RUSS COLLEGE of Engineering and Technology

Highway Embankments in Ohio: Soil Properties and Slope Stability

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Background

- Roadway embankments constitute one of the most common geotechnical facilities in the U.S.
- Despite its seemingly straightforward nature, design and construction of highway embankments are complicated by the fact that a number of key issues (bearing capacity, settlement, drainage, erosion, and slope stability) must be all taken care of.
- In Ohio, highway embankments have been designed often using soil properties that are based on previously published default values or that are derived from empirical correlations found in literature.
- This practice has become popular, since it reduces project cost and time.

Background

- Main problem with soil property data found in the literature is that it was in most cases determined for soils found outside Ohio and in some cases outside the U.S. Applicability of the literature data to Ohio soils has not been fully investigated.
- Because of the popular short-cut approach, there have been some cases in Ohio where embankment slopes suffered slope instability problems.
- Structural stability of roadway embankments is vital to the state economy and public safety.

Background

- The geotechnical research team at Ohio University recently conducted a comprehensive study on shear strength properties of soils and stability of highway embankments for the Ohio Department of Transportation.

Project Tasks

Task 1 (Literature Review & Site Selection)

- (a) Conduct a literature review to survey geological conditions existing in Ohio
- (b) Establish site selection criteria in consultation with Ohio DOT personnel
- (c) Contact Ohio DOT District Geotechnical Engineers and request a list of highway embankm-ent sites suitable for soil sampling/testing
- (d) Finalize selection of nine highway embankment sites

Project Tasks

Task 2 (Field & Laboratory Soil Testing)

- (a) Calibrate equipment that will be used in the field
- (b) Perform subsurface exploration work at each highway embankment site
- (c) Subject soil samples recovered from the sites to index property and shear strength tests in the laboratory
- (d) Analyze all laboratory test data

Project Tasks

Task 3 (Empirical & New Correlations)

- (a) Evaluate default soil property values and empirical correlations found in the literature in light of the field and laboratory test data accumulated in the project
- (b) Analyze the field and laboratory test data together statistically to develop new correlations among basic index properties, field measurements, and shear strength properties for each major embankment soil type found in Ohio

Project Tasks

Task 4 (Slope Stability Analysis & Guidelines)

- (a) Feed the average properties of each major soil type encountered into a series of computerized embankment slope stability analysis
- (b) Formulate a set of guidelines concerning both the design and construction of highway embankment structures in Ohio

Task 1

Engineering Characteristics of Ohio Soil Series (by Johnson 1975: Report OHIO-DOT-12-75)

Parent Materials of Ohio Soils

- Bedrock & Residual Soils
- Lake Deposits
- Glacial Deposits
- Alluvial Deposits

Task 1

Bedrocks in Ohio

Western Ohio
Limestone
& Shale

Eastern Ohio
Sandstone
& Shale

Task 1

Soil Deposits in Ohio

Glacial Till

Lake Deposits

Unglaciated

Task 1

Site Selection Criteria

- Embankment fill height over 25 ft (7.6 m)
- Embankment soil fill cohesive
- Site located on major highway
- Site recommended by ODOT or subcontractor
- Site represents unique geographical location or geological condition not duplicated many times previously
- Slopes at the site not experiencing any instability problems
- A lack of gravel size particles and rock fragments
- No guardrails
- Relatively level grassed area in median or beyond shoulder
- Age was determined to be a nonfactor

Task 1

Site Locations (9 Sites)

Site 6
(ERI-2)

Site 7
(HAN-75)

Site 2
(FAY-35)

Site 1
(HAM-275)

Site 3
(LAK-2)

Site 5
(MRW-71)

Site 8
(MUS-70)

Site 9
(NOB-77)

Site 4
(ATH-33)

Task 1

Distributions of Selected Field Sites

- Three (3) sites in northern Ohio
- Four (4) sites in central Ohio
- Three (2) sites in southern Ohio
- Seven (7) ODOT districts
- Two (2) sites (east, west) in the lake deposits area
- Four (4) sites in the glaciated region
- Four (3) sites in the unglaciated region

Task 2

Standard Penetration Test (SPT)

- Oldest and most commonly used in-situ soil test method
- Drop a 140-lb (64-kg) hammer 30 inches (0.76 m) to drive a split-spoon barrel
- SPT-N value = number of hammer blows per 1-ft (0.3-m) penetration
- SPT-N value depends on several factors such as the hammer type, actual drop height, inclination of the hole, hole diameter, presence of liner inside split-spoon barrel, and test depth.

Task 2

Subsurface Exploration Work

- Use of automatic SPT hammer
- Calibration of SPT equipment
- Dedicated equipment and personnel (QC)
- Continuous SPT to 25-ft (7.6-m) depth in initial hole
- Direct visual logging of soil layers
- Four (4) surrounding holes to provide twelve (12) Shelby tube soil samples at three (3) selected depth ranges

Task 2

SPT Automatic Hammer Calibration

- Maximum Energy Transferred to Rods (EMX):

$$EMX = \int F(t) \cdot V(t)\,dt$$

PAK model pile driver analyzer

where F(t) = force measured at time t; and V (t) = velocity measured at time t.

- Energy transfer ratio (ETR) = EMX/(Theoretical SPT Hammer Energy) = EMX/(0.35 kip-ft)
- Calibration by GRL Engineering, Inc. (Cleveland, OH; Tel. 216-292-3076); depth 1 to 25.5 ft
- Results: ETR = 78.8 to 84.4% (ave. 81.6%) for Truck #55 with CME automatic hammer & AWJ rods.

Task 2

Normalization of SPT-N value

- SPT-N values are normalized to an overburden pressure of 1 tsf (13.9 psi, 95.7 kPa) and to an energy transfer rate of 60% (= energy typically applied by the safety hammer in the U.S.)

$$(N_1)_{60} = C_N * N_{60} = C_N * (ETR/60) * N$$

where $(N_1)_{60}$ = fully normalized SPT N value; C_N = depth or over-burden pressure correction factor; N_{60} = N value measured with 60 % hammer efficiency; ETR = energy transfer ratio (%); and N = raw N value.

Task 2

Normalization of SPT-N Value

- A few different methods proposed for C_N
- Peck et al. (1974)

$$C_N = 0.77 \log\left(\frac{20}{\sigma_0'}\right)$$

where σ'_0 = effective overburden pressure (tsf)

- Terzaghi et al. (1996)

$$C_N = \left(\frac{100}{\sigma_0'}\right)^{0.5}$$

where σ'_0 = effective overburden pressure (kPa)

Task 2

Normalization of SPT-N Values

- Seed et al. (1975)

$$C_N = 1 - 1.25 \log\left(\frac{\sigma_0'}{p_a}\right)$$

- Skempton (1986)

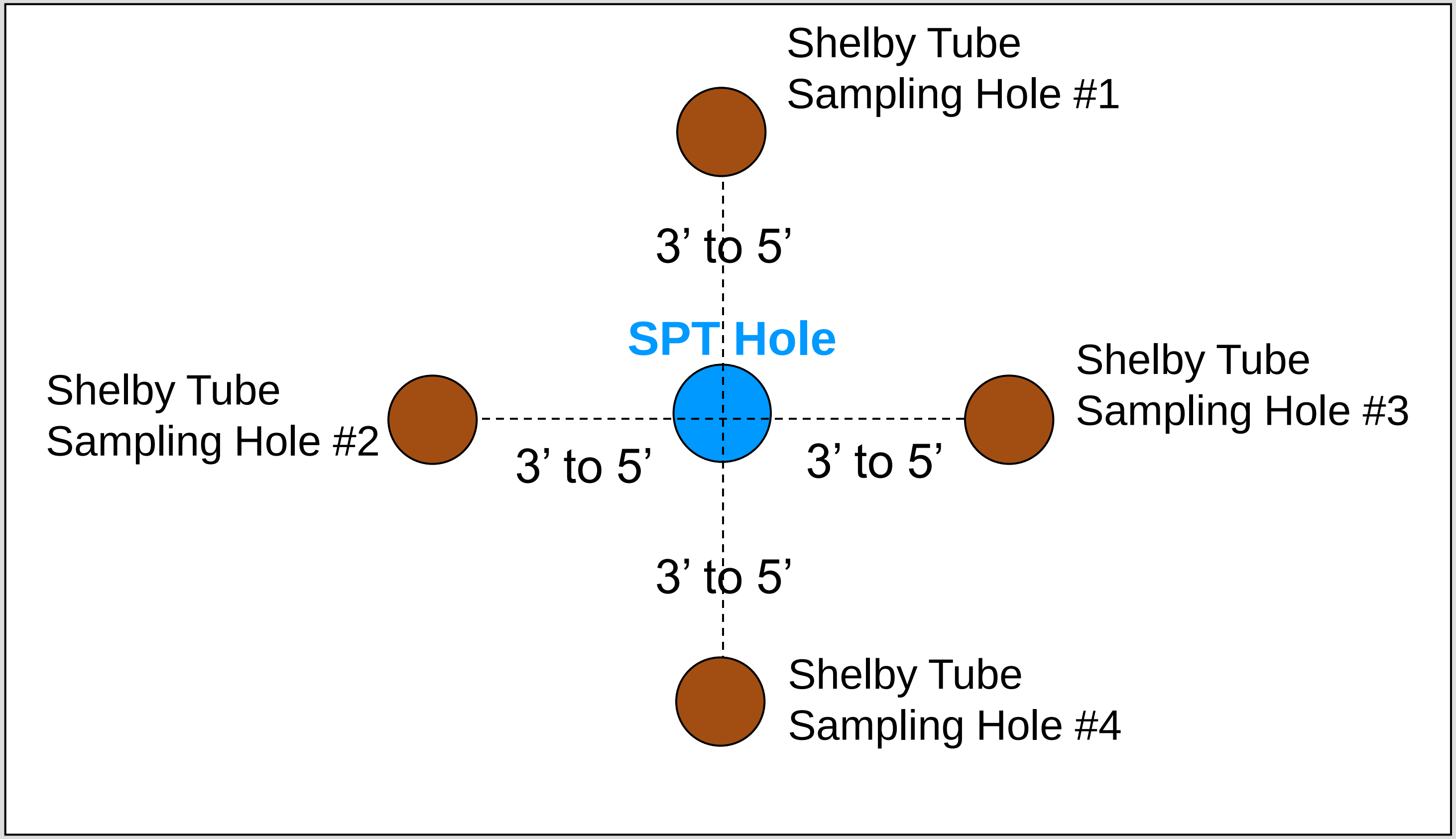
$$C_N = \frac{2}{1 + \left(\sigma_0' / p_a\right)}$$

where σ'_0 = effective overburden pressure (psf); and p_a = atmospheric pressure (= 2,000 psf = 1 tsf)

