Drained-Strength Parameters from Direct Shear Tests for Slope Stability Analyses in Overconsolidated Fissured Residual Soils

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An experimental investigation of the drained shear strength of soils from three sites within the Durham Triassic Basin of North Carolina is described. Multistage direct shear tests conducted on specimens trimmed from block samples were utilized to evaluate the postpeak shear strength of these materials. In general, the observed reduction in strength with accumulated displacement beyond the peak strength was in good agreement with data reported in the literature for other overconsolidated soils. The residual friction angle for the soils tested was found to follow the trend of decreasing friction angle with increasing plasticity index. These results support the need for utilizing drained shear strength parameters and effective stress analyses for predicting the stability of slopes cut in overconsolidated residual soils. In a companion paper in this Record a case study is presented of a well-documented slope failure within the Durham Triassic Basin that indicates the appropriateness of using noncircular failure geometrics and suggests the applicability of utilizing postpeak strengths obtained from drained direct shear tests.

This paper is the first of two companion papers dealing with slope stability analyses in overconsolidated fissured residual soils. An experimental investigation of the drained shear strength of soils from three sites within the Durham Triassic Basin of North Carolina is described. The companion paper by Putrich et al. in this Record presents an evaluation and analysis of a documented slope failure utilizing some of the data presented here. The impetus for this study was the realization that failures were still occurring in slopes for which stability analyses based on undrained strengths had predicted stability. For example, utilizing the unconfined compressive strength in conjunction with a realistic noncircular failure surface on the basis of a field investigation after the slide, a failed slope was shown to have a factor of safety of approximately 2.5. A circular arc analysis utilizing the unconfined strength shows the slope to be even slightly safer.

It is well known that the long-term stability of overconsolidated clay slopes is governed by effective-strength parameters (i.e., drained conditions). The explanation for this behavior was originally presented by Vaughan and Walbanke (*1*) and later by Skempton (*2*), who concluded that failure of cut slopes in overconsolidated fissured clays occurred years after the excavation because of the slow dissipation of excess negative pore pressures. This study shows that the overconsolidated fissured residual soils investigated in this experimental program may be viewed in a similar manner.

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GEOGRAPHIC AND GEOLOGIC BACKGROUND

Three sites have been investigated in this study; two are sites of future cut slopes that will result from construction of the East-West Freeway through Durham, North Carolina, referred to as the Anderson Street and LaSalle Street sites, and the third is the site of a previously documented slope failure in nearby Apex, North Carolina, the Railroad Underpass/US-64 site. Geographically these three sites are all located in the Durham Triassic Basin, which is the northernmost portion of the Deep River Basin. The Deep River Basin is a troughlike topographic lowland trending northeast to southwest that extends 100 mi in length and varies from 5 to 20 mi in width. Figure 1 shows the Deep River Basin deposit in relation to the Atlantic Seaboard Triassic deposits and the general locations of the three sites investigated in this study.

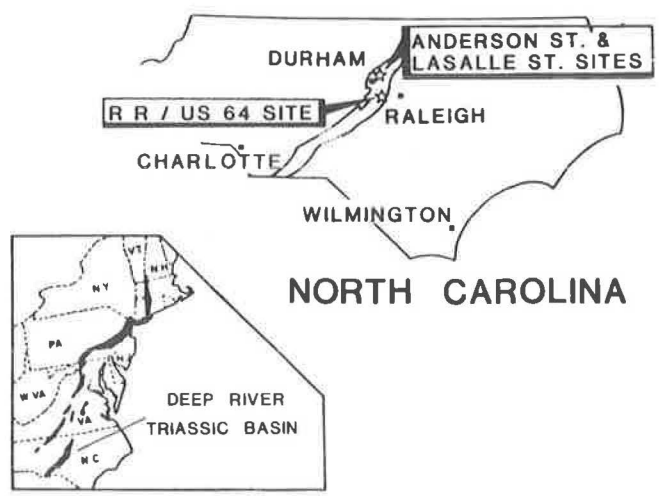

FIGURE 1 Location of Deep River Triassic Basin and test sites.

Geologically the Deep River Basin contains Triassic sedimentary rocks that are clastic deposits consisting of claystone, shale, siltstone, and sandstone. These deposits are characterized by abrupt changes in composition. The sediments of the Deep River Basin are composed largely of debris eroded from nearby pre-Triassic metamorphic and igneous rocks. In places they contain large amounts of debris derived from nearby granite intrusives. These sediments were deposited as alluvial fans, stream-channel and floodplain deposits, and lake and swamp deposits. Because of the nature of the depositional environment, a large percentage of the clays and clayey silts are fissured. Because the soils were deposited under wet conditions, subsequent drying and shrinkage caused cracks and openings classified as fissures. Soil samples from two of the three sites investigated in this study contained fissures with undulating surfaces.

Triassic sedimentary deposits are characterized by inclinations to the southeast at angles ranging from 5 to 45 degrees. This characteristic dipping of the deposits is believed to have a major impact on the stability of cut slopes in the Triassic deposits. The geometry of the failed soil mass appears to be dependent on the orientation of the soil layering. As a point of illustration, the documented slide at the Railroad Underpass/US-64 site is discussed in detail in the companion paper of this experimental study (Putrich et al. in this Record). The soil profile within the slope had layering that was inclined into the cut; therefore, the soil mass had an inherent tendency to slide into the cut. Conversely, the bedding planes on the opposite side of the railway cut dip away from the cut and the slope has remained stable (has not experienced sliding) since the end of construction more than 10 years ago. This illustrates the importance of the orientation of the layering of the subsurface profile with respect to the cut and its direct influence on the probability of slope failure, in addition to the determination of the appropriate shear strength used in the design of these slopes. As was reported by Leith and Fisher (*3*), the slope failure frequency in the Triassic Basin was anomalously high based on a statewide survey of highway cuts at that time. It can be inferred that the characteristic southeasterly dip of the Triassic sedimentary soil profile was a major factor in inducing a large number of those failures. Henkel (*4*) supports this position by stating that local geologic environment greatly influenced the stability of natural slopes, and therefore stability analyses using circular failure geometries and classical design methods developed for uniform soils are unconservative. It is critical that strength parameters determined are applicable to sliding along these bedding planes and are appropriate in the evaluation of long-term stability.

EXPERIMENTAL PROGRAM

The following is a description and rationalization of the laboratory testing program used in this study for the determination of the shear strengths of soils from three locations in the Durham Triassic Basin. Two of the three locations are sites of future bridge structures for the East-West Freeway project. The third is the site of a previous slope failure that was investigated and documented by the North Carolina Department of Transportation (NCDOT) in 1978.

