Figure 2. Relationship between coefficient of consolidation field data and sand drain spacing.

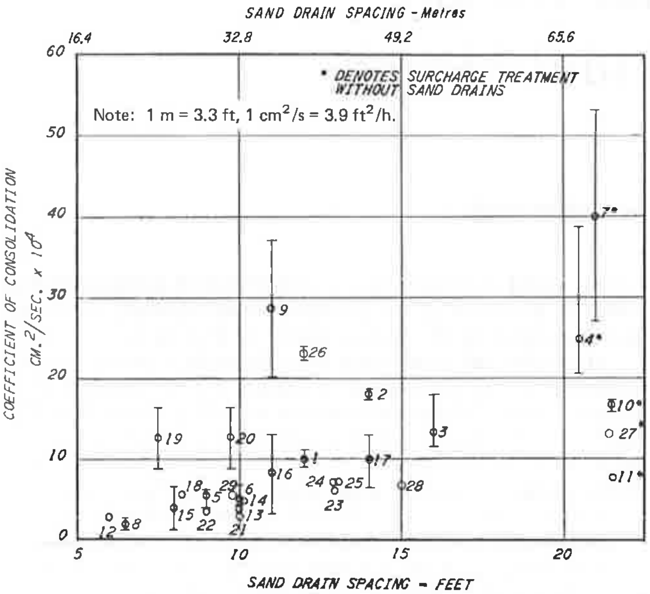
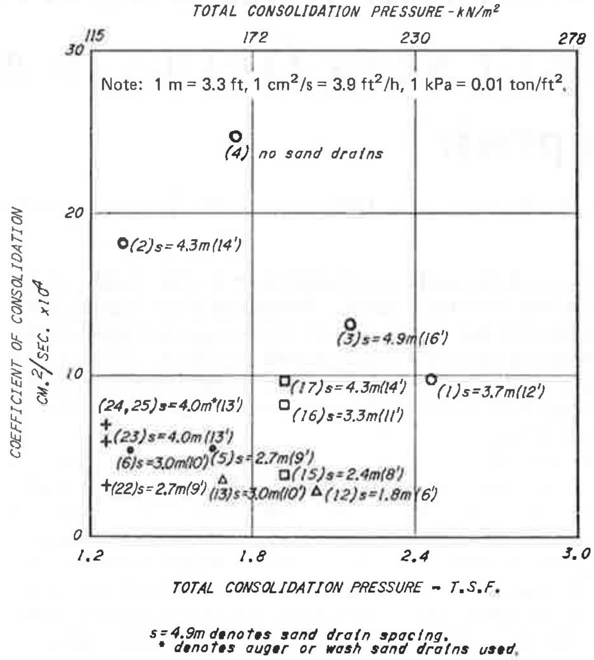


Figure 3. Relationship between coefficient of consolidation field data and total consolidation pressure.



drain spacing and nondisplacement sand drains produce a lesser magnitude of disturbance than displacement sand drains.

The relatively high coefficient of consolidation for point 4 with respect to points 1, 2, and 3 is probably due to the omission of sand drains for point 4 and the resulting absence of the disturbance effect due to sand drain installation.

Figures 2 and 3 show that in at least some cases the spacing used with conventional displacement sand drains does materially affect the field coefficient of consolidation and thus the field settlement rate. It is possible that for some of the other data the use of closer sand drain spacings resulted from design considerations such as lower laboratory coefficients of consolidation.

This study led to the following conclusions:

1. A review of field settlement platform data showed that the range in the settlement rate was much narrower than that indicated by the laboratory test data.
2. The field settlement data corroborated that, for the design of displacement sand drains in tidal marsh deposits, the average coefficient of consolidation from conventional laboratory consolidation samples should be used and that any increase in horizontal over vertical permeability should be neglected.
3. The plot of coefficient of consolidation versus sand drain spacing shows a significant trend for a large number of different tidal marsh deposits. For some of these data a closer spacing of conventional displacement sand drains showed a resulting reduction in the coefficient of consolidation as measured from field data and appears to be due to disturbance effects. These data also show that the total consolidation pressure has a marked effect on the field-measured coefficient of consolidation, as would be expected from the laboratory consolidation test data.

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The Iowa K-Test

R. L. Handy, A. J. Lutenegger, and J. M. Hoover, Department of Civil Engineering, Iowa State University, Ames

A simple and rapid laboratory test that uses standard 9.44 cm³ (0.03 ft³) compacted soil specimens for strength comparisons is presented and discussed. The test gives discrete evaluations of undrained c, ϕ, and other strength parameters from single soil specimens. The specimens are subjected to vertical compression while confined in a split steel mold, which acts as a spring, so that spreading of the mold provides a measure of lateral stress. Thus, K, or the ratio of soil horizontal to vertical stress, may be continuously monitored and used to obtain strength parameters and moduli as the test progresses. In addition, a direct measure of soil-to-steel friction as a function of normal stress is obtained. The K-test simulates an undrained, rapid field-loading situation and appears particularly applicable for transportation facilities. This paper presents representative results on several soils, discusses errors in the assumptions, and describes some potential uses of the test for design and control purposes.

Lateral stress induced from an applied vertical pressure on soil has been of fundamental concern since the classical work of Coulomb on retaining structures (1). Rankine (2) suggested that a ratio of lateral to vertical stress is a discrete property of granular soils, the lateral stress being reduced by the soil's internal resistance to sliding—termed the active state. Rankine (3) later suggested that the value of the ratio (K) is not only important for design of retaining structures but also for simple, direct analyses of bearing capacity.