Task 2

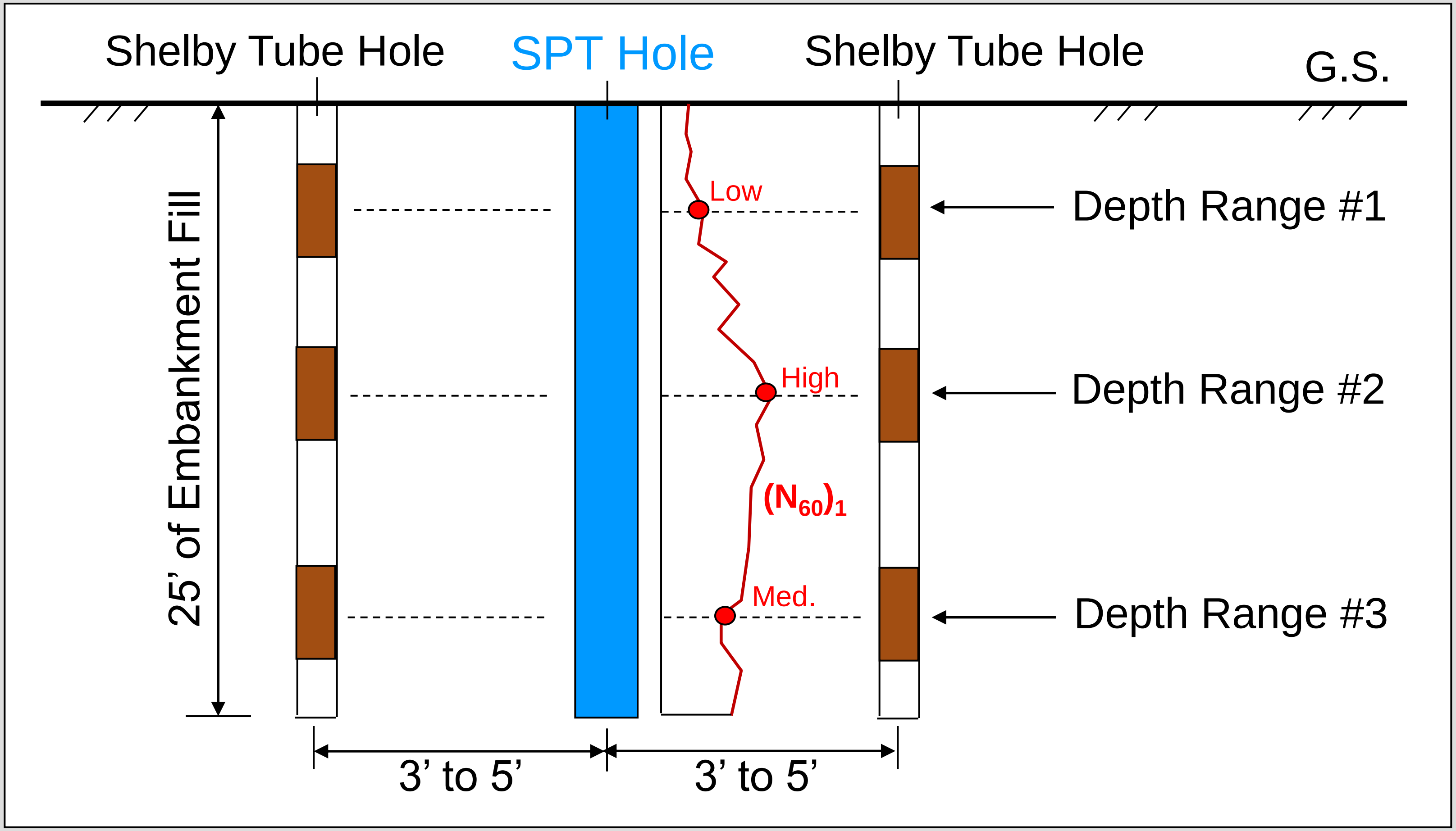
- Apply the approach proposed by Seed et al. (1975) to normalize N_{60} values, as it represented the average of all the C_N values.
- Determine the three soil sampling depths by selecting high, medium, and low $(N_{60})_1$ values.
- High $(N_{60})_1$ value should be below 40 to prevent Shelby tube from crushing.
- If soil type changes through depth, place at least one sampling depth within each soil type.

Task 2



Plan View of Master Plan

Task 2



Side View of Master Plan

Task 2

- AASHTO Soil Classification System

	A-4	A-5	A-6	A-7-6
(-) No. 200	36 min.			
Liquid Limit (LL)	40 max.	41 min.	40 max.	41 min.
Plasticity Index (PI)	10 max.		11 min.	
Description	Silty soils		Clayey soils	
Note				PI > (LL-30)

[Note] Max. dry unit weight (typical) = 120 pcf for A-4 soils; 110 pcf for A-6 & A-7-6 soils --- Ref. ODOT (2006), "Construction Inspection Manual of Procedures," Columbus, OH, pp. 962-963.

Task 2

- Further Breakdowns of A-4 & A-6 Soils by Ohio DOT
- A-4a: A-4 Soils with 36-49% (-) Sieve No. 200
- A-4b: A-4 Soils with at Least 50% (-) Sieve No. 200
- A-6a: A-6 Soils with PI Between 11 and 15
- A-6b: A-6 Soils with PI at Least 16
- [Note] A-4b & A-6b soils are more problematic but rare compared to A-4a & A-6a soils.

Task 2

List of Fundamental Laboratory Tests

- Visual soil descriptions
- Moisture contents & Dry unit weight
- Atterberg limits (plastic; liquid → plasticity index)
- Grain size analysis (mechanical sieve; hydrometer)
- Soil classifications by AASHTO/ODOT method
- Specific gravity
- Unconfined compression strength

Task 2

Advanced Laboratory Test

- Consolidated-undrained (C-U) triaxial compression test with pore pressure measurement
- Stage 0 (Extrusion & Mounting)
- Stage 1 (Saturation)
- Stage 2 (Consolidation)
- Stage 3 (Axial Loading)
- ASTM D-4767-04: “Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils”

Task 2

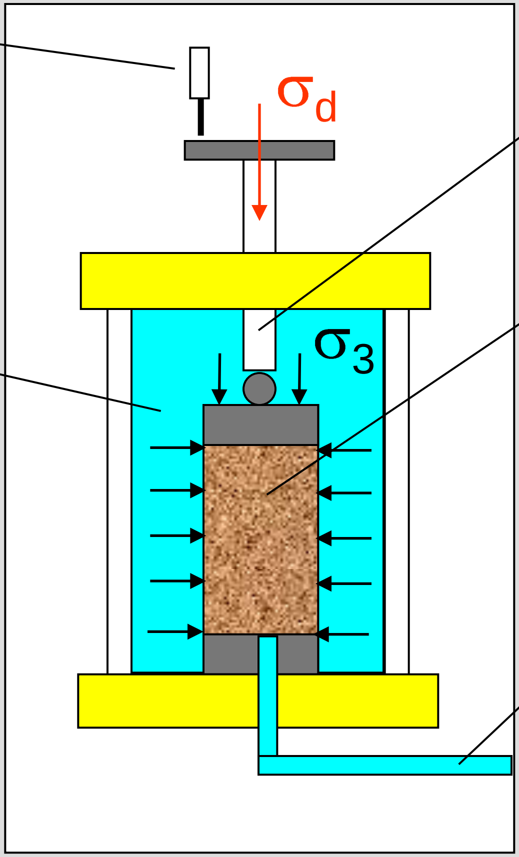
Key Specifications of ASTM D-4767-04

- Specimen's height/diameter ratio of 2.0 to 2.5
- Back pressure 5 psi (35 kPa) less than chamber pressure
- Saturation to minimum B value of 0.95, where B = $\Delta u/\sigma_3$
- Consolidation stage to follow the procedure outlined in ASTM D-2435. Determine t_{50}
- Loading rate set at 4%/(10*t_{50}) so that pore pressure can achieve equilibrium
- Load specimen to 15% axial strain, a 20% drop in deviator stress, or 5% additional strain beyond deviator stress peak.
- Check for presence of large stones after test

Task 2

Axial deformation censor

Chamber confining pressure applied through water



Loading piston equipped w/ load cell

Soil specimen encased in rubber membrane

Drainage line w/ pressure transducer

Components of Triaxial Test Set-Up

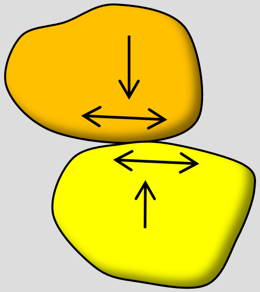
Task 2

Soil Shear Strength: Saturated vs. Unsaturated

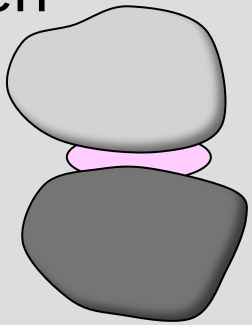
- The additional shear strength possessed in the unsaturated state is tenuous and can be lost easily upon wetting.
- It is a sound practice to design embankments with the assumption that unsaturated soils can become saturated over time. This eventual saturation can be caused by a rising water table, poor surface drainage, an unusually wet season, and leaking underground structures.

Task 2

- Shear Strength Parameters
- Internal Friction Angle (ϕ; ϕ') describes the frictional properties of individual particles and interlocking between particles. It is known to depend on soil mineral type, gradation, soil particle shape, and void ratio.



- Cohesion (c; c') describes the bonding between soil particles due to cementation, electrostatic attractions, and covalent bonding.



Task 2

- Short-Term & Long-Term Shear Strengths
- Short-term (end of construction) shear strength of cohesive soils is characterized by ϕ of 0 and $c = c_u$ (undrained cohesion) – total stress parameters.
- Long-term shear strength of cohesive soils is characterized by ϕ' (> 0) and c' -- effective stress parameters.

Task 2

Laboratory Determination of Soil Shear Strength

- Perform triaxial tests at three confining pressure levels
- Minimum confining pressure needs to be larger than over-burden pressure to assure normally consolidated soil behaviors
- Plot the results from three tests in p-q and p′-q ′ diagrams, where $p = (\sigma_{1f} + \sigma_3)/2$; $q = (\sigma_{1f} - \sigma_3)/2$;

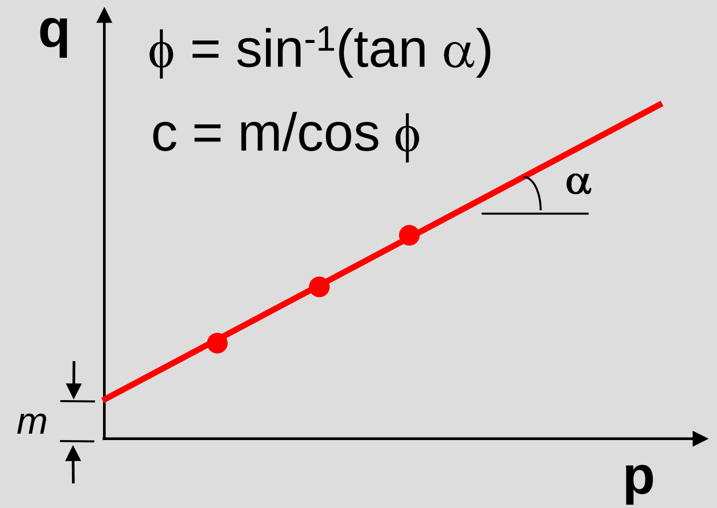
  $p' = (\sigma'_{1f} + \sigma'_3)/2$; and