Testing Equipment

The device used to evaluate shear strength in this study was a standard motor-driven direct shear device made by Soiltest Company. The device accommodates 2.5-in.-diameter samples and has a maximum displacement of 0.25 in. per run. The device was modified to enable reversing of the upper one-half of the shear box to its original position after each run. Therefore, a test can be conducted with the desired amount of accumulated one-directional displacement for each value of effective normal stress (or normal stress increment).

The reversible direct shear test was chosen as suitable for determining the shear strength of soils in this study for the following reasons:

1. The test allows for the accumulation of adequate horizontal displacement to attain postpeak shear strength considered applicable to slope stability analysis.
2. The test can be conducted as a drained shear test, where the displacement rate is adjusted to allow for drainage during the shearing of the sample. Drained-strength parameters are known to be important in the long-term stability of cut slopes.
3. Samples can be positioned and aligned in the shear box so that the plane on which shearing takes place can be selected before testing. This enables shear strength determination of seams, fissures, slickensides, and in general any plane of weakness considered susceptible to failure in the field.
4. There is a body of direct shear data accumulated from studies on a variety of soils with which the results of this study can be compared. Two such studies are those by Kenney (*5*) and by Lupini et al. (*6*).
5. The direct shear test can be conducted by using only one sample to develop a range of strength envelopes. This minimizes the number of samples necessary in the determination of shear strength.

No laboratory test is without its weaknesses. Some of the difficulties and intrinsic weaknesses in this method of direct shear testing are as follows:

1. The test is time consuming if it is to be conducted completely drained (3 to 6 working days of test time);
2. Particular attention must be given to ensure that the shearing surface is approximately planar and that the proposed plane of shearing corresponds to the actual plane of shearing (i.e., the shearing surface must be aligned with the plane of separation of the shear box); this is most important in determining the strength seams or fissures within a soil sample; and
3. Bromhead and Curtis (*7*) note in their study of residual strength determination that because of the reversing of the shear to return the box back to its original position, there is no allowance for complete particle orientation in the shear surfaces, and the reversible direct shear test therefore yields a slightly higher "residual strength" than other tests such as that for torsional shear.

Laboratory Technique

In order to develop and validate a standard laboratory technique for this series of direct shear tests, it was necessary to

identify the key variables and discuss the tests conducted to evaluate their significance.

Rate of Displacement

Since the direct shear tests must be conducted as drained tests (slow tests), it was important to select an initial displacement rate that would allow for drained behavior during shear. Using the relationship between liquid limit (W_L) and the coefficient of consolidation (c_v), as shown in Figure 2 (*8*), it was possible to determine the required time to failure in order to ensure drained behavior. Because the soils are overconsolidated, the top curve was used as a conservative estimate for c_v. Knowing c_v, one can back-calculate the time required for consolidation corresponding to a specific average percentage of consolidation [$t = (H_{dr})^2T/c_v$] and the time to failure. ASTM D 3080 recommends a time to failure for drained shear tests as $50 \times t_{50}$, and Bishop and Henkel (*9*) recommended $10 \times t_{100}$. Both methods gave approximately the same required time to failure and subsequent suggested displacement rates of 2.2×10^{-3} in./min. The conservative nature of the relationship between W_L and c_v was confirmed by consolidation data from the Anderson Street site (on the soil having a W_L of 55) shown in Figure 3. Even the lowest c_v-value of 0.3×10^{-2} cm2/sec is only slightly lower than the projected value based on Figure 2. As a second check on the rate of displacement, one long-term test was conducted at one-third the standard displacement rate. The slower rate did not vary the shear strength developed in the faster test, and therefore confirms that the faster rate of displacement allowed drained behavior during shear. On the basis of these data, a standard displacement rate of 2.2×10^{-3} in./min was adopted for all testing.

Amount of Horizontal Displacement

In standardizing the amount of horizontal displacement for the direct shear testing in this study, two factors were considered:

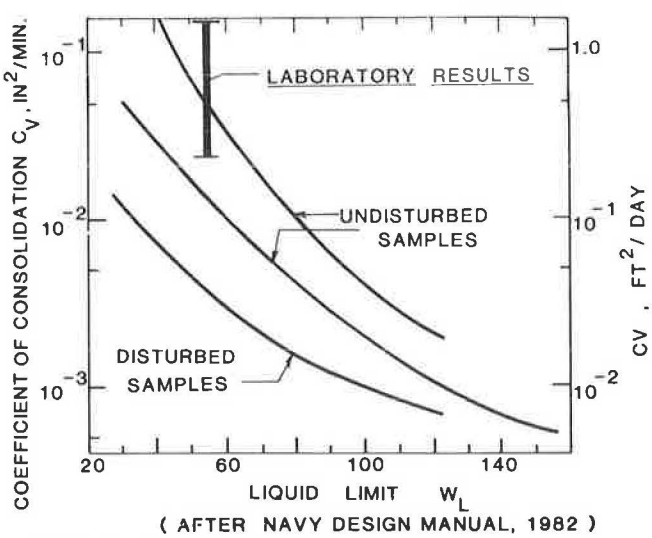

FIGURE 2 Relationship between coefficient of consolidation (c_v) and liquid limit (W_L) (*8*).

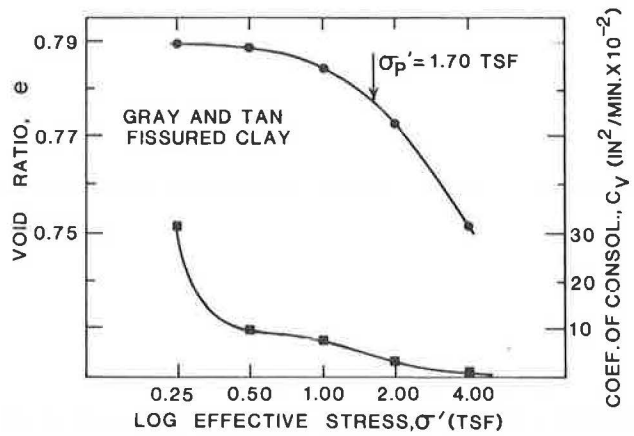

FIGURE 3 Consolidation results from Anderson Street sample.

first, the maximum one-directional displacement that the direct shear machine will allow per run (0.25 in./run) and, second, the need for adequate horizontal displacement to reach the postpeak shear strengths under consideration in this study (softened to residual strength range).

