CHAPTER 5

SOIL PARAMETERS

5.1 INTRODUCTION

The focus of the laboratory testing program was to develop soil parameters that will be used to perform analyses of the full-scale lateral load tests. The laboratory tests included soil classification, unit weight, strength (UU, CU, and CD triaxial tests), and consolidation.

Tests were performed on soil samples obtained from the field test facility (natural soils) and on samples of imported materials that were used as backfill around the piles, pile caps, and bulkhead. Section 5.2 describes the results of tests on the natural soils, which consist of clayey and silty sands. Test results for the two backfill soils, crusher run gravel and New castle sand, are described in Section 5.3.

5.2 NATURAL SOILS

5.2.1 Soil Description

Test results for samples obtained at different depths are described according to the project benchmark, which was established at an arbitrary elevation of 100.00 feet. The actual elevation of the benchmark is unknown, but judging from the USGS Radford North quadrangle map, it is approximately 1700 feet above mean sea level. The ground surface in the area of the test foundations was relatively flat. The average surface elevation, after stripping the topsoil, was 97.5 ft.

The soil conditions at the site, which covers an area about 100 feet by 50 feet, are quite uniform. The soil profile revealed by six borings and two test pits was as follows:

Elevation (ft)	Soil Description
97.5 to 94.0	Brown silty sand and sandy lean clay with fine sands and frequent small roots.
94.0 to 88.5	Dark brown, moist sandy lean clay with occasional gravel.
88.5 to 84.5	Brown moist sandy silt with lenses of silty sand.
84.5 to 80.5	Brown, moist sandy silt and silty sand.
80.5 to 77.5	Light brown sandy lean clay and sandy silt with trace of gravel.

In general, the soils at the site consist of sandy clay, sandy silt, and silty sand with thin layers of gravel. In accordance with the Unified Soil Classification System (ASTM D2487), the soils are classified as ML, CL, SC, and CL-ML. Chapter 4 contains a description of the subsurface conditions encountered during the in situ investigation.

5.2.2 Index Properties and Unit Weights

Index tests were performed to provide data necessary for classifying the soil and for developing correlations with various soil parameters. The percentage of soil passing the number 200 sieve, Atterberg limits, and natural moisture contents were determined in general accordance with ASTM D1140, D4318, and D2216. Summaries of results from these tests are shown in Table 5.1 and Figure 5.1.

Moist unit weights of the natural soil were estimated from triaxial and consolidation samples, and sand cone tests that were performed in the near-surface soils in accordance with (ASTM D 1556). Most of the values of unit weight fall between 115 and 125 pcf as shown in Figure 5.1 (d).

5.2.3 Consolidation Tests

Three one-dimensional consolidation tests were performed in general accordance with ASTM D2435, on specimens trimmed from undisturbed samples. Samples BH-4, ST-4, and BH-4 ST-5 were trimmed conventionally from undisturbed soil samples to represent vertical consolidation properties of the soil. Sample BH-4, SST-3 was trimmed such that it represented horizontal consolidation properties of the soil. The test specimens were loaded at twenty-four hour intervals using a load-increment ratio of one, and they were unloaded at twenty-four hour intervals using a load-increment ratio of four. Stress-strain curves from these tests are included in the Appendix D (Figure D.1), and the test results are summarized in Table 5.2.

5.2.4 Strength Tests

A total of 31 triaxial tests were performed on specimens trimmed from undisturbed Shelby tube samples and block samples. Of the 31 triaxial tests, 22 were UU (Unconsolidated-Undrained), 3 were CD (Consolidated-Drained), 3 were CU (Consolidated-Undrained), and 3 were staged CU tests. The UU specimens were tested at their natural moisture content. The CD and CU specimens were saturated by applying back pressure. The CD, CU and 21 of the UU specimens were carved from Shelby tube samples. These specimens were all trimmed to a nominal diameter of 1.4 inches and a nominal height of 3 inches.

Ten of the UU specimens were carved from block samples. Four of these were trimmed vertically and the remaining 6 were trimmed horizontally. Seven of the specimens were trimmed to a nominal diameter of 1.4 inches and a nominal height of 3 inches. The other 3 specimens were trimmed vertically to a nominal diameter of 2.8 inches and a nominal height of 5.6 inches.

Total stress shear strength parameters. A summary of the UU triaxial results is presented in Table 5.3. Plots of p versus q at failure are shown in Figure 5.2(a) for tests performed on samples obtained at 4 different elevations. The same results are re-plotted in Figure 5.2(b) as a function of q at failure and cell pressure. The undrained strength parameters were estimated using the curves and transformation equations shown in Figure 5.2(b), with emphasis on values measured at cell pressures less than 7 psi. The total stress parameters for the natural soils, determined in this manner, are as follows:

Elevation (ft)	ϕ (deg)	c (psi)
96.0	38	7.0
92.8	35	6.0
90.5	23	4.7
88.2	28	4.4

There was no discernable difference in undrained behavior between the vertically and horizontally trimmed specimens, or between the small diameter and large diameter specimens.

Values of ε_{50}, the strain required to mobilize 50 % of the soil strength, were estimated from the triaxial stress strain curves. The estimated variation of ε_{50} with depth is shown in Figure 5.1(f). The values increase with depth from about 0.005 in the stiff upper crust to 0.025 in the underlying softer soils. These are in good agreement with Reese et al.'s (1997) recommended ε_{50} values of 0.005 for stiff clay and 0.020 for soft clay.

Stress-strain curves for the 10 UU tests performed on block sample specimens are included in Appendix B. The confining pressure for these tests ranged from 0 to 4 psi. Values of the initial tangent modulus, E_i, were estimated by transforming the stress-strain data using the hyperbolic formulation described by Duncan and Chang (1970). The transformation procedure is shown in Figures D.2, D.3, and D.4 for the natural soils. The estimated values of E_i are shown in Figure 5.3(a). E_i does not vary significantly over the range of confining pressures that were used. Consequently, an average value of E_i = 6,200 psi was selected for use in the analyses.

Effective stress parameters. Summaries of the CU and CD test results are presented in Tables 5.4 and 5.5. Plots of effective stress, p′ versus q, at failure for the CU and CD tests are shown in Figure 5.4. No variation of effective strength parameters with depth was apparent from the data. Values of ϕ' and c' determined from these tests are as follows:

	ϕ' (deg)	c' (psi)
lower bound	27	0
best fit	32	0
upper bound	32	4.9

5.3 BACKFILL SOILS

5.3.1 Soil Description and Index Properties

Two soil types were used as backfill in the lateral load tests: New Castle sand and crusher run gravel. These materials were selected because they are representative of the types of backfill materials often used for pile caps, footings, and other buried structures.

New Castle sand. New Castle sand is a relatively clean, fine sand consisting predominantly of subangular grains of quartz. Plots of 2 grain size distribution curves are shown in Figure D.5. About 70 % of the sand passes the No. 40 sieve and less than 1 % passes the No. 200 sieve. The coefficient of uniformity is 2.0, the coefficient of curvature is 2.8, and the Unified Classification is SP. The specific gravity of solids, determined in general accordance with ASTM D854, is 2.65. The maximum and minimum densities determined in general accordance with ASTM D4253 and ASTM D4254 are 105 and 87.3 pcf, respectively.

