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CONSTANT-RATE-OF-STRAIN AND CONTROLLED-GRADIENT CONSOLIDATION TESTING

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ABSTRACT: Controlled-gradient (CG), constant-rate-of-strain (CRS), and conventional incremental-loading (STD) consolidation testing are compared and evaluated. Undisturbed samples of three soils common to Kentucky were used in the testing program. Results of 15 CG, 14 CRS, and 32 STD consolidation tests are evaluated. Feasibility of the new test methods for routine testing is briefly discussed and recommendations are made for refinements in testing procedures.
KEY WORDS: consolidation, constant-rate-of-strain, controlled-gradient, laboratory tests, pore pressure, soil mechanics, strain rate

Introduction

The foundation design of a structure requires a reliable estimate of the magnitude and rate of settlement. Buildings and bridges must be designed to withstand such estimated differential and total settlements. Highway embankments must be designed to minimize settlements that produce uneven road surfaces and pavement distress. Information needed to estimate magnitude and rate of settlement is obtained by laboratory consolidation tests of soil samples taken from the proposed site. For the past 40 years, specimens from such samples have been subjected to standard incremental-loading tests where increasingly large increments of load, and the resulting measured deformations, have been used to estimate settlement. This laboratory test requires approximately two weeks and yields information which requires much interpretation. These shortcomings eventually led to the development of controlled-gradient (CG) and constant-rate-of-strain (CRS) consolidation tests.

Soil is not a homogeneous material having easily defined engineering properties, and different methods of testing may yield different values of soil properties. The objectives of this research are (1) to compare test data obtained from CG and CRS tests with those obtained from conventional incremental-loading (STD) tests, and (2) to determine the feasibility of using CG and(or) CRS testing in routine investigations.

The CRS test was first described in 1959 by Hamilton and Crawford *(1)* as a rapid means of determining the preconsolidation pressure, P_c. In the CRS test, imposed boundary conditions are similar to those in the STD test, but with one-way drainage. The specimen is confined laterally by the same type of ring used in the conventional test apparatus (oedometer), and drainage of pore water is permitted at the top only. In the original CRS test, however, the specimen is loaded at a constant rate of strain instead of incrementally. The strain rate is chosen such that "significant" pore pressure does not develop in the specimen; thus, effective stress is assumed equal to the applied stress. Continuous stress-strain points provided a well-defined stress-strain curve; this is not possible in the STD test.

Hamilton and Crawford pointed out that, in the STD test, gas bubbles in test specimens accounted for most of the initial compression observed at loads below P_c. Lowe, et al., *(2)* agreed with their findings and proposed a solution in 1964 involving the use of back pressure to saturate specimens. Results of STD tests with and without back pressure indicated that back pressure had little effect on the compressibility of the soil but does affect the rate of consolidation.

Theory for the CRS test was developed by Smith and Wahls *(3)* and, independently, by Wissa, et al. *(4)*. Both works extend Terzaghi's theory to the boundary conditions of the CRS test and yield the following expressions for average effective stress, σ_v', and coefficient of consolidation, C_v:

$$C_v = H^2 \, \Delta\sigma_v \, / \, 2 \, \Delta t \, u_b \qquad 1$$

and

$$\sigma_v' = \sigma_v - 2 \, u_b \, / \, 3, \qquad 2$$

where H = length of drainage path in the specimen,

σ_v = total vertical stress,

t = time, and

u_b = excess pore pressure measured at the base of the specimen.

Wissa, et al., arrived at the same solutions for C_v and σ_v' by assuming that strain was a parabolic function of vertical distance, z, from the drainage face. The theory was also extended to the case where the coefficient of volume change, m_v, was not assumed constant but was assumed to vary linearly with the logarithm of σ_v'. For u_b/σ_v less than about five percent, the linear and nonlinear theories yield approximately the same result.

The CG test proposed by Lowe, et al., *(5)* was similar to the CRS test except the specimen was loaded at a rate such that the excess pore pressure generated at the base (undrained end) of the specimen remained constant. Thus a constant hydraulic gradient was established across the consolidating specimen. An assumption of a parabolic distribution of pore pressure across the specimen yielded equations for C_v and σ_v' identical to those derived for the CRS tests.

Since the equations derived for the reduction of CG and CRS test data are the same, both tests may be reduced using the same procedure. Readings of time, deflection, load, and pore pressure are taken at various intervals during the test and are used to calculate strain, ϵ, σ_v', and C_v. Plots are then produced and preconsolidation pressure, P_c, compression ratio, CR, and swell ratio, SR, may be determined using graphical constructions proposed by Casagrande *(6)* and Schmertmann *(7)* for the STD test. Plots of σ_v versus time from the CG test and u_b versus σ_v' from the CRS test are also helpful in the determination of P_c.

Equipment

Controlled-Gradient Equipment

Equipment capable of performing a CG consolidation test was obtained from Soils Mechanics Equipment of Spring Valley, New York. The equipment was intended to perform CG, CRS, and STD consolidation tests and was hence termed the "Universal" consolidometer. However, only CG tests were performed with this equipment because STD tests were more easily performed using conventional equipment and because the response of the pore-pressure measuring system was too slow for the CRS tests. Slow response of the pore-pressure measuring system was attributed to the fact that no provision was made for de-airing the pore-water lines in this apparatus; air trapped in the lines created a slow response to pressure change, or pore pressure lag. In the CG test, pore pressure was maintained constant (or nearly so); consequently, pore-pressure lag was not a significant problem.

A simplified schematic of the control portion of the equipment is shown in Figure 1. The basic function of the equipment is to load the specimen at a rate that will maintain a constant pore pressure, u_b, at the base of the specimen. Provision is also made for saturating each specimen by back pressure.

The pore-pressure duplicator converts the pore pressure plus back pressure at the base of the specimen to an equal air pressure. The difference between this pressure and the back pressure is the excess pressure generated by loading the specimen. Excess pore pressure is displayed on the differential pressure gage. The air pressure is also input to the load pacer, which regulates the load applied to the specimen such that the pore pressure plus the back pressure remains at a constant, preset value. A schematic of the Universal consolidometer is shown in Figure 2.

Constant-Rate-of-Strain Equipment

Modified triaxial equipment was used to perform CRS tests. A triaxial chamber was fitted with an oedometer ring, and a triaxial loading press was used to deform the specimen at a constant rate. A schematic of the CRS consolidometer is shown in Figure 3. The specimen may be back-pressured, top and bottom, to insure saturation. The load, applied to the specimen by the loading ram, was measured by a strain-gage-type load cell and signal-conditioning equipment. Pore pressure was measured at the undrained end (bottom) of the specimen by means of a strain-gage-type pressure transducer mounted directly in the base of the apparatus. Height change was measured by a linear variable displacement transducer which recorded the movement of the chamber as it was moved upward on the loading ram by the loading press.

Complete saturation of the pore-pressure cavity and back-pressure line was achieved by filling the chamber with de-aired water and applying a vaccum to the water, thus displacing the air in the cavities with de-aired water. This eliminated the problem of pore-pressure lag.

Conventional Equipment

STD consolidation tests were performed using Karol-Warner and Anteus pneumatic loading equipment. The Anteus pneumatic loading chamber had provision for back-pressuring specimens to insure saturation. Load was imposed on the specimens by applying a constant, regulated air pressure to the pneumatic-hydraulic loading assembly and measuring the change of height of the specimen with a dial indicator having a resolution of 0.0001 inches (0.0025 mm). The back-pressure system of the Anteus consolidometer was similar in all respects to that shown in the CRS schematic (Figure 3).

Site and Soil Descriptions

Much of the research with CG and CRS consolidation testing has been conducted using remolded soil samples and soils which are not commonly found in Kentucky. In an effort to compare and evaluate CG, CRS, and STD consolidation testing techniques for typical Kentucky soils, undisturbed (2.5-foot (0.8-m) long Shelby tube) soil samples were taken from three physiographic regions in Kentucky. Soil samples from one location in each of three regions within the state do not represent all "typical" Kentucky soils. In fact, considerable soil variability exists not only across the state but also within a particular physiographic region. Rather, the intent was to obtain undisturbed samples which would provide suitable specimens for the comparison of the various tests. With this in mind, three soils which exhibited a wide range of engineering properties were chosen for testing. Samples were obtained from the Western Coal Field, Mississippian Plateaus, and Bluegrass Regions; Site Numbers One, Two, and Three, respectively. Index properties of the soils are summarized in Table 1.

Samples at Site Number One were taken from soils of the Green River Valley, a significant feature of the Western Coal Field Region. This wide valley was formed in weak shales of the area and has been filled with alluvial material to depths of 175 feet (53 m).

The Mississippian Plateaus Region is characterized by a deep residual soil profile weathered from cherty Mississippian limestones. The level-to-rolling terrain and lack of surface drainage contribute to the weathering process. Many soils are red in color and contain large amounts of nontronite, an iron-rich montmorillonitic clay.

The Bluegrass Region soils are residual, weathered from the Lexington and Cynthiana Limestones to produce a brown, phosphate-rich soil. The soils are usually well drained internally, due to joints and cracks in the limestone, and have a fragmentary structure.

Test Procedures

Undisturbed, 2 1/2-foot (0.76-m) long, 2 7/8-inch (73-mm) diameter, thin-walled tube (Shelby tube) samples were taken at 5-foot (1.5-m) intervals at Site Numbers One and Two and continuously at Site Number Three. The disturbed material in the ends of each tube was removed, and samples from the tubes were cut into 6-inch (152-mm) lengths, waxed, and labeled according to site number, borehole number, and sample number.

Each test specimen was trimmed to 2.5 inches (64 mm) diameter by 1.0 inch (25 mm) high using a stainless steel trimming shoe and then transferred to a Teflon-lined consolidation ring in the apparatus. A small seating load of approximately 0.1 tsf (1 kPa) was applied to the specimen. Where back pressure was used, the chamber was filled with distilled water and a back pressure was applied to the top and bottom of the specimen. A back pressure of 10 psi (69 kPa) was used in all CG and CRS tests and in some STD tests. The value of 10 psi (69 kPa) for back pressure was chosen since it was greater than the maximum pore pressure which existed in any sample in situ, but was small enough not to affect the maximum load capability of the equipment. Back pressure was monitored for at least 12 hours before testing. In all cases, primary consolidation under the seating load was completed before beginning the loading phase.

Controlled-Gradient Loading

Following the back-pressure saturation phase, the drainage line to the bottom of the specimen was closed, creating one-way drainage at the top of the specimen. Pore-pressure buildup was measured at the bottom of the specimen. Although the loading system in the CG apparatus automatically adjusted the load to maintain a constant pore pressure at the bottom of the specimen, manual adjustment was necessary until the desired excess pore pressure (2-3 psi (14-21 kPa)) was attained, to prevent overreaction by the automatic loading system. Once the desired pore pressure was attained, automatic control was restored and the loading continued to a preset load (usually 32 tsf (3 MPa)). When the preset load was reached, the load was held constant at this value, pore pressure was allowed to dissipate, and compression under constant load with zero excess pore pressure (secondary compression) was recorded. The specimen was then unloaded manually in small increments to measure rebound data. Throughout the test, readings were taken of load, pore pressure, and height of the specimen at various time intervals.

Constant-Rate-of-Strain Loading

Drainage of pore water from the bottom of the specimen was prevented, prior to loading, as in the CG test. The specimen was loaded by compressing it at a constant, predetermined rate using a gear-driven loading frame. The loading machine was stopped at a preset load (32 tsf (3 MPa)), and excess pore pressure was allowed to dissipate at the final, constant deformation. The specimen was finally unloaded at the same rate at which it was loaded. Readings were taken of load, pore pressure, and height of the specimen in the same manner as in the CG test.