The initial estimate of required horizontal displacement was based on data from other studies involving direct and rotational shear tests (*6, 10–14*). This body of information suggests that 0.5 in. per normal stress increment is an ample quantity to reach postpeak strengths. To evaluate this contention, one complete residual envelope was developed using a total of 1.0 in. displacement for each value of effective normal stress. This test showed a relatively small reduction in the residual strength resulting from the increase of accumulated horizontal displacement from 0.5 to 1.0 in. Therefore, 0.5 in. of horizontal displacement per normal stress increment, requiring two runs at 0.25 in./run, was adopted as standard procedure.

Test Drainage Conditions and Sample Soaking Prior to Testing

Drained shear testing means that no excess pore pressures are generated during shear as a result of free drainage. All tests were conducted "wet"; that is, the specimens being sheared were submerged throughout the duration of the test. This allowed for pore fluid to communicate freely through the upper and lower porous stones with water in the shear box reservoir surrounding the sample.

In order to evaluate the effect of sample soaking before direct shear tests were conducted, samples from both Anderson Street and LaSalle Street were soaked for 1 month and 2 weeks, respectively, under a stress approximately equal to the in situ effective stress. This extended soaking had minimal effects on the shear strength. The probable explanation for this behavior is the high degree of saturation of the samples before the long-term soaking. Without the use of back pressure, the degree of saturation would not be expected to increase substantially. For practical purposes, the standard procedure (i.e., soaking the sample overnight before testing) produced the same results as extensive soaking. This should not be considered standard behavior at other sites where soils may be desiccated due to

exposure or lowering of the groundwater and are subsequently at a low degree of saturation at the time of sampling.

Number of Specimens

Another consideration in the direct shear test procedure is the number of soil specimens needed to develop the residual shear strength envelope. The two options available are to use one specimen per normal stress increment or the same specimen for all the normal stress increments necessary to complete a strength envelope. On the basis of recommendations by Townsend and Gilbert (*15*), a single specimen is tested under various normal stresses; this is referred to as the multistage approach. This issue was investigated by conducting two test series on specimens from the same block sample. The results indicated that the use of three specimens in the second test series generated shear strength parameters considerably higher than those obtained by using one specimen under various normal stresses and significantly higher strength than that predicted based on data accumulated from other studies on similar soil. This behavior suggests less particle alignment on the failure surface as a result of the smaller accumulated displacement during the three single-shear tests as compared with the multistage tests. It was concluded that the one-specimen multistage approach to testing was an appropriate procedure, and it was adopted as standard for the remainder of the laboratory testing program.

Field Sampling

In order to secure samples for use in the laboratory direct shear tests, exploratory trenches were excavated by NCDOT personnel at both the Anderson Street and LaSalle Street sites in June 1983. Block samples were taken from specific locations within the trenches that had previously been identified as potential shear planes significant in the stability of the proposed cut slopes. Bedding planes, seams, and fissures were measured for dip and direction and recorded by NCDOT geologists. All the block samples were wrapped in plastic and tape, numbered, and returned to North Carolina State University for moisture-controlled storage.

One month before the trench sampling at the LaSalle Street and Anderson Street sites, samples were taken from the Railroad Underpass/US-64 location, also to be used in the direct shear testing. Two samples were secured by pushing 4-in.-diameter metal tubes into the soil on the face of the previously failed slope close to the existing bridge structure (i.e., the US-64 overpass). The tubes were sealed and returned to North Carolina State University for storage.

In order to catalogue the soil samples in storage, a system of numbering was developed that identified the samples according to the following criteria:

1. Type of sample—block sample (SBS) or Shelby tube sample (STS);
2. Number of sample and site name; designated sample number and letter *a*, *b*, or *r* assigned to the LaSalle Street, Anderson Street, and Railroad Underpass/US-64 samples, respectively; and
3. Location of the site where the sample was taken, for example, EB1-A (End Bent 1-A of the proposed bridge structure at the site).

These three parts are combined to give a catalogue number for each sample (example: SBS-3a, EB1-A).

Laboratory Sample Preparation Technique

Because of the difference in the nature of soils found at the three sites under investigation, different sampling and specimen preparation methods were adopted that best suited each of the materials. In all the laboratory sampling and preparation techniques employed, the objective was to obtain specimens that most accurately represented the in situ conditions of the site under question and could be aligned and positioned most easily in the direct shear device so that the plane of shearing (i.e., plane of separation of the shear box) corresponded to the plane of weakness thought most susceptible to failure in the field (i.e., fissures or seams).

For the LaSalle Street soils, the stiff siltstone with relatively soft thin clay seams was prepared for testing according to the following procedure. Block samples with the most distinct seams and adequate sample dimensions were cut into blocks 2.6 in. wide by 2.0 in. high with a hand saw. The clay seams in the brown siltstone were kept as horizontal as possible throughout the sample cutting and trimming so that the samples, once placed in the direct shear box, would fail along these seams as much as possible. These samples were trimmed further to a 2.5-in. diameter by using a motorized sampling device made by Soiltest Company and a razor blade mounted on a plexiglass strip as a trimming tool (Figure 4, *left*).

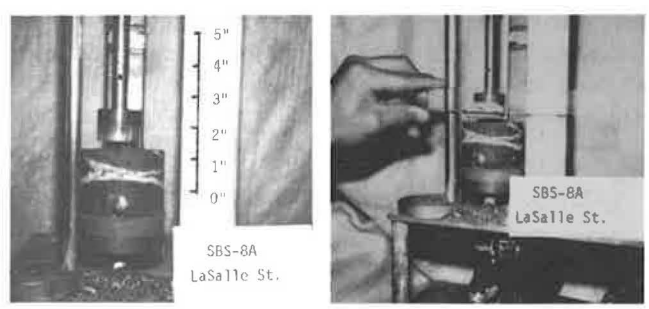

FIGURE 4 Laboratory sample preparation, LaSalle Street sample.

For the Anderson Street soils, the fissured clay was prepared for testing by using the following procedure. Block samples with the most distinct fissures and adequate sample dimensions were sawed into blocks by using a sample cutting box that was designed to accommodate a variety of sample sizes. Block samples were confined by plastic wrap and tape binding before placement in the sample cutting box. The samples were oriented in the box so that the planes of fissure were horizontal and the samples would be sheared across the apparent planes of weakness. Because the planes of fissure were undulating and discontinuous, it was often difficult or impossible to determine

FIGURE 5 Laboratory sample preparation, Anderson Street sample.

the exact location and orientation of the fissures within the sample. Figure 5 (*left*) shows the sample cutting box; a portion of the original field sample SBS-10b and a sawed block of the same material are shown in the left and right sides of the photograph, respectively. Once the soil had been sawed into right-angle blocks, a consolidation ring with a 2.5-in. inside diameter was pressed into each block by using a modified drill press configuration. To maintain the integrity of the soil blocks and minimize soil moisture loss during the filling of the consolidation rings, the sample was bound with plastic and heavy-duty tape. After being filled with soil, the ring was recovered from the soil block and trimmed to the required 0.75-in.-high specimen. Figure 5 (*right*) shows the ring-pressure device and consolidation ring before they are pressed into the soil block.

