Foundation Engineering

Course Syllabus and Schedule | Department of Damas and Water Resources Eng.
Spring 2020 | University of Anbar

Course Description: Foundation engineering combines the study of soil behavior (the material you learned in soil mechanics) with topics from engineering mechanics and structures (structural analysis, concrete, and steel design) in order to design all manner of geotechnical structures. It may be one of the most rigorous courses you will take as part of your Civil Engineering program. In general, we discuss practical concepts of soil behavior, develop mechanistic methods of analysis, and apply our knowledge of soil properties and basic mechanics to the design of earth structures and foundations.

General Information:

Instructor: **Nabeel Shaker Mahmood**
PhD Civil Engineering, Consultant Geotechnical Engineer
email: nabeel.sm@uoanbar.edu.iq
office: DWE 101
office hours: Tuesday 11AM-12PM, Thursday 9:45 AM-11:45 AM.

Reference Texts: Foundation Design – Principles and Practice, Third Edition, by Donald P. Coduto, 2014, Pearson Education, Inc.

Course Objectives:

When you complete this course you will be able to:

a. Material behavior and site characterization:
   Plan a subsurface Exploration
   Select drilling, sampling and field property measurement tools
   Specify necessary laboratory tests
   Interpret field and laboratory data to get design properties
b. Design and Analysis of Shallow Foundations:
   Idealize a soil profile for analysis and design
   Use Bearing Capacity Equations correctly
   Determination of Correction Factors
   Evaluate the effects of water and layered soil systems on foundation performance
   Predict foundation settlement (consolidation, elastic)
c. Design and Analysis of Retaining Structures:
   Select proper earth pressure calculation method
   Calculate earth pressures for layered systems
   Evaluate the effects of water and drainage provisions
   Determine internal stability requirements of MSE walls
d. Design and Analysis of Deep Foundations
   Identify major deep foundation types
   Calculate side and tip capacity of driven piles in clay (alpha)
   Calculate side and tip capacity of driven piles in sand (beta)
   Specify pile material types for various applications
   Evaluate pile capacity in the field
e. Introduction to Slope Stability Analysis:
   Identify analysis type to be used for different slope and soils conditions
   Predict stability in homogeneous sand using infinite slope procedures
   Predict stability in homogeneous clays using circular arc charts

Predict stability in all homogeneous soil types using log spiral charts

ABET Outcomes:

a. Math and science principles are applied daily in the design and analysis process.
b. Field and laboratory data are analyzed and interpreted to obtain design properties.
c. Major geotechnical structures are designed from a geotechnical perspective.
d. Semester-long interaction amongst students is encouraged on homework and design submittals.
e. Design submittals are open-ended requiring student formulation of the problem solving process.
f. Public safety in design is emphasized for every major structure type considered.
g. Two design submittals (engineer to client) are required.
h. Not addressed
i. Through external research required for design and creation of design tools.
j. Contemporary geotechnical projects either in design or under construction are discussed along with their impact on society.
k. Required use of spreadsheets, mathematical assistants and CADD along with using current state of practice design concepts.

Participation:

Come to class on time. Class attendance is in accordance with the published university course schedule. Attendance to class is required and comprises a portion of your final grade. Quizzes will be periodically administered to keep a record of your participation and preparedness. Any absence should be coordinated before the absence, if possible.

Homework Assignments:

Daily homework will be due at the beginning of the next class after it is assigned unless otherwise noted in class. All homework assignments should be turned in before class begins. Work turned in late will be penalized in increments of 10% per day. Work will not be accepted beyond two days late without special coordination affected prior to the due date.

Course Grading: The final grade will be assigned based on this distribution:

Homework and quizzes	20%
Exam 1	10%
Exam 2	10%
Final Exam	60%

Academic Integrity:

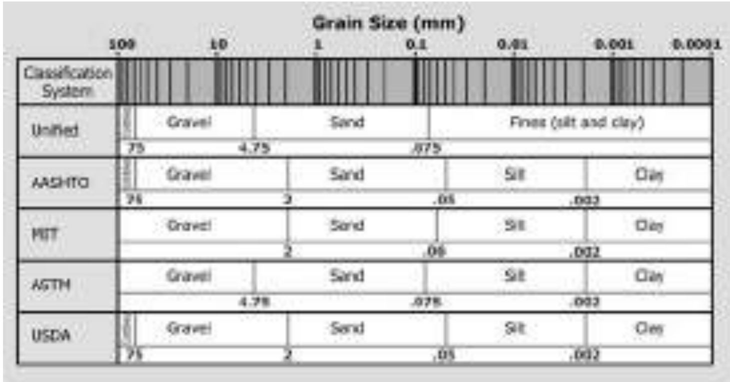
The engineering profession does not need, and should not tolerate, dishonesty. Students are required to be familiar with and abide by the University's Academic Integrity Policy. Students may consult with each other about homework assignments. However, each student is responsible for understanding the principles behind the correct homework solution (not just the correct answer). Cheating (e.g. copying other students work) on homework assignments will NOT be tolerated. Violations will be dealt with immediately and severely in accordance with the university policies.

Disability:

Students in this course with disability requiring an accommodation should contact the professor as soon as possible or contact the head of the department.

Class 1: Review of Soil Mechanics

1- Particle size



2- Index Properties Atterberg limits (LL, PL, PI) why important (Classification, correlations with swelling, shear strength…)

3- Unite weight (wtere = 62.4 psf or 9.81 kPa, 1 gm /cm3, 1000 kg/m3)

4- Specific Gravity (Gs) Gs water =1

5- Dr = emax-e/ emax-emin

6- Stress conditions in soil:
   a. Total stress
   b. Effective stress

7- Shear strength:
   a. what is the shear strength: resistance to sliding along a failure surface
   b. the two component of shear strength (cohesion, friction)
   c. T= c+s'n tan f (Mohr-coulomb failure criterion)
   d. Tests:
      i. Sand:
         1. Direct shear test
         2. Triaxial test
      ii. Clay:
         1. Unconfined compression
         2. Triaxial test
            a. UU test
            b. CU test
            c. CD test

   e. Type of analysis:

i. Total stress analysis
ii. Effective stress analysis
iii. Which one is more critical

8- Settlement:
   a. Elastic settlement
   b. Consolidation settlement (curve)
   c. Secondary Settlement

9- Psi= 6.8 kPa
   Psf= 0.047 kPa (kPa= 20.8 psf)

HW: Read through chapter 3 (quiz next class)
Solve Problems 5, 6, and 11 (Coduto)

Site Exploration

Purpose:

1- Determine properties, location and thickness of strata.
2- Location of bed rock.
3- Location of GWT.
4- Problems and concerns.

The information obtained from the site exploration program will be utilized to:

- Evaluate whether the site is suitable for project construction or not.
- Design appropriate foundations for the project.

Phases:

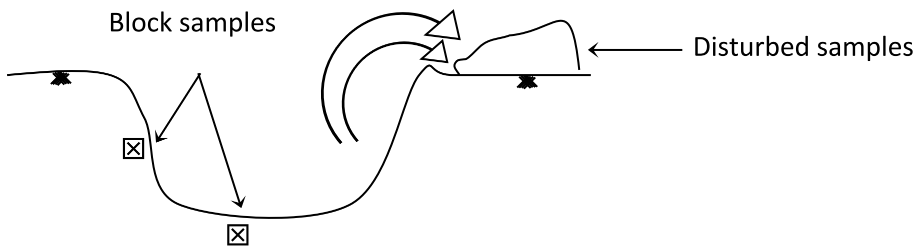
1- Reconnaissance Investigation
   - Search for all existing information (previous site exploration reports, topographic maps, geologic maps, soil surveys, aerial photographs)
   - Field reconnaissance to inspect:
     - Accessibility of the site
     - Exposed soil and rock
     - Ground water flow
     - Conditions of nearby structures
   - A limited number of borings or test pits may be required if the obtained information is not sufficient.
   - For preselected sites, this phase is limited in scope. However, it is important when several proposed sites are under consideration for major projects such as dams and highways.

2- Exploration for Preliminary Design:
- This is the main phase of the exploration program and is typically accomplished with borings or test pits.
- The soil and rock strata that will be affected by the project must be investigated (depth, thickness, properties).
- Soil and rock used as construction materials must be investigated (quantity, quality).

3- Explorations for Detailed Design:
- Additional explorations may be required to provide the designer with adequate information prior or during construction.
- Critical parts of structures (such as spillways, tunnels and shear walls) may require additional borings so that the subsurface conditions are well defined.

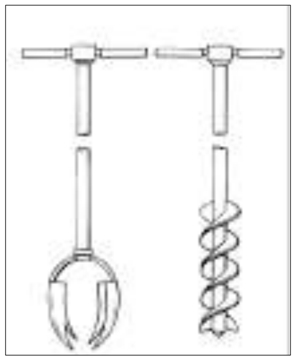
Methods of Site Investigation:
1- Test Pits and Tunnels.
2- Exploratory borings.
3- Geophysical Methods.

1- Test Pits and Tunnels:
- Suitable for all types of soil above GWT
- To obtain disturbed and undisturbed samples.



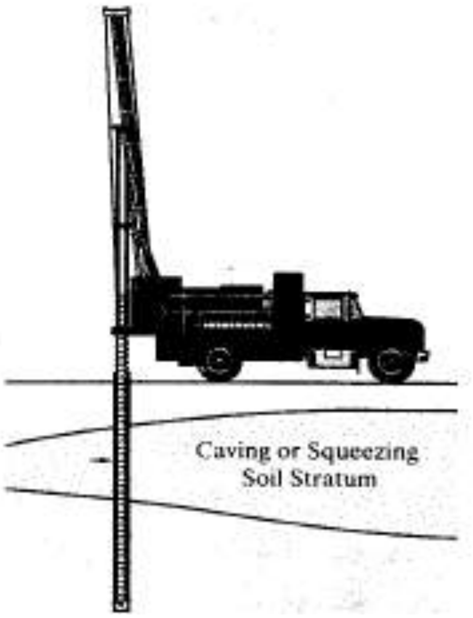
2- Exploratory borings:
  -Typically 75 to 600 mm (3-24 in) in diameter and 3 to 30 m (10-100 ft) deep.
  - There are a wide variety of boring equipment and techniques.

1- Auger Drilling:
  -Hand auger:
    - For shallow foundations and small structures.
    - Depth (3-5 m)
    - Not suitable for sandy and silty soils below GWT.

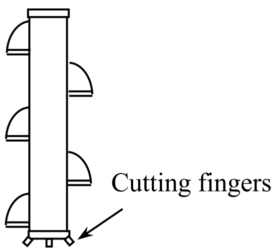


Hand augers (Das, 2011)

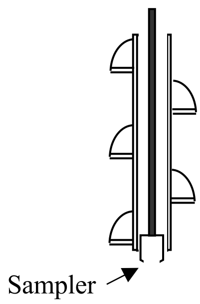
- Power driven auger (two types):
  a- Continuous flight auger
  b- Hollow stem auger



Power driven auger (Coduto, 2013)

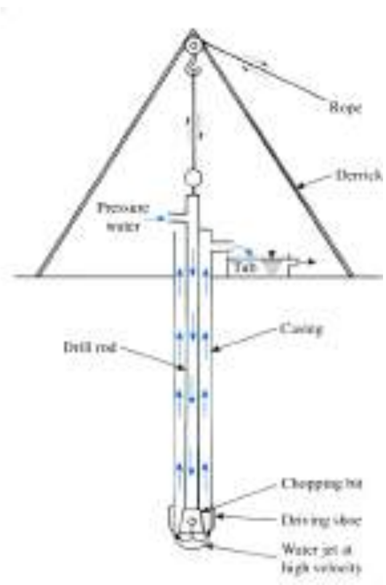


Continuous flight auger

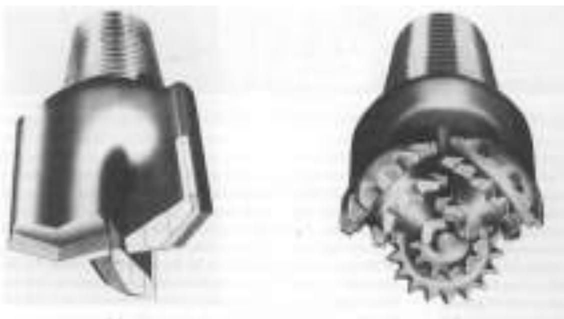


Hollow stem auger

3- Wash Boring Drilling:
- To drill through sand, and soft clay and silt.
- Is rarely used now.