  $q' = (\sigma'_{1f} - \sigma'_3)/2$

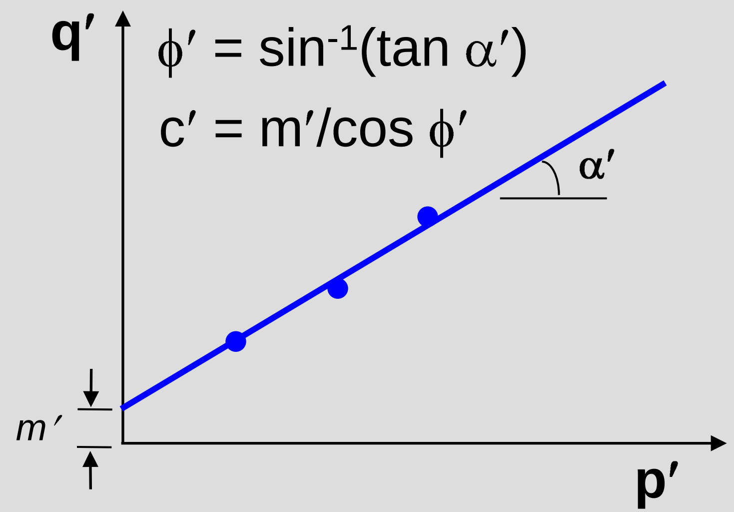
Stress path method

Task 2

- p-q and p′-q′ Diagrams



Using total stresses at failure



Using effective stresses at failure

[Note] $m = m' = c = c' = 0$ for granular soils and normally consolidated clays

Task 2

<u>Unconfined Compression Test</u>

- Performed only on cohesive soils
- Rapid test to obtain undrained cohesion strength (c_u):

  $c_u = \sigma_1/2$

- Considered as a special case of U-U test
- Involves no confining pressure;
- No drainage

Task 2

Field Test Data from Site 7 (HAN-75) – March 28, 2008

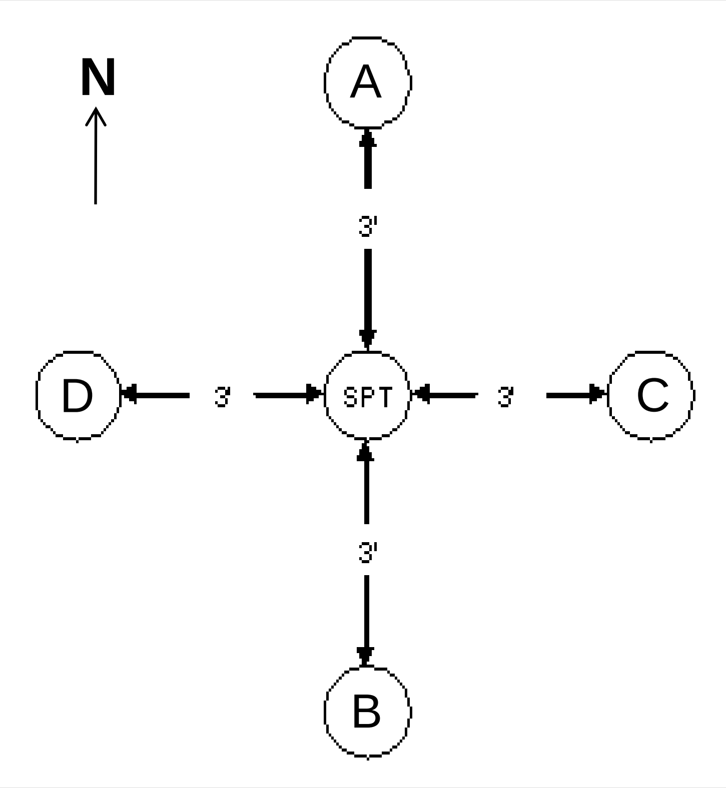
- I-75, Approx. 0.5 miles north of Exit 142 (Bluffton exit)
- SPT hole placed in outside shoulder area of northbound lanes

Depth (ft)	N Value	$(N_{60})_1$	Depth (ft)	N Value	$(N_{60})_1$
1.0-2.5	19	74.8	13.0-14.5	12	17.3
2.5-4.0	13	37.8	14.5-16.0	25	34.5
4.0-5.5	14	34.0	16.0-17.5	17	22.6
5.5-7.0	16	34.1	17.5-19.0	33	42.2
7.0-8.5	15	28.7	19.0-20.5	10	12.3
8.5-10.0	23	40.2	20.5-22.0	21	25.0
10.0-11.5	9	14.6	22.0-23.5	21	24.3
11.5-13.0	20	30.2	23.5-25.0	25	35.8

[Notes] Ave. unit weight of soil = 130 pcf (assumed).
No groundwater table encountered.

Task 2

- Shelby Tube Sampling at Site 7 (HAN-75)



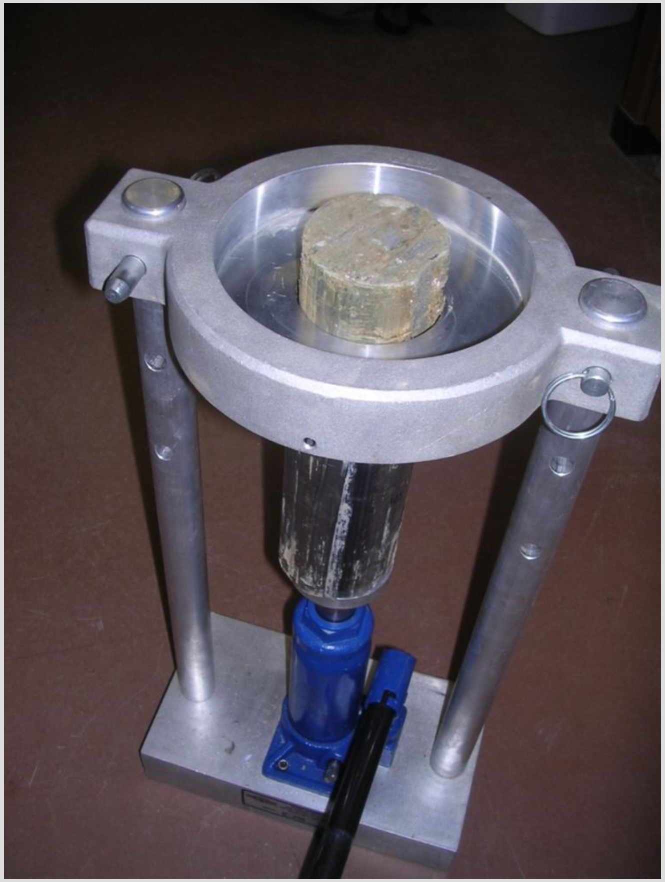
Tube ID	Depth (ft)	Recovery (in)
A-1	5.5-7.0	18.0
A-2	10.0-11.3	15.6
A-3	16.0-17.8	21.6
B-2	10.0-11.9	22.8
B-3	16.0-17.8	21.6
C-1	5.5-7.3	21.6
C-3	16.0-18.0	24.0
D-1	5.5-6.9	16.8
D-2	10.0-11.4	16.8

[Note] Tube length = 36.0 inches.

