FOUNDATION ANALYSIS & DESIGN WORKSHOP
(FAD TOOLS)

John Chan
Anwar Hirany, PhD
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Anthony M. DiGioia, Jr., PhD, PE
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Event
June 14-16, 2011
AGENDA

June 15 (Tuesday)
8:00  Breakfast
8:15  Welcome & Introduction (John Chan)
8:30  Soil Mechanics Refresher and Subsurface Investigations (Anwar Hirany) (Session 1)
12:00 Lunch
1:00  Selection of Geotechnical Design Parameters (Tony DiGioia) (Session 2)
4:00  Workshop end for the day
6:00  Group Dinner

June 16 (Wednesday)
8:00  Breakfast
8:30  Foundation Design Approaches (Tony DiGioia) (Session 3)
10:30 Foundations for Single Poles (Tony DiGioia) (Session 4)
12:00 Lunch
1:00  Foundations for Single Poles (Session 4 Continued)
2:30  Foundations for Lattice Towers (Tony DiGioia) (Session 5)
4:30  Workshop end for the day
<table>
<thead>
<tr>
<th>Time</th>
<th>Event</th>
</tr>
</thead>
<tbody>
<tr>
<td>8:00</td>
<td>Breakfast</td>
</tr>
<tr>
<td>8:30</td>
<td>Foundations for Lattice Towers (Tony DiGioia) (Session 5 Continued)</td>
</tr>
<tr>
<td>10:00</td>
<td>Foundations for H-Frames (Tony DiGioia) (Session 6)</td>
</tr>
<tr>
<td>12:00</td>
<td>Lunch</td>
</tr>
<tr>
<td>1:00</td>
<td>Foundations for H-Frames (Tony DiGioia) (Session 6 Continued)</td>
</tr>
<tr>
<td>3:00</td>
<td>Feedback for Workshop (John Chan)</td>
</tr>
<tr>
<td>4:00</td>
<td>Workshop complete</td>
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</tbody>
</table>
SESSION 1

SOIL MECHANICS REFRESHER

Anwar Hirany, Ph.D., P.E. – EPRI
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972-556-6505
Overview

• Introduction
• Overview of geotechnical concepts
• Correlation of soil properties
• Overview of subsurface investigations
Introduction

• Foundation definition
• Foundation engineering
• Foundation types
Foundation Definition

A foundation can be described simply as the support system that rests on or in the ground and transfers to the ground the weight of the supported structure or equipment and the loads acting on it.

– e.g. simple peg to support camping tent OR complex system of very deep foundations to support off-shore drilling platforms
Foundation Engineering

Determination of:

1. Location
2. Size & shape
3. Structural details
4. Construction details

of a foundation that satisfies:
Three basic requirements

I. Safety from failure of soil/rock, foundation elements, structure

II. Freedom from objectionable deformation (settlement/heave, tilt, distortion)

III. Economics
1. Location

i. At minimum depth (frost, scour, soil movement)
ii. Additional depth to increase bearing capacity/reduce settlement
iii. Depth limitations because of high water table, rock, or adjacent development
iv. Change location because of adjacent structures, underground defects, etc.
v. Future influences
2. Size & Shape

i. Magnitude and configuration of loads

ii. Bearing capacity and settlement behavior under these loads

iii. Space available for foundation
3. Structural Details

i. Loads
ii. Bearing stress
iii. Distribution of bearing stress (uniform or not)
iv. Space available
v. Type of structure (steel, concrete, masonry)
4. Construction Details

i. How will it be built?
ii. Is soil treatment necessary?
iii. Will excavation be wet or dry?
iv. Will excavation walls need support?
Foundation Types

• Shallow (spread type)
  – spread footings
  – combined
  – raft/mat
  – strap/strip

• Deep (shaft type)
  – piles
  – drilled shafts
  – anchors
# Foundation Types

<table>
<thead>
<tr>
<th>Structure Type</th>
<th>Predominant Loading Mode</th>
<th>Spread</th>
<th>Drilled Shaft</th>
<th>Driven Pile</th>
<th>Direct Embedment</th>
<th>Rock anchor/socket</th>
<th>Anchors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lattice Tower</td>
<td>Uplift/Compression</td>
<td>X</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>Single Shaft</td>
<td>High Overturning Moment</td>
<td>X</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>Framed Structure</td>
<td>Overturning Moment w/ Uplift/Comp.</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Externally Guyed</td>
<td>Uplift (guys) Compression (fndn)</td>
<td>X</td>
<td></td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>
Foundation Engineering

• Basically foundation engineering is a trial and error process in which we seek to optimize design

• Often may have to consider many alternatives to satisfy:
  Safety
  Aesthetics
  Economics
Solution

• should be technically sound
• should minimize costs
• should allow construction to be completed within given time
Questions?
Overview of Geotechnical Concepts
Overview

- Soil types
- Definitions (weight-volume relationships)
- Effective stress
- Consolidation
- Stress history
- Shear strength parameters
Overview

• Geotechnical site assessment
• Importance of site assessment
• Foundation capacity
• Serviceability considerations
• Load testing
References
References


Soil Types

• Coarse grained (sand, gravel, cobbles)
• Fine grained (silt, clay)
• Organic (peat, organic silt/clay)
• Mixture (silty sand, sandy clay, SGC)
• Cemented/moisture sensitive
• Rock (various levels of weathering)
• Landfill (PCB, refrigerators)
Question

When water is added to soil, will its volume:

– increase
– decrease
– remain the same

ANSWER:
IT DEPENDS……
Alluvial Fan

Source: USGS
(http://libraryphoto.cr.usgs.gov/keyword.htm)
Alluvial Fan

Source: USGS
(http://libraryphoto.cr.usgs.gov/keyword.htm)
Question

In general, does the water flow faster in the vertical direction or in the horizontal direction within a soil mass?
Definitions
(Weight-Volume Relationships)
Definitions

\[ V = \text{Volume of soil} \]
\[ V_a = \text{Vol. of air} \]
\[ V_s = \text{Vol. of solids} \]
\[ V_v = \text{Vol. of voids} = V_a + V_w \]
\[ V_w = \text{Vol. of water} \]

\[ W = \text{Weight of soil} \]
\[ W_a = \text{Wt. of air} = 0 \]
\[ W_s = \text{Wt. of solids} \]
\[ W_w = \text{Wt. of water} \]
Definitions

• Moisture content ($\omega\%$)
  – weight of water/weight of solids ($W_w / W_s$)

• Void ratio ($e$)
  – volume of voids/volume of solids ($V_v / V_s$)

• Porosity ($\eta\%$)
  – volume of voids/total vol of soil ($V_v / V$)

• Degree of saturation ($S\%$)
  volume of water/volume of voids ($V_w / V_v$)
Definitions

• Specific gravity

\[ G_s = \frac{\text{weight of solids}}{\text{weight of equal vol. of water}} \]
\[ G_s = \frac{W_s}{V_s \cdot \gamma_w} \]  
[in which: \( \gamma_w = \text{unit wt. of water} = 62.4 \text{ pcf} \)]

• Unit weight (dry, moist, saturated)

\[ \gamma_{\text{dry}} = \frac{\text{weight of dry soil}}{\text{total vol of soil}} = \frac{W_s}{V} \]
\[ \gamma_{\text{moist}} = \frac{\text{weight of moist soil}}{\text{total vol of soil}} = \frac{(W_s + W_w)}{V} \]
\[ \gamma_{\text{sat}} = \frac{\text{weight of sat soil}}{\text{total vol of soil}} = \frac{(W_s + W_w)}{V} \]

• Relative density

\[ \left( \frac{e_{\text{max}} - e}{e_{\text{max}} - e_{\text{min}}} \right) \times 100\% \]
Effective Stress

- Shear strength of soil governed by effective stress
- Mathematically:

\[
\text{eff. stress} = \text{total stress} - \text{pore water pressure} \{\text{pwp}\}
\]
Effective Stress

• Applied external load initially taken up by water and effective stress remains unchanged
  \[ \text{eff. stress} = (\text{ini. stress} + \text{ext. load}) - (\text{ini. pwp} + \text{ext. load}) \]

• Additional pressure in water from external load (excess pwp) is above hydrostatic pressure and dissipates with time
Effective Stress

- Dissipation is immediate in coarse-grained soils; time-dependent in fine-grained soils
- Pore water pressure dissipation results in increase in effective stress
  \[(\text{eff. stress})_{\text{final}} = (\text{ini stress} + \text{ext load}) - (\text{pwp})_{\text{final}}\]
Consolidation

• Dissipation of excess pore water pressure results in water being removed from soil

• Removal of water causes volume change in soil

• This process is called consolidation
Stress History

• Soil behavior is governed by stress history
  – Soil has a “memory” - remembers the maximum load ever experienced

• Overconsolidation Ratio (OCR):
  \[ OCR = \frac{\text{max. vert. stress}}{\text{present vert. stress}} \]

• Normally consolidated (NC) soil OCR = 1

• Heavily overconsolidated (HOC) soil OCR > 4 to 8
Question

• When a soil is subject to load, does its strength INCREASE or DECREASE with time?

ANSWER:
IT DEPENDS……….
Stress History

• NC soil \((OCR=1)\) decreases in vol. when sheared
  – water is pushed out during shear
  – long-term (drained) strength > short-term (undrained) strength

• HOC soil \((OCR>4\ \text{to}\ 8)\) increases in vol. when sheared
  – water is sucked in during shear
  – long-term (drained) strength < short-term (undrained) strength
Question

Within a soil mass, is:

– horizontal stress = vertical stress
– horizontal stress < vertical stress
– horizontal stress > vertical stress

ANSWER:
IT DEPENDS………..
Stress History

• Horizontal stress coefficient $K_0$:
  
  $K_0 = \frac{\text{horizontal stress}}{\text{vertical stress}}$

• $K_0$ is function of stress history

• Generally $K_0$ increases as OCR increases

• Ranges in value from 0.3 to more than 3
Question

• Can OCR be less than 1?
Shear Strength Parameters

• For fine-grained soils need both undrained and drained parameters
  – undrained shear strength, $s_u$, or unconfined compressive strength, $q_c$, $[s_u = q_c/2]$
  – effective stress friction angle ($\phi'$)

• For coarse-grained soils generally need drained parameters only
  – effective stress friction angle ($\phi'$)

NOTE: in the absence of cementation
  $c' = 0$ for ALL soil types
Cohesive Sand???
Beach Soil Mechanics
Question

Why is the shear strength of soil expressed in terms of an angle ($\phi$) between two lines/planes?

OR

Why is the frictional resistance expressed in terms of an angle ("FRICTION ANGLE") between two lines/planes?
Geotechnical Site Assessment

- Collect existing subsurface data
- Develop preliminary geological model
- Perform field reconnaissance
- Evaluate data
- Supplement data with soil borings if needed
- Establish design values for soil parameters
Soil Parameters

- Water table
- Soil type
- Unit weight
- Shear strength parameters \((s_u/\phi)\)
- Stress history \((K_0\text{- horizontal stress coeff.})\)
Soil Parameters

- Pressuremeter modulus (lateral/moment)
- Corrosivity (pH, resistivity, chlorides, sulfates)
- Field density tests
- In situ tests (SPT, CPT, PMT)
- Index tests (Atterberg limits, moisture content)
# Variability of Soil Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Normal Range</th>
<th>COV (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Unit weight</strong></td>
<td>80 – 130 pcf</td>
<td>&lt;10</td>
</tr>
<tr>
<td><strong>Friction angle</strong></td>
<td>20 – 40 deg</td>
<td>5 - 15</td>
</tr>
<tr>
<td><strong>Undrained shear strength</strong></td>
<td>0.1 – 4 tsf</td>
<td>20 - 55</td>
</tr>
<tr>
<td><strong>$K_o$ (horiz. stress/vert. stress)</strong></td>
<td>0.3 – 4</td>
<td>15 - 55</td>
</tr>
<tr>
<td><strong>PMT modulus</strong></td>
<td>50 – 150 tsf</td>
<td>15 - 65</td>
</tr>
</tbody>
</table>
Importance of Geotechnical Assessment

• Uplift resistance of drilled shaft in sand:

\[ Q_{\text{soil}} = 0.5 \gamma D^2 K_0 \tan(\phi) \pi B \]

in which:
- \( D = \) foundation depth = 15 ft
- \( B = \) foundation diameter = 4 ft
- \( \gamma = \) soil unit weight = 80 to 130 pcf
- \( \phi = \) soil friction angle = 20 to 40 deg
- \( K_0 = \) in situ horizontal stress coeff. = \((1 - \sin\phi)\) to 4
  (ground water depth = 30 ft.)
## Importance of Geotechnical Assessment

### Uplift resistance from soil

<table>
<thead>
<tr>
<th>Uplift resistance from soil</th>
<th>Lower bound (ref)</th>
<th>Change γ only</th>
<th>Change φ only</th>
<th>Change Ko only</th>
<th>Upper bound (change all)</th>
</tr>
</thead>
<tbody>
<tr>
<td>γ = 80 pcf, φ = 20 deg, Ko = (1-sin φ)</td>
<td>γ = 130 pcf, φ = 20 deg, Ko = (1-sin φ)</td>
<td>γ = 80 pcf, φ = 40 deg, Ko = (1-sin φ)</td>
<td>γ = 80 pcf, φ = 20 deg, Ko = 4</td>
<td>γ = 130 pcf, φ = 40 deg, Ko = 4</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Q (kips)</th>
<th>27</th>
<th>44</th>
<th>34</th>
<th>165</th>
<th>617</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q/Q_{ref}</td>
<td>1</td>
<td>1.6</td>
<td>1.3</td>
<td>6.1</td>
<td>22.8</td>
</tr>
</tbody>
</table>

Q = \text{Uplift resistance from soil}
Foundation Capacity

- Compression
- Uplift
- Lateral/moment
- Torsional
- Combined
Foundation Capacity

• Deterministic
  – most prevalent
  – number of different methodologies
• Probabilistic (reliability based)
  – relatively new concept in foundation design
Foundation Capacity

• Undrained condition
  – short term transient loads
  – generally not applicable to coarse grained soils (sand)
  – generally governs if soil is normally consolidated (OCR=1)

• Drained condition
  – long term sustained loading
  – generally governs if soil is heavily overconsolidated (OCR>4)
Serviceability Considerations

- Strength
- Duration of critical load combinations
- Tolerable movements
- Design Philosophy (generalization)
  - soil - least reliable component
  - foundations - capacity probably okay if movement governs
  - structures - don’t have foundation movements cause failure
Load Testing

- Short term conditions
- Requires experienced personnel/contractor
- Interpretation of results requires experience
- Not inexpensive to conduct
Load Testing

• Proof of the pudding?
  – Valid for prevailing conditions at time of test
  – Valid for specific test location
  – Results affected by loading rate
Questions?
Correlation of Soil Properties
Correlation of Soil Properties

• Soil is a complex engineering material formed by various geologic, environmental, and chemical processes
• Many of these processes are continuing and modifying the soil in-situ
• Soil properties are not unique or constant and vary vertically and horizontally
Correlation of Soil Properties

• Properties vary with many environmental factors:
  – time, stress history, water table fluctuations, decomposition

• Because of complexity of soil behavior, empirical correlations are used extensively to evaluate soil parameters

• Property estimates are made most often by correlations to results of laboratory index tests and in-situ tests:
  – calibration studies
  – back-calculation from full-scale load test data
Correlation of Soil Properties

- Comprehensive characterization of soil at a site requires elaborate and costly testing program
- Design engineer has limited soil information and therefore must rely upon correlations
CAUTION!!!!!

• Cautious of using broad, generalized correlations
• Source, extent and limitations of each correlation should be examined carefully
• Ensure extrapolation not being done beyond original boundary conditions
• Prefer “LOCAL” calibrations where available
CAUTION!!!!!
\[
\frac{E_{\text{PMT}}}{p_0} = 19.3 N^{0.63}
\]

\((n=443, r^2=0.393, \text{S.D.}=61.4 p_0)\)

- **Clay**
  - Alluvial
  - Diluvial

- **SPT N Value**
  - Tokyo
  - Nagoya
  - Osaka
  - Sakaide
CAUTION!!!!!!