HISTORY OF SELECTED K-TESTS

One of the earliest attempts at direct measurement of the active state ratio was made by Goodrich (4). His apparatus consisted of a cast-iron cylinder that had a circular hole near the bottom, into which a plug was carefully fitted. Soil pressure on the plug activated a buzzer, and the force required to push the plug back, breaking the electrical circuit, was measured.

Tschebotarioff (5) described a lateral earth pressure meter consisting of a thick-walled cylinder, one-half of which consists of 12 horizontal half-circular ball-bearing mounted rings, each ring connected to a dynamometer with SR-4 strain gauges. Values of $K_0 = \sigma_h/\sigma_v$ are computed for each ring, assuming constant σ_v throughout depth of specimen. Since some lateral movement occurs, the measured value of K_0 may not be a correct representation.

A simpler apparatus developed by Sowers (6) used SR-4 gauges applied directly to the walls of a horizontally slotted thin-walled mold. Since little specimen movement was allowed, the test was considered advantageous for K_0 estimation. Other means for measurement of K_0 were devised by Terzaghi (7) and Obercian (8).

A widely used method to obtain a measure of K is the Hveem stabilometer (ASTM D2844-69) wherein lateral stress in a cylindrical specimen is measured as fluid pressure in a liquid-filled cell (9). Stabilometer R values may be modified to account for variable resistance of a soil to lateral deformation (10).

Housel (11) proposed a more conventional triaxial stabilometer test and pressure transmission factor, the latter being a function of changes in lateral and vertical stresses. In this test, lateral deformation is considerable. Recognizing this fault, Wedzinski (12) measured induced lateral pressures in silts in a closed-system water-filled triaxial cell where magnitude of pressure was observed by means of movement of a closed air bubble in a side-mounted capillary tube.

INDIRECT EVALUATIONS OF K FROM TRIAXIAL DATA

For design purposes, K is commonly evaluated by triaxial tests at several preset values of lateral stress (σ_3) or by direct shear tests at several preset values of normal stress (σ_n); derived values are internal friction angle (ϕ) and cohesion (c). For cohesionless soils, K is found from the Rankine formula,

$$K_{(c=0)} = (1 - \sin\phi)/(1 + \sin\phi) \tag{1}$$

Conventional shear tests are not realistic for several reasons, one being that the lateral stress σ_h (or the normal stress σ_n in direct shear) is held constant, whereas in field loading these stresses gradually increase as a consequence of increasing vertical load. Nevertheless, stress-strain relations evolved from triaxial tests are widely interpreted as accurately representing field behavior and are used in sophisticated computer-based techniques. Yet in tests conducted with the lateral stress held at a single value, vertical stress climbs to a peak and then declines, whereas in tests where σ_h floats upward, vertical stress likewise climbs and there may be no peak strength but simply a continuing dependence of vertical stress on lateral restraint. Field failure occurs when sufficient lateral restraint cannot be mobilized or when vertical deformation is excessive for a given load.

IOWA CONTINUOUS K-TEST

The present K-testing concept began when a commercial split Proctor mold was left unlocked, a compacted soil specimen was placed inside, and it was squeezed between steel end plates while mold expansion was monitored with a dial gauge. The mold was calibrated using a tangential force, and later air pressure in a rubber membrane. Figure 1 shows the results from testing three different soils with this primitive apparatus. The higher the lateral stress ratio, the lower the stability. The silt test sprang the mold.

c and ϕ

The data of Figure 1 show that K was not constant but varied as a function of σ_1. This may be accounted for by converting K values to equivalent c and ϕ, more commonly used in design. From a Mohr circle, a c-ϕ soil at failure produces (5)

$$K = \sigma_3/\sigma_1 = \tan^2(45^\circ - \phi/2) - (2c/\sigma_1)\tan(45^\circ - \phi/2) \tag{2}$$

If c = 0, the second term is 0, and Equation 2 is identical to Equation 1. If $\phi = 0$, Equation 2 becomes

$$K_{(\phi=0)} = 1 - 2c/\sigma_1 \tag{3}$$

Thus, for a c-soil, K increases as σ_1 increases. This behavior is shown by the clay and early stages of the silt in Figure 1. Thus, as a generalization, a constant K with respect to σ_1 should signify a granular or ϕ-soil, and a K that increases with σ_1 should signify a c-soil.

Most soils have both internal friction and cohesion. Since any two points in a K-test represent two Mohr circles, if failure is in progress these will define a tangential failure envelope such as AA′ (Figure 2). Linear extrapolation to the abscissa gives

$$\phi = \sin^{-1}[(\sigma_1 - \sigma_3)/(\sigma_1 + \sigma_3 - 2\sigma_i)] \tag{4}$$

For any two Mohr circles that describe a failure condition, Equation 4 may be written for each circle. Solving for σ_i gives

$$\sigma_i = (\sigma_{1_1}\sigma_{3_2} - \sigma_{1_2}\sigma_{3_1})/(\sigma_{1_1} - \sigma_{1_2} - \sigma_{3_1} + \sigma_{3_2}) \tag{5}$$

Thus, the numerical procedure to reduce any pair of K-test data points is as follows: σ_i is obtained by Equation 5, ϕ by Equation 4, and c from

$$c = -\sigma_i \tan\phi \tag{6}$$

The above procedure applied to the data of Figure 1 gives relationships such as shown in Figure 3 for the clay. In this case, ϕ starts high and decreases to 0 as the load is applied, the soil compresses, and pore pressures develop. Simultaneously, c increases. Error in c also increases to the right due to the longer lever arm (AA′ in Figure 2) as Mohr circles shift to the right.

