The Shear Behavior of Soil with Constant Structure

By

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Synopsis

This paper presents an experimental investigation of the components of mobilized soil shear resistance independent (I) and dependent (D) of effective stress when also considering strain (S) as a variable. To study the important effect of soil structure, the writer developed a test (IDS-test) that separates these components at approximately constant structure. Many such IDS-tests demonstrated the special and surprising behavior of the I-component in a variety of remolded and undisturbed soils. This led to the concept of the Constant Structure Mohr Envelope (CSME), the formulation of its shape in equation (6), and the experimental confirmation of equation (6). All this then led to a new method for measuring a soil’s mobilized shear resistance from only internal bonds, I₀. The new method seems to produce reasonable results, has a low sensitivity to stress-dilatency effects, and permits determining I₀ as a function of strain.

This paper demonstrates that when some remolded and undisturbed cohesive soils carry load under creep strain rates they undergo soil structure modifications that transfer load to those groups of particles oriented to best resist particle dispersion. This transfer can increase dramatically the soil’s mobilized shear resistance sensitivity to a change in effective stress. The transfer also leads to a temporary, with continued strain, stiffening and strengthening of the soil. Overconsolidation also produces this transfer, without significant, if any, increase in bonds. This seemingly common transfer phenomena may help to explain the important non-bond contribution to such soil behavior as “aging” and “quasi-preconsolidation” effects.

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1. Introduction

Dr. Bjerrum always maintained a lively interest in the subject of fundamental soil shear resistance behavior. His own doctoral research, reported in NGI Bulletin No. 5, dealt with the Hvorslev effective shear strength components. In his major state-of-the-art paper to the 8th International Conference he again reviewed fundamental behavior and generously referred to the work of Hvorslev and that of myself and co-workers. In both 1951 and 1973, and in the years throughout, he attempted to draw practical implications from fundamental shear strength research and theory.

My doctorate dissertation also dealt with the Hvorslev components, and through 1971 I actively pursued research in this area. This included a 1962–63 USA National Science Foundation Fellowship under Dr. Bjerrum at the NGI. After 1963 I corresponded with Dr. Bjerrum concerning my continuing research and he contributed in a substantial way. In view of these events I look upon the invitation to participate in this volume as a very suitable opportunity to bring much of this research together in one place, to update it since the last publication in 1965, and to suggest explanations for the observed shear mobilization behavior of soils tested at constant structure.

2. The constant structure Mohr envelope (CSME)

First, consider what the term “structure” means herein. We all know that soils have an extremely complex fabric involving a three dimensional arrangement of particles, with the particles having a wide range of size, shape and mineralogy. Furthermore, there exists a complex interplay of attractive and repulsive forces between particles depending on particle spacings and orientation, the electrical and chemical activity of the particles and of natural cementing agents between particles, the age of the soil deposit, its degree of saturation, the chemical content of the pore water, etc. To permit identifying a particular state of the fabric and internal forces composing such a complex material, the writer will simply use the catch-all term of “soil structure” or just “structure”.

As explained in T. W. Lambe’s beautiful work (Lambe, 1953), particle dispersion represents one of the important ways in which clay structure can change. Dispersion means a more parallel, less flocculated, orientation of the platy or elongated particles typical of clay minerals. It can result from chemical additives which reduce the internal attractive forces between particles, particularly if at the edge-face contact zones. Or, dispersion can result from the mechanical strains between particles during consolidation and shear. Strain changes structure. The remolding of sensitive soils and the resulting strength
change can provide dramatic evidence of the mechanical strength reduction consequences of soil structure change by strain, in this case often at constant void ratio or density. C. Crawford (1963) presented photographs illustrating a striking example of such a structure change by the strains of remolding.

The reader should now imagine a soil in which changes in the level of ambient effective stress will produce only negligible changes in particle orientation, packing, etc., therefore, negligible change in structure. Begin with an essentially identical set of isotropically consolidated soil specimens with each specimen subjected to a different level of initial effective stress. Then compress each to a failure in a similar manner in a conventional triaxial test machine, with failure occurring at 10% strain in all tests. Fig. 1 presents the conventional Mohr-Coulomb effective stress failure envelope obtained from such a test series. Each of the three specimens began with the same soil structure and each reached the same percent at failure along the same strain path. Therefore, the writer believes they all would have the same structure at failure. For this imagined case, the conventional failure envelope also represents a constant-structure Mohr envelope (CSME).

Dr. E. E. de Beer (1965) expressed a similar idea and he referred to the resulting failure envelope as the "real intrinsic curve." He plotted experimental intrinsic curves from tests on reassembled sands, with each test in a series beginning at the same void ratio after consolidation but before compression. Thus, he started compression with a very simple, disarranged structure, and one not sensitive to change by ambient effective stress. His "intrinsic curves" represent approximate CSMEs to the extent, unreported by de Beer, that identical void ratio and strain conditions existed at failure. Section 6.2.1 analyzes further this work by de Beer.

Consider the various envelopes also shows in the hypothetical example of Fig. 1., with each envelope representing the shear resistance mobilized by the soil at various, constant levels of strain less than the failure 10%. Each of these envelopes represents a constant-structure and condition and therefore represents another CSME for the same soil.

In this paper the writer chose to work with a constant structure envelope defined in terms of the conventional shear and effective normal stress coordinates and Mohr circles determined from major and minor principal stresses, σ’ and σ’0, only. This neglects the effect of the intermediate principal stress, σ’2. To the extent that σ’2 has an important effect on structure, this set of coordinates introduces inaccuracy. The concepts developed herein might, or might not, have significantly more generality in terms of octahedral stresses.

The reader might ask why introduce a sometimes fictitious, new type of Mohr envelope? The reason -- to investigate strength fundamentals without the serious complicating effects of varying soil structure. The slope of the CSME at any point, such as x in Fig. 1, represents the shear resistance mobilization sensitivity of that particular structure to a change in the level of effective stress. Furthermore, if the structure remains constant along such an envelope, then strength components such as real cohesion, or the alternate term "bond shear strength" used herein, remain constant. Then the intercept of the CSME with x = 0 line, denoted y0 in Fig. 1, represents the real mobilized bond shear resistance of that structure. The intercept with the y = 0 axis represents the mobilized effective tension resistance of that structure, σ’0. The slope at any point, dσ/df, expresses the sensitivity of that structure’s shear mobilization to a change in effective stress. An investigator can obtain these more fundamental strength properties with such an envelope from a CSME. He cannot from conventional Mohr envelopes, even in the form of constant strain contours, because volumetric strain, and therefore structure, usually varies very significantly at different levels of effective stress along conventional envelopes.

At this point the reader has only the de Beer tests on sands as evidence of the possible real existence of such CSMEs. De Beer approximated the CSME condition at failure and drew some important, more fundamental conclusions -- especially with respect to the curvature of such an envelope. However, Professor P. Row, by use of his famous stress-dilatancy theory, also produces such a structure, and therefore a special case of constant structure, envelopes mathematically via a structure-dilatancy correction. He thus eliminates the effects of different structure along a Mohr envelope and produces a no-dilatancy CSME. Use of this no-dilatancy CSME has permitted measuring some of the more fundamental soil shear strength properties.

The writer attempted to construct parts of CSMEs and then reason the behavior of complete CSMEs. He also eventually produced complete CSMEs by means of the experiments and their mathematical interpretation as described subsequently.

3. The IDS-test

3.1 GENERALIZING THE HVORLSLEV COMPONENTS

Starting in 1958, the writer began an experimental investigation aimed at determining the Hvorslev effective shear stress components as functions of strain. However, as explained in detail by Schmertmann & Otterberg (1963b), and Schnaurtehrn (1962a), these components, as originally defined by Hvorslev, prohibited any determination as functions of strain. The Hvorslev components first required a slight modification and a resultant generalization, as explained by Schnaurtehrn (1963a). This modification included a slight relaxation of the Hvorslev requirement of exactly the same void ratio for soil specimen to have equal cohesion, in favor of a definition that permitted small, seemingly negligible, differences in void ratio in favor of much better assurance of equal structure.

The problem required an experimental technique that would test a soil structure, subject to modification by strain, at two levels of effective stress. The change in mobilized shear resistance with a change in effective stress then would indicate that structure’s sensitivity to such a change, and therefore presumably measure its friction behavior. Extrapolating this change to ε’ = 0 would presumably produce the cohesion intercept. At first the writer designed the test designed to accomplish the separation of Cohesion and Friction as functions of Strain as the CSME-test. Later, as explained subsequently, the writer realized that CSME curvature prevented these components from being fundamental friction and cohesion and changed the test notation to the IDS-test. 'I' denotes that component seemingly independent of effective stress change and 'D' that component seemingly linearly dependent on effective stress change. The following presents an outline of the history and details of the performance of IDS-tests.

3.2 TEST DEVELOPMENT TO THE USE OF A SINGLE SPECIMEN

Consider a soil subjected to initial, or virgin, compression followed by simple unloading. Fig. 2 illustrates a unit change in effective stress during initial compression produces a much greater change in void ratio, and presumably structure, than similar change during unloading.

If one desires to change effective stress with a minimum of change in structure, one should confine effective stress changes to the rebound region designated by 3-4 in Fig. 2.

Imagine three initially identical specimens consolidated to point 3 in Fig. 2. Then allow one to rebound to q’1 at point "A", another point "B" and third to point "C".

Although each reaches a progressively reduced effective stress, all other aspects of structure should remain almost the same. Further imagine each of these almost identical specimens subjected to similar compression tests in such a way that q’1 never exceeds point 3 in Fig. 2. Fig. 3 illustrates results qualitatively typical of many experiments. Because specimen A has a greater effective stress at all strains than B and B greater than C, the soil structure at A can mobilize greater shear resistance than at B at all strain, and B greater than C. At any strain ε 1, either before, at, or after any definition of failure, the mobilized resistances of specimens A, B and C can be represented by three Mohr circles all applicable to that soil at ε 1. Fig. 4 illustrates such circles. The envelope defined by these circles, represents a piece of the CSME that applies to the soil shown in Fig. 2 after shear mobilization to strain ε 1, in Fig. 3.

The slope of this piece of the CSME expresses the shear resistance sensitivity of that constant structure to a probing change in effective stress -- a change sufficient to cause a measurable change in shear resistance but not sufficient to produce a significant change in soil structure. Note the slope λ(σ’0 - τ) = dσ’/dε and D = τ’ - dτ’/dε.

As shown by Schmertmann & Otterberg (1963b) any piece, such as x-y, of a CSME appeared linear in their
3.3 VARIETY OF IDS-TESTS

It also proved possible and sometimes useful to perform IDS-tests of different types. At first the writer performed only tests where each of the curves in Fig. 3 represented the stress-strain mobilization at a constant q’
. The purpose of the constant-q’ form of the test was to better assure that major structural changes would not take place in the soil during strain as a result of q’ increasing the pre-consolidation q’
 at point 3 in Fig. 2. But, maintaining constant-q’ required continuous adjustment of pore pressure during the strain. In principle any type of test that develops the Fig. 3 curves between two or more different levels of effective stress will provide the data for the required Mohr circle separations at constant stress in Fig. 4. Experience has shown that curves similar to A and C would also result from IDS-tests at constant-q’ or constant-q, or two levels of constant volume. For the constant-volume IDS-tests the investigator controls volume between two pre-selected levels and has to measure pore pressures to obtain the effective stress differences between curves A and C—see Schmertmann (1963a, p. 265) for an example.

