on Walls

Because walls usually have to resist horizontal stresses, the existing K condition should have a major influence on wall design. Of course the actual pressure against a wall will vary with its geometry, method of construction, ability to deflect under load, and the associated wall support conditions. It does seem obvious, however, that the preexisting K condition will have an important effect on the stresses that will eventually act on any wall, and thus on the its design. The Soil Mechanics and Foundation Division of the ASCE (1970) devoted an entire specialty conference to this subject and the papers therein discuss the importance of the preexisting K condition.

From the slope stability discussion, and considering a wall as ordinarily supporting a near-vertical slope, it follows that walls supporting cuts should be designed for K. The horizontal deflection of a wall depends approximately linearly on K, as shown by Dibiagio (1966) in Figure 5 . Thus, with higher K conditions the designer will have to either permit greater wall deflections to reduce stresses, or accept higher stresses on the wall in return for less deflection.

For the case of fills behind walls, the method of filling and the soil used in the filling can dominate the stresses on the wall. As previously discussed, the compaction of backfill will increase the K condition. Pufahl et. al. (1983) have shown that pressures against structural members cast directly against undisturbed, swelling clays can reach passive pressures upon saturation. They also demonstrated in the laboratory that a compacted clay can also reach passive failure in one-dimensional swelling, and presumably could do so behind a wall if the wall could not deflect adequately.

For the special case of cellular cofferdams and bins to contain particulate materials, the K condition again obviously controls the design. Figure 6 shows the results of MPMT and DMT measurements for K in a large cellular cofferdam. These pressures were considerably in excess of what the engineers expected from $K_o = (1-\sin\phi) = 0.40$ in this coarse river sand and gravel dumped through water to fill the cell. K increasing to about 0.6 in the mid and lower part of the cell fill fits well with Handy's (1985) explanation of how arching effects increase K within a soil sheared vertically and restrained horizontally by a wall. The K-profile in Figure 6 therefore suggests that significant arching may have occurred within this cofferdam cell.

Stress Distribution in Soils

Harr (1977, p. 222), has made a highly original and convincing argument that the preexisting K condition has a major effect on the distribution of stresses within a particulate soil mass. He based his reasoning on probabilistic rather than theory-of-elasticity concepts. His results appear more logical for a particulate mass, and conform better to various research measurements and field experience than those obtained from the widely used elastic solutions. The writer has used Harr's equations to prepare Figure 7, which illustrates the effect of K on the vertical stress increase distribution under the center of a uniform and parabolic circular loading on the surface of a particulate mass. According to these concepts, knowing the K condition is a requirement for the accurate computation of stress distribution.

Pile/Shaft Skin Friction

Friction piles and drilled shafts are common, and by definition derive all or most of their support from side friction. Tapered piles also typically have a high side friction component. Even so-called end bearing piles often behave as friction piles under their working loads.

Engineers generally calculate side friction under drained conditions by multiplying the vertical stress by the lateral stress coefficient they believe will act to produce the pile/shaft friction, here denoted K_f. They then multiply K_f by a soil/pile friction coefficient to get skin friction capacity. It seems logical to suppose that the typical high lateral stresses surrounding a driven displacement pile will diminish with time toward the preexisting K condition. In the other direction, drilled shafts placed in a high-K environment will probably have K_f gradually increase with time. Although K_f should move up or down towards K, it will probably never reach K. As an example of using these concepts in design, Schmertmann & Crapps (1983) assumed for a drilled shaft that K_f with time would approach the average of the initial K after pouring concrete and the insitu K of about 4 (Table 1, item 12). This approach seemed satisfactory when compared with load test results.

Settlement and Deformations

All geotechnical engineers appreciate that upon loading a preconsolidated soil will settle less than if not preconsolidated. The preconsolidation point represents the stress condition at which the soil structure yields, and thus becomes more plastic and compressible. But, the magnitude of the vertical stress at yielding (the effective preconsolidation stress) depends on the initial K condition. The yield envelopes used in critical state soil mechanics (see Schofield and Wroth (1968) for examples) demonstrate this dependence on K.

Engineers also generally appreciate that the soil modulus, whether the one-dimensional M, Young's E, or some other modulus, depends to an important degree on the preexisting effective stress conditions. Most constitutive equations use the average of the three principle effective stresses to control the magnitude of various moduli. Because the K stresses constitute two of the three averaged stresses, they are in this sense twice as important as the vertical stress and thus can dominate a deformation problem.

Poulos and Davis (1972) presented an example of the importance of K to obtain the proper E from lab tests. E for kaolin varied by a factor of about 4 when K varied by a factor of about 2. Simons and Som (1969) presented an example showing a reduction in E by a factor of 2.4 when not taking account of K in lab stress path testing. These authors (their Fig. 7) also followed Lambe (1964, eqn. 7) in pointing out that the K-condition has a major influence on the ratio of vertical/volumetric strain and therefore settlement predictions. Harr (1977, p. 334) has also shown how increasing K can greatly reduce settlement. As noted previously, the K condition is especially important for the evaluation of lateral deformations. Figure 5 illustrated this from one FEM analysis.

Liquefaction Potential

Considering all the above, it seems reasonable to anticipate that the K condition would affect not only static stability and deformations, as measured by such behavior as bearing capacity and modulus, but also cyclic and dynamic loading stability as measured by resistance to liquefaction. Seed and Peacock (1971) did indeed find that the potential for liquefaction reduced dramatically as K increased. Therefore, any laboratory tests for liquefaction potential must begin with the proper K condition.