• Many common correlations have been developed for:
  – insensitive clays of low to moderate plasticity
  – unaged quartz sands reconstituted in the laboratory
• Extrapolation to “special” soils not recommended because correlations may not be applicable
“Special” Soils

- very soft clays
- organic clays
- sensitive clays
- expansive clays
- fissured clays
- cemented soils
- calcareous sands
- micaceous sands
- collapsible soils
- frozen soils
Test Variability

• Variability of soil property also may be introduced by type of laboratory/in-situ test

• Each test will provide different test result because of differing boundary conditions and loading mechanism
Test Variability

Laboratory Strength Tests

TC  TE  DS  DSS  PSC  PSE

Field Tests

SPT  CPT  PMT  DMT  VST

SYMBOLS:
TC - triaxial compression
TE - triaxial extension
DS - direct shear
DSS - direct simple shear
PSC - plane strain compression
PSE - plane strain extension

SPT - standard penetration test
CPT - cone penetration test
PMT - pressuremeter test
DMT - dilatometer test
VST - vane shear test
Test Boundary Conditions (Ref: EPRI EL-6800)

Note: Plane strain tests (PSC/PSE) used for long features
Triaxial tests (TC/TE) used for near symmetrical features
Direct shear (DS) normally substituted for DDS to evaluate $\phi$

Figure 4.6. Relevance of Laboratory Strength Tests to Field Conditions
Variability
Important Properties for Foundation Design

• Unit Weight ($\gamma$)/Relative Density
• Friction angle ($\phi$)
• Undrained Shear Strength ($s_u$)
• Horizontal Stress Coefficient ($K_0$)
• Modulus of Elasticity ($E$)
Reference

SESSION 2

SUBSURFACE INVESTIGATIONS

Anwar Hirany, Ph.D., P.E. - EPRI
Subsurface Investigation

• Foundation Design Requirements
  – must be stable
  – must not move excessively
  – must be economical or at least be cost-effective

The above can only be accomplished if we can evaluate ground conditions correctly and adequately
Exploration Purpose

1. Select type and depth of suitable foundation
2. Evaluate load bearing capacity
3. Estimate settlement
4. Determine potential problems (expansive soil, etc.)
5. Establish construction procedures
Exploration Requirements

DETERMINE:
1. Nature of deposits (geology, recent history of filling/excavation, etc.)
2. Depth, thickness, composition of each stratum
3. Location and variation of ground water
4. Engineering properties of each stratum

Extent of exploration will depend upon project size
Exploration Costs

• Generally between 0.05 to 0.2 percent of total structure cost
• May go up to 0.5 to 1.5 percent for complex structures like dams, bridges, power plants
Strategy for Site Characterization

• Need to develop an optimal exploration program to promote the most cost-effective design

• To accomplish this:
  – weigh costs of exploration versus savings in foundation design that may result from more detailed information
Qualitative Exploration Model

- Exploration cost increases
  - with level of exploration
  - with less efficient methods
- Foundation cost decreases
  - with level of exploration (or more reliable techniques)
- Combine above to optimize
Consider Extremes

• It is expensive (or impossible) to determine all geotechnical data precisely
• Foundations are over designed for worst conditions (i.e. no exploration)
• Proper site characterization will eliminate extremes
Exploration stages

- **Existing Information**
  - Collect Existing Data (physiography, regional geology, other)
  - Analyze Data
  - Prepare Engineering Geologic Map

- **Geologic Reconnaissance**
  - Perform Aerial & Ground Field Reconnaissance
  - Revise Engineering Geologic Map

- **Preliminary Data Evaluation**
  - Estimate Soil/Rock Type & Depth
  - Assess Potential Geologic Problems
  - Determine Gaps in Data

- **Subsurface Investigation**
  - Test Borings: In Situ & Lab Testing
  - Geophysical Testing
Existing Information

Quick and economic way of collecting information on general site conditions from:

- Regional geologic data
- Topographic maps
- Remote sensing
- Existing information from adjacent sites
Regional Geologic Data
Regional Geologic Data

• Physiography
  – genesis and formation of landforms
  – correlate landforms to materials of which they are composed

• Surficial geology
  – geology of soil deposits in upper few feet of the subsurface

• Bedrock geology
  – rocks present in an area
Regional Geologic Data

• Data useful for route selection of transmission lines, highways, railroads, etc.
• Determine areas of problematic conditions such as flood plains, collapsing/expansive soils, karst topography
Topographic Maps

• Available from USGS
• Can identify landforms, cultural features, floodplains, karst topography, etc. (1:24,000)
Topographic Maps

Source: USGS
Remote Sensing

• Black and white vertical aerial photographs
• Infrared and multispectral photographs and images
• High altitude photographs
• Satellite images
• Others

Source: USGS
Remote Sensing

Source: USGS
Aerial Photographs

• Show terrain, ground cover, and adjacent land use
• Also reveal probable foundation conditions
  – identify landforms by:
    • topography, drainage patterns, erosion, tone (coloration), vegetation and cultural features
  – perform landform analysis to determine:
    • residual soils, aeolian soils, waterlaid soils, etc.
Existing Information

- Public records of cities
- Building owners on adjacent sites
- Adjacent railroads, highways, utilities
- Contractors
Geologic Reconnaissance

Quick and economic way of collecting information on general site conditions from:

- Ground geologic surveys
- Aerial reconnaissance (transmission lines, etc.)
Ground Geologic Surveys

• Always important to make visual inspection of site
• Get detailed information on
  – landforms, topography, soil and rock conditions
  – drainage ditches, creep, shrinkage cracks, earth fissures
  – vegetation
  – construction on nearby sites, roadway damage
Aerial Reconnaissance

- Generally used for transmission lines, highways, etc.
- Helicopter or fixed wing
- Helicopter has advantage of slow speed and close-in observation
Field Exploration (site investigation)

• Perform drilling and sampling/geophysical testing
• Obtain site specific data
• Data obtained:
  – geometric (from drilling, sampling, etc.)
  – property (from in-situ tests)
  – observational (water levels, voids, etc.)
• All data must be recorded at time of observation
• Need qualified personnel to observe and record data
Geophysical Testing

• Useful for determining preliminary properties and geometries of subsurface materials
• Most useful when correlated with nearby borings
• Cannot be used to measure geotechnical properties per se
  – measure some physical characteristic (seismic velocity/resistivity)
  – these are correlated to infer subsurface conditions
Geophysical Testing

• Many techniques exist, but two most applicable to geotechnical investigations
• Seismic Refraction
• Electrical Resistivity
• Without aid of nearby borings for correlations, these should be considered as preliminary exploration
Drilling and Sampling

• General Planning
  – data required
  – depth of investigation
  – number and spacing of borings
Drilling and Sampling

• Boring Depth:
  – borings should be extended to a firm stratum or at least to a depth at which the stress increase from the foundation is less than 10% of the in-situ effective stress
  – for bedrock, drill through at least 5 to 10 ft (depending upon core recovery)
Drilling and Sampling

• Boring Spacing (rule of thumb)
  – multi-story buildings 30 – 100 ft
  – one-story factories 60 – 200 ft
  – dams 100 – 200 ft
  – highways 1000 – 2000 ft
  (spacing decreased for highly varying deposits)

– transmission lines:
  IT DEPENDS!!!
Drilling and Sampling

• Purpose:
  – advance a hole
  – obtain a sample
• Preliminary methods
  – hand augers (for shallow exploration)
  – probes (to determine “refusal”)
  – wash borings (“dated” procedure)
  – machine auger borings (hollow stem/flight)
• Detailed methods
Preliminary Methods

• Hand augers
  – depths of 10 to 15 ft
  – inexpensive
  – samples for classification purpose only

• Probes
  – drill rod driven into soil
  – dynamic cone penetrometer
  – determine bedrock undulations
  – not very reliable
Preliminary Methods

• Wash borings
  – advanced by chopping bits equipped with holes for water jets
  – water pushes chopped soil particles to the surface
  – not in common use anymore
Preliminary Methods

• Machine augers
  – continuous flight augers
  – commonly used to advance borings
  – generally auger has hollow stem for inserting sampling/testing tools
  – serves to advance hole and carry cuttings to surface AND cases the hole to prevent caving
Detailed Methods

- Include three steps
  - drill through soil and keep hole open
  - recover samples
  - perform in-situ tests
Detailed Methods

• Drilling accomplished by
  – hollow-stem flight augers, driven casing, or rotary drilling (need drilling mud to keep hole open)

• Recover samples by
  – Spoon sampling (driven – disturbed)
  – Tube sampling (pushed – “undisturbed”)
  – Coring (rotary drilling/sampling)

• In-situ testing (SPT, PMT, VST, etc.)
Split Spoon Sampler

• Most commonly used
• Also used for Standard Penetration Test
• Retrieves intact but disturbed sample
• Driving resistance yields standard penetration N value (discussed later)
Split Spoon Sampler

- Drive Weight 140°
- 30° Free Fall
- Drill Rod
- Drive Head
- Drive Pipe
- Drive Coupling
- Drive Shoe

Split Tube Sampler in Undisturbed Soil
Disturbed Samples

• Can be used for
  – grain-size analysis
  – determination of Atterberg limit
  – specific gravity analysis
  – organic content determination
  – soil classification
Tube Sampling

• Relatively undisturbed samples can be obtained from samplers that are pushed into the soil
• Thin walled tubes generally used for retrieving fine-grained soils for laboratory testing
• Sample tubes are sealed at both ends with wax and shipped to laboratory
Sample Disturbance

• Squeezing/heaving minimized by keeping hole filled with fluid (water or drilling mud)
• Seal samples immediately, pack in cushioned container, protect from temperature extremes, and transport carefully
Tube Samplers

- Tube samplers used for fine-grained soils
- Difficult to obtain undisturbed sample of sands (specially below water table)
  - can use freezing techniques, but that changes void ratio
  - Bishop sampler – force air through an outer tube to evaporate some moisture creating capillary tension to hold sample together
Undisturbed Samples

- Used for
  - consolidation testing
  - permeability testing
  - shear strength testing
Coring

- Used when rock is encountered
- Rotary drilling process is employed
- Samples are retrieved in a core barrel
  - single-tube barrel
  - double-tube barrel
- Good quality sampling from double-tube barrels of N size (nominal 2 in. dia. core) or larger
Data Recording

- Data recorded on boring logs
- Many boring log formats exist
- Information recorded should be as complete and accurate as possible
<table>
<thead>
<tr>
<th>DEPTH (Ft)</th>
<th>SOIL SAMPLING</th>
<th>ROCK CORING</th>
<th>PROFILE IDENTIFICATION, DESCRIPTION, REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sample No.</td>
<td>Blows per 6in.</td>
<td>Blows per 6in.</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>35</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Symbols:
- G: Gravel
- S: Sand
- S: Silt
- C: Clay

Proportions:
- trace (t) 1-10 %
- little (l) 10-20 %
- some (s) 25-30 %
- and (a) 35-50 %

Field classification:
- < 5% fines: GW, GP, SW, SP
- > 12% fines: GM, GC, SM, SC
- W: well graded, P: poorly graded
CHECK LIST OF BORING INFORMATION NEEDED

Project, location, and personnel (contractor, driller, inspector)

Date and time to start and finish hole

Boring number, location, and surface elevation¹

Type of drilling equipment used, including rig type and drilling bits

Method of advancing hole (hollow stem auger, etc.) and sizes of hole and casing or auger; note use of drilling mud

Ground water elevation(s) and observation date(s) and time

For driven casing
   -- size of drive weight, free fall distance, and method of operation
   -- blows per foot to advance casing

For split spoon sampling
   -- description and size of sampler
   -- size and type of drive hammer, free fall distance, and method of operation
   -- blows per 6 in (300 mm) to drive sampler one foot (600 mm) (SPT N value)

For thin-wall tube sampling
   -- description and size of sampler
   -- stress or force to push sampler²
   -- sample recovery and distance sampler pushed

For rock coring
   -- description and size of core barrel
   -- water pressure, feed rate, bit pressure, and rotation rate²
   -- core recovery and RQD¹

Upper and lower elevation, description, and classification of each sample or core¹

Appropriate details for in-situ tests such as vane shear¹

Results of simple tests such as pocket penetrometer¹

Remarks and miscellaneous conditions
   -- loss of circulating water or sample³
   -- occurrence of boulders, cavities, or voids³


¹ Requires trained personnel.

² Useful but not mandatory.

³ Essential but often overlooked and not recorded.
In-Situ Test Methods

• Standard Penetration Test (SPT)
• Cone Penetration Test (CPT)
• Vane Shear Test (VST)
• Pressuremeter Test (PMT)
• Other
# USEFULNESS OF IN-SITU TESTS IN COMMON SOIL CONDITIONS

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Sand</th>
<th>Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Gravel</td>
<td>Loose</td>
</tr>
<tr>
<td>SPT</td>
<td>M-L</td>
<td>H</td>
</tr>
<tr>
<td>MCPT</td>
<td>M-L</td>
<td>H</td>
</tr>
<tr>
<td>ECPT</td>
<td>L</td>
<td>H</td>
</tr>
<tr>
<td>CPTU</td>
<td>L</td>
<td>H</td>
</tr>
<tr>
<td>VST</td>
<td>N</td>
<td>N</td>
</tr>
<tr>
<td>DMT</td>
<td>L</td>
<td>H</td>
</tr>
<tr>
<td>BMT</td>
<td>M</td>
<td>M</td>
</tr>
<tr>
<td>SBPMT</td>
<td>L</td>
<td>M</td>
</tr>
<tr>
<td>GEOPH*</td>
<td>H</td>
<td>H</td>
</tr>
</tbody>
</table>


Notes:  
H - Highly applicable  
M - Moderately applicable  
L - Limited applicability  
N - Not applicable

SPT - Standard Penetration Test  
CPT - Cone Penetration Test  
MCPT - Mechanical CPT  
ECPT - Electrical CPT  
CPTU - Piezometric CPT  
VST - Vane Shear Test  
DMT - Dilatometer Test  
PMT - Pressuremeter Test  
SBPMT - Self-Boring PMT  
GEOPH - Geophysical Methods

*Note that these are profiling methods which do not give soil properties.
**Relative Applicability of In-Situ Tests to Obtain Soil Parameters for Foundation Design**

<table>
<thead>
<tr>
<th>Test</th>
<th>Soil Identification</th>
<th>Soil Profile</th>
<th>$\phi$</th>
<th>$s_u$</th>
<th>$K_o$</th>
<th>$E$</th>
<th>$C_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPT</td>
<td>H</td>
<td>M</td>
<td>M</td>
<td>M</td>
<td>L</td>
<td>N</td>
<td>M</td>
</tr>
<tr>
<td>MCPT</td>
<td>M</td>
<td>H</td>
<td>M-L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>N</td>
</tr>
<tr>
<td>ECPT</td>
<td>M</td>
<td>H</td>
<td>M</td>
<td>M</td>
<td>L</td>
<td>M</td>
<td>M</td>
</tr>
<tr>
<td>CPTU</td>
<td>H</td>
<td>H</td>
<td>M</td>
<td>M</td>
<td>M</td>
<td>M</td>
<td>M</td>
</tr>
<tr>
<td>VST</td>
<td>L</td>
<td>L</td>
<td>N</td>
<td>H</td>
<td>L</td>
<td>N</td>
<td>N</td>
</tr>
<tr>
<td>DMT</td>
<td>M</td>
<td>H</td>
<td>M-L</td>
<td>L</td>
<td>M</td>
<td>M</td>
<td>N</td>
</tr>
<tr>
<td>PMT</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>M</td>
<td>L</td>
<td>H</td>
<td>L</td>
</tr>
<tr>
<td>SBPMT</td>
<td>M</td>
<td>L</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>H</td>
</tr>
<tr>
<td>GEOPH</td>
<td>L</td>
<td>M</td>
<td>N</td>
<td>N</td>
<td>N</td>
<td>N</td>
<td>N</td>
</tr>
</tbody>
</table>

**Source:** Modified after Campanella and Robertson (53), p. 25.

**Notes:**
- **H** - Highly applicable
- **M** - Moderately applicable
- **L** - Limited applicability
- **N** - Not applicable
- $\phi$ - Effective stress friction angle
- $s_u$ - Undrained shear strength
- $K_o$ - In-situ horizontal stress coefficient
- $E$ - Elastic (Young's) modulus
- $C_c$ - Compressibility index

**NOTE:** These parameters are interpreted; you DO NOT obtain direct values from in-situ tests
Figure A-4. Equipment Used to Perform the SPT

Source: Kovacs, et al. (6), p. 3.
Standard Penetration Test

• Detailed procedure described in ASTM D 1586
  – Sampler connected to drill rods placed at bottom of hole
  – 140 lb weight dropped 30 in. to collar attached to drill rods until 18 in. penetration is achieved (or 100 blows)
  – Number of blows for each 6 in. interval is recorded
  – Number of blows for final 12 in. is reported as N value
## Major Sources of Error in the Standard Penetration Test

<table>
<thead>
<tr>
<th>Cause</th>
<th>Effect</th>
<th>Influence on N Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inadequate cleaning of hole</td>
<td>SPT is not made in original in-situ soil, and therefore soil may become trapped in sampler and be compressed as sampler is driven, reducing recovery</td>
<td>Increases</td>
</tr>
<tr>
<td>Failure to maintain adequate head of water in the borehole</td>
<td>Bottom of borehole may become quick</td>
<td>Decreases</td>
</tr>
<tr>
<td>Careless measurement of hammer drop</td>
<td>Hammer energy varies (generally, variations cluster on the low side)</td>
<td>Increases</td>
</tr>
<tr>
<td>Hammer weight inaccurate</td>
<td>Hammer energy varies (driller supplies weight; variations of 5 to 7 percent are common)</td>
<td>Increases or decreases</td>
</tr>
<tr>
<td>Hammer strikes drill rod collar eccentrically</td>
<td>Hammer energy reduced</td>
<td>Increases</td>
</tr>
<tr>
<td>Lack of hammer free fall because of ungreased sheaves, new stiff rope on weight, more than two turns on cat-head, incomplete release of rope during each drop</td>
<td>Hammer energy reduced</td>
<td>Increases</td>
</tr>
<tr>
<td>Sampler driven above bottom of casing</td>
<td>Sampler driven in disturbed, artificially densified soil</td>
<td>Increases greatly</td>
</tr>
<tr>
<td>Careless blow count</td>
<td>Inaccurate results</td>
<td>Increases or decreases</td>
</tr>
<tr>
<td>Use of non-standard sampler</td>
<td>Correlations with standard sampler invalid</td>
<td>Increases or decreases</td>
</tr>
<tr>
<td>Coarse gravel or cobbles in soil</td>
<td>Sampler becomes clogged or impeded</td>
<td>Increases</td>
</tr>
<tr>
<td>Use of bent drill rods</td>
<td>Inhibited transfer of energy of sampler</td>
<td>Increases</td>
</tr>
</tbody>
</table>

**SPT Correlations**

### N versus $\bar{\phi}_{tc}$ Relationships

<table>
<thead>
<tr>
<th>N Value (blows/ft or 305 mm)</th>
<th>Relative Density</th>
<th>Approximate $\bar{\phi}_{tc}$ (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 4</td>
<td>very loose</td>
<td>$&lt; 28$</td>
</tr>
<tr>
<td>4 to 10</td>
<td>loose</td>
<td>28 to 30</td>
</tr>
<tr>
<td>10 to 30</td>
<td>medium</td>
<td>30 to 36</td>
</tr>
<tr>
<td>30 to 50</td>
<td>dense</td>
<td>36 to 41</td>
</tr>
<tr>
<td>&gt; 50</td>
<td>very dense</td>
<td>$&gt; 41$</td>
</tr>
</tbody>
</table>

*(a) Approximate values based on empirical data.*

---

*Source:* Peck, Hanson, and Thornburn (12), p. 310.