Furthermore, the investigator need not restrict the IDS-test to the triaxial compression test at controlled strain rate. In principle he or she can perform an IDS-test with any shear testing equipment capable of generating stress-strain curves with a controlled difference in effective stress conditions. For example Schmertmann (1963b) performed IDS-tests (the "X" points in Fig. 11) with strain controlled, direct, simple shear equipment. Topping (1970) performed the IDS-tests using axial extension and produced results testing kaolinite similar to those from axial compression tests. The IDS-test can also be performed using stress control methods and two specimens, as shown by Schmertmann (1963a, pp. 267-268).

4. The constant-I-component

Altogether, the writer has performed, or supervised the performance of, about 500 IDS-tests at the University of Florida. These tests produced much information about the I and D components and their possible significance. This paper can only summarize some of these results. One of the most important discoveries concerned the remarkable and unexpected behavior of the I-component. As explained previously, the I-component represents a cohesion-related concept. The D-component seems friction-related. The fundamental work of Hvorslev suggested that any effective "true" cohesion, and therefore presumably also the I-component, should vary with the variables of void ratio, the degree of dispersion or flocculation of the soil structure, the nature of the pore fluid, previous stress history effects such as overconsolidation, clay mineralogy, etc. But, I, as measured by its maximum value I
 (see Fig. 5), proved essentially independent of all of these variables. The following summarizes much of the data leading to this strange conclusion.

4.1 THE CONSTANCY of I

4.1.1 Particle dispersion

Table 1 presents a comparison of I
 from remolded kaolinite and Bothwell blue clay, before and after the addition of trace amounts of sodium phosphate dispersants, judging by the large change in PI, the reduction in shrinkage limit (SL), and the large reduction in equilibrium void ratio at the constant effective stress conditions for all tests compared, the phosphates produced dramatic changes in structure. Yet, I
 remains virtually constant. Note that unless otherwise designated, the IDS-tests which produced these and subsequent data were of the constant-q’ type.

4.1.2 Void ratio and overconsolidation

In 1960 Dr. J. Hall (then a graduate student) and the writer performed a most informative series of IDS-tests using a large number of saturated, machine extruded specimens with initial structure virtually identical. We then subjected these specimens to various levels of overconsolidation and overconsolidation, after which we subjected them to a constant-q’ IDS-test. Fig. 6 presents the results obtained, showing both void ratio changes during consolidation and during the IDS-test itself, and I
 vs. q’/7. The writer performed the first series of tests, shown with solid symbols, and Dr. Hall performed the second (open symbols) over the summer of 1960 when the writer was absent. These two separate sets of data matched very well, and show clearly that I
 is virtually independent of final void ratio and instead depends linearly on the average value of q’, actually used during the one-specimen test curve-hopping procedure described in 3.2. Fig. 6 also illustrates the meaning of the average q’ value, I
.

We also performed a similar series of tests on a natural but remolded clay (Raini) with very different grain size distribution and mineralogy, and obtained very similar results.

The tentative conclusion was both surprising and inescapable—I
 did not vary with the void ratio changes, and presumably great structural changes, associated with virgin consolidation followed by rebound, provided one compared I
 using the same q’-r
.

4.1.3 Simple overconsolidation

Consider two points at the same q’-r
, such as B and B’ in Fig. 2, one normally consolidated (B) and one overconsolidated (B’). They both have the same effective stress but certainly different structure as a result of the void ratio reduction associated with overconsolidation. For example, test numbers 108 and 112 in Fig. 6 have a B-B’ relationship. Fig. 7(a) presents the IDS-test stress-strain curves obtained from tests 108 and 112, and also the subsequent check test H-45. Fig. 7(b) shows the resultant I-components as a function of strain. Note that I
 stays about the same, and perhaps even decreases somewhat as a result of the overconsolidation. In great contrast, Fig. 7(b) shows a dramatic increase in the stress-strain mobilization of the D-component, illustrated here via dp. As a further example of this important discovery, Fig. 8(a) and (b) show the same comparison for a remolded natural clay (Raini) with different mineralogy. To date we find this behavior in all soils tested. Clearly, the structure changes associated with overconsolidation had little if any effect on the I-component, but did dramatically affect the D-component.

4.1.4 Primary and secondary consolidation time effects

As also indicated by the notes in Fig. 6, varying the length of time allowed for the completion of primary consolidation from 24 to 120 hours, or whether or not an intermediate complete unloading occurred, had little
Consider the nature of the structural change that increases $d_e$. Fig. 9 also shows via a heavy solid line the low-strain part of the $d_e$-strain mobilization curve from a constant-$\phi$ ID5-test on deliberately dispersed kaolinite (by Na$_2$P$_2$O$_5$). O-EFPK, prior to specimen extrusion. As discussed in 4.1.1, and shown in Table 1, the addition of this dispersant did produce a dramatic dispersion of structure (destroys edge-face particle contacts and forms a more parallel particle arrangement).

Note the heavy line matches almost exactly the $d_e$ mobilization of the distilled water kaolinite after a 5-week secondary. This suggests that the structure change during secondary may be closely related to dispersion. Over-consolidation increases particle orientation and therefore disperses structure. The similar effect of secondary on $d_e$ supports the dispersion hypothesis. The writer discusses further this important hypothesis in Sections 10.5 and 10.9.

4.1.5 Anisotropic consolidation

In an extensive series of tests with remolded kaolinite and Boston blue clays reported in detail by Schmertmann and Hall (1951a), the authors showed (see examples in Figs. 21(a) and 21(b)) that the mobilization of the 1-component

IDS-test component obtained from a comparative series, again on remolded kaolinite, with all other test conditions the same. Note again that $I_n$ remains about the same (differences in strain-mobilization probably mostly experimental variations). Again in contrast, there occurs a dramatic increase in the strain-rate of $d_e$ mobilization, especially at low strains. Compare with Figs. 7 and 8 and it appears that secondary compression has much the same effects, at least at low strain, as simple overconsolidation.

Similarly, the structural change effects of longer secondary compression times produced, as with overconsolidation, structural changes which greatly affected $d_e$, but did not affect $I_n$.

### Table 2. $I_n$ vs. time for primary consolidation.

<table>
<thead>
<tr>
<th>Clay</th>
<th>Primary Time (Min.)</th>
<th>$I_n$ (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kaolinite</td>
<td>22</td>
<td>0.57</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>54</td>
<td>0.57</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>60</td>
<td>0.56</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>100</td>
<td>0.53</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>250</td>
<td>0.56</td>
</tr>
<tr>
<td>Erod</td>
<td>25</td>
<td>0.52</td>
</tr>
<tr>
<td>Erod</td>
<td>55</td>
<td>0.52</td>
</tr>
<tr>
<td>Erod</td>
<td>80</td>
<td>0.54</td>
</tr>
</tbody>
</table>

![Fig. 7. Comparisons of shear component mobilization in NC and DC (R = 3) extruded kaolinite clay.](image)

![Fig. 8. Comparisons of shear component mobilization in NC and DC (R = 4) extruded Erod clay.](image)

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Fig. 6. Demonstration that $I_n$ independent of void ratio and simple laboratory overconsolidation in extruded, distilled water kaolinite (O-EFPK).

The writer investigated further the effect of different primary times on the $I_n$ value in remolded kaolinite and Erod clays by providing progressively more internal and external drainage aids to speed consolidation. Table 2 presents the results and shows that changing the primary time by a factor of 10 appeared to have no effect on $I_n$.

The writer also tried various combinations of sudden loading and constant-rate loading to produce the consolidation, various times in secondary following by unloading and reloading, etc. as explained briefly in Table 3. In all cases consolidation ended with a normally consolidated clay at the same final effective stress and produced the same $I_n$. It seemed clear that the method of primary consolidation had no effect on $I_n$.

The effects of allowing progressively increasing times for secondary compression (under constant effective stress conditions) proved informative. Fig. 9 shows the...
Table 3. Method for hydraulic consolidation. (All tests at approximately \(\varepsilon_1' = 3.30\) and \(\varepsilon_5' = 2.60\) kPa/cm²).

<table>
<thead>
<tr>
<th>Clay</th>
<th>Method of consolidation</th>
<th>(I_0) (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extruded Kaolinite 4</td>
<td>6 days in &quot;secondary&quot;, without drainage, then immediate IDS-test</td>
<td>0.56</td>
</tr>
<tr>
<td>Extruded Kaolinite 5</td>
<td>6 days in secondary at (\varepsilon_1' = 0.50) after consolidation to 3.50, then returned to (\varepsilon_1' = 3.50) for 3 day</td>
<td>0.57</td>
</tr>
<tr>
<td>Extruded Kaolinite 6</td>
<td>7 days at (\varepsilon_1' = 3.50), then held at 0.20 for 8 days; then returned to 3.50 for 3 day</td>
<td>0.57</td>
</tr>
<tr>
<td>Extruded Kaolinite 7</td>
<td>Consolidated to (\varepsilon_1' = 3.50) for 1 day and tested, both with back pressure of 3.00</td>
<td>0.55</td>
</tr>
<tr>
<td>Extruded Kaolinite 8</td>
<td>Consolidated to (\varepsilon_1' = 3.50) with constant rate of loading for 10 days; then held at 3.50 for one day</td>
<td>0.63</td>
</tr>
<tr>
<td>Extruded Kaolinite 9</td>
<td>Consolidated to (\varepsilon_1' = 3.50) with constant rate of loading for 10 days; then held at 3.50 for one day</td>
<td>0.59</td>
</tr>
</tbody>
</table>

during the IDS-test following anisotropic consolidation to various \(K'_v\) values, including \(K'_v\), did not change the \(I_0\) component with subsequent compressive strain. Once again, the \(D\) (or \(d_e\)) component changed drastically by increasing as \(K'_v\) decreased. Note that in this reference, \(c' = I_0\) and \(\tan\phi' = d_e\). Once again, the soil structure differences associated with different levels of \(K'_v\) consolidation did not produce changes in \(I_0\).

4.1.6 Clay mineralogy and grain size

Table 4a presents a list of the \(I_0\) values obtained from constant-\(\varepsilon_1'\) IDS-tests on remolded clays with a variety of mineralogy and plasticity. Despite the \(P_f\) varying from 4 to 10% and clay minerals from kaolinite to montmorillonite, the average \(I_0\) values at the same \(\varepsilon_1'\) test levels vary from only 0.47 to 0.62, with most around the average 0.53 kg/cm². Again the writer concluded that the likely major differences in soil structure associated with these major differences in clay mineralogy have only a minor effect on \(I_0\).

Not all soils will produce the same \(I_0\) at the same effective stress levels. For example, consider the non-plastic quartz soils listed in supplemental Table 4b. One might expect that non-plastic soils would not have a cohesion-related component. This proved untrue as shown in Table 4b. However, the writer observed a definite trend of reducing \(I_0\) with increase in grain size at the same effective stress, and will return to this important point in subsequent Section 10.