As further evidence of the importance of K to liquefaction, Robertson and Campanella (1984) have proposed a preliminary K_D-liquefaction correlation. See Marchetti (1980) for a description of how he uses K_D from the DMT as an index for K.

When evaluating liquefaction potential using insitu penetration tests such as the SPT, these tests themselves reflect the importance of K in the same direction, but not necessarily to the same degree, as their importance to liquefaction. It would be of value to determine how much of the liquefaction resistance comes from the K condition vs. other effects (relative density, dynamic prestraining, cementation, etc.). After all, as discussed previously, K may reduce and therefore liquefaction potential may increase significantly with events and time.

Evaluating Ground Treatment

Various types of ground treatment increase the K condition as one of their important effects. As already mentioned, vibratory roller compaction, dynamic compaction, surcharging, and compaction grouting all have the beneficial effects of increasing K. The reader can probably add others. However, the potential for increasing K probably depends on the magnitude of the insitu K condition to begin with. For example, the data shown in the following Table 2 indicate that Test Section 1 already had a such a high K condition that the dynamic compaction effort used could not raise it further.

Table 2 - Ave. DMT Results from before and after Dynamic Compaction in a Test Area, using 33 ton weight dropping 105′ (all tests in approx. center between prints 24′ apart)

In Test Section	No. drops	depth interval	No. Tests	K before	K after
1	2	3-20′	26	1.30	1.34
2	6	5-24′	30	0.66	1.17
3	6	6-27′	32	0.98	1.19

One can also argue that engineers should measure the after-treatment K so they will know how much of the improvement (say in modulus) results from improvement in K. It remains possible that subsequent developments may reduce K and release some or all of this K-related improvement. Knowing the before and after treatment K conditions should help designers to at least consider the consequences of such potential future loss of K.

Evaluation of Lab and Field Tests

The results of almost all laboratory and field tests for engineering parameters and performance depend on the K condition. This has already been discussed in some detail for the CPT, SPT, preconsolidation (yield) stress determination, modulus values, and liquefaction testing. Any type of triaxial stress path testing, for example Lambe (1964), requires choosing lateral stress conditions for the testing. It seems apparent that laboratory tests which attempt to model field behavior either must match the field K condition or correct for any mismatch. Both require knowledge of K.

The interpretation of field tests requires K to separate the various components that contribute to the overall test result. For example, q_c from the CPT in sands results primarily from friction angle strength and the magnitude of the effective lateral stresses. Thus, to evaluate Ø (or relative density) requires knowing K. See Schmertmann (1982) for one discussion of their separation. Figure 1 shows another example where the high-K condition in the surface 10 ft accounts in part for the higher SPT N-values in this 10 ft. Part of the increase also results from the increased density from compaction.

Reading K Profiles

The discussion thus far in this section of the paper has only considered using K to help solve specific geotechnical engineering problems. It seems likely that K profiles at a site may reveal information about its history or current behavior that will allow the engineer to better understand the overall site conditions. Measuring K then becomes part of the general site exploration rather than information obtained to solve specific problems on the site. The profiles in Figures 1 and 2 provide some examples. The relatively low-K plateau of about 0.5 in the central portions of the profile in Figure 1 suggest that this residual soil has been thoroughly weathered above about the 30 ft depth. The high-K "crust" at the surface suggests excavation and/or surface compaction effects. The possible increase in K at the bottom of the profile suggests the weathering transition to the parent rock, which itself probably has a high-K condition. The horizontal stress profile in Figure 2 shows a major and minor peak which suggested to the writers previous dessication cracking, infilling and swelling behavior at these levels. The high stress near the present ground surface was thought to represent the current production of horizontal expansion stresses.

As mentioned previously, Marchetti (1980) uses a K_D lateral stress index parameter, which he then relates to K in uncemented soils. He routinely profiles K_D to help him understand the history of a site. Others have also found profiling K_D to be useful. For example, Lutenegger and Donchev (1983) used K_D profiles to help locate metastable layers within loess deposits.

Summary

Many years ago Skempton (1961, p. 351) said "Yet without a knowledge of K_o the stress state of a particular soil cannot be properly defined nor can a number of practical problems be solved". From the topics in this section of the paper it seems abundantly clear that the K condition can constitute a key variable in many and perhaps most geotechnical analyses. Engineers should therefore treat it as a potential key site condition and measure, evaluate and use K just as they would any other site condition of possible major importance. Engineers now have at least 17 methods to measure K, as discussed in the next section.

METHODS FOR MEASURING K

Table 3 lists 7 laboratory methods, and Table 4 lists 10 field methods for the evaluation of the insitu K condition, making a total of at least 17 available methods. In general the laboratory methods were developed earlier and for use with cohesive soils and the field insitu methods came later for use in a greater variety of soil types. Abdelhamid and Krizek (1976) also presented a list of methods to measure K that includes some methods not in Tables 3 and 4. The reader may well know of still other published or unpublished methods.

The scope of this paper does not include discussing in detail the various capabilities and limitations of the different methods. Rather, consider a few general comments that might help you decide on suitable methods for your job situations.