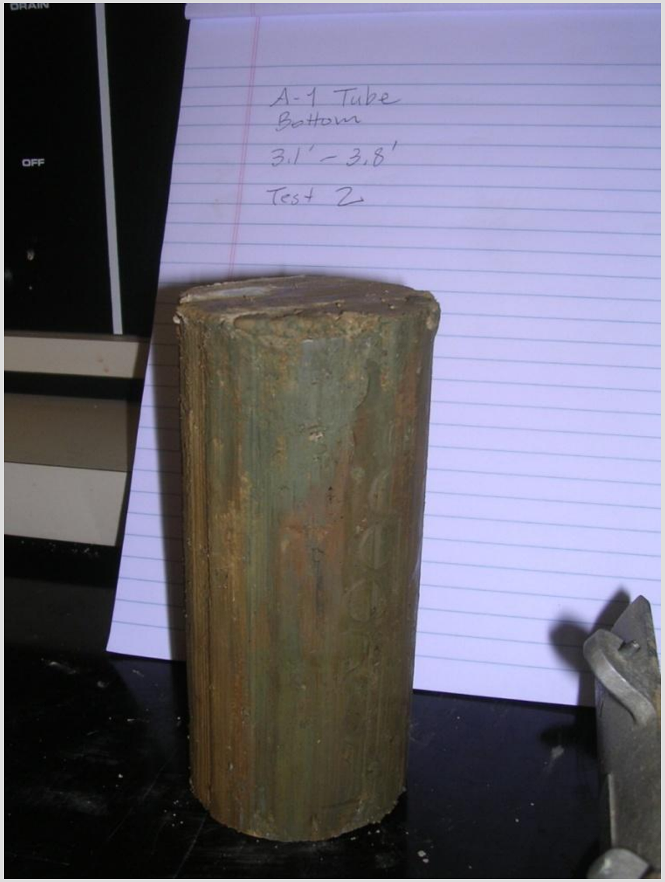
Task 2

Cutting of Shelby Tube into Shorter Sections

Task 2

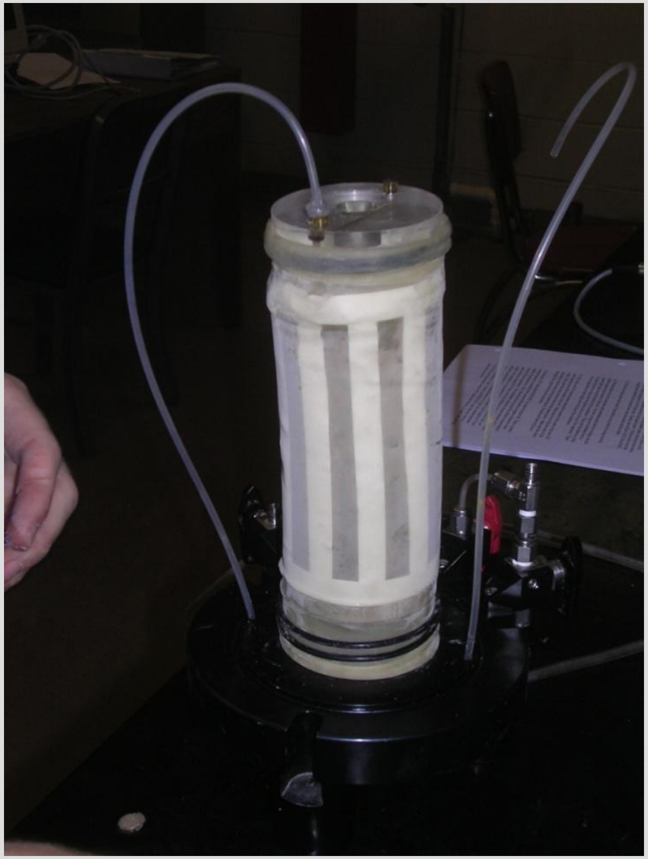


Hydraulic Jacking of Soil Sample

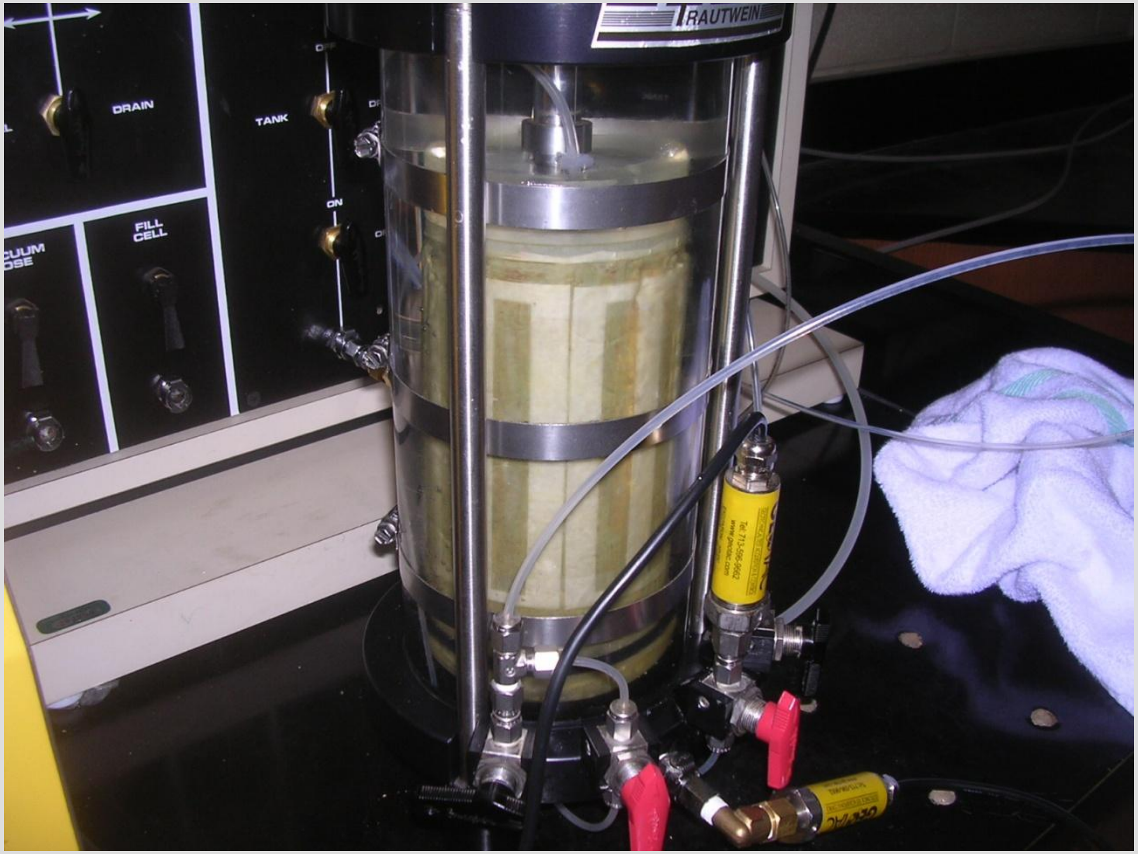


Specimen Examined Before Test

Task 2

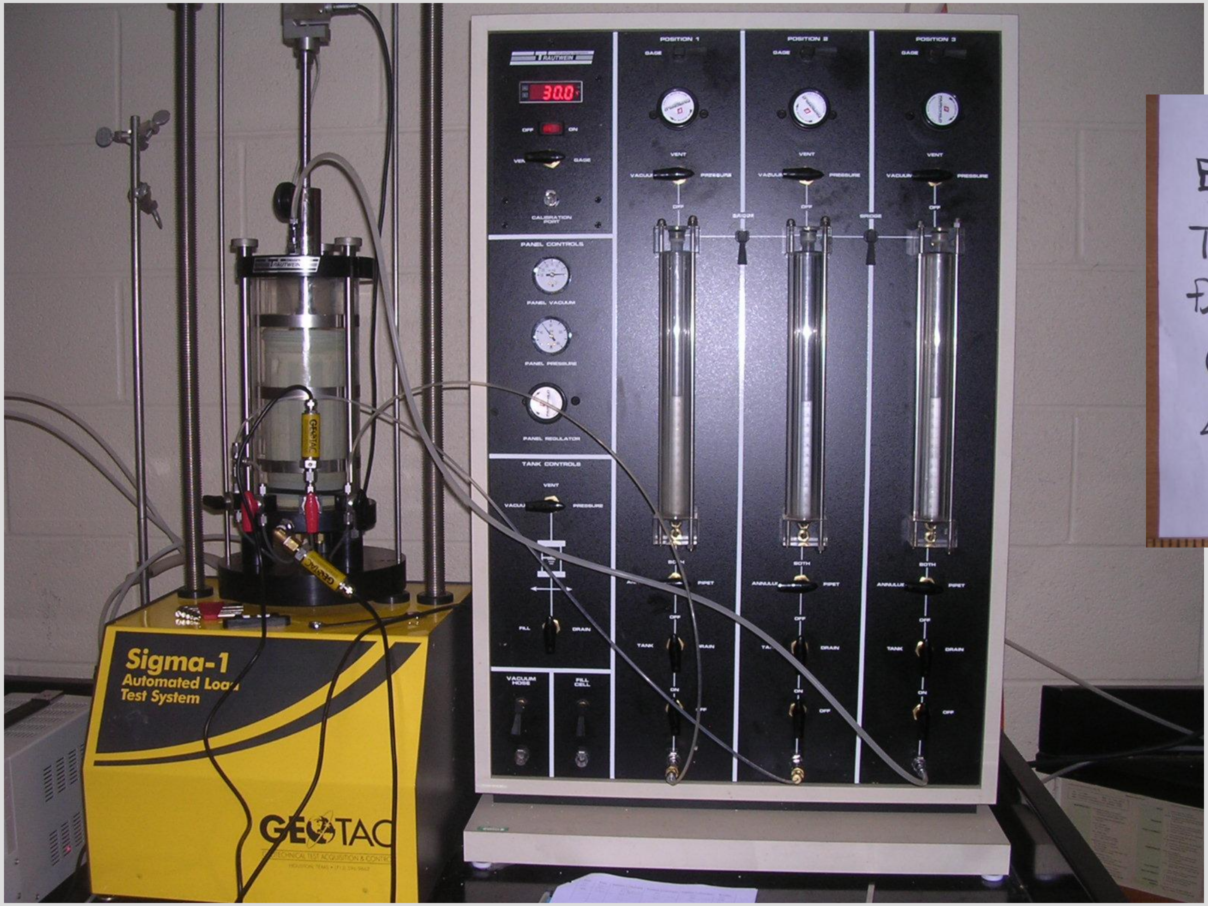


Mounting of Soil Specimen

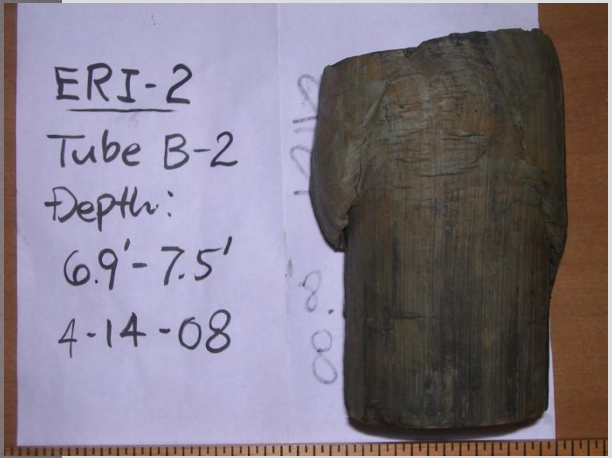


Soil Specimen Going Through Initial Saturation

Task 2



Triaxial Compression Test in Progress



Examination of Soil Specimen After Test

(Specimens usually do not exhibit clearly defined shear failure planes.)

Task 2

- Laboratory Test Results for Site 7 (HAN-75) – Part 1

Depth (ft)	Tube	G_s	LL (%)	PL (%)	PI (%)	Type
6.55	D-1	2.69	41	19	22	A-7-6
6.75	C-1	2.69	41	19	22	A-7-6
7.00	A-1	2.69	45	21	24	A-7-6
10.95	A-2	2.69	47	22	25	A-7-6
10.95	B-2	2.69	47	22	25	A-7-6
11.05	D-2	2.69	38	20	18	A-6b
17.45	A-3	2.68	39	19	20	A-6b
17.45	B-3	2.68	39	19	20	A-6b
17.65	D-3	2.68	39	19	20	A-6b

[Notes] Blue Color = A-7-6 Soils; Green Color = A-6b Soils
Gs = Specific gravity; LL = Liquid limit; PL = Plastic limit; PI = Plasticity index

Task 2

- Laboratory Test Results for Site 7 (HAN-75) – Part 2

Depth (ft)	Tube	%G	%S	%M	%C
6.55	D-1	2	19	32	46
6.75	C-1	2	19	32	46
7.00	A-1	3	16	33	48
10.95	A-2	1	16	32	50
10.95	B-2	1	16	32	50
11.05	D-2	1	19	36	44
17.45	A-3	3	17	34	47
17.45	B-3	3	17	34	47
17.65	D-3	3	17	34	47

[Notes] Blue Color = A-7-6 Soils; Green Color = A-6b Soils
%G = % Gravel; %S = % Sand; %M = % Silt; %C = % Clay

Task 2

- Laboratory Test Results for Site 7 (HAN-75) – Part 3

Depth (ft)	Tube	w (%)	γ_d (pcf)	q_u (psi)	$(N_{60})_1$
6.55	D-1	20.0	110.1	24.6	34
6.75	C-1	20.0	110.1	24.6	34
7.00	A-1	21.4	107.2	39.4	34
10.95	A-2	21.4	107.2	39.4	15
10.95	B-2	21.6	105.1	34.4	15
11.05	D-2	20.1	108.8	35.9	30
17.45	A-3	18.5	111.3	61.2	23
17.45	B-3	18.5	111.3	61.2	42
17.65	D-3	18.5	111.3	61.2	42

[Notes] Blue Color = A-7-6 Soils; Green Color = A-6b Soils
w = Moisture content; γ_d = Dry unit weight; q_u = Unconfined compr. strength

Task 2

- Laboratory Test Results for Site 7 (HAN-75) – Part 4

Depth (ft)	Tube	γ_d (pcf)	w_f (%)	σ_3 (psi)	σ_{1f} (psi)	u_f (psi)
6.55	D-1	111.9	21.4	55.0	90.2	32.8
6.75	C-1	113.5	22.1	47.1	80.5	27.7
7.00	A-1	113.0	22.2	40.0	71.5	22.3
10.95	A-2	110.7	23.7	41.9	70.3	26.0
10.95	B-2	112.6	22.1	48.9	75.9	32.1
11.05	D-2	NA	NA	NA	NA	NA
17.45	A-3	113.5	19.7	45.1	92.1	20.3
17.45	B-3	114.9	17.5	52.3	109.6	23.5
17.65	D-3	116.7	18.2	61.3	128.6	23.9

[Notes] 1. Blue Color = A-7-6 Soils; Green Color = A-6b Soils.
2. All triaxial test readings raised by backpressure of 30.0 psi.
3. w_f = Final moisture content; σ_3 = Chamber pressure; σ_{1f} = Major principal stress at failure; and u_f = pore water pressure at failure

Task 2

- Laboratory Test Results for Site 7 (HAN-75) – Part 5

Depth (ft)	Tube	t_{50} (min)	ε_f (%)	ϕ (deg)	ϕ' (deg)
6.55	D-1	60.0	15.0	14.0	26.2
6.75	C-1	46.0	15.0	15.2	27.6
7.00	A-1	19.0	15.0	16.4	28.0
10.95	A-2	40.0	15.0	14.7	28.2
10.95	B-2	36.0	15.0	12.5	26.5
11.05	D-2	NA	NA	NA	NA
17.45	A-3	9.0	15.0	20.0	29.1
17.45	B-3	9.3	15.0	20.7	30.2
17.65	D-3	NA	15.0	20.7	28.3