The erratic, zig-zag nature of the plot suggests experimental error, but it also may reflect a test periodicity, such as stick-slip. That is, gradual develop-

Figure 1. Lateral stress ratios from several soils compacted with standard effort and optimum moisture content, tested in a split Proctor mold.

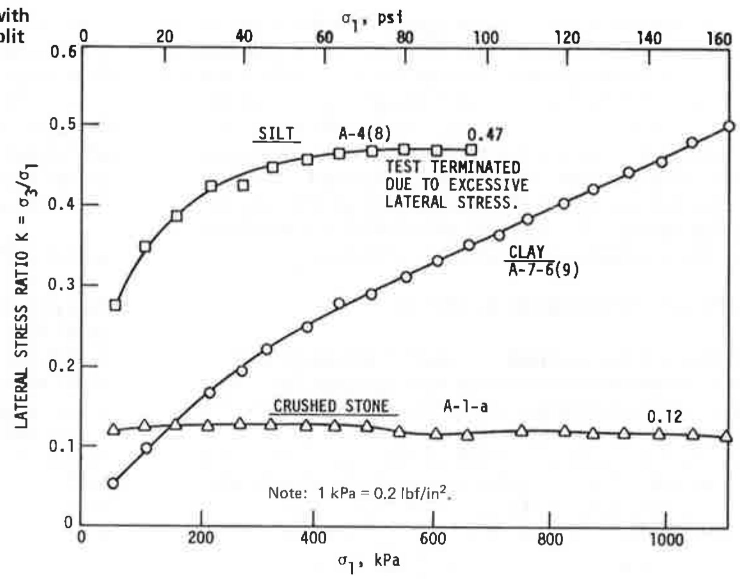


Figure 2. Mohr envelope from two K-test points.

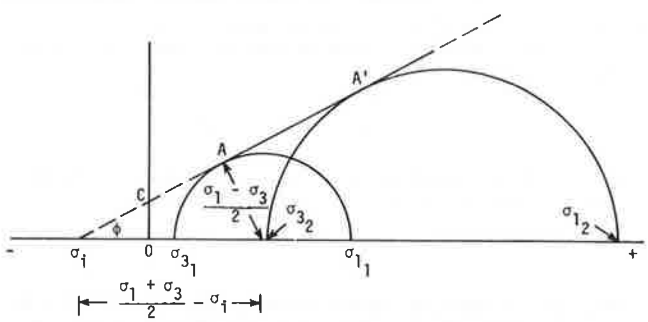
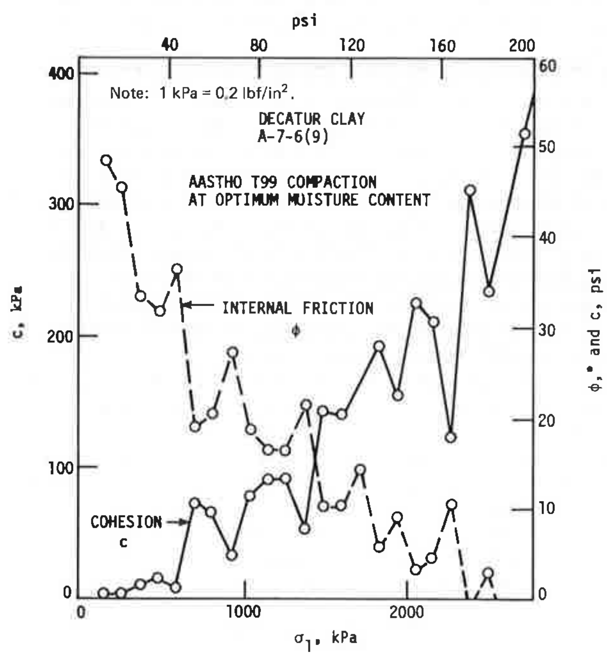


Figure 3. Increase in c and decrease in ϕ for a clay under increasing load in the K-test.



ment of cohesion may be periodically interrupted by a sudden, temporarily disruptive slip. Evidence for this is that the curves are not random ups and downs, since the highs of c (and lows of ϕ) usually last for two or three points, whereas the lows of c (and highs of ϕ) are single points.

Similar treatment of the data for crushed stone gave a consistent ϕ around 51°, with c hovering about 0. The loess behaved first as a weak granular material, with ϕ about 20° and c around 14 kPa (2 lb/in^2), then showed effects from pore pressure: ϕ dropped and c increased with increasing σ_1.

In summary, the trends of c and ϕ are about as expected, although actual indicated values may be incorrect.

Mold Redesign

A special K-test mold was made with the wall thicker at the back (Figure 4) in the zone of maximum bending moment. The opposite side is slotted, with an internal Teflon strip acting as a seal. The interior of the mold is polished and chromium plated to resist abrasion. A 0.0025-mm (0.0001-in) dial gauge is mounted externally to monitor expansion. With uniform internal pressure the change in mold radius is not uniform but gives two radial bulges about 90° to one another and 45° from the mold slot. The mold is slightly larger than the specimen diameter, thus avoiding an initial passive condition from soil being forced into the mold (i.e., the horizontal confining stress initially must not exceed the vertical stress, or an initial passive stress state must be overcome before obtaining active state K data).