The laboratory methods suffer from the common problem of uncertain effects from sampling disturbance of the soil structure. For example, any insitu aging effects might be lost in the sampling and trimming operations. The difficulties of getting adequate samples from cohesionless deposits are also well appreciated. Some of the laboratory methods involve the tacit assumption that the soil will behave during an increment of laboratory compression in the same way it behaved during its last increment of field compression. On the other hand, the engineer may have the lab samples available anyway and the tests for K may involve little extra effort -- especially if convenient equipment is also readily available, such as the K_0-cell by Campanella and Vaid (1972).

Most of the field methods have the advantage of speed and economy, which may also allow the profiling of horizontal stress (and K) as in Figure 2. They have the inherent disadvantage of requiring the insertion of some type of instrument which has at least the potential for changing the K condition to be measured. Here the philosophy varies: Some instruments such as the SBP attempt to achieve insertion with zero disturbance. Others like the MPMT and Menard Geocell attempt insertion with a minimum but unkown amount of disturbance. Still others such as the spade, DMT and K_0 stepped blade induce a fixed and reproducible disturbance with the objective to extrapolate or correlate to the pre-insertion insitu condition.

When evaluating these various K tests against ground truth the question always arises as to what should one take for ground truth? Engineers usually compare the results of these methods with one another, or to some other value they believe better matches ground truth. In the writer's experience, no single method is always superior to any other method. When used appropriately they can all produce reasonable results superior to and more informative than non-measured guesses or estimates.

Tables 3 and 4 also include references from which the reader may learn about the wide range of suitable conditions for and advantages and disadvantages of the test methods to measure K.

SUMMARY & CONCLUSIONS

1. The K conditions at a site can dominate soil engineering behavior and design analyses, and significantly affect safety. Table 5 summarizes the effects of K in most of the geotechnical analyses discussed in this paper:

TABLE 5 - QUALITATIVE EFFECTS OF CHANGING K ON ENGINEERING BEHAVIOR
(arrows show direction of usual less conservative behavior)

Engineering Behavior	Insitu: Low K — High K
Bearing capacity	← safety decreases
Slope stability	safety decreases →
Fracture of dams	← safety decreases
Pressure on walls	increases →
Pile friction	← decreases
Settlement/deformation	← increases
Liquefaction	← safety decreases
Ground treatment improvement	more difficult →

2. Insitu soil K conditions vary over a range greater than 0.2 to 6 -- a ratio of over 30. The variation may be considerable, not only between sites but also within a site or along a single profile. Anisotropic K conditions may also exist. Many natural and man-made factors affect K in a particular soil layer. Thus, without measurements the engineer may have a difficult task to make a reasonable estimate of K conditions at a site.

3. The K conditions at a site can control the design and/or interpretation of laboratory and field test results.

4. Vertical profiling of K may add significantly to understanding the geotechnical history and conditions at a site.

5. A variety of laboratory, and particularly field methods now exist for evaluating insitu K conditions. A suitable and practical method exists for most geotechnical problems in soils finer than gravel.

The writer suspects Mother Nature has a bag full of surprises for us when it comes to insitu K conditions. Some of these will be pleasant and others unpleasant in their consequences. Reduce the unpleasant possibilities, and at the same time have the enjoyment of understanding our sites better, by measuring and then using K on a routine basis.

REFERENCES

ABDELHAMID, M.S. and KRIZEK, R.J. (1976), "At-Rest Lateral Earth Pressure of a Consolidating Clay," ASCE Journal of the Geotechnical Division, Vol. 102, No. GT7, July, pp. 721-738. (see Table 1)

AL-SHAIKAH-ALI, M.M.H. (1977), "A Method of Determining the Insitu K_o in the Laboratory," written contribution to the Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, p. 349.

AL-SHAIKAH-ALI, M.M.H., DAVIS, A.G. and LLOYD, F.J. (1981), "Insitu Measurement of K_o in a Stiff Fissured Glacial Till by Hydraulic Fracturing," Ground Engineering, Vol. 14, No. 1, January, pp. 19-25.

ARTHUR, H.G. (1976), "Teton Dam Failure," ASCE Engineering Foundation Conference Proceedings on The Evaluation of Dam Safety, Pacific Grove, CA, pp. 61-71.

ASCE SOIL MECHANICS AND FOUNDATIONS DIVISION (1970), Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures, Cornell University, June, Proceedings Vol., 328 pp.

BAGUELIN, F., JEZEQUEL, J.F. and SHIELDS, D.H. (1978), The Pressuremeter and Foundation Engineering, Transtech Publications, Germany.

BLIGHT, G.E. (1970), "In Situ Strength of Rolled and Hydraulic Fill," ASCE Journal of the Soil Mechanics and Foundation Division, Vol. 96, No. SM3, May, pp. 881-899. (See Figure 6).

BISHOP, A.W. (1958), "Test Requirements for Measuring the Coefficient of Earth Pressure at Rest," Proceedings, Brussels Conference on Earth Pressure Problems, Vol. 1, pp. 2-14.

BISHOP, A.W. and HENKEL, D.J. (1957), "The Measurement of Soil Properties in the Triaxial Test, Edward Arnold, London.

BJERRUM, L. (1967), "Progressive Failure in Slope of Overconsolidated Plastic Clay and Clay Shells," The 3rd Terzaghi Lecture, ASCE Journal of the Soil Mechanics and Foundations Division, Vol. 93, No. SM5, Sept., pp. 1-49.