---

**Figure 4-12. N versus $\bar{\phi}_{tc}$**

*Source:* Peck, Hanson, and Thornburn (12), p. 310.
Cone Penetration Test

• Detailed procedure given in ASTM D 3441
• Conical penetrometer tip (also called Dutch cone) is slowly pushed into ground and monitored
• Modern devices contain transducers to measure both tip bearing and side friction as instrument is advanced
## Major Sources of Error in the Cone Penetration Test

<table>
<thead>
<tr>
<th>Cause</th>
<th>Effect</th>
<th>Influence on Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel or cobbles in soil</td>
<td>Impedes penetration of penetrometer tip (can break tip or rods)</td>
<td>Increases $q_C$ greatly</td>
</tr>
<tr>
<td></td>
<td>Causes penetrometer to wander off vertical</td>
<td></td>
</tr>
<tr>
<td>Worn penetrometer tip</td>
<td>Tip may become dull and/or surface roughness may become greater or lesser than standard</td>
<td>Increases or decreases $q_C$ and $f_s$ slightly</td>
</tr>
<tr>
<td>Soil clogging end of friction sleeve (mechanical tips only)</td>
<td>Adds an erroneous end bearing component to $f_s$</td>
<td>Increases $f_s$ up to about 80 percent</td>
</tr>
<tr>
<td>Rusted or clogged inner rods (mechanical tips only)</td>
<td>Impedes free travel of inner rods because of friction against outer rods</td>
<td>Increases $q_C$ and $f_s$</td>
</tr>
<tr>
<td>Hard soils (mechanical tips only)</td>
<td>Causes elastic compression of inner rods, giving false indication that penetration has occurred</td>
<td>Measurement of $q_C$ and $f_s$ may not be possible</td>
</tr>
<tr>
<td>Leaky water seal (electrical tips only)</td>
<td>Electrical transducers may become corroded</td>
<td>Increases or decreases $q_C$ and $f_s$</td>
</tr>
<tr>
<td>Improper calibration (electrical tips only)</td>
<td>Inaccurate measurements</td>
<td>Increases or decreases $q_C$ and $f_s$</td>
</tr>
</tbody>
</table>

Source: Kulhawy, et al. (23), p. 5-30.
Vane Shear Test

• Procedure described in detail in ASTM D 2573
• Test involves pushing a four-bladed vane into a clay stratum and slowly rotating it while measuring the resisting torque
• Peak torque is related to undrained shear strength
• After peak torque is determined, vane is quickly rotated about ten times to remold the soil
• Torque is measured again to determine remolded or residual strength
Pressuremeter Test

• Procedure described in detail in ASTM D 4719
• Test equipment consists of three parts
  – probe – metal cylinder covered with inflatable rubber membrane
  – pressure-volume control unit
  – tubing
Pressuremeter Test

Figure C-1. Menard Pressuremeter Equipment

Source: Baguelin, et al. (2), p. 47.
Pressuremeter Test

• After calibration, probe is inserted into the borehole at desired location
• Pressure is applied to the probe in ten equal steps and maintained for a constant period (60 secs) for each step
• Volumetric expansion of probe is measured
• Test ends when probe is expanded to twice its deflated volume or when pressure limit of device is reached
Pressuremeter Test

- Three characteristic pressures can be defined
  - $p_0$ - associated with in-situ horizontal stress
  - $p_f$ – represents creep or yield pressure
  - $p_l$ – limit pressure
- Pressuremeter Modulus
  $$E_p = 2(1+\mu)(V_0 - v_m)(\Delta p/\Delta v)$$
  $\mu$ = Poisson’s ratio, $v_m = (v_0 + v_f)/2$
Pressuremeter Plot
## TYPICAL VALUES OF PRESSUREMETER MODULUS AND LIMIT PRESSURE

<table>
<thead>
<tr>
<th>Type of Soil</th>
<th>Pressuremeter Modulus $E_p$ (kN/m$^2$)</th>
<th>Limit Pressure $p_2$ (kN/m$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peat, very soft clay</td>
<td>200 - 1,500</td>
<td>20 - 150</td>
</tr>
<tr>
<td>Soft clay</td>
<td>500 - 3,000</td>
<td>50 - 300</td>
</tr>
<tr>
<td>Firm clay</td>
<td>3,000 - 8,000</td>
<td>300 - 800</td>
</tr>
<tr>
<td>Stiff clay</td>
<td>8,000 - 40,000</td>
<td>600 - 2,500</td>
</tr>
<tr>
<td>Loose silty sand</td>
<td>500 - 2,000</td>
<td>100 - 500</td>
</tr>
<tr>
<td>Silt</td>
<td>2,000 - 10,000</td>
<td>200 - 1,500</td>
</tr>
<tr>
<td>Sand and gravel</td>
<td>8,000 - 40,000</td>
<td>1,200 - 5,000</td>
</tr>
<tr>
<td>Till</td>
<td>7,500 - 40,000</td>
<td>1,000 - 5,000</td>
</tr>
<tr>
<td>Recent fill</td>
<td>500 - 5,000</td>
<td>50 - 300</td>
</tr>
<tr>
<td>Old fill</td>
<td>4,000 - 15,000</td>
<td>400 - 1,000</td>
</tr>
</tbody>
</table>

Source: Reference (62), p. 310

Note: 1 kN/m$^2$ = 1/100 tsf or 1/7 psi
## ASSESSMENT OF IN-SITU TESTS

<table>
<thead>
<tr>
<th>Comparison Basis</th>
<th>Standard Penetration Test</th>
<th>Cone Penetration Test</th>
<th>Vane Shear Test</th>
<th>Pressure-meter Test</th>
<th>Flat Dilatometer Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simplicity of apparatus</td>
<td>Simple, rugged</td>
<td>Complex, rugged</td>
<td>Simple, rugged</td>
<td>Complex, delicate</td>
<td>Simple, rugged</td>
</tr>
<tr>
<td>Ease of testing</td>
<td>Easy</td>
<td>Easy</td>
<td>Easy</td>
<td>Complex</td>
<td>Easy</td>
</tr>
<tr>
<td>Continuous profile or point values</td>
<td>Point</td>
<td>Continuous</td>
<td>Point</td>
<td>Point</td>
<td>Semi-continuous</td>
</tr>
<tr>
<td>Basis for interpretation</td>
<td>Empirical</td>
<td>Empirical, theory</td>
<td>Theory</td>
<td>Empirical, theory</td>
<td>Semi-empirical, theory</td>
</tr>
<tr>
<td>Suitable soils</td>
<td>Most types</td>
<td>Most types</td>
<td>Softer clays</td>
<td>Most types</td>
<td>Most types</td>
</tr>
<tr>
<td>Suitability in practice</td>
<td>Routine</td>
<td>Routine</td>
<td>Routine</td>
<td>Limited</td>
<td>Routine</td>
</tr>
</tbody>
</table>

Source: Modified from Mitchell (1), pp. 121, 123.
## HISTORICAL USE, MOBILIZATION AND ACCESS REQUIREMENTS, AND COSTS OF IN-SITU TESTS

<table>
<thead>
<tr>
<th>Test</th>
<th>Historical Use</th>
<th>Availability</th>
<th>Access</th>
<th>Unit Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPT</td>
<td>Substantial</td>
<td>Excellent</td>
<td>Truck, trailer</td>
<td>Medium</td>
</tr>
<tr>
<td>MCPT</td>
<td>Substantial</td>
<td>Good</td>
<td>Limited portability - truck, trailer</td>
<td>Low</td>
</tr>
<tr>
<td>ECPT</td>
<td>Moderate</td>
<td>Good</td>
<td>Limited portability - truck, trailer</td>
<td>Low</td>
</tr>
<tr>
<td>CPTU</td>
<td>Limited</td>
<td>Poor</td>
<td>Limited portability - truck, trailer</td>
<td>Medium</td>
</tr>
<tr>
<td>VST</td>
<td>Substantial</td>
<td>Excellent</td>
<td>Limited portability - truck, trailer</td>
<td>Medium</td>
</tr>
<tr>
<td>DMT</td>
<td>Limited</td>
<td>Fair</td>
<td>Limited portability - truck, trailer</td>
<td>Low</td>
</tr>
<tr>
<td>PMT</td>
<td>Moderate</td>
<td>Good</td>
<td>Limited portability - truck, trailer</td>
<td>Medium</td>
</tr>
<tr>
<td>SBPMT</td>
<td>Limited</td>
<td>Poor</td>
<td>Limited portability - truck, trailer</td>
<td>High</td>
</tr>
</tbody>
</table>

Qualitative Relationship Between Relative Test Cost and Accuracy
## ESTIMATES OF IN-SITU TEST VARIABILITY

<table>
<thead>
<tr>
<th>Test</th>
<th>COV(^a) (%)</th>
<th>COV (%)</th>
<th>COV (%)</th>
<th>COV (^b) (%)</th>
<th>COV (^c) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard Penetration Test (SPT)</td>
<td>5(^d) to 75(^e)</td>
<td>5(^d) to 75(^e)</td>
<td>12 to 15</td>
<td>14(^d) to 100(^e)</td>
<td>15 to 45</td>
</tr>
<tr>
<td>Mechanical Cone Penetration Test (MCPT)</td>
<td>5</td>
<td>10(^f) to 15(^g)</td>
<td>10(^f) to 15(^g)</td>
<td>15(^f) to 22(^g)</td>
<td>15 to 25</td>
</tr>
<tr>
<td>Electrical Cone Penetration Test (ECPT)</td>
<td>3</td>
<td>5</td>
<td>5(^f) to 10(^g)</td>
<td>7(^f) to 12(^g)</td>
<td>5 to 15</td>
</tr>
<tr>
<td>Vane Shear Test (VST)</td>
<td>5</td>
<td>8</td>
<td>10</td>
<td>14</td>
<td>10 to 20</td>
</tr>
<tr>
<td>Dilatometer Test (DMT)</td>
<td>5</td>
<td>5</td>
<td>8</td>
<td>11</td>
<td>5 to 15</td>
</tr>
<tr>
<td>Pressuremeter Test (PMT)</td>
<td>5</td>
<td>12</td>
<td>10</td>
<td>16</td>
<td>10 to 20(^h)</td>
</tr>
<tr>
<td>Self-Boring Pressuremeter Test (SBPMT)</td>
<td>8</td>
<td>15</td>
<td>8</td>
<td>19</td>
<td>15 to 25(^h)</td>
</tr>
</tbody>
</table>

Notes:

- a - COV = standard deviation/mean
- b - COV(Total) = \([COV(\text{Equipment})^2 + COV(\text{Procedure})^2 + COV(\text{Random})^2]^{\frac{1}{2}}\)
- c - Because of limited data and the judgment involved in estimating COV values, ranges represent probable magnitudes of field test measurement error
- d - Best case scenario for SPT test conditions
- e - Worst case scenario for SPT test conditions
- f - Tip resistance CPT measurements
- g - Side resistance CPT measurements
- h - It is likely that results may differ for \(p_0\), \(p_f\), and \(p_L\), but the data are insufficient to clarify this issue

Source: Orchant, et al. (3), p. 4-63.
### Laboratory Test Requirements for Different Soil Types

<table>
<thead>
<tr>
<th>Category</th>
<th>Test</th>
<th>Soil Type</th>
<th>Coarse-Grained*</th>
<th>Fine-Grained*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(&gt; #200 Sieve)</td>
<td></td>
<td>(&lt; #200 Sieve)</td>
</tr>
<tr>
<td>Index Tests</td>
<td>Atterberg Limits</td>
<td>N</td>
<td>M</td>
<td></td>
</tr>
<tr>
<td>Compaction</td>
<td></td>
<td>R</td>
<td>J</td>
<td></td>
</tr>
<tr>
<td>Grain Size</td>
<td></td>
<td>M</td>
<td>J</td>
<td></td>
</tr>
<tr>
<td>Relative Density</td>
<td></td>
<td>R</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>Specific Gravity</td>
<td></td>
<td>J</td>
<td>R</td>
<td></td>
</tr>
<tr>
<td>Unit Weight</td>
<td></td>
<td>R</td>
<td>R</td>
<td></td>
</tr>
<tr>
<td>Water Content</td>
<td></td>
<td>J</td>
<td>M</td>
<td></td>
</tr>
<tr>
<td>Strength Tests</td>
<td>Direct Shear</td>
<td>R</td>
<td>J</td>
<td></td>
</tr>
<tr>
<td>Triaxial Shear</td>
<td></td>
<td>J</td>
<td>R</td>
<td></td>
</tr>
<tr>
<td>Unconfined Compression</td>
<td></td>
<td>N</td>
<td>M</td>
<td></td>
</tr>
<tr>
<td>Vane Shear</td>
<td></td>
<td>N</td>
<td>R</td>
<td></td>
</tr>
<tr>
<td>Compressibility Tests</td>
<td>1-D Consolidation</td>
<td>J</td>
<td>M</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Triaxial Consolidation</td>
<td>J</td>
<td>R</td>
<td></td>
</tr>
</tbody>
</table>

*N - Not applicable, R - Recommended, J - Judgment of investigator, M - Mandatory*
Foundation Analysis & Design Workshop
Session 2

SELECTION OF GEOTECHNICAL DESIGN PARAMETERS
Foundation Design Process

- Foundation Design Program
  - Foundation Performance Criteria
  - Construction Drawings and Specifications
    - Construction Monitoring
    - As-Built Information

Subsurface Investigation
- In-Situ Test Data
- Laboratory Test Data

Geotechnical Design Parameters

- Needed Geotechnical Design Parameters for Selected Foundation Types and Design Models

Design Models
- Selection of Foundation Types

- Design Loads
- Foundation Performance Criteria

Geology Study of Proposed Route
- Logs from Past Drilled Borings
Program to Establish Geotechnical Design Parameters

It’s Important to establish a specific and uniform method to establish geotechnical design parameters.

WHY?
Program to Establish Geotechnical Design Parameters

Assign Parameters to Subsurface Profile at Each Boring Site

Develop a Longitudinal Profile

Assign Parameters to Structure Sites Between Borings
## Unified Soil Classification System

### Field Identification Procedures

<table>
<thead>
<tr>
<th>Group Symbols</th>
<th>Typical Names</th>
</tr>
</thead>
<tbody>
<tr>
<td>GW</td>
<td>Well graded gravels, gravel-sand mixtures, little or no fines</td>
</tr>
<tr>
<td>GP</td>
<td>Poorly graded gravels, gravel-sand mixtures, little or no fines</td>
</tr>
<tr>
<td>GM</td>
<td>Silty gravels, poorly graded gravel-sand-silt mixtures</td>
</tr>
<tr>
<td>GC</td>
<td>Clayey gravels, poorly graded gravel-sand-clay mixtures</td>
</tr>
<tr>
<td>SW</td>
<td>Well graded sands, gravely sands, little or no fines</td>
</tr>
<tr>
<td>SF</td>
<td>Poorly graded sands, gravelly sands, little or no fines</td>
</tr>
<tr>
<td>SM</td>
<td>Silty sands, poorly graded sand-silt mixtures</td>
</tr>
<tr>
<td>SC</td>
<td>Clayey sands, poorly graded sand-clay mixtures</td>
</tr>
</tbody>
</table>

### Identification Procedures on Fraction Smaller than No. 40 Sieve Size

<table>
<thead>
<tr>
<th>Group Symbols</th>
<th>Typical Names</th>
</tr>
</thead>
<tbody>
<tr>
<td>ML</td>
<td>Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity</td>
</tr>
<tr>
<td>CE</td>
<td>Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays</td>
</tr>
<tr>
<td>OL</td>
<td>Organic silts and organic silt-clays of low plasticity</td>
</tr>
<tr>
<td>MH</td>
<td>Inorganic silts, maraneous or diatomaceous fine sandy or silty soils, elastic silts</td>
</tr>
<tr>
<td>CH</td>
<td>Inorganic clays of high plasticity, clay clays</td>
</tr>
<tr>
<td>OH</td>
<td>Organic clays of medium to high plasticity</td>
</tr>
<tr>
<td>Pt</td>
<td>Peat and other highly organic soils</td>
</tr>
</tbody>
</table>

* Boundary classifications. Soils possessing characteristics of two groups are designated by combinations of group symbols. For example, GW-GC.