Several procedures that involve air pressure were used to calibrate the K-test molds. More successful, however, has been an artificial soft thermoplastic soil specimen with K nominally equal to 1.0. In this way, horizontal stress is assumed to equal the applied vertical stress, and a horizontal stress versus mold opening calibration is prepared. Calibrations are linear, enabling data reduction by linear equations.

Stress Path Interpretation of K-Test Data

Graphs such as Figure 3, while of interest for showing how strength may develop in a soil under increasing load, are of little direct use in design where mean values of c and ϕ are needed across the anticipated range of field loading. It may be more convenient to plot a p-q diagram, as in Figure 5, and fit a line by least squares regression, converting slope and intercept to ϕ and c (13). This also has the advantage of plotting several tests on a single graph.

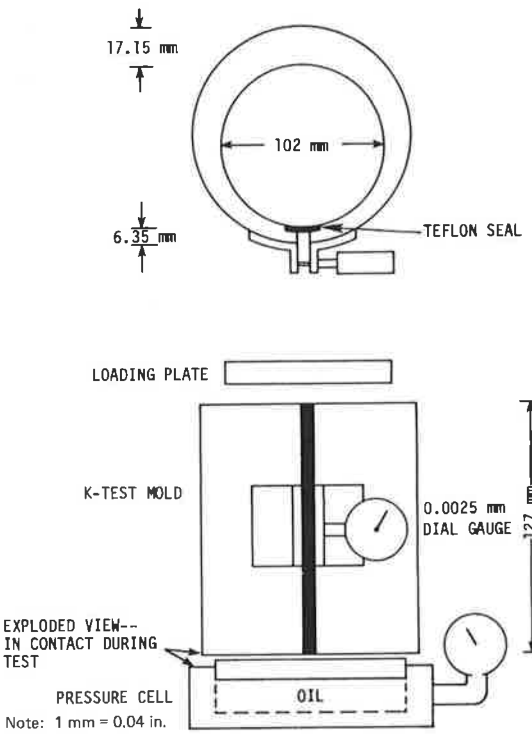
Tests of Dry Sand

Results from 20 K-tests on dry standard Ottawa sand with initial void ratios varying from 0.55 to 0.74 were analyzed by the stress path method (14). Stress path plots were exceptionally linear, and all regression coefficients exceeded 0.999. The friction angle ϕ varied from 30° to 41° and was inversely related to the initial void ratio, $\phi = 64.5° - 44.2\ (e_0)$; $r = 0.68$.

Taylor and Leps (15) report essentially the same trend for Ottawa sand but with ϕ lower by 5° to 10°, depending on the normal stress. Though cohesion values should be 0, c from the K-tests averaged 17.6 ± 15.4 kPa (2.6 ± 2.2 lb/in²), the ± value indicating standard deviation, and showed no relation to e_0. An important source of this error is boundary stress between soil and mold and between soil and end-loading platens. Such friction must be quite high in the case of the sand, which tended to score the mold, and is discussed later.

In summary, K-tests of Ottawa sand showed excellent linearity and precision in the determination of c and ϕ by the stress path method. Furthermore, the trend of ϕ in relation to void ratio is as it should be; however, on the average, ϕ appeared to be overestimated by about 5°, and c by about 20 kPa (3 lb/in²).

Figure 4. Constant-E model Iowa K-test mold for Proctor or equivalent-size specimens.

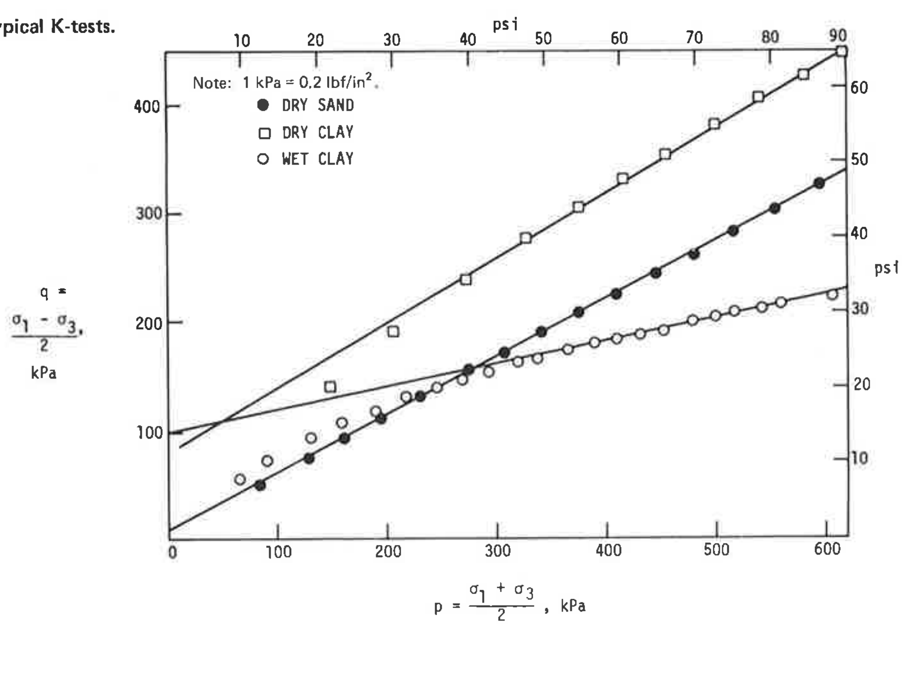


Glacial Till

Figure 6 shows the results of 25 K-tests on a glacial till-derived Shelby series soil molded by AASHTO T-99 compaction. Since c and ϕ of cohesive soils vary during the test, the use of a linear stress path is an averaging approximation, the degree of departure from a line causing a decrease in correlation coefficient (r), which varied from 0.86 to 0.999. The ϕ and r values are lower for wetter samples, indicating development of positive pore pressures.