BJERRUM, L. and ANDERSEN, K.H. (1972), "Insitu Measurement of Lateral Pressures in Clays," Proceedings of the 5th European Conference on Soils Mechanics and Foundation Engineering, Madrid, Vol. 1, pp. 11-20 (reprinted in NGI No. 91).

BJERRUM, L., NASH, J.K.T.L., KENNARD, R.M., and GIBSON, R.E. (1972), "Hydraulic Fracturing in Field Permeability Testing," Geotechique, Vol. 22, No. 2, pp. 319-332.

BLIGHT, G.E. (1967), "Horizontal Stresses in Stiff and Fissured Lecustine Clays," Proceedings of the Fourth Regional Conference for Africa on

Soil Mechanics and Foundation Engineering, December, pp. 95-99.

BLIGHT, G.E. (1969), "Foundation Failures of Four Rock Fill Slopes," ASCE Journal of the Soil Mechanics and Foundation Division, Vol. 95, No. SM3, May, p. 743.

BROWN, C.B. and KING, I.P. (1966), "Automatic Embankment Analysis: Equilibrium and Instability Conditions," Geotechnique, Vol. XVI, No. 3, September, pp. 209-219.

CAMBRIDGE INSITU (1984), Advertisement in Geotechnique, Vol. XXXIV, No. 4, p. A2.

CAMPANELLA, R.G. and VAID, Y.P. (1972), "A Simple K_o Triaxial Cell," Canadian Geotechnical Journal, Vol. 9, pp. 249-260.

CHANG, M.F., MOH, Z.C., LIU, H.H., and VIRANUVUT, S. (1977), "A Method for Determining the In Situ K_o Coefficient," Proceedings 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Vol. 1, pp. 61-64.

CLARKE, B.G., and WROTH, C.P. (1984), "Analysis of Duton Green Retaining Wall Based on Results of Pressuremeter Tests," Geotechnique, Vol. 34, No. 4, pp. 549-561.

CLEMENCE, S.P. and PEPE, F.D. Jr. (1984), "Measurement of Lateral Stress Around Mulihelix Anchors in Sand," ASTM Geotechnical Testing Journal, Vol. 7, No. 3, September, pp. 145-152.

COOK, W.H. (1971), "A Field Study of Effect of Compaction and Changes in Lateral Stress on Buried Rigid Plate Load Tests in Sand," Masters Thesis, Department of Civil Engineering, University of Florida, 182 pp.

DAHLBERG, R. (1974), "Penetration, Pressuremeter, and Screw-Plate Test in a Preloaded Natural Sand Deposit," Proceedings of the European Symposium on Penetration Testing (ESOPT), Vol. 2:2, pp. 69-87. (see Fig. 10)

DALTON, J.C.P., and HAWKINS, P.G. (1982), "Fields of Stress — Some Measurements of the In-Situ Stress in a Meadow in the Cambridgeshire Country Side," Ground Engineering, Vol. 15, No. 4, May, p. 15.

D'APPOLONIA, D.J., WHITMAN, R.V., and D'APPOLONIA, E. (1969), "Sand Compaction With Vibratory Rollers," ASCE Journal of the Soils Mechanics and Foundation Division, Vol. 95, No. SM1, January, pp. 263-284 (see Fig. 19).

DIBIAGIO, E. (1966), University of Illinois Ph.D. dissertation, as presented by Morgenstern, M.R. and Eisenstein, Z. in "Methods of Estimating Lateral Loads and Deformations," ASCE Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures, Cornell University, 1970, p. 62.

DMT DIGESTS, prepared and distributed by G.P.E., Inc., Gainesville, FL, J.H. Schmertmann, Editor.

DUNCAN, J.M. and DUNLOP, P. (1969), "Slopes in Stiff-Fissured Clays and Shells," ASCE Journal of the Soil Mechanics and Foundations Division, Vol. 95, No. SM2, March, pp. 467-492.

DUNLOP, P. and DUNCAN, J.M. (1970), "Development of Failure Around Excavated Slope," ASCE Journal of the Soil Mechanics and Foundations Division, Vol. 96, No. SM2, March, pp. 471-493.

DURGUNOGLU, H.T. and MITCHELL, J.K. (1975), "Static Penetration Resistance of Soils, I-Analysis, II-Evaluation of Theory and Implications for Practice," ASCE Specialty Conference on the In Situ Measurement of Soil Properties, Raleigh, Vol. I, pp. 151-189.

GOULD, J.P. (1970), "Lateral Pressures on Rigid Permanent Structures," ASCE Proceedings of the Specialty Conference Lateral Stresses in the Ground and Design of Earth-Retaining Structures, Cornell University, pp. 219-269. (See pp. 262-265)

HAIMSON, B.C. (1973), "Simple Method for Estimating In-situ Stresses at Great Depths," ASTM Symposium on Field Testing and Instrumentation of Rock, Philadelphia, STP 554.

HANDY, R.L. (1985), "The Arch in Soil Arching," ASCE Journal of the Geotechnical Engineering Division, Vol. 111, No. 3, March.

HANDY, R.L., REMMES, B., MOLDT, S., LUTENEGGER, A.J., and TROTT, G. (1982), "In Situ Stress Determination by Ohio Stepped Blade," ASCE Journal of the Geotechnical Division, November, p. 1415.