Crusher run gravel. Crusher run gravel was obtained from the Sisson and Ryan Stone Quarry, located in Shawsville, Virginia. The material is produced by processing and screening quartz and limestone rock to produce a well-graded mixture containing angular to subangular grains that range in size from ¾-inch gravel to silt-size particles. The gravel is produced to meet the requirements of VDOT Road and Bridge Specification Section 205, Crusher Run Aggregate. The material obtained for this project also meets the more stringent gradation requirements of VDOT Road and Bridge Specification Section 208, 21B-Subbase and Base Material.

Plots of 4 grain size distribution curves are shown in Figure D.6. Approximately 40 to 50 % of the material passes the No. 4 sieve, 10 to 20 % passes the No. 40 sieve, and 5 to 10 % passes the No. 200 sieve. The soil passing the No. 200 sieve classifies as nonplastic silt, ML. The coefficient of uniformity is 23, the coefficient of curvature is 2.8, and the Unified Classification for the crusher run aggregate ranges between a GW-GM and a SW-SM. This material is referred to as gravel or crusher run gravel in this report.

5.3.2 Standard Density Relationships

New Castle sand. Moisture-density relationships were determined for the New Castle sand using the modified Proctor procedure (ASTM D1557). The maximum dry unit weight was found to be 107 pcf at an optimum water content of 12 %. The maximum and minimum densities determined in general accordance with ASTM D4253 and ASTM D4254 are 105 and 87.3 pcf, respectively.

Crusher run gravel. CD triaxial tests were performed using crusher run gravel that was scalped on the ½-inch sieve size. The grain size distribution curve for the scalped gravel is shown in Figure D.6. Density tests were performed on unscalped and scalped samples to provide a means of correlating field (unscalped) densities with lab (scalped) densities, as shown in Figure 5.5.

The maximum dry densities were determined using the wet method, Method 1B. The dry method, Method 1A, did not yield realistic results because of bulking and segregation of the material during placement into the mold. The minimum dry densities were determined by pouring soil into the mold using a hand scoop (Method A) and by filling the mold by extracting a soil filled tube (Method B). The results were essentially the same, and an average value was used. Based on these results, Figure 5.5 was developed to correlate density values measured in the field on unscalped material with density values measured in the lab on scalped material. The line shown in Figure 5.5 can be represented by the following equation:

$$Drs = 1.053Dru - 6.4 \qquad \text{Equation 5.1}$$

where Drs is the relative density of the scalped material, and Dru is the relative density of the unscalped material. The differences between the two becomes insignificant at relative densities greater than 60 %. For example, a field relative density measurement of 65 % (Dru = 65 %) corresponds to a lab scalped value of Drs = 62 %.

Moisture-density relationships were determined for the crusher run gravel using the Modified Proctor (ASTM D1557) and the Standard Proctor (ASTM D698) methods. The results are as follows:

Modified Proctor	Unscalped	Scalped
maximum dry density	147.4 pcf	146.1 pcf
optimum water content	4.9 %	5.1 %
Standard Proctor		
maximum dry density	135.7	-
optimum water content	7.6	-

5.3.3 In-Place Densities

Excavated zones around the pile caps and single piles were backfilled using two methods. The first method involved placing New Castle sand into the excavation in a loose condition. This was achieved by end-dumping and shoveling dry sand into the excavation with no additional compaction effort. The drop height was maintained at a constant level during sand placement.

The second method involved placing backfill in 8-inch-thick lifts and compacting it with a Wacker "jumping jack" compactor. This method was used to achieve a dense backfill condition for both the New Castle sand and the crusher run gravel. The soil water content was maintained near its optimum water content during placement.

Because of intermittent rains during construction, the natural soil at the bottom of the excavations was often wet, and medium to soft in consistency. A dryer and thicker lift of backfill (10 to 12 inches deep) was placed on the excavated surface to "bridge" over soft and wet soils. Consequently, the initial lift of backfill was less dense than the backfill in the upper lifts.

Nuclear density gauge and sand cone tests were performed during backfill placement in general accordance with ASTM D3017 and ASTM D1556, respectively. Nuclear density gauge tests were performed after compacting each lift of backfill. Sand cone tests were performed to calibrate the nuclear gage for both backfill materials. The statistical distribution of field density results are shown in Figure D.7. A summary of the average results from the moisture-density tests are shown in Table 5.6. The following values were selected for use in subsequent analyses:

Backfill	γ_m (pcf)	D_r (%)
compacted sand	104	60
uncompacted sand	92	10
compacted gravel	134	55

The densities were reduced from the values shown in Table 5.6, to account for the lower density soil in the bottom of the excavations. These reduced values represent the average density of the backfill in the excavations.

5.3.4 Strength Tests

Shear strength versus relative density relationships for the backfill soils were developed using the results of the CD triaxial tests, which were performed on reconstituted 2.8-inch-diameter specimens at low confining pressures. A suite of tests were performed at relative densities ranging from loose to very dense. Table 5.7 contains a summary of CD test results for New Castle sand samples that were tested at relative densities of 20, 60, and 80 percent. Table 5.8 contains a summary of CD test results for crusher run aggregate samples that were tested at relative densities of 50, 70, and 90 percent. Stress-strain curves for the 19 CD tests are shown in Figures D.8 through D.13.

Test specimens were prepared using the method of undercompaction developed by Ladd (1978). The advantages of this procedure is that it uses the same type of compaction energy that was used in the field, and it provides a means of obtaining consistent and repeatable results, with minimal particle segregation. Specimens are prepared to a target relative density by placing soil in layers, inside a forming jacket, and compacting each layer with a small tamper. The compaction density of each layer is varied linearly from the bottom to the top, with the bottom (first) layer having the lowest density. A nearly uniform density is achieved throughout the specimen because compaction of each succeeding layer further densifies the underlying lower layers, which are compacted initially to densities below the target density.

The results were normalized using the $\phi_o - \Delta\phi$ approach (Duncan et al. 1980) to account for curvature of the failure envelope caused by changes in the level of confining stress. Equation 5.2 is used to determine the friction angle, ϕ':

$$\phi' = \phi_o - \Delta\phi \log\left(\frac{\sigma'_3}{p_a}\right) \qquad \text{Equation 5.2}$$

where ϕ_o is the friction angle at 1 atmosphere confining pressure, $\Delta\phi$ is the change in ϕ' over one log cycle, σ'_3 is the effective confining pressure, and p_a is the atmospheric pressure.

Mathematical expressions were developed for calculating ϕ_o and $\Delta\phi$ based on the relative density of the soil. These expressions were developed using the following procedure:

1. ϕ' was plotted on a semi-log scale as a function of the effective confining pressure, normalized by atmospheric pressure, as shown in Figure 5.6. The solid symbols represent CD test results. The 3 open symbols in Figure 5.6(a) represent the results of CU tests with pore pressure measurements.

2. Straight lines were fit through each set of data points to determine ϕ_o and $\Delta\phi$ values. These values are tabulated in Figure 5.6 for New Castle sand and crusher run gravel.

3. The ϕ_o and $\Delta\phi$ values were plotted as functions of relative density in Figure 5.7. Equations representing best fit straight lines were developed for the data as shown in Figure 5.7(a) for the sand and Figure 5.7(b) for the gravel.