** All sieve sizes on this chart are U.S. standard.
Assign Parameters to Subsurface Profile at Each Boring Site

**Laboratory Tests**
- Insitu Density & Moisture Content
- Atterberg Limits
- Direct Shear Tests
- Unconfined Compression Tests
- Point Load Rock Tests
- Compaction Tests (Backfills)

**Correlations**
- Granular Soils
- Cohesive Soils
- Rock Mass
- Concrete/Rock Bond Strength

**In-Situ Field Test & Samples**
- Standard Penetration Resistance (Split Spoon Samples)
- Shelby Tube Samples
- Pressuremeter Tests
- Vane Shear Tests
Split Spoon Sample
Shelby Tube Sample
Rock Core Barrels

Single Tube

Double Tube

Triple Tube
Pressuremeter Test
Vane Shear Test
Atterberg Limits
Triaxial Compression Test
Point Load Rock Test
## Table 1A

**Empirical Values for $\phi$, $D_r$, and Unit Weight of Granular Soils Based on Standard Penetration Resistance**

<table>
<thead>
<tr>
<th>Description</th>
<th>Very Loose</th>
<th>Loose</th>
<th>Medium</th>
<th>Dense</th>
<th>Very Dense</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relative density, $D_r$</td>
<td>0</td>
<td>0.15</td>
<td>0.35</td>
<td>0.65</td>
<td>0.85</td>
</tr>
<tr>
<td>Standard penetration number, $N$</td>
<td>4</td>
<td>10</td>
<td>30</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>Approximate angle of internal friction, $\phi^\circ$</td>
<td>25°-30°</td>
<td>27°-32°</td>
<td>30°-35°</td>
<td>35°-40°</td>
<td>38°-43°</td>
</tr>
<tr>
<td>Approximate range of moist unit weight ($\gamma$) pcf</td>
<td>70–100+</td>
<td>90–115</td>
<td>110–130</td>
<td>110–140</td>
<td>130–150</td>
</tr>
</tbody>
</table>

* Use larger value of $\phi$ for granular soils with 5% or less fine sand or silt, or both
Assign Parameters to Subsurface Profile at Each Boring Site

Correlations - Granular Soils - Moist Unit Weight — γ

<table>
<thead>
<tr>
<th></th>
<th>Lower Bound</th>
<th>Upper Bound</th>
<th>Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>VERY LOOSE</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LOOSE</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MEDIUM</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DENSE</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>VERY DENSE</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Standard Penetration Resistance, N (blows/ft)
Assign Parameters to Subsurface Profile at Each Boring Site

Correlations - Granular Soils - Angle of Internal Friction $-\Phi$

<table>
<thead>
<tr>
<th>Very Loose</th>
<th>Loose</th>
<th>Medium</th>
<th>Dense</th>
<th>Very Dense</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lower Bound</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Upper Bound</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Standard Penetration Resistance, N (blows/ft)

Very Loose | Loose | Medium | Dense | Very Dense

0 5 10 15 20 25 30 35 40 45 50

0 5 10 15 20 25 30 35 40 45 50

Angle of Internal Friction, $\phi$
Assign Parameters to Subsurface Profile at Each Boring Site

Correlations - Granular Soils - Modulus of Deformation - $E_D$

**Notes:**

1. N-values utilized to enter the chart are defined as standard penetration resistance for 2" O.D. sampler, not corrected for overburden pressure.
2. Refer to J.H. Schwertmann, (4).
3. 1 KSI = 6.895 MPa
   1 FT = 0.3048 m
Engineering Property Correlations
Cohesive Soils

<table>
<thead>
<tr>
<th>Consistency</th>
<th>Very Soft</th>
<th>Soft</th>
<th>Medium Stiff</th>
<th>Stiff</th>
<th>Very Stiff</th>
<th>Hard</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard Penetration Resistance, N</td>
<td>0</td>
<td>2</td>
<td>4</td>
<td>8</td>
<td>16</td>
<td>32</td>
</tr>
<tr>
<td>(blows per foot)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total Unit Weight of Saturated Soil</td>
<td>100-120</td>
<td>110-130</td>
<td>120-140</td>
<td>&gt;130</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(pcf)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**UNDRAINED SHEAR STRENGTH OF COHESIVE SOILS (3)**

<table>
<thead>
<tr>
<th>Consistency</th>
<th>Very Soft</th>
<th>Soft</th>
<th>Medium Stiff</th>
<th>Stiff</th>
<th>Very Stiff</th>
<th>Hard</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined Compressive strength, ( q_u ) (tsf)</td>
<td>0</td>
<td>0.25</td>
<td>0.50</td>
<td>1.00</td>
<td>2.00</td>
<td>4.00</td>
</tr>
<tr>
<td>Standard Penetration Resistance, N</td>
<td>0</td>
<td>2</td>
<td>4</td>
<td>8</td>
<td>16</td>
<td>32</td>
</tr>
<tr>
<td>(blows per foot)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Unconfined Compression Test

\[ q_u = \text{Unconfined Compression Strength} \]

\[ S_u = \frac{q_u}{2} \]

\[ \frac{L}{D} = 2 \]

\[ D \approx 2" \]

COHESIVE SOIL SAMPLE

Unconfined Compression Strength

\[ q_u = \frac{S_u}{2} \]

\[ S_u = \text{Undrained Shear Strength} \]
Assign Parameters to Subsurface Profile at Each Boring Site

Correlations - Cohesive Soils - Saturated Unit Weight $\gamma$

(Use With Care)

Saturated Unit Weight, $\gamma$ (pcf)

Standard Penetration Resistance, N (blows/ft)

Lower Bound
Upper Bound
Mean
Assign Parameters to Subsurface Profile at Each Boring Site

Correlations – Cohesive Soils – Unconfined Compressive Strength $-q_u$

(Use With Care)

Correlation Table:

<table>
<thead>
<tr>
<th>Data Point</th>
<th>Linear Fit</th>
</tr>
</thead>
<tbody>
<tr>
<td>VERY SOFT</td>
<td>STIFF</td>
</tr>
<tr>
<td>SOFT</td>
<td>STIFF</td>
</tr>
<tr>
<td>MEDIUM</td>
<td>STIFF</td>
</tr>
<tr>
<td>STIFF</td>
<td>STIFF</td>
</tr>
<tr>
<td>HARD</td>
<td>HARD</td>
</tr>
</tbody>
</table>

Unconfined Compressive Strength, $q_u$ (tsf)

Standard Penetration Resistance, N (blows/ft)
Assign Parameters to Subsurface Profile at Each Boring Site

Correlations – Cohesive Soils – Modulus of Deformation – $E_D$

RECOMMENDED MODULUS OF DEFORMATION

NOTES:


2. 1 ksi = 6.9 MPa
   1 tsf = 95.8 kPa
### Assigning Engineering Properties to Rock Mass

The engineering properties of a rock mass are for all practical purposes entirely a function of the number, type, spacing, and orientation of rock defects such as:

<table>
<thead>
<tr>
<th>Joints</th>
<th>Shear Zones</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weathering</td>
<td>Foliation</td>
</tr>
<tr>
<td>Faults</td>
<td>Solution Channels</td>
</tr>
<tr>
<td>Bedding Planes</td>
<td></td>
</tr>
</tbody>
</table>
## Point Contributions to the RMR\textsubscript{76} of a Rock Mass (Hoek and Brown, 1980)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Point load strength (ksi)</th>
<th>Uniaxial compression strength (ksi)</th>
<th>Strength of intact rock material</th>
<th>RQD (%)</th>
<th>Spacing (ft)</th>
<th>Joints Condition</th>
<th>Groundwater Condition</th>
<th>Joints Orientation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&gt; 1.16</td>
<td>0.6 – 1.16</td>
<td>0.3 – 0.6</td>
<td>0.15 – 0.3</td>
<td>&gt; 9.8</td>
<td>JC1, JC2, JC3, JC4, JC5</td>
<td>GW1, GW2, GW3, GW4</td>
<td>Very favorable</td>
</tr>
<tr>
<td>Points</td>
<td>15</td>
<td>12</td>
<td>7</td>
<td>4</td>
<td>3</td>
<td>5</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>90 –100</td>
<td>75 – 90</td>
<td>50 – 75</td>
<td>25 – 50</td>
<td>&lt; 0.2</td>
<td>10</td>
<td>6</td>
<td>-2</td>
</tr>
<tr>
<td>Points</td>
<td>20</td>
<td>17</td>
<td>13</td>
<td>8</td>
<td>5</td>
<td>10</td>
<td>6</td>
<td>-7</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>20</td>
<td>20</td>
<td>10</td>
<td>5</td>
<td>10</td>
<td>6</td>
<td>-15</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>25</td>
<td>30</td>
<td>10</td>
<td>5</td>
<td>10</td>
<td>6</td>
<td>-25</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>30</td>
<td>40</td>
<td>10</td>
<td>5</td>
<td>10</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>40</td>
<td>50</td>
<td>10</td>
<td>5</td>
<td>10</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>50</td>
<td>60</td>
<td>10</td>
<td>5</td>
<td>10</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>60</td>
<td>70</td>
<td>10</td>
<td>5</td>
<td>10</td>
<td>6</td>
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<td></td>
<td>80</td>
<td>70</td>
<td>80</td>
<td>10</td>
<td>5</td>
<td>10</td>
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<td>90</td>
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<td>90</td>
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<td>6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>90</td>
<td>100</td>
<td>10</td>
<td>5</td>
<td>10</td>
<td>6</td>
<td></td>
</tr>
</tbody>
</table>

### Joint condition descriptions:
- **JC1**: Very rough surfaces, not continuous, no separation, unweathered wall rock.
- **JC2**: Slightly rough surfaces, separation less than 1 mm, slightly weathered wall rock.
- **JC3**: Slightly rough surfaces, separation less than 1 mm, highly weathered wall rock.
- **JC4**: Slickensided surfaces, or gauge less than 5 mm, or separation 1–5 mm continuous.
- **JC5**: Soft gauge greater than 5 mm, or separation greater than 5 mm continuous.

### Groundwater condition descriptions:
- **GW1**: Completely dry.
- **GW2**: Moist only, interstitial water.
- **GW3**: Water under moderate pressure.
- **GW4**: Severe water problem.
Assigning Engineering Properties to Rock Mass

RMR$_{76}$ Values, Rock Descriptions and Rock Strength Based on RMR$_{76}$ (Hoek and Brown, 1980)

<table>
<thead>
<tr>
<th>RMR$_{76}$</th>
<th>81-100</th>
<th>61-80</th>
<th>41-60</th>
<th>21-40</th>
<th>&lt; 20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class No.</td>
<td>I</td>
<td>II</td>
<td>III</td>
<td>IV</td>
<td>V</td>
</tr>
<tr>
<td>Description</td>
<td>Very good rock</td>
<td>Good rock</td>
<td>Fair rock</td>
<td>Poor rock</td>
<td>Very poor rock</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Class No.</th>
<th>I</th>
<th>II</th>
<th>III</th>
<th>IV</th>
<th>V</th>
</tr>
</thead>
<tbody>
<tr>
<td>RMR$_{76}$</td>
<td>81-100</td>
<td>61-80</td>
<td>41-60</td>
<td>21-40</td>
<td>&lt; 20</td>
</tr>
<tr>
<td>Cohesion of the Rock Mass $c'$</td>
<td>$&gt; 6.3$ ksf</td>
<td>$4.2-6.3$ ksf</td>
<td>$3.1-4.2$ ksf</td>
<td>$2.1-3.1$ ksf</td>
<td>$&lt; 2.1$ ksf</td>
</tr>
<tr>
<td>Friction Angle of the Rock Mass $\phi'$</td>
<td>$&gt; 45$ degrees</td>
<td>$40-45$ deg.</td>
<td>$35-40$ deg</td>
<td>$30-35$ deg</td>
<td>$&lt; 30$ deg.</td>
</tr>
</tbody>
</table>

Conversion: 1 ksf = 47.88 kPa
## Assigning Engineering Properties to Rock Mass

### Calculation of RMR$_{76}$ values for full-scale foundation load tests and estimated strength and deformation parameters

<table>
<thead>
<tr>
<th>Load test site</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer No.</td>
<td>1</td>
<td>1</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>Depth (m)</td>
<td>0-</td>
<td>0.75-</td>
<td>1.5-</td>
<td>0.6-</td>
<td>1.2-</td>
<td>1.5-</td>
<td>1.7-</td>
</tr>
<tr>
<td></td>
<td>1.2</td>
<td>1.5-</td>
<td>2.1</td>
<td>1.2</td>
<td>1.5</td>
<td>1.7</td>
<td>1.9</td>
</tr>
<tr>
<td>RMR$_{76}$ parameter</td>
<td>1.1-</td>
<td>0.8-</td>
<td>0.2-</td>
<td>0.8-</td>
<td>1.5-</td>
<td>2.1-</td>
<td>2.7</td>
</tr>
<tr>
<td>Strength of intact rock</td>
<td>12</td>
<td>4</td>
<td>15</td>
<td>12</td>
<td>12</td>
<td>4</td>
<td>7</td>
</tr>
<tr>
<td>RDQ</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>8</td>
<td>8</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Spacing of joints</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>10</td>
<td>10</td>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td>Condition of joints</td>
<td>12</td>
<td>12</td>
<td>12</td>
<td>12</td>
<td>12</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td>Groundwater</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Adjustment for joint orientations</td>
<td>-7</td>
<td>-7</td>
<td>-7</td>
<td>-7</td>
<td>-7</td>
<td>-7</td>
<td>-7</td>
</tr>
<tr>
<td>RMR$_{76}$</td>
<td>35</td>
<td>20</td>
<td>27</td>
<td>38</td>
<td>35</td>
<td>45</td>
<td>37</td>
</tr>
<tr>
<td>Friction Angle, $\Phi^*$</td>
<td>34</td>
<td>30</td>
<td>32</td>
<td>35</td>
<td>34</td>
<td>36</td>
<td>34</td>
</tr>
<tr>
<td>Cohesion, $c$ (kPa)</td>
<td>134</td>
<td>100</td>
<td>115</td>
<td>144</td>
<td>134</td>
<td>158</td>
<td>139</td>
</tr>
<tr>
<td>Modulus of deformation, $E$ (GPa)</td>
<td>4.13</td>
<td>1.38</td>
<td>2.48</td>
<td>4.85</td>
<td>4.13</td>
<td>6.76</td>
<td>4.60</td>
</tr>
</tbody>
</table>

* Lower bound RMR$_{76}$ value assumed for rock layer due to insufficient data.
1 KSF = 47.88 kPa
Assigning Engineering Properties to Rock Mass
Assigning Engineering Properties to Rock Mass

Core Recovery (CR) and Rock Quality Designation (RQD)

For Each Rock Core Run

Core Recovery (CR)

\[
CR (\%) = \frac{\text{Total Length of Rock Core Recovered}}{\text{Total Length of Rock Core Drilled}}
\]

Rock Quality Designation (RQD)

\[
RQD = \frac{\text{Total Length of Rock Core Pieces} = 4 \text{ inches}}{\text{Total Length of Rock Core Drilled}}
\]
Assigning Engineering Properties to Rock Mass

Effective Stress Friction Angle, $\Phi'$, vs RMR$_{76}$ of Rock Mass

Friction Angle, $\Phi'$ (degrees)

RMR$_{76}$

Hoek and Brown (1980)
Assigning Engineering Properties to Rock Mass

Effective Stress Cohesion, $c'$ vs RMR$_{76}$ of Rock Mass

1 ksf = 47.88 kPa
Assigning Engineering Properties to Rock Mass

Modulus of Deformation, E, vs RMR$_{76}$ of Rock Mass

- Berafim and Pereira (1983) RMR < 61
- Berafim and Pereira (1983) RMR > 61
- Bieniawski (1978)

Mathematical Models:

- $E = 0.564 \text{ RMR}_{76}^{0.958}$ (ksi) for RMR < 60
- $E = 290 \text{ RMR} - 14500$ (ksi) for RMR > 60

Conversion: $1 \text{ksi} = 6.9 \text{ MPa}$

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## Rock End Bearing Capacity and Concrete/Rock Bond Strength