4- Rotary Drilling:
- To drill through hard soil and rock by rotating drilling bits attached to the drilling rod.
- Water or air flow through the drilling rod forces the cutting to the surface.
- Samples (cores) can be taken by replacing the drilling bit by a coring bit.



a

b

Rotary drilling bits a- Drilling bits b- Coring bit (Fang, 1991)

* Observation of water Tables:
- Water encountered in a borehole should be recorded (Depending on the permeability of the soil, water table level may take 24 hrs to several weeks to stabilize).
- Piezometers may be used to obtain accurate and continues measurements of water level and pore water pressure.
- Water samples may be required to conduct some chemical tests.

Soil Sampling:

*** Most common samplers:**

- Split spoon sampler (mainly for cohesionless soils)
- Thin-wall (Shelby) Tube:
  - To obtain undisturbed samples from soft and medium cohesive soils
  - 2 to 6 inch in diameter, and 24 to 30 in in length.

*** Types of Samples:**

1- Disturbed Samples >> for index and physical tests.

2- Undisturbed Samples>> for strength and permeability tests.

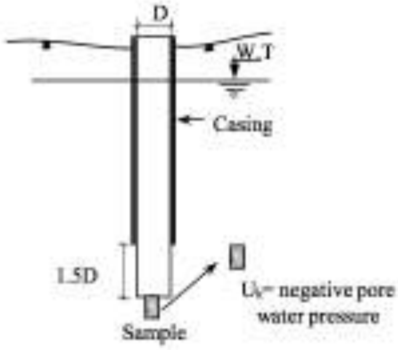
*** Types of Disturbance:**

1- Drilling disturbance:

a- Stress relief.

b- Piping

c- Casing



2- Sampling Disturbance:

a- Compression and shearing the soil during inserting the sampler

$$A_r = \frac{\pi/4D_1^2 - \pi/4D_2^2}{\pi/4D_2^2} = \frac{D_1^2 - D_2^2}{D_2^2} \times 100\%$$

Ar : Area ratio

To give good undisturbed samples:

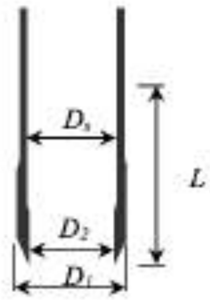
Ar ≤ 20 % for medium and stiff soil

Ar ≤ 10 % for soft and medium soil

b- Soil jamming:

Jamming may occurred when L/D ~ 2.5 for soft soil

L/D ~ 4.5 for stiff soil



$$C_r = \frac{D_s - D_2}{D_2} \times 100\%$$

Cr = Clearance ratio.

To minimize jamming Cr must be between (0.3 % -0.4%)

c- Method of sampler driving:

- Minimum disturbance results from fast and steady driving with a static pressure.
- Maximum disturbance results from hammering.

3- Other causes of disturbance:

- Change in moisture content.
- Chemical changes.
- Mixing and segregation of soil.
- Disturbance during recovery and transport

Spacing of Boreholes:

There are no hard rules to calculate the spacing of boring. The spacing can be increased or decreased depending on structure type and subsurface soil conditions.

Guideline for Spacing of Borings (Das, 2011)

Type of project	Spacing (m)	Spacing (ft)
Multistory building	10–30	30–100
One-story industrial plants	20–60	60–200
Highways	250–500	800–1600
Residential subdivision	250–500	800–1600
Dams and dikes	40–80	130–260

Subsurface soil conditions:

Regular – Good quality- Use upper limit.
Regular – Poor quality- Use lower limit.
Irregular – Poor quality- Reduce lower limit by 30%

Spacing should be selected so that the cost of the exploration program is between 0.1 to 0.5 % of the total cost of the project.

Depth of Boreholes

It depends on:

- Foundation type (shallow or deep foundation)
- Structural load.
- Subsurface soil conditions.
- The depth at which engineering parameters are required for the design.

Guidelines by the American Soicty of Civil Engineers (1972):

• The net increase in stress caused by the new construction $\Delta\sigma$ at the end of boring is about 10% or less of the stress increment at the footing base.
• Boreholes should penetrate all unsuitable soil layers such as highly compressible fills and organic soils.
• Minimum depth of boring into bedrock is 3m (10ft).

Depth of boring suggested by Sowers (1979) from Coduto (2013)

Subsurface Conditions	Minimum Depth of Borings (S = number of stories; D = anticipated depth of foundation) (m)	(ft)
Poor	$6\,S^{0.7}+D$	$20\,S^{0.7}+D$
Average	$5\,S^{0.7}+D$	$15\,S^{0.7}+D$
Good	$3\,S^{0.7}+D$	$10\,S^{0.7}+D$

HW#2: Subsurface Exploration

You have been asked to formulate a boring plan (recommended boring depths, spacing, boring method, sampling method..etc) for a four-story office building and parking lot. The office building has a footprint of 50 m x 70 m and the parking area is 150 x 250 m (Figure 1). Your back ground search and field reconnaissance revealed that the soil profile consists a 2 m loose fill overlying thick clay deposits. Try to do the best job of characterizing the site for the least amount of money. Also, discuss the logic behind your exploration program.

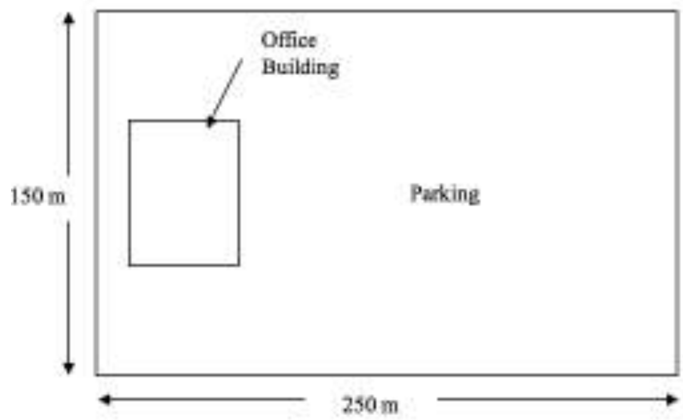


Figure 1. Plan View of Office Building and Parking Facility

3- Geophysical Methods:

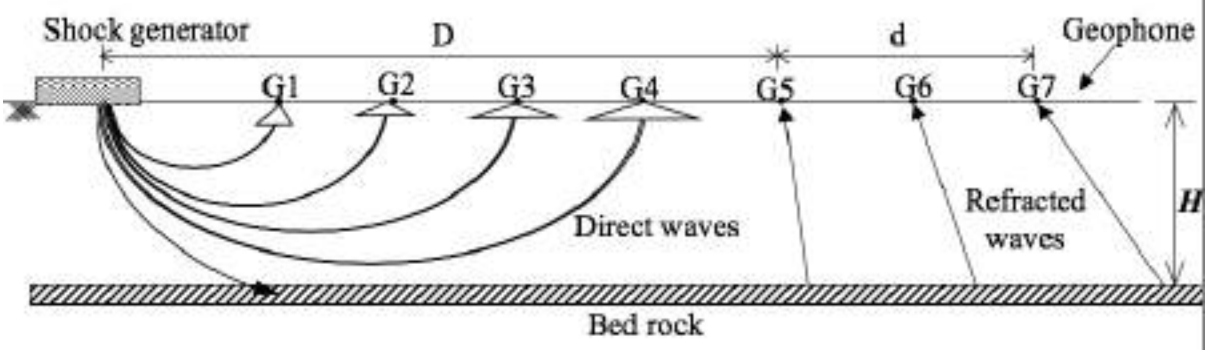
- For rapid and sometimes cost effective evaluation of subsoil conditions (thickness of layers, GWT level, contacts..etc.)
- For major projects and large areas
- Used for preliminary exploration but they are not alternative to the direct methods.

1- Seismic Methods:

Various layers of soil or rock have different wave velocities

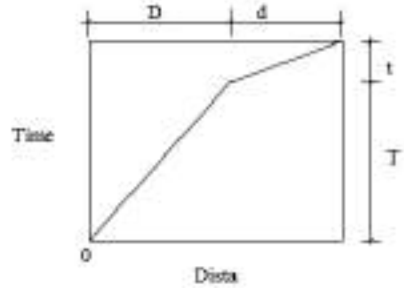
*Refraction Method:

- Impacting the surface (generate P waves and S waves) and recording the arrival time of the direct and the refracted waves.
- For depths up to 300 m.



$$v_1 = \frac{D}{T} \qquad v_2 = \frac{d}{t}$$

$$H = \frac{D}{2}\sqrt{\frac{v_2 - v_1}{v_2 + v_1}}$$



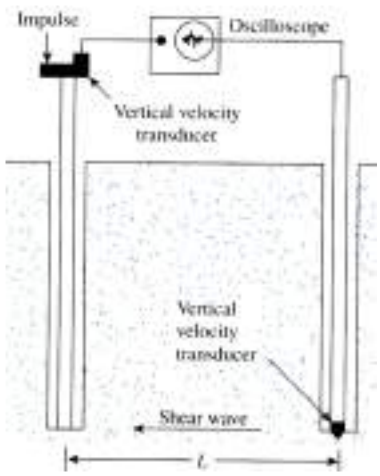
*Cross-Hole Method:

$$vs = \frac{L}{t}$$

$$G = \frac{v_s^2}{\rho}$$

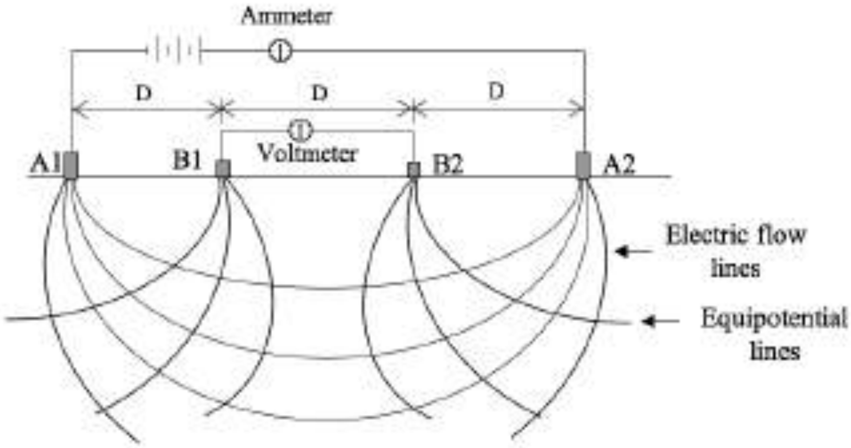
G= Shear modulus of soil

P= Mass density of soil



Cross-hole method (Das 2011)

2- Resistivity Methods:
Resistivity of soils depends on water content and concentration of dissolved ions.
Wenner Method:

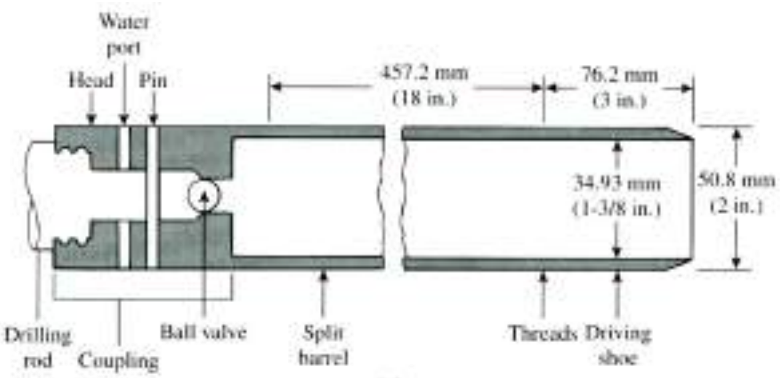


D= 3-10 m
R = 2π D V/I
R= Electric resistivity (ohm/m).
V= Drop in voltage between electrodes B1 & B2.
I= Current imposed on electrodes A1 & A2.

<u>In Situ – Testing</u>

1- Standard Penetration Test (SPT)

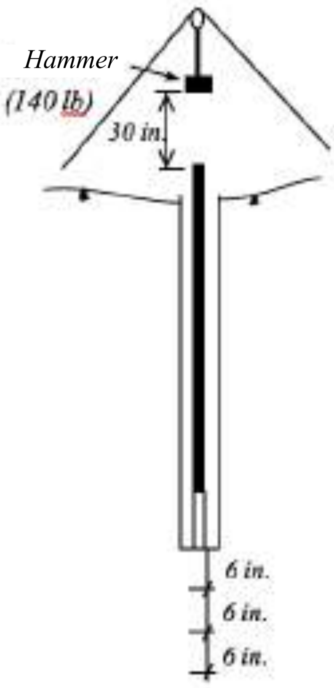
- Split-spoon sampler can be used to obtain disturbed samples from soils without coarse gravel or rock.