Incremental (Conventional) Loading

The STD tests were performed in accordance with ASTM D 2435-70, Standard Method of Test for One-Dimensional Consolidation Properties of Soils. Two-way drainage (top and bottom of the specimen) was allowed; load increments of 0.25, 0.50, 1.0, 2.0, 4.0, 8.0, 16.0, and 32.0 tsf (0.025, 0.05, 0.1, 0.2, 0.4, 0.8, 1.6, 3.2 MPa) were applied; and the specimens were rebounded to 1.0 tsf (0.1 MPa). STD tests in which back pressure was used were loaded only to 16 tsf (1.6 MPa). Each load increment was maintained constant on the specimen for 24 hours before the next increment was applied. Deformation at 100-percent primary consolidation and C_v were obtained from plots of deformation versus square root of time and deformation versus logarithm of time as recommended in the ASTM standard.

Test Results

Results of 22 STD tests without back pressure, 10 STD tests with back pressure, 15 CG tests, and 14 CRS tests are evaluated herein. To minimize effects of inhomogeneity, only test results from samples from the same tube were compared, except for Site Number Three where a uniform profile and close spacing of boreholes permitted comparison between sampling tubes. Five such comparisons were made for Site Number One, six for Site Number Two, and two for Site Number Three. Typical results of these comparisons were shown in the graphical form of ϵ versus σ_v' and C_v versus σ_v' in Figures 4 through 7. In addition, plots of σ_v versus time from the CG tests and u_b versus σ_v' from the CRS tests are also shown in these figures. These plots can be used to determine P_c.

Readings taken in each test are indicated by individual points in Figure 4 only. This serves to demonstrate that CG and CRS tests produce well-defined curves, since continous data may be obtained from these tests, while the number of data points in the STD test is limited to the number of load increments applied.

Analysis and Discussion

Comparisons of Stress-Strain Data

In general, the agreement among CG, CRS, and STD stress-strain curves at Site Number Two was very good. Some scatter was shown in the stress-strain curves for the different test methods at Site

Numbers One and Three. Factors related to test methods which could have caused some of the scatter will be discussed later; however, differences in properties among test specimens could well be a contributing factor. Even though test specimens taken from the same sampling tube may appear homogenous, small differences in structure may produce different consolidation characteristics. Since most curves do agree well, this effect apparently was not pronounced; however, it should not be entirely discounted.

A more quantitative method of comparing CG, CRS, and STD stress-strain data is to compare the numerical values obtained from the stress-strain curves for use in settlement analysis. The values of compression ratio (CR), swell ratio (SR), and preconsolidation pressure (P_c) from each test are summarized in Table 2. Numerical comparisons are also shown in graphical form in Figures 8 through 16. Horizontal bars on the plots indicate scatter in STD test data. Since only one CG and one CRS test were performed on specimens from each sampling tube (except in the case of Site Number Two, H-2, S-5, in which case two CRS tests were performed), it was not possible to determine scatter in CG or CRS test data.

CR obtained from CG test data is compared with CR from STD test data in Figure 8, and CR from CRS test data is compared with CR from STD test data in Figure 9. CR from CG test data agrees more closely with CR from the STD test than does CR from the CRS test. The CRS test gave slightly lower values of CR than did the STD test; however, the discrepancy is not significant.

The same type of comparisons are made in Figures 10 and 11 of the values of swell ratio (SR) obtained from the various tests. Again, the horizontal bars indicate scatter in the STD test data; the vertical bar shown in Figure 11 indicates scatter in SR from two CRS tests. Scatter in SR data is significant. The CG and CRS tests gave higher values of SR than did STD tests. The method of unloading the specimen can apparently affect SR values significantly. The CG test was unloaded in small increments; the STD test was unloaded to a pressure of 1.0 tsf (10 kPa) instantaneously; and the CRS test was unloaded at a constant strain rate. Considerable scatter also exists in SR values from STD tests at Site Numbers One and Three. The scatter in STD test results may be due to back-pressure effects and will be discussed later.

Finally, the same type of comparison is shown in Figures 12 and 13 for values of P_c obtained from the various tests using Casagrande's construction. Scatter in values of P_c obtained from STD test results is considerable and is not due to back-pressure effects. In most cases, the agreement is good, considering the scatter in P_c values from the STD tests. The largest discrepancies in values of P_c given by the different tests occurred in the highly overconsolidated soils of Site Number Three.

Comparisons of C_v Data

C_v is, by definition, a function of soil compressibility, m_v, and soil permeability, k. The theory used to derive the equations of consolidation assumes that m_v and k are constant and that drainage of pore water occurs only in the vertical direction. Deviation of actual conditions from those assumed render any estimate of C_v only approximate.

In general, C_v-σ_v' curves determined appear scattered but tend to show some convergence above the apparent value of P_c. The reason for the convergence of the curves above the apparent P_c deserves some consideration. In the CG and CRS tests, values of C_v calculated in the early portions of the tests using Equation 6 were very erratic and often very high. Due to the unreasonable nature of these values, they were subsequently omitted from the data shown in this report. Values of C_v calculated using Equation 6 for the early portions of the CG and CRS tests were unreasonable because the steady-state conditions, upon which Equation 6 is based, do not exist in the early portions of the test.

In the CG test, some time is required to establish the pore-pressure gradient. This involves manual adjustment of the load in the early phases of the test. As a result, the term $\Delta\sigma_v/\Delta t$ in Equation 6 is very small. Only when pore pressure increases and the automatic loading system takes control will the calculated values of C_v be realistic since the tendency for pore pressure to increase is accompanied by a decrease in the loading rate ($\Delta\sigma_v/\Delta t$).

In the CRS test, the same phenomenon occurs for the same reasons. Values of C_v are erratic until significant pore pressures are measured. This phenomenon illustrates the fact that the minimum strain rate in the CRS test is one that generates at least some measurable pore pressures. The actual magnitude of pore pressure generated, however, does not appear to affect the magnitude of C_v. This is shown in C_v results from CRS tests on samples from Site Number Two, H-2, S-5: CRS-15 reached a maximum pore pressure of 32 psi (220 kPa) while CRS-21 only reached a maximum pore pressure of 5 psi (34 kPa), yet both tests gave reasonably close values for C_v above P_c.

A possible solution to the determination of C_v in the early stages of the CG and CRS tests might be to impose an initial pore-pressure gradient prior to loading the specimen. This could be accomplished by applying a back pressure at the bottom of the specimen greater than that at the top of the specimen. This would, after some time, create a steady-state flow much the same as in a constant-head permeability test. Once the steady-state condition was established, the equation for C_v (Equation 6) would apply. This possible solution was not attempted in this research; and it is, therefore, recommended that future work investigate this technique.

Preconsolidation Pressures

Preconsolidation values obtained from consolidation tests is an indication of the maximum past vertical effective stress that has acted on the soil. A marked increase in compressibility occurs at this stress; yet, it is not so evident and abrupt that it can be precisely determined. Rather, the gradual increase in compressibility, as shown by the stress-strain data, points to a range of effective stresses within which the maximum past pressure or preconsolidation pressure may lie. The graphical procedure proposed by

Casagrande to determine P_c should not be considered rigorous or precise but rather an aid in locating this range of values. For this reason, P_c as determined by this method might be referred to as an "apparent" P_c. Values of P_c as determined by this method have been tabulated; however, alternative methods of determining P_c do exist for the CG and CRS tests.

The method of estimating P_c from CG data is to plot σ_v versus time. The point at which a change in slope occurs is indicative of P_c. For purposes of comparison, the values of P_c as estimated from the Casagrande construction are given in Table 2. Estimation of P_c from CRS data is accomplished by noting the value of σ_v at which pore pressure tends to increase.

Test Variables

Back Pressure -- All CG and CRS tests were performed under a back pressure of 10 psi (69 kPa). This is a relatively low value of back pressure and should not have affected the results significantly. To determine the effect, if any, that back pressure had on the data for the samples tested, STD tests were performed both with a back pressure of 10 psi (69 kPa) and without back pressure. To illustrate the effect of back pressure more clearly, Figure 14 shows a comparison of CR from STD tests with and without back pressure. The vertical and horizontal bars indicate a range of CR for two tests with back pressure and two tests without back pressure, respectively. The comparison is good, which shows that back pressure of 10 psi (69 kPa) has little or no effect on the value of CR for the soils tested. Figure 15 shows the same comparison for SR. In this case, the tests without back pressure consistently gave higher values of SR. Although the mechanism of this phenomenon is not understood, the back-pressure effect observed in these STD tests does not explain the higher SR values observed in the CG and CRS tests which were back-pressured. Therefore, the higher SR observed in CG and CRS tests is, apparently, not due to the fact that the tests were conducted under back pressure. Finally, the same comparison was made for P_c, as shown in Figure 16 where no significant back-pressure effects were observed.

The purpose of using back pressure in consolidation testing is to ensure saturation and duplication of in situ pore pressures. The samples from Site Numbers One and Two were fully saturated; samples from Site Number Three were above the water table and, therefore, were partially saturated. Thus, the use of back pressure for these soils is questionable. Even so, the back pressure used did not saturate the specimens; therefore, the assumption of complete saturation, used in developing the consolidation theory, was not fulfilled. Site Number Three should serve only as an indication of the effects of testing partially saturated soils.

Strain Rate -- Strain rate determines the pore pressures that will be generated in the testing and thus the applicability of the theory. Theories used in the CG and CRS tests assume parabolic pore-pressure distributions across the test specimen. If a specimen is strained at too slow a rate, little or no pore pressure will be generated and, although the calculation of σ_v' may not be affected, the effect on the determination of C_v will be pronounced. Since the term u_b appears in the denominator of the expression for C_v, a value of u_b equal to or approaching zero will cause the expression to be meaningless, since the theory assumes steady-state conditions which require some pore-pressure buildup. On the other hand, if pore pressures become excessive, assumptions made in deriving the theory will again be violated because the pore-pressure distribution will not be parabolic. Previous work *(3)* has shown that the term u_b/σ_v is a good indicator of excessive pore pressures. The maximum allowable value of u_b/σ_v was suggested by Smith and Wahls *(3)* to be about 50 percent. Pore pressure may be reduced by testing shorter specimens; however, this technique was not used for the tests reported herein. Minimum values of pore pressure are usually not a problem since the strain rate may then be increased and testing time reduced. Pore pressures of 1 psi (7 kPa) or greater are desirable.

Pore-pressure control through strain-rate selection is, of course, not a problem in the CG test, and herein lies the advantage of CG testing. Pore-pressure gradient is set to a constant value thoroughout the test. In the CRS test, however, a strain rate must be preselected so as to keep pore pressures within tolerable limits. This is a problem when testing a particular type of soil for the first time. Thus, a method of preselecting strain rate for the CRS test is needed.

To analyze the problem, a comparison was made of the range of strain rates in the CG test necessary to maintain a given pore-pressure gradient, the strain rate selected in the CRS test, and the maximum value of u_b/σ_v in the CRS test. Results of this comparison are shown in Table 3. In all cases, the values of u_b/σ_v in the CRS tests were below 32 percent, which is well within the 50-percent limit suggested by Smith and Wahls. Furthermore, the CRS test, in which the highest value of u_b/σ_v (32 percent) was encountered, showed good agreement in both stress-strain data and C_v data with the STD and CG tests. Thus, all strain rates shown are acceptable. The strain rate in the CG test usually decreases as the test progresses because of the tendency for pore pressures to increase as the test progresses. In almost all cases, the strain rate selected for the CRS test was between the maximum and minimum strain rates used in the CG test. Samples tested from Site Number Two, H-3, S-6, contained sand lenses and were therefore highly permeable, accounting for the extremely fast strain rate observed in the CG test. The effect of strain rate for a given soil type is shown by the two CRS tests performed for Site Number Two, H-2, S-5: strain rates varied by a factor of four and produced no significant changes in the stress-strain or C_v data. This indicates that selection of a strain rate may not be a critical factor and that selection of a rate within a fairly wide range of values will suffice.