The Railroad Underpass/US-64 soil was very similar to the Anderson Street material. The field samples were contained in small 4-in.-diameter metal tubes, which acted as lateral confinement and therefore facilitated the pressure of the consolidation rings directly into the tubes to recover a sample. No preparation was needed to trim the field sample and therefore the entire procedure of obtaining the direct shear specimen was completed in one step. Once the soil-filled consolidation ring was removed from the tube, it was trimmed to the same height as the other sample. In general, for the laboratory, field block samples are the best with which to work. Several Shelby tube samples were taken at the Anderson Street site but were not employed as a source of specimens in the direct shear testing. Field block samples are considered to be more useful than Shelby tube samples for the following reasons:

1. More soil sample is available, and therefore there is a greater degree of freedom in selecting the part of the soil mass from which the testing specimen can be taken; and
2. There is less disturbance to the soil sample (i.e., the sample must be extruded from the Shelby tube to examine it, whereas the block sample can simply be unwrapped, examined, and rewrapped with little or no disturbance).

EXPERIMENTAL RESULTS

A series of eight direct shear tests was conducted on samples from the Railroad Underpass/US-64 and the Anderson Street and LaSalle Street locations. The goal of the test program was to describe the postpeak strength parameters of materials within zones of the soil stratigraphy that had previously been identified as potential shear zones significant in slope stability analyses of cut slopes at these sites. In addition, tests of Atterberg limits and x-ray diffraction analyses were conducted to evaluate the plasticity characteristics and mineral composition, respectively, of the soils from the three sites.

Soil Description and Classification

The results of the Atterberg limits tests, natural water content determinations, liquidity index values, and the percent passing the No. 200 sieve for the soils at the three sites are compiled in Table 1. The Atterberg limits of these soils were determined by two methods:

1. The standard liquid limit and plastic limit test methods according to ASTM D423-66 and D424-59 and

TABLE 1 SOIL CLASSIFICATION DATA

Sample No.	Location	Water Content, w_n (%)	Liquid Limit, W_L (%)		Plastic Limit, W_{PL} (%)		Plasticity Index, PI (%)		Liquidity Index, LI (%)		Percent Passing No. 200 Sieve	USCS Symbol
			CP	SAT	CP	SAT	CP	SAT	CP	SAT		
1r	Railroad Underpass/US-64	31.0	49.5	50.0	20.2	19.5	29.3	30.5	0.37	0.38	82.0	CH
2r	Railroad Underpass/US-64	30.6	—	55.0	—	22.0	—	33.0	0.26	—	78.0	CH
13b	Anderson Street	33.6	63.0	57.5	20.7	27.7	42.3	30.0	0.30	0.20	91.6	CH
10b	Anderson Street	26.0	55.0	52.2	20.0	22.2	35.0	30.0	0.17	0.13	88.0	CH
10b	Anderson Street	32.0	56.4	52.8	21.4	22.0	35.0	30.8	0.30	0.32	88.0	CH
14b	Anderson Street	32.3	57.5	57.0	—	24.0	—	33.0	—	0.25	85.1	CH
2a	LaSalle Street (parent material)	16.8	34.0	—	16.4	—	17.6	—	0.02	—	76.5	CL
1a	LaSalle Street (parent material)	17.4	37.0	32.0	20.6	21.0	16.4	11.0	–0.20	–0.33	76.0	CL
2a	LaSalle Street (thin discontinuous seam material)	20.9	40.9	39.0	13.7	19.6	27.2	19.4	0.26	0.07	82.0	CL
4a	LaSalle Street (thick seam material)	32.1	56.5	55.1	20.0	22.1	36.5	33.0	0.33	0.30	88.0	CH

Note: CP = cone penetrometer determination; SAT = standard Atterberg test determination; USCS = Unified Soil Classification System.

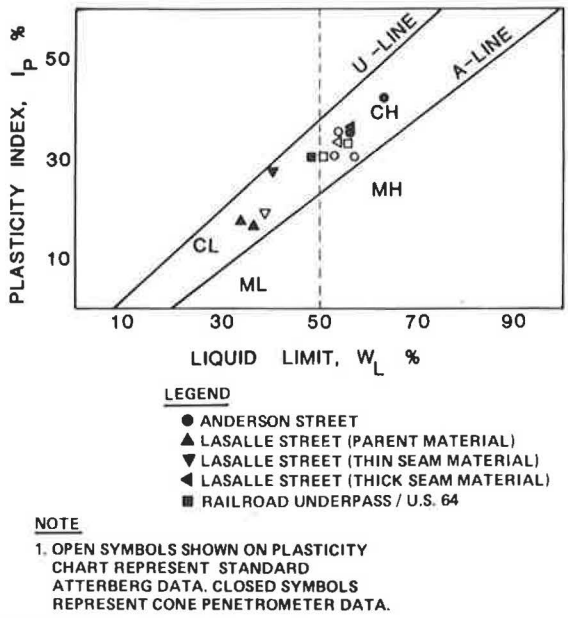

FIGURE 6 Plasticity chart.

2. The cone penetrometer liquid limit determination (British Standard No. 1377) and the plastic limit determination using two cones of different mass from a study by Wood and Wroth (*16*). Figure 6 shows the Casagrande plasticity chart with the results from all Atterberg limits test. The open symbols represent standard ASTM results and the solid symbols represent cone penetrometer results.

Railroad Underpass/US-64

The soil used in the direct shear tests was a gray and tan fissured clay with some fine sand. The term "fissured" is used here in the same way that Terzaghi used it to define a stiff, fissured clay (*17*, pp.161–165):

> When it is dropped, a big chunk of the clay breaks into polyhedric, angular and subangular fragments with dull or shiny surfaces. The diameter of the fragments may range between less than one centimeter and more than twenty centimeters.

Anderson Street

The soil used in the direct shear tests was highly plastic light gray and tan streaked fissured clay with traces of fine sand and plant roots in the fissures. The soil is classified a CH material according to the Unified Soil Classification System (USCS). This fissured clay broke into small chunks and blocks if not handled carefully. The fracture surfaces examined after breakage were distinct and appeared shiny and undulating.