HARR, M.E. (1977), Mechanics of Particulate Media, McGraw Hill International Book Company.

JAMIOLKOWSKI, M., LADD, C.C., GERMAINE, J.T., and LANCELLOTTA, R. (1985), "New Developments in Field and Laboratory Testing in Soils," Preprint, Theme Lecture prepared for XI International Conference on Soil Mechanics and Foundation Engineering, scheduled for San Francisco, 1985, 97 pp.

KANE, H. (1973), "Confined Compression of Loess," Proceedings of the 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, Vol. 2.2, pp. 115-122.

KENNEY, T.C. (1967), "Field Measurements of In Situ Stresses in Quick Clays," Proceedings of the Geotechnical Conference Oslo, 1967, Vol. 1, Reprinted in Norwegian Geotechnical Insitute Publication No. 76, pp. 49-55.

LAMBE, T.W. (1964), "Methods of Estimating Settlement," ASCE Journal of the Soil Mechanics and Foundations Division, Vol. 90, No. SM5, September, pp. 43-67.

LO, K.Y. and LEE, C.F. (1973), "Stress Analysis and Slope Stability in Strain-Softening Materials," Geotechnique, Vol. XXIII, No. 1, March, pp. 1-11.

LUTENEGGER, A.J. and DONCHEV, P. (1983), "Flat Dilatometer Testing in Some Metastable Loess Soils," International Symposium on In Situ Testing, Paris, Vol. II, pp. 337-340.

MARCHETTI, S. (1980), "In Situ Test by Flat Dilatometer," ASCE Journal of the Geotechnical Division, Vol. 106, No. GT 3, March, pp. 299-321. See also Discussion and Closure in Vol. 107, No. GT6, June, 1981, pp. 831-837.

MASSARSCH, K.R. (1975), "New Method for Measurement of Lateral Earth Pressures in Cohesive Soils," Canadian Geotechnical Journal, Vol. 12, No. 1, February, pp. 142-146.

MASSARSCH, K.R., HOLTZ, R.T., HOME, B.G. and FREDRIKSSON, A. (1975), "Measurement of Horizontal In Situ Stresses," Proceedings ASCE Specialty Conference on insitu measurement of soil properties, Raleigh, Vol. 1, pp. 266-286.

MAYNE, P.W. and KULHAWY, F.H. (1982), "K_o-OCR Relationships in Soil," ASCE Journal of the Geotechnical and Engineering Division, June, pp. 851-872.

POULOS, H.G., and DAVIS, E.H. (1972), "Laboratory Determination of In Situ Horizontal Stress in Soil Masses," Geotechnique, Vol. XXII, No. 1, pp. 177-182.

PUFAHL, D.E., FREDLUND, D.G., and RAHARDJO, H. (1983), "Lateral Earth Pressures in Expansive Clay Soils," Canadian Geotechnical Journal, Vol. 20, No. 2, p. 228.

ROBERTSON, P.K. and CAMPANELLA, R.G. (1984), "The Flat Plate Dilatometer Test for Liquefaction Assessment," University of British Columbia, Dept. of Civil Engineering, Soil Mechanics Series No. 79.

SCHMERTMANN, J.H. (1972), "Effects of In Situ Lateral Stress on Friction -- Friction-Cone Penetrometer Data in Sands," Proceedings FUGRO Sondeer Symposium, pp. 37-39.

SCHMERTMANN, J.H. (1979), "Statics of SPT," ASCE Journal of the Geotechnical Engineering Division, Vol. 105, No. GT5, May, pp. 655-670. (see Fig. 7)

SCHMERTMANN, J.H. (1982), "A Method for Determining the Friction Angle in Sands From the Marchetti Dilatometer Test (DMT)," Proceedings of the 2nd European Symposium on Penetration Testing, Amsterdam, Vol. 2, pp. 853-861.

SCHMERTMANN, J.H. and CRAPPS, D.K. (1983), "Use of Insitu Penetration Test to Aid Pile Design and Installation," Proceedings of the GEOPILE '83 Conference, Hollywood, FL, April (published by Associated Pile & Fitting Corp., P.O. Box 1043, Clifton, NJ 07014).

SCHMERTMANN & CRAPPS, INC., Consulting Geotechnical Engineers, Gainesville, FL, various job file Nos.

SCHOFIELD, A. and WROTH, P. (1968), "Critical State Soil Mechanics," McGraw-Hill Publishing Company, Ltd., 310 pp.

SEED, H.B. and PEACOCK, W.H. (1971), "Test Procedures for Measuring Soil Liquefaction Characteristics," ASCE Journal of the Soil Mechanics and Foundation Division, Vol. 97, No. SM8, Aug, pp. 1099-1119, (see Fig. 6)

SHERARD, J.L., DECKER, R.S., and RYKER, N.L. (1972), "Hydraulic Fracturing in Low Dams of Dispersive Clay," ASCE Proceedings of the Specialty Conference on Performance of Earth and Earth-Supported Structures, Purdue University, June, Vol. I, Part 1, pp. 653-689.

SIMONS, N.E., and SOM, N.N. (1969), "The Influence of Lateral Stresses on the Stress Deformation Characteristics of London Clay," Proceedings of the 7th Internation Conference on Soil Mechanics and Foundation Engineering, Mexico, Vol. 1, pp. 369-377.