Figure 6 shows the expected trends—the higher the moisture content, the lower are ϕ and c. Extrapolation to predict the moisture content at which ϕ and c are 0 gives 29 and 32 percent, respectively. The plastic limit of this soil is 32 percent. From these data, it appears that pore pressure is a most important variable, particularly when comparing results of K-tests with more conventional laboratory tests. This does not mean that the uncorrected total stress parameters ϕ and c cannot be useful for design purposes. On the contrary, where anticipated loading rate and conditions inhibit or prohibit drainage, ϕ and c may be more useful than effective stress parameters ϕ' and c' because field pore pressures are not very predictable. As will be shown, the influence of pore-water pressure in the K-test may be di-

Figure 5. Stress paths from typical K-tests.



rectly monitored through a measured reduction in side friction.

E and ν

An estimate of the vertical deformation modulus E_z, useful in pavement design and required for finite element modeling, may be directly obtained from K-test stress-strain plots (Figure 7). E_z is not a true elastic modulus since the soil is in failure but should be a fairly accurate reflection of field behavior. The stress-strain relation is much more linear than in triaxial tests where lateral stress is constant and may explain the success of elastic theory for predicting soil stresses in the field under load. The E_z for Ottawa sand increased during the test, which is indicative of compaction.

Another measure required for characterization of stress under load for application of elastic theory or the finite element method is Poisson's ratio (ν). For a homogeneous, isotropic, elastic material it can be shown that

$$\nu = (\epsilon_r \sigma_z - \epsilon_z \sigma_r)/[2\epsilon_r \sigma_r - \epsilon_z(\sigma_r + \sigma_z)] \quad (7)$$

where ϵ_r and ϵ_z are radial and vertical unit strains, and σ_r and σ_z are corresponding stresses. Since these values of stress and strain are measured in the K-test, a direct evaluation of Poisson's ratio may be obtained.

Figure 6. K-test ϕ (top) and c (middle), and γ_d (bottom) versus compaction moisture content for a glacial till soil.

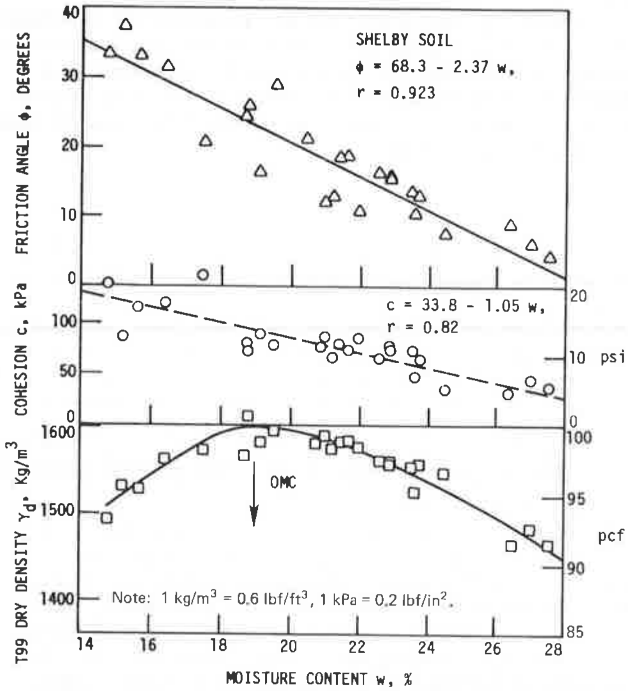
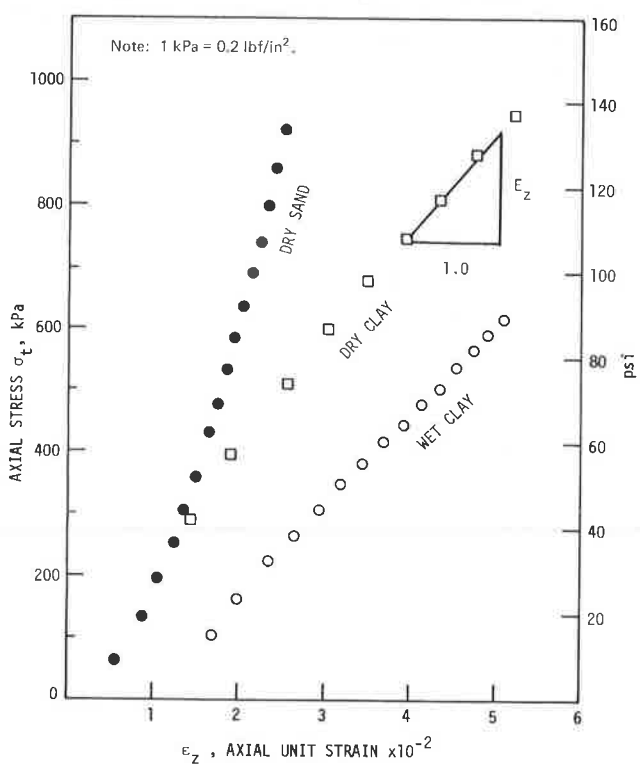


Figure 7. Typical stress versus strain diagram from K-test.