Split-Spoon Sampler (Das, 2011)

- The sampler is driven into the bottom of a borehole by a hummer (140 lb) drops a distance of (30 in.) on the top of the drill rode.
- The number of blows required for the sampler penetration of three intervals (6 in.) intervals are recorded.
- N value for the Standard Penetration Test (SPT) is the number of blows required to drive the sampler the last two intervals (12 in.) [ASTM D1586].
- Stop the test if the number of blows at any 6 in interval reaches 50, or if the total blows counts reaches 100.
- Typically SPT is performed at intervals of 1.5 m (5 ft).

- Many factors may affect N values at a given depth such as hammer type and efficiency, borehole diameter, sampler type, and rode length, so it must be corrected.



$$N_{60} = \frac{E_m C_B C_S C_R N}{0.60}$$ Skempton (1985)

Where

N_{60} = standard penetration number, corrected for field conditions
E_m = hammer efficiency
C_B = correction for borehole diameter
C_S = sampler correction
C_R = correction for rod length
N = measured penetration number

From Coduto (2001)

TABLE 4.3 SPT HAMMER EFFICIENCIES (Adapted from Clayton, 1990).

Country	Hammer Type (per Figure 4.10)	Hammer Release Mechanism	Hammer Efficiency E_m
Argentina	Donut	Cathead	0.45
Brazil	Pin weight	Hand dropped	0.72
China	Automatic	Trip	0.60
	Donut	Hand dropped	0.55
	Donut	Cathead	0.50
Colombia	Donut	Cathead	0.50
Japan	Donut	Tombi trigger	0.78–0.85
	Donut	Cathead 2 turns + special release	0.65–0.67
UK	Automatic	Trip	0.73
US	Safety	2 turns on cathead	0.55–0.60
	Donut	2 turns on cathead	0.45
Venezuela	Donut	Cathead	0.43

From Coduto (2001)

TABLE 4.4 BOREHOLE, SAMPLER, AND ROD CORRECTION FACTORS (Adapted from Skempton, 1986).

Factor	Equipment Variables	Value
Borehole diameter factor, C_B	65–115 mm (2.5–4.5 in)	1.00
	150 mm (6 in)	1.05
	200 mm (8 in)	1.15
Sampling method factor, C_S	Standard sampler	1.00
	Sampler without liner (not recommended)	1.20
Rod length factor, C_R	3–4 m (10–13 ft)	0.75
	4–6 m (13–20 ft)	0.85
	6–10 m (20–30 ft)	0.95
	>10 m (>30 ft)	1.00

- In granular soils, N is also affected by over burden stress:

$$N_{1,60} = N_{60}\sqrt{\frac{P_a}{\sigma'_o}}$$

Where:

$N_{1,60}$ = standard penetration number corrected to for overburden stress

N_{60} = standard penetration number, corrected for field conditions

Pa ≈100 kN/m^2 or 2000 psf (atmospheric pressure)

- Uses of SPT data (some examples of the available correlations):

1- Relative Density:

$$Dr(\%) = 122 + 0.75[222N_{60} + 2311 - 711OCR - 779(\frac{\sigma'_o}{Pa}) - 50C_u^2]^{0.5}$$ [Kulhawy & Mayne, 1990]

C_u = uniformity coefficient of sand

OCR = Over consolidation ratio

Pa ≈100 kN/m^2 or 2000 psf (atmospheric pressure)

$$Dr = \sqrt{\frac{N_{1,60}}{C_P C_A C_{OCR}}}$$

$C_P=60+25\log D_{50}$

$C_A=1.2+0.05\ \log(t/100)$

t= age of soil [time since deposition (years)], usually taken as 1000 years

$C_{OCR}=OCR^{0.18}$ (for sand, usually do not use OCR, in this case OCR=1)

Properties for sand based on SPT (Terzaghi and Peck, 1967)

Blow count	Consistency	Relative Density	Friction Angle
0-4	Very loose	0-15	26-30
5-10	Loose	16-35	28-35
10-30	Medium	36-65	35-42
31-50	Dense	66-85	38-46
>50	Very Dense	>85	>42

Properties for clay based on SPT (Terzaghi and Peck, 1967)

Blow count	Consistency	Undrained Cohesion, Cu (psf)
>2	Very soft	<250
2-4	Soft	250-500
5-8	Medium Stiff	500-1000
9-15	Stiff	1000-2000
16-30	Very Stiff	2000-4000
>30	Hard	>4000

2- Angle of Friction:

$$\emptyset' = tan^{-1}\left[\frac{N_{60}}{12.2+20.3\left(\frac{\sigma vo}{Pa}\right)}\right]^{0.34}$$ [Kulhawy & Mayne, 1990]

$$\emptyset' = \sqrt{20(N_1)_{60}} + 20$$ [Hatanka and Uchida, 1996]

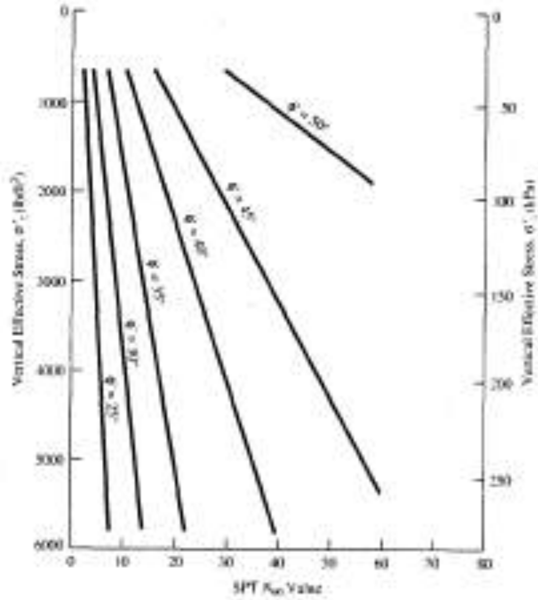
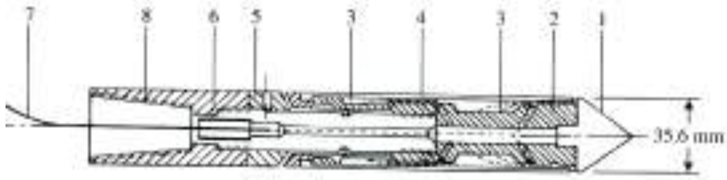


Figure 4.11 Empirical correlation between N_{60} and ϕ' for uncemented sands (Adapted from DeMello, 1971).

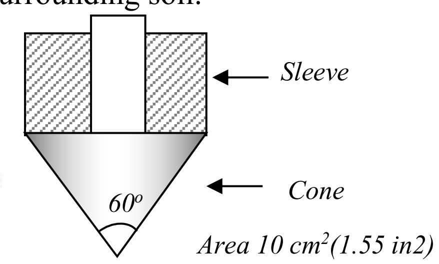
2- Cone Penetration Test (CPT)

- Also known as Static Penetration Test
- No boreholes are necessary, no samples recovered, not suitable for soils with gravel, the cost of the test is high
- Test procedure: The cone is pushed at a steady rate of 20 mm/sec (0.8in./sec) and both cone (or point) resistance (qc) friction resistance (fc) are recorded through transducers connected to the cone and the sleeve. fc is the sum of friction and adhesion developed between the sleeve and the surrounding soil.



1 Conical point (10 cm²)
2 Load cell
3 Strain gauges
4 Friction sleeve (150 cm²)
5 Adjustment ring
6 Waterproof bushing
7 Cable
8 Connection with rods

Electric Cone Penetrometer (Das, 2011)



Advantage vs. Disadvantage

Uses of CPT data (some examples of the available correlations):

$$Dr\ (\%) = \sqrt{\left(\frac{q_c}{312 Q_c OCR^{0.18}}\right)}\sqrt{\frac{P_a}{\sigma'_0}} \times 100\%$$ (Kulhawy and Mayne, 1990)

Qc= Cmpressiblity factor

=0.91 for highly compressible sands

=1.00 for moderately compressible sands

=1.09 for slightly compressible sands

Sand with high fines content or high mica content is "high compressible"

Pure quantize sand is "slightly compressible"

$$\emptyset' = tan^{-1}\left[0.1 + 0.38 log\left(\frac{qc}{\sigma'_0}\right)\right]$$ (Kulhawy and Mayne, 1990)

$$c_u = \frac{q_c - \sigma_0}{N_K}$$ Robertson and Campanella (1983)

N_K= 15 for electric cone

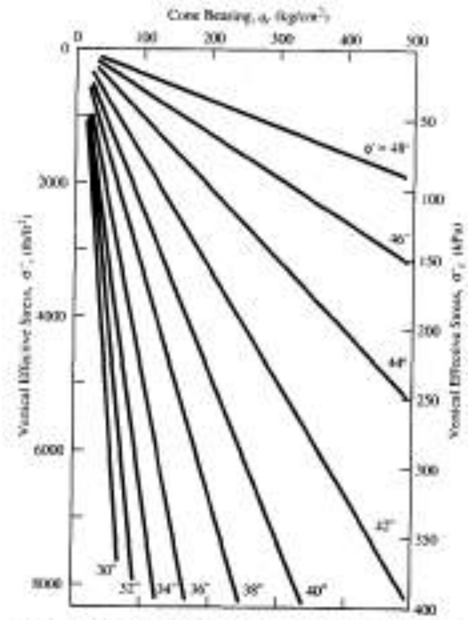


Figure 4.16 Relationship between CPT results, overburden stress and effective friction angle for uncemented, normally consolidated quartz sands (Adapted from Robertson and Campanella, 1983).

From Coduto (2001)

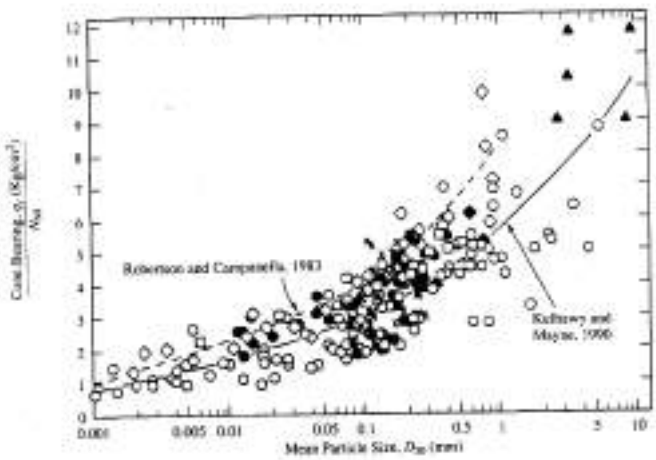


Figure 4.17 Correlation between q_c/N_{60} and the mean grain size, D_{50} (Adapted from Kulhawy and Mayne, 1990.) Copyright © 1990 Electric Power Research Institute, reprinted with permission.

From Coduto (2001)

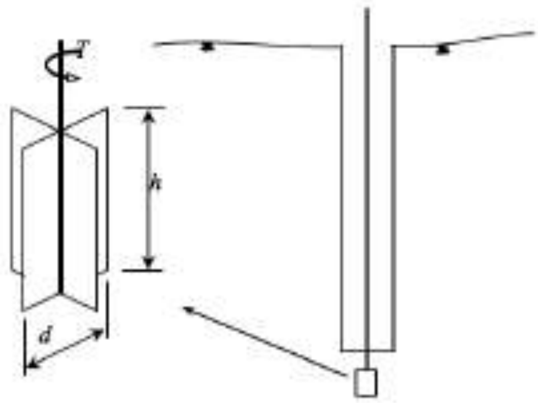
3- Vane Shear Test

- To evaluate undrained shear strength of soft and medium clay with undrained shear strength.
- Driving the vane into the bottom of a borehole and recording the maximum torque (T) required to rotate the vane.
- Cu is too high and it must be reduced for design purposes.

$$c_u = \frac{T}{\pi d^2 (\frac{h}{2} + \frac{d}{2})} = kT$$

$k = vane\ factor$

$$Sensitivity = \frac{c_u}{(c_u) remolded}$$



5- Rock Quality Designation(RQD)

- To evaluate rock quality from the recovered rock cores.

$$RQD = \frac{\sum \text{Length of recovered pieces} \geq 4 \text{ in.}}{\text{length of rock cored}}$$

Relationship between the RQD and the rock qua;ity (Deere, 1963)

RQD	Rock Quality
0–0.25	Very poor
0.25–0.5	Poor
0.5–0.75	Fair
0.75–0.9	Good
0.9–1	Excellent

5- **Other methods (Plate load test, pressuremeter)**

Sample Description (general)

A-Soil:

1- Consistency or density:

Clay →
- very soft
- soft
- firm
- stiff
- very stiff
- hard

Sand →
- very loss
- loss
- medium dense
- dense
- very dense

2- Structure or texture: fissured, laminated….etc.
3- Color : brown, grey,.…etc.
4- Subsidiary constitutes
5- Principle soil type.
6- Additional description: with plant root, with little gypsum …etc.