SKEMPTON, A.W. (1961), "Horizontal Stresses in an Over-consolidated Eocene Clay," 5th International Conference on Soil Mechanics and Foundation Engineering, Paris, Vol. 1, pp. 351-357.

SONNENFELD, S.L., SCHMERTMANN, J.H., and WILLIAMS, R.C. (1984), "A Bridge Site Investigation Using SPTs, MPMTs and DMTs from Barges," ASTM Symposium on Marine Testing, San Diego, January.

TEDD, P. and CHARLES, J.A. (1981), "In Situ Measurement of Horizontal Stress in Overconsolidated Clay Using Push-In Spade-Shaped Pressure Cells," Technical Note, Geotechnique, Vol. XXXI, No. 4, December, pp. 554.

VAN WANBEKA, A. and RENARD, J. (1972), "Experimental Determination of Pressures Inside a Soil Mass and Against a Cast in Place Wall," Proceedings of the 5th European Conference on Soil Mechanics and Foundation Engineering, Madrid, Vol. 1, pp. 585-595. (in French)

VEISMANIS, A. (1974), "Laboratory Investigation of Electrical Friction-Cone Penetrometers in Sands," Proceedings of the European Symposium on Penetration Testing (ESOPT), Vol. 2:2, pp. 407-419.

WROTH, C.P. (1975), "In Situ Measurement of Initial Stresses and Deformation Characteristics," Proceedings of the Conference on In Situ Measurement of Soil Properties, ASCE, Raleigh, N.C., II, pp. 181-230.

TABLE 1 - EXAMPLES OF INSITU K MEASUREMENTS

Item	K	Soil	depth (ft)	type test	Notes	Ref.
1	0.40 - 0.60	8 Norwegian clays	n.a.	lab triax., hydr. fract.	OCR = 1.2-1.5	Bjerrum & Anderson (1972)
2	0.45 - 0.73	3 Australia NC clays	approx. 25	lab p_c	K_o	Poulos & Davis (1972, Table 1)
3	0.60	Italian NC silty clay	10-50	SBP Triax., Oed.	K_σ	Jamiolkowski et.al. (1985, Fig. 47)
4	0.48 - 0.56	Italian clay	40-90	SBP	OCR = 1.1-1.3	Jamiolkowski et. al. (1985)
5	1.6 0.8	S. Africa Lacustrine clays	10 16	Lab capillary	shrinkage cracks closed, cracks open	Blight (1967)
6	3	do.	5-15	Lab capillary	assoc. with slope failure	Blight (1969)
7	0.5 1.2	old rolled clay dam in S. Africa	13 36	Lab capillary	anisotropy in s_u closely matched K	Blight (1970)
8	2.8 1.5 2.0-2.5	London Clay	20 110	Lab capillary, backfigure slide		Skempton (1961)
9	4.4 3.1,2.2 1.9	London Clay	10 30 40	pushed Glotzl spade and SBP	on high side, 2 sites	Tedd & Charles (1981)
10	3.3 2.0	London Clay	10 55	SBP		Clark & Wroth (1984)

TABLE 1 continued - EXAMPLES OF INSITU K MEASUREMENTS

Item	K	Soil	depth (ft)	type test	Notes	Ref.
11	4.7 2.6	Gault clay	10-40	SBP SB load cell	major & minor directions,	Dalton & Hawkins (1982)
12	3.5 5.4 6.4	FL Miocene clay	20-40	DMT MPMT K_o blade	production production research	Sonnenfeld et. al (1985)
13	1.3,0.5 0.2-4.1	till,loess clays	16,32 4,36	K_o blade	Fig. 2	Handy
14	0.25	approx. NC FL cl. sand	60-70	DMT	nearby cavities,	S&C files (1981) #441
15	0.48 0.44	River valley sand $55\%<D_r<65\%$	20-80	DMT SBP	ave. 35 tests ave. 19 tests,	Jamiolkowski et. al. (1985) (Figs. 46,54)
16	0.3-0.7	cse. sand cofferdam fill	10-60	DMT MPMT	Fig. 6, arching possible	S&C files (1982) #463
17	0.33 2.5 0.9	sand at bottom of excavation	50-80 6 25	MPMT	before excavat., after excavation	Dahlberg (1974)
18	0.5-0.6 2.0 0.9	mine tailings sand	5-13 4 8	DMT	very loose, NC, after vibr. rolling	S&C files (1984) #542
19	2.7 1.1	compacted sand	2 2	buried load cells	perpendic. to rolling directn., parallel	D'Appolonia et. al. (1969)
20	0.66-1.34	estuarine sand	3-27	DMT	Table 2, bef-aft dyn.-compaction	DMT DIGEST 5 (1985)

TABLE 3 - LAB METHODS FOR K
(apply to cohesive soils)

No.	Method	Ref.
1	Triaxial, no lateral strain	Bishop & Henkel (1957) Bishop (1958)
2	Capillary stress, A parameter	Skempton (1961)
3	Oedometer OCR/K correlations	Mayne & Kulhawy (1982)
4	K_o cell	Campanella & Vaid (1972)
5	Horiz/Vert oedometer	Poulos & Davis (1972)
6	Hydraulic fracturing	Al-Shaikh-Ali (1977)
7	Triax. Deviator Stress	Chang et. al. (1977)