LaSalle Street

From the LaSalle Street site, two types of soil were tested in direct shear. The first type of soil was a brown micaceous clayey siltstone of slight to medium plasticity (referred to in this paper as the parent material and classified as a CL material according to the USCS). The second type of soil was a medium to highly plastic light gray fissured clay that was interbedded in the siltstone in a range of thicknesses from ⅛ to 1 in. The "thin" seams (i.e., up to ⅛ in.) are classified CH according to the USCS. All seam material contained some plant roots. The water content and plasticity characteristics and the occurrence of fissures in these seams were found to be a function of the seam thickness. Field investigations by NCDOT indicated that the clay may be as thick as 1 ft.

Mineralogy Analysis

X-ray diffraction analyses were used in order to determine the type and relative quantity of clay minerals present in representative samples of soils from each of the sites. These results are given as supplemental information to enhance the data base of soil characteristics. The testing utilized modified procedures for dispersion of soil minerals and conventional x-ray diffraction analysis based on procedures by Jackson (*18*). The tests were conducted under the supervision of Sterling Weed of North Carolina State University.

The following is a summary of the clay mineral content of the soils from the three sites under study:

LaSalle Street
Parent material, brown micaceous clayey siltstone; mica (weathered) ≈ vermiculite >> montmorillonite > halloysite.
Thin gray discontinuous clay seam material; montmorillonite > halloysite > mica (weathered).
Thick gray fissured clay seam material; montmorillonite >> halloysite ≈ mica (weathered).
Anderson Street
Gray with tan fissured clay; montmorillonite >> halloysite ≈ mica (weathered).
Railroad Underpass/US-64
Gray and tan fissured clay; montmorillonite >> halloysite > mica (weathered).

DIRECT SHEAR TESTS RESULTS

The direct shear test program involved eight series of tests that resulted in the development of eight complete sets of strength envelopes (i.e., softened, residual, and lower bound residual). Of these eight series, one was conducted on soil from the Railroad Underpass/US-64 site, three on soil samples from the Anderson Street site, and four on the two soil types from the LaSalle Street site.

Shear Stress Versus Displacement Behavior

The results of the eight series of direct shear tests conducted in this study are presented in plots of shear stress versus displacement (Figure 7) and summarized in Table 2. Each of the eight series is designated by a number from 1 through 8. A list of comments that reference the eight test series and identify the procedures implemented and variables investigated follows:

Test 1: standard procedure; one sample, three normal stress levels, and two runs per increment (i.e., total one-directional displacement = 0.50 in. per increment); chosen standard rate of displacement = 2.2×10^{-3} in./min.

Test 2: three samples, standard procedure (see Test 1); three samples were used to validate the standard technique utilizing one sample to develop a complete envelope.

Test 3: standard procedure with modification of one additional run conducted on the third increment to verify the effect of reducing the rate of displacement to one-third of the standard rate (8.0×10^{-4} in./min).

Test 4: standard procedure with modification of allowing the sample to soak for 1 month before testing positioned and aligned under a stress of 0.25 tsf (i.e., approximately the existing in situ overburden).

Test 5: standard procedure with sample aligned and posi-

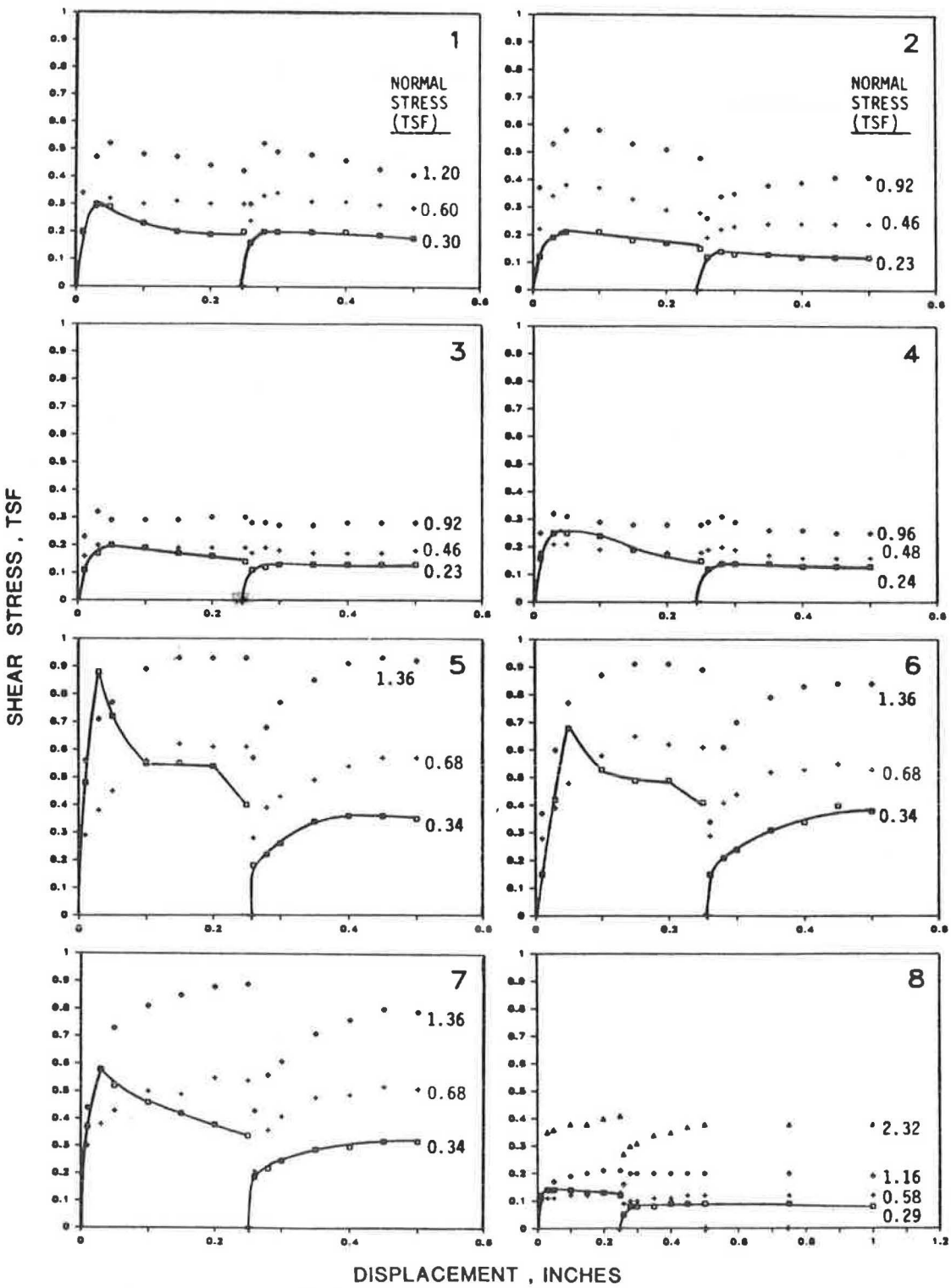

FIGURE 7 Shear stress versus displacement data: Series 1–8.