<table>
<thead>
<tr>
<th>Location</th>
<th>Rock Type</th>
<th>Rock Properties</th>
<th>Side Shear Reached</th>
<th>End Bearing Reached</th>
<th>Test / Design</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Burgin, Kentucky</td>
<td>Hard Limestone</td>
<td>Comp. Strength 15,000-20,000 psi</td>
<td>19 tfs (0.66 inches)</td>
<td>127 tfs (0.17 inches)</td>
<td>&gt; 9.0</td>
<td>Ultimate Loads Not Reached</td>
</tr>
<tr>
<td>Decatur, Alabama</td>
<td>Med. Hard Limestone</td>
<td>Comp. Strength 15,000-22,000 psi</td>
<td>30 tfs (0.65 inches)</td>
<td>370 tfs (1.5 inches)</td>
<td>&gt; 5.0</td>
<td>Ultimate Loads Not Reached</td>
</tr>
<tr>
<td>Edison, New Jersey</td>
<td>Weathered Shale</td>
<td>60-70% Recovery RQD = 7%</td>
<td>3 tfs (1.7 inches)</td>
<td>34 tfs (0.5 inches)</td>
<td>&gt; 3.0</td>
<td>Ultimate Reached in Side Shear</td>
</tr>
<tr>
<td>Lathe, New York</td>
<td>Shale</td>
<td></td>
<td>18 tfs (0.68 inches)</td>
<td>216 tfs</td>
<td></td>
<td>Ultimate Loads Not Reached</td>
</tr>
<tr>
<td>Hong Kong</td>
<td>Weathered Granite</td>
<td></td>
<td>18 tfs (0.68 inches)</td>
<td>194 tfs (0.28 inches)</td>
<td>&gt; 4.5</td>
<td>Ultimate Loads Not Reached</td>
</tr>
<tr>
<td>Chester, New York</td>
<td>Bedded Sandstone</td>
<td>13 tfs (1.8 inches)</td>
<td>16 tfs (0.68 inches)</td>
<td>161 tfs (0.08 inches)</td>
<td></td>
<td>Ultimate Reached in Side Shear</td>
</tr>
<tr>
<td>Ohio River</td>
<td>Shale with coal seams</td>
<td>350-500 psi</td>
<td>9.3 tfs (0.3 inches)</td>
<td>113 tfs (1.4 inches)</td>
<td>&gt; 6.0</td>
<td>Ultimate Loads Not Reached</td>
</tr>
<tr>
<td>Milton, Mass</td>
<td>Argillite Shale</td>
<td>Comp. Strength 3200 psi</td>
<td>15 tfs @ 0.55 inches</td>
<td>Ultimate</td>
<td>250 tfs @ 0.65 inches</td>
<td>Ultimate Reached in Side Shear and End Bearing</td>
</tr>
<tr>
<td>Chicago, Illinois</td>
<td>Hard Limestone</td>
<td>Comp. Strength 12,200 psi</td>
<td>20 tfs (0.26 inches)</td>
<td>725 tfs (0.13 inches)</td>
<td>Ultimate Loads Not Reached</td>
<td></td>
</tr>
<tr>
<td>Milwaukee, Wisconsin</td>
<td>Fractured Limestone</td>
<td>RQD 60</td>
<td>9.55 tfs (0.36 inches)</td>
<td>115 tfs (1.23 inches)</td>
<td>Ultimate Loads Not Reached</td>
<td></td>
</tr>
<tr>
<td>Springfield, VA</td>
<td>Weathered green &amp; black granite</td>
<td>85% Recovery RQD 50</td>
<td>1.99 tfs (0.17 inches)</td>
<td>Ultimate</td>
<td>230 tfs (3.71 inches)</td>
<td>Ultimate Reached in End Bearing</td>
</tr>
<tr>
<td>Hannibal, MO</td>
<td>Lime Rock</td>
<td></td>
<td>16.1 tfs (1.09 inches)</td>
<td>100.3 tfs (114 inches)</td>
<td>&gt; 7.1</td>
<td>Ultimate Loads Not Reached</td>
</tr>
<tr>
<td>Hannibal, MO</td>
<td>Shale</td>
<td></td>
<td>7.46 tfs (0.20 inches)</td>
<td>93.9 tfs (4.31 inches)</td>
<td>&gt; 4.1</td>
<td>Ultimate Loads Not Reached</td>
</tr>
<tr>
<td>Grand Rapids, MI</td>
<td>Mod. Weathered Gypsum w/ Clay Shale</td>
<td>Comp. Strength Approx. 3,000 - 8,000 psi</td>
<td>17.0 tfs (2.97 inches)</td>
<td>81.4 tfs (1.20 inches)</td>
<td>&gt; 5.1</td>
<td>Ultimate Loads Not Reached</td>
</tr>
<tr>
<td>Lexington, Missouri</td>
<td>Gray - Black Shale w/ thin Limestone &amp; Coal Seams</td>
<td>Comp. Strength Approx. 121.5 psi SPT 100 REC 100%</td>
<td>8.03 tfs (0.22 inches)</td>
<td>101.5 tfs (0.54 inches)</td>
<td>&gt; 5.5</td>
<td>Ultimate Loads Not Reached</td>
</tr>
<tr>
<td>Lexington, Missouri</td>
<td>Gray - Black Shale w/ thin Limestone &amp; Coal Seams</td>
<td>Comp. Strength Approx. 336.5 psi</td>
<td>8.85 tfs (0.21 inches)</td>
<td>72 tfs (2.39 inches)</td>
<td>&gt; 4.7</td>
<td>Ultimate Reached in End Bearing</td>
</tr>
<tr>
<td>Albany, New York</td>
<td>Sound, black shale (Skeena Hill Formation)</td>
<td>RQD 95% RQD 23%</td>
<td>35.1 tfs (0.22 inches)</td>
<td>289.5 tfs (0.30 inches)</td>
<td>&gt; 6.7</td>
<td>Ultimate Loads Not Reached</td>
</tr>
<tr>
<td>Providence, RI</td>
<td>Dark gray fractured graphic shale</td>
<td>SPT 100</td>
<td>3.06 tfs (0.27 inches)</td>
<td>61.1 tfs (2.07 inches)</td>
<td>7.1</td>
<td>Ultimate Loads Approached</td>
</tr>
<tr>
<td>Aspen, Colorado</td>
<td>Shale Bedrock</td>
<td>RQD 85% REC 100%</td>
<td>4.75 tfs (0.07 inches)</td>
<td>163 tfs (0.09 inches)</td>
<td>&gt; 7.1</td>
<td>Ultimate Loads Not Reached</td>
</tr>
<tr>
<td>Portsmouth, N.H.</td>
<td>Granite</td>
<td></td>
<td>11.75 tfs (0.82 inches)</td>
<td>228 tfs (0.77 inches)</td>
<td>&gt; 2.7</td>
<td>Ultimate Loads Not Reached</td>
</tr>
<tr>
<td>Atlanta, Georgia</td>
<td>Fractured &amp; partially weathered rock</td>
<td>RQD 55%</td>
<td>9 tfs (0.204 inches)</td>
<td>165 tfs (1.92 inches)</td>
<td>&gt; 3.2</td>
<td>Ultimate Reached in End Bearing</td>
</tr>
<tr>
<td>Aspen, Colorado</td>
<td>Hard, reddish-brown Silicate</td>
<td>RQD 90 REC 100%</td>
<td>7.71 tfs (0.51 inches)</td>
<td>61.5 tfs (1.86 inches)</td>
<td>7.2</td>
<td>Ultimate Reached in Side Shear and End Bearing</td>
</tr>
</tbody>
</table>
Based on an evaluation of full scale foundation load tests, the following ultimate rock/concrete bond strengths are recommended for use with MFAD, HFAD and TFAD:

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Ultimate Rock/Concrete Bond Strength (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Claystone</td>
<td>2</td>
</tr>
<tr>
<td>Shale</td>
<td>7</td>
</tr>
<tr>
<td>Siltstone</td>
<td>15</td>
</tr>
<tr>
<td>Limestone</td>
<td>20</td>
</tr>
<tr>
<td>Sandstone</td>
<td>25</td>
</tr>
</tbody>
</table>

(1) The bond strengths listed are for weathered rock conditions.
Granular Backfills – Grainsize Distributions and Relative Density Values

![Sieve Analysis Diagram]

<table>
<thead>
<tr>
<th>Description of Backfill</th>
<th>Max. Dry Density $\gamma_{max}$ (pcf)</th>
<th>Min. Dry Density $\gamma_{min}$ (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1, Brown-red crushed limestone (SW)</td>
<td>139</td>
<td>102</td>
</tr>
<tr>
<td>C2, Gray-black shale fragments, some silt (crushed stone) (GW)</td>
<td>130</td>
<td>96</td>
</tr>
<tr>
<td>C3, Crushed limestone/sandstone (SW-SM)</td>
<td>137</td>
<td>109</td>
</tr>
<tr>
<td>C4, Gray crushed silty limestone (GW-GM)</td>
<td>123</td>
<td>100</td>
</tr>
<tr>
<td>C5, Crushed Limestone (GP)</td>
<td>96</td>
<td>86</td>
</tr>
<tr>
<td>C6, Crushed basalt (GW)</td>
<td>120</td>
<td>97</td>
</tr>
<tr>
<td>S&amp;G 1, Brown sandy gravel, trace silt (GP-GM)</td>
<td>140</td>
<td>117</td>
</tr>
<tr>
<td>S&amp;G 2, Light yellow sand, some gravel (sand &amp; gravel mix) (SP)</td>
<td>127</td>
<td>113</td>
</tr>
<tr>
<td>S&amp;G 3, Sand and rounded gravel (SP)</td>
<td>132</td>
<td>120</td>
</tr>
<tr>
<td>G1, Gravel (GP)</td>
<td>105</td>
<td>90</td>
</tr>
</tbody>
</table>

Conversion: 1 pcf = 0.157kN/m³
Granular Backfills – Internal Friction Angles
Granular Backfills – Deformation Modulus Values

Conversions: 1 ksi = 6.89 MPa
1 tsf = 95.7 kPa
Design Verification During Construction & Documentation

Why is this activity important?

What should be done? And by Whom?
Assignment of Geotechnical Design Parameters
# Boring Log B-1, Str. 9A

## Drilling and Sampling Information
- **Date Started**: 7/3/08
- **Date Completed**: 7/3/08
- **Hammer Wt.**: 140 lb.
- **Hammer Drop**: 30 in.
- **Dist Foreman**: T. Simpson
- **Spoon Sampler OD**: 2 in.
- **Inspector**: M.B.
- **Rock Core Dia**: 2 in.
- **Drilling Method**: CFA, MH
- **Shelly Tube OD**: 3 in.

## Soil Classification

### Surface Elevation

<table>
<thead>
<tr>
<th>Depth</th>
<th>Sample Type</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.4</td>
<td>Tepsoil</td>
<td>LEAN CLAY (CL) with silt, brown, moist, STIFF to VERY STIFF</td>
</tr>
<tr>
<td>6.5</td>
<td>Fat Clay (CH), Brown and Gray, moist, STIFF to VERY STIFF</td>
<td></td>
</tr>
<tr>
<td>12.2</td>
<td>Weathered Limestone</td>
<td></td>
</tr>
<tr>
<td>17.7</td>
<td>Boring Terminated at 17.7 feet</td>
<td></td>
</tr>
</tbody>
</table>

## Test Data

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard Penetration Test</td>
<td>2-4.5 [5]</td>
</tr>
<tr>
<td>Density (g/cc)</td>
<td>6.07 [13]</td>
</tr>
<tr>
<td>Bulk Density (g/cc)</td>
<td>3.33 [6]</td>
</tr>
<tr>
<td>Plastic Limit (%)</td>
<td>21.8</td>
</tr>
<tr>
<td>Liquid Limit (%)</td>
<td>12.7</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>9</td>
</tr>
</tbody>
</table>

### Notes
- Run 1 12.7 to 17.7 ft.
- REC = 86%
- RQD = 62%
**Subsurface Profile – Boring B1, Str. 9A**

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>Depth (FT)</th>
<th>SOIL DESCRIPTION</th>
<th>SPT (B/M)</th>
<th>γ (pcf)</th>
<th>φ (DEG)</th>
<th>qu (tsf)</th>
<th>Sa (Ksi)</th>
<th>Ed (Ksi)</th>
<th>RMR76</th>
<th>Classification</th>
<th>γ (pcf)</th>
<th>φ (DEG)</th>
<th>c' (Ksf)</th>
<th>E (Ksi)</th>
<th>Concrete/Rock Bond Strength (Ksf)</th>
</tr>
</thead>
</table>

**Notes:**
1. For Cohesive Soils, see Pages 2-21 through 2-23
2. For Granular Soils, see Pages 2-16 through 2-18
3. For Rock, see Pages 2-24 through 2-34
4. For Rock, E (KSI) = 0.564 (RMR76)\(^{1.96}\)
Class Problem 2 - Boring Log TB-1

Assignment of Geotechnical Design Parameters
# Boring Log TB-1

## DRILLING and SAMPLING INFORMATION
- **Date Started:** 6/20/06
- **Date Completed:** 6/20/06
- **Hammer Wt.:** 140 lbs.
- **Hammer Drop:** 10 in.
- **Drill Foreman:** W. Bates
- **Spoon Sampler OD:** 2.0 in.
- **Inspector:** M. McBrayer
- **Rock Core Dia.:** -- in.
- **Boring Method:** HSA
- **Shelby Tube OD:** -- in.

## SOIL CLASSIFICATION

<table>
<thead>
<tr>
<th>Surface Elevation</th>
<th>Depth ft</th>
<th>Sample Type</th>
<th>Sample Graphic</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>460.7</td>
<td>4.2</td>
<td>SS</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3.0</td>
<td>SS</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2.0</td>
<td>SS</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>SS</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.0</td>
<td>SS</td>
<td>X</td>
<td></td>
</tr>
</tbody>
</table>

---

<table>
<thead>
<tr>
<th>Test Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard Penetration Test: 11-9-10, 13.8</td>
</tr>
<tr>
<td>Sample Type: SS</td>
</tr>
<tr>
<td>Remarks: Boring location and elevation determined by LORI surveyors. Boring backfilled with soil cuttings at completion. Boring Location: N 35° 56.03800 W 118° 59.33700</td>
</tr>
</tbody>
</table>

## Soil Characteristics
- **Tosstol:** Light brown, slightly moist, very stiff, silty, clay (CL) with small roots.
- **Brown, moist, loose to very loose, silty, sand and gravel (SM) with trace clay.**
- **Brown, moist, very loose, fine to coarse sand (SP) with trace silt and gravel.**
- **Brown, very moist to wet, very loose, silty, fine to medium sand (SP-SM) with trace clay and gravel.**

## Additional Information
- **Sample Type:** SS - Driven Split Spoon, ST - Pressed Shelby Tube, CA - Continuous Flight Auger, RC - Rock Core, CT - Continuous Tube.
- **Depth to Groundwater:** 44.0 ft.
- **Clay Depth:** 23.0 ft.
## Subsurface Profile – Boring TB-1

### Notes:
1. For Cohesive Soils, see Pages 2-21 through 2-23
2. For Granular Soils, see Pages 2-16 through 2-18
3. For Rock, see Pages 2-24 through 2-34
4. For Rock, \( E \) (KSI) = 0.564 \((\text{RMR}_{76})^{1.96}\)
FOUNDATION DESIGN APPROACHES
Foundation Design Approaches

Foundation Design Requires a Blending of Soil/Foundation Interaction Modeling and Engineering Judgement
Foundation Design Approaches

Allowable Stress Design Approach

Reliability-Based Design Approach
Foundation Design Approaches

Maximum Component Design Load - $Q_D$

Component Nominal Capacity - $R_n$

Assumed Safety Factor = ?

How far do you distance $R_n$ from $Q_D$?

$Q_D \leq \frac{R_n}{\text{Assumed Safety Factor}}$

Component Design Load – $\Phi_D$

Component Nominal Capacity - $R_n$
Foundation Design Approaches

Load and Resistance Factor Design (LRFD) Format
Reliability-Based Design Approach
Foundation Design Approaches

\[ R_5 > \text{Effect of } \left[ \text{Dead Load} + \gamma Q_{50} \right] \]

where \( R_5 \) is the 5% required nominal design capacity

\( \gamma \) is the load factor to modify line reliability

\( Q_{50} \) is the 50-year return period climatic event
It can be shown that:

$$R_5 = m_m \left[ 1 - 0.01 \left( 1.64 - 0.00925 V_m \right) V_m \right] R_n$$

Where

$m_m$ is the slope of the least square fit line of a plot of interpreted test capacities, $R_T$, of full-scale load tests, versus predicted nominal ultimate foundation capacities, $R_n$.

$V_m$ is the coefficient of variation of the ratio $m = R_T/R_n$ in %

Thus:

$$\Phi_5 = m_m \left[ 1 - 0.01 \left( 1.64 - 0.00925 V_m \right) V_m \right]$$

and

$$R_5 = \Phi_5 R_n$$
Foundation Design Approaches
Foundation Design Approaches

How Do We Obtain Values of the $m_m$ and $V_m$
For a Particular Foundation Design Model
Foundation Design Approaches

Strength Factors are Determined for Foundation Design Models by a Calibration Process or in Some Cases by Professional Judgment.
Foundation Design Approaches

The Schematic Model for MFAD 5.0
Foundation Design Approaches

From this figure

\[ m_m = 1.02 \]
\[ V_m = 27.2\% \]
Foundation Design Approaches

From this figure

\[ m_m = 1.04 \]
\[ V_m = 5.4\% \]
FOUNDATION FOR SINGLE POLES
## Mode of Foundation Loads and Design Models

<table>
<thead>
<tr>
<th>Mode of Loading</th>
<th>Design Model</th>
<th>Drilled Shafts</th>
<th>Direct Embedment</th>
</tr>
</thead>
<tbody>
<tr>
<td>DRILLED SHAFT</td>
<td>MFAD 5.0</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>DIRECT EMBEDDED POLE</td>
<td>LPILE 5.0</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td>HANSEN</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td>CAISSON</td>
<td>Yes</td>
<td>No</td>
</tr>
</tbody>
</table>

Note: LPILE, Hansen & CAISSON can be used for the design of direct embedded poles but considerable judgment is needed by the designer.
Foundations for Single Poles

The Schematic Drilled Shaft Design Model for MFAD 5.0
MFAD 5.0 Calibration For Drilled Shafts

From this figure
\[ m_m = 1.02 \]
\[ V_m = 27.2\% \]
Foundations for Single Poles

The Schematic Direct Embedment Design Model for MFAD 5.0
Foundations for Single Poles

MFAD 5.0 Calibration for Direct Embedded Poles

From this figure

\( m_m = 1.08 \)

\( V_m = 30.6\% \)
Foundations for Single Poles

The Schematic Drilled Shaft Design Model for LPILE 5.0
From this figure

\[ m_m = 1.36 \]
\[ V_m = 27.4\% \]
Foundations for Single Poles

THE SCHEMATIC DRILLED SHAFT DESIGN MODEL FOR HANSEN
From this figure

\[ m_m = 1.42 \]
\[ V_m = 38.8\% \]
Foundations for Single Poles

The Schematic Drilled Shaft Design Model for CAISSON

- Concrete Strength: 4 ksi
- Steel yield strength: 60 ksi
- Pier Diameter: 7 ft
- Distance of top of pier above ground: 1 ft

**Soil Data**

Soil data are entered from ground level down.
- Soil type: C for cohesive soil (clay), S for cohesionless soil (sand)
- Enter undrained shear strength CU for cohesive soil.
- Enter Rankine Coefficient of earth pressure KP for cohesionless soil.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Type</th>
<th>Thickness (ft)</th>
<th>Density (lbs/ft²·ft³)</th>
<th>Strength (psf)</th>
<th>Rankine Coeff.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C</td>
<td>2</td>
<td>150</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>S</td>
<td>4</td>
<td>50</td>
<td>2.77</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>S</td>
<td>15</td>
<td>50</td>
<td>2.46</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>C</td>
<td>2</td>
<td>55</td>
<td>1000</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>S</td>
<td>4</td>
<td>55</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>S</td>
<td>12</td>
<td>55</td>
<td>2.77</td>
<td></td>
</tr>
</tbody>
</table>

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From this figure

\[ m_m = 1.02 \]

\[ V_m = 0.42 \]
Foundations for Single Poles

Design Example 1

Laterally Loaded Drilled Shaft Hansen Design Model
Foundations for Single Poles

Design Example 1

THE SCHEMATIC DRILLED SHAFT DESIGN MODEL FOR HANSEN
Foundations for Single Poles

Design Example 1

**Cohesive Soils (Undrained Loading)**

where:

\[ q_h = K_c \cdot S_u \]

- \( K_c \) = Hansen’s bearing capacity coefficient shown on the next page; and
- \( S_u \) = undrained shear strength of cohesive soil.