TABLE 4 - FIELD METHODS FOR K

No.	Method	Ref.
1	Menard PMT (MPMT)	Baguelin, et.al. (1978, pp. 152,573)
2	Menard Geocell	Van Wanbeka & Renard (1972)
3	Instrumented vertical pipe	Kenny (1967)
4	Buried Load Cells	D'Appolonia et.al. (1969)
5	Hydraulic Fracturing	Bjerrum & Andersen (1972) Bjerrum (1972) Al-Shaikh-Ali et.al. (1981)
6	Self boring Pressuremeter (SBP)	Wroth (1975) Clark & Wroth (1984)
7	Thin pressure plate penetrometer (Glotzl spade cell)	Massarsach (1975) Tedd & Charles (1981)
8	Flat plate dilatometer (DMT)	Marchetti (1980) Schmertmann (1981) Jamiolkowski, et.al. (1985, p. 51)
9	Self boring (SB) Load Cell	Dalton & Hawkins (1982)
10	K_o stepped blade	Handy et.al. (1982)

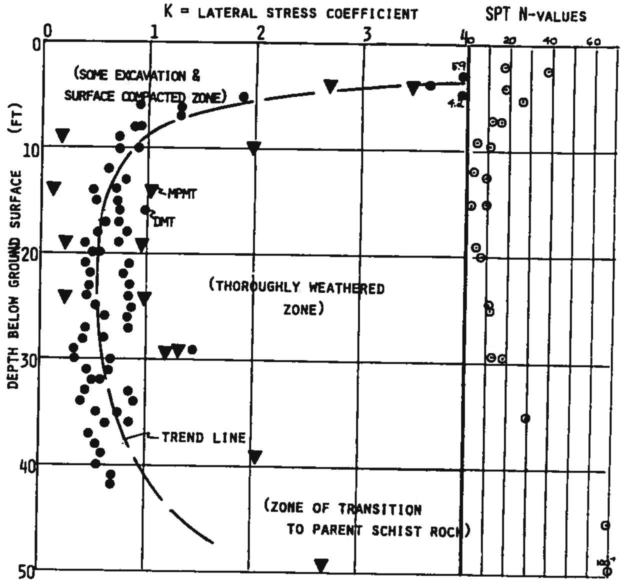


FIGURE 1 - FIELD K-PROFILE IN A MICACEOUS, SANDY SILT, RESIDUAL SOIL IN THE N.C. PIEDMONT (from 2 sets of parallel DMT, MPMT, SPT data)

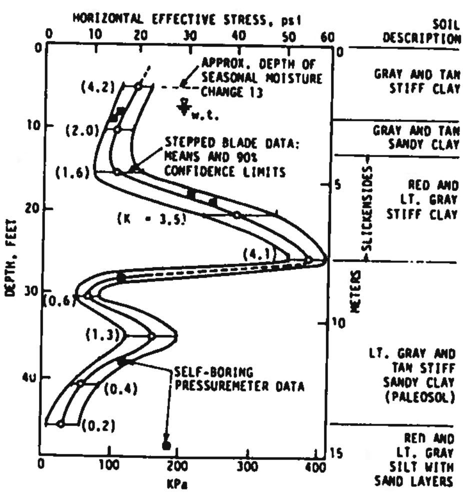
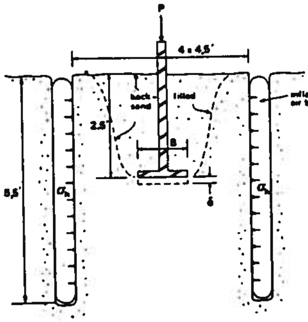


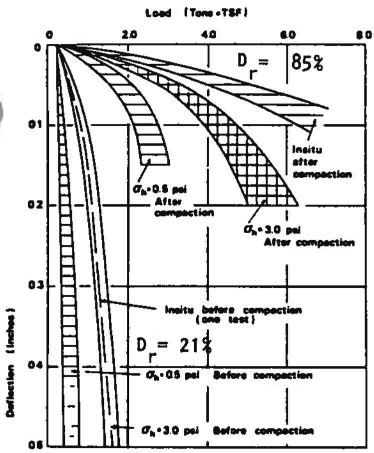
FIGURE 2

FIELD PROFILE OF HORIZONTAL STRESS IN BEAUMONT CLAY, HOUSTON, TX

Nos. in ( ) denote K values
From: Handy et. al. (1982, Fig. 8)