Using these expressions and the relative density values presented in Section 5.3.3, the following estimates of ϕ_o and $\Delta\phi$ were calculated for the backfill materials.

Backfill	D_r (%)	ϕ_o (deg)	$\Delta\phi$ (deg)
compacted sand	60	40.3	7.8
uncompacted sand	10	32.1	4.5
compacted gravel	55	45.0	8.3

Effective stress friction angles were calculated using these values of ϕ_o and $\Delta\phi$, and Equation 5.2. This was done for the backfill soils at depths of 0.75, 1.5, and 3 feet, as shown in Table 5.9. The effective cohesion is zero for the backfill soils.

Values of the initial tangent modulus, E_i, were estimated by transforming the stress-strain data using the hyperbolic formulation described by Duncan and Chang (1970). The transformed stress-strain plots are shown in Figures D.8, D.9, and D.10 for New Castle sand and Figures D.11, D.12, and D.13 for crusher run gravel. E_i values from the transformed stress-strain plots are shown as functions of relative density in Figure 5.3(b) for New castle sand and Figure 5.3(c) for crusher run gravel. Based on these plots, the following values of E_i will be used for the backfill soils:

Backfill	E_i (psi)
compacted sand	9,700
uncompacted sand	5,000
compacted gravel	5,300

5.4 SUMMARY

The natural soils encountered at the Kentland Farms field test site consisted of sandy silt, sandy clay, and silty sand, with thin lenses of gravel. Two types of backfill soils were used: a poorly graded fine sand (New Castle sand) and a well graded silty gravel (crusher run gravel).

A laboratory testing program was developed to measure soil properties and to provide a basis for estimating the values of all the parameters that will be used to perform analyses of the full-scale lateral load tests. The results that will be used in the analyses described in Chapter 7, are summarized below:

- Distributions of ϕ, c, γ_m, and ε_{50} are shown in Figure 5.8, for the natural soils.

- Shear strengths parameters for the backfill soils (ϕ' and γ_m) are summarized in Table 5.9. The effective cohesion is zero for the backfill soils.

- Values of initial tangent modulus for the natural soil and backfill soils are shown in Figure 5.3.

Table 5.1. Summary of index test results on samples of natural soil.

Borehole and sample No.	Elevation (ft)	Finer than No. 200 Sieve[a] (%)	Liquid limit[b] (%)	Plasticity index[c] (%)	USCS[d]	Moist unit weight (pcf)	Natural moisture[e] content (%)	Dry unit weight (pcf)
BH-1, cuttings	67.5 +/-	85.9	Non-viscous	Non-plastic	ML	-	20.6	-
BH-2, SS-1	95.0 – 93.5	59.8	40.5	21.8	CL	-	18.3	-
BH-2, SS-2	92.5 – 91.0	60.6	39.1	17.8	CL	-	22.2	-
BH-2, SS-3	90.0 – 88.5	51.5	34.0	8.6	ML	-	23.9	-
BH-2, SS-4	86.5 – 85.0	63.6	37.0	9.1	ML	-	27.8	-
BH-2, SS-5	82.0 – 80.5	30.7	29.5	9.0	SC	-	21.6	-
BH-2, SS-6	77.0 – 75.5	-	-	-	-	-	3.8	-
BH-3, SS-1	95.0 – 93.5	64.9	39.8	16.3	CL	-	21.3	-
BH-3, SS-2	92.0 – 90.5	63.3	35.0	6.2	ML	-	21.8	-
BH-3, SS-3	90.0 – 88.5	63.2	38.7	13.9	ML	-	24.2	-
BH-3, SS-4	87.0 – 85.5	61.4	35.3	12.3	ML	-	28.5	-
BH-3, SS-5	82.0 – 80.5	51.2	31.0	7.7	CL	-	26.1	-
BH-3, SS-6	77.0 – 75.5	52.2	15.0	3.9	CL-ML	-	10.8	-
BH-4, SS-1	95.5 – 94.0	73.4	33.5	9.0	ML	-	22.6	-
BH-4, cuttings	87.5 – 86.5	55.0	41.0	17.8	CL	-	26.2	-
BH-4, SS-4	87.7 – 86.2	72.9	32.1	9.1	CL	-	27.9	-
BH-4, SS-6	83.2 – 81.7	-	-	-	-	-	25.9	-
BH-4, cuttings	81.5 – 81.0	38.7	34.3	10.1	SC	-	27.7	-
BH-4, SS-7	81.0 –79.5	68.0	21.1	1.6	ML	-	22.2	-
BH-4, ST-3	90.0-87.7	-	-	-	-	111	24.0	89.5
BH-4, ST-5	85.5-83.2	-	-	-	-	124	-	-

Table 5.1. Concluded.

Borehole and sample No.	**Elevation (ft)**	**Finer than No. 200 Sieve[a] (%)**	**Liquid limit[b] (%)**	**Plasticity index[c] (%)**	**USCS[d]**	**Moist unit weight (pcf)**	**Natural moisture[e] content (%)**	**Dry unit weight (pcf)**
BH-5, cuttings	78.5 – 76.5	61.8	24.6	4.6	ML-CL	-	24.4	-
BH-5, ST-1	97.5-95.4	-	-	-	-	125	21.0	103.3
BH-5, ST-2	93.9 - 91.6	-	-	-	-	115	26.0	91.3
BH-5, ST-3	91.6 – 89.3	-	-	-	-	121-124	23.0*	98.4-100.8
BH-5, ST-4	89.3 – 87.0	-	-	-	-	115-119	26.0*	91.3-94.4
BH-5, ST-5	87.0 – 84.7	68.3	30.6	28.8	-	117	21.6	96.2
BH-5, ST-6	84.7 – 82.4	-	-	-	-	106-122	25.0*	84.5-98.6
BH-6, SS-1	97.0 – 95.5	34.2	37.8	10.0	SC	-	22.6	-
BH-6, ST-2	94.0 – 91.7	70.0	37.9	16.3	CL	-	21.6	-
Block 3	96.0 – 95.0	68.4	37.6	11.6	ML	121-125	20.0-22.6	99-104

Notes

[a]ASTM D1140
[b]ASTM D 4318
[c]ASTM D 4318 (PI = LL – PL)
[d]ASTM D 2487 (USCS = Unified Soil Classification System)
[e]ASTM D 2216
* average moisture content

Type of sample

Cuttings (auger cuttings)
SS (split spoon)
ST (Shelby tube)
Block (hand cut)

Table 5.2. Summary of consolidation test results on samples of natural soil.

Sample	Elevation (ft)	P_o (tsf)	P_p (tsf)	OCR	C_{ec}	C_{er}	Permeability (cm/sec)
BH-5, ST-3*	90.5	0.42	5-10	-	0.13-0.14	0.025-0.040	K_h=6.8x10^{-7}
BH-5, ST-4	87.5	0.60	5-10	8.3-16.7	0.14-0.17	0.030-0.035	k_v=1.6x10^{-7}
BH-4, ST-5	84.5	0.78	5-10	6.4-12.8	0.17-0.18	0.013-0.030	k_v=1.2x10^{-6}
* Sample BH-5, ST3 was trimmed horizontal.							

Table 5.3. Summary of UU test results on samples of natural soil.