4.1.7 Rate of strain effect

Subsequently in their closures (1962) presented data from a "7-test series of constant-\(\varepsilon_1'\) IDS-tests on

Table 4a. Data indicating \(I_0\) approximately independent of clay mineralogy at constant effective stress.*

<table>
<thead>
<tr>
<th>Remolded clay</th>
<th>(P_f) %</th>
<th>&lt;0.002 mm %</th>
<th>Activity</th>
<th>Clay minerals</th>
<th>(I_0) (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dispersed kaolinite (Q-EPK)</td>
<td>4</td>
<td>60</td>
<td>0.07</td>
<td>Kaolinite – 99%</td>
<td>0.55-0.57</td>
</tr>
<tr>
<td>Ekaire (residual)</td>
<td>9</td>
<td>20</td>
<td>0.45</td>
<td>Kaolinite – 15%</td>
<td>0.50-0.55</td>
</tr>
<tr>
<td>Jacksonville</td>
<td>14</td>
<td>13</td>
<td>1.08</td>
<td>Montmorillonite – 10%</td>
<td>0.55-0.57</td>
</tr>
<tr>
<td>Benton blue</td>
<td>19</td>
<td>53</td>
<td>0.36</td>
<td>Illite – 10%</td>
<td>0.50-0.55</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>21</td>
<td>60</td>
<td>0.25</td>
<td>Chlorite – 25%</td>
<td>0.50-0.55</td>
</tr>
<tr>
<td>Lake Alluaug</td>
<td>105</td>
<td>85</td>
<td>1.24</td>
<td>Montmorillonite – 85%</td>
<td>0.55-0.57</td>
</tr>
</tbody>
</table>

* All \(I_0\) results from constant-\(\varepsilon_1'\) IDS-tests using: isotropic consolidation to 3.50 kg/cm² \(\varepsilon_1'\) = 3.30 kg/cm², \(\varepsilon_5'\) = 2.60 kg/cm².

Table 4b. Supplemental data from non-plastic soils

<table>
<thead>
<tr>
<th>Soil</th>
<th>(P_f) %</th>
<th>&lt;0.002 mm %</th>
<th>Clay minerals</th>
<th>(I_0) (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crushed quartz</td>
<td>0</td>
<td>0.007</td>
<td>Quartz</td>
<td>0.40</td>
</tr>
<tr>
<td>Beach sand</td>
<td>0</td>
<td>0.14</td>
<td>Quartz</td>
<td>0.22</td>
</tr>
<tr>
<td>Ottawa sand</td>
<td>0</td>
<td>0.8</td>
<td>Quart</td>
<td>0.08</td>
</tr>
</tbody>
</table>

kaolinite in which they varied the constant rate-of-strain by a factor of over 5000. They showed, at least for this clay, that rate-of-strain did not significantly affect the \(I_0\) value. However, reducing the rate-of-strain did significantly increase the strain-ratio of \(d_e\) mobilization. Once again, any structural changes from this variable appeared to affect the D-component and not the I-component.

4.1.8 Type of pore fluid

In 1962 the writer performed a test series involving only 5 constant-\(\varepsilon_1'\) IDS-tests with kaolinite saturated by water and other fluids. The same kaolinite powder used previously in saturated, extruded kaolinite specimens was here first compacted into a mold while still hot from drying in the oven. A vacuum was then applied to the top of the specimen and fluid allowed to come in from the bottom until near-saturated. All specimens were then consolidated isostatically to 3.50 kg/cm² and IDS-tested using curve hopping between the constant-\(\varepsilon_1'\) values of 2.30 and 2.60 kg/cm² and using the same compressive strain rate for each of the 5 tests. Table 5 presents supplementary data on the pore fluids used, etc. Despite the dramatic difference in dielectric and viscosity properties of these fluids, they all produced about the same I-component, as shown in Fig. 10. However, once again the \(d_e\) component varied dramatically with the pore fluid and most drastically with its viscosity.

As also shown in Fig. 10, \(I_0\) increased from about 0.55 kg/cm² for the water-saturated extruded kaolinite, compared to about 0.75 kg/cm² for the specimens first compacted dry and then saturated. Although the dry material had a higher void ratio, presumably it also had a greater number of solid-contact points which somehow

\[ \text{Fig. 10: Effect of different pore fluids on } I_0 \text{ and } D \text{ mobilization in kaolinite, } \text{dry.} \]

4.2. \(I_0\) NOT COHESION

All, or at least many, of the variables discussed in 4.1 should have had a dramatic effect on any true soil cohesion, assuming such cohesion to derive from the interplay of particle geometry and attractive and repulsive force between particles – particularly at or near the zones of closest particle approach or contact. Yet, \(I_0\) and \(l_0\) appear independent of these variables. The conclusion seems inescapable that the I-component cannot represent true soil cohesion in the sense that soil engineers ordinarily imagine it. However, the \(\varepsilon_1'\) = 0 axis intercept must include any true cohesion.Either true cohesion has a much different origin and behavior than visualized or \(l_0\) and \(I_0\) result primarily from other effects. Perhaps true cohesion represents such a small part of \(l_0\) that significant changes in such cohesion from changes in the 4.1 variables does not change \(I_0\) sufficiently to detect easily. The writer believes this "minor-part-of-I" explanation.

Table 5. Test conditions for constant-\(\varepsilon_1'\) IDS-tests using kaolinite saturated with different pore fluids (match with Fig. 10).

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Porous fluid</th>
<th>Bulk properties</th>
<th>Computed void ratios</th>
<th>Time for primary consol. (min.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>255</td>
<td>Dry nitrogen</td>
<td>0.02</td>
<td>1</td>
<td>0.000</td>
</tr>
<tr>
<td>256</td>
<td>Benzene</td>
<td>0.65</td>
<td>2</td>
<td>0.899</td>
</tr>
<tr>
<td>257</td>
<td>Methyl alcohol</td>
<td>0.59</td>
<td>31</td>
<td>0.810</td>
</tr>
<tr>
<td>258</td>
<td>Distilled water</td>
<td>1.00</td>
<td>81</td>
<td>0.998</td>
</tr>
<tr>
<td>259</td>
<td>25% alcohol</td>
<td>2.10</td>
<td>75</td>
<td>1.065</td>
</tr>
</tbody>
</table>

Av. of three Distilled water * 1.028 0.895 0.898 99.9 70

* Specimens machine extruded in saturated condition.
5. 1-component linear with effective stress

5.1 CONSIDERING $I_a$

The reader probably noted in Fig. 6 that $I_a$ varies linearly with the average of the two constant $\sigma_1''$ levels of curve hopping in these constant-$\sigma_1''$ IDT-tests. This behavior produced a paradox! The 1-component represents that portion of the mobilized shear resistance of a soil that appears, after linear extrapolation, to be independent of constant structure Mohr envelope. Fig. 11 shows $I_m$ versus the average effective stress $\bar{\sigma}'$, of the pieces of the CSME defined by the two Mohr circles obtained at each strain in an IDT-test (see Fig. 4). Table 6 presents the legend for the various symbols used in Fig. 11.

Fig. 11 presents overwhelming evidence that all the soils tested possess a fundamental shear resistance behavior that, when probed by the IDT-test over the stress ranges investigated, produces $I_m$ that varies linearly with CSME effective stress. Fig. 12 presents a summary of the various linear relationships detailed in Fig. 11. Note the remarkable similarity in the slopes of most of the linear relationships summarized in Fig. 12.

5.2 CONSIDERING $I$ AT ANY STRAIN

Section 5.1 treated only the maximum value of the $I$ component, $I_m$. Almost all IDT-tests have shown that $I_m$ usually occurs at very low strain. Each of the individual tests reported in each of the various parts of Fig. 11 probably had its $I_m$ occur at about the same low strain for all tests in that part. However, $I_m$ was taken at whatever strain it occurred and Fig. 11 includes no specific attempt to treat strain as an independent variable.

K.-H. Ho (1971) showed that a linear relationship between $I$ and $\sigma_1'''$ existed at all strains investigated in each of the three artificial soils he tested. Table 7 summarizes the results from his tests on duplicate specimens of remolded normally and overconsolidated kaolinite.
Table 7. K.-H. Ho's test to check linearity of Iₒ vs. αₗ at any strain.

<table>
<thead>
<tr>
<th>Soil</th>
<th>Type IDS-test</th>
<th>No. tests in series</th>
<th>Av. linear corr. coeff.*</th>
<th>αₗ values checked (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC kaolinite</td>
<td>const - αₗ / nₒ / αₗ /²</td>
<td>13</td>
<td>0.99</td>
<td>1, 2, 3, 4, 5</td>
</tr>
<tr>
<td></td>
<td>- αₗ / nₒ / αₗ /² - vol.</td>
<td>3</td>
<td>0.99</td>
<td>1, 2, 3, 4, 5</td>
</tr>
<tr>
<td>OC kaolinite</td>
<td>const - αₗ / nₒ / αₗ /²</td>
<td>3</td>
<td>0.98</td>
<td>1, 2, 3, 4, 5</td>
</tr>
<tr>
<td></td>
<td>- αₗ / nₒ / αₗ /² - vol.</td>
<td>3</td>
<td>0.99</td>
<td>1, 2, 3, 4, 5</td>
</tr>
<tr>
<td>Cemented glass beads</td>
<td>const - αₗ / nₒ / αₗ /²</td>
<td>9</td>
<td>0.94</td>
<td>0.1, 1.5, 2</td>
</tr>
</tbody>
</table>

* Range for all tests at all strains = 0.92 - 1.06.

and also on duplicate specimens of glass beads cemented with "hydrocal". He used small sample statistical theory and computed the correlation coefficients for the least-squares linear relationship between Iₒ and αₗ / nₒ / αₗ /². He found for both soils, and with each of the four different types of IDS-tests he used, and at all strains investigated (including well past the strain of Iₒ = 1.4) that the correlation coefficients varied between 0.92 and 1.00. Table 7 lists the average values of correlation coefficient for the 1.4 to 5 levels of constant strain considered in each test series. He concluded that in all of his tests Iₒ vs. αₗ / nₒ / αₗ /² was linear at every strain investigated. Although He did not prove the generality of this linear relationship at every strain for all soils, he did prove the principle that this linearity exists for at least some soils. The writer has little doubt it exists for most soils.