It can be seen from consolidation theory that the one variable which determines how fast and how much pore pressure will increase in the CRS test (or tend to increase in the CG test) is C_v. Thus, any method of preselecting strain rate should be based on the value of C_v for the soil to be tested. Lower values of C_v should dictate lower strain rates. Unfortunately, the only method of determining C_v directly is the consolidation test. Attempts have been made to correlate C_v with liquid limit, LL *(8)*, but such correlations have been only moderately convincing. The C_v-LL correlation for the soils tested in this study is shown in Figure 17. C_v values shown in this correlation are rough estimates of C_v above the apparent preconsolidation pressure, P_c. Extension of the C_v-LL relationship to a LL-strain rate relationship provides a means of preselecting strain rate. The LL-strain rate relationship is shown in Figure 18. Since CRS tests exhibited vastly different pore pressures, the median strain rate in the CG test was correlated with LL. In spite of the poor correlation, a preliminary selection of a range of possible strain rates is possible based on LL values. This technique may prove useful due to the wide range of strain rates which will produce u_b/σ_v less than 50 percent and still generate at least 1 psi (7 kPa) pore pressure.

Durations of Tests

Significant differences in the durations of time required to complete the tests were noted. The STD tests with back pressure required one less day to complete because the loading system was limted to 16 tsf (1.6 MPa). The STD tests without back pressure were loaded an additional 24-hour increment (32 tsf, (3.2 MPa)) and thus required 9 days to complete. The time required to complete a STD test in this study was always the same; the time required to complete a CG or CRS test was variable. In the CG test, the time to completion depends on the value of the pore-pressure gradient selected and the compressibility and permeability of the soil. CG tests reported herein required between 1 and 6 days to complete, with 3.3 days being the average time to completion. Time to completion in the CRS test depends on the selected strain rate and the compressibility and permeability of the soil. CRS tests reported herein required between 1 and 4 days to complete, with 1.9 days being the average completion time.

Conclusions

1. No significant differences were observed in CG, CRS, or STD test data above P_c.

2. Of the three test methods considered, the CRS test required the least time and was the least difficult to perform. CRS tests reported herein required an average of 1.9 days to complete and required no manual adjustments at any time during the test.

3. CG and CRS tests must be monitored frequently. Since the CG and CRS tests require more than one working day to complete, a data-acquisition system is needed to monitor these tests.

4. Considerable latitude exists in the selection of a satisfactory strain rate for the CRS test. A strain rate should be selected which generates at least 1 psi (7 kPa) pore pressure but does not generate pore pressures in excess of 30 to 50 percent of the applied stress at any time during the test. Selection of strain rate may be based on the liquid limit of the soil to be tested. Until further investigation into the selection of strain rate in the CRS test is conducted, the following guidelines may be used:

If the liquid limit of the soil to be tested is greater than 60, use a strain rate of 50×10^{-4} percent/minute.

If the liquid limit of the soil to be tested is less than 60, use a strain rate of 100×10^{-4} percent/minute.

5. Values of C_v should be considered valid only above the preconsolidation pressure. Rate of consolidation below P_c is controlled by compressibility rather than permeability, and the theory used to derive the equations for C_v does not apply. Techniques for determining C_v in the early stages of the CG and CRS tests need to be developed. One possible technique would be to establish an initial pore-pressure gradient prior to loading the specimen.

6. P_c may be determined in the CRS test from plots of pore pressure, u_b, versus vertical effective stress and in the CG test from plots of applied stress versus time.

7. Standard testing procedures, including methods for pore-pressure gradient and strain-rate selection, should be developed for both the CG and CRS consolidation tests.

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TABLE 1. INDEX PROPERTIES OF SOILS

DEPTH (FEET)	DEPTH (METERS)	SAMPLE NUMBER	LIQUID LIMIT	PLASTICITY INDEX	NATURAL MOISTURE CONTENT (%)	CLASSIFICATION		GRADATION (%)		
						UNIFIED	AASHTO	SAND	SILT	CLAY
SITE NUMBER ONE										
5 - 7	1.5 - 2.1	1	32	11	13	CL	A-6	25	45	30
9 - 11.5	2.7 - 3.5	2	28	6	26	ML-CL	A-4	11	54	35
14 - 16.5	4.3 - 5.0	3	31	9	22	ML-CL	A-4	13	54	33
20 - 22.5	6.1 - 6.9	4	27	1	22	ML	A-4	19	55	26
25 - 26.5	7.6 - 8.1	5	27	1	23	ML	A-4	24	54	22
30 - 31.5	9.1 - 9.6	6	NP	NP	23	SM	A-4	64	23	13
SITE NUMBER TWO										
5 - 7.5	1.5 - 2.3	1	47	24	21	CL	A-7-6	44	12	44
10 - 12.5	3.0 - 3.8	2	54	35	25	CH	A-7-6	32	12	56
15 - 17.5	4.6 - 5.3	3	54	28	25	CH	A-7-6	32	14	54
20 - 22.5	6.1 - 6.9	4	48	33	27	CL	A-7-6	28	24	48
25 - 27.5	7.6 - 8.4	5	62	17	32	MH	A-7-5	10	26	64
30 - 32.5	9.1 - 9.9	6	26	13	27	CL	A-6	44	29	27
35 - 37.5	10.7 - 11.4	7	84	36	48	MH	A-7-5	0	18	82
SITE NUMBER THREE										
0.5 - 3	0.2 - 0.9	1	42	14	24	ML	A-7-6	20	35	45
3 - 5.5	0.9 - 1.7	2	41	11	25	ML	A-7-5	20	32	48
5.5 - 8	1.7 - 2.4	3	50	14	29	ML	A-7-5	12	32	56

TABLE 2. COMPARISON OF STD, CG, and CRS TEST RESULTS

SITE NUMBER ONE							
			STD-02*	STD-03	CG-04	CRS-05	
H-2, S-2	CR		0.138	0.112	0.129	0.112	
	SR		0.001	0.006	0.013	0.007	
	P_c	(tsf)	0.945	0.910	1.83	2.84	
		(kPa)	90.5	87.1	175	272	
			STD-04	STD-05*	CG-08	CRS-06	
H-1, S-2	CR		0.122	0.102	0.107	0.087	
	SR		0.007	0.001	0.012	0.019	
	P_c	(tsf)	0.881	0.840	0.997	2.33	
		(kPa)	84.4	80.4	95.5	223	
			STD-06	CG-10	CRS-12		
H-2, S-3	CR		0.112	0.092	0.085		
	SR		0.004	0.009	0.013		
	P_c	(tsf)	1.18	2.00	2.86		
		(kPa)	173	192	2.74		
			STD-08*	STD-09	CG-06	CRS-07	
H-1, S-5	CR		0.088	0.115	0.085	0.093	
	SR		0.002	0.001	0.009	0.015	
	P_c	(tsf)	0.878	1.55	1.49	1.26	
		(kPa)	841	148	143	121	
SITE NUMBER TWO							
			STD-11	CG-13			
H-3, S-2	CR		0.219	0.163			
	SR		0.003	0.01[illegible]			
	P_c	(tsf)	9.59	9.7[illegible]			
		(kPa)	918	935			
			STD-12*	STD-13	CG-12	CRS-17	
H-3, S-4	CR		0.171	0.223	0.175	0.154	
	SR		0.006	0.009	0.015	0.018	
	P_c	(tsf)	4.37	5.77	5.88	5.49	
		(kPa)	418	553	563	526	
			STD-14*	STD-15	STD-17*	CG-02	CRS-19
H-3, S-5	CR		0.201	0.241	0.184	0.210	0.148
	SR		0.021	0.023		0.039	0.036
	P_c	(tsf)	5.25	9.12	5.48	6.73	5.15
		(kPa)	503	874	525	644	493
			STD-16	CG-14	CRS-15	CRS-21	
H-2, S-5	CR		0.279	0.207	0.179	0.169	
	SR		0.039	0.020	0.036	0.025	
	P_c	(tsf)	8.56	9.81	8.11	8.38	
		(kPa)	820	939	777	802	
			STD-18	STD-19*	CG-03	CRS-11	
H-3, S-6	CR		0.150	0.194	0.126	0.151	
	SR		0.012	0.011	0.012	0.017	
	P_c	(tsf)	4.43	7.24	5.26	6.11	
		(kPa)	424	693	504	585	
			STD-21	STD-22	STD-23*	CG-11	CRS-18
H-3, S-7	CR		0.435	0.381	0.446	0.392	0.308
	SR		0.048		0.032	0.043	0.044
	P_c	(tsf)	3.17	2.61	3.40	2.92	2.36
		(kPa)	304	250	326	280	226
SITE NUMBER THREE							
			STD-28	STD-29*	CG-15	CRS-20	
H-2&3, S-2	CR		0.147	0.113	0.118	0.133	
	SR		0.018	0.004	0.013	0.028	
	P_c	(tsf)	5.15	6.13	12.1	7.04	
		(kPa)	493	587	1159	674	
			STD-30	STD-31*	GC-16	CRS-09	
H-2&3, S-3	CR		0.131	0.095	0.108	0.137	
	SR		0.024	0.017	0.019	0.021	
	P_c	(tsf)	5.95	4.33	9.07	2.22	
		(kPa)	570	415	869	213	