**Granular Soils (Drained Loading)**

where:

\[ q_h = K_q \cdot \sigma_o \]

- \( K_q \) = Hansen’s bearing capacity coefficient shown on the next page;
- \( \sigma_o \) = overburden pressure at depth \( z = \gamma_T z \);
- \( \gamma_T \) = the effective soil density; and
- \( z \) = depth.
Design Example 1

where:

- \( z = \text{depth} \); 
- \( B = \text{drilled shaft diameter} \); 
- \( \Phi = \text{effective friction angle of granular soil} \).
Foundations for Single Poles

DESIRED EXAMPLE 1
EATERLY LOADED DRILLED SHFT - CONSOLIDATED SOIL
BY: EBA DATE: 2/11/11  PROJ NO: 2011-144
CHECKED BY: EBA DATE: 2/17/11  SHEET NO: 1 of 6
DIJA, GRAY & ASSOCIATES, LLC

DETERMINE THE REQUIRED EMBEDMENT DEPTH, D, FOR THE
CONCRETE-DRILLED SHFT SHOWING USING THE HANSEN
METHOD - COMPUTE THE MAXIMUM MOMENT AND SHEAR.

Mom = 1000 ft.kips
Vom = 10 kips

Soil Data
Medium stiff to stiff clay

Unit wt. \( \gamma = 120 \text{pcf} \)
Wet density \( \rho_w = 66 \text{pcf} \)

Undrained compression strength \( q_u = 10 \text{ksi} = 70 \text{kfpf} \)

Undrained shear strength \( S_u = \frac{q_u}{2} = 3.5 \text{ksi} \)

Friction angle \( \phi = 0 \)
Water table at 50 ft.

Strength factor \( \phi_s = 0.75 \) (See page 12-24)

Nominal loads for geological design of the drilled shaft:

\[ M_n = \frac{M_n}{\phi_s} = \frac{1000 \text{ ft-kips}}{0.75} = 1333 \text{ ft-kips} \]

\[ V_n = \frac{V_n}{\phi_s} = \frac{10 \text{ kips}}{0.75} = 13.33 \text{ kips} \]
Foundations for Single Poles

**Design Example 1**

LATERALLY LOADED DRILLED SHAFT - Cohesive Soil

**Subject:**

**By:**

**Checked by:**

**Date:** 2/11

**Promo No.:** 2011-101

**Sheet No.:** 2 of 6

**DiGioia, Gray & Associates, L.L.C.**

---

**Simplified Lateral Pressure Diagram**

\[ M_0 = 140.88 \text{ kips} \]

\[ V_0 = 14.08 \text{ kips} \]

**Equilibrium Equations**

\[ Z F_0 = 0 \]

\[ V_0 - \frac{1}{2} N_{CCB} B^2 - N_{CCB} (D_0 - B) + N_{CCB} (D - D_0) = 0 \]

**Solve for** \( D_0 \)

\[ D_0 = \frac{D}{2} + \frac{B}{4} + \frac{V_0}{2 N_{CCB}} \] **EQUATION 1**
Equilibrium Equations (continued)

\[ \sum M_A = 0 \]

\[ = M_m + \frac{1}{2} N C B^2 + \frac{3}{2} B + N C B (D_o - D) Y + \frac{K_c - G}{2} \]

\[ - N C B (D - A) (D_o + \frac{D - D_o}{2}) = 0 \]

Expand and simplify:

\[ \sum M_A = 0 = M_m + \frac{N C B}{6} (6 D_o^2 - 3 D^2 - E^3) \quad \text{EQ. 2} \]

Method of Solution

Trial and Error Solution Method

First Trial - Try D = 17.0 ft

- For \( X/B = 0.670/B = 0.57 \) (17/4) = 2.86
  - From NC curve on page 10-23, \( N_C = 6.4 \)

- From Equation 1:
  
  \[ D_o = \frac{D}{2} + \frac{B}{4} + \frac{V_n}{2 N C B} = \frac{17}{2} + \frac{4}{2} + \frac{14.08}{2 (6.4) (1.0) (4)} \]
  
  \[ D_o = 9.78 \text{ ft} \]

- Check Equation 2:

\[ \sum M_A = M_m + \frac{N C B}{6} (6 D_o^2 - 3 D^2 - E^3) \]

\[ \sum M_A = 14.08 + \frac{4.56 (4) (4)}{6} \left[ \frac{6 (9.78)^2 - 3 (9.78)^2 - (4)^3}{6} \right] \]

\[ = 89 \text{ ft kips} \approx 0 \quad \text{OK} \]
Foundations for Single Poles

**Design Example 1**

LATERALLY LOADED DRILLED SHAFT - COHESIVE SOIL

CALCULATE MAXIMUM MOMENT AND MAXIMUM SHEAR

**Determine Point of Zero Shear, \( x_0 \) and \( M_{MAX} \)

\[ M_{MIN} = 1106 \text{ ft-kips} \]

\[ V_m = 440 \text{ ft-kips} \]

\[ V = 0 \]

\[ M = M_{MAX} \]

\[ \sum \text{Mom} = 0 \]

\[ M_{MIN} = M_h + V_h x_0 - \frac{1}{2} (N_c C x_0^2 + x_0 \alpha) \]

\[ M_{MAX} = 1410 (2.10^3) - \frac{1}{6} (6.4 \times 10^3 x_0^2) \]

\[ V_{MAX} = 14126 \text{ ft-kips} \]

**Maximum Shear at \( D_0 \)**

\[ \sum \text{V} = 0 \]

\[ V_{MAX} = V_m - \frac{1}{6} N_c C B^2 - N_c C B (D_0 - \frac{B}{2}) \]

\[ V_{MAX} = 14148 - (6.4 \times 10^3 x_0^2 - 9.78 - \frac{B}{2}) \]

\[ V_{MAX} = -185 \text{ kips} \]
Subject: Design Example 1
Laterally Loaded Drilled Shaft - Cohesive Soil

By: AMD Date: 2/5/11
Proj. No.: 2011-181

Checked By: EB Date: 2/7/11
Sheet No.: 5 of 6

Calculate the maximum moment and maximum shear to be used for design of the concrete reinforcement:

\[ M_u = 0.5 \times M_{max} \]
\[ M_u = 0.71 \times (1428 \text{ ft-kips}) \]
\[ M_u = 1014 \text{ ft-kips} \]

\[ V_u = 0.5 \times V_{max} \]
\[ V_u = 0.71 \times (185 \text{ kips}) \]
\[ V_u = 131 \text{ kips} \]
Using the LRFD design format, determine D for the Hansen, MFAD 4.0 and LPILE design methods.
### Design Example 2

<table>
<thead>
<tr>
<th>Design Model</th>
<th>$\Phi_5$</th>
<th>The Required Nominal Design Moment Capacity - $M_{nD}$ (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hansen</td>
<td>0.71</td>
<td>1408</td>
</tr>
<tr>
<td>MFAD 4.0</td>
<td>0.60</td>
<td>1667</td>
</tr>
<tr>
<td>LPILE</td>
<td>0.84</td>
<td>1190</td>
</tr>
</tbody>
</table>

**DESIGN EQUATION**

$$M_{50} \leq \phi_5 M_{nD} \quad \text{or} \quad M_{nD} \geq M_{50} / \phi_5$$

Where $M_{nD}$ is the required nominal design moment capacity.
Design Example 2
Required Nominal Design Moment Capacity ($M_{nD}$) Versus Embedment Depth

- **Assuming a Safety Factor of 2 ($\Phi = 0.5$)**
  - $M_{np} = 1667$ (MFAD)
  - $M_{np} = 1408$ (HANSEN)
  - $M_{np} = 1190$ (LPILE)

- **Embedment Depths and Moment Capacities**
  - 12.0 ft: $M_{np} = 1667$ (MFAD)
  - 12.9 ft: $M_{np} = 1408$ (HANSEN)
  - 13.5 ft: $M_{np} = 1190$ (LPILE)

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Design Example 2
Applied Design Moment ($M_{50}$)
Versus Embedment Depth
Using the LRFD design format, determine D for the Calibrated Hansen, MFAD 4.0 and CAISSON design methods.
Design Example 3

Laterally Loaded Drilled Shaft

<table>
<thead>
<tr>
<th>Design Model</th>
<th>$\Phi_5$</th>
<th>The Required Nominal Design Moment Capacity - $M_{nD}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hansen</td>
<td>0.71</td>
<td>1408</td>
</tr>
<tr>
<td>MFAD</td>
<td>0.60</td>
<td>1667</td>
</tr>
<tr>
<td>CAISSON</td>
<td>0.42</td>
<td>2381</td>
</tr>
</tbody>
</table>

**DESIGN EQUATION**

$M_{50} \leq \phi_5 M_{nD}$ or $M_{nD} \geq M_{50} / \phi_5$

Where $M_{nD}$ is the required nominal design moment capacity.
Foundations for Single Poles

Design Example 3
Required Nominal Design Moment Capacity
Versus Embedment Depth

- $M_n = 1408$ k-ft
- $M_n = 1697$ k-ft
- $M_n = 2381$ k-ft

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Foundations for Single Poles

Design Example 3

Applied Design Moment ($M_{50}$) versus Embedment Depth
Class Problems Using MFAD

• Click Start
• Select All Programs
• Select Launch FAD Tools
• On the Tool Selection box, select MFAD
## Class Problems

<table>
<thead>
<tr>
<th>Class Problem</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Do Design Example 1 using MFAD 5.0</td>
</tr>
<tr>
<td>2</td>
<td>Do Design Example 2 using MFAD 5.0</td>
</tr>
<tr>
<td>3</td>
<td>Do Design Example 3 using MFAD 5.0</td>
</tr>
</tbody>
</table>
Foundations for Single Poles

MFAD Class Problem 4 – Drilled Shaft

• Case: Fine sand – dense
• Description: DS in fine sand – dense
• Foundation Data
  – Name: 5 ft drilled shaft
  – Shaft diameter = 5.0 ft
  – Stick up = 1.0 ft
  – Initial depth of embedment = 10.0 ft
• Geotechnical Parameters
  – Name: Fine sand – dense
• Applied Loads
  – Name: DS Loads for MFAD
  – Load Name: DS Loads
• Performance Criteria
  – Name: DS Criteria
  ✓ Total Deflection = 1.0 in
  ✓ Total Rotation = 1 deg
  ✓ Nonrecoverable Deflection = 0.5 in
  ✓ Nonrecoverable Rotation = 0.5 deg

Total Unit Weight = 125.0 pcf
Deformation Modulus = 6.0 ksi
Friction Angle = 39 deg
Undrained Shear Strength = 0.0 ksf

Ground Water at 30 ft
Foundations for Single Poles

MFAD Class Problem 5 – Direct Embedded Pole with Granular Backfill

- Case: Fine sand – dense
- Description: Direct embedded pole in fine sand – dense
- Foundation Data
  - Name: 5 ft direct embedded pole
  - Shaft diameter = 5.0 ft
  - Initial depth of embedment = 10.0 ft
- Geotechnical Parameters
  - Name: Fine sand – dense
- Annulus Backfill Properties - Soil
  - Annulus Thickness = 0.5 ft
  - Total Unit Weight = 130.0 pcf
  - Deformation Modulus = 3.0 ksi
  - Friction Angle = 40.0 deg
  - Undrained Shear Strength = 0 ksf
- Performance Criteria
  - Name: DS Criteria
  - same criteria as drilled shaft
Foundations for Single Poles

MFAD Class Problem 6 – Direct Embedded Pole with Concrete Backfill

- Case: Fine sand – dense with concrete
- Description: Direct embedded pole in fine sand – dense with concrete backfill
- Foundation Data
  - Name: 5 ft direct embedded pole
  - Shaft diameter = 5.0 ft
  - Initial depth of embedment = 10.0 ft
- Geotechnical Parameters
  - Name: Fine sand – dense
- Annulus Backfill Properties - Concrete
  - Annulus Thickness = 0.5 ft
  - Total Unit Weight = 140.0 pcf
  - Concrete Strength = 3000 psi
- Performance Criteria
  - Name: DS Criteria
  - same criteria as drilled shaft

<table>
<thead>
<tr>
<th>Total Unit Weight</th>
<th>= 125.0 pcf</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deformation Modulus</td>
<td>= 6.0 ksi</td>
</tr>
<tr>
<td>Friction Angle</td>
<td>= 39 deg</td>
</tr>
<tr>
<td>Undrained Shear Strength</td>
<td>= 0.0 ksf</td>
</tr>
</tbody>
</table>

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MFAD Class Problem 7 – Drilled Shaft

- Case: Fine sand – dense on rock
- Description: DS in fine sand – dense on rock
- Foundation Data
  - Name: 5 ft drilled shaft
  - Shaft diameter = 5.0 ft
  - Stick up = 1.0 ft
  - Initial depth of embedment = 10.0 ft
- Geotechnical Parameters
  - Name: Fine sand – dense on rock
- Applied Loads
  - Name: DS Loads for MFAD
  - Load Name: DS Loads
- Performance Criteria
  - Name: DS Criteria
  - Total Deflection = 1.0 in
  - Total Rotation = 1 deg
  - Nonrecoverable Deflection = 0.5 in
  - Nonrecoverable Rotation = 0.5 deg

Total Unit Weight = 125.0 pcf
Deformation Modulus = 6.0 ksf
Friction Angle = 36 deg
Undrained Shear Strength = 0.0 ksf

Total Unit Weight = 140.0 pcf
Deformation Modulus = 360.0 ksf
Friction Angle = 32 deg
Rock Cohesion = 2.4 ksf
Rock/Conc Bond Strength = 2.0 ksf

Depth to Ground Water = 15 ft
Foundations for Single Poles

MFAD Class Problem 8 – Direct Embedded Pole with Granular Backfill

- **Case:** Fine sand – dense on rock
- **Description:** Direct embedded pole in fine sand – dense on rock

**Foundation Data**
- Name: 5 ft direct embedded pole
- Shaft diameter = 5.0 ft
- Initial depth of embedment = 10.0 ft

**Geotechnical Parameters**
- Name: Fine sand – dense on rock

**Annulus Backfill Properties - Soil**
- Annulus Thickness = 0.5 ft
- Total Unit Weight = 130.0 pcf
- Deformation Modulus = 3.0 ksi
- Friction Angle = 40.0 deg
- Undrained Shear Strength = 0 ksf

**Performance Criteria**
- Name: DS Criteria
- same criteria as drilled shaft

---

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Foundations for Single Poles

MFAD Class Problem 9 – Direct Embedded Pole with Concrete Backfill

- Case: Fine sand – dense on rock with concrete
- Description: Direct embedded pole in fine sand – dense on rock with concrete backfill
- Foundation Data
  - Name: 5 ft direct embedded pole
  - Shaft diameter = 5.0 ft
  - Initial depth of embedment = 10.0 ft
- Geotechnical Parameters
  - Name: Fine sand – dense on rock
- Annulus Backfill Properties - Soil
  - Annulus Thickness = 0.5 ft
  - Total Unit Weight = 140.0 pcf
  - Concrete Strength = 3000 psi
- Performance Criteria
  - Name: DS Criteria
  - same criteria as drilled shaft

Total Unit Weight = 125.0 pcf
Deformation Modulus = 6.0 ksi
Friction Angle = 39 deg
Undrained Shear Strength = 0.0 ksf

Total Unit Weight = 140.0 pcf
Deformation Modulus = 380.0 ksi
Friction Angle = 32 deg
Rock Cohesion = 2.4 ksf
Rock/Conc Bond Strength = 2.0 ksf

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FOUNDATION FOR LATTICE TOWERS
Foundations for Lattice Towers

- Drilled Shafts
- Steel Grillages
- Reinforced Concrete Spread Footings
- Rock Foundations
Foundations for Lattice Towers

Typical Steel Grillage Foundations

(a) (b) (c)
Foundations for Lattice Towers

Pyramidal Grillage Foundations
# Foundations for Lattice Towers

## Foundation Design Models for Various Foundation Types

<table>
<thead>
<tr>
<th>Design Model</th>
<th>Foundation Types</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uplift Compression and Lateral Load Models (MFAD, LPILE, HANSEN, CAISSON)</td>
<td>Drilled Shaft: Yes, Spread Footing: Yes</td>
</tr>
<tr>
<td>TFAD</td>
<td>Drilled Shaft: Yes, Spread Footing: No</td>
</tr>
</tbody>
</table>
Design Model Calibration

- Using full-scale load tests of drilled shafts in uplift, each of the design models can be calibrated for use in design.
Foundations for Lattice Towers

Drilled Shaft Foundations Loaded in Uplift – Side Shear Design Model

\[ R_n = \overline{W_c} + \overline{T} \]
\[ R_5 = \Phi_5 R_n \]
\[ R_5 = \Phi_5 (\overline{W_c} + \overline{T}) \]

where:
\[ R_n \] = ultimate nominal uplift strength;
\[ \overline{W_c} \] = effective weight of the concrete drilled shaft;
\[ \overline{T} \] = effective total side shear geotechnical resistance; and
\[ R_5 \] = design strength.
Foundations for Lattice Towers

Drilled Shaft Foundations Loaded in Uplift

- Side Shear Design Model - Cohesive Soils

The value of $T$, the shaft side resistance under undrained loading conditions, can be calculated from:

$$ T = \alpha S_u (\pi B_s D) $$

where:
- $\alpha$ = the adhesion factor
- $S_u$ = undrained shear strength of soil
- $B_s$ = the shaft diameter
- $D$ = the shaft depth

Values for $\alpha$ can be obtained from this figure:
Calibration of Side Shear Design Model with a Single Strength Factor Drilled Shafts Loaded in Undrained Uplift (EPRI Test Data – 1984)

From this figure
\[ m_m = 1.01 \]
\[ V_m = 27.2\% \]
Summary – Side Shear Design Model – Drilled Shafts Loaded in Undrained Uplift

\[ n = 65 \]
\[ m_m = 1.01 \]
\[ V_m = 27.2\% \]
\[ \Phi_5 \text{ (Lognormal)} = 0.63 \]
Design Example 1

Determine the Nominal and Design Uplift Capacity of a Drilled Shaft Foundation Loaded in Undrained Uplift Using the Cylindrical Shear Design Model
Foundations for Lattice Towers

Design Example 1
Uplift Capacity of a Drilled Shaft in Cohesive Soil

Φ₅ = 0.63 (See Page 5-11)
Foundations for Lattice Towers

Design Example 1
Uplift Capacity of a Drilled Shaft in Cohesive Soil

\[ R_m = W_c + T \]

\[ W_c = \text{Effective weight of concrete foundation} \]
\[ = \frac{h}{a} \gamma_c b^2 D = \frac{h}{a} (0.15)(3)^2(15) = 15.9 \text{ kips} \]

\[ T = \text{Effective side shear resistance} \]
\[ T = \alpha \cdot S_w \cdot (h b D) \]

From SRS & Kulhawy curve can find:
\[ \alpha = 0.21 + 0.27/5(30') \]
\[ = 0.21 + 0.27/0.785 \]
\[ = 0.57 \]

\[ T = 0.57 (1.50)(3)(15) = 120.9 \text{ kips} \]

\[ R_m = W_c + T = 15.9 + 120.9 = 136.8 \text{ kips} \]

\[ R_s = \phi_s (R_m) \]
\[ = 0.63(136.8) \]
\[ = 86 \text{ kips} \]
Class Problem 1
Find the nominal ($R_n$) and Design Uplift Capacity ($R_D$) of the drilled shaft shown using the side shear design model.