6. The shape of the CSME

6.1 THE SHAPE EQUATION

Equation (1) describes the piece of the CSME illustrated in Fig. 4.

\[ \tau = I_0 + d \cdot \alpha' \]  

Equation (2) expresses the slope of this piece of envelope:

\[ d = \frac{d \tau}{d \alpha'} \]  

The data on Fig. 12, which summarizes the various parts of Fig. 11, show clearly that the L-component behaves in accord with equation (3):

\[ I_0 = \tau = I_0' + \alpha'_L \cdot \alpha' \]  

Putting equations (2) and (3) into (1), and dropping the understood subscript, produces:

\[ \tau = I_0' + \frac{\beta_0}{\alpha'} \cdot \frac{d \tau}{d \alpha'} \]  

from which:

\[ \frac{d \tau}{d \alpha'} = \tau - I_0' - \frac{\beta_0}{\alpha'} \cdot \frac{d \tau}{d \alpha'} \]  

Solving Equation (5) produces the equation (6), which expresses the shape of a constant structure Mohr envelope over the stress range in which Equation (3) holds:

\[ \tau = I_0' + \frac{\beta_0}{\alpha'} \cdot \alpha' \]  

Equation (6) seems like an unlikely expression for a shear resistance envelope. Its slope becomes infinite at the \( \alpha' = 0 \) axis and becomes zero and negative at high values of \( \alpha' \). Fig. 11a illustrates the CSME shape using \( I_0 ', \beta_0 \) values more or less typical of those obtained from IDS-tests on remolded clays. Fig. 11b blows up the low-\( \alpha' \) range of this CSME. Fig. 11a indicates the approximate maximum \( \alpha' = 50 \) kg/cm² used on clays for which this shape CSME applies. It appears that this CSME would not be reason-

able for \( \alpha' > 7 \) kg/cm². This does not necessarily mean an error exists in Eq. (6). It may mean that the particular structure described by this CSME cannot exist, or at least cannot mobilize additional shear resistance, at \( \alpha' \) greater than \( 7 \) kg/cm². For example, it could cause crushing at the grain contacts. Also consider equation (7) in the next Section 6.2.1. The slope of this CSME remains reasonable to extremely high \( \alpha' \) much beyond the stress levels which will produce extensive breaking of the grains.

Fig. 12b shows that the CSME appears reasonable at least to the minimum \( \alpha' \) level that effective stress compression, constant-\( \alpha' \), IDS-tests can probe -- as shown by the smallest possible Mohr's circles and the piece of the CSME so determined. Note that several clays were tested down to near-minimum possible \( \alpha' \) levels (Fig. 11a) (b) (f) and (g) and equation (3) appears to remain valid. The very steep slopes near the \( \alpha' = 0 \) axis predicted by Eq. (6) only occur very near the axis and have no significant effect on any test interpretations made herein.

The dimensionless \( \alpha' \) and \( \beta_0 \) terms in equation (6) quantify very different aspects of soil structure. As described further in Section 10.2, \( \beta_0 \) seems to be a grain contact crushing constant. Its value does not change with the pressure units used for \( I_0 ' \) and \( \alpha' \). But, \( \alpha' \) seems to depend on the soil friction, dilatancy and dispersion aspects of structure. As shown in Fig. 12b one can also think of \( \alpha' \) as a reference (Delta\( \alpha' \)) expressed by the secant \( d \tau / d \alpha' \) level 0 to 1 interval. Its magnitude depends on the units used for \( \alpha' \).

6.2 INDEPENDENT EXPERIMENTAL CHECKS OF CSME SHAPE

6.2.1 de Beer tests

As noted in Section 2.2, E. de Beer (1965) has already presented data which at least approximately defined CSME, which he called "intrinsic curves". This writer reconstructed intrinsic curves from de Beer's Fig. 1 and 5 for the two remolded sands he investigated. These intrinsic curve envelopes apply only to the failure condition from drained, triaxial compression tests. Furthermore, de Beer defined his effective stress coordinate as the mean, or oedometric, effective stress, \( \alpha' \). After carefully plotting the intrinsic curves from his data, the writer constructed tangents graphically to these curves and obtained the L-component intercepts with the \( \alpha' = 0 \) axis. Because the envelope represents a drained failure condition, denote the L-component as \( I_0 L \). For all cases checked \( I_0 L \) started at 0 when \( \alpha' = 0 \) and then increased almost linearly with \( \alpha' \) at a slope whose tangent (\( \beta \) in equation (6)) varied between 0.04 and 0.06. Equation (6) fits the de Beer intrinsic curves rather well. For example, Equation (6) in the form:

\[ \frac{\beta_0}{\alpha' (\alpha')^2} \]  

fits the data for Molnár at \( \alpha' = 38.9 \% \) as measured by the conicose scat ing angles presented in Table 8. The comparative agreement demonstrates how very well equa-

Table 8. Comparison of de Beer's intrinsic curve shape and that predicted from Equation (6).

<table>
<thead>
<tr>
<th>( \phi' ) (°)</th>
<th>Curve secant e' (Degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>de Beer</td>
<td>Eqns. (7)</td>
</tr>
<tr>
<td>0.1</td>
<td>45.0</td>
</tr>
<tr>
<td>0.2</td>
<td>40.3</td>
</tr>
<tr>
<td>0.3</td>
<td>34.0</td>
</tr>
</tbody>
</table>

6.2.2 IDS-tests using different \( \alpha' \) curve-hopping levels

The CSME has a continually changing slope, as indicated by equation (6). Then, as illustrated in Fig. 44 using an expanded \( x \) scale for clarity, the larger the curve hopping \( \alpha' \), the smaller the L-component. The relative magnitude of the L-component obtained using different levels of \( \alpha' \), compared to some reference level, can be predicted by appropriate use of equation (6). Such a mathematical prediction can then be carried out experimentally to verify or refute the reasonableness of equation (6). Table 9 presents a comparison of mathematical and experimental L-component ratios referenced to a standard \( \alpha' \) of 20 \% (often used in constant-\( \alpha' \), IDS-tests). The very good agreement between such use of equation (6) and experiments further supports the validity of the CSME shape equation (6).

The use of Equation (6) also permits computing the error in the L-component that occurs when extrapolating to the \( \alpha' = 0 \) axis on the basis of a secant approximation of the curve of the CSME determined in an IDS-
The curves from the graphical fit and those obtained from equation (6) matched so well that a graphical comparison would make any separation difficult to see. Instead, consider the comparative materials presented in Table 10. First note the very high curve-fit correlation coefficients, which confirm the excellent fit provided by Equation (6) for the envelopes obtained directly from the CSE tests. Because Hio (1971) tested these same soils, Table 10 also includes a comparison between $I_P$ and $\beta$ obtained by Hio using the IDS-test method (Section 7.3) compared with $I_P$ and $\beta$ obtained from the best-fit Equation (6) to the CSE-tests. The agreement seems reasonable and further confirms the validity of the CSE-test equation (6).

The CSE-test as described above allows a small amount of additional axial strain to take place as the load against the load cell diminishes with decreasing effective stress and the load cell, fixed only at one end, compresses the sample specimen slightly with this release. Table 10 includes the axial compressive strain at the initial equilibrium (after step 1) and the final strain at the 5th, $\epsilon_{eq}$ almost 0, equilibrium condition. The difference between these strains varied from 0.09 to 0.16%. The CSE-test as presented assesses this additional strain, as well as any unmeasured changes in radial strain, produces a negligible change in the soil structure during this method of probing that structure's ability to mobilize shear resistance at varying levels of effective stress. The CSE-test may not prove suitable for soils that swell appreciably during the change in stress condition in this test, and thus perhaps significantly change structure during this reduction.

Streetsman's experience with the CSE-test also indicates that the investigator must allow considerable time to reach her specified almost-equilibrium state after each step reduction of effective stress (change in deviator stress per hour less than 1% of total change during each step reduction). She required a total of 4 days for the 5-step tests on glass beads and 7 days for kaolinite.

6.3 CONCLUSION REGARDING SHAPE

From the success of the independent checks described in 6.2.1 and 6.2.2, and 6.2.3 in this section, it is established that Equation (6) adequately describes the shape of the CSEME for the range of soils and stress levels investigated. However, the experiments described herein provide only limited data on soils with $P_I$ exceeding 45%. Such clays, especially if they present swelling complications, may, or may not, behave differently from equation (6).

7. Determining the bond shear resistance from the CSEME

7.1 DEFINITIONS

As introduced in Section 2., the CSEME (illustrated in Fig. 1) has an intercept with the $\epsilon_{eq} = 0$ axis. This intercept, denoted $I_P$, must represent the shear mobilization cap-

ability, when $\epsilon_{eq} = 0$, of the soil structure whose CSEME produces that $I_P$ intercept. The writer assumes that any shear resistance at this condition must result from internal particle–particle bonds within the soil. This research does not distinguish between the many possible causes of such internal bonds, but simply sums their effects and designates the total mobilized bond resistance as $I_P$. For a given soil, following a given stress path, $I_P$ attains some maximum value, $I_P_{max}$, denoted $I_P$ for simplicity. $I_P$ represents the mobilized bond shear resistance and $I_P$ the bond shear strength. As with any other shear resistance component, $I_P$ and $I_P$ may be path dependent.

The writer recognizes that a more fundamental explanation for friction behavior relates friction to the stress-dependent making and breaking of the numerous atomic bonds that form between stress reductions forced into contact, and thus perhaps significantly change structure during this reduction.

Streetsman's experience with the CSE-test also indicates that the investigator must allow considerable time to reach her specified almost-equilibrium state after each step reduction of effective stress (change in deviator stress per hour less than 1% of total change during each step reduction). She required a total of 4 days for the 5-step tests on glass beads and 7 days for kaolinite.

7.2 THUS CSE-TEST METHOD

As described in Section 6.2.3, some soils permit a direct determination of the CSEME to almost the $\epsilon_{eq} = 0$ intercept. One can then find the intercept by fitting Equation (6) through the part of the CSEME obtained from the CSE-test and using the resulting formula to obtain the value of $I_P$. Table 10 includes $I_P$ values obtained by this method and they agree approximately with those obtained by Hio (1971) using the IDS-test methods described in the next section.

7.3 THE IDS-TEST METHOD

The IDS-test produces only a small piece of the CSEME, but this does for a spectrum of CSMEs as those develop with changing strain. In Section 5. the writer presents experimental results that seem to prove that the $I_P$ component varies linearly with effective stress over the full range of effective stress investigated, including as close to the $\epsilon_{eq} = 0$ axis as feasible in our experiments.
extrapolation for \( I_c \). On the other hand, the CSE-test requires only a 1-level elimination of the effects of effective stress to extrapolate for \( I_c \), but at only one strain. However, the CSE-test can be repeated on the same specimen brought to a new, greater strain after each test.

### 7.4 Using the IDS-Test Method on Undisturbed Soils

This section presents a number of examples of the use of the IDS-method to determine \( I_c \). The first example reports on tests with artificially prepared quartz silt. The remainders involve undisturbed samples using, in part, data from other investigators who perhaps inadvertently performed IDS-tests. Only the test series on the Manglerud clay had the specific intent of determining bond strength.

#### 7.4.1 Partially saturated quartz silt

The \( I_c \)-\( \Psi_c \) results reported in Fig. 11, part (b), came from either dry or completely saturated specimens of a re-assembled quartz silt. We also tested this silt in a partially saturated state. One group of 5 constant-\( \Psi_c \)-IDS-tests on this silt had dry densities and degrees of saturation within the relatively narrow range of 74.1 to 79.4 pcf and 114 to 19.6 % respectively. Fig. 15 shows the \( I_c \)-\( \Psi_c \) results from these tests, together with the \( \beta \)-slope determined from Fig. 11(a).

Of course, the capillary attraction forces resulting from partial saturation produced an apparent bond in the partially saturated specimens. Assuming the \( \beta \)-slope through the partially saturated tests equals that through the dry and saturated tests, and extrapolating to the \( \psi = 0 \) axis, produces an \( I_c \) intercept of \( I_c = 0.27 \) kgc/m³. As also shown in Fig. 15, this results from a capillary effective stress of 1.5 kgc/m³. This example shows how an engineer might by this method evaluate the apparent bond contribution of partial saturation or the equivalent effective stress due to partial saturation.