Side Friction Experiments

A major criticism of the K-test is the influence of friction between the soil and its confining steel mold and end platens. Platen friction is also a problem in triaxial tests where its effects are standardized by adapting a standard height-diameter ratio, normally 2.0. In the K-test, this ratio is closer to 1:1, and there is the additional friction from the mold.

Two approaches to the side friction problem are to minimize it and to measure it. Measurement offers some unique advantages:

1. Soil-to-steel friction is directly obtained as a function of normal stress, giving soil-to-steel sliding friction parameters (c_s and ϕ_s) that are potentially useful in design of pile, earth movers, and farm implements.
2. If the measured soil-to-steel friction parameters are assumed to apply to end platens of similar material and finish, the average boundary stresses will be known, and there exists a potential for a complete solution of stresses within the specimen. This would appear to offer a substantial advance over present triaxial testing.
3. Pore pressure effects may be evaluated directly from the influence on side friction, rather than through pressure transducers.

Side friction was measured by supporting the K-test mold on a pressure cell, such that the soil specimen would move downward and push on the base cell piston (14). In this way, all side friction was mobilized upward. Summing vertical forces on the soil gives

$$F_t = F_b + F_s \quad (8)$$

where

F_t = the top load on the soil,
F_b = the bottom load, and
F_s = the side friction.

Substituting stresses times respective end and side areas and solving,

$$\tau_s = (A_e/A_s)(\sigma_t - \sigma_b) \quad (9)$$

$$\tau_s = (R/2H)(\sigma_t - \sigma_b) \quad (10)$$

Table 1. K-test soil and soil-to-steel friction data for Shelby soil.

Moisture Content, W (%)	Soil Parameters ϕ (degrees)	Soil Parameters c (kPa)	Soil-to-Steel Parameters ϕ_s (degrees)	Soil-to-Steel Parameters c_s (kPa)	Effective Stress Soil Parameters ϕ' (degrees)	Effective Stress Soil Parameters c' (kPa)
12.4	39.1	132	28.0	18	39.1	132
15.2	37.4	87	27.0	5	38.0	85
15.3	34.8	119	25.5	18	35.4	128
16.5	31.6	121	23.2	10	33.3	130
17.3	30.5	119	18.2	21	36.6	118
18.8	25.9	71	15.4	3	35.6	66
19.5	23.8	43	13.1	17	39.8	80
20.1	22.6	17	11.5	23	36.2	74
21.7	18.6	74	7.7	12	38.5	57
22.6	16.1	66	6.1	10	38.2	50
22.9	17.3	76	5.7	26	44.2	43

Notes: 1 kPa = 0.145 lbf/in^2.
Parameters are uncorrected for side friction.
Regressions for total stress are ϕ = 70.9 - 2.39w, r = 0.988; ϕ_s = 61.6 - 2.45w, r = 0.981; c = 199 - 5.5w, r = 0.775; c_s = 4.9 + 0.55w, r = 0.269; and ϕ_s = 1.03 - 11.3, r = 0.992.
Regressions for effective stress are ϕ' = 31.2 + 0.35w, r = 0.417; and c' = 250 - 9.0w, r = 0.877.

where

τ_s = the average side frictional stress,
A_e and A_s = end and side areas of the cylindrical soil specimen, respectively,
R and H = its radius and height, and
σ_t and σ_b = the nominal vertical stresses on its top and bottom.

Typical plots of K-test side friction versus normal stress obtained from the mold expansion give linear relationships. Slope of the plot is the soil-to-steel friction angle (ϕ_s) and the intercept is soil-to-steel adhesion (c_s). In every case, it was found that the slope $\phi_s < \phi$, and the intercept $c_s < c$. However, neither ϕ_s/ϕ nor c_s/c describes any specified ratio, as often assumed for pile design.

A summary of soil and side friction parameters is given in Table 1. Regressions of total stress data in Table 1 indicate that for the Shelby soil $\phi_s = \phi - 11.3°$. A series of tests on compacted friable loess gave a similar result: $\phi_s = \phi - 12.8°$. Part of the approximate 12° difference between ϕ_s and ϕ may be an overestimation of ϕ by the K-test, whereas the vertical and horizontal forces to calculate ϕ_s are directly measured, and there should be no analogous error in its determination.

The adhesion c_s (Table 1) is low and erratic but tends to increase with increasing moisture content. Since the tests were performed immediately after compaction, there was little time for soil-steel adhesion to develop.

An unusual feature of the K-test is the evaluation of pore pressure from the effect on side friction rather than measurement under dynamic loading. The equation for side friction on an effective stress basis (indicated by ') is

$$\tau_s' = c_s' = (\sigma_h - u)\tan\phi_s' \qquad (11)$$

where

τ_s' = shearing stress,
c_s' = adhesion,
ϕ_s' = soil-to-steel friction angle,
σ_h = total horizontal stress obtained from mold expansion, and
u = pore water pressure.