Examples: 1 2 3 4 5 6
- stiff fissured brown silty CLAY (with little gypsum)

1 3 4 5 6
- dense grey gravelly SAND (with some mica)

B- Rock:

1- Color
2- Hardiness (Brittle, Weak, Medium, Hard)
3- Structure (Laminated, Cohesive….etc)
4- Weathering (Slightly, Moderately, Highly Weathered)
5- Cavities
6- Voids
7- Rock type (Limestone, Sandstone….etc)

Geotechnical Report

A typical geotechnical report may include the following items:

1- Executive Summary
2- Introduction:
   - The scope of the investigation.
   - Description of the proposed project
3- Site Conditions:
   - Location of the site
   - Geology (provide maps, seismic activity)
   - Description of the site and nearby structures and any unique features in the site.
   - Limitations
4- Site Exploration:
   - Number and locations of boreholes on the site plan
   - Equipment used for drilling and sampling
   - Methods of sampling
5- Subsurface Conditions:
   - Description of the surface conditions
   - Description of the water table conditions
6- Evaluation and Engineering Analysis of the Data
   - Select foundation types to provide an adequate factor of safety against bearing capacity failure and maintain settlement within the allowable limits.
   - Evaluation of earth pressure, slope stability, liquefaction …etc.
7- Recommendations:
   - Type of foundations
   - Bearing capacity and settlement at the proposed depths of foundations
   - Site earth work preparation.
   - Using of in-situ soil and rock as construction materials
   - Specific precautions to protect subsurface structures from corrosion
   - Dewatering
8- References
9- Appendices
   a. Laboratory and field tests results
   b. Boring logs
   c. Profiles
   d. Calculation sheets
   e. Maps and photos

UNIVERSAL ENGINEERING SCIENCES
BORING LOG

PROJECT NO.: 1630.1100007.0000
REPORT NO.: 887472

PROJECT: McDonald's #035-6574
North Rhett and Remount Road
North Charleston, South Carolina

BORING DESIGNATION: **B-2** SHEET: **1 of 1**
SECTION: TOWNSHIP: RANGE:

CLIENT: McDonald's USA, LLC
LOCATION: See Boring Location Plan
REMARKS: Automatic Hammer Used

G.S. ELEVATION (ft):
WATER TABLE (ft): 5.41
DATE OF READING: 3/18/2011

DATE STARTED: 3/18/11
DATE FINISHED: 3/18/11
DRILLED BY: Southern Drill, Inc
TYPE OF SAMPLING: SPT

DEPTH (FT.)	SAMPLE	BLOWS PER 6" INCREMENT	N (BLOWS/FT.)	W.T.	SYMBOL	DESCRIPTION	-200 (%)	MC (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	K (FT./DAY)	ORG. CONT. (%)
0						TOPSOIL (6")						
						COASTAL PLAIN - Very Loose Light Brown SAND w/ Trace Clay Seams (SP)						
	X	1-1-1	2									
						Very Loose Light Brown SAND (SP)						
5	X	1=18"	1=18"	▼								
	X	1=18"	1=18"									
						Very Loose Light Reddish Brown SAND (SP)						
10	X	1-0-1	1									
						Loose to Very Loose Light Brown SAND (SP)						
15	X	2-4-4	8									
20	X	1-1-2	3									
						Boring Terminated @ 20'						

DESIGN OF SHALLOW FOUNDATIONS

We need shallow foundations to transmit structural load to soil so that the foundation:

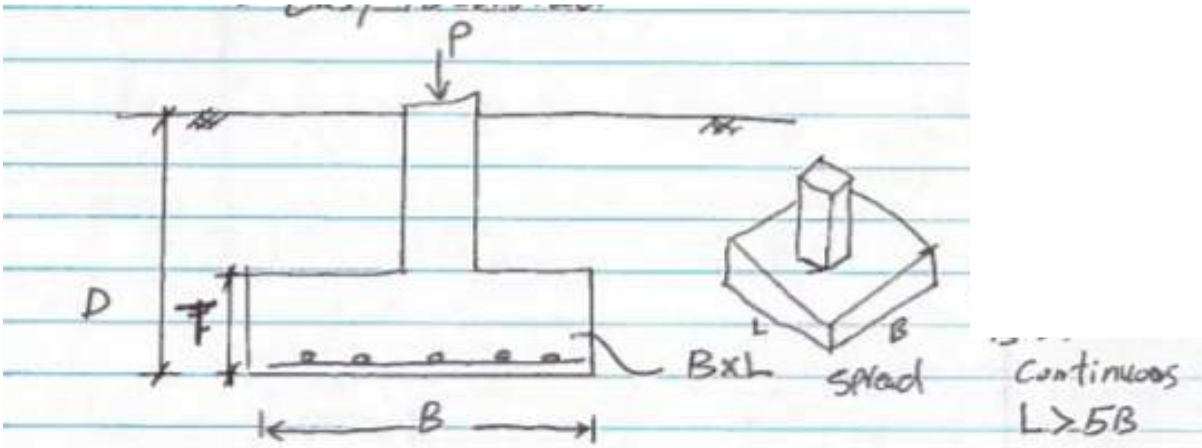
1- Safe against shear failure in soil
2- Cannot undergo excessive settlement

Shallow foundation when D< 4B

Types of shallow foundations:

1- Spread Footing

- **Most common**
- **Low cost**
- **Easy to construct**



* B = width ≥ 16" cont.
  ≥ 3 ft spread

* P = Structure load < Dead load / live load

* t = Thickness, enough to resist shear failure in concrete
  - Punching (two way)
  - one way shear

* # and size of Reinforcement (flexural Reinf.)
  Reinforcement

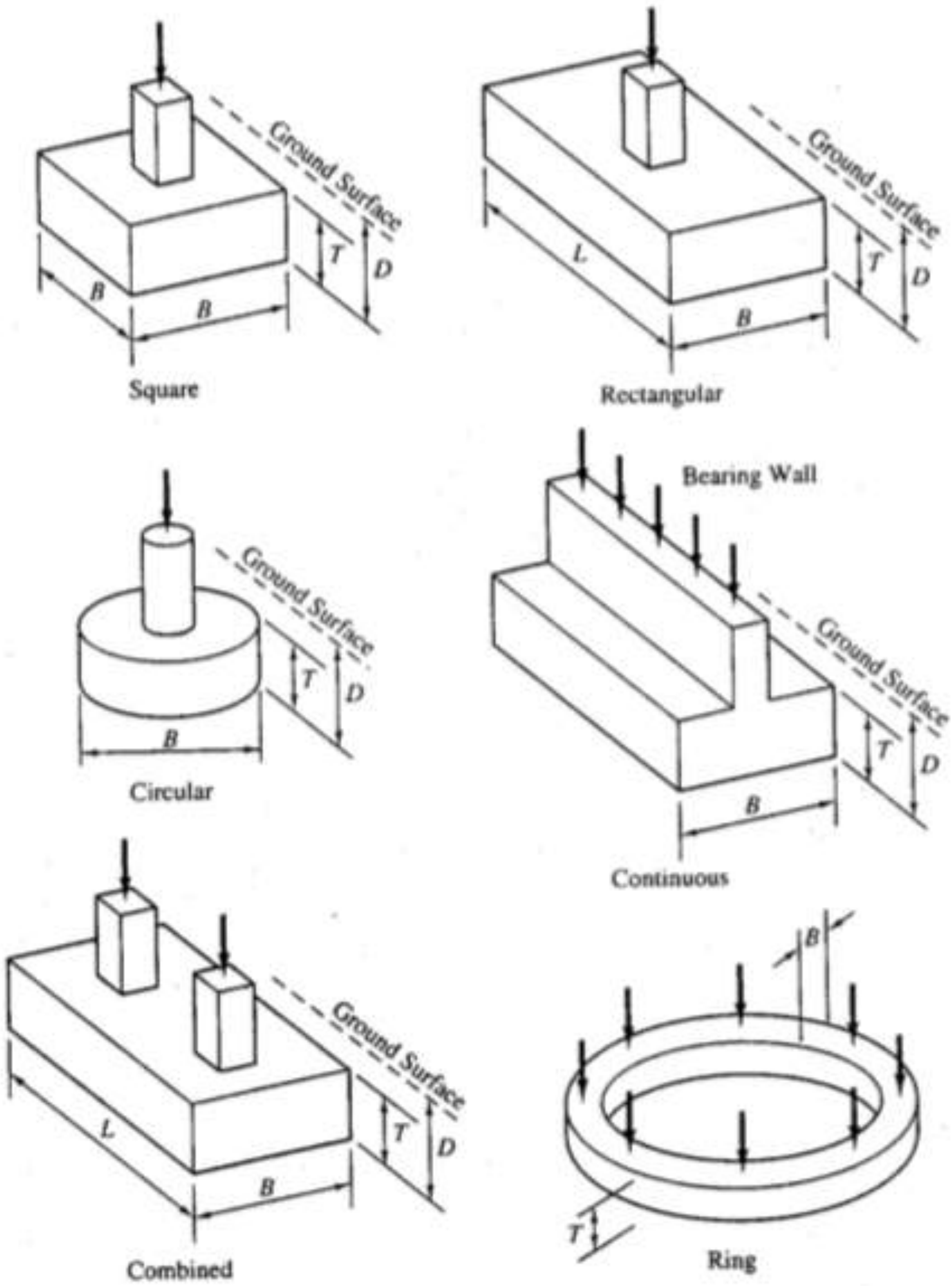


Figure 5.1 Spread footing shapes and dimensions.

2- Strap footings

when close to property line==== reduce differential settlement

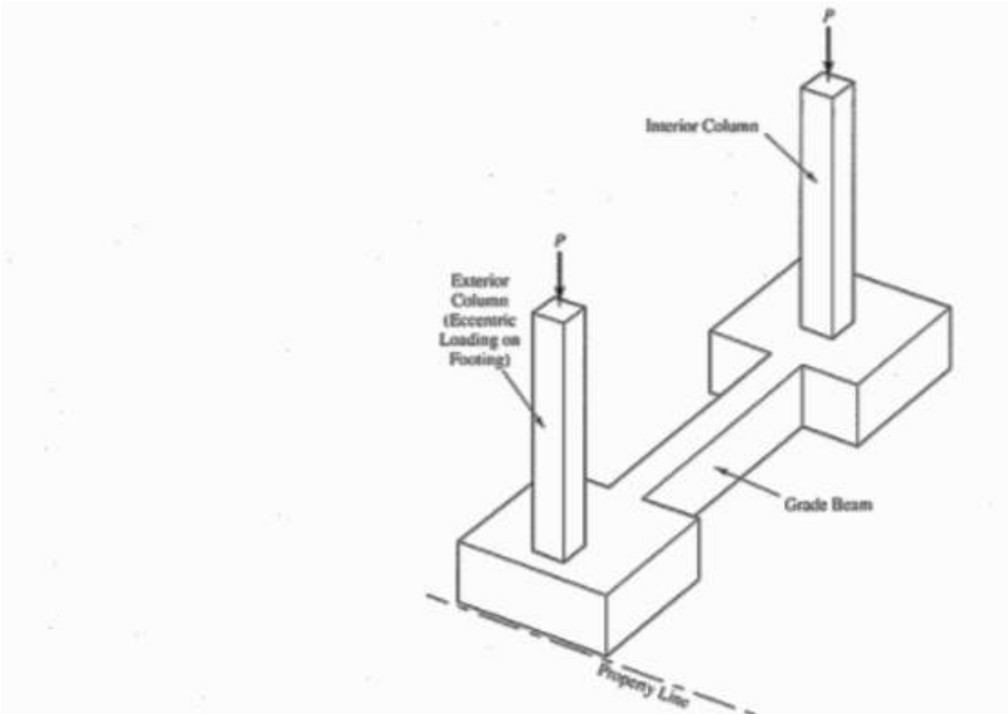


Figure 5.2 Use of a strap footing to support exterior columns when construction cannot extend beyond the property line.