[Notes] Blue Color = A-7-6 Soils; Green Color = A-6b Soils
t_{50} = Time for 50% consolidation; and ε_f = axial strain at failure

Task 2

Soils Encountered at Highway Embankment Sites

Site 6 (ERI-2)
A-7-6 Soils

Site 7 (HAN-75)
A-6b & A-7-6 Soils

Site 2 (FAY-35)
A-4a & A-6a Soils

Site 1 (HAM-275)
A-7-6 Soils

Site 3 (LAK-2)
A-4a & A-6a Soils

Site 5 (MRW-71)
A-4a & A-6a Soils

Site 8 (MUS-70)
A-4b & A-6a Soils

Site 9 (NOB-77)
A-6a & A-6b Soils

Site 4 (ATH-33)
A-6a & A-7-6 Soils

Task 3

Empirical Correlations

- SPT-N Value vs. Unconfined Compression Strength q_u for Cohesive Soils – Terzaghi et al (1996)

SPT-$(N_{60})_1$	Stiffness	Unconfined Strength (psi)
< 2	very soft	< 3.6
2-4	soft	3.6-7.3
4-8	medium stiff	7.3-14.5
8-15	stiff	14.5-29
15-30	very stiff	29-58
> 30	hard	> 58

Task 3

<u>Empirical Correlations</u>

- SPT-N Value vs. Unconfined Compression Strength q_u for Cohesive Soils – Terzaghi et al (1996)

SPT-$(N_{60})_1$	Unconf. Strength (psi) Terzaghi: A-4	Unconfined Strength (psi) Measured: A-4
< 2	< 3.6	(No data)
2-4	3.6-7.3	(No data)
4-8	7.3-14.5	(No data)
8-15	14.5-29	45.1
15-30	29-58	19.1, 30.2, 30.3, 46.1, 48.9
> 30	> 58	20.8, 25.2, 41.0, 71.3, 79.0

[Note] Values in red are outside the Terzaghi range (45.5% outside).

Task 3

<u>Empirical Correlations</u>

- SPT-N Value vs. Unconfined Compression Strength q_u for Cohesive Soils – Terzaghi et al (1996)

SPT-$(N_{60})_1$	Unconf. Strength (psi) Terzaghi: A-6	Unconfined Strength (psi) Measured: A-6
< 2	< 3.6	(No data)
2-4	3.6-7.3	(No data)
4-8	7.3-14.5	(No data)
8-15	14.5-29	47.8
15-30	29-58	18.4, 20.8, 21.2, 25.8, 28.0, 30.3, 35.9, 61.2
> 30	> 58	20.2, 36.6, 38.0, 57.3, 61.2

[Note] Values in red are outside the Terzaghi range (57.1% outside).

Task 3

Empirical Correlations

- SPT-N Value vs. Unconfined Compression Strength q_u for Cohesive Soils – Terzaghi et al (1996)

SPT-$(N_{60})_1$	Unconf. Strength (psi) Terzaghi: A-7-6	Unconfined Strength (psi) Measured: A-7-6
< 2	< 3.6	(No data)
2-4	3.6-7.3	(No data)
4-8	7.3-14.5	(No data)
8-15	14.5-29	18.9, 21.3, 21.2, 24.3
15-30	29-58	16.9, 18.7, 24.8, 30.6, 39.4, 41.8
> 30	> 58	24.6, 39.4, 46.9

[Note] Values in red are outside the Terzaghi range (46.2% outside).

Task 3

Empirical Correlations

- SPT-N Value vs. Unconfined Compression Strength q_u – Dept. of Navy (1982)

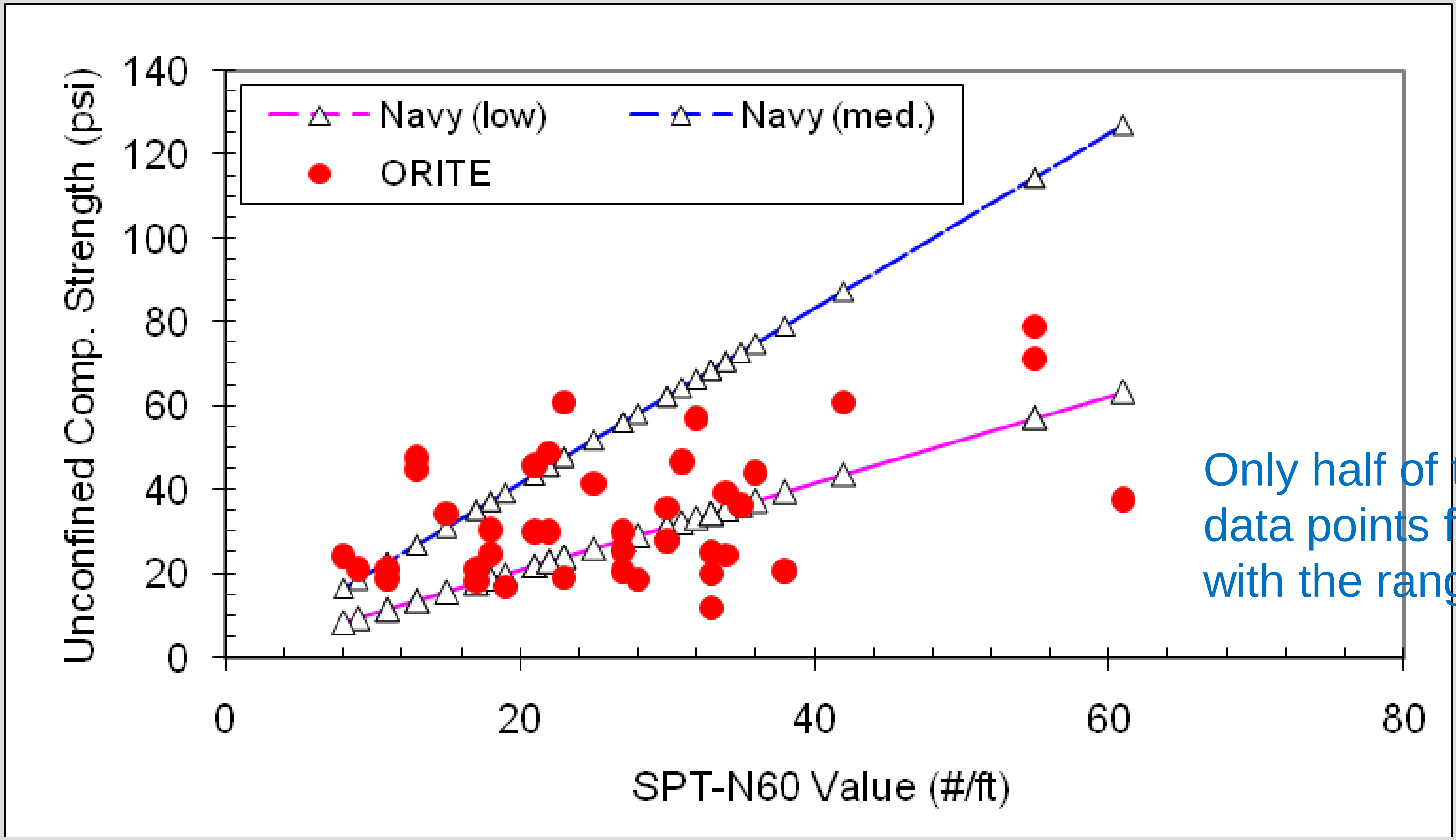
SPT-N_{60}	q_u of clays (low plasticity) & clayey silts	q_u of clays (med. plasticity)	q_u of clays (high plasticity)
5	5.2 psi	10.4 psi	17.4 psi
10	10.4 psi	20.8 psi	34.7 psi
15	15.6 psi	31.3 psi	52.1 psi
20	20.8 psi	41.7 psi	69.4 psi
25	26.0 psi	52.1 psi	86.8 psi
30	31.2 psi	62.5 psi	104.1 psi

[Note] Low Plasticity (LL < 40); Med. Plasticity (LL 40 to 60); and High Plasticity (LL > 60).

Task 3

SPT-N Value vs. Unconfined Compression Strength

q_u – Dept. of Navy (1982)



Only half of the data points fell with the range

Task 3

Empirical Correlations

- Soil Type vs. Compacted Unit Weight

Soil Type	Default - Navy $\gamma_{d\text{-}max}$ (pcf)	Actual - ORITE γ_d (pcf)
A-4	94 to 119	110 to 138 (ave. 125)
A-6	94 to 119	109 to 132 (ave. 119)
A-7-6	75 to 119	98 to 123 (ave. 107)

[Ref.] Design Manual 7.2 by U.S. Dept. of Navy (1982).

Task 3

Empirical Correlations

- Soil Type vs. Effective Friction Angle

Soil Type	ϕ' (deg.) Dept. of Navy	Number of Data Points - ORITE	ϕ' (deg.) Measured by ORITE
A-4	32	19	28.8 to 37.4 (Ave. 33.6)
A-6	28	31	28.3 to 37.8 (Ave. 32.7)
A-7-6	19-28	25	24.5 to 35.6 (Ave. 27.4)

[Ref.] Design Manual 7.2 by U.S. Dept. of Navy (1982).

Task 3

<u>Empirical Correlations</u>

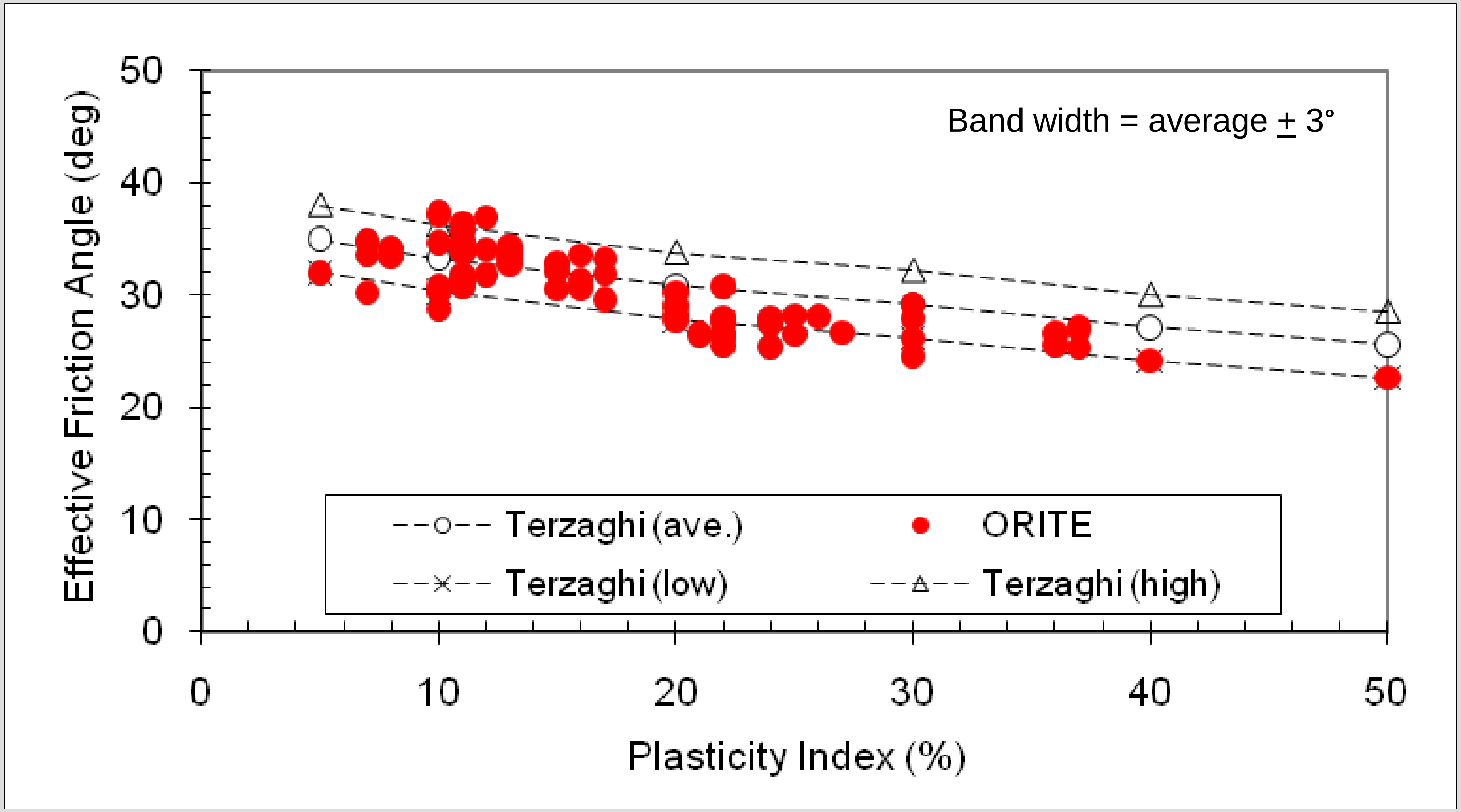
- Friction Angle ϕ' vs. Plasticity Index PI – Terzaghi et al. (1996)