Borehole No.	Sample	Elevation (ft)	Cell pressure (psi)	Strain rate (% / min)	p_{max} (psi)	q_{max} (psi)	Axial strain at failure (%)
BH-5	ST-2 #6	93.9 to 91.6	5	1.0	25.0	20.0	3.42
	ST-2 #4		10		38.4	28.4	2.43
	ST-2 #3		20		59.3	39.3	3.10
	ST-2 #1		30		71.9	41.9	6.52
BH-5	ST-3 #3	91.6 to 89.3	5	1.0	25.3	20.3	4.62
	ST-3 #6		10		33.5	23.5	5.88
	ST-3 #2		20		53.1	33.1	4.96
	ST-3 #4		30		67.4	37.4	5.35
BH-5	ST-4 #3	89.3 to 87.0	5	1.0	20.2	15.2	16.36
	ST-4 #2		10		27.3	17.3	14.53
	ST-4 #1		20		45.1	25.1	8.87
	ST-4 #4		30		57.0	27.0	4.62
Block 3	#2, vert., 1.4 in dia.	95.5	3	0.3	12.9	9.8	2.21
	#3, vert., 1.4 in dia.		2		19.6	17.6	2.72
	#4, vert., 1.4 in dia.		0		14.5	14.5	0.93
	#5, vert., 1.4 in dia.		4		24.6	20.6	1.91
	#6, horz., 1.4 in dia.	95.5	0	0.3	13.6	13.6	1.35
	#7, horz., 1.4 in dia		4		25.0	21.0	2.31
	#8, horz., 1.4 in dia		2		19.3	17.3	1.96
	#9, vert., 2.8 in dia.	95.5	2	0.3	29.71	27.71	2.84
Block 1	#10, vert., 2.8 in dia	95.5	4	0.3	22.21	18.21	3.57
	#11, vert., 2.8 in dia		0		13.67	13.67	1.89
Failure criterion: maximum deviator stress.							

Table 5.4. Summary of CU test results on samples of natural soil.

Borehole No.	Sample	Elevation (ft)	B*	Cell pressure (psi)	Strain rate (% / min)	p'_{max} (psi)	q_{max} (psi)	Axial strain at failure (%)
BH-4	ST-3 #3	89.0	0.93	10	0.10	45.2	25.6	10.76
	ST-3 #4	88.5	0.95	20	0.08	41.9	23.5	5.33
	ST-3 #5	88.0	0.98	30	0.06	50.7	26.0	8.51
BH-5	ST-1 #2	96.5	0.93	-	-	-	-	-
	Stage 1	-	-	3.55	0.1	-	-	-
	Stage 2	-	-	8.96	0.1	-	-	-
	Stage 3	-	-	15.04	0.1	-	-	-
BH-5	ST-5 #3	86.0	0.95	-	-	-	-	-
	Stage 1	-	-	5	0.1	15.3	10.1	-
	Stage 2	-	-	10.1	0.1	22.6	13.9	-
	Stage 3	-	-	14.9	0.1	30.7	17.7	4.76
BH-5	ST-6 #1	84.5	0.97	10.3	0.1	9.0	4.5	20.67
	ST-6 #2	84.0	0.98	21.3	0.1	20.6	11.3	20.23
	ST-6 #5	83.0	0.97	30.2	0.07	39.6	21.6	14.09
BH-5	ST-6 #4	83.5	0.93	-	-	-	-	-
	Stage 1	-	-	5.04	0.07	15.5	9.9	-
	Stage 2	-	-	10.0	0.07	21.0	12.5	-
	Stage 3	-	-	15.03	0.07	31.6	17.8	7.46

Notes
* B = Skempton's pore pressure coefficient.
Failure criterion: maximum deviator stress.

Table 5.5. Summary of CD test results on samples of natural soil.

Borehole No.	Sample	Elevation (ft)	B*	Cell pressure (psi)	Strain rate (% / min)	p'_{max} (psi)	q_{max} (psi)	Axial strain at failure (%)
BH-5	ST-5 #4	85.5	0.94	10.0	0.104	29.6	19.6	4.40
	ST-5 #5	85.1	0.95	20.0	0.060	48.2	28.2	7.94
	ST-5 #6	84.8	0.96	30.0	0.010	68.8	38.8	14.23

Notes
* B = Skempton's pore pressure coefficient.
Failure criterion: maximum deviator stress

Table 5.6. Average results from field moisture-density tests.

	Nuclear gage results			**Corrected results based on sand cone tests**		
Soil type	**g_m (pcf)**	**w_c (%)**	**g_{dry} (pcf)**	**g_m (pcf)**	**g_{dry} (pcf)**	**D_r (%)**
Crusher run gravel (unscalped)	141.0	4.7	134.7	134.1	128.1	67.4
Crusher run gravel (scalped)	-	-	-	-	-	64.6
New castle sand (compacted)	109.4	4.5	104.7	104.0	99.5	72.8
New castle sand (uncompacted)	92.1	3.6	88.9	88.9	88.9	10.6

Table 5.7. Summary of CD test results on compacted New Castle sand samples.

Relative density (%)	**Cell pressure (psi)**	**Maximum deviator stress (psi)**	**p' (psi)**	**q (psi)**	**Axial strain at failure (%)**
20	2.1	6.9	5.5	3.5	4.0
20	3.4	11.5	9.2	5.7	4.0
20	5.2	14.5	12.4	7.2	4.0
60	1.4	9.0	5.8	4.5	1.5
60	2.3	13.3	8.9	6.7	1.6
60	3.6	15.9	11.5	8.0	6.2
60	5.1	21.1	15.7	10.6	2.2
80	2.0	14.7	9.4	7.4	1.8
80	3.4	23.6	15.2	11.8	2.1
80	4.8	26.6	18.1	13.3	2.1

Table 5.8. Summary of CD test results on compacted crusher run gravel samples.

Relative density (%)	**Cell pressure (psi)**	**Maximum deviator stress (psi)**	**p' (psi)**	**q (psi)**	**Axial strain at failure (%)**
50	2.0	15.7	9.8	7.9	1.1
50	3.1	16.6	11.4	8.3	4.0
50	4.9	22.8	16.3	11.4	9.7
70	2.1	27.9	16.0	13.9	1.9
70	3.4	30.8	18.9	15.4	3.0
70	5.8	57.0	34.3	28.5	2.4
90	2.1	39.0	21.6	19.5	2.6
90	2.8	45.8	25.7	22.9	2.7
90	4.1	64.3	36.2	32.2	3.2

Table 5.9. Friction angles for New Castle sand and crusher run gravel.