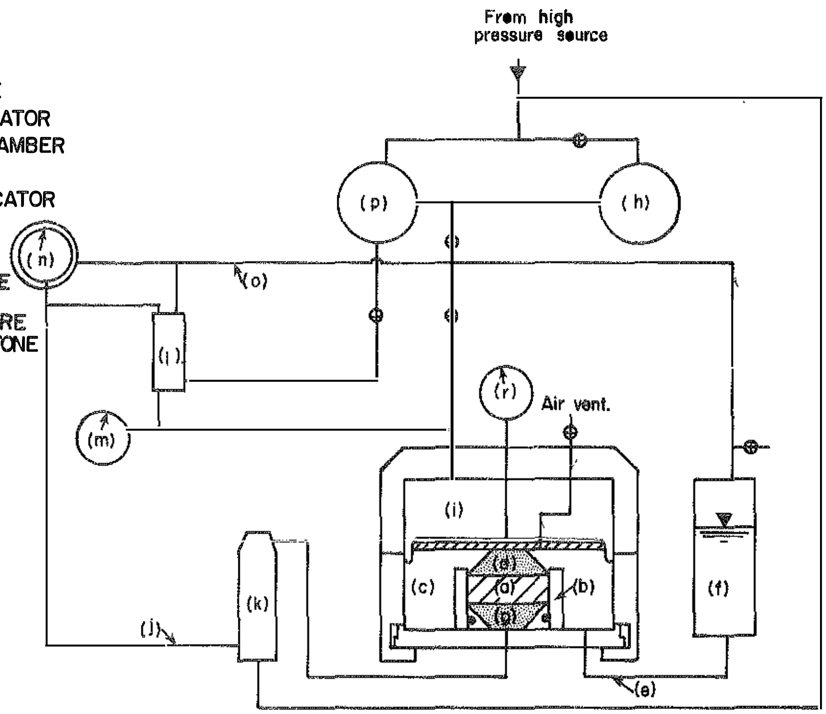
*Back Pressured

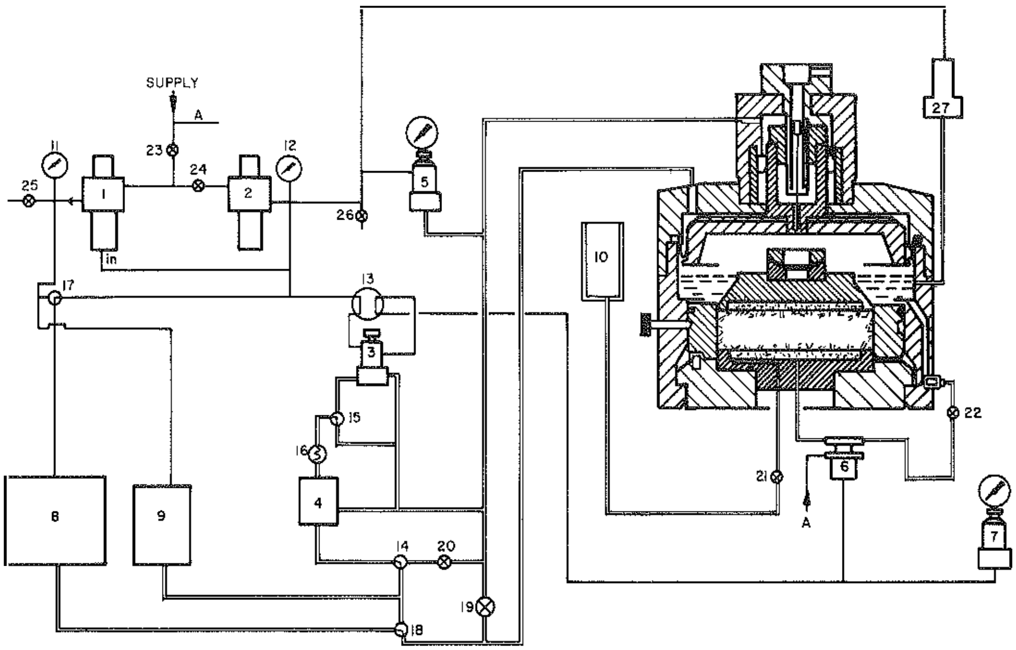
TABLE 3. COMPARISONS OF STRAIN RATES AND PORE PRESSURES FROM CG AND CRS TESTS

TEST IDENTIFICATION	PORE-PRESSURE GRADIENT IN CG TEST (psi)	PORE-PRESSURE GRADIENT IN CG TEST (kPa)	RANGE OF STRAIN RATES IN CG TEST (10^{-4}%/min)	STRAIN RATE IN CRS TEST (10^{-4}%/min)	MAXIMUM VALUE OF u_b/σ_v IN CRS TEST(%)
SITE NUMBER ONE H-2 S-2	2.0	14	4 - 500	250	27
SITE NUMBER ONE H-1 S-2	2.6	18	26 - 600	150	10
SITE NUMBER ONE H-2 S-3	2.1	14	160 - 710	160	4
SITE NUMBER ONE H-1 S-5	2.2	15	110 - 400	150	7
SITE NUMBER TWO H-3 S-4	2.5	17	14 - 1700	68	2
SITE NUMBER TWO H-3 S-5	3.0	21	16 - 38	50	32
SITE NUMBER TWO H-2 S-5	2.8	19	18 - 200	160 & 38	16 & 6
SITE NUMBER TWO H-3 S-6	2.8	19	3200	870	1
SITE NUMBER TWO H-3 S-7	2.7	19	19 - 300	65	4
SITE NUMBER THREE H-2&3 S-2	2.8	19	13 - 200	38	7
SITE NUMBER THREE H-2 S-3	3.0	21	6 - 340	160	7

Figure 1. Simplified Schematic of CG Test Equipment.

(a) SPECIMEN
(b) CONSOLIDATION RING
(c) BACK PRESSURE CHAMBER
(d) UPPER POROUS STONE
(e) OUTFLOW LINE
(f) RESERVOIR
(g) BOTTOM POROUS STONE
(h) BACK PRESSURE REGULATOR
(i) PNEUMATIC LOADING CHAMBER
(j) PORE PRESSURE LINE
(k) PORE PRESSURE DUPLICATOR
(l) LOAD PACER
(m) TOTAL LOAD GAUGE
(n) DIFFERENTIAL PRESSURE GAUGE BETWEEN BACK PRESSURE AND PRESSURE IN BOTTOM POROUS STONE
(o) BACK PRESSURE LINE
(p) LOAD REGULATOR
(r) EXTENSOMETER



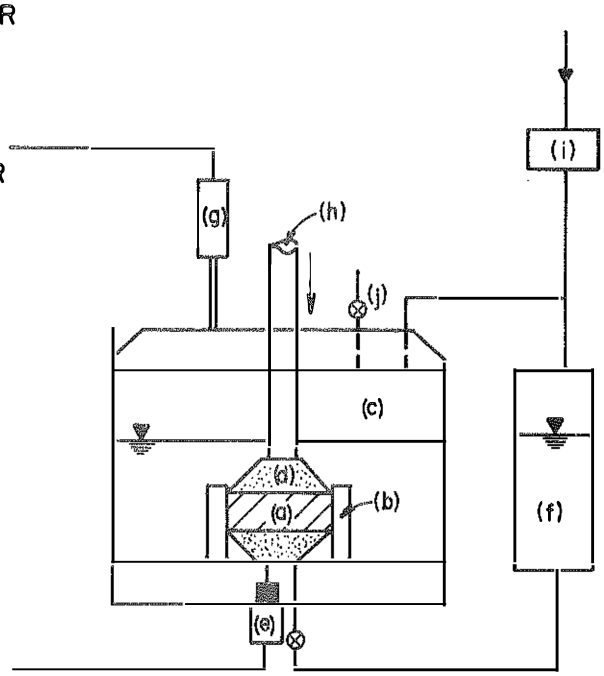


1. LOAD REGULATOR
2. BACK PRESSURE REGULATOR
3. LOAO PACER
4. FLOW REGULATOR
5. LOAD DIAL
6. PORE PRESSURE DUPLICATOR
7. PORE PRESSURE DIAL
8. HIGH LOAD RESERVOIR
9. LOW LOAD RESERVOIR
10. SATURATION WATER INLET
11. TOTAL PRESSURE GAUGE
12. BACK PRESSURE GAUGE
13. LOAD-UNLOAD SELECTOR
14. INCREMENTAL LOAD SELECTOR
15. STRAIN RATE LOAD SELECTOR
16. STRAIN RATE VALVE
17. HIGH-LOW LOAD SELECTOR
18. "
19. "
20. TERMINAL VALVE
21. SATURATION WATER VALVE
22. EQUALIZER VALVE
23. LOAD SUPPLY VALVE
24. BACK PRESSURE SUPPLY VALVE
25. LOAD VENT
26. BACK PRESSURE VENT
27. WATER TRAP

Figure 2. Schematic of CG Test Equipment.

Figure 3. Schematic of CRS Test Equipment.

(a) SPECIMEN
(b) CONSOLIDATION RING
(c) CHAMBER
(d) POROUS STONE
(e) PORE PRESSURE TRANSDUCER
(f) RESERVOIR
(g) DISPLACEMENT TRANSDUCER
(h) LOADING RAM
(i) BACK PRESSURE REGULATOR
(j) AIR VENT



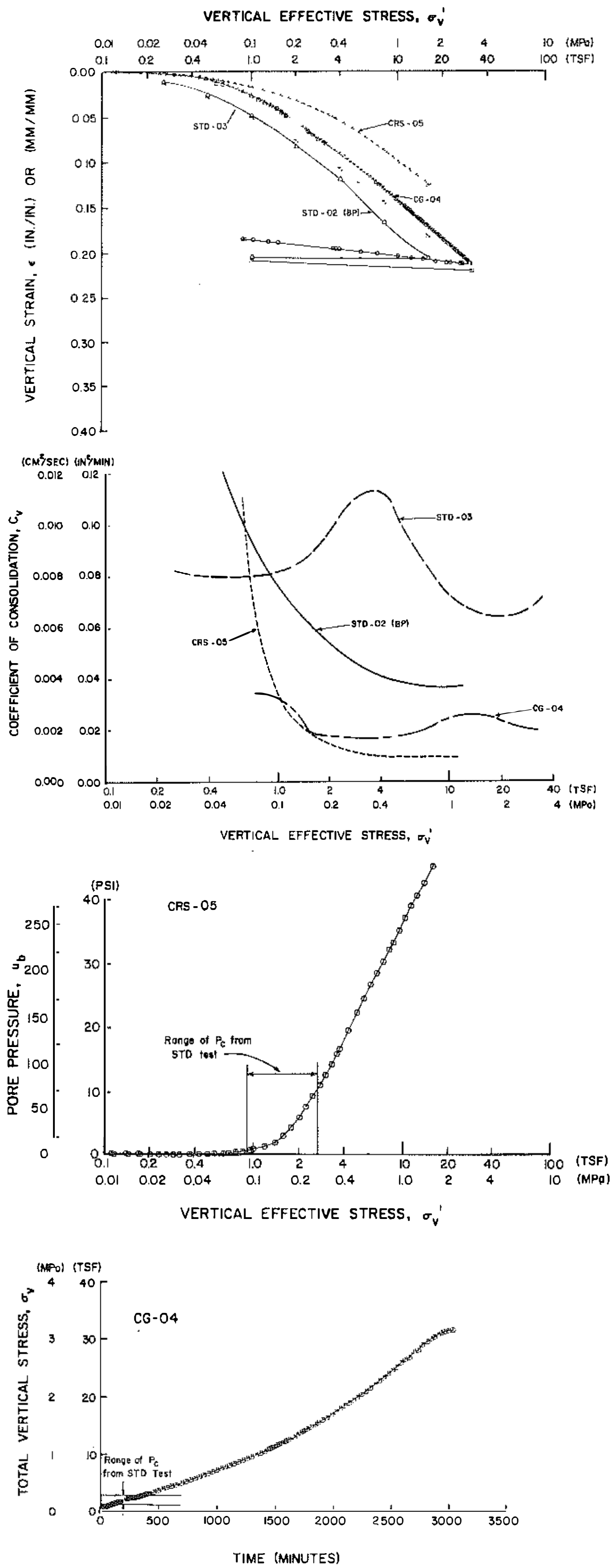


Figure 4. Comparison of STD, CG, and CRS Test Results; Site Number One, H-2, S-2.