3- Mat (raft) foundation

- If spread footing area > 50 % of the building foot print
- To minimize differential settlement in variable soils

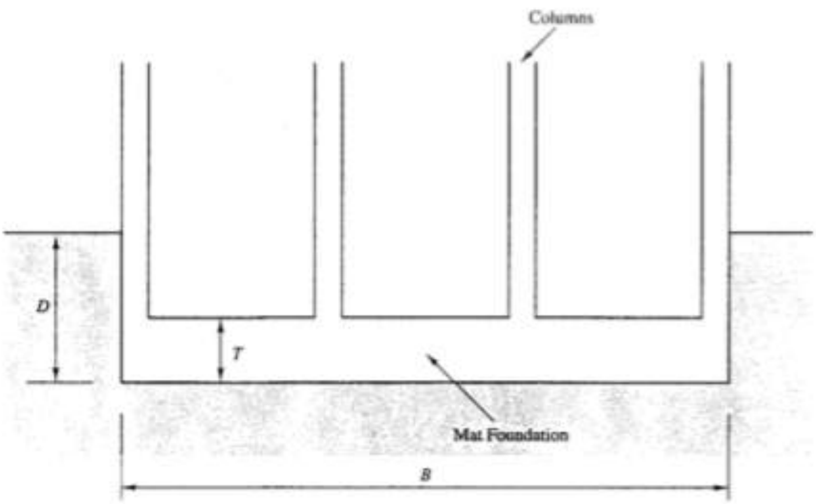


Figure 5.9 A mat foundation.

Footing Depth

Considerations:

1- Below inadequate soil layers

- Organic
- Fill
- Compressible or weak

2- Below frost penetration depth

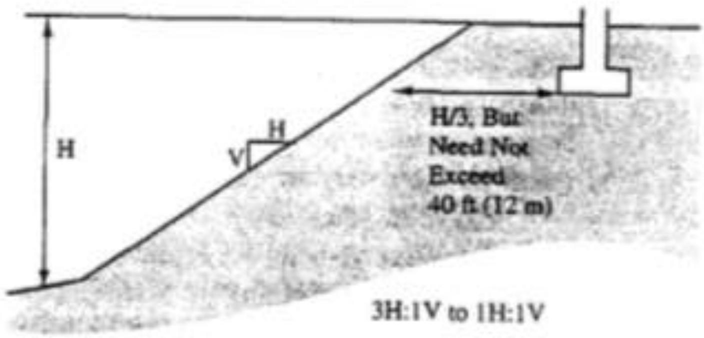
- water expands 9% when it freezes- causes "frost heave"
  - some cold areas 25 to 50 mm, but nonuniform
  - water may forms ice lens
- Depth of frost heave from building cods (Example: Chicago 1.1 m)

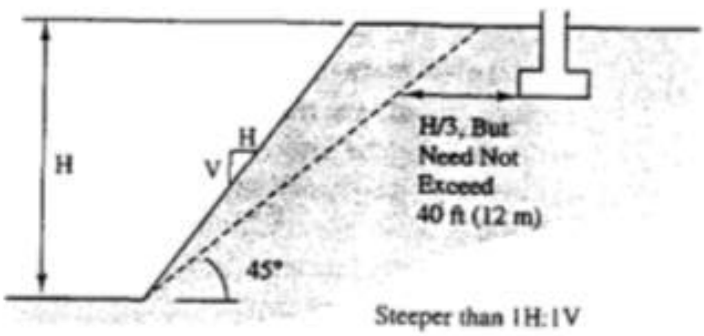
3- Depth of expansive soil:

- **Below water fluctuations**
- **Highly expansive soils === other considerations**

4- footing near slopes

Need to step back criteria





5- Below Scouring Depth

6- Other considerations

- **Avoid working below ground water**
- **Avoid excavation support**
- **Avoid substructures and utilities**

Minimum depth – Tables

TABLE 8.1 MINIMUM DEPTH OF EMBEDMENT FOR SQUARE AND RECTANGULAR FOOTINGS

Load P (k)	Minimum D (in)
0–65	12
65–140	18
140–260	24
260–420	30
420–650	36

Load P (kN)	Minimum D (mm)
0–300	300
300–500	400
500–800	500
800–1100	600
1100–1500	700
1500–2000	800
2000–2700	900
2700–3500	1000

TABLE 8.2 MINIMUM DEPTH OF EMBEDMENT FOR CONTINUOUS FOOTINGS

Load P/b (k/ft)	Minimum D (in)
0–10	12
10–20	18
20–28	24
28–36	30
36–44	36

Load P/b (kN/m)	Minimum D (mm)
0–170	300
170–250	400
250–330	500
330–410	600
410–490	700
490–570	800
570–650	900
650–740	1000

Bearing capacity of shallow foundations

Bearing capacity = shaer strength failure

Occurs when applied pressure > > soil shear strength

1- General shear failure

- Most common
- Failure occurs suddenly
- Formed bulges on the surface
- Ultimate failure occurs on side (rotation) even though bulges may appear on both sides
- Sand and clay loaded rapidly

2- Local shear failure

- Settlement +bulging (sometimes)
- Failure surface gradually extends to the surface
- Medium sand and clay

3- Punching shear failure

- Failure surface will nit extend to the surface
- Large settlement
- Loose sand and soft clay

** Vesic (1970) – from lab testing

- D/B < 2 footing can fail in any mode
- D/B > 4 punching shear

** Rules of thumb:

1- Shallow in rock and undrained ==== general shear failure

2- Dense sand (Dr<67%) ==== General shear failure

3- Loose to medium sand (30<Dr< 67%)==== local shear failure

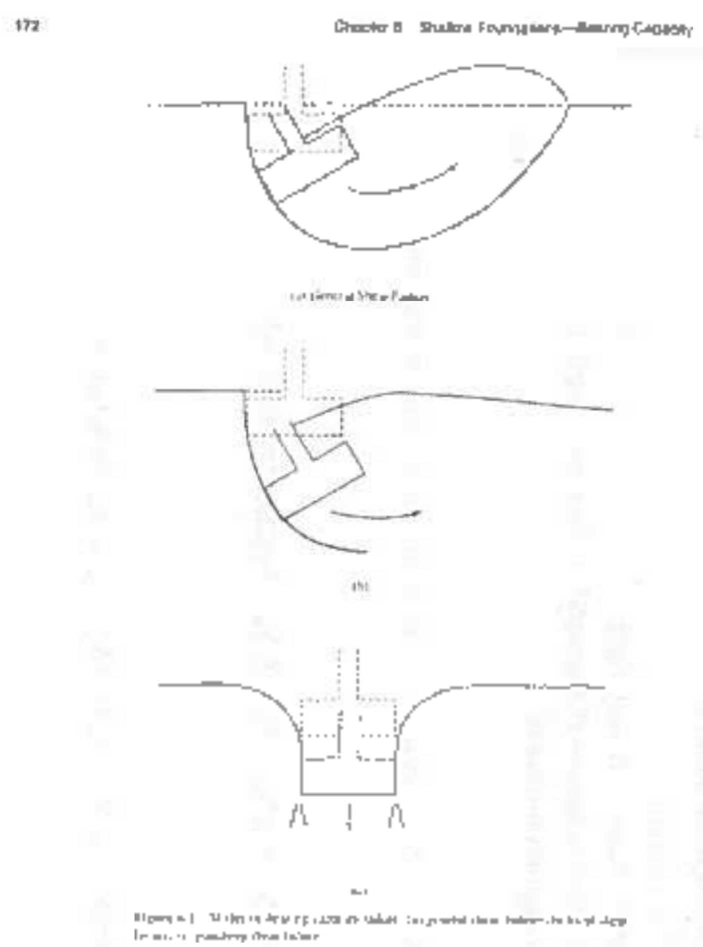
4- Very loose sand (Dr<67%) ====Punching

In practice:

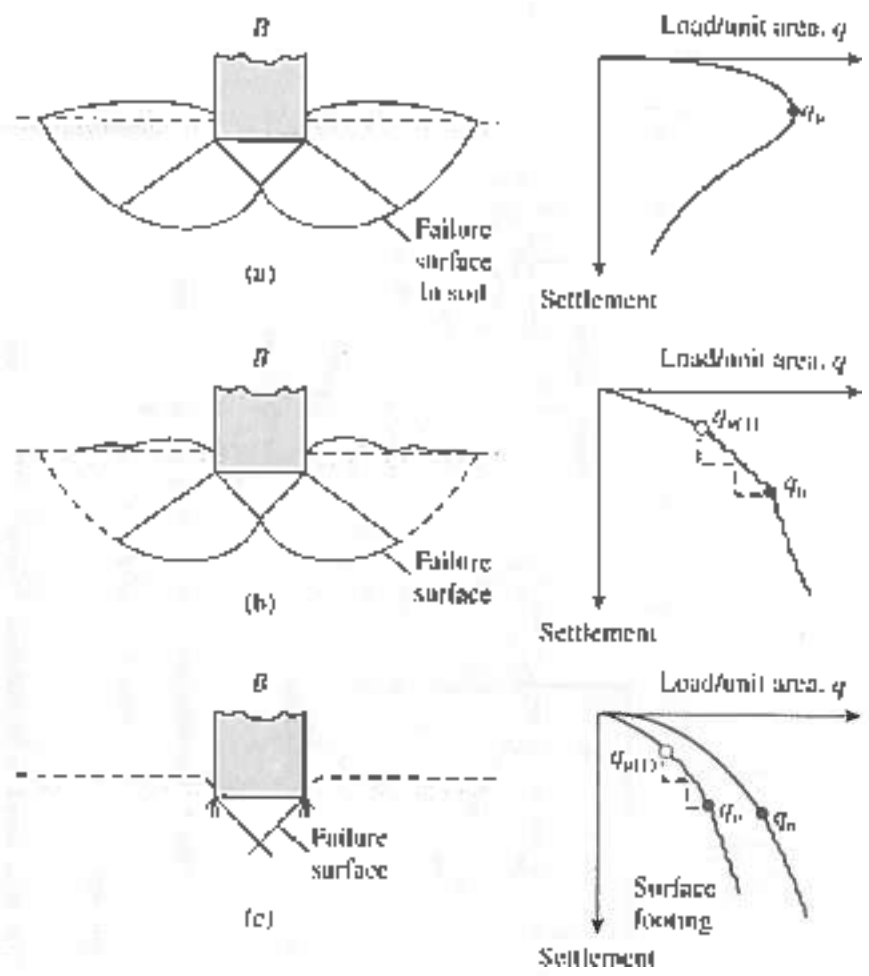
1- Check only general shear failure

2- Check settlement (from settlement analysis) << settlement from local or punching

Exception: Punching shear governs the design if strong layer above very weak layer



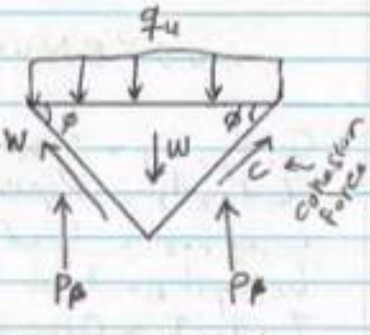
From Coduto (2001)



From Das (2011)

① Terzaghi's Bearing Capacity ~~Equation~~ Theory (General Shear failure):

② - For continuous Foundation $L > 5B$
- Axial loads
-



$\Sigma F_y = 0 = -q_u + 2P_p + 2C\sin\phi - W$

$P_p = f(W_T, \text{Surcharge}, \text{Cohesion})$

Pp = Passive force

$q_u = CN_c + \cancel{\gamma D_f}\,q\, N_q + 0.5\,\gamma B N_\gamma$ (cont.)

$q = \gamma D_f$

$N_c, N_q, N_\gamma = f(\phi)$ = Bearing Capacity factors
Table (7.1) 3rd Ed. (6.1) 2nd ed.