#### 7.4.2 A natural cemented sand

The writer cut specimens for constant-\( \Psi_c \)-IDS-tests from an undisturbed block of air-dry, medium to coarse sand, cemented with 5 % kaolinite, which he obtained from the vicinity of Tupelo, Mississippi, U.S.A. For a further description of this soil, as well as example IDS-test data, see Schnurnann (1962b, pp. 173, 179). Although so two specimens from this natural block of soil could be considered identical due to obvious variability in cementing, 6 of the specimens appeared similar with initial void ratios varying between 0.72 and 0.82 and degrees of saturation between 57 and 98 %. Fig. 16 presents the

#### 7.4.3 Undisturbed Norwegian quick clay

As noted in the introduction to section 7.4, this series of tests had the specific purpose of evaluating bond strength. The soil tested was Manglerud quick clay, which the NGI used previously, and subsequently, as a test clay for research purposes. This series consisted of 6 constant-volume IDS-tests, 5 in conventional triaxial compression and 1 in simple, direct shear. NGI personnel obtained these samples and handled them with great care. The pore pressure measurement system employed a back pressure of 6.0 kgc/m³. The strain rate was held at 1% per day axial compression, and the difference between the two constant-volume states of the soil was about 0.01% total volume. The interested reader can find all the details from these tests in an NGI Internal Report by Schnurnann (1963d).

Fig. 17 presents the results in \( f \) vs. \( \psi \) plot using expanded scales. Each different shaped symbol represents a different test. The smaller, lighter form of the symbol denotes data obtained at strains less than ultimate shear failure. A dashed line connects these points until reaching the longest, emphasised point, which denotes the \( f \)-component at ultimate failure. Based on these results, the writer concluded that bond shear resistance at ultimate failure, \( f_{ult} \), in this quick clay equalled essentially 0.05 kgc/m², and probably equaled no more than 0.01 kgc/m² prior to this failure. Note that the ultimate shear strengths in these tests were low, varying from 0.12 to 0.33 kgc/m², with failure strain between 1.1 and 2.6 %.

#### 7.4.4 Two normally consolidated clays tested by "perfect sampling" IDS-tests.

The perfect sampling procedure as described, for example, by Norszyn and Seed (1965) involves the testing of two specimens of clay before and after a sample, perfect sampling procedure in the triaxial chamber. The procedure produces two specimens at almost the same structure but with different initial effective stress conditions. Comparing stress-strain behavior between the two specimens during subsequent undrained compression tests thus represents, in effect, a two-specimen version of a constant-volume IDS-test. Schnurnann (1965b) discusses this equivalency in more detail.

Norszyn and Seed (1965) investigated undisturbed samples of the San Francisco Bay mud in this manner and presented their results in sufficient detail for the writer to extract those data needed for an \( I_c \) extrapolation at undrained failure. Using these authors' data (their Figures 22, 23 and Table 6) the writer obtained the points plotted in Fig. 18 herein. Note that these points fall very close to a straight line, as expected. This line has a \( \beta = 0.05 \), also within the expected range. Extrapolating these data points to the \( \psi = 0 \) intercept produces a bond resistance at undrained failure, \( I_c = 0.07 \) kgc/m².

Ladd and Lambe (1963) presented the data from a similar set of perfect-sampling tests using normally consolidated clay from Kawasaki, Japan. These authors also provided sufficient information to use their test data to

\[ \text{Fig. 15: } I_c \text{ determination in compacted, partially saturated, ground quartz silt, 100% passing 355 sieve.} \]
7.5 AN APPROXIMATE, ONE SPECIMEN METHOD

The IDS-test method for extrapolating for \( I_{cu} \) described in 7.3 and 7.4 requires a series of at least 2 comparative tests on the same soil. Errors can result from the inevitable differences between supposedly duplicate specimens. Random errors are less serious provided one tests enough specimens. However, systematic errors may also occur, such as a progressive destruction of cohesion on bonds when consolidating to higher effective stresses prior to IDS-testing, or progressive swell-destruction of bonds resulting from progressive reduction of effective stresses prior to IDS-testing. For such reasons, as well as economy, consider the following 1-test method for estimating \( I_{cu} \).

Study of Fig. 12 shows that the \( \beta \)-slope values for remolded soils appear to vary over a rather limited range. For instance, \( \beta \) varies from 0.15 to 0.22 for cohesive soils. The undisturbed clay tests reported in 7.4 and 7.4 also fit this range. Fig. 20 illustrates this slope variation, with the average value of 0.185 indicated. Also plotted on this figure are the 1-test \( I_{cu} \) values from two constant-\( \sigma' \) IDS-tests on undisturbed clays. In the absence of better information on the \( \beta \)-slope applicable for each clay (as in (16e) for the BHC), an investigator can assume the average value. Then the vertical distance between the 1-point test and the average slope, or a more applicable slope if available, represents the estimated \( I_{cu} \) value for each of these clays.

8. Comparing \( I_{cu} \) with other methods

Now consider the very important test of how the bond resistance values discussed in Section 7 compare with results using other methods of predicting or estimating the fundamental cohesion strength component. The writer will now consider three such methods in order of increasing sophistication, ending with Rowe's useful stress-diagram method.

8.1 JAKOBSON'S ORIGIN COHESION

Jakobson (1953) proposed a field method that may produce a result closely related to the \( I_{cu} \) bond shear resistance. He first performed a series of vane tests with depth at several sites in Sweden where he found uniform, soft, normally consolidated clays with an underwater surface. He found the now well known linear variation of undrained shear strength with depth. A short extrapolation to the surface then gave him the undrained vane shear strength at a point of zero, pre-vane, effective stress. He termed this strength the "origin cohesion". In all three examples he reported origin cohesion between 0.05 and 0.08 kg/cm². Subsequent data by others shows such values as typical for post-Pleistocene clays. Note that Jakobson reported magnitudes of origin cohesion similar to those noted in Table 11. More important, note that the \( I_{cu} \) values of 0.07 and 0.06 kg/cm² reported in 7.4 for the undisturbed San Francisco and Kawasaki clays fall exactly within the range of Jakobson's origin cohesion.

In this method let us assume that everything (meaning initial effective stress, the effective stress changes due to vane insertion and rotation, the difference in soil structure as a result of varying effective stress, etc.) varies linearly with depth. Then extrapolating to the surface should produce the undrained strength at zero effective stress, and therefore the bond shear resistance at undrained vane failure.

The writer had the opportunity to try Jakobson's origin cohesion method in San Francisco Bay mud, but in clay somewhat different than that tested by Noorany and Seed (1965). Fig. 21 presents the results from 5 vane shear depth profiles below a thin drying crust at a tidal flat site in San Francisco Bay. These very consistent results, obtained with the high-quality Jonell and Nilson vane shear equipment, produce an origin cohesion of 0.09 to 0.11 kg/cm². Considering the possible difference in the clays (average water content 76 vs. 90%, liquid limit 81 vs. 88%, plastic limit 30 vs. 53%, and average field vane sensitivity 5 vs. 9, when comparing the Fig. 21 site with the N & S site), and the rate effects between field vane tests and N & S's laboratory tests, the writer believes 0.09 to 0.11 represents excellent agreement compared to 0.07 kg/cm² determined from Fig. 16.

8.2 TENSION TESTS

The writer (1972) performed a series of 16 hydraulic fracturing tests at three sites in Leda clay in Ottawa,
Table 13 demonstrates clearly that the two independent experimental techniques arrived at about the same value of $I_{so}$. Note that $I_{so}$ determined by axis extension IDS-tests gave essentially the same bond shear resistance as axial compression IDS-tests of the constant-$\phi$ type (0.08 kgc/m² in Table 11). The agreement improved further after correcting the Bishop tests, approximately, for the effects of progressive volume change during testing in accord with Rowe’s stress-dilatancy theory.

8.3 Rowe’s Stress Dilatancy Method

P. Rowe (1962) has proposed, and he and his colleagues have expanded upon, a comprehensive theory to correct laboratory shear strength test results for the internal and external energy demands associated with the volume change (dilatancy) effects of soil structure. His work represents one of the few examples in soil mechanics of a major theoretical discovery later confirmed by experiments. In contrast, the writer’s work presents experimental discoveries with attempts to fit a theory around them. Rowe’s dilatancy method compares a soil at different effective stress conditions after eliminating the effects of different soil structure by means of a mathematical technique. The writer’s IDS-method attempts to eliminate soil structure effects by experimental techniques. Both approaches have their relative advantages and disadvantages. Both aim at evaluating the more fundamental shear strength behavior of particular systems such as soil.

It is of course of great interest to compare results using these two very different approaches toward the same objective. K. H. Ho (1971) compared the methods using duplicate specimens of three artificial soils. He very carefully performed IDS-tests of various types and at the same time made the very accurate volume change measurements required to make the Rowe stress-dilatancy soil structure corrections. Table 14 presents the comparisons that Ho obtained between $I_{so}$ and the Rowe interparticle cohesion parameter $c_{s}$. (Rowe et al. 1984). Table 15 describes the same tests already reported in Table 7, but the $I_{so}$ and $c_{s}$ values represent average values from all the strains considered in each case. Table 14 demonstrates clearly that both methods produce about the same results in measuring the more fundamental cohesion parameter $c_{s}$ and $I_{so}$ at least on the average.

Table 13. topshe's Comparison between extension IDS-tests and effective stress tension tests using remolded kaolinite at different stress levels for a soil with a confidence interval of 95%.

<table>
<thead>
<tr>
<th>Type test</th>
<th>No. tests</th>
<th>$I_{so}$ (kN/m²)</th>
<th>95% confidence interval</th>
</tr>
</thead>
<tbody>
<tr>
<td>Const-$\phi$ axial</td>
<td>16</td>
<td>0.084 ± 0.025</td>
<td></td>
</tr>
<tr>
<td>G-B tension</td>
<td>11</td>
<td>0.112 ± 0.041</td>
<td></td>
</tr>
<tr>
<td>G-B tension with approx. stress dist. correction</td>
<td>11</td>
<td>0.084 ± 0.032</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 21: Results from shear tests in San Francisco Bay mud illustrating determination of Jacobson’s origin cohesion.

Thus, two completely independent approaches to the same problem have yielded very similar results.

The reader familiar with stress-dilatancy might well wonder why the IDS-tests produce seemingly accurate results for $I_{so}$ without applying stress-dilatancy corrections to the IDS-test data. The small, approximately negligible, difference in soil structure between the two comparative effective stress situations in the IDS-test accounts, in part, for this success. The soil at both effective stress conditions has approximately the same dilatancy rate at all strains. For example, Fig. 22 illustrates this using one of Ho’s tests on glass beads saturated with hydraulic. Ho and the writer have found similar parallel behavior in dilatancy rates in all comparative tests. Thus, the Rowe dilatancy correction factor $(1 - \Delta \phi/\Delta \sigma_{v})$, appropriately denoted $c_{s}$ herein, has at least approximately the same value for both the higher and lower effective stress conditions at any strain in an IDS-test.