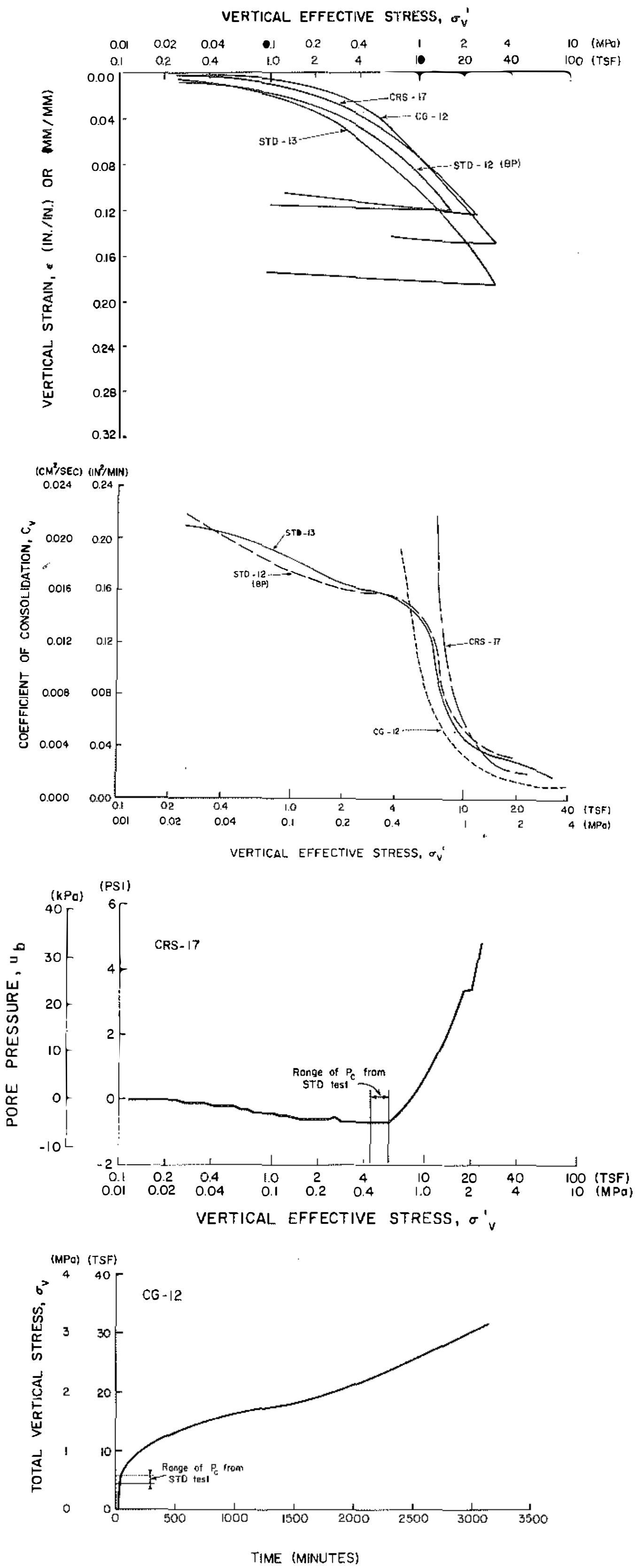


Figure 5. **Comparison of STD, CG, and CRS Test Results; Site Number Two, H-3, S-4.**

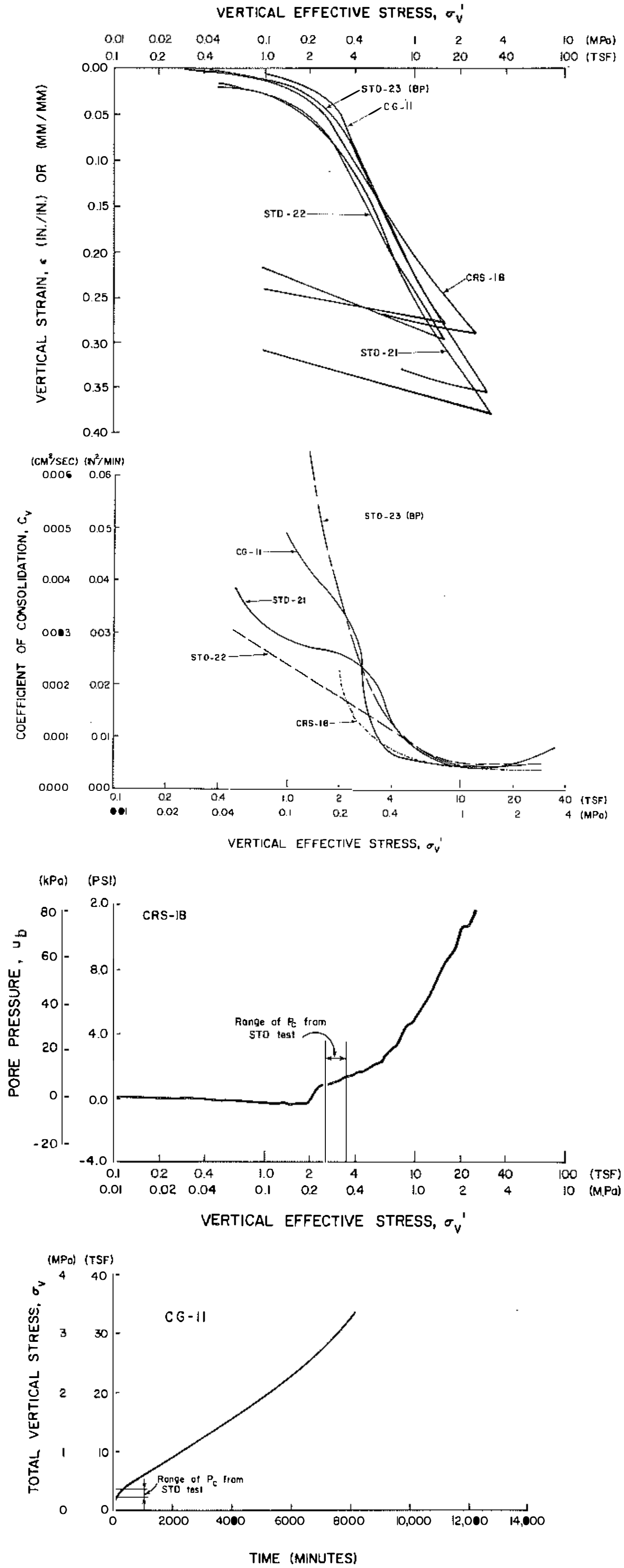


Figure 6. Comparison of STD, CG, and CRS Test Results; Site Number Two, H-3, S-7.

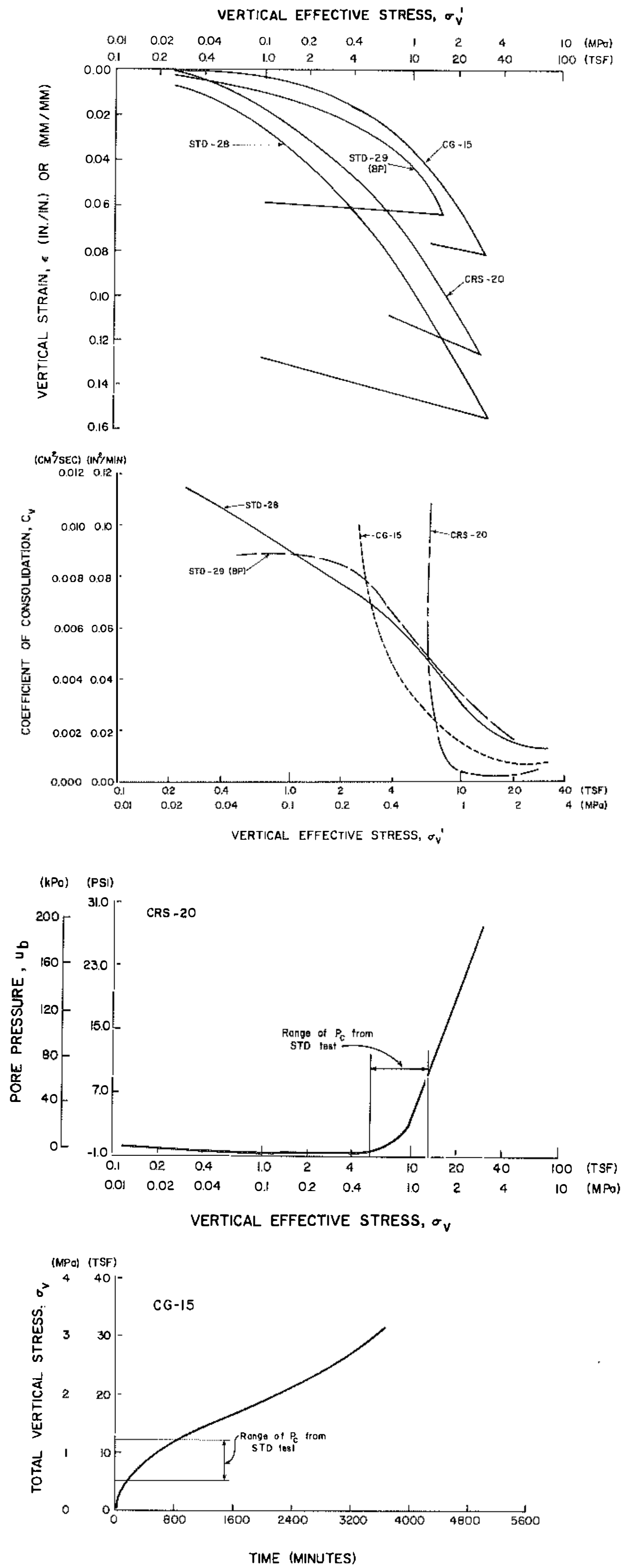


Figure 7. **Comparison of STD, CG, and CRS Test Results; Site Number Three, H-2 and 3, S-3.**

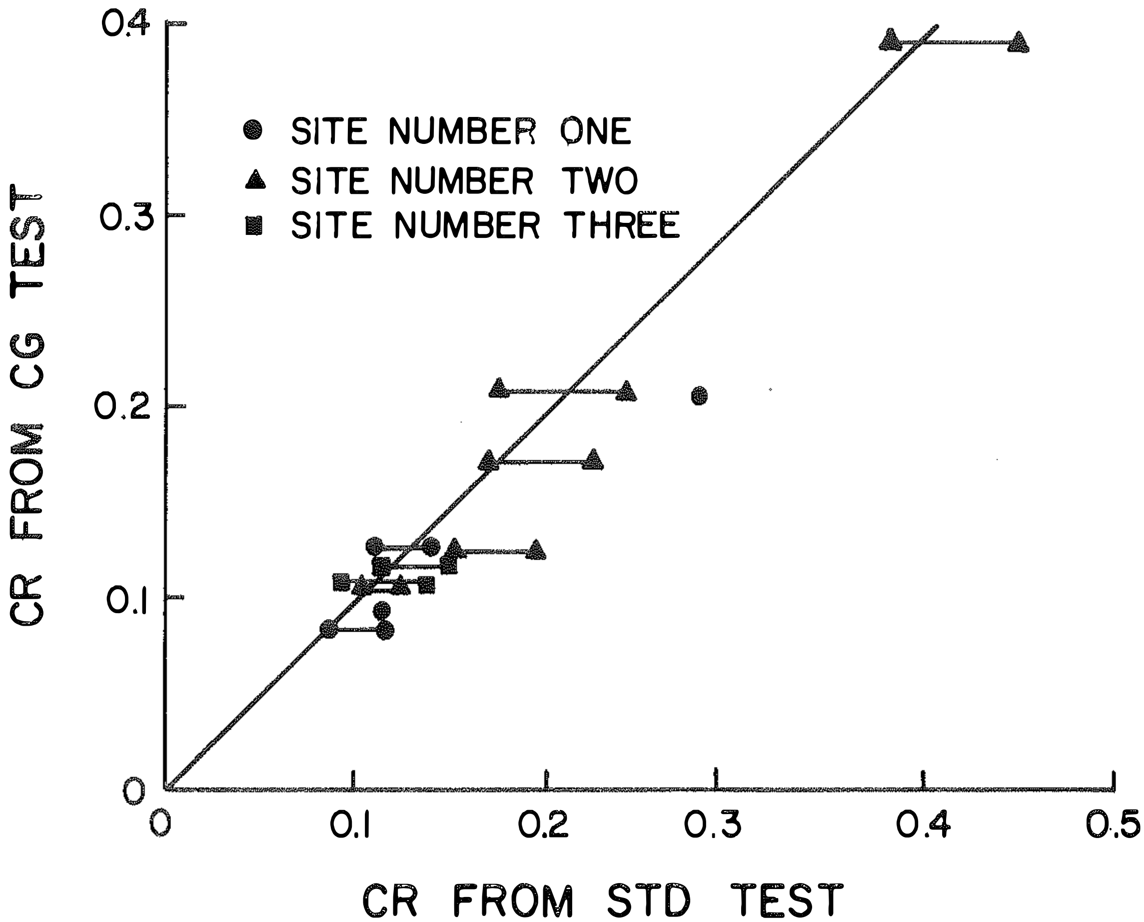


Figure 8. CR Comparison, CG Test and STD Test.