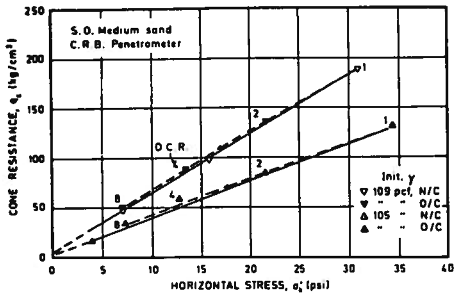
(a) CROSS SECTION THROUGH A PLATE TEST IN AIR-BAG-SURROUNDED 'CUBE' OF SAND



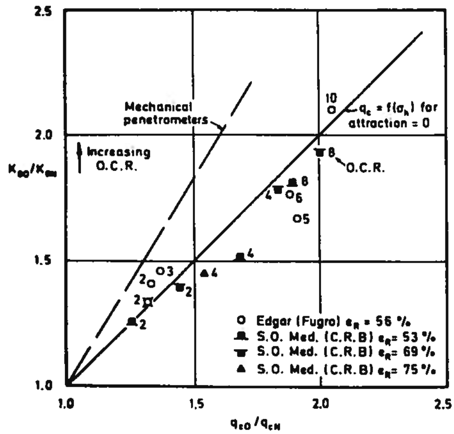
(b) RANGE OF LOAD TEST RESULTS, 4-5 TESTS WITHIN EACH RANGE

FIGURE 3 - RESULTS FROM COMPARATIVE 1.0 FT^2 PLATE BEARING TESTS, PERFORMED IN THE FIELD WITH & WITHOUT USING AIR BAGS TO CONTROL LATERAL STRESS, BEFORE & AFTER VIBRATORY ROLLER

from: COOK (1971), SCHMERTMANN (1972)



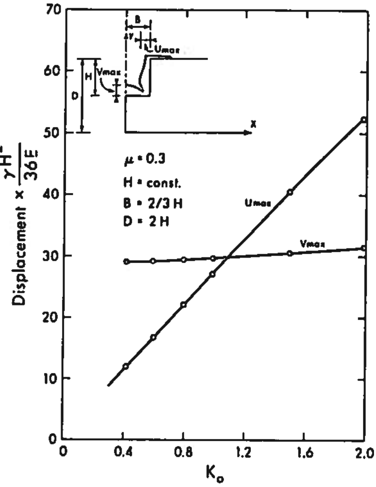
Effect of Overconsolidation: q_c vs $\sigma_h{}'$



Cone Resistance vs Lateral Stress Ratio: Normalised Plot

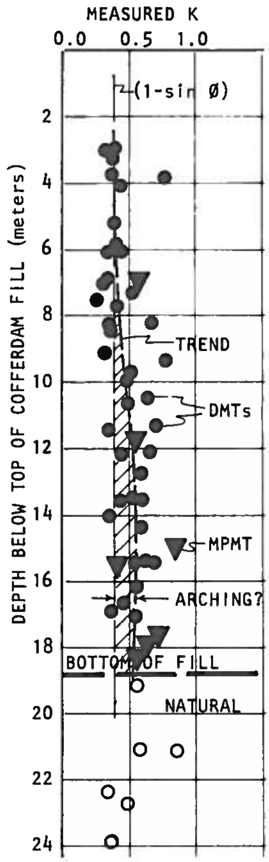
FIGURE 4 - EXAMPLE DATA FROM LARGE SAND CALIBRATION CHAMBERS SHOWING THAT LATERAL STRESS DETERMINES STATIC CONE BEARING CAPACITY

from: VEISMANIS (1974, Fig. 15)



<u>FIGURE 5</u> - MAXIMUM VALUES OF HEAVE & LATERAL DISPLACEMENTS FOR DIFFERENT K_O VALUES

from: DIBIAGIO (1966)



<u>FIGURE 6</u> - K DATA AND PROFILE FROM INSIDE A CIRCULAR COFFERDAM FILLED WITH RIVER COARSE SAND AND GRAVEL

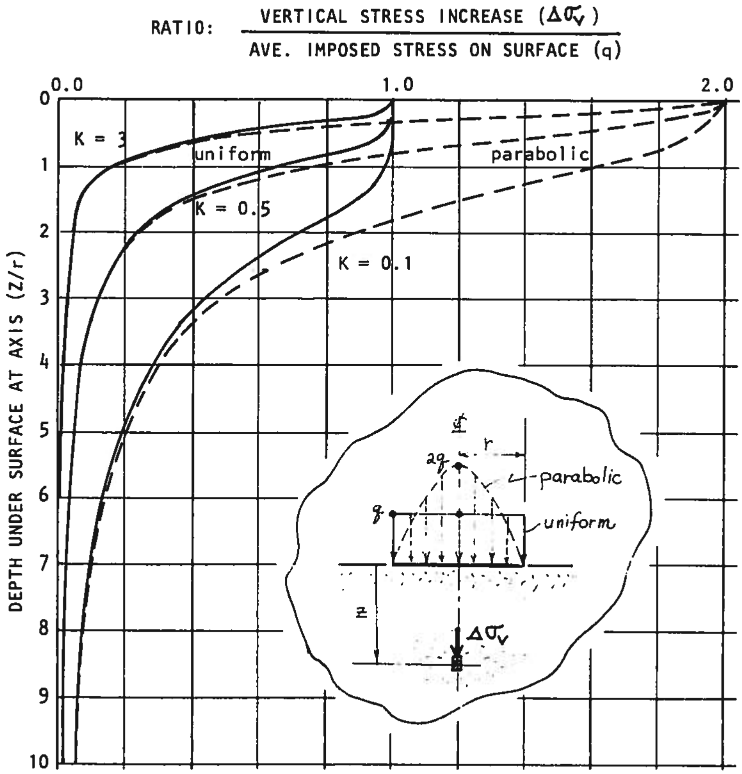


FIGURE 7 - VERTICAL STRESS INCREASE BELOW AXIS OF A CIRCULAR AREA LOAD ON SURFACE, USING PROBABILISTIC PARTICULATE MECHANICS, AND SHOWING IMPORTANCE OF INITIAL K CONDITION

Computed from HARR (1977, eqns. 7-57, -63)