A mathematical study of the problem produced the following equation:

$$I_{so} = I_{so} \left(1 + \frac{\Delta \phi}{\Delta \sigma_{v}} \right)$$

Neglecting to correct the K-component for dilatancy produces an error in $I_{so}$ in 10% or less over the range 0.92 $\leq \Delta \sigma_{v} \leq 1.23$ and 20% or less over the range 1.70 $\leq \Delta \sigma_{v} \leq 1.56$. A $\Delta \sigma_{v} > 1$ means an increase in volume with strain and an equation (8) correction ratio of less than 1.00.

However, when it comes to the linear extrapolation for the $I_{so}$ value as in the many examples in Sections 7.3 and 7.4, the situation becomes more favorable because the stress-dilatancy correction also changes $c_{s}$ in the same direction as the K-component. Table 15 presents the approximate error in $I_{so}$ one may expect due to neglecting the stress-dilatancy correction as made by Rowe. The actual error depends on the magnitude of $I_{so}$, the $\beta$-slope and the stress-dependent component $a_{d}$. Table 15 suggests a relatively minor error in $I_{so}$ with soil structures having dilatancy rates within the ordinary range.

Table 14. Ho's comparison between average values of $I_{so}$ and stress-dilatancy $c_{s}$ for all strains investigated.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Test Type</th>
<th>Average $I_{so}$ (kN/m²)</th>
<th>Average $\Delta \sigma_{v}$ (kN/m²)</th>
<th>Average $c_{s}$ (kN/m²)</th>
<th>$\Delta \phi/\Delta \sigma_{v}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC kaolinite</td>
<td>const-$\phi$</td>
<td>0.047 ± 0.059</td>
<td>0.101 ± 0.051</td>
<td>0.088 ± 0.128</td>
<td>1.30</td>
</tr>
<tr>
<td>all</td>
<td>const-$\phi$</td>
<td>0.080 ± 0.079</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>OC kaolinite</td>
<td>const-$\phi$</td>
<td>0.114 ± 0.065</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>all</td>
<td>const-$\phi$</td>
<td>0.064 ± 0.086</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Glass beads</td>
<td>const-$\phi$</td>
<td>0.056 ± 0.048</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>all</td>
<td>const-$\phi$</td>
<td>0.052 ± 0.033</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cemented glass beads</td>
<td>const-$\phi$</td>
<td>0.028 ± 0.040</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The last column in Table 14 includes the approximate, average dilatancy rate, $\Delta \phi/\Delta \sigma_{v}$, for some of Ho’s tests. In all cases these average rates are positive and therefore, from Table 15, $I_{so}$ would be too high as a result of neglecting the dilatancy correction. But Table 14 indicates that in only three of the six cases does the average $I_{so}$ exceed $I_{0}$. This comparison suggests that other factors have greater importance than the relatively minor effect of neglecting the stress-dilatancy correction to $I_{so}$. Finally, K. H. Ho (1971) did attempt dilatancy corrections to $I_{so}$ but found no further improvement in the $I_{so} - c_{s}$ comparison.

8.4 Conclusions and Comparisons with Other Methods

It seems clear from the general agreement success of these different alternate methods of estimating the bond shear resistance within cohesive soils that the IDS-test
also provides an independent method for determining bond resistance. One can argue about the relative accuracy of one method vs. another, but they all deal with fundamental shear behavior. The TDS-test CSME method and the stress-dilation method probably provide the most opportunity for learning about soil shear behavior and producing a practical spottor.

9. Time-transfer of IDS-components

Dr. Bjerrum developed a special interest in how time-effects relate to a soil's shear behavior. In Bjerrum (1975) he placed great emphasis on the observations by Schirmer and Hall (1961a) concerning the time-transfer from the J-component to the D-component during an anisotropic consolidation. P. Rowe (1957) also observed that the small, empirical effective stress cohesion intercept appeared to diminish with time. It is of course well known that clay increases undrained strength substantially with an increase in rate of strain. But, what do we know about rate-of-strain effects in the direction of low rates, especially with respect to the fundamental shear components?

9.1 RATE-OF-STRAIN EFFECTS

As discussed in 4.1.7, changing the time-rate of strain in one set of comparative IDS-tests on remolded kaolinite produced only minor changes in the I-component but a dramatic increase in the strain-rate of mobilization of the D-component. The writer also observed similar comparisons during more casual observations of results from tests on other clays.

However, as shown by Schirmer and Hall (1961a, p. 268 and Fig. 28), decreasing the average rate of strain in an IDS-test, by changing from a strain-controlled test completed in 1 day to a fixed-crest-controlled test completed in 3 days per increment, produced a large transfer from I to D. This transfer increased with strain and time. Note that some of the strain-controlled tests discussed in 4.1.7 also took 30 days. It appears that forcing the strain, even very slowly, tends to maintain the I-component, while allowing strain adjustment under constant stress conditions tends to transfer I to D.

9.2 PRIMARY AND SECONDARY CONSOLIDATION TIME EFFECTS

Section 4.1.4 has already presented a discussion of the non-importance of such time-effects to the I-component. Table 4 and 5 indicated the time required for primary consolidation and the manner of performing this consolidation appeared to have no significant effect. Fig. 10 shows that lengthening secondary time by a factor of about 10 had almost no effect on $J_{II}$. Once again, we observed a dramatic increase in the low-strain rate of mobilization of the D-component.

An important corollary result from progressively longer times in secondary compression is that such compression can produce a substantially stiffer soil structure. For example, Schirmer (1965a) showed the initial tangent modulus from the testing of specimens similar to those reported in Fig. 10 increased by a factor of about 10 as a result of longer secondary times. Bjerrum and Lo (1963) also observed such stiffening, but to a lesser degree, during aging of undisturbed samples of a Norwegian clay. The quasi-procussional consolidation effect reported by Leonards and Ramiah (1960) provides further evidence of the low-strain structure-stiffening effect of increasing time in secondary compression. It appears that these important stiffening effects may have an association with the I- to D-component transfer occurring during secondary compression.

9.3 EFFECTS DURING ANISOTROPIC CONSOLIDATION

Schirmer and Hall (1961a) reported on a comprehensive series of constant-q IDS-tests on remolded specimens of kaolinite and Boston blue clay after one day anisotropically consolidated to various K' levels. L. Bjerrum (1975) took special note of these experiments. He had a special interest in the clearly demonstrated time transfer from the I-component to the D-component during the secondary compression phase of the K' consolidation. For example, consider Figs 23a23b and 23b, taken from the 1961 reference. In 1961 we still used the old test and component notation. These figures show, as explained in the reference, that after some secondary time following different levels of K' consolidation the deviator stress transferred almost entirely to the D-component. Then, during the subsequent increase in strain and strain rate imposed by the IDS-tests, the I-component quickly regressed itself in all K' < 1 tests to the same magnitude as if K' = 1. In contrast, the D-component progressively increased its slow-strain mobilization as K' decreased, and D remained greater at all IDS strain tests than if tested at K' = 1.

It appears that the very low rates of strain during secondary compression permitted the soil to change structure so as to increase its shear sensitivity to effective stress change. This increased sensitivity gradually diminished with increasing IDS-test strain.

9.4 COMPONENT TRANSFER DURING CREEP TESTS

R. G. Bea (1960, 1965) performed a particularly interesting series of creep experiments on remolded, duplicate specimens of kaolinite and Boston blue clay. He subjected specimens to various levels of progressive loading and allowed them to creep under these constant loads. After various load and time intervals and, therefore accumulated strain, he subjected these specimens to constant-q IDS-tests. By comparing the I- and D-component distributions after creep with those distributions obtained from specimens compressed to the same strain, but without creep loading, he could evaluate the effect of creep on the component mobilization and distribution. Fig. 24, taken from Bea's thesis, presents the results he obtained from one series of tests on kaolinite. He obtained similar results from Boston blue clay, see Bea (1963).

Fig. 24 shows the component distributions found after three creep tests, compared with the component distributions from two duplicate tests on kaolinite specimens IDS-tested without prior creep. The IDS-tests started at the strains indicated by the light, dashed vertical lines. Also shown are the first I and D component points determined after a small additional strain during the subsequent IDS-test, as well as the subsequent variation of these components with strain during the IDS-test. Clearly, as emphasized by the shading, the previous creep produced an abrupt increase in the D-component compared to that which otherwise would have been mobilized at that strain without creep. After creep, this kaolinite and also the Boston blue clay, had greatly increased initial tangent moduli and a structure able to resist considerably greater compressive load than the prior creep had not occurred. However, these increases in ability to mobilize shear resistance dissipated after about 1 to 2% increase in axial strain during the IDS-test. Note that the above behavior seems quite consistent with that observed to result from the K' consolidation effects discussed in 9.3, and the secondary compression effects discussed in 9.2. All these behaviors seem to result from the same, or at least similar, soil structure phenomena.

R. G. Bea (1960) also showed that prior creep could produce an increase in the D-component to a level greater than the soil could mobilize without an interval of creep. Fig. 25, also taken from Bea's thesis, illustrates this phenomenon. This Boston blue clay actually mobilized apparent effective stress friction angles, tan $\phi'$, of 30 degrees while it could only mobilize about 24 degrees in an IDS-test strain much more rapidly to the same strain. Extrapolating to the strain condition at the beginning of the IDS-test produces apparent friction angles as great as 33 to 34 degrees.

The question of the component distribution at the end of creep, corresponding to the 0% strain during the subsequent IDS-test, of course has great interest. One can obtain estimates of such components by extrapolating the data obtained during subsequent strain by the IDS-test to the condition of 0 strain. A great deal of experience has shown that the D-component usually varies very gradually at low IDS-test strain while the I-component often increases very abruptly. Typically, the I-component has a very high modulus compared to the D-component. As also explained and illustrated by Schirmer and Hall.
Hall (1961a), it therefore seemed logical to those authors to extrapolate the D-component to 0 strain and then compute the I-component that existed to provide the proper total of I- and D-components. After doing this, Nebelmann and Hall found a dramatic reduction in the I-component mobilized at the 0 IDS-test strain condition. The I-component dropped to near 0 in some cases and to -0.08 kg/cm² in one case.

The above negative value of I seemed impossible at the time and was attributed to experimental error. However, Beas obtained negative values that cannot be ignored when using similar, seemingly very logical, extrapolation procedures for the components at 0 strain. He reported negative values of 0-strain I-component in over half of his tests. For the remolded kainite he reported negative values ranging from -0.12 to -0.20 kg/cm², and for remolded Boston blue clay (from -0.07 to -0.52 kg/cm²). These 0-strain I-components did not correlate with the creep strain rate just prior to the IDS-test. However, they did become increasingly negative with the incidental over-consolidation that occurred during creep in relation to the highest a₀ used in the subsequent IDS-test (OCR = 1.6 q above ±2).

9.5 COMMENTS ON TIME-TRANSFER EFFECTS

All of the above time effects indicate that during a period of constant loading, or very slowly increasing loading, both involving very small the rate of strain, the structure of a soil readjusts in such a way as to: 1) increase its reserve strength for subsequent small increases in strain, 2) this increase in strain is accompanied by an increase in creep in shear modulus, and 3) the increase in shear resistance result almost entirely, and perhaps entirely, from an increase in the D-component.