PI (%)	ϕ' (deg)	PI (%)	ϕ' (deg)
10	33.3	50	25.6
20	30.8	60	24.6
30	29.2	70	23.8
40	27.1	80	23.1

[Note] The actual ϕ' value may be off by at least $\pm$ 3 degrees.

Task 3

Friction Angle ϕ' vs. Plasticity Index PI – Terzaghi et al. (1996)



Task 3

Friction Angle ϕ' vs. Plasticity Index PI – Terzaghi et al. (1996)

Soil Type	Results
All	55 (77%) out of 73 data points inside correlation band
A-4	PI = 7 to 13; 13 (68%) out of 19 data points inside band
A-6a	PI = 11 to 16; 20 (91%) out of 22 data points inside band
A-6b	PI = 16 to 20; 9 (100%) out of 9 data points inside band
A-7-6	PI = 21 to 37; 14 (61%) out of 23 data points inside band

- Standard deviation (σ) = 2.5
- More than half (64%) of measured values reside within Terzaghi's ave. ϕ' value $\pm$ 1σ.
- Most (96%) of measured values reside within Terzaghi's average ϕ' value $\pm$ 2σ.

Task 3

<u>Empirical Correlations</u>

- Soil Type vs. Soil Cohesion

Soil Type	Default – Navy $c_{\text{-moist}}$ (psi)	Actual – ORITE $c_{\text{-moist}}$ (psi)	Default-Navy $c'_{\text{-saturated}}$ (psi)	Actual-ORITE $c'_{\text{-saturated}}$ (psi)
A-4	9	5 to 22 (ave. 12)	1	1 to 8 (ave. 5)
A-6	12	7 to 20 (ave. 10.5)	2	2 to 9 (ave. 4)
A-7-6	12-15	8 to 23 (ave. 11)	2	1 to 6 (ave. 3)

[Ref.] Design Manual 7.2 by U.S. Dept. of Navy (1982).

Task 3

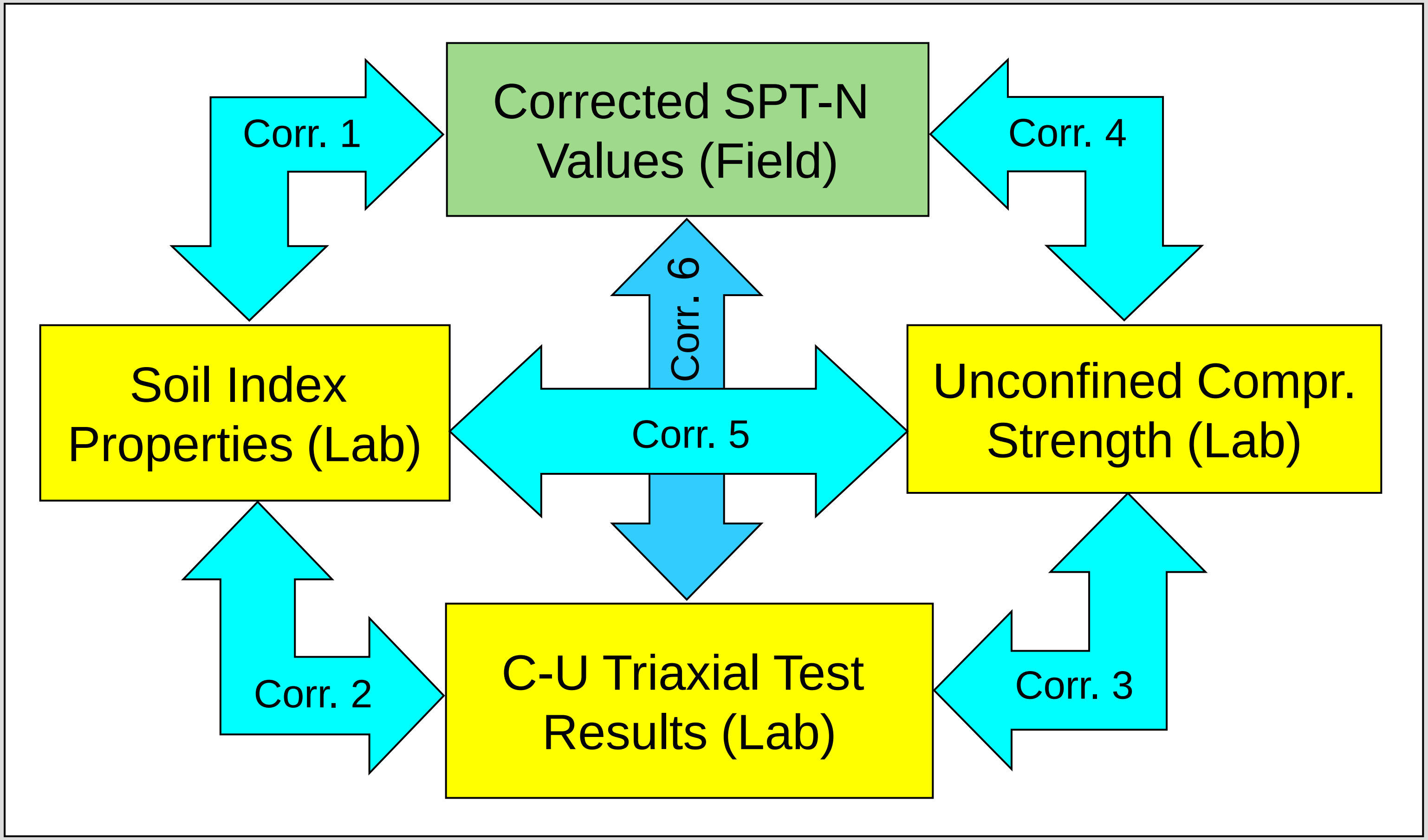


Diagram Showing Different Correlation Paths

Task 3

Field Test

- Original SPT-N values

  → Corrected SPT-N values N_{60} , $(N_{60})_1$

Soil Index Properties

- AASHTO soil classification
- Specific gravity (G_s)
- Moisture content (w)
- Dry unit weight (γ_d)
- Relative compaction (R_c)
- % gravel
- % sand
- % silt
- % clay
- Liquid limit (LL)
- Plastic limit (PL)
- Plasticity index (PI)

Task 3

Unconfined Compression Test

- Strength (q_u)
- Undrained cohesion (c_u) **Y**
- Moisture content
- Dry unit weight
- Relative compaction

C-U Triaxial Compression Test

- Dry unit weight
- Relative compaction
- Final moisture content
- Time for 50% consolidation (t_{50})
- Angle of internal friction (ϕ) **Y**
- Undrained cohesion (c_u) **Y**
- Effective angle of internal friction (ϕ') **Y**
- Effective cohesion (c') **Y**

Task 3

Single-Variable Models for Statistical Analysis

- Higher Polynomial → $Y = a_0 + a_1X + a_2X^2 + \ldots$
- Logarithmic → $Y = b + Ln(X)$
- Exponential → $Y = b\, e^{mX}$
- Power → $Y = b\, x^m$
- Hyperbolic → $Y = (b + mX)/X$
- Reciprocal → $Y = b + m(1/X)$

Task 3

- Single-Variable Regression Results for A-4a Soils

Independent Variable x	Model	Equation (coeff. of determination)
Time for 50% consolidation	Hyperbolic	$\phi = (24.19x - 0.556)/x$ ($r^2 = 0.923$)
% clay	Linear	$c_u = -1.469x + 55.38$ ($r^2 = 0.949$)
% gravel	Hyperbolic	$c_u = (15.97x - 24.36)/x$ ($r^2 = 0.939$)
% silt	2nd Polyn.	$c_u = -0.256x^2 + 22.05x - 454.72$ ($r^2 = 0.900$)
Time for 50% consolidation	Hyperbolic	$\phi' = (28.95x + 15.10)/x$ ($r^2 = 0.988$)
Plasticity index	Hyperbolic	$\phi' = (35.13x - 15.82)/x$ ($r^2 = 0.923$)
% clay	2nd Polyn.	$c' = -0.1655x^2 + 8.596x - 96.136$ ($r^2 = 0.989$)
Plasticity index	2nd Polyn.	$c' = -0.641x^2 + 13.28x - 60.08$ ($r^2 = 0.955$)

[Note 1] The above results are based on analysis of data from the all nine sites.

[Note 2] No single-variable regression analysis results are possible for A-4b soils due to a small sample size.

[Note 3] Units are – ϕ & ϕ'(degrees), c_u & c (psi), t_{50} (minutes), and PI (%).

Task 3

- Single-Variable Regression Results for A-6a Soils

Independent Variable x	Model	Equation (coeff. of determination)
Time of 50% consolidation	Hyperbolic	$\phi = (18.85x + 8.17)/x$ ($r^2 = 0.930$)
Unconf. compr. strength (q_u)	Hyperbolic	$\phi = (27.17x - 245.7)/x$ ($r^2 = 0.828$)
Specific gravity	2^{nd} Polyn.	$c_u = -1{,}846x^2 + 9975x - 13{,}459$ ($r^2 = 0.823$)
Time for 50% consolidation	Hyperbolic	$\phi' = (30.37x + 19.34)/x$ ($r^2 = 0.992$)
% gravel	Hyperbolic	$\phi' = (31.86x + 10.93)/x$ ($r^2 = 0.979$)
Liquid limit	Hyperbolic	$\phi' = (32.21x + 31.35)/x$ ($r^2 = 0.945$)
% sand	Hyperbolic	$\phi' = (38.13x - 108.5)/x$ ($r^2 = 0.927$)
Time for 50% consolidation	2^{nd} Polyn.	$c' = 0.165x^2 - 2.701x + 12.15$ ($r^2 = 0.979$)
% clay	2^{nd} Polyn.	$c' = -0.936x^2 + 57.40x - 873.1$ ($r^2 = 0.977$)
% gravel	2^{nd} Polyn.	$c' = -2.07x^2 + 22.63x - 55.84$ ($r^2 = 0.934$)
% silt	Linear	$c' = 1.380x - 49.71$ ($r^2 = 0.929$)

[Note 1] The above results are based on analysis of data from all nine sites.
[Note 2] Units are – ϕ & ϕ'(degrees), c_u & c (psi), t_{50} (minutes), q_u (psi), and LL (%).