Figure 9. CR Comparison, CRS Test and STD Test.

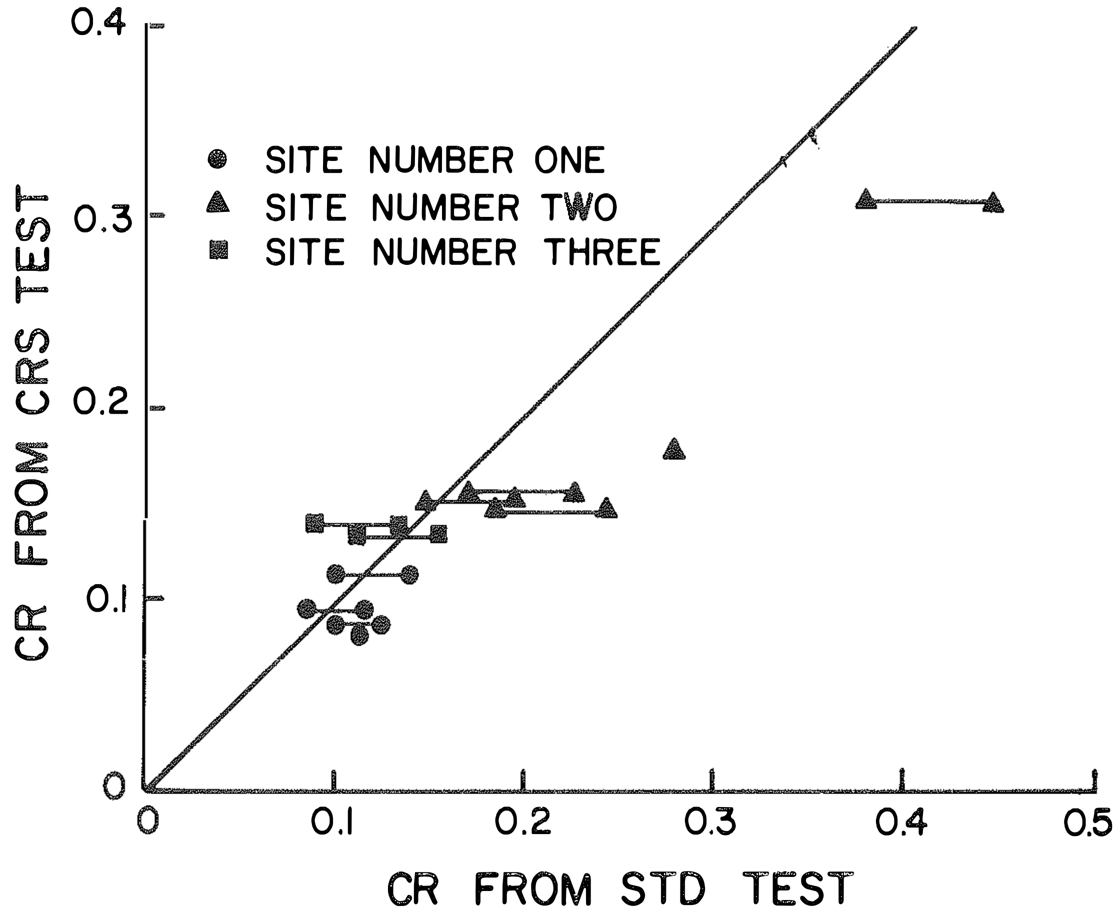
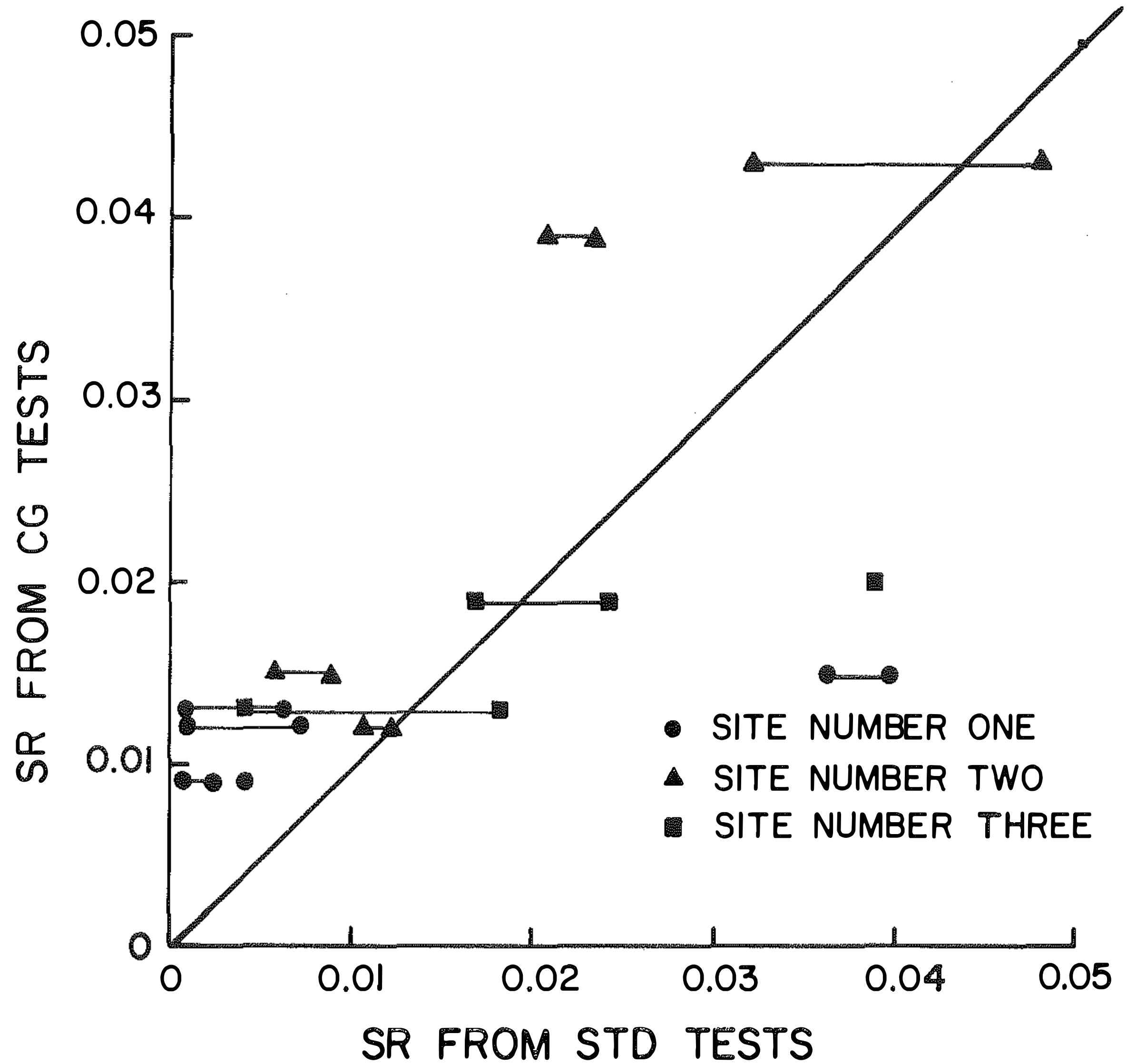


Figure 10. SR Comparison, CG Test and STD Test.



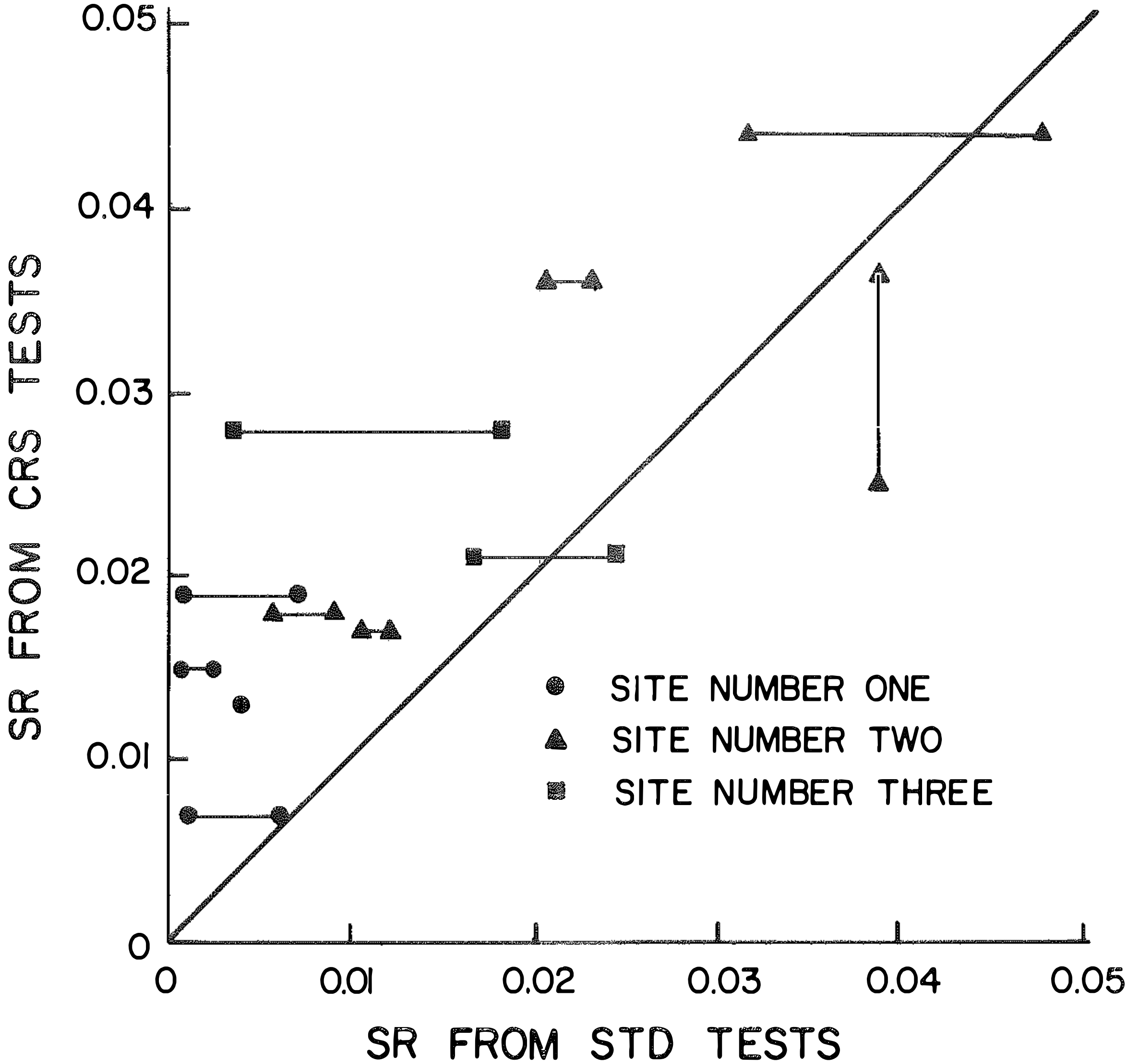
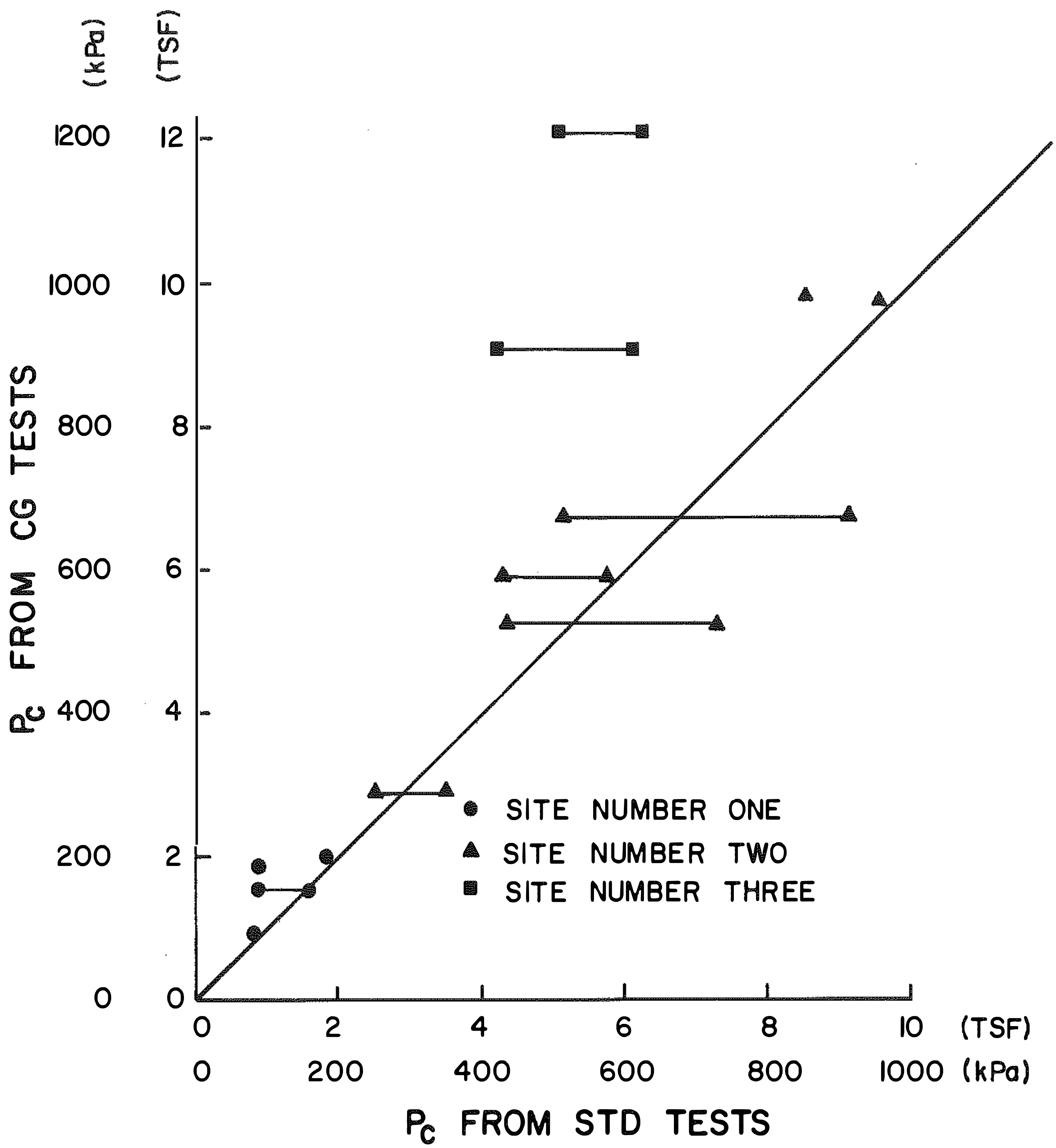