Fig. 26 presents a quantitative illustration of this creep behavior with pieces of CSMEs obtained from some of Beas's tests on remolded Boston blue clay, both with (1.2) and without (3.9) prior creep. Note the negative I extrapolation at the end of creep (the entire CSME still passed through I₀ = 0.14). Also, from comparison with the IDS-test results without creep one can see that this clay increased in shear mobilization capability, and sensitivity a₀, d in %, at the creep to εₙ₁ = 1.6 % not occurred. It appears that the soil structure yields to those elements within the structure that have increased rigidity and also an increased sensitivity to a subsequent change in effective stress.

During strain-rest the passage of time allows the I-component to reduce dramatically and more easily become negative. The previously established "fact" that only a part of the I-component can consist of true bond shear resistance (or true cohesion) clearly demonstrates that the yielding of the I-component cannot be explained simply by a time-reduction in some type of true cohesion behavior. Although Bjerrum's recent work (1973), as well as his work with T. C. Kenney (1968), showed a developing understanding of time-transfer from the less rigid to the more rigid shear resistance framework within a soil, Bjerrum attributed this to a relaxation of the true cohesion within the soil. The work presented herein seems to show clearly that any such true cohesion transfer can only account directly for a small portion of the I- to D-component transfer. If such small cohesion transfer does exist, it may, or it may not, indirectly trigger the behavior producing the major portion of this transfer.

The component time-transfer effects discussed thus far occurred in kainite and Boston blue clay first remolded by machine extention. Do they also occur in undisturbed clays? The work by Bjerrum and Wu (1960) and Bjerrum and Lo (1963) suggests that they do in at least some undisturbed Norwegian clays. As another example, Beas also tested undisturbed specimens from the same block sample of BBC that supplied the source for the extruded specimens.

Table 16 presents the results of Beas's constant-α IDS-tests on undisturbed BBC after undrained creep. It includes a comparison between the shear components at the end of creep and the same strain in a similar IDS-test, but without prior creep.

Table 16. The effect of creep on the I and D components in undisturbed BBC (stresses in kg/cm²).

<table>
<thead>
<tr>
<th>Test</th>
<th>Creep</th>
<th>Days</th>
<th>C</th>
<th>D</th>
<th>ε₁</th>
<th>εₙ₁</th>
<th>a₀</th>
<th>d₀</th>
<th>I₀</th>
<th>Iₙ₁</th>
<th>A₀</th>
<th>Aₙ₁</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-13</td>
<td>2.02</td>
<td>2.00</td>
<td>11</td>
<td>15.0</td>
<td>0.30</td>
<td>0.31</td>
<td>11.0</td>
<td>0.51</td>
<td>1.10</td>
<td>1.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>B-14</td>
<td>2.00</td>
<td>2.00</td>
<td>15.0</td>
<td>15.1</td>
<td>0.27</td>
<td>0.27</td>
<td>9.5</td>
<td>0.54</td>
<td>1.00</td>
<td>1.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Table 27. Example illustrating constant I constant because (α - d) constant.

Fig. 27. Example illustrating constant I constant because (α - d) constant.

10. One explanation for the observed shear behavior

Over the span of this research the writer has devoted much thought to finding one or more hypotheses that might explain all the observed behavior in a logical, hopefully simple, manner consistent with the observations and with the particular nature of soil. To date he can offer only the following for others to consider and improve upon.

10.1 THE CONSTANT I-COMPONENT DUE TO CONSTANT β

This remarkable constant-I behavior, as detailed in Section 4.1, may perhaps now be explainable - at least in a mathematical sense. Section 4.1 demonstrated that for a given soil, and a given effective stress, the I-component remained essentially the same no matter what was done to change the soil structure. Fig. 27 illustrates two CSMEs for the same soil but at different structural conditions. In both cases I₀ represents only a minor fraction of the total I-component. In both cases, the IDS-piece-of-CSME, when extruded, produced the same I-component. Why? Start with the following equation (9) for I:

I = v - d₀

Substituting v from Equation (6) into (9) produces the following equation (10):

I = v - d₀ = (α - d₀) = a₀ - d₀

After recognizing that (I - I₀) = v = α, Equation (10) rearranges to the following:

(α - d₀) = (a₀ - d₀)

As was shown in Figs 11 and 12 herein, as well as some of the Section 7.4 example cases from undisturbed clays, the value of β at I₀ seems remarkably independent of the soil structure. Therefore, any two structural conditions 1 and 2 of the same soil in Fig. 27 should have at least approximately the same value of β. A constant β despite a changing α produces a constant curvature and hence a constant difference between the α and d₀ slops at any given α₀. In Fig. 27 the writer of the procedure plots of CSMEs around a₀ = 2.0 using the same values of I₀ and β, but different α values to express the different soil structure. As must be true at any a₀, the two values of (a₀ - d₀) at α₀ = 2.0 are the same.

Equation (10) shows a constant β and (α - d₀) difference must result in the same I-component at a given a₀, as also shown in Fig. 27. This bold provided I₀ does not change significantly with changes in structure or if I₀ is so small any significant change in I₀ has a negligible affect on the apparent constancy of the total I-component.

10.2 THE CONSTANT β AND GRAN SIZE

This research has shown that β represents an approximate constant which expresses the curvature of the CSME. All or after the strain of I₀, β does not vary with structure for a given soil during the forced rate-strain in an IDS-test, nor even very much in a particular class of soils. However, as noted in Section 5.1, β does appear to vary with grain size. After looking into this more carefully and plotting in Fig. 28 the variation of I₀ for various remolded soils as a function of the log of the D₉₅ grain size, a remarkable fact emerges. Not only does I₀ vary linearly with the level of cohesion, as explained previously in Section 5 in great detail, but I₀ also varies inversely with log D₉₅. However, note the one exception to the general trend - the glass spheres have a much lower than their air voids, and the glass spheres have proven consistently slightly negative (see Tables 10, 14).

From Fig. 28 it appears clear that the value of β somehow depends on grain size. But, the zero or negative value of β for near-spherical glass particles shows that size alone is probably not the real critical parameter. The writer suggests that instead the curvature of the particle at contact controls β. It seems reasonable to postulate that the smaller the particles in contact, and therefore the more irregular and plate-shaped their geometries, then the smaller the average radial of curvature at the contact points. Because of the higher void ratios associated with finer particles, and their progressively less spherical geometries, a given effective stress produces progressively greater particle contact pressures as size decreases. This produces greater opportunity for local crushing, particle breaking, and any other nonlinear behavior, and thus progressively reduces the stress sensitivity (slope) of the CSME with increasing a₀. Such a theory would show why the glass spheres do not fit with the other soils in Fig. 28. They have an almost spherical shape and therefore very high radial of contact relative to their size when compared to the rough, pitted and peaked surface of natural grains. Such a theory would also explain the constancy of β - the same or similar soil has I₀ for one or similar particles in contact, with statistically similar
The writer has also observed on numerous occasions that the formation of a distinct failure plane in a specimen during an IDS-test would produce a relatively abrupt drop in \( d_i \) and a corresponding increase in \( d_j \). The last data point in Fig. 1 shows such a drop in the \( d_i \) component resulting from a failure plane developing just before the point. A failure plane represents a localized zone of very high shear strain. Very large shear strains dispersed to a particle size of the order of \( 10^{-6} \) m. However, if the particles must be taken up by those particles least likely to reorient. These are also the particles with the greatest interference effects. The immediate subsequent shear resistance of the soil increases in rate of strain now resides primarily in those particles with greater interference effects and therefore greatly increased. Because of the large strain of the \( L \) and \( D \) components remains the same, a greater \( d_j \) must cause a drop in the extrapolated S-component (see CSME 1 in Fig. 26). The subsequent forced increase in \( d_j \) does not significantly essential, elastic mobilization of those rigidly rigid particle interference zones of shear resistance. As soon as interference movements again begin and \( \beta \) again controls CSME curvature, \( X \) increases to its curvature-controlled \( I_{\text{soil}} \) (as at CSME 2 in Fig. 26). As the soil continues to strain it again mobilizes its ordinary particle-interference structure and not just the above rigid zones it is temporarily "loosening up." With this rigid-deduction to ordinary mobilization transfer the value of \( d_j \) and therefore the D-component, rapidly diminishes during the very low increase of soil strain it takes to accomplish the transfer. \( d_j \) then gradually increases again provided the soil structure has the capability of increasing its net particle interference effects with continued strain.

The above explanation presents a soil carrying a deviator stress under creep-rate-of-strain conditions would have a high \( d_j \) mobilization, and therefore a low extrapolated \( I_{\text{soil}} \)-component. With the IDS-test forced increase in rate of strain, the 1-component would rapidly rise and \( d_j \) (and therefore the D-component) rapidly decrease with strain. This would be exactly the effect, as related in Fig. 24, 25, and 26. This behavior would not ordinarily be observed in an IDS-test beginning after isotropic consolidation because the mobilized D-component would begin at zero.

10.7 \( I_{\text{soil}} \) DRAINED VS. UNDRAINED

If \( I_{\text{soil}} \) represents the shear resistance of a soil due to its internal bonds between particles, then its mobilization must also be in strain and pass strain path dependent. As detailed in 5.2, \( I_{\text{soil}} \) is the same, a greater \( d_j \) mobilized as functions of strain in at least some soils (each of 3 \( I_{\text{soil}} \) investigated). \( I_{\text{soil}} \) might well be also anisotropic and stress and strain path dependent.

The data from Ho in Table 14 seems to show that both \( I_{\text{soil}} \) and \( c_1 \) values obtained from constant-volume (undrained) IDS-tests have a greater magnitude than when determined from free-draining tests. It has been seen that forcing a structure to strain at constant volume forces that structure to mobilize more of its internal bond shear resistance. Perhaps more interference movements per unit shear strain must occur at constant volume compared to when the soil structure can increase or decrease volume if free to do so.

Note also in Table 14 that for the constant-volume test \( c_2 \) exceeds \( I_{\text{soil}} \) substantially. However, if \( c_1 \) and \( c_2 \) were theoretically equivalent after a proper stress-dilatancy correction to \( I_{\text{soil}} \) and remembering that the constant-volume q-correlation = 1.0, then these two parameters should be identical. That they are not perhaps indicates that the Rowe dilatancy correction does not completely account for all soil-strain effects.

10.8 IMPLICATIONS FROM THE SUCCESS OF CURVE-HOPPING

The success of the one-specimen IDS-test curve-hopping technique demonstrated by Schnurmann (1962b) presents a puzzle which R. Kondner (1964) and Schnurmann (1964b) discussed. One might explain the success of curve-hopping by assuming the soil elastic over the effective stress range of the curve-hopping (usually approximately 20% of initial effective stress). However, we know that soils do not behave elastically and especially not at strains beyond the initial high-modulus, low-strain portion of any type of soil shear test. Kondner rejected such an elastic explanation and the writer agrees.