Depth	**Friction angle, f¢ (deg)**		
	compacted sand γ_m = 105 pcf	uncompacted sand γ_m = 92 pcf	compacted gravel γ_m = 138 pcf
0.75	51	39	56
1.5	49	37	53
3	47	36	51

Note: $$f' = f_o - \Delta f \log\left(\frac{s_3}{p_a}\right)$$

s_3 = confining pressure

p_a = atmospheric pressure (2117 psf)

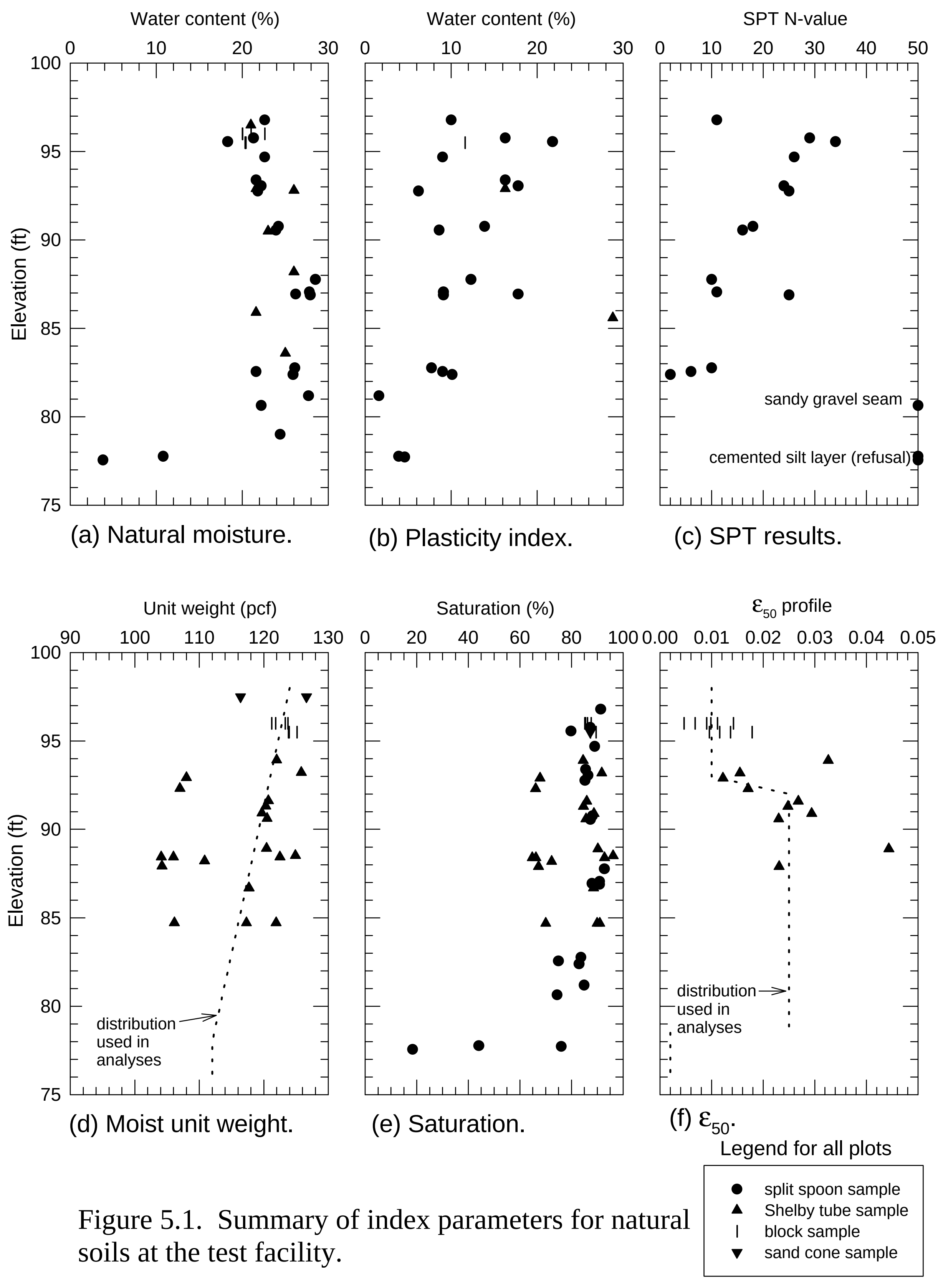
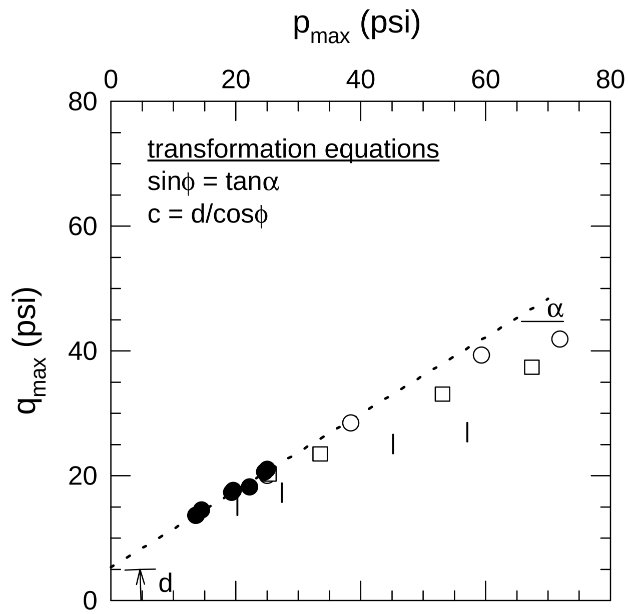
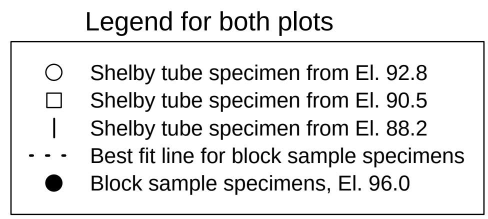


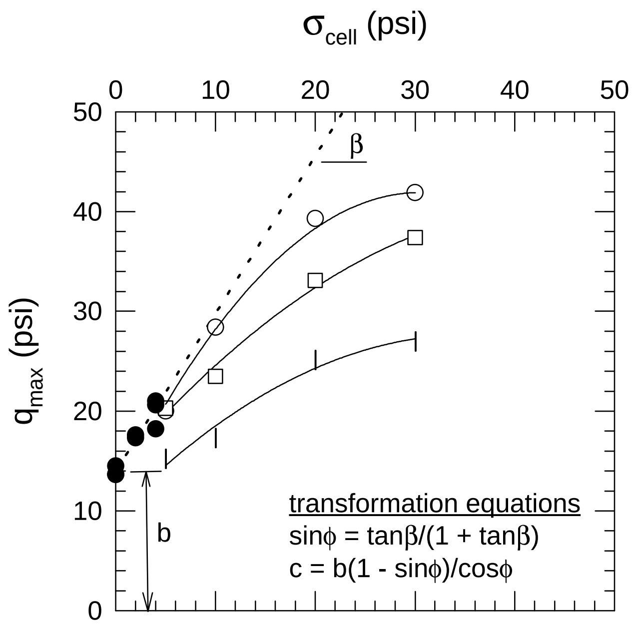
Figure 5.1. Summary of index parameters for natural soils at the test facility.



Parameter from plot (a)		
Elev.	α (deg)	d (psi)
96.0	31.6	5.4
92.8	30.0	5.0
90.5	28.0	4.0
88.2	25.0	3.9

(a) p_{max} versus q_{max}.





Parameters from plot (b)		
Elev.	β (deg)	b (psi)
96.0	57.7	14.0
92.8	58.0	11.8
90.5	49.0	8.5
88.2	41.5	7.3

Summary of results		
Elev.	ϕ (deg)	c (psi)
96.0	38	7.0
92.8	35	6.0
90.5	32	4.7
88.2	28	4.4

(b) σ_{cell} versus q_{max}.