Figure 11. SR Comparison, CRS Test and STD Test.

Figure 12. P_c Comparison, CG Test and STD Test.



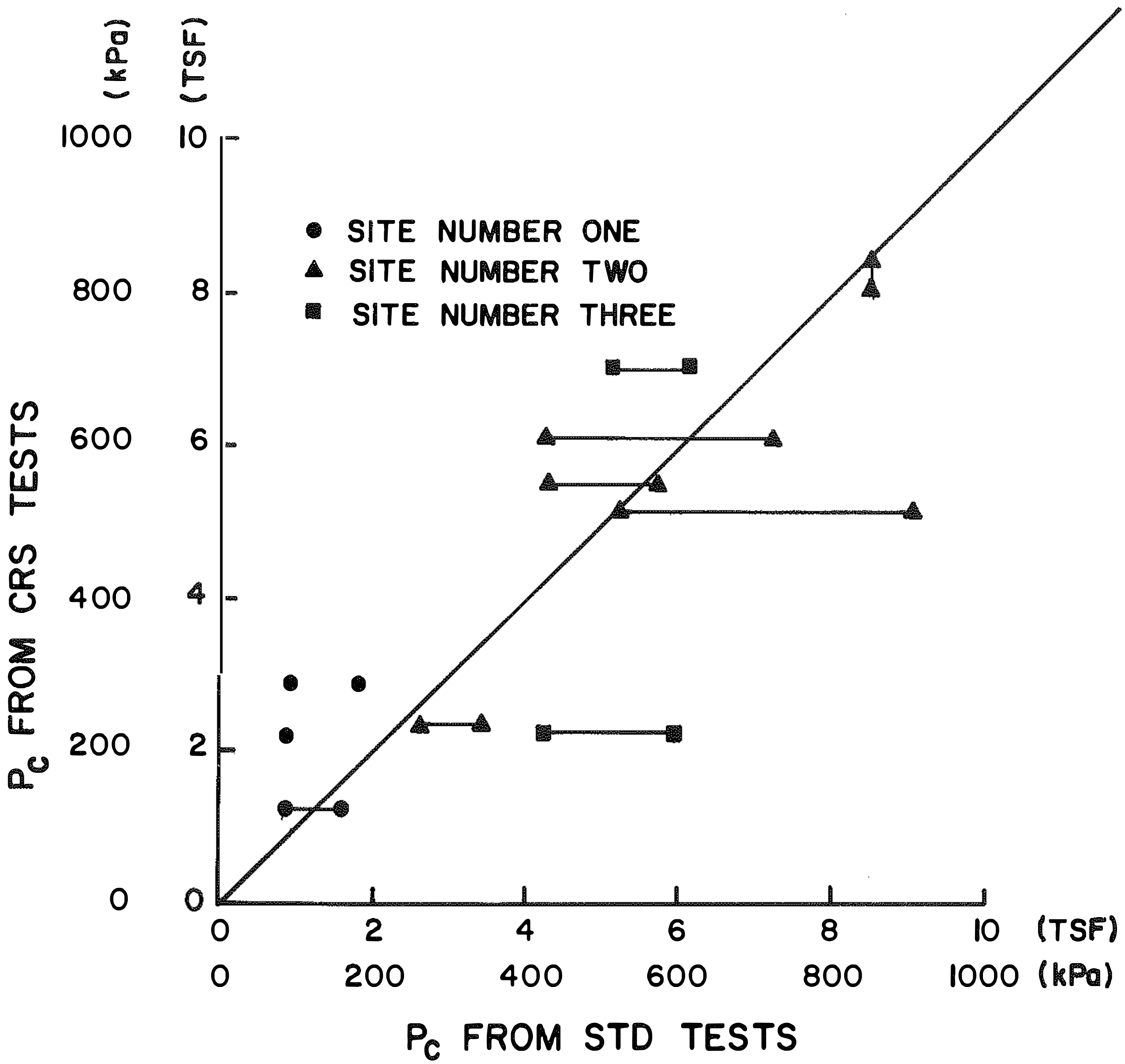
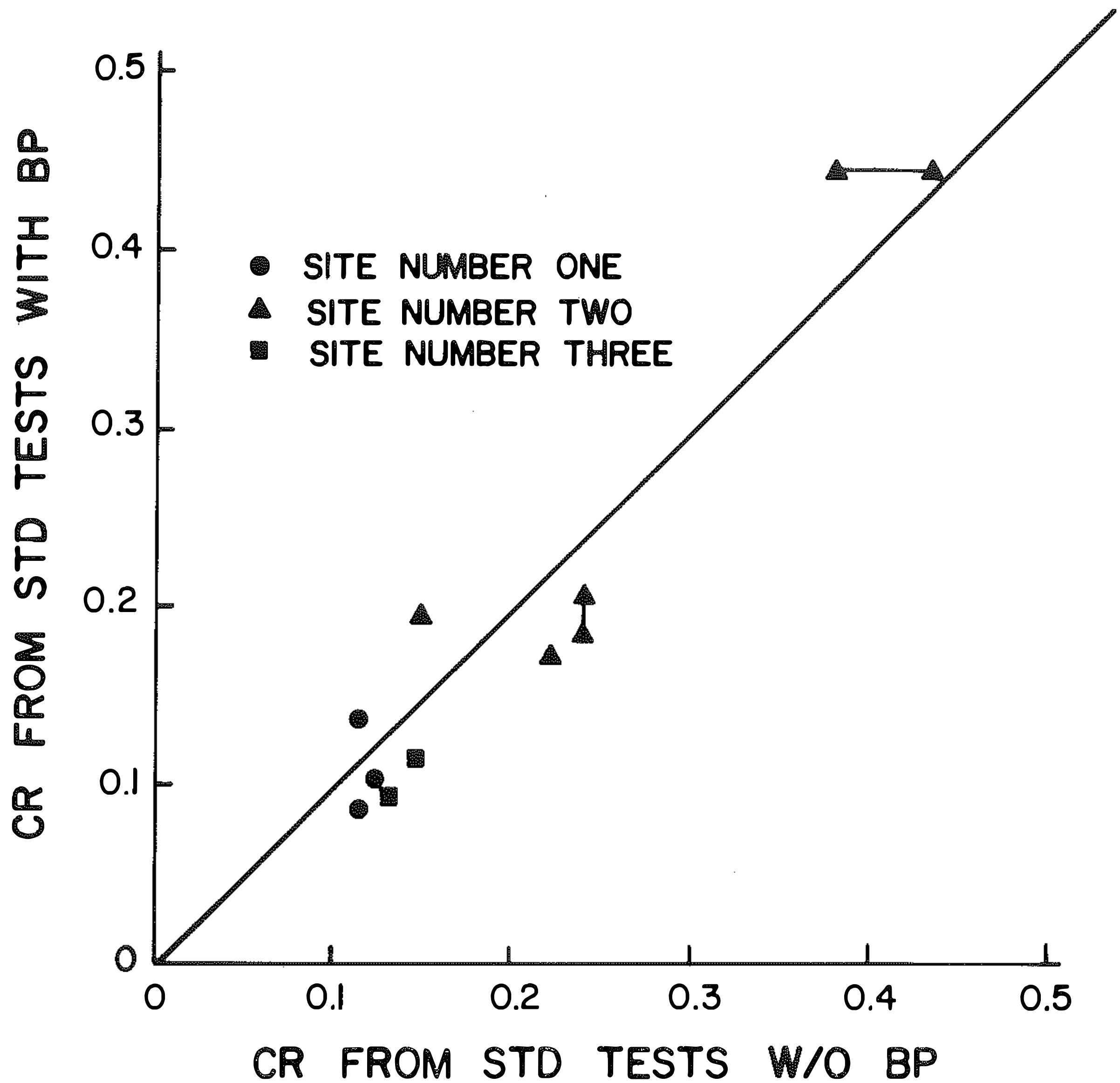


Figure 13. P_c Comparison, CRS Test and STD Test.

Figure 14. CR Comparison, STD Tests with and without Back Pressure.