$q_u = 1.3\,CN_c + \cancel{\gamma D_f}\,q\,N_q + 0.4\,B\gamma N_\gamma$ (square)

$q_u = 1.3\,CN_c + \cancel{\gamma D_f}\,q\,N_q + 0.3\,\gamma B N_\gamma$ (circular)

Local Shear failure:

use the same general Shear failure Eq. with reduced value of c' and ϕ

$c'_{adj} = \frac{2}{3}\,c'$

$\phi'_{adj} = \tan^{-1}\left(\frac{2}{3}\tan\phi'\right)$

c_{adj}, ϕ_{adj} = adjusted c' and ϕ

3

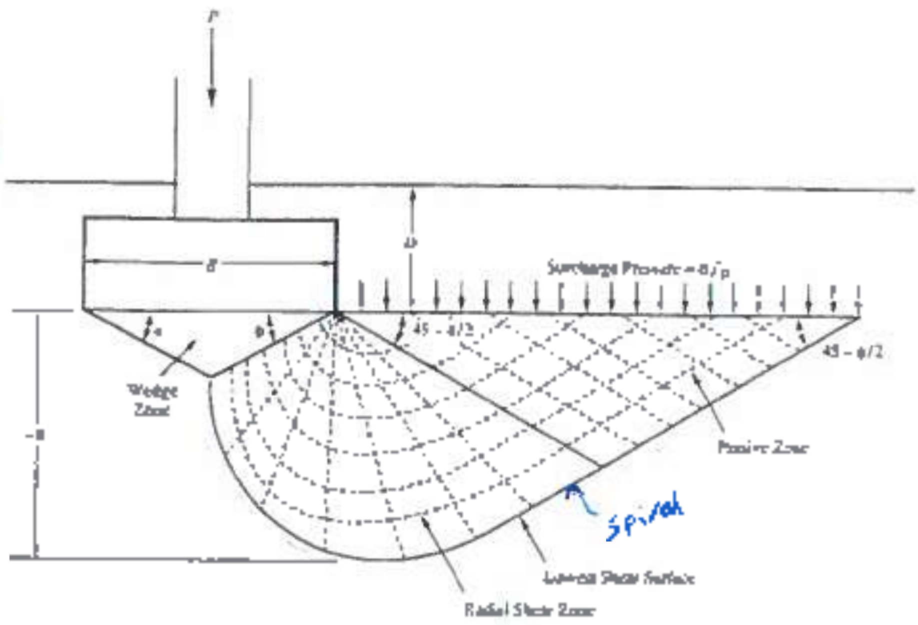


Figure 6.5 Geometry of failure surface for Terzaghi's bearing capacity formula

From Coduto (2001)

4

Terzaghi assumptions:

- D<=B
- No sliding between the foundation bottom and the soil
- The soil is homogenous and semi-infinite
- General shear failure
- Shear= c+tanφ'
- The foundation is very rigid
- No shear between the ground surface and depth D
- No applied moment

5 $q_u = c'N_c + qN_q + \frac{1}{2}\gamma BN_\gamma$ (continuous or strip foundation)

$q_u = 1.3c'N_c + qN_q + 0.4\gamma BN_\gamma$ (square foundation)

$q_u = 1.3c'N_c + qN_q + 0.3\gamma BN_\gamma$ (circular foundation)

TABLE 6.1 BEARING CAPACITY FACTORS

ϕ' (deg)	Terzaghi (for use in Equations 6.4–6.6)			Vesić (for use in Equation 6.13)		
	N_c	N_q	N_γ	N_c	N_q	N_γ
0	5.7	1.0	0.0	5.1	1.0	0.0
1	6.0	1.1	0.1	5.4	1.1	0.1
2	6.3	1.2	0.1	5.6	1.2	0.2
3	6.6	1.3	0.2	5.9	1.3	0.2
4	7.0	1.5	0.3	6.2	1.4	0.3
5	7.3	1.6	0.4	6.5	1.6	0.4
6	7.7	1.8	0.5	6.8	1.7	0.6
7	8.2	2.0	0.6	7.2	1.9	0.7
8	8.6	2.2	0.7	7.5	2.1	0.9
9	9.1	2.4	0.9	7.9	2.3	1.0
10	9.6	2.7	1.0	8.3	2.5	1.2
11	10.2	3.0	1.2	8.8	2.7	1.4
12	10.8	3.3	1.4	9.3	3.0	1.7
13	11.4	3.6	1.6	9.8	3.3	2.0
14	12.1	4.0	1.9	10.4	3.6	2.3
15	12.9	4.4	2.2	11.0	3.9	2.6
16	13.7	4.9	2.5	11.6	4.3	3.1
17	14.6	5.5	2.9	12.3	4.8	3.5
18	15.5	6.0	3.3	13.1	5.3	4.1
19	16.6	6.7	3.8	13.9	5.8	4.7
20	17.7	7.4	4.4	14.8	6.4	5.4
21	18.9	8.3	5.1	15.8	7.1	6.2
22	20.3	9.2	5.9	16.9	7.8	7.1
23	21.7	10.2	6.8	18.0	8.7	8.2
24	23.4	11.4	7.9	19.3	9.6	9.4
25	25.1	12.7	9.2	20.7	10.7	10.9
26	27.1	14.2	10.7	22.3	11.9	12.5
27	29.2	15.9	12.5	23.9	13.2	14.5
28	31.6	17.8	14.6	25.8	14.7	16.7
29	34.2	20.0	17.1	27.9	16.4	19.3
30	37.2	22.5	20.1	30.1	18.4	22.4
31	40.4	25.3	23.7	32.7	20.6	26.0
32	44.0	28.5	28.0	35.5	23.2	30.2
33	48.1	32.2	33.3	38.6	26.1	35.2
34	52.6	36.5	39.6	42.2	29.4	41.1
35	57.8	41.4	47.3	46.1	33.3	48.0
36	63.5	47.2	56.7	50.6	37.8	56.3
37	70.1	53.8	68.1	55.6	42.9	66.2
38	77.5	61.5	82.3	61.4	48.9	78.0
39	86.0	70.6	99.8	67.9	56.0	92.2
40	95.7	81.3	121.5	75.3	64.2	109.4
41	106.8	93.8	148.5	83.9	73.9	130.2

From Coduto (2001)

6

Vesic's Bearing Capacity Equation (General Eq.)

$$q_u = c N_c s_c d_c i_c b_c g_c + \cancel{\gamma D_f} q N_q s_q d_q i_q b_q g_q$$

$$+ 0.5 \gamma B N_\gamma s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma$$

s_c, s_q, s_γ = Shape factors.
d_c, d_q, d_γ = depth factors.
i_c, i_q, i_γ = Load inclination factors.
b_c, b_q, b_γ = base inclination factors.
g_c, g_q, g_γ = Ground inclination factors.

Simplified Bearing Capacity Eq for Saturated Clay ($\phi_u = 0$) [Meyerhof, 1951]

$$q_u = c N_c S_c + q N_q S_q + 0.5 \gamma B N_\gamma S_\gamma$$

Neglect depth factors

$N_c = 5.14$, $N_q = 1$, $N_\gamma = 0$

$S_c = 1 + \left(\frac{B}{L}\right) \frac{N_q}{N_c} = 1 + \left(\frac{B}{L}\right) \frac{1}{5.14} = 1 + 0.2 \frac{B}{L}$

$S_q = 1 + \frac{B}{L} \tan\phi = 1$

$q_u = \left(1 + 0.2 \frac{B}{L}\right) 5.14\, C_u + q$ Rect

$q_u = 5.14\, C_u + q$ Strip

Des lee

Base Inclination

$$b_c = 1 - \frac{\alpha}{147°}$$

$$b_q = b_\gamma = \left(1 - \frac{\alpha \tan \phi'}{57°}\right)^2$$

Ground Inclination

$$g_c = 1 - \frac{\beta}{147°}$$

$$g_q = g_\gamma = [1 - \tan \beta]^2$$

Vesic's Bearing Capacity Equation:

$$q_{ult} = c'N_c s_c d_c i_c b_c g_c + \sigma'_{zD} N_q s_q d_q i_q b_q g_q + 0.5\gamma' B N_\gamma s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma$$

Shape factors:

1   = 0

From Coduto (2001)

$$s_c = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$$

for c=t. factor $S_c = 1$

$B/L = 0$

$$s_q = 1 + \left(\frac{B}{L}\right)\tan\phi'$$

and if $\phi = 0$ $S_q = 1$

$\phi a = 0 \Rightarrow \phi q = 0$

$$s_\gamma = 1 - 0.4\left(\frac{B}{L}\right)$$

Depth Factors:

$$d_c = 1 + 0.4\,k$$

$$d_q = 1 + 2k\tan\phi'(1 - \sin\phi')^2$$

$$d_\gamma = 1$$

k= D/B if (D/B<=1)
k= $\tan^{-1}$ (D/B) if (D/B >1)
$\tan^{-1}$ (D/B) in radians

Load Inclination Factors:

$$i_c = 1 - \frac{mV}{Ac'N_c} \geq 0$$

B x L

$$i_q = \left[1 - \frac{V}{P + \frac{Ac'}{\tan\phi'}}\right]^m \geq 0$$

$$i_\gamma = \left[1 - \frac{V}{P + \frac{Ac'}{\tan\phi'}}\right]^{m+1} \geq 0$$

For loads inclined in the B direction:

$$m = \frac{2 + B/L}{1 + B/L}$$

For loads inclined in the L direction:

$$m = \frac{2 + L/B}{1 + L/B}$$

Factor of safety

Allowable stress design (ASD)

$q_a = q_u/F$

q_a = Allowable bearing Capacity @ the foundation level (gross)
(↳ Struct load + W_f)
q_u = ultimate B.C
F = factor of Safety
↳ depends on: - soil Type
- site Data
- structure Type

P = P always net unless it is mentioned that it is P_{gross}

$F = (2.5 - 3.5)$

$q = (q_a)_{app.} = \frac{P + W_f}{A}$

q_{app} = Bearing Pressure (applied)
P = Vertical column load
W_f = Weight of foundation and the weight of soil above the foundation
Assume $\gamma_{soil} = \gamma_{con}$, so
$W_f = A \times D \times \gamma_{con}$ γ_{con} (150 lb/ft³) (24 kN/m³)
or $q_a = \frac{P}{A} + \frac{A D \gamma_{con}}{A} = \frac{P}{A} + D \cdot \gamma_{con}$
A = area base area

Assume $q_u = 120$ kPa
$q_{all} = \frac{q_u}{F.S} = \frac{120}{3} = 40$ kPa

Therefore $\frac{P + W_f}{A} \le 40$ kPa

Design procedures:

$8 \quad q_{app} = \frac{P}{A} + \gamma D$

① Known: ~~dimensions~~ dimensions, P
req. : Check F
(a) Find $q_a = q = \frac{P + W_f}{A}$
(b) find q_u from BC eq.
(c) Find $F = \frac{q_u}{q_a}$

Given	Req.	procedure
dim., P	F	- Find q_u, set $q_a = q_{app}$, - Check $F = \frac{q_u}{q_{app}}$
dim., F	P	- Find q_u, - Cal q_a, - set $q_{app} = q_a$
F, P	dim.	