What then can explain the success of curve-hopping? The only explanation the writer can imagine involves thinking of a soil mass as composed of a great many zones, or "cells" within the mass wherein most of its shear resistance resides. Such resistance-cells form, are destroyed, and thereby subsequently reform with the continuing strain. All such cells depend primarily on an effective stress prestress for their shear resistance capability. The sudden decrease of ambient effective stress during a curve-drop from the higher to the lower effective stress level reduces this effective stress prestress level in many of these resistance cells to the point that they cannot sustain the same level of shear resistance and they fail. The many such failures within the IDS-test specimen results in the sudden drop in that soil's ability to mobilize shear resistance at the strain of the effective stress drop. Fig. 29, spots 1-2 illustrate this drop. Then the IDS-test strain continues along \( 2-3 \) and new cells form and the soil resistance-cell system recovers from the failures effects of the previous abrupt effective stress reduction. With repackaging of the soil the effective stress level the soil can again mobilize its initial magnitude of shear resistance along path 3-4.

The Fig. 29 path 1-2-3-4 ends at a point seemingly too incipient, or not real. However, the path has been directly from 1 to 4 without the effective stress drop. The writer thinks the probable explanation for this superficially elastic behavior is that the number of cells progressively forming, collapsing and reforming with the continuing strain between points 1 and 4 is so large that the temporary, abrupt collapse of the cells during step 1-2 does not significantly affect the shear resistance cell system at the strain of 4. The writer adds: The overall mobilized shear resistance behavior of a soil structure represents the statistical summation of a very large number of formation-collapse events taking place within the structure. Any dislocation in the smooth flow of these events by a sudden effective stress change is only strain-temporary and is subsequently healed by the continuing many events that take place with the continuing strain.

10.9 POSSIBLE NON-BONDING EXPLANATION OF "AGING" AND "QUASI-PRECONSOLIDATION"

When a soil can readjust its structure under drained conditions, such as during long periods of time at constant load, the more easily dispersed groups or zones of particle yield to the greater Shear stress on the soil. When the shear stress on the soil becomes greater and stiffer as a result of this yield-transfer of mobilized shear. But, it also becomes relatively more effective stress sensitive (\( d_j \) increases). An subsequent full mobilization of the shear capability of the new structure produces a yielding of the stiffer particle zones. After some interval of new strain to produce and exceed this yielding, with its accompanying many resistance-cell events, the soil's mobilized shear resistance behavior returns to that which would have occurred had the drained dispersion interval not happened (as in Figs. 9 x = 3, \( I_{\text{soil}} \)).

Such behavior would explain why soil "aging" and "quasi-consolidation" effects occur and possibly help evaluate when they would and would not occur. Clays are ordinarily the soils with the most complicated structure and the most subject to particle dispersion and orientation effects. Such behavior is therefore potentially most important in clays. However, in principle it can also apply in such soils where the work here it would seem that it cannot occur when a structure has already been dispersed so that all particles have a minimum potential energy orientation. This would not easily such a structure (as on shear planes at the residual state).
no greater particle interference zones to temporarily "lean on".

The above has not introduced the possibility of additional interparticle bonds forming with time. Such would of course contribute to the "aging" and "quasi-preconsolidation" behavior. Table 11 suggests that dispersion may increase such bonds. Cementing will also. However, the writer does not consider them essential to the phenomena.

10.10 COMMENT ON THE USE OF REMOLED K A O L I N I T E VS. U N D I S T U R B E D C L A Y

Because of the research need to compare tests while changing as few variables as possible, the writer made extensive use of tests from duplicate specimens remolded by machine extrusion – especially kaolinite. The reader might well wonder if the results obtained and the interpretations offered would apply to other clays – especially undisturbed clays?

As described and documented herein and in the references cited, several other extruded natural clays behaved the same as kaolinite with respect to IDS-tests, CSME curvature, component time-transfer, etc. After satisfying himself on this point the writer then chose to use kaolinite more extensively, but not exclusively. It offered the advantages of availability in large quantities, ease of preparation and handling, a low secondary compression index, and the convenience of its relative high permeability reducing the time required for pore pressure dissipation or equilibrium and making it easier to prepare saturated specimens. The interested reader can read Schmertmann (1963d) for more information about this kaolinite.

As also documented herein and elsewhere, the curve-fitting method and the IDS-test separation of components has proved possible in many undisturbed soils. Section 7.4 showed that the I vs. q* method of extrapolation for I* seems to work as well in undisturbed as in remolded soils. Section 9.5 ended with data and discussion to support the idea that the component time-transfer behavior also occurs in undisturbed clays. All the available evidence by the writer knows of supports the position that the various shear behavior phenomena investigated and described herein will occur in situ in undisturbed clays. Of course one should expect great variations in degree.

11. Conclusions

A. Relating to the constant structure Mohr Envelope (CSME)

1. To achieve a more fundamental understanding of soil shear resistance behavior, one can investigate this behavior when keeping soil structure constant.

2. The concept and experimental use of the CSME provides a useful vehicle to explain constant structure shear behavior.

3. This research has produced sufficient data to derive the following:

\[ v = I_s - n_0 - \beta \cdot p' \cdot q' \cdot I_s \]

expressing the shape of the CSME applicable to the soils and stress range investigated.

4. The \( \beta \) factor in the CSME equation seems to depend primarily on grain size and shape and probably reflects the nonlinear behavior occurring at particle contact points. The \( \beta \) factor seems to depend on soil structure.

5. Relating to cohesion and the I-component

6. The unreported constancy of the I-component ("cohesion" intercept) with changes in structure results from the constancy of \( \beta \) and does not describe an irreversible, fundamental shear component. The I-component varies linearly with effective stress along the CSME because of the nature of CSME curvature.

7. One can determine a fundamental cohesion, or bond shear resistance, I_s of any soil as a function of strain by new experimental methods described herein. The available alternate methods seem to check these I_s.

8. Typical remolded cohesive soils and natural, young, uncemented clays have maximum I_s of about 0.05 to 0.15 kfx. Cementation or partial saturation can greatly increase I_s. Drained structure dispersion probably increases the non-cementation part of I_s.

9. Because of CSME curvature, I_s often represents only a minor part of the total I-component. Even sands with I_s = 0 indicate a positive value for their I-component.

10. The strain-rate of I-component mobilization usually occurs much more rapidly than that of the D-component. But, the writer believes it premature to conclude from this alone that "cohesion" mobilizes quickly, while "friction" only gradually, with strain. The shape of the CSME can explain this behavior.

11. I_s appears to mobilize at very low strain in cohesive soils compared to the strain required for overall soil failure. I_s may, or may not, diminish greatly at overall failure.

12. Strain under undrained conditions appears to force a soil to mobilize more of its bond shear resistance than under drained conditions.

B. Relating to the D-component and dispersion

13. Deliberate chemical dispersion of structure, or over-consolidation, or increasing strain in a shear test, or increasing time in secondary compression, or increasing creep time under deviator loading, or decreasing a forced rate-of-strain, or using less viscous pore fluids all have the common effect of increasing a cohesive soil's subsequent shear resistance sensitivity to a change in effective stress.

14. The above 13, common effects suggest a common cause. The writer suggests that all the above manipulations produce an increased dispersion of soil structure, which leads to a transfer of a soil's shear resistance to those particles with greater resistance to dispersion.

15. After a period of time at constant loading, a soil becomes stronger and stiffer as a result of the continuing dispersion. The dispersion particle movements force the soil to "lean on" those groups of particles with greater resistance to dispersive recrystallization. These groups have greater strength, greater stiffness, and -- being brought nearer to failure -- a greater sensitivity to changes in effective stress.

D. Relating to "aging" behavior

16. The above dispersion explanation for the increase in a soil's stiffness and stiffer with time may help explain soil "aging" and "quasi-preconsolidation" behavior.

17. One need not exclude the formation of new interparticle bonds to explain aging and quasi-preconsolidation behavior. Possibly, for the same reasons, one need not resort to the making and breaking of bonds to explain the undrained shear sensitivity of some soils.

18. Because of the more complicated soil structure possibilities in clays, they have a greater susceptibility to grain crushing/breaking and to dispersion and particle reorientation-interference effects. It seems reasonable to expect that the various shear resistance phenomena described herein would occur most dramatically in cohesive soils. However, in principle, one should also find such behavior as "aging" and "quasi-preconsolidation" in silts and sands.

E. Some general thoughts from this work

19. The overall behavior of a soil mass subject to strain at constant or increasing load represents the integration of effects from a huge number of resistance cell-formation and resistance cell-failure events that occur with changes in effective stress, strain and time.

20. The writer believes that d_e (= tan \( \theta' \)) represents a measure of a soil's mobilization of particle interference effects to resist applied shear. It appears that under constant-stress conditions a soil has more freedom to readjust its structure to more selectively, or at least more efficiently, mobilize its particle-interference strength against the next increment of applied shear than when subjected to a forced continuing strain, as in strain-controlled tests.

21. The maximum value of d_e probably cannot be determined from a strain-controlled test. The value of d_e will vary with the strain-time path used. Faster rates of forced strain will cause d_e to decrease. Longer creep times will cause d_e to increase.

22. The experiments described herein employed mostly remolded soils. However, all the present data indicate that the previous conclusions will also apply to soils in-situ.

List of notations

- \( c_v \) = early notation for \( I_s \)
- \( c_i \) = P. Rowe's interparticle cohesion parameter, determined after using a stress-strain-flux correction. \( c_i \) = an average value.
- CSE-test = experimental procedure for determining a constant structure envelope from a single specimen at one strain.
- CFS-test = early notation for the IDS-test.
- CSME = abbreviation for Constant Structure Mohr Envelope.
- \( D_s \) = alternate expression for the D-component, \( D_s = \Delta v/\Delta u' \) (see Fig. 4).
- \( D \) = component of mobilized shear resistance seemingly linearly dependent on effective stress, \( I_s' \), at constant structure (see Fig. 4).
- \( D' \) = notation for \( D \) to emphasize that component is a function of stain.
- \( D_s' \) = that grain size in a soil with 50% finer sized, by weight.
- \( e \) = void ratio.
- \( I \) = component of mobilized shear resistance seemingly independent of effective stress, \( \phi' \), at constant structure. Note 1 + D = e. See Fig. 4.
- \( I_s \) = notation for I to emphasize that component is a function of strain.
- \( I_{s,v} \) = the maximum value of \( I_s \) determined from a graph of \( I_s \) vs. strain, the bond strength.
- \( I_{v,s} \) = the value of \( I_s \) at a drained failure condition.
- IDS-test = experimental procedure for determining the I and D components of \( I \) as functions of strain.
- \( I_{v,s} \) = the maximum value of \( I_s \) determined from a graph of \( I_s \) vs. strain.
- \( I_{s,v} \) = the mobilized shear resistance when \( \phi' = 0 \), the bond shear resistance (see Fig. 15). \( I_{s,v} \) = an average value.
- \( I_{s,v} - I_{s,0} \) = the value of \( I_s \) determined from an extrapolation of \( I_s \) to \( \phi' = 0 \).
- \( I_{s,v} \) = the value of \( I_s \) determined from an extrapolation of \( I_s \) to an undrained failure condition to \( \phi' = 0 \).
- \( I' \) = the ratio of \( \phi' q_{\phi'} \) at a point.