TABLE 2 SUMMARY OF DIRECT SHEAR TEST RESULTS

Test No.	Sample No.	Sample Location and Description	Shear Strength: Softened ϕ'_s (degrees)	Softened C'_s (tsf)	Residual ϕ'_r (degrees)	Residual C'_r (tsf)	Lower-Bound Residual ϕ'_{rl} (degrees)	Lower-Bound Residual C'_{rl} (tsf)
1	lr	Railroad Underpass/US-64: gray and tan fissured clay with some fine sand	19.8	0.10	14.2	0.10	19.0	0.00
2[a]	SBS-14b(1), 10b(2), 10b(3), EB1-A	Anderson Street: gray and tan fissured clay with trace fine sand and roots	25.3	0.05	22.4	0.04	24.0	0.00
3	SBS-10b(4), EB1-A	Anderson Street: gray and tan fissured clay with trace fine sand and roots	12.1	0.10	12.0	0.07	16.9	0.00
4	SBS-10b(1), EB1-A	Anderson Street: gray and tan fissured clay with trace fine sand and roots	10.3	0.10	10.1	0.08	14.4	0.00
5	SBS-1a(1), EB1-A	LaSalle Street: brown micaceous siltstone with thin gray discontinuous clay seams and trace roots in the seams	28.4	0.22	28.1	0.19	33.8	0.00
6	SBS-1a(2), EB1-A	LaSalle Street: brown micaceous siltstone with thin gray discontinuous clay seams and trace roots in the seams	24.8	0.27	23.8	0.23	31.4	0.00
7	SBS-3a(1), EB1-A	LaSalle Street: brown micaceous siltstone with thin gray continuous clay seam and trace roots in the seams	28.1	0.17	25.3	0.16	30.5	0.00
8	SBS-4a(2), EB1-A	LaSalle Street: light gray clay, thick seam material with trace roots	8.4	0.06	8.4	0.03	9.0	0.00

[a]Peak strength: ϕ'_p = 28.2 degrees, C'_p = 0.10 tsf (peak strength as determined using one sample per normal stress increment).

tioned to force failure to occur through the majority of the thin discontinuous seams.

Test 6: standard procedure (sample alignment as in Test 5) with modification of allowing the sample to soak for 2 weeks before testing positioned and aligned under a stress of 0.25 tsf (i.e., approximately the existing in situ overburden).

Test 7: standard procedure with sample aligned and positioned to force failure to occur on the thin silty clay seam.

Test 8: one sample, four normal stress levels, and four runs per increment; the fourth increment was used to define the strength envelope at a relatively large normal stress; the total displacement of 1 in. per increment was incorporated to show the effect of doubling the standard amount of displacement.

For the fissured clays found at both the Anderson Street and Railroad Underpass/US-64 sites, a peak strength value is thought to be an unconservative estimate of the available in situ shear strength with regard to stability of cut slopes in these soils. This postpeak strength phenomenon can be explained by the progressive failure theory, which states that the peak strength is passed at any one point along a failure surface within a cut slope because of fissures or discontinuities that act as stress concentrators forcing that point to pass the peak strength with a given amount of displacement. Once the peak strength has passed at one point along the failure surface, the stress is shifted to another point, causing it to pass the peak, and so on. In this way a progressive failure can be initiated and the strength along the entire length (or majority) of a slip surface will decrease as a function of displacement to a lower strength range bounded by the softened and residual strengths.

Figure 8 shows the idealized response of a stiff, fissured clay during a drained direct shear test (*19*), which illustrates the reduction in shear strength as a function of displacement that would occur at one point and eventually progress to a portion

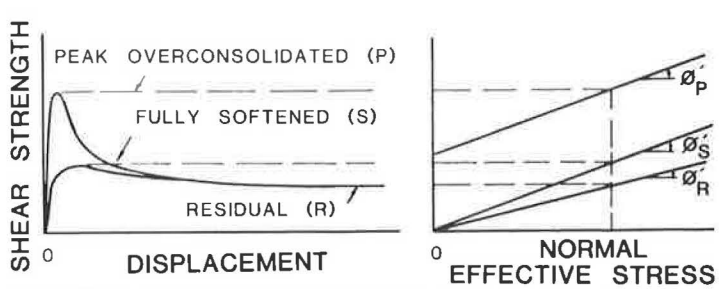

FIGURE 8 Idealized stress-displacement response of stiff fissured clay (*19*).

or the entire length of a slip surface. The curves of stress versus displacement in Figure 7 from this study are generally in agreement with the idealized response shown in Figure 8.

The softened strength and zero cohesion residual strength are generally recognized as the upper and lower boundaries for the range of in situ strength mobilized for slope failures in fissured clays. The softened strength is defined as the peak stress response for a "remolded" normally consolidated clay (i.e., critical-state condition) and is shown on the idealized stress-displacement curve of Figure 8. The residual strength value also indicated in Figure 8 is defined as that value of stress at which further accumulated displacement will not result in a lowering of strength (i.e., steady-state condition).

In this study, displacements of 0.5 in. per stress increment or an accumulated one-directional displacement of 1.5 in. for a single sample (multistage test) were used to mobilize a postpeak strength that appeared to be a relatively steady-state condition. The stress value at this condition is taken as the residual strength. Numerous studies have been conducted to date that examine techniques for determining the residual strength of clay and the amount of displacement required to achieve that minimum strength. Two of these studies, mentioned previously [LaGatta (*14*) and Lupini et al. (*6*)], specifically investigated residual strength for a variety of clay soils including Pepper shale, Cucaracha shale, and London clay. Results from these studies indicated that displacement of approximately 0.5 in. per normal stress increment would mobilize the postpeak strength, but that as much as 35 in. of displacement was needed to mobilize the true residual strength for some soils. An analysis of the data from these studies indicated that the additional accumulated displacement between 0.5 in. and that corresponding to the true residual value resulted in a reduction in shear strength ranging from approximately 6 percent to a maximum reduction of approximately 40 percent.