**Stiff Clay**
- Unit Weight – 120 pcf
- Undrained Shear Strength = 1.5 ksf

**Concrete**
- Unit Weight = 150 pcf

**Ground Water Level**
- Depth = 30 ft
Foundations for Lattice Towers

Drilled Shaft Compression Capacity

The ultimate nominal capacity, \( Q_m \) of a drilled shaft loaded in compression is given by:

\[
Q_m = Q_B + Q_s
\]

where:

- \( Q_m \) = ultimate nominal end-bearing capacity of the base
- \( Q_B \) = ultimate nominal end-bearing capacity of the base
- \( Q_s \) = ultimate nominal cylindrical shear capacity of the shaft

Design Capacity - \( Q_D \)

\[
Q_D = \phi_{5eb}Q_B + \phi_{5cs}Q_s
\]

where:

- \( \phi_{5eb} \) is the strength factor applied to the ultimate nominal end-bearing capacity
- \( \phi_{5cs} \) is the strength factor applied to the ultimate nominal side shear capacity
The ultimate nominal end-bearing capacity, $Q_B$, for circular drilled shafts can be computed as follows:

**Cohesive Soils (undrained loading)**

$$Q_B = A_B N_C S_U$$  \hspace{1cm} EQ. 1

where:
- $A_B$ = the area of the base
- $N_C$ = a bearing capacity factor $\approx 9$
- $S_U$ = undrained shear strength

**Granular Soils (drained loading)**

$$Q_B = A_B (0.3 \gamma B_B N_Y + \gamma D N_q)$$  \hspace{1cm} EQ. 2

where:
- $\gamma$ = effective density of soil
- $B_B$ = base diameter
- $D$ = shaft depth below ground surface
- $N_Y$ & $N_q$ = bearing capacity factors

Since the value of $B_B$ is relatively small, it is normally dropped from EQ.2.

Values $N_q$ are shown on the figure on the following page.
Foundations for Lattice Towers
Foundations for Lattice Towers

Design Example 2

Find the Nominal ($R_n$) and Design Capacity ($R_D$) of a Drilled Shaft in a Granular Soil and Subjected to a Compression Load
Design Example 2
Compressive Capacity of a Drilled Shaft in Granular Soil

Soils Data: (given)
- Medium to Dense Granular
- Moist Unit Wt. of Soil, \( v_s = 180 \text{pcf} \)
- Effective Internal Friction Angle, \( \phi = 35^\circ \)
- Water Table at Depth of 30 ft

Calculate Nominal Compression Load Capacity, \( Q_n \)

\[ Q_n = Q_{nd} + Q_{as} \] (See Page 5-16)

- \( Q_{nd} = \) Nominal Load Carrying Capacity of the Base
- \( Q_{as} = \) Effective Total Nominal Cylindrical Shear Capacity of the Shaft
Foundations for Lattice Towers

Design Example 2 -- Calculate the Design Example

CALCULATE THE DESIGN COMPRESSION CAPACITY, $Q_0$

WHERE $Q_0 =$ DESIGN STRENGTH

$Q_{0b} = \left( C N_2^b + \frac{1}{2} B N_y N_b^b + \frac{1}{6} N_b^b \right) A_F$

$N_2 N_y N_b^b =$ BEARING CAPACITY FACTORS

$\frac{1}{2} B N_y N_b^b = 0$ \text{BECAUSE} B IS SMALL

$C N_2 = 0$ \text{BECAUSE} COHESION, $C = 0$

$Q_{0b} = \left( 0 N_b^b \right) A_F$

$Q_{0b} = \left( 0.1 D N_b^b \right) A_F$

$Q_{0b} = 130 \times 0.3 \times 20 = 58 \times 7.111$

$Q_{0b} = 1030.7$ Kip

$Q_{ns} = K_0 \left( \tan \frac{4}{5} \right) \sigma_2 \times \frac{1}{2} B \times D$

$K_0 = \left( 1 - \sin \frac{4}{5} \right) = \left[ 1 - \sin \left( 35^\circ \right) \right] = 0.426$

$Q_{ns} = 0.426 \left[ \tan \left( 35^\circ \right) \right] \times \frac{1}{2} \times 130 \times 0.3 \times 20 \times \frac{1}{2} \times 3 \times 20$

$Q_{ns} = 73.1$ Kip
Design Example 2

Compute Nominal Strength, $Q_n$

$Q_n = Q_{ne} + Q_{ns}$

$Q_n = 1070.7 \text{ kips} + 73.1 \text{ kips}$

$Q_n = 1143.8 \text{ kips}$

Compute Design Strength, $Q_d$

$Q_d = 0.45(1070.7 \text{ kip}) + 0.56(73.1 \text{ kip})$

$Q_d = 481.2 \text{ kips} + 40.9 \text{ kips}$

$Q_d = 522.1 \text{ kips}$
Foundations for Lattice Towers

Design Example 3

Find the Nominal ($R_n$) and Design Capacity ($R_D$) of a Drilled Shaft in a Cohesive Soil and Subjected to a Compression Load
Design Example 3
Compressive Capacity of a Drilled Shaft in Cohesive Soil

SOILS DATA: (GIVEN)
- STIFF TO VERY STIFF COHESIVE
- MOIST UNIT WT. OF SOIL
  \( \gamma_s = 130 \text{ psf} \)
- UNDRAINED SHEAR STRENGTH
  \( C = 2000 \text{ psf} \)
- WATER TABLE AT A DEPTH OF 30 FT

CALCULATE NOMINAL COMPRESSION LOAD CAPACITY, \( Q_n \)

\[
Q_n = Q_{as} + Q_{as}
\]

\( Q_{as} = \) NOMINAL LOAD CARRYING CAPACITY OF THE BASE
\( Q_{as} = \) EFFECTIVE TOTAL NOMINAL CYLINDRICAL SHEAR CAPACITY OF THE SHAFT

(See Page 5-16)
Design Example 3

\[
Q_d = \frac{3}{4} E_b \cdot R_{Ng} + \frac{3}{4} CS \cdot R_{Ns}
\]

WHERE \( Q_d = \) Design compression capacity, kips

\[
Cylindrical Shear = \frac{1}{2} CS \cdot 0.85 \ (AASHTO 2007)
\]

ENI bending = \( \frac{0.8B \cdot 0.85}{2} \) \( 0.15 \) \( E_b \) \( A \) \( (AASHTO 2007) \)

\[
G_{NB} = (c \cdot N_c^* + \frac{1}{2} \cdot B \cdot N_b^* \cdot N_c^*) \cdot A_f
\]

\[
N_c^*, N_b^* - Bearing capacity factors
\]

\[
\frac{1}{2} B \cdot N_b \cdot N_c = 0 \text{ because } B \text{ is small}
\]

\[
G_{NB} = \left[ 2000 \cdot 9 \cdot \left( 9 \right) \cdot \left( 20 \right) \cdot \left( 180 \right) \cdot A \cdot (A) \cdot (A) \right] \cdot A_f
\]

\[
G_{NB} = \left( 18 \cdot 4 \cdot 9 \cdot 7 + 2.6 \cdot 6 \cdot 7 \right) \times 7.1 \ ft^2
\]

\[
G_{NB} = 146.3 \text{ kips}
\]

\[
G_{NS} = \alpha^* \cdot \gamma \cdot D
\]

\[
\alpha^* = \text{Reinforcement factor} = 0.21 + 0.5 \cdot \frac{1}{50} = 0.21 + 0.1 = 0.31
\]

\[
G_{NS} = 0.31 \cdot 2000 \cdot 0.8^2 \times \gamma \times 3 \text{ ft} \times 20 \text{ ft}
\]

\[
G_{NS} = 181 \text{ kips}
\]
Design Example 3

**Compute Nominal Strength, \( Q_n \)**

\[ Q_n = Q_{nb} + Q_{ns} \]
\[ Q_n = 146.3 \text{ Kips} + 181.0 \text{ Kips} \]
\[ Q_n = 327.3 \text{ Kips} \]

**Compute Design Strength, \( Q_d \)**

\[ Q_d = 0.55 \left( 146.3 \text{ Kips} \right) + 0.65 \left( 181 \text{ Kips} \right) \]
\[ Q_d = 80.5 \text{ Kips} + 117.7 \text{ Kips} \]
\[ Q_d = 198.2 \text{ Kips} \]
Foundations for Lattice Towers

Class Problem 2

Find the Nominal ($R_n$) and Design Compression ($R_D$) Capacities of the Drilled Shaft Shown Below

Stiff Clay
- Unit Weight = 120 pcf
- Undrained Shear Strength = 1.5 ksf

Concrete
- Unit Weight = 150 pcf

Groundwater Level
- Depth = 30 ft
Foundations for Lattice Towers

Side Shear Design Model for Spread Footings Loaded in Uplift

$$R_n = W_c + W_s + T$$

$$R_n = \Phi_5 R_n$$

$$R_n = \Phi_5 \left( W_c + W_s + T \right)$$

Common sense tells us in this case that $W_c$ and $W_s$ are each significant contributors to uplift resistance for spread footings.

However, let’s initially use the single strength factor approach in the calibration process.
Calibration of Side Shear Design Model with a Single Strength Factor-Spread Footings Loaded in Drained Uplift

From this figure

\[ m_m = 1.38 \]

\[ V_m = 46.8\% \]
Foundations for Lattice Towers

Should we consider using two strength factors in this case?

It makes sense, let’s do it!
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Calibration of Side Shear Design Model and Two Strength Factors-Spread Footings Loaded in Drained Uplift

From this figure

\[ m_m = 1.73 \]

\[ V_m = 68.4\% \]
Summary of Calibration Results – Side Shear Design Model

Spread Footings in Drained Uplift

Single Strength Factor - $R_5 = \Phi_5 \left( \overline{W_c} + \overline{W_s} + T \right)$

$n = 57$

$m_m = 1.38$

Two Strength Factors - $R_n = \Phi_{DW} \overline{DW} + \Phi_{5GR} \overline{GR}$

$n = 57$

$m_m = 1.73$

$Mod\ COV_m = 68.4\%$

$\Phi_{5GR} = 0.51$

$\Phi_{DW} \ (Assumed) = 0.9$
## Summary of Calibration Results – Spread Footings in Uplift

<table>
<thead>
<tr>
<th>Foundation Type</th>
<th>Design Model</th>
<th>Mode of Loading</th>
<th>Strength Factor, $\Phi_5$ (Lognormal)</th>
<th>Single $\Phi_5$</th>
<th>Two $\Phi_5$’s</th>
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<tr>
<td>Spread Footing</td>
<td>Side Shear</td>
<td>Drained Uplift</td>
<td>$\Phi_{DW}$ Dead Wt. Factor, $\Phi_{5GR}$</td>
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<td>0.9</td>
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</table>
Design Example 4

Uplift Capacity of a Spread Footing with a Granular Backfill

Side Shear Design Model Using a Single Strength Factor
Foundations for Lattice Towers

Design Example 4

Foundation Loads
(50-Yr Return Period)

Q_{50} = 100 kips
Granular Site –
Drained Loading

GEOTECHNICAL DATA
Compact Granular Backfill
- \gamma_T = 130 PCF
- Friction Angle = 35°
- Ratio of Horizontal Stress to Vertical Stress = 1.0

CONCRETE DATA
Density = 150 PCF

Groundwater Level
at D = 20 ft
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Design Example 4

Find the uplift capacity, $R_u$, of the concrete spread footing shown using the shear method.

**Trial 1 - Assume $D = 80$ ft**

**W**

- $W_1$ (water weight)
- $W_2$ (dead load)
- $W_3$ (live load)
- $T$ (force)

**Soil Data**
- Medium to dense, compacted granular backfill.
- Initial unit wet of soil $\gamma_s = 130$pcf
- Effective internal friction angle: $\phi = 35^\circ$
- Water table at 20 ft
- Unit wet of concrete, $\gamma_c = 150$pcf

**Strength Reduction Factor**
- $R_b = 0.80$ (limit state approach; see page 5-33)

**Calculate Uplift Strength - $R_u$**

$$R_u = R_m \cdot \phi$$

Where $R_m =$ nominal uplift strength of foundation

$$R_m = \frac{W_c + W_3 + T}{B}$$

(See Page 5-28)

Effective wet of foundation

$$W_c = \phi_c B^2 T_e \sqrt{(D \cdot T_e) (T_s)^2}$$

$$= 0.15 \left[ (4)^2 (2) + (8 - 2) (2)^2 \right] = 14.4 \text{ kips}$$
Foundations for Lattice Towers

Design Example 4

\[
W_s = \text{effective wt. of soil} = \left[ B^2 (D - T_r) - T_s^2 (D - T_r) \right] = 0.13 \left[ (6)^2 (8 - 2) - (2)^2 (8 - 2) \right] = 25.0 \text{ kips}
\]

\[
T = \text{fractional soil resistance} = 2 K \frac{W_s B D^2 \tan \phi}{\phi}
\]

\[ K = 1.0 \text{ (well compacted backfill)} \]

Therefore,

\[
T = 2 (1.0) (0.13) (6) (8)^2 \tan(25^\circ) = 70.2 \text{ kips}
\]

Compute nominal strength - \( P_n \)

\[
P_n = (W_c + W_s + F)
\]

\[
P_m = 14.4 + 250 + 70.2 = 334.6 \text{ kips}
\]

\[
P_n = 109.6 \text{ kips}
\]

Compute design strength - \( P_e \)

\[
P_e = \Phi_e P_m
\]

\[
P_e = 0.6 (110) = 66 \text{ kips}
\]
Foundations for Lattice Towers

Design Example 4

\[ P_h = \frac{A_h}{2} \left( B \right) \left( D_e \right) \]

\[ \bar{V} = \frac{\bar{D}_c}{\bar{v}} \]

\[ \bar{D}_c = \bar{P} \bar{V} \]

\[ \bar{V} = \bar{D}_e \]

\[ \bar{D}_e = \bar{P}_e \]

\[ \bar{P}_e = A_e \bar{V} \]

\[ A_h = \frac{4}{4} \left( A \bar{V} B \left( D_e \right) \right) \]

\[ \bar{v} = 2 \bar{A} \bar{B} \left( D_e \right)^2 \bar{D}_e \bar{v} \]
The ultimate nominal compression capacity, \( Q_n \), of a spread foundation is given by:

\[
Q_{nB} = q_n( A_F ) = \left( cN_c + \frac{1}{2} B \gamma N_\gamma + qN_q \right) A_F
\]

where: 
- \( A_F \) = base area of foundation
- \( C \) = effective cohesion of soil
- \( B \) = width of the foundation
- \( q \) = effective surcharge at depth \( D \)
- \( D \) = depth of the foundation
- \( \gamma \) = effective density of the soil

\( N_c, N_\gamma \) and \( N_q \) are dimensionless bearing capacity factors.
Foundations for Lattice Towers

Spread Footings – Compression Capacity

Values for $N_c$, $N_y$, and $N_q$ are given in the Figure below:

$$N_q = e^{\tan \theta} \tan^2 \left(\frac{45 + \theta}{2}\right)$$
$$N_c = \left(N_q - 1\right) \cot \theta$$
$$N_y = 2(N_q + 1) \tan \theta$$
Foundations for Lattice Towers

Spread Footings – Compression Capacity

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<th>$\phi$</th>
<th>$N_r$</th>
<th>$N_k$</th>
<th>$N_r/N_k$</th>
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* After Vesic (1973)
Design Example 5

Find the Nominal ($Q_n$) and Design ($Q_D$) Capacities of a Reinforced Concrete Footing on a Granular Soil and Subjected to a Compression Load
Foundations for Lattice Towers

Design Example 5 – Compression Capacity of a Spread Footing in Granular Soil

Find the ultimate nominal and design bearing capacities, $Q_n$ and $Q_d$, of a concrete spread foundation using the general shear failure model.