② Known: ~~Demisions~~ Dimensions, F → if it is not given use (F = 3)
Req. : P (Ex. multi story building you ~~don't know~~ want to know the max load)
(a) find q_u (BC eq.)
(b) find $q_a = \frac{q_u}{F}$
(c) $q_a = \frac{P + W_f}{A}$ → find P

$q_a = q_{app} = \frac{P}{A} + \gamma D$
$q_a = \frac{q_u}{F}$
q_u in terms of B

③ Known: F, P
Req. : Deminsions
(a) $q_a = \frac{P + W_f}{A}$ in terms of B&L
(b) find q_u $\left(q_a = \frac{q_u}{F}\right) \Rightarrow q_u = q_a \times F$
(c) Find B from q_u $(q_u = cN_c \cdots + 0.5B\gamma N_\gamma)$

More complicated, ~~b/c of F_{c}, F_q~~
b/c S_c, S_q, and S_γ in terms of B&L
d_c, d_q, and d_γ

10

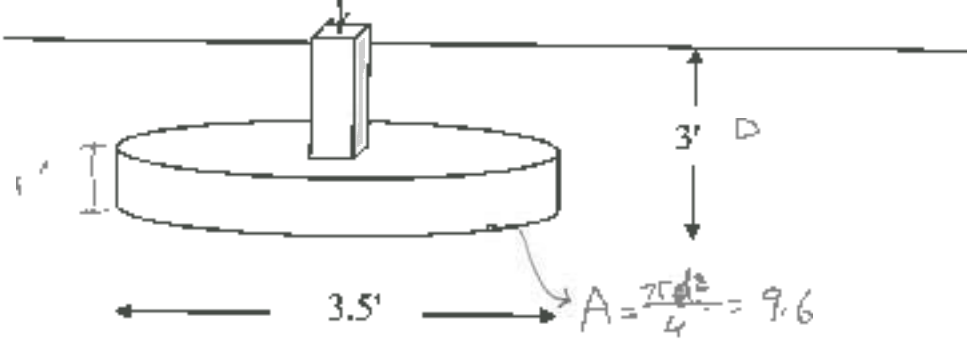
CVEG 4143 BEARING CAPACITY CLASS PROBLEM

class prob 1

You have just taken the job as City Engineer of Pea Ridge, Arkansas. One of your first tasks is to evaluate the safety of an elevated water tank in the national park. The tank weighs approximately ~~40~~ 82 tons when full and is supported on four legs. Each leg has a circular foundation as shown below. ~~Determine the allowable capacity of the foundations. Use both general shear considerations and local shear considerations.~~ F.S

$\frac{82}{4} = 20 \text{ ton} = 40{,}000 \text{ lb}$

$P = 40{,}000$



$A = \frac{\pi d^2}{4} = 9.6$

$q_a = q = \frac{P + W_f}{A}$

Properties:

$W_f = A * D * \gamma_c = 9.6 * 3 \times 150 \text{ pcf} = 4320 \text{ lb}$

Silt (ML)

$\phi = 25^\circ$
$\gamma = 110$ pcf
$c = 170$ psf

$q_a = \frac{40{,}000 + 4320}{9.6} = 4616 \text{ psf}$

$q_u = 1.3\, c N_c + q N_q + 0.3\, \gamma B N_\gamma$

$q = \gamma D = 110 * 3 = 330$

from Table for $\phi = 25$; $N_c = 25.1$, $N_q = 12.7$, $N_\gamma = 9.2$

$q_u = 1.3(170)(25.1) + (330)(12.7) + 0.3(110)(3.5)(9.2)$

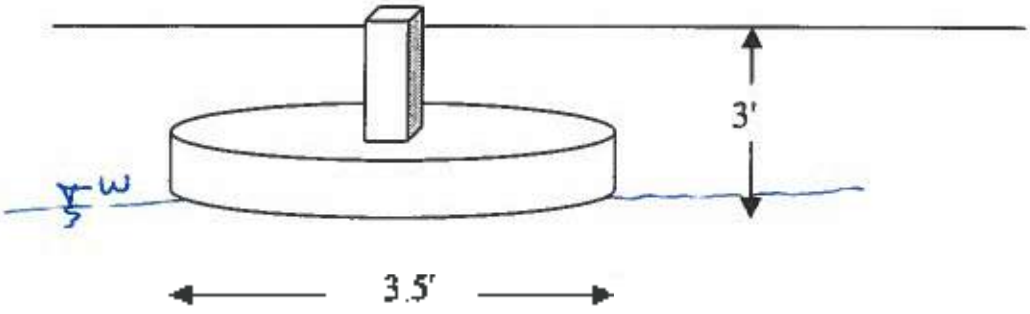
$q_u = 10{,}800 \text{ psf}$

$F = \frac{q_u}{q_a} = \frac{10{,}800}{4{,}616} = 2.33 < 2.5$ Not ok

CVEG 4143 BEARING CAPACITY CLASS PROBLEM ② (1)

You have just taken the job as City Engineer of Pea Ridge, Arkansas. One of your first tasks is to evaluate the safety of an elevated water tank in the national park. The tank weighs approximately 20 tons when full and is supported on four legs. Each leg has a circular foundation as shown below. Determine the allowable capacity of the foundations. Use both general shear considerations and local shear considerations.

Solve the proplom for CH and W.T at 3ft



Properties:

Silt (~~ML~~) (CH)

ϕ_u = ~~29°~~ 0

γ = 110 pcf

c_u = ~~120 psf~~ 800 psf

$q_a = 4616$ psf from #1

$q_u = 1.3 C_u N_c + \bar{q} N_q + \cancelto{0}{0.3 \gamma B N_\gamma}$

for $\phi_u = 0$, $N_c = 5.7$, $N_q = 1$, $N_\gamma = 0$

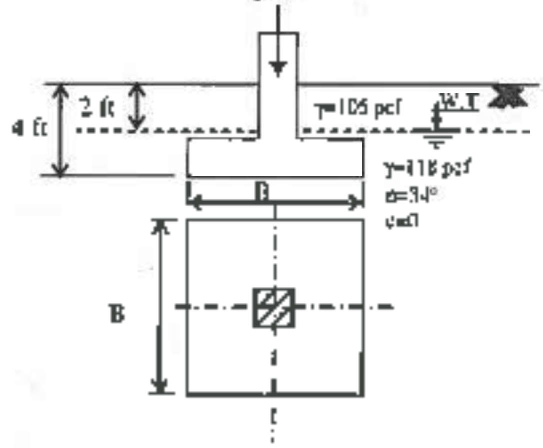
No correction is needed (Total stress analysis)

$q_u = 1.3(800)(5.7) + (330)(1) = 6260$

$$F = \frac{q_u}{q_a} = \frac{6260}{4616} = 1.35 < 2.5$$

using total stress analysis highly ↓ B.C

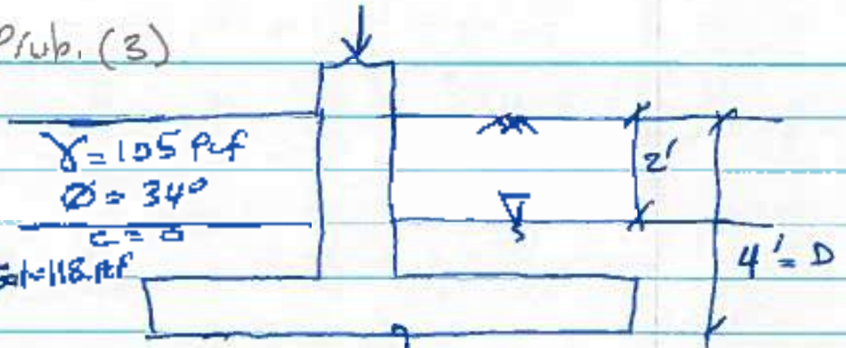
A square foundation has to be constructed as shown below. Determine the size of the footing with F=3.



$P_{gross} = 150{,}000\ lb$ (12)

~~Example~~: class Prob. (3)

Req. B @ F = 3



Solution:

$$q_a = \frac{P + W_f}{A} = \frac{150000}{B^2}$$

$$q_a = \frac{q_u}{F} \Rightarrow q_a = \frac{1}{3}(q_u) \text{ ---- eq(1)}$$

$$q_a = \frac{1}{3}(q N_q S_q d_q + 0.5 \gamma B N_\gamma S_\gamma d_\gamma)$$

For $\phi = 34°$, from Table 7.2, $N_q = 29.4$, $N_\gamma = 41.1$

Shape f. $S_q = 1 + \left(\frac{B}{L}\right)\tan\phi = 1.67$

$$S_\gamma = 1 - 0.4\left(\frac{B}{L}\right) = 0.6$$

depth f.

$$d_q = 1 + 2k\tan\phi(1-\sin\phi)^2 = 1 + \frac{1.05}{B}$$

$$k = \frac{D}{B}$$

$$d_\gamma = 1$$

Corr. for W.T.: $q = 2\times(105) + 2(118-62.4) = 321.2\ lb/ft^2$

$\gamma \longrightarrow \gamma'$ third term $= 118-62.4 = 55.6$ pcf

$$q_a = \frac{1}{3}\left[321.2(29.4)(1.67)\left(1+\frac{1.05}{B}\right) + 0.5(55.6)(B)(41.1)(0.6)(1)\right]$$

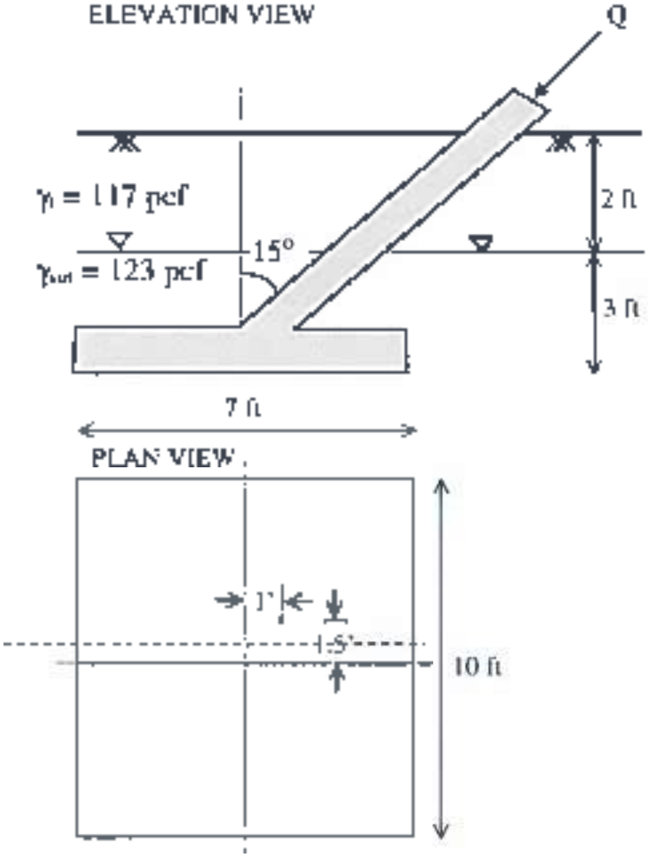
sub in eqn1

$$\frac{15000}{B^2} = 5263.9 + \frac{5527.1}{B} + 228.3B$$

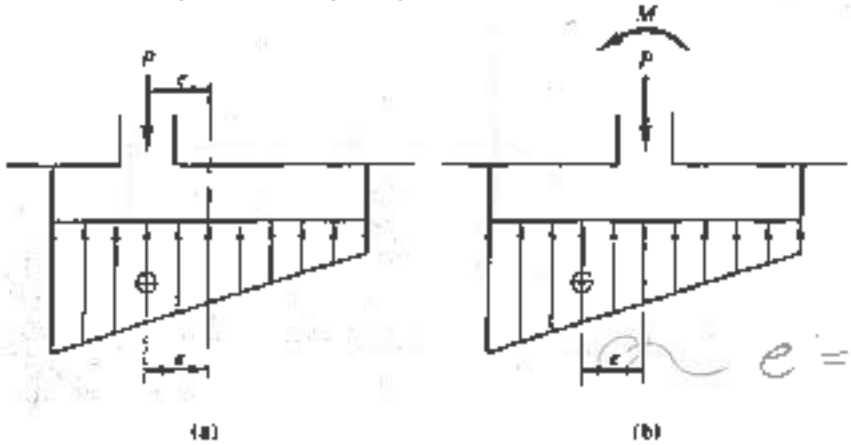
by trail and error, $B \simeq 4.5$ ft

Determine the adequacy of the footings (shown below) against a *general* shear failure. The total load is 100 tons. Use a factor of safety of three.

Soil Properties:
$\phi = 33°$
$C = 100$ psf



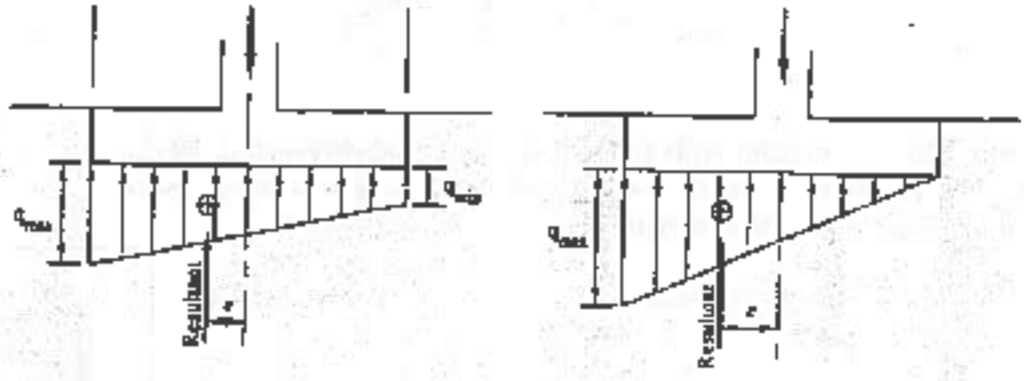
One- way Eccentricity (From Coduto, 2001)



$e = \frac{M}{P}$

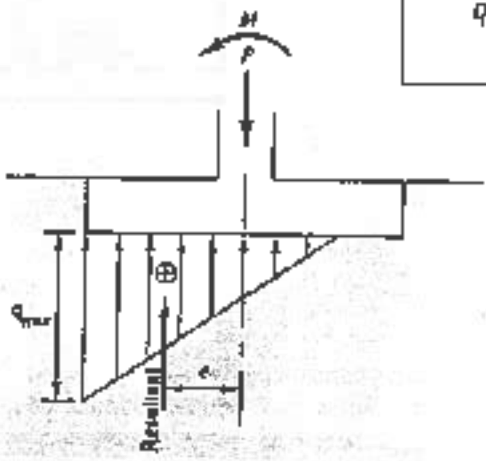
(a) (b)

Figure 5.14 (a) Eccentric and (b) moment loads on shallow foundations.