Figure 5.2. Natural soil strength parameters based on UU triaxial tests.

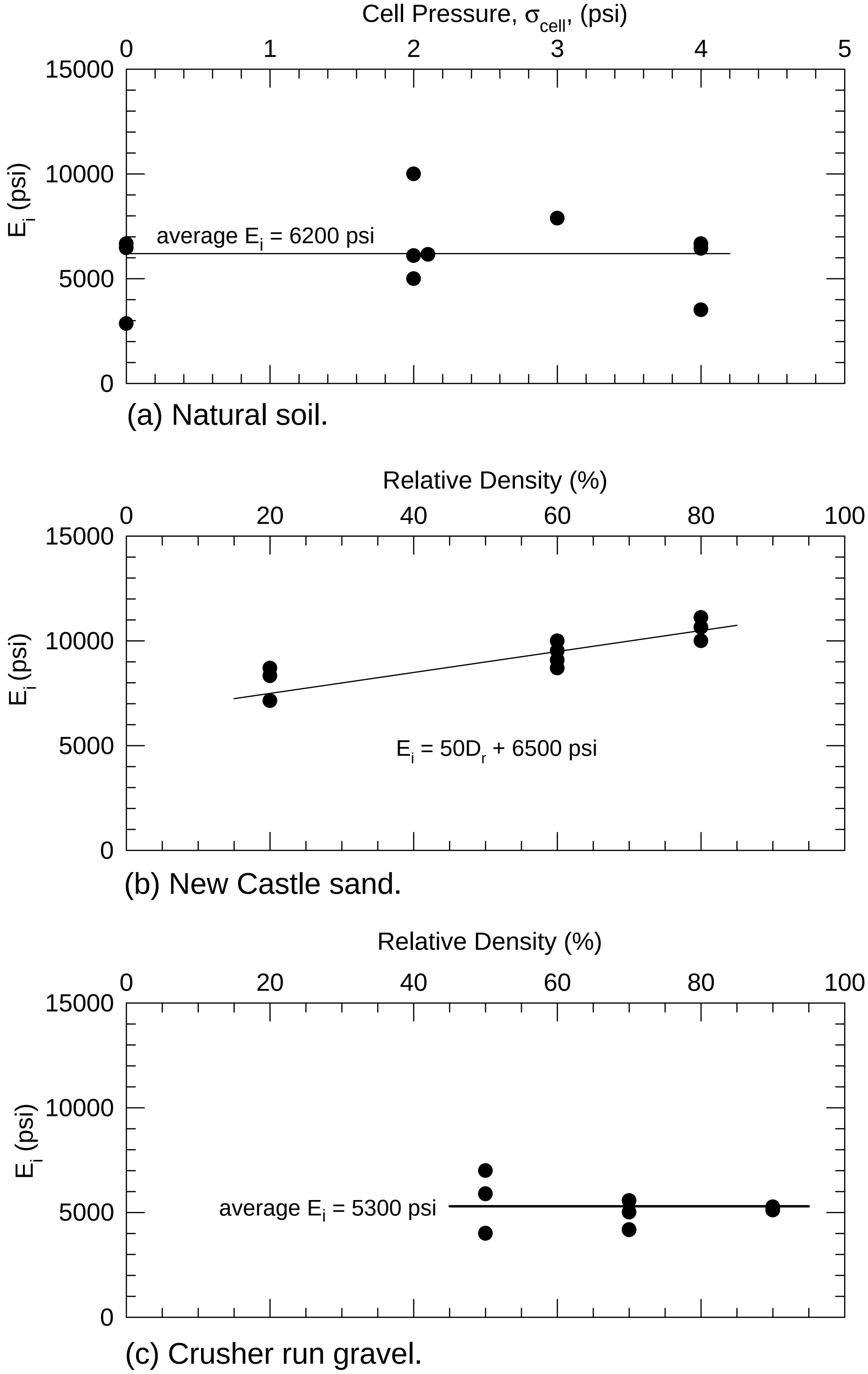


Figure 5.3. Initial tangent modulus (E_i) for natural soil, New Castle sand, and crusher run gravel.

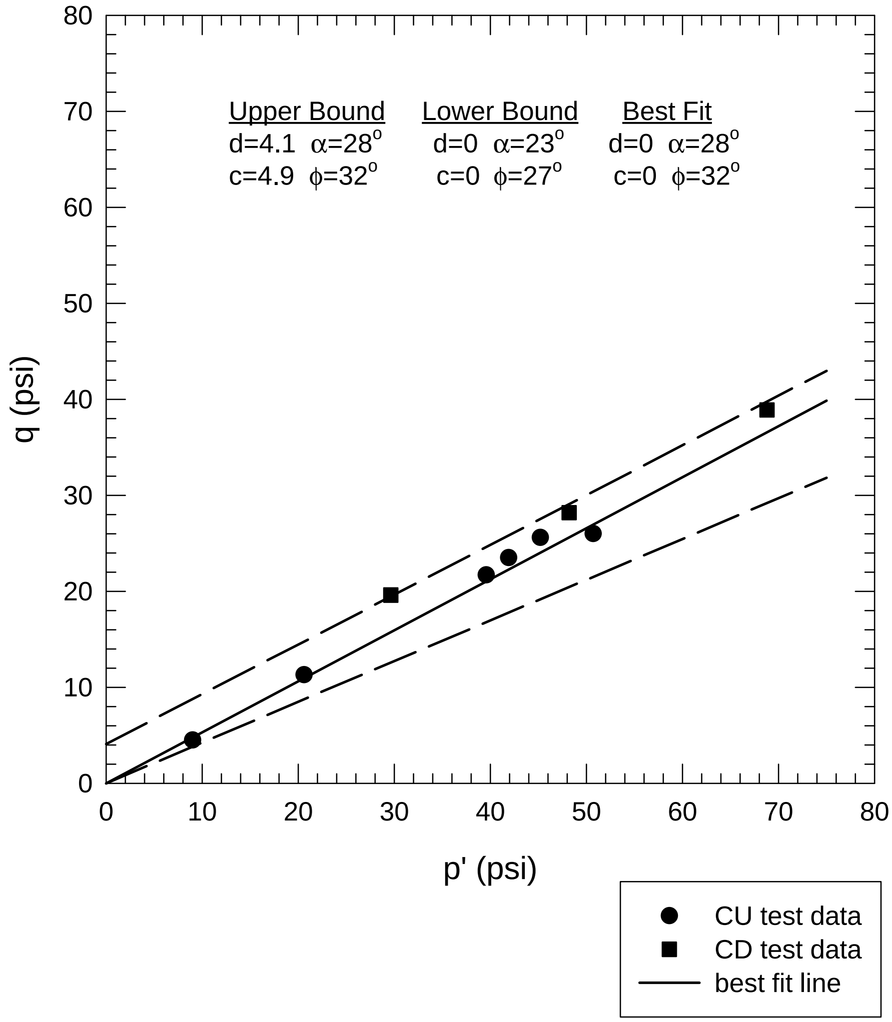


Figure 5.4. Maximum values of p' versus q for CU and CD triaxial tests on natural soil.