Solving,

$$u = \sigma_h \left\{1 - [\tan\phi_s/\tan\phi_s' + (c_s' - c_s)/\tan\phi_s']\right\} \qquad (12)$$

Since c_s is small and probably not affected by pore pressure (i.e., $c_s' = c_s$) the equation may be simplified to

$$u = \sigma_h\ (1 - \tan\phi_s/\tan\phi_s') \qquad (13)$$

As previously discussed, ϕ and ϕ_s for a wet soil change as the test progresses due to development of pore water pressure. Side friction should be particularly sensitive to pore water pressure because there is no dilatancy. The soil is pressed against an impermeable steel face; therefore, pore pressure developed at the face should be essentially the same as that within the specimen.

In the initial stages of a side friction test, the slope is ϕ_s' if u = 0. This is shown by the first six points of the wet clay sample (Figure 5). Pore pressure then determined from Equation 13 gives effective stress data (Table 1).

Regressions of the effective stress parameters (Table 1) show a higher dependence of c' on moisture content than the total stress parameter c, and a higher correlation coefficient. As should be expected, ϕ' changes relatively little with moisture content, and appears to increase with increasing w. However, the standard error indicates about a 70 percent possibility that the relation is due to chance. Wetting has been observed to substantially increase ϕ' of nonclay minerals while reducing that of clays (13).

Actual Stress Distribution

A previously cited advantage of the K-test is the direct determination of side friction as a function of measured normal stress. If the end platens are of the same material and finish as the mold, radial end friction should also be characterized. If all soil-to-steel friction is fully mobilized, vertical stresses at the top and bottom (σ_t and σ_b) should plot on the soil-to-steel side-friction envelope rather than on the abscissa of $\tau = 0$. The indicated horizontal shearing stresses at the platens (τ_t and τ_b) must decrease to 0 and reverse directions near the platen centers, so τ_t and τ_b are positive extremes. Downward within the soil specimen, τ also must decrease to 0 and then reverse. The mean stress on horizontal planes within the specimen, therefore, must be within the shaded trapezoid of Figure 8.

Approximations to a Mean Failure Envelope

The precise topography of stress distribution within the K-test specimen is not known but should be within the capability of finite element modeling once boundary conditions and soil behavior are fully defined. However, the test purpose is to define soil behavior, so a stress distribution must be assumed. If desired, the approximation may be tested on the basis of derived behavior.

The first approximation used in this paper assumes $\sigma_t = \sigma_1$ and $\sigma_h = \sigma_3$, a very simple procedure adapted for routine tests and not requiring bottom pressure readings. This is shown by the solid-line uncorrected failure circles (Figure 8).

A second approximation that should be closer to reality can be based on assumed validity of the soil-to-steel failure envelope, which is directly measured (Figure 8). The top and bottom normal stresses (σ_t and σ_b) also directly measured, must plot on or slightly below this envelope. The horizontal shearing stress τ is a maximum at the upper platen but must decrease to 0 and reverse both across and downward within the soil specimens; hence the upper σ_1 limb of the Mohr circle must pass somewhere through the shaded area of Figure 8,

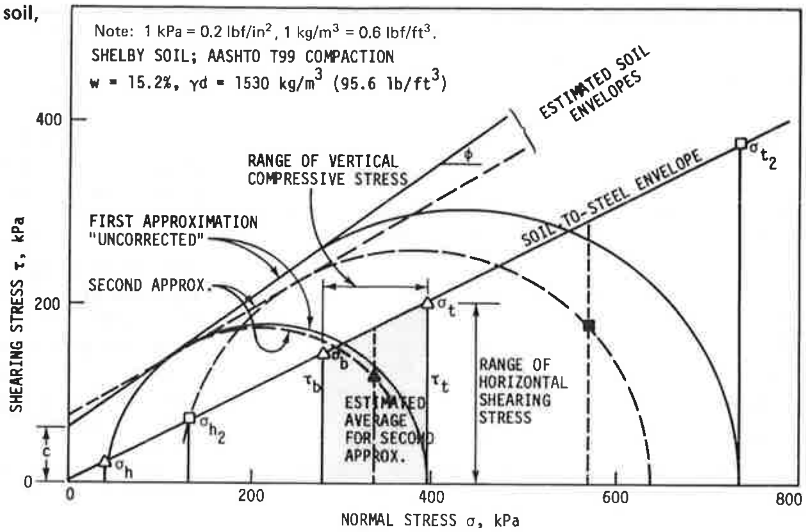


Figure 8. Two data points from a test on Shelby soil, showing two methods to obtain ϕ and c.

rather than simply through its lower right corner. The lower σ_3 limb likewise should plot through σ_h plotted on the soil-to-steel envelope, and since the measured σ_h is an average, its indicated τ also represents an average.

The problem is to ascertain a realistic average stress condition for the upper limb. On the basis of the assumption that σ_b and σ_t may be averaged, and the mean τ is 60 percent of that at the specimen boundaries, the solid points and corresponding dashed second approximation circles were drawn. The resulting failure envelopes give

Approximation	ϕ	c
First	35.6°	103 kPa
Second	30.5°	131 kPa

That the second approximation is better than the first is shown by the lower ϕ and slightly higher c, consistent with observations that uncorrected K-test values are too high, and, with exception of the Ottawa sand, c values are too low compared to other data. The analysis also shows why the initial assumption that $\sigma_t = \sigma_1$ gives fairly reasonable results since the Mohr circle passing through (σ_t, 0) is a good preliminary estimate.