Task 3

- Single-Variable Regression Results for A-6b Soils

Independent Variable x	Model	Equation (coeff. of determination)
% clay	Hyperbolic	$\phi = (32.42x - 563.5)/x$ ($r^2 = 0.988$)
Time for 50% consolidation	Hyperbolic	$\phi = (9.685x + 49.67)/x$ ($r^2 = 0.983$)
Plasticity index	Hyperbolic	$\phi = (53.46x - 660.9)/x$ ($r^2 = 0.966$)
% clay	2nd Polyn.	$c_u = -0.142x^2 + 10.96x - 190.8$ ($r^2 \approx 1.000$)
% gravel	2nd Polyn.	$c_u = 0.225x^2 - 5.468x + 37.43$ ($r^2 \approx 1.000$)
Plasticity index	2nd Polyn.	$c_u = -2.351x^2 + 85.94x - 768.7$ ($r^2 \approx 1.000$)
Time for 50% consolidation	Hyperbolic	$\phi' = (29.75x + 6.659)/x$ ($r^2 = 0.998$)
% gravel	Hyperbolic	$\phi' = (28.48x + 23.77)/x$ ($r^2 = 0.980$)
% clay	Hyperbolic	$\phi' = (25.56x + 178.1)/x$ ($r^2 = 0.956$)
Time for 50% consolidation	2nd Polyn	$c' = 0.186x^2 - 7.47x + 55.74$ ($r^2 \approx 1.000$)
Plastic limit	2nd Polyn	$c' = 2.391x^2 - 96.16x + 966.6$ ($r^2 \approx 1.000$)
% clay	2nd Polyn	$c' = -0.124x^2 + 9.403x - 163.5$ ($r^2 \approx 1.000$)

[Note 1] The above results are based on analysis of data from all nine sites.
[Note 2] Units are – ϕ & ϕ'(degrees), c_u & c (psi), t_{50} (minutes), q_u (psi), PI & PL (%).

Task 3

- Single-Variable Regression Results for A-7-6 Soils

Independent Variable x	Model	Equation (coeff. of determination)
% gravel	Hyperbolic	$\phi = (11.20x + 3.578)/x$ ($r^2 = 0.972$)
% sand	Hyperbolic	$\phi = (16.39x - 26.58)/x$ ($r^2 = 0.935$)
% gravel	Hyperbolic	$c_u = (6.293x + 2.951)/x$ ($r^2 = 0.827$)
Time for 50% consolidation	Hyperbolic	$\phi' = (26.14x + 36.55)/x$ ($r^2 = 0.994$)
% sand	Hyperbolic	$\phi' = (26.91x + 3.683)/x$ ($r^2 = 0.991$)
% gravel	Hyperbolic	$\phi' = (27.72x - 0.708)/x$ ($r^2 = 0.989$)
Plasticity index	Hyperbolic	$\phi' = (30.24x - 75.15)/x$ ($r^2 = 0.876$)
Uncomf. compr. strength (q_u)	2nd Polyn.	$c' = 0.145x^2 - 6.767x + 79.38$ ($r^2 = 0.876$)
% sand	Exponen.	$c' = 1.058\exp(0.097x)$ ($r^2 = 0.853$)

[Note 1] The above results are based on analysis of data from all nine sites.
[Note 2] Units are – ϕ & ϕ'(degrees), c_u & c (psi), t_{50} (minutes), and PI (%).

Task 3

- Multi-Variable Linear Regression Model

$$\boxed{Y = a_0 + a_1X_1 + a_2X_2 + \ldots + a_nX_n}$$

- Ranking of Correlations According to r^2 values
- Backward Scheme or Forward Scheme

SPSS

Task 3

- Multi-Variable Linear Regression Analysis Results

Soil Type	Independent Variables	Equation (coeff. of determination)
A-4a	% sand (x_1), dry unit weight (x_2)	$\phi' = 28.457 + 1.557x_1 - 0.282x_2$ ($r^2 = 0.726$)
A-6a	% gravel (x_1), moisture content (x_2)	$c' = 28.097 - 0.742x_1 - 0.999x_2$ ($r^2 = 0.954$)

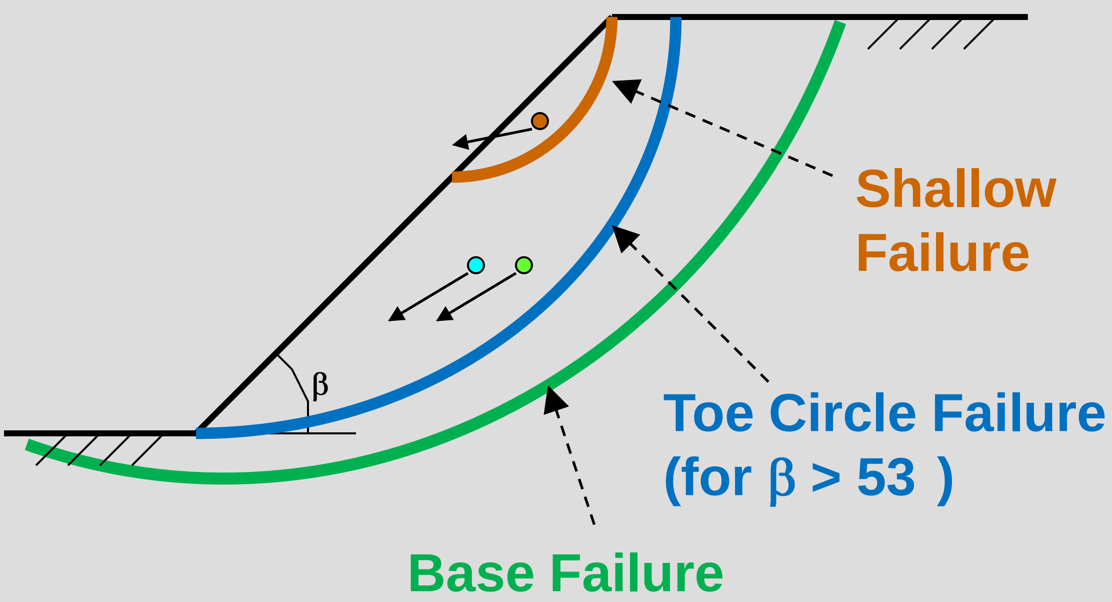
[Note] The above results are based on analysis of data from the all nine sites.

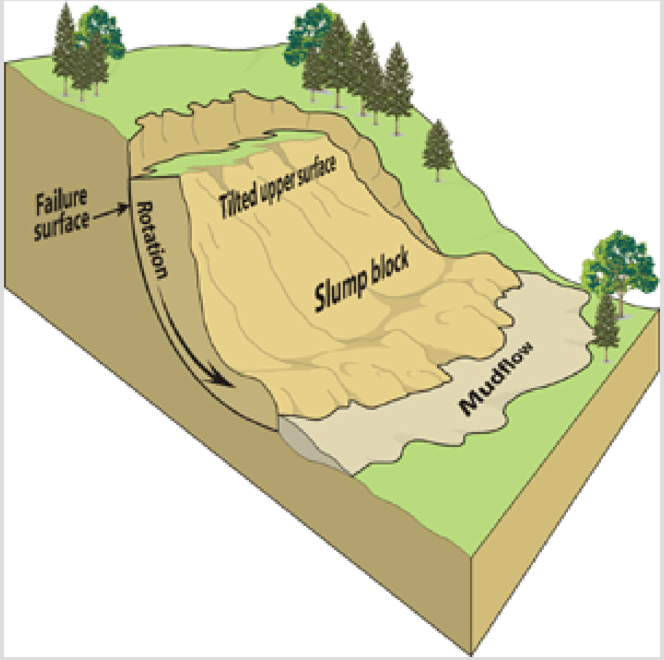
Only two results shown above were reasonable. All the other results were not meaningful due to multiple collinearity problems.

Multi-collinearity exists when there is a strong correlation between two or more predictors (independent variables). If there is perfect collinearity between predictors, it then becomes impossible to obtain unique estimates of the regression coefficients. There are simply infinite number of combinations of coefficients that would work equally well.

Task 4

- Slope Stability Analysis
- Three Different Types
- Approximately Circular Shape
- Short-Term & Lon-Term Analyses





Task 4

Stability of Highway Embankments

- Factors for stability – shear strength of embankment soil; unit weight of embankment soil; height of embankment; steepness of embankment slope; pore pressures in embankment soil; and shear strength of subsoil
- Stability of embankments on firm subsoils – Sliding of soil mass over firm base; Both short-term and long-term conditions are critical
- Stability of embankments on soft subsoils – Shear failure deep within soft subsoil layer; Short-term conditions are more critical

Task 4

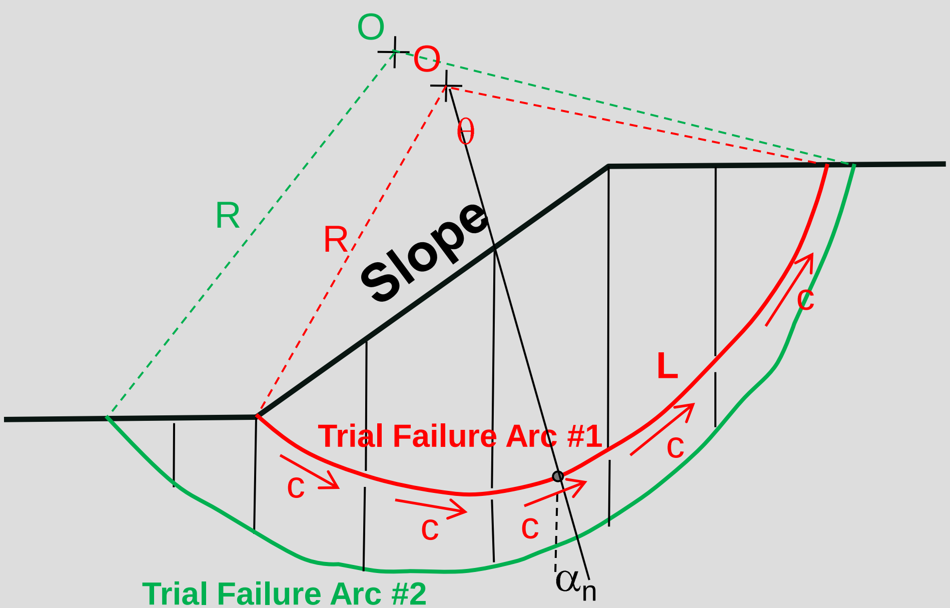
<u>Stability of Highway Embankments</u>

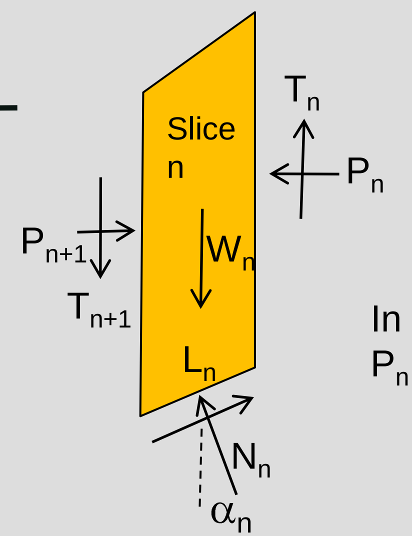
- Embankments constructed of a mixture of cohesive soils and rock fragments – Long-term stability may be a concern especially if the rock fragments were derived from sedimentary rock (ex. shale).