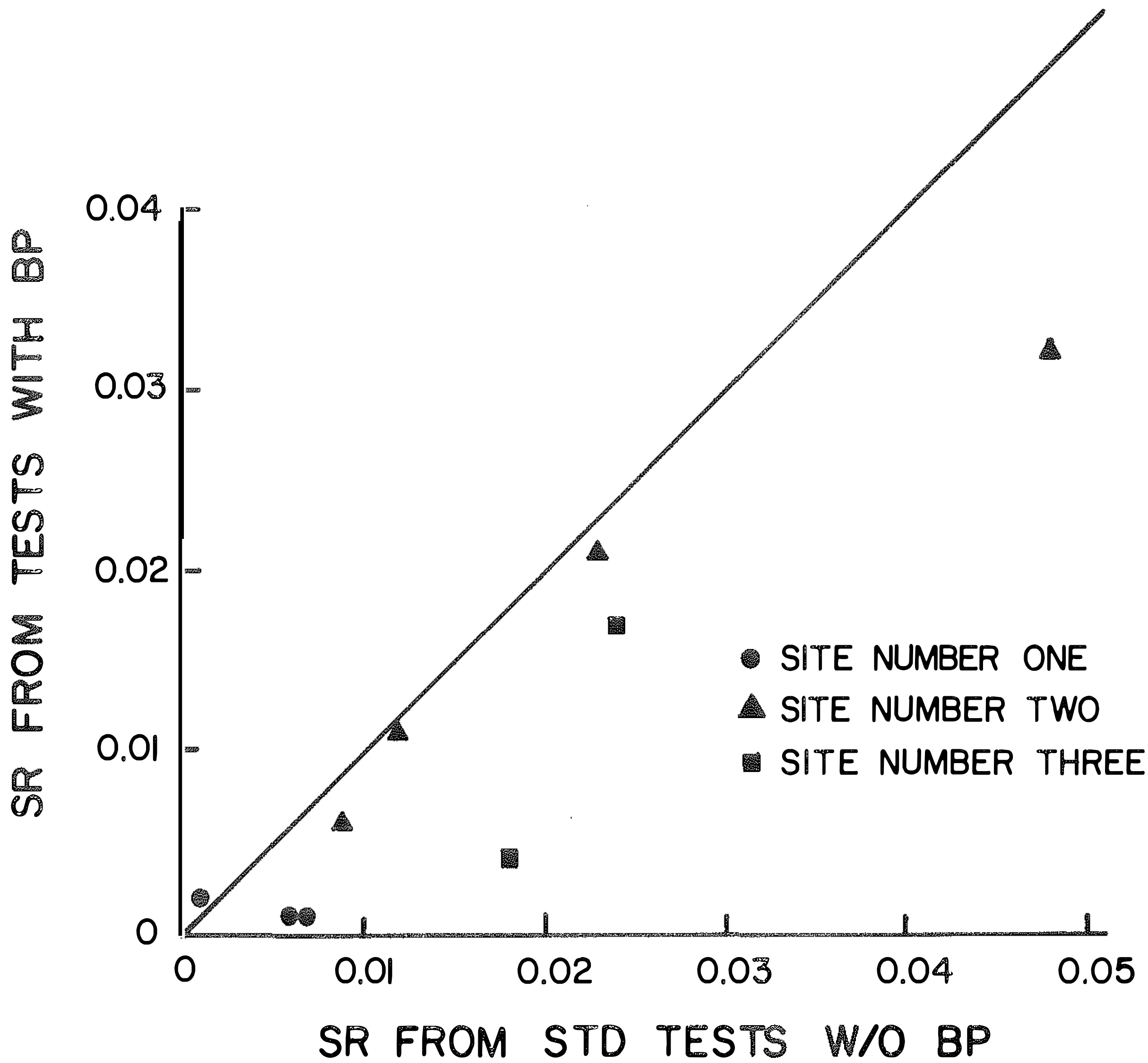
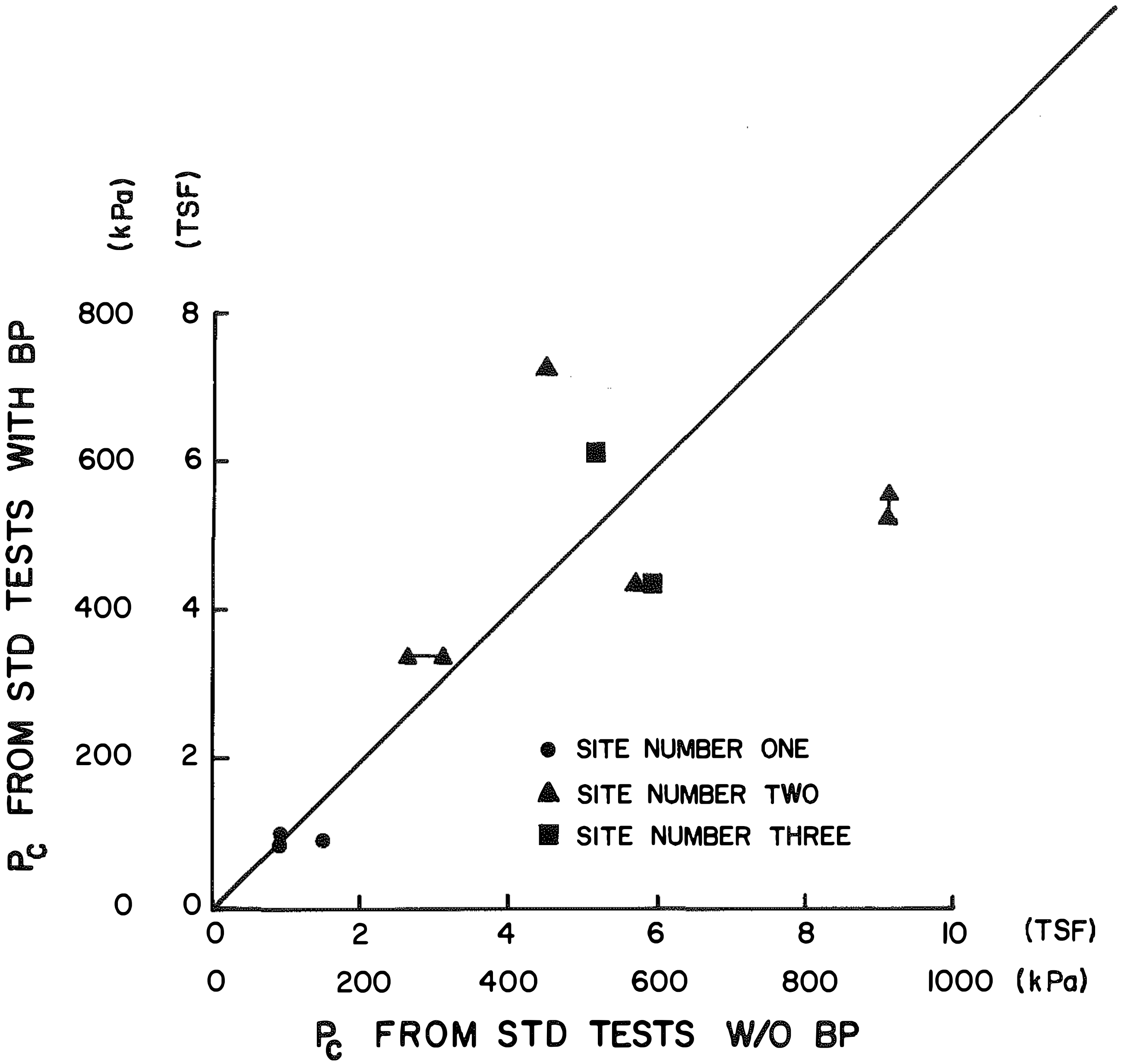


Figure 15. SR Comparison, STD Tests with and without Back Pressure.

Figure 16. P_c Comparison, STD Tests with and without Back Pressure.



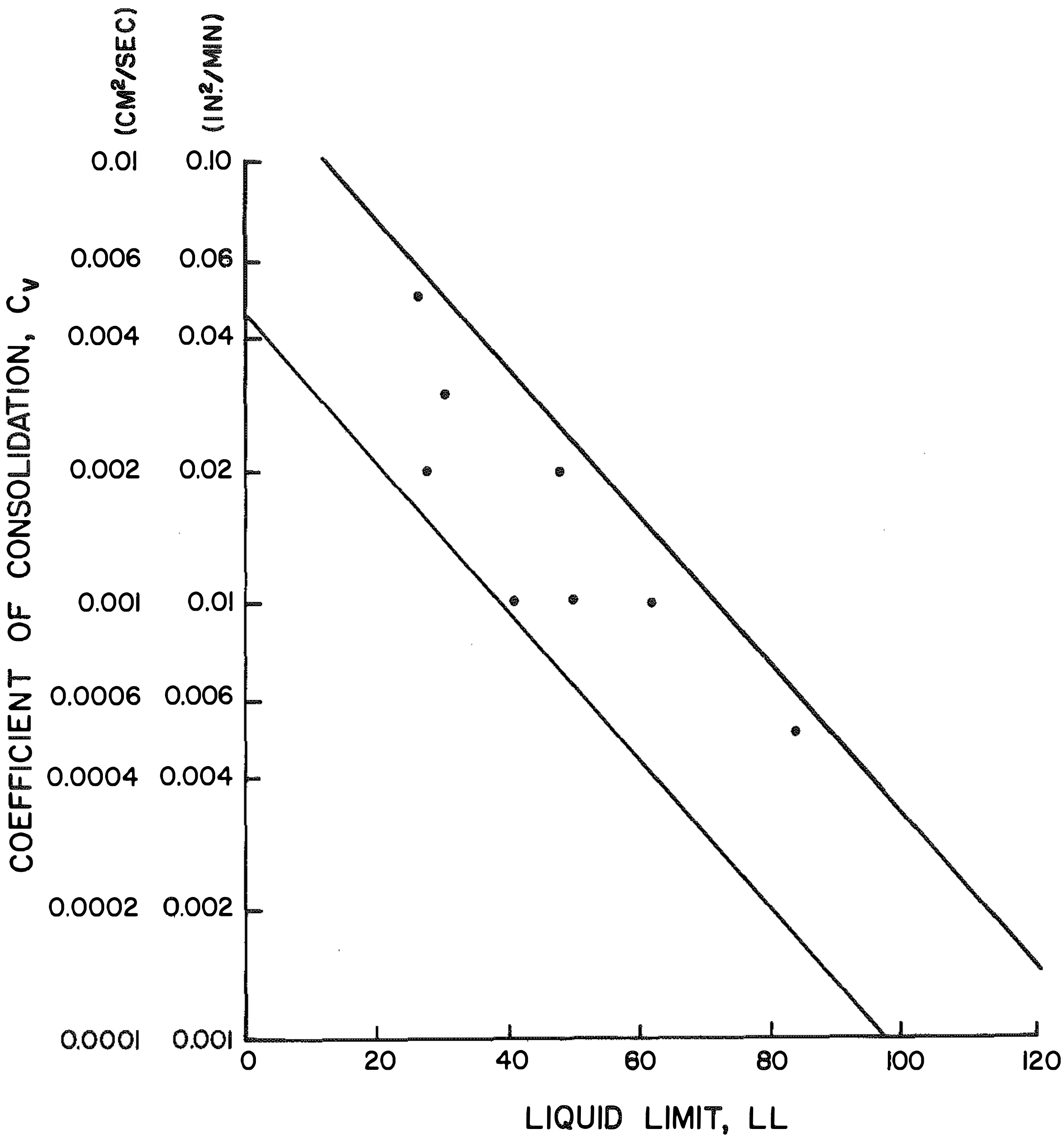


Figure 17. C_v from Consolidation Tests versus Liquid Limit.

Figure 18. Median Strain Rate from CG Test versus Liquid Limit.