Height-to-Diameter Ratio

Stress distribution within a K-test specimen should depend on the height-to-diameter ratio (H/D) and on whether the mold is supported, since support causes all side friction to act one way. Support and a large H/D will increase the roll of side friction relative to end friction. Preliminary investigations with a 25.4-mm (1-in) diameter K-test mold indicate that an H/D ratio close to 1.0, with the mold supported, minimized ϕ and c for the sand and appeared reasonable.

Circumferential Friction

Circumferential friction of soil on steel acts on the inner mold surface, opposing opening of the mold and reducing the measured σ_h. A calculation of this effect shows that it is minor and may be ignored. Secondly, since only the vertical component of side friction is measured in the test, total side friction is not axisymmetric and is underestimated. Extent of this error depends on the direction of movement: The maximum amount of circumferential slip occurs adjacent to the mold split, about 1.3 mm (0.050 in), whereas the maximum amount of vertical deflection typically is about 6.4 mm (0.25 in). As an extreme, the correct $\tau = \tau_s$ sec/tan (1.3/6.4) = 1.02 τ_s, a 2 percent error, which is minor and may be ignored.

SOME POTENTIAL USES OF THE K-TEST

The K-test modulus of deformation (E_z) would appear to have an excellent potential for rigid pavement design or for comparison with, and fill-in between, relatively expensive plate bearing tests. It should be emphasized that the K-test requires about 15 min and utilizes specimens already available from Proctor compaction tests.

The speed and ease with which K-tests provide c and ϕ values mean extremely rapid calculation of bearing capacity pertinent to flexible pavement design, at a cost well below that of comparable California bearing ratio (CBR) tests. It will be noted that a laboratory CBR uses a rigid steel mold, whereas structural deterioration of flexible pavements often begins at the edges where lateral confining stress is low and is not correctly modeled in CBR.

K-test specimens may be vacuum or capillary saturated prior to test; however, molding at slightly above optimum moisture content may achieve the same result in less time.

Earthwork Control

Present methods for compaction control are of necessity indirect; although strength is the desired criterion, strength tests have been too inaccurate (as with the Proctor penetrometer) or too time consuming and expensive. We therefore rely on density, moisture content, and soil identification, all three needing arduous attention if failures are to be avoided. K-tests of cores could provide a much more rapid accept-reject criterion—if the soil is too clayey, this will adversely affect K-test results. Since only a simple unconfined compression tester is needed, Shelby tube specimens may be extruded, immediately tested in the field, and results compared with laboratory controls. In addition, calculation of bearing capacity may give a rapid determination of weight requirements for field compactions.

Earthwork Specifications

The choice between 90 and 95 percent T-99 or T-180 density is usually a matter of engineering skill and judgment based on anticipated loads and performance records of particular soils. By means of K-tests on specimens molded at variable compactive efforts, the advantage and amount of additional field compactive effort may be quickly and directly ascertained; also the K-test is sensitive to overcompaction, as indicated by a drop in c and increase in ϕ.

Pile

Use of the K-test to evaluate side friction data directly, and thus predict skin friction on pile, needs no further elaboration. Tests conducted show that the method of using fractionally reduced soil ϕ and c values may be in error, normally but not necessarily conservative.

The above suggested uses may seem somewhat overstated or even flamboyant at first encounter. The K-test can be fast, cheap, and readily computerized. Since the known errors are biased rather than random, corrections appear possible. Meanwhile, an arbitrary reduction in ϕ and c may be made or their overestimation covered in a factor of safety. A major advantage of a cheap and rapid test is to evaluate variability of soils, which has been a major problem in soil testing. Similar comparisons seldom are possible by triaxial testing because of the prohibitive cost; an abbreviated version; however, is the stage-triaxial test, which is increasing in use, particularly by consulting firms. The latter is a stepwise analogue of the K-test, requiring judgment of where failure is about to occur in order to increase confining stress; in the K-test, this is automatic.

CONCLUSIONS

1. A soil test to measure the Rankine active stress ratio K has been devised to utilize standard Proctor-size compacted specimens, a load frame, and a split steel K-test mold. The test has been used extensively for comparative evaluations of undrained soil strength parameters.

2. The Iowa K-test requires about 15 min to perform and provides a continuous record of uncorrected soil cohesion (c), internal friction (ϕ), and deformation moduli (E_z and ν), as the test progresses.

3. Supplemental use of a base pressure cell allows simultaneous direct measurement of soil-to-steel friction as a function of normal stress and, hence, of the soil-to-steel friction parameters (c_s and ϕ_s).

4. Errors in the uncorrected soil c and ϕ data are discussed, and a preliminary method for correction is presented. An accurate correction will require a much more extensive analysis of stress distribution; however, preliminary analyses and comparative tests indicate that the uncorrected data are not so seriously in error that they would not be covered by usual factors of safety. Since many tests can be performed in a reasonable time span, a probabilistic approach to design and construction control becomes feasible.

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The first Iowa K-test was conducted October 11, 1974, by R. L. Handy and J. M. Hoover, and was observed by Darwin E. Fox. Since that time, thousands of tests have been conducted in conjunction with Federal Highway Administration project, Chemical Compaction Aids for Fine-Grained Soils, in order to assess effects of chemical additives, moisture content, and compaction energy on strength parameters. Developmental research on the K-test has been sponsored by the Iowa State University Engineering Research Institute and the Iowa State University Research Foundation.

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