Soils Data: (Given)
- Medium to dense granular
- Moist unit wt of soil $y_s = 130$pcf
- Effective internal friction angle $\phi = 35^\circ$
- Water table at a depth of 20 ft
- Footing area $A_f = 6' x 6' = 36$ ft$^2$ (Given)

Calculate nominal bearing capacity, $Q_n$

$$Q_n = q_n (A_f)$$

$$Q_n = (c N_c + \frac{1}{2} b Y N_y + q N_d) A_f$$

(See Page 5-39)

For simplicity, neglect difference in soil and concrete unit weight.
Foundations for Lattice Towers

Design Example 5

CALCULATE NOMINAL BEARING CAPACITY (CONT.)

\[ N_b = \tan^2(45 + \frac{\theta}{2}) = 33.3 \quad (\text{REISSNER 1924}) \]
\[ N_c = (N_q - 1) \cos \theta = 46.12 \quad (\text{FRANTZ 1921}) \]
\[ N_q = 2(N_b + 1) \tan \theta = 48 \quad (\text{VESIC 1973}) \]
\[ q_n = \sqrt[4]{(46.12)^2 + \left( \frac{1}{2} \right)^2 \left( 130 \right)^2 (14) + \left( 130 \right)^2 (8)^2 (13.3)} \]
\[ q_n = 53.3 \quad \text{kip/ft}^2 \]
\[ q_n = 0 + 18.7 \text{kip/ft}^2 + 34.6 \text{kip/ft}^2 = 53.3 \text{kip/ft}^2 \]

COMPUTE ULTIMATE NOMINAL BEARING CAPACITY, \( Q_n \)

\[ Q_n = q_n A_f = 53.3 \text{ kip/ft}^2 \times 36 \text{ ft}^2 = 1919 \text{ kips} \]

COMPUTE DESIGN BEARING CAPACITY, \( Q_D \)

\[ Q_D = \phi \times Q_n \]

WHERE \( \phi \) IS THE STRENGTH FACTOR AND \( \phi = 0.4 \). AVERAGE OF RATIONAL METHOD - SAND, TABLE 15.5.1 "ASHTO LAYD BRIDGE DESIGN SPECIFICATION CUSTOMARY US UNITS" THIRD EDITION.

\[ Q_D = 0.4 (1919 \text{ kips}) \]
\[ Q_D = 768 \text{ kips} \]
Foundations for Lattice Towers

Class Problem 3
Find the Ultimate Nominal ($Q_n$) and Design Compression ($Q_D$) Capacities for the Concrete Spread Foundation shown below

Soil
Stiff to V. Stiff Clay
Moist Unit Weight = 130 pcf
Undrained Shear Strength = 2.2 ksf
Water Level at a Depth of 20 ft
Foundations for Lattice Towers

Design Example 6

Nominal \((Q_n)\) and Design \((Q_D)\) Capacities of a Reinforced Concrete Footing on a Cohesive Soil and Subjected to a Compression Load
Foundations for Lattice Towers

Design Example 6
Bearing Capacity of a Spread Footing in Cohesive Soil

Find the ultimate nominal and design bearing capacities, $Q_u$ and $Q_d$, of a concrete spread foundation using the general shear failure model.

Soils Data:
- Given
- Stiff to very stiff cohesive
- Moist unit weight of soil $\gamma_s = 130$pcf
- Undrained shear strength $C = 2000$psf
- Water table at a depth of 20ft
- Footing area $A_f = 6' \times 6' = 36$ ft$^2$ (Given)

Calculate nominal bearing capacity, $Q_n$

$Q_n = \frac{q_n}{A_f}$

$Q_n = \left( \frac{C}{\gamma_s} + \frac{C}{\gamma_s} N_{k_8} + q N_b \right)A_f$  (See Page 5-39)

For simplicity, neglect difference in soil and concrete unit weight.
Foundations for Lattice Towers

Design Example 6
Bearing Capacity of a Spread Footing in Cohesive Soil

CALCULATE NOMINAL BEARING CAPACITY (CONT.)

\[ N_b = \sigma_b \tan \theta \tan \phi = 1 \quad \text{(Reissner 1924)} \]

\[ N_c = (N_b - 1) \tan \phi = 5.14 \quad \text{(Fellenius 1921)} \]

\[ N_y = 2(N_b + 1) \tan \phi = 0 \quad \text{(Vesic 1973)} \]

\[ q_c = 2000 \frac{\gamma h^2}{k} \frac{1}{2} \left( \frac{h}{c} \right)^2 \left( 1 - 0.5 \frac{h}{c} \right) \left( 1 + 0.5 \frac{h}{c} \right) \]

\[ q_n = 10.28 \text{ ksi ft}^2 + 0 + 1.04 \text{ ksi ft}^2 = 11.32 \text{ ksi ft}^2 \]

COMPUTE ULTIMATE NOMINAL BEARING CAPACITY, \( Q_n \)

\[ Q_n = q_n \times A_f = 11.32 \text{ ksi ft}^2 \times 36 \text{ ft}^2 = 407.5 \text{ kips} \]

COMPUTE DESIGN BEARING CAPACITY \( Q_d \)

\[ Q_d = \phi_d \times Q_n \]

WHERE \( \phi_d \) IS THE STRENGTH FACTOR AND EQUAL 0.55, AVERAGE OF RATIONAL METHOD - CLAY, TABLE 10.5-3.1 "MAYAUF CRFD BRIDGE DESIGN SPECIFICATION CUSTOMARY US UNITS." THIRD EDITION.

\[ Q_d = 0.55 \times 407.5 \text{ kips} \]

\[ Q_d = 224 \text{ kips} \]
Foundations for Lattice Towers

We Now Know How to Design for Uplift and Compression Loads. How do We Consider the Lateral Shear Load?
TFAD 5.0
TFAD executes the flow diagram shown on the previous page. TFAD uses the following design models for the design of drilled shafts.

**Lateral Shear Loads**
- MFAD 5.0 – Lateral Resistance Only

**Uplift Loads**
- Cylindrical Shear Design Model

**Compression Loads**
- Cylindrical Shear & Vesic Design Models

Note: For the design of direct embedded legs, neglect the Vesic end bearing capacity.
Foundations for Lattice Towers

Class Problems Using TFAD

• Click Start
• Select All Programs
• Select Launch FAD Tools
• On the Tool Selection box, select TFAD
Find the Nominal ($R_n$) and Design ($R_D$) Uplift Capacities of the drilled shaft shown on Page 5-13 using TFAD
Class Problem 6 Using TFAD

Find the Nominal ($R_n$) and Design ($R_D$) Compression Capacities of the drilled shaft shown on Page 5-15 using TFAD
Class Problem 7 – Using TFAD

- Case: Hard cohesive soil
- Description: DS in hard cohesive soil
- Foundation Data
  - Name: 4 ft drilled shaft
  - Shaft diameter = 4.0 ft
  - Stick up = 1.0 ft
  - Initial depth of embedment = 8.0 ft
- Geotechnical Parameters
  - Name: Hard cohesive soil
- Applied Loads
  - Name: DS Loads for TFAD

<table>
<thead>
<tr>
<th>Mode</th>
<th>Axial</th>
<th>Shear</th>
<th>Load Case</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Uplift with Associated Shear</td>
<td>350</td>
<td>25</td>
<td>1/2” ice</td>
</tr>
<tr>
<td>Maximum Compression with Associated Shear</td>
<td>325</td>
<td>50</td>
<td>Extreme Wind</td>
</tr>
<tr>
<td>Maximum Shear under uplift with Associated Uplift</td>
<td>10</td>
<td>30</td>
<td>Maintenance</td>
</tr>
<tr>
<td>Maximum Shear under compression with Associated Compression</td>
<td>275</td>
<td>50</td>
<td>Broken Conductor</td>
</tr>
</tbody>
</table>

Total Unit Weight = 135.0 pcf
Deformation Modulus = 4.0 ksi
Friction Angle = 0 deg
Undrained Shear Strength = 3.4 ksf

Ground Water = 30 ft
Foundations for Lattice Towers

TFAD Class Problem 8 – Drilled Shaft

- Case: Hard cohesive soil on rock
- Description: DS in hard cohesive soil on rock
- Foundation Data
  - Name: 4 ft drilled shaft
  - Shaft diameter = 4.0 ft
  - Stick up = 1.0 ft
  - Initial depth of embedment = 8.0 ft
- Geotechnical Parameters
  - Name: Hard cohesive soil on rock
- Applied Loads
  - Name: DS Loads for TFAD

<table>
<thead>
<tr>
<th>Mode</th>
<th>Axial</th>
<th>Shear</th>
<th>Load Case</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Uplift with Associates Shear</td>
<td>350</td>
<td>25</td>
<td>1/2&quot; ice</td>
</tr>
<tr>
<td>Maximum Compression with Associated Shear</td>
<td>325</td>
<td>50</td>
<td>Extreme Wind</td>
</tr>
<tr>
<td>Maximum Shear under uplift with Associated Uplift</td>
<td>10</td>
<td>30</td>
<td>Maintenance</td>
</tr>
<tr>
<td>Maximum Shear under compression with Associated Compression</td>
<td>275</td>
<td>50</td>
<td>Broken Conductor</td>
</tr>
</tbody>
</table>

- Total Unit Weight = 135.0 pcf
- Deformation Modulus = 4.0 ksi
- Friction Angle = 0 deg
- Undrained Shear Strength = 3.4 ksf

- Total Unit Weight = 140.0 pcf
- Deformation Modulus = 360.0 ksi
- Friction Angle = 32 deg
- Rock Cohesion = 2.4 ksf
- Rock / Concrete Bond Strength = 2.0 ksf

Depth to Ground Water = 15 ft
Foundations for H-Frames
Foundations for H-Frames

- Drilled Shafts
- Direct Leg Embedment
- Reinforced Concrete Spread Footings
## Foundations for H-Frames

### Foundation Design Models for Various Foundation Types

<table>
<thead>
<tr>
<th>Design Model</th>
<th>Drilled Shaft</th>
<th>Direct Embedment</th>
<th>Spread Footing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uplift Compression and Lateral Load Models (MFAD, LPILE, HANSEN, CAISSON)</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>MFAD 5.0</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
</tr>
</tbody>
</table>
Foundations for H-Frames

Suggested Superposition Approach

[Diagram showing the suggested superposition approach for H-frames with labeled steps and decision points.]

Given B, Determine D_1 to Resist M & V
Use MFAD, PILE, HANSEN, or CAISSON

Use Lateral Resistance Only

Check if D_1 is sufficient to Resist U
Use Cylindrical Shear Design Model

Yes

Check if D_1 is sufficient to resist C. Use cylindrical shear & vesic design models

Yes

Increase D_1 to D_1 to Resist U. Use cylindrical shear design model.

No

Select D_2

Yes

Select D_3

No

Increase D_1 to D_2 to Resist U. Use Cylindrical Shear Design Model

Select D_1

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HFAD 5.0

HFAD executes the flow diagram shown on the previous page. HFAD uses the following design models.

- **Lateral Shear and Moment Loads**
  - MFAD 5.0 – Lateral Resistance Only

- **Uplift Loads**
  - Cylindrical Shear Design Model

- **Compression Loads**
  - Cylindrical Shear & Vesic Design Models

Note: For the design of direct embedded legs, neglect the vesic end bearing capacity.
Foundations for H-Frames

Design Example 1 – Using HFAD

- Case: Soft to medium stiff soil
- Description: DS in cohesive soil
- Foundation Data
  - Name: 6 ft drilled shaft
  - Shaft diameter = 6.0 ft
  - Stick up = 1.0 ft
  - Initial depth of embedment = 12.0 ft
- Geotechnical Parameters
  - Name: Soft to medium stiff soil
- Applied Loads
  - Name: DS Loads for HFAD
  - Load Name: DS Loads

Uplift Leg
- \( V = 25.0 \text{ kips} \)
- \( M = 1045.0 \text{ kip-ft} \)
- \( U = 125.0 \text{ kips} \)

Compression Leg
- \( V = 20.0 \text{ kips} \)
- \( M = 867.0 \text{ kip-ft} \)
- \( C = 50.0 \text{ kips} \)

Total Unit Weight = 119.0 psf
Deformation Modulus = 0.33 ksi
Friction Angle = 0 deg
Undrained Shear Strength = 0.6 ksf
Foundations for H-Frames

Class Problems using HFAD

• Click Start
• Select All Programs
• Select Launch FAD Tools
• On the Tool Selection box, select HFAD

![Tool Selection](image)
Foundations for H-Frames

HFAD Class Problem 1 – Drilled Shaft

- Case: Soft to medium stiff soil on rock
- Description: DS in cohesive soil on rock
- Foundation Data
  - Name: 6 ft drilled shaft
  - Shaft diameter = 6.0 ft
  - Stick up = 1.0 ft
  - Initial depth of embedment = 12.0 ft
- Geotechnical Parameters
  - Name: Soft to medium stiff soil on rock
- Applied Loads
  - Name: DS Loads for HFAD
  - Load Name: DS Loads

\[
\begin{align*}
\text{Uplift Leg} & : \\
V &= 25.0 \text{ kips} \\
M &= 1045.0 \text{ kip-ft} \\
U &= 125.0 \text{ kips}
\end{align*}
\]

\[
\begin{align*}
\text{Compression Leg} & : \\
V &= 20.0 \text{ kips} \\
M &= 867.0 \text{ kip-ft} \\
C &= 50.0 \text{ kips}
\end{align*}
\]

- Total Unit Weight = 119.0 pcf
- Deformation Modulus = 0.33 ksi
- Friction Angle = 0 deg
- Undrained Shear Strength = 0.8 ksf

- Total Unit Weight = 140.0 pcf
- Deformation Modulus = 360.0 ksi
- Friction Angle = 32 deg
- Rock Cohesion = 2.4 ksf
- Rock / Concrete Bond Strength = 2.0 ksf

Depth to Ground Water = 15 ft
Foundations for H-Frames

HFAD Class Problem 2 – Direct Embedded Pole with Granular Backfill

- Case: Soft to medium stiff soil
- Description: Direct embedded pole in cohesive soil
- Foundation Data
  - Name: 6 ft direct embedded pole
  - Shaft diameter = 6.0 ft
  - Initial depth of embedment = 12.0 ft
- Geotechnical Parameters
  - Name: Soft to medium stiff soil
- Applied Loads
  - Name: DS Loads for HFAD
  - Load Name: DS Loads
- Annulus Backfill Properties - Soil
  - Annulus Thickness = 0.5 ft
  - Total Unit Weight = 130.0 pcf
  - Deformation Modulus = 3.0 ksi
  - Friction Angle = 40.0 deg
  - Undrained Shear Strength = 0 ksf

Uplift Leg
- \( V = 25.0 \text{ kips} \)
- \( M = 1045.0 \text{ kip-ft} \)
- \( U = 125.0 \text{ kips} \)

Compression Leg
- \( V = 20.0 \text{ kips} \)
- \( M = 867.0 \text{ kip-ft} \)
- \( C = 50.0 \text{ kips} \)

Total Unit Weight = 119.0 pcf
Deformation Modulus = 0.33 ksi
Friction Angle = 0 deg
Undrained Shear Strength = 0.6 ksf

Ground Water Level at Surface

50 ft

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Foundations for H-Frames

HFAD Class Problem 3 – Direct Embedded Pole with Concrete Backfill

- **Case:** Soft to medium stiff soil with concrete
- **Description:** Direct embedded pole in cohesive soil with concrete backfill
- **Foundation Data**
  - Name: 6 ft direct embedded pole
  - Shaft diameter = 6.0 ft
  - Initial depth of embedment = 12.0 ft
- **Geotechnical Parameters**
  - Name: Soft to medium stiff soil
- **Applied Loads**
  - Name: DS Loads for HFAD
  - Load Name: DS Loads
- **Annulus Backfill Properties - Concrete**
  - Annulus Thickness = 0.5 ft
  - Total Unit Weight = 140.0 pcf
  - Concrete Strength = 3000 psi

Uplift Leg
- $V = 25.0$ kips
- $M = 1045.0$ kip-ft
- $U = 125.0$ kips

Compression Leg
- $V = 20.0$ kips
- $M = 867.0$ kip-ft
- $C = 50.0$ kips

Total Unit Weight = 119.0 pcf
Deformation Modulus = 0.33 ksi
Friction Angle = 0 deg
Undrained Shear Strength = 0.8 ksf

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Foundations for H-Frames

HFAD Class Problem 4 – Direct Embedded Pole with Granular Backfill

- Case: Soft to medium stiff soil on rock
- Description: Direct embedded pole in cohesive soil on rock
- Foundation Data
  - Name: 6 ft direct embedded pole
  - Shaft diameter = 6.0 ft
  - Initial depth of embedment = 12.0 ft
- Geotechnical Parameters
  - Name: Soft to medium stiff soil on rock
- Applied Loads
  - Name: DS Loads for HFAD
  - Load Name: DS Loads
- Annulus Backfill Properties - Soil
  - Annulus Thickness = 0.5 ft
  - Total Unit Weight = 130.0 pcf
  - Deformation Modulus = 3.0 ksi
  - Friction Angle = 40.0 deg
  - Undrained Shear Strength = 0 ksf

Uplift Leg
- U = 125.0 kips
- V = 25.0 kips
- M = 1045.0 kip-ft

Compression Leg
- C = 50.0 kips
- V = 20.0 kips
- M = 867.0 kip-ft

Total Unit Weight = 119.0 pcf
Deformation Modulus = 0.33 ksi
Friction Angle = 0 deg
Undrained Shear Strength = 0.6 ksf

Total Unit Weight = 140.0 pcf
Deformation Modulus = 360.0 ksi
Friction Angle = 32 deg
Rock Cohesion = 2.4 ksf
Rock / Concrete Bond Strength = 2.0 ksf

Depth to Ground Water = 10 ft
Foundations for H-Frames

HFAD Class Problem 5 – Direct Embedded Pole with Concrete Backfill

- Case: Soft to medium stiff soil on rock with concrete
- Description: Direct embedded pole in cohesive soil on rock with concrete backfill
- Foundation Data
  - Name: 6 ft direct embedded pole
  - Shaft diameter = 6.0 ft
  - Initial depth of embedment = 12.0 ft
- Geotechnical Parameters
  - Name: Soft to medium stiff soil on rock
- Applied Loads
  - Name: DS Loads for HFAD
  - Load Name: DS Loads
- Annulus Backfill Properties - Concrete
  - Annulus Thickness = 0.5 ft
  - Total Unit Weight = 140.0 pcf
  - Concrete Strength = 3000 psi

![Diagram of Direct Embedded Pole with Concrete Backfill]