The objective of this experimental testing program on selected fissured clays of the Triassic Basin was not to determine the absolute lower bound shear strength of these soils, as was the case in the two studies mentioned earlier. The primary objective was to develop a reliable testing technique using the revisable direct shear device in order to establish a range of postpeak strengths thought to control instability of cut slopes in the type of soils tested, in particular, (*a*) first-time slides and the design of cut slopes in previously unfailed materials and (*b*) those somewhat lower strengths that govern the stability of a previously failed soil mass. The applicability of these results is evaluated in the case study analysis of a previously documented slope failure in the companion paper by Putrich et al. in this Record.

Shear Strength Envelopes

The shear strength envelopes (shear stress versus normal stress plots) for the eight series of direct shear tests are presented in Figure 9 and are designated 1 through 8, corresponding to the stress displacement curves in Figure 7. The softened and residual strength envelopes shown for Tests 1 and 3 through 8 in Figure 9 are determined from multistage single-sample tests. Test 2 utilized three samples, one for each load increment. This allowed for the determination of the peak strength envelope in addition to the softened and residual envelopes as shown. The range of normal stresses investigated was chosen to correspond to the in situ stress conditions anticipated on the failure planes in the cut slopes. A dashed line has been used to indicate that the shear strengths below the lowest stress increment are not known and that the dashed portion of these lines is a continuation of the slope of the strength envelopes for the stress range tested to the vertical intercept. An apparent effective cohesion (c') is given for the purpose of defining the linear equation of the envelope.

In utilizing three samples of the Anderson Street fissured clay in Test 2, comparatively higher shear strengths resulted than those in Test 3 where only one sample was used in the multistage approach. One may conclude that the accumulated displacement in the multistage single-sample test significantly reduces both the softened and residual shear strength values compared with shear strengths determined with the minimal displacement developed in the three-sample approach. In light of the fact that the displacement used in these eight series of tests was relatively small compared with the studies by LaGatta and Lupini et al., cited earlier, the lower-bound residual envelope has been introduced. A dashed line is used to indicate that this is a theoretical envelope, constructed by passing a straight line through the original, and the residual strength corresponds to the largest normal stress applied. This is a best estimate of the "practical" lower limit of shear strength of these materials. In theory, a lower shear strength could be achieved if the amount of accumulated displacement was increased by a factor of, say, 10 or more, but as will be shown by Putrich et al., this "practical" lower-bound residual strength, although greater than the "true" residual, significantly underestimates the lower-bound in situ shear strength for a landslide that occurred in these soils. Therefore, for the purpose of this study, the lower-bound residual as shown in Tests 1 through 8, Figure 9, is considered to be a reasonable first approach to describing a useful lower range of shear strength reflecting the behavior of these soils.

One of the focuses of the testing program was to evaluate the strength of somewhat continuous clay seams varying from ⅛ in. to 1 ft thick interbedded in a stiff siltstone found at the LaSalle Street site. Triassic sedimentary deposits are characterized by inclined bedding planes that dip from 5 to 45 degrees and a northeast-southwest trending strike. There had been several wedge-type failures in similar soil deposits within the Triassic Basin before this study for which stability analyses based on both undrained and drained peak strengths were predicted to be stable. These slopes were thought to fail along the sited seams of fissured clay. Therefore, there was an interest in evaluating the strength of the siltstone (referred to as parent material) and the highly plastic fissured clay seams individually. In comparing the results of Test 8 (2-in. clay seam: ϕ_r = 8.4 degrees) with those of Tests 5, 6, and 7 (parent material with the clay seams: range of ϕ_r = 23.8 to 28.1 degrees) a significant difference in shear strength is noted (as much as 300 percent greater residual friction angle for the intact parent material). It is apparent that significant differences in slope geometry would result from a design based on the results of Tests 5, 6, and 7 versus Test 8. This significantly lower shear strength would be considered in cut slope design if there was