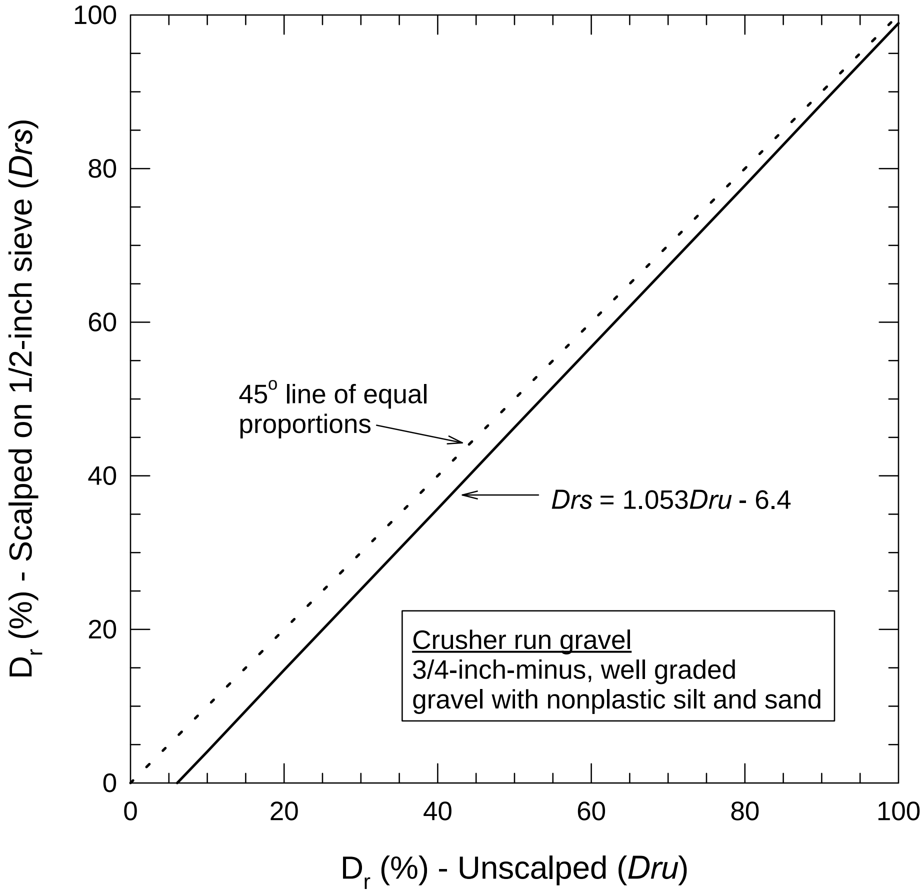
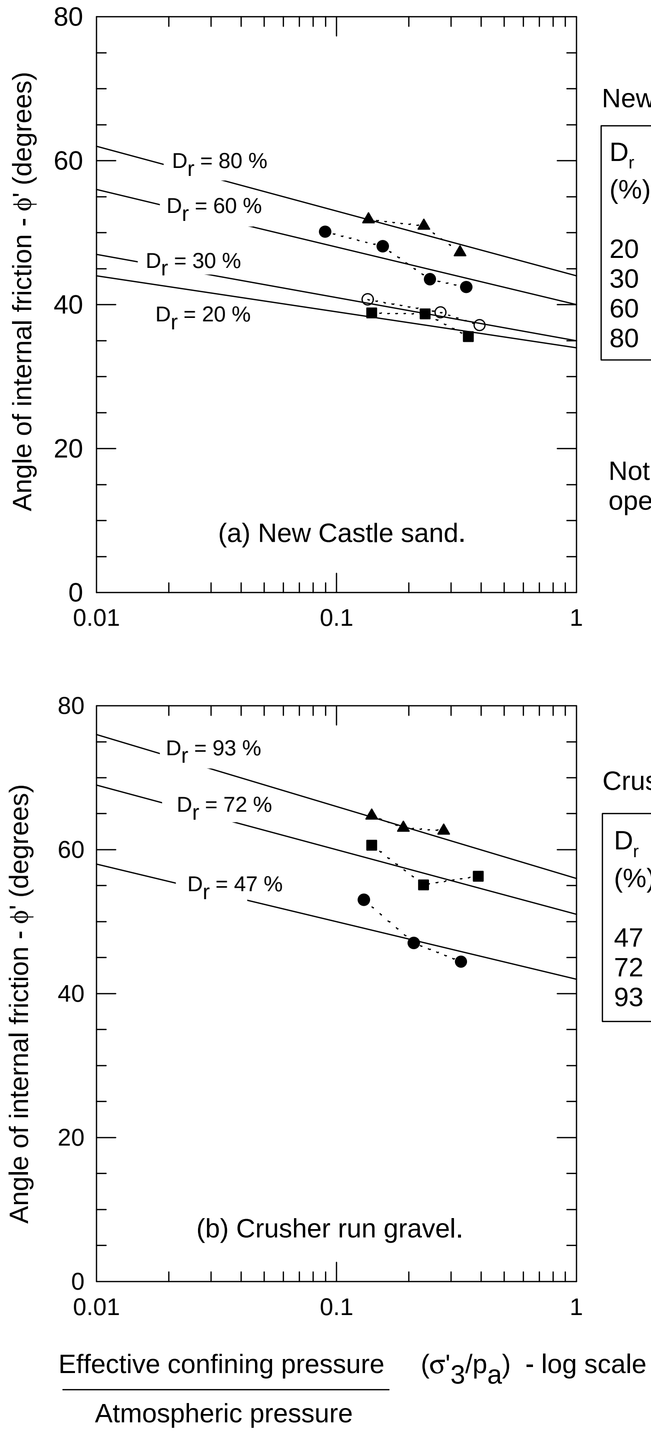


Figure 5.5. Relative density (D_r) comparison of scalped and unscalped crusher run gravel.



New Castle Sand

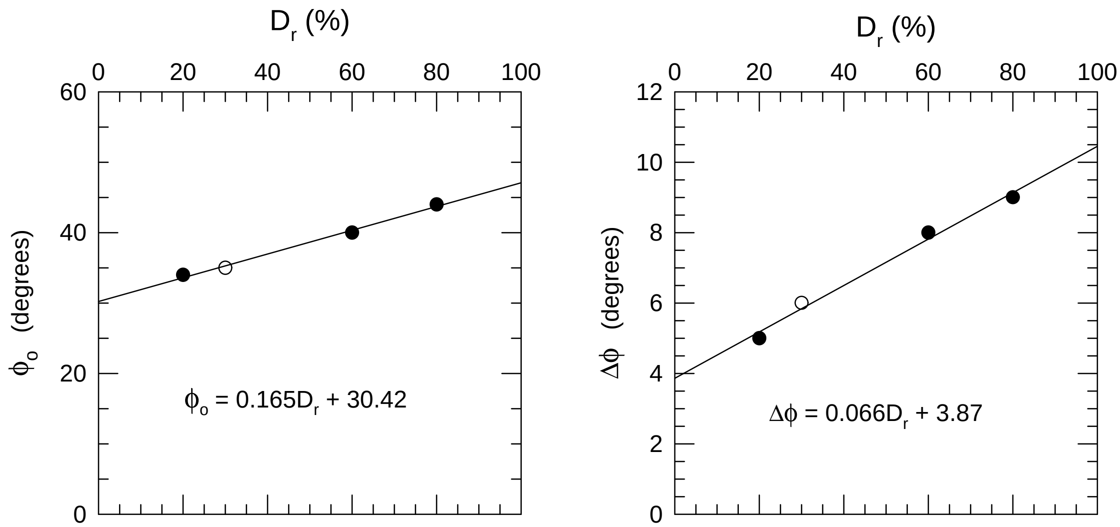
D_r (%)	ϕ_o (deg)	$\Delta\phi$ (deg)	Triaxial Test
20	34	5	CD
30	35	6	CU
60	40	8	CD
80	44	9	CD

Note: Solid symbols represent CD results, open symbols represent CU results.