- Slope Stability Analysis by Method of Slices

$$F_S = \frac{\Sigma(cL_n + W_n\cos\alpha_n)\tan\phi}{\Sigma(W_n\sin\alpha_n)}$$

F_s = factor of safety; c = cohesion; L = total length of failure arc = Rθ; W_n = weight of slice n; α = angle of inclination for line connecting O and center of slice's bottom; ϕ = internal friction angle.





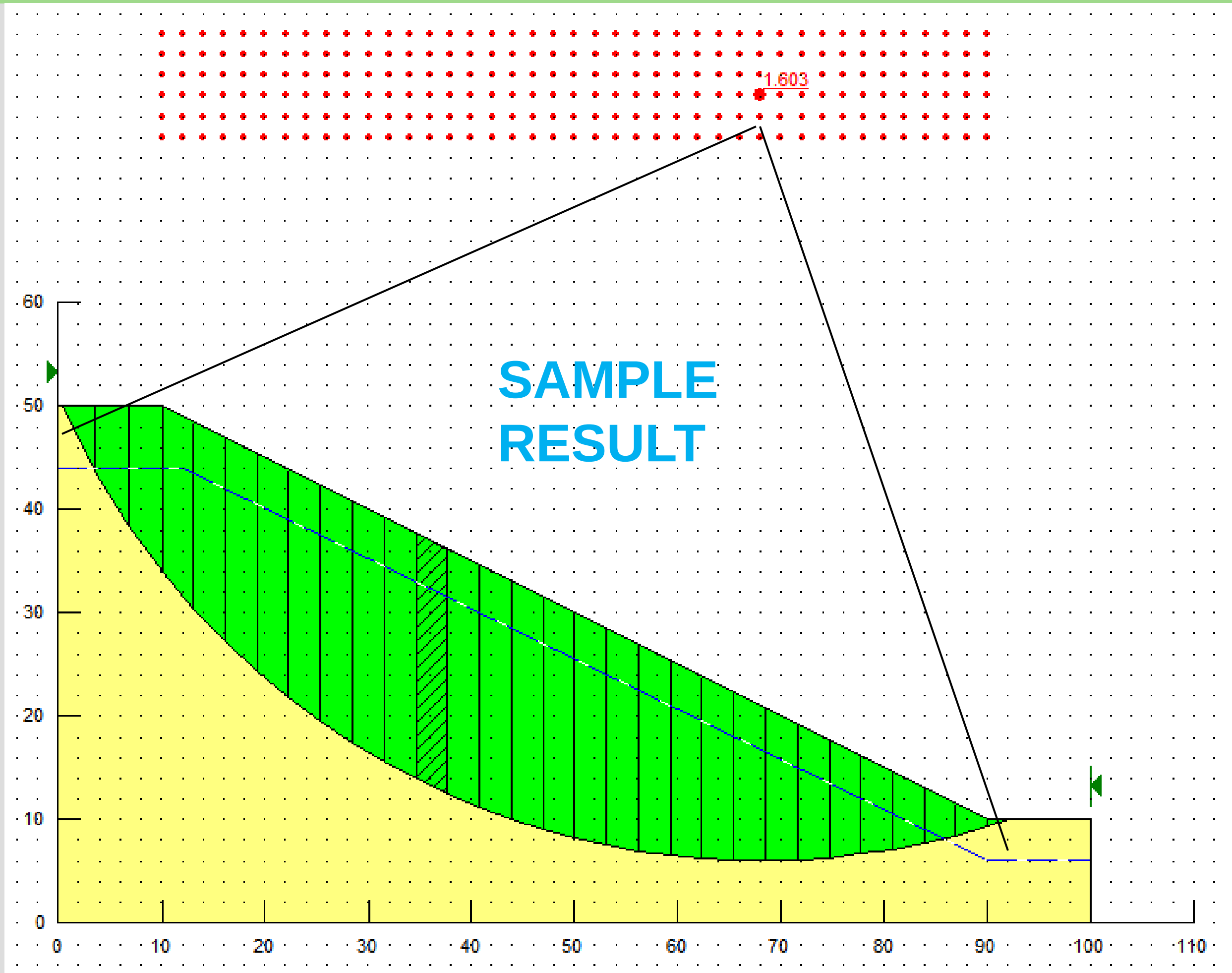
In ordinary method,
$P_n = P_{n+1}$ and $T_n = T_{n+1}$

Task 4

Slope Stability Analysis

- Computer Software GEOSLOPE
- Embankment Height -- 20, 30, and 40 ft
- Embankment Slope – 3H:1V (α = 18.4), 2.5H:1V (α = 21.8), and 2H:1V (α = 26.6)
- Same fill material extended below the embankment to form a foundation soil layer
- Short-Term (γ = moist unit weight; c_u; and ϕ = 0)
- Long-Term (γ = moist unit weight above water table; γ_{sat} = moist unit weight below water table; c'; and ϕ')

Task 4



40-ft (12.2-m) High Embankment in A-7-6 Soil, Slope 2H:1V, Long-Term

Task 4

- List of Average Soil Properties Used in Analysis

Type	γ (pcf)	γ_{sat} (pcf)	c_u (psi)	ϕ (º)	c' (psi)	ϕ' (º)
A-4a	121.2	138.5	12.1	24.4	4.90	33.4
A-6a	119.8	138.2	11.9	20.0	3.40	33.5
A-6b	119.6	137.9	8.90	15.4	4.50	30.8
A-7-6	104.5	128.2	5.80	12.9	3.30	27.4

- F_s Values for Homogeneous Embankments

Type	Slope 3H:1V	Slope 2.5H:1V	Slope 2H:1V
A-4a	4.64 (S); 3.84 (L)	4.40 (S); 3.53 (L)	4.24 (S); 3.24 (L)
A-6a	4.59 (S); 3.25 (L)	4.36 (S); 2.96 (L)	4.20 (S); 2.64 (L)
A-6b	3.38 (S); 3.50 (L)	3.21 (S); 3.22 (L)	3.09 (S); 2.92 (L)
A-7-6	2.38 (S); 2.82 (L)	2.26 (S); 2.60 (L)	2.18 (S); 2.35 (L)

Height = 20 ft

S = Short-Term

L = Long-Term

Task 4

- F_s Values for Homogeneous Embankments

Type	Slope 3H:1V	Slope 2.5H:1V	Slope 2H:1V
A-4a	3.38 (S); 3.08 (L)	3.16 (S); 2.79 (L)	3.00 (S); 2.49 (L)
A-6a	3.35 (S); 2.66 (L)	3.12 (S); 2.38 (L)	2.96 (S); 2.09 (L)
A-6b	2.46 (S); 2.81 (L)	2.30 (S); 2.55 (L)	2.18 (S); 2.27 (L)
A-7-6	1.74 (S); 2.26 (L)	1.62 (S); 2.05 (L)	1.54 (S); 1.82 (L)

Height = 30 ft

S = Short-Term

L = Long-Term

- F_s Values for Homogeneous Embankments

Type	Slope 3H:1V	Slope 2.5H:1V	Slope 2H:1V
A-4a	2.73 (S); 2.74 (L)	2.52 (S); 2.47 (L)	2.34 (S); 2.18 (L)
A-6a	2.70 (S); 2.40 (L)	2.49 (S); 2.15 (L)	2.32 (S); 1.87 (L)
A-6b	1.99 (S); 2.50 (L)	1.83 (S); 2.25 (L)	1.71 (S); 1.99 (L)
A-7-6	1.40 (S); 2.02 (L)	1.29 (S); 1.82 (L)	1.20 (S); 1.60 (L)

Height = 40 ft

S = Short-Term

L = Long-Term

Task 4

Conclusions

- The default soil property values available in the literature do not represent the average properties possessed by cohesive soil fills in Ohio very well.
- The empirical ϕ' vs. PI correlation published by Terzaghi et al. is applicable to A-4 and A-6 soils found in Ohio.
- The empirical q_u vs. $(N_{60})_1$ correlation published by the U.S. Dept. of Navy is not very reliable for cohesive soils in Ohio.

Task 4

Conclusions

- Many statistically strong single-variable correlations were identified for predicting shear strength properties of highway embankment fill soils.
- Very few linear multi-variable correlations surfaced for shear strength properties of Ohio embankment fill materials, due to multiple collinearity problems existing among the data set.

Task 4

Conclusions

- Highway embankment slopes made from A-4a soils exhibited the highest resistance against slope failure. Highway embankment slopes made from A-7-6 soils exhibited the lowest resistance against slope failure.

Task 4

- **Geotechnical Guidelines**
- Level 1: Short-Term Analysis
- Set $\phi = 0$. Use the following default undrained cohesion for each of the three major soil types found in Ohio:
- A-4a & A-4b soils …………………… c = 9 to 12 psi
- A-6a & A-6b soils …………………… c = 8 to 11 psi
- A-7-6 soils …………………………… c = 6 to 11 psi

Task 4

- **Geotechnical Guidelines**
- Level 1: Long-Term Analysis
- Use the following default shear strength parameter values for each of the four major soil types found in Ohio:
- A-4a soils …………………….. $\phi' = 32$; $c' = 3.8$ psi
- A-6a soils …………………….. $\phi' = 28$; $c' = 1.8$ psi
- A-6b soils …………………….. $\phi' = 28$; $c' = 1.6$ psi
- A-7-6 soils …………………….. $\phi' = 24.5$; $c' = 3.3$ psi

Task 4

- **Geotechnical Guidelines**
- Level 1: Long-Term Analysis (alternative)
- Determine liquid and plastic limits of the soil. Compute plasticity index (PI). Estimate the effective friction angle using the Terzaghi's empirical ϕ' vs. PI correlation chart. For A-4 and A-6 soils, use the average ϕ' value resulting from the chart. For A-7-6 soils, the lower the average ϕ' value shown in the chart by 2.5 (one standard deviation).

Task 4

- **Geotechnical Guidelines**
- Level 2: Short-Term & Long-Term Analyses
- Take advantage of some index property data available from laboratory tests. Use any of the correlation equations (w/ r^2 values > 0.8) previously shown for effective friction angle in:
- Slide #64 …………………. A-4a soils
- Slide #65 …………………. A-6a soils
- Slide #66 …………………. A-6b soils
- Slide #67 …………………. A-7-6 soils

Task 4

- **Geotechnical Guidelines**
- Embankment slopes built with A-6b soils should not be taller than 30 ft (9.1 m). Steepest slope shall be 2H:1V.
- Embankment slopes built with A-7-6 soils should not be taller than 20 ft (6.1 m). Steepest slope shall be 2H:1V.

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Thank you for listening to my presentation!

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