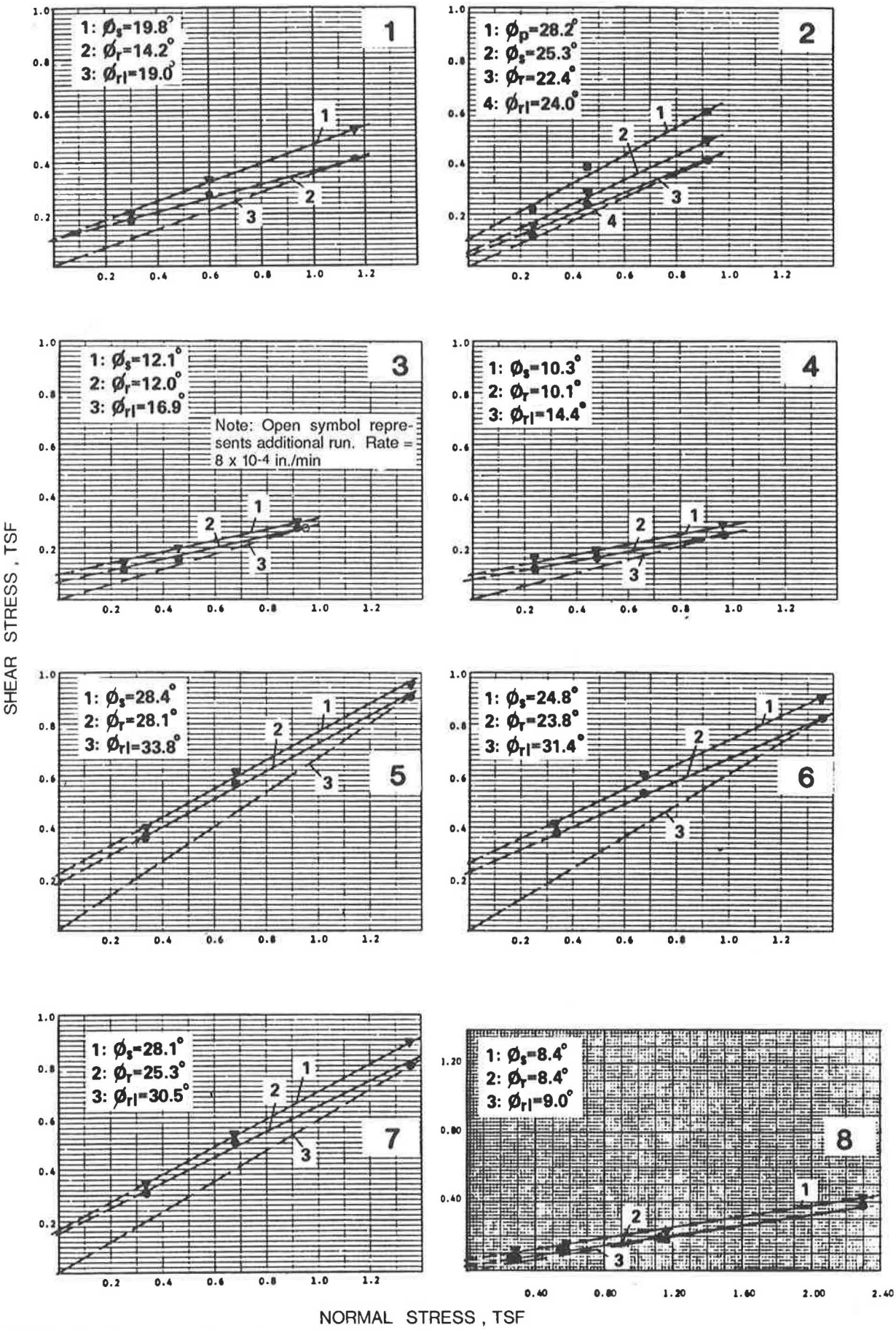

FIGURE 9 Shear strength envelopes: Series 1–8.

sufficient evidence (e.g., soil boring data, test pit excavation data, information from documented nearby slope failures) indicating that the weak seams and zones were extensive and that their orientation with respect to the planned excavation would lead to a slope failure. One can conclude that the type of sample taken from the field and the selectiveness with which the samples are tested can have a significant effect on the shear strength results. In the case of this study, special efforts were made to recover and test soil samples of the somewhat continuous interbedded clay seams and the more prevalent siltstone parent material from the LaSalle Street site.

Evaluation of Test Results

In order to evaluate the results of these direct shear tests, a summary of empirical relationships between the drained residual friction angle and the plasticity index based on the results compiled from a number of studies involving the measurement of the residual strength of soils [Lupini et al. (*6*)] is shown in Figure 10. One may conclude from these results, despite the wide range of curves, that the residual friction angle decreases with increasing plasticity index. The approximate upper and lower boundaries of the data are indicated by heavy dashed lines.

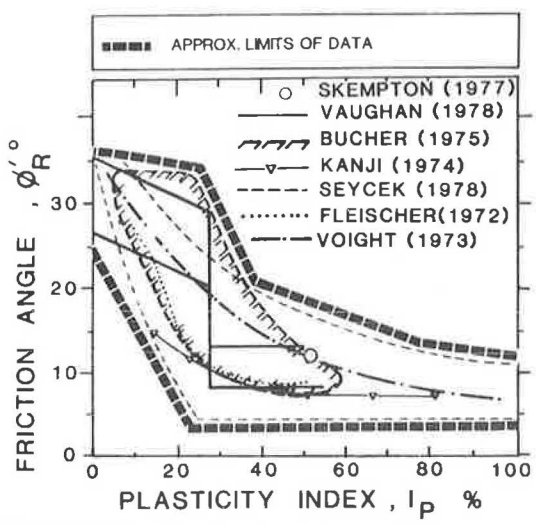

FIGURE 10 Relationship between friction angle and plasticity index (*6*).

The residual and lower-bound friction angles determined in this study are presented in Figure 11 with respect to the approximate upper and lower bounds taken from Figure 10 as a comparison. Despite the variation in the empirical relations presented in Figure 11, the test results follow the general trend of decreasing friction angle with increasing plasticity index, plot within the given limits of the range of expected results, and are in general agreement with this larger body of data.

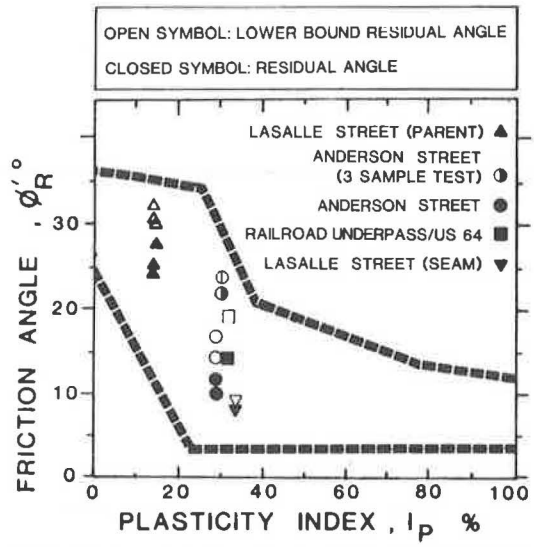

FIGURE 11 Relationship between friction angle and plasticity index including test results: Series 1–8 (*6*).

CONCLUSIONS

On the basis of the results of the direct shear tests performed in this study and the literature cited in this paper, the following conclusions are advanced:

1. The results of direct shear tests conducted on the fissured overconsolidated residual soils investigated showed a reduction in strength with accumulated displacement beyond the peak strength.
2. The amount of horizontal displacement used in the direct shear testing discussed was sufficient to mobilize postpeak shear strengths, but was probably not a sufficient amount to mobilize the "true" residual (i.e., absolute lower-bound) shear strength, on the basis of test results cited from rotational shear tests on similar soils. There is evidence to suggest, however, that the range of postpeak strengths examined in this study approximates in situ soil strengths mobilized in a documented slope failure in the same soils, and that the "true" residual strength values would greatly underestimate the in situ strength. (See paper by Putrich et al. in this Record for slide analysis of referenced slope failure.)
3. The data generated in this study were found to be in general agreement with the trend of decreasing residual friction angle with increasing plasticity index reported in the literature.
4. The type of soil samples taken from the field and the selectiveness with which the samples are tested can have a significant effect on laboratory results and the resulting slope design and stability of that cut slope. This is especially true for soil deposits with weak seams that follow the inclination of the soil bedding planes and are oriented so that planes dip into a proposed excavation. The importance of the selectiveness in sampling and testing soils is illustrated by a series of tests conducted on an intact siltstone with thin discontinuous clay seams (approximately ⅛ in.), which resulted in a residual

friction angle 300 percent greater than the same test that forced failure through a 2-in.-thick clay seam in the same siltstone material. The direct shear device allows the flexibility to evaluate the strength of seams and weak zones in soils by forcing the failure surface to pass through a specific zone of interest within a soil sample.

5. Block samples were found to be more useful than tube samples for obtaining test specimens from material containing seams or thin weakness planes because they allow for a greater degree of freedom in selecting the part of the soil mass to be tested and they are generally less disturbed by the sampling and extruding processes.

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