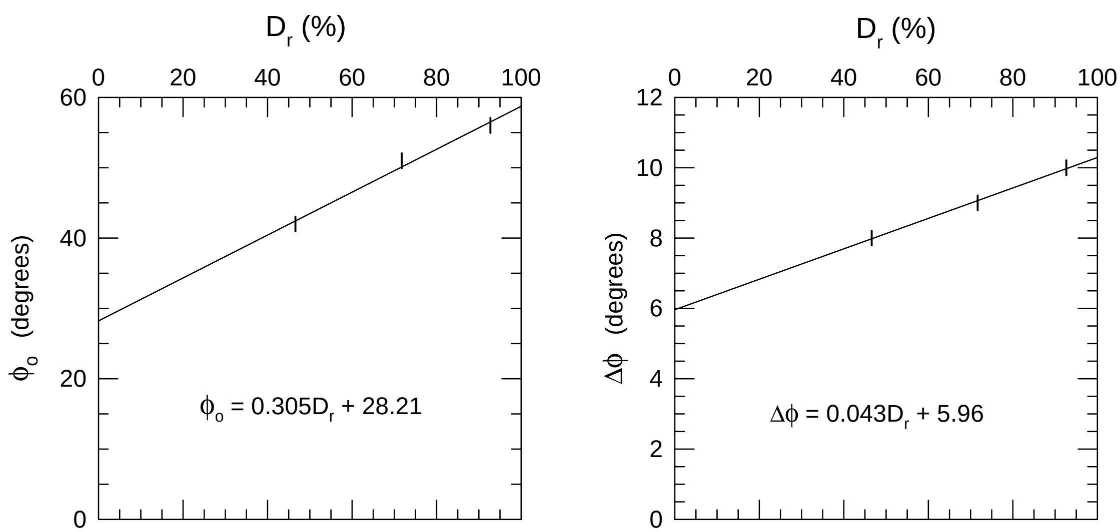
Crusher Run Gravel

D_r (%)	ϕ_o (deg)	$\Delta\phi$ (deg)	Triaxial Test
47	42	8	CD
72	51	9	CD
93	56	10	CD

Figure 5.6. Effect of density on strength of New Castle sand and crusher run gravel.



(a.) New Castle sand.



(b.) Crusher run gravel.

Equation 5.2

$\phi' = \phi_o - \Delta\phi \log_{10}(\sigma'_3/p_a)$
where p_a = atmospheric pressure
σ'_3 = effective confining pressure

Note: Solid symbols represent CD results, open symbols represent CU results.

Figure 5.7. Distribution of ϕ_o and $\Delta\phi$ for New Castle sand and crusher run gravel.

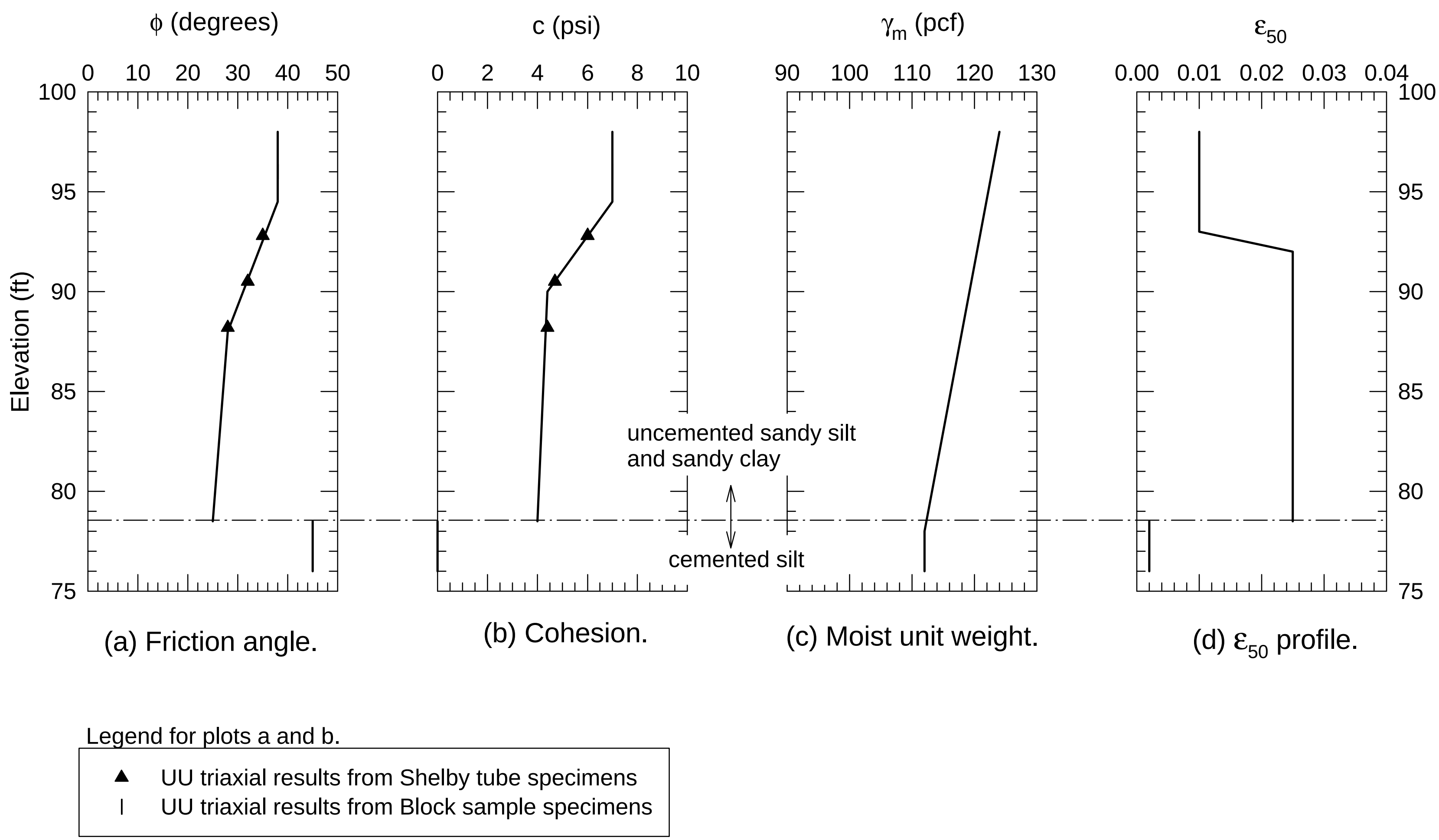


Figure 5.8. Soil parameter distributions for analytical models.