(a) $e < B/6$ (b) $e = B/6$

$$q_{max} = \frac{P}{BL}\left(1 + \frac{6e}{B}\right)$$
$$q_{min} = \frac{P}{BL}\left(1 - \frac{6e}{B}\right)$$



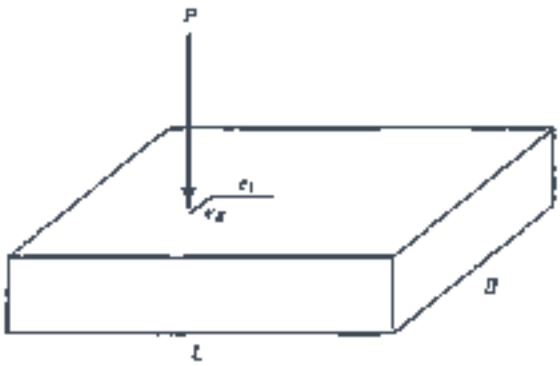
(c) $e > B/6$

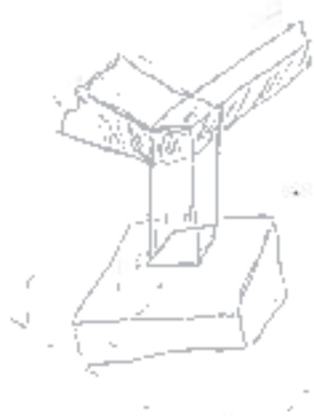
$$q_{max} = \frac{4P}{3L(B - 2e)}$$

e= M/P and

P is the gross load = Net structure load + q

Two-way Eccentricity





When:

$$\frac{6e_B}{B} + \frac{6e_L}{B} \leq 1.0$$

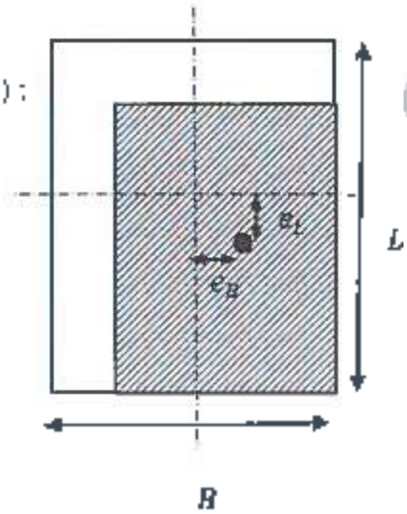
$$q_{max} = \frac{P}{BL}\left(1 \mp \frac{6e_B}{B} \mp \frac{6e_L}{L}\right)$$

F= q_a/q_{max}

Equivalent (Effective) uniformly loaded Area Method (Meyerhof, 1953):

Intermediate value between average and maximum bearing pressure

Will be used to calculate bearing capacity and settlement



1- Determine equivalent dimensions (one or two way eccentricity) :

$$B' = B - 2e_B$$
$$L' = L - 2e_L$$

2- Calculate q_u (ultimate bearing capacity):
   a. Use effective dimensions to calculate shape factors
   b. Use original dimensions to calculate depth factors

3- $q_a = q_{eq} = (P+W_f)/A'$
   - A'= from B' and L'

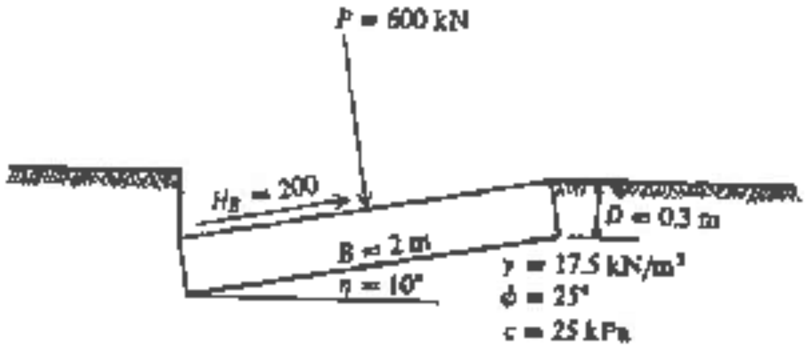
4- $F = q_u/q_a$

See Example 7.4 pp. 239

1-

A 5-ft square, 2-ft deep spread footing is subjected to a concentric vertical load of 60 k and an overturning moment of 30 ft-k. The overturning moment acts parallel to one of the sides of the footing, the top of the footing is flush with the ground surface, and the groundwater table is at a depth of 20 ft below the ground surface. Determine whether the resultant force acts within the middle third of the footing, compute the minimum and maximum bearing pressures, and show the distribution of bearing pressure in a sketch.

2- A square footing 2 X 2 m has to be constructed as shown below. Are the footing dimensions adequate for the given loads if we use a safety factor F = 3?



3- <u>**(Bonus question):**</u> Based on what we have discussed in class about friction and adhesion between soils and construction materials such as concrete and steel, check sliding safety of the footing due to the effect of the force H_B. For now, neglect the footing weight and the lateral resistance of the soil (passive resistance).

Given
dime, P, F

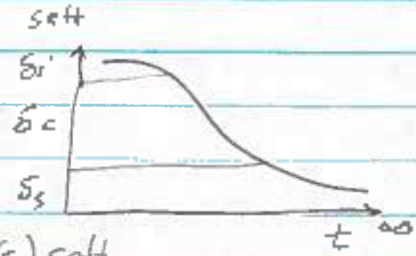
- either find F and check F given
- or find Pall and check with P given.

Check

① - q_u
- $q_{all} = q = \frac{P + W_f}{A}$
- $F = \frac{q_u}{q_{all}}$ (check if $F > 3$)

② - q_u
- Find q_{all} from $q_{all} = \frac{q_u}{F \to \text{given (3)}}$
- Find P_{all} ($q_{all} = \frac{P_{all} + W_f}{A}$)
- check P_{all} with P

Settlement Analysis:



Sand → Initial (Elastic) sett

Clay → Consolidation Settlement + Elastic

Usually to a depth where $\Delta\sigma \le 0.1q$

Settlement in Sand:

The analysis based on in-situ tests → CPT, → SPT, → plateload

why? Difficult to get und. samples from sand

*** (1) See Next page (P. 6)

(2) Schmertmann's Method:

$$\delta = C_1 C_2 C_3 (q_{gross} - \sigma'_{zD}) \sum \frac{I_\varepsilon H}{E_s}$$

different soil not perfectly elastic

C_1 = depth factor

C_2 = Creep factor

C_3 = Shape factor

I_ε = Strain influence factor

$\sigma'_{zD} = \gamma D$

E_s = Modulus of elasticity (See Page (5))

TABLE 2.1 TYPICAL ALLOWABLE TOTAL SETTLEMENTS FOR FOUNDATION DESIGN

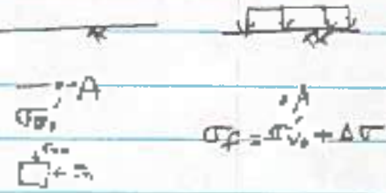
Type of Structure	Typical Allowable Total Settlement, δ_a (in)	(mm)
Office buildings	0.5–2.0 (1.0 is the most common value)	12–50 (25 is the most common value)
Heavy industrial buildings	1.0–3.0	25–75
Bridges	2.0	50

TABLE 2.2 ALLOWABLE ANGULAR DISTORTION, θ_a (COMPILED FROM WAHLS, 1994; AASHTO, 1996; AND OTHER SOURCES)

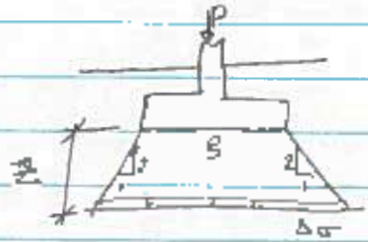
Type of Structure	θ_a
Steel tanks	1/25
Bridges with simply-supported spans	1/125
Bridges with continuous spans	1/250
Buildings that are very tolerant of differential settlements, such as industrial buildings with corrugated steel siding and no sensitive interior finishes.	1/250
Typical commercial and residential buildings.	1/500
Overhead traveling crane rails.	1/500
Buildings that are especially intolerant of differential settlement, such as those with sensitive wall or floor finishes.	1/1000
Machinery[a]	1/1500
Buildings with unreinforced masonry load-bearing walls	
Length/height ≤ 3	1/2500
Length/height ≥ 5	1/1250

[a] Large machines, such as turbines or large punch presses, often have their own foundation, separate from that of the building that houses them. It often is appropriate to discuss allowable differential settlement issues with the machine manufacturer.

* Settlement is due to change in stress
  - → Add structures or fill
  - → Change in G.W.T
  - → lateral movement



How to calculate ($\Delta\sigma$):



1- Approximate method:

$$\Delta\sigma = \frac{P}{(B+Z)(L+B)}$$

bou_sen-nesk

2- Boussinesq Method:

* Square and cont.

$$\Delta\sigma_Z = I_\sigma \, q_s$$

induced

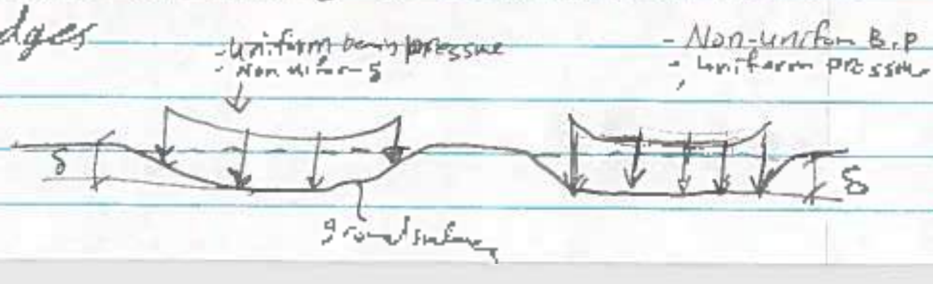
σ_Z = Induced Stress due to load
I_σ = Stress influence factor Fig 7.2
q_s = Net bearing Pressure = $\frac{P_{net}}{A_f}$

③ Add the approximate equation

$$\Delta\sigma = \left[1 - \frac{1}{R} \; \cdots\right]$$

* Point, Rectangular, circular footing → see FE Review Manual
  ↳ $I_\sigma - I_Z$

* Foundation Stiffness:
  Large foundations such as mat, need to calculate $\Delta\sigma$ and S @ the centre and the edges



3A

CVEG 4143 **Settlement** **Homework# 5**

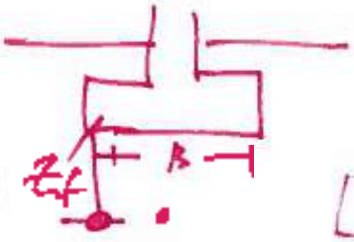
Solve 2.5, 2.6, and 2.7 [Coduto ,2001]

In Coduto (2014), these problems are 5.11, 5.12, and 5.13

2.5 A seven-story steel-frame office building will have columns spaced 7 m on center and will have typical interior and exterior finishes. Compute the allowable total and differential settlements for this building.

2.6 A two-story reinforced concrete art museum is to be built using an unusual architectural design. It will include many tile murals and other sensitive wall finishes. The column spacing will vary between 5 and 8 m. Compute the allowable total and differential settlements for this building.

2.7 A 40 ft × 60 ft one-story agricultural storage building will have corrugated steel siding and no interior finish or interior columns. However, it will have two roll-up doors. Compute the allowable total and differential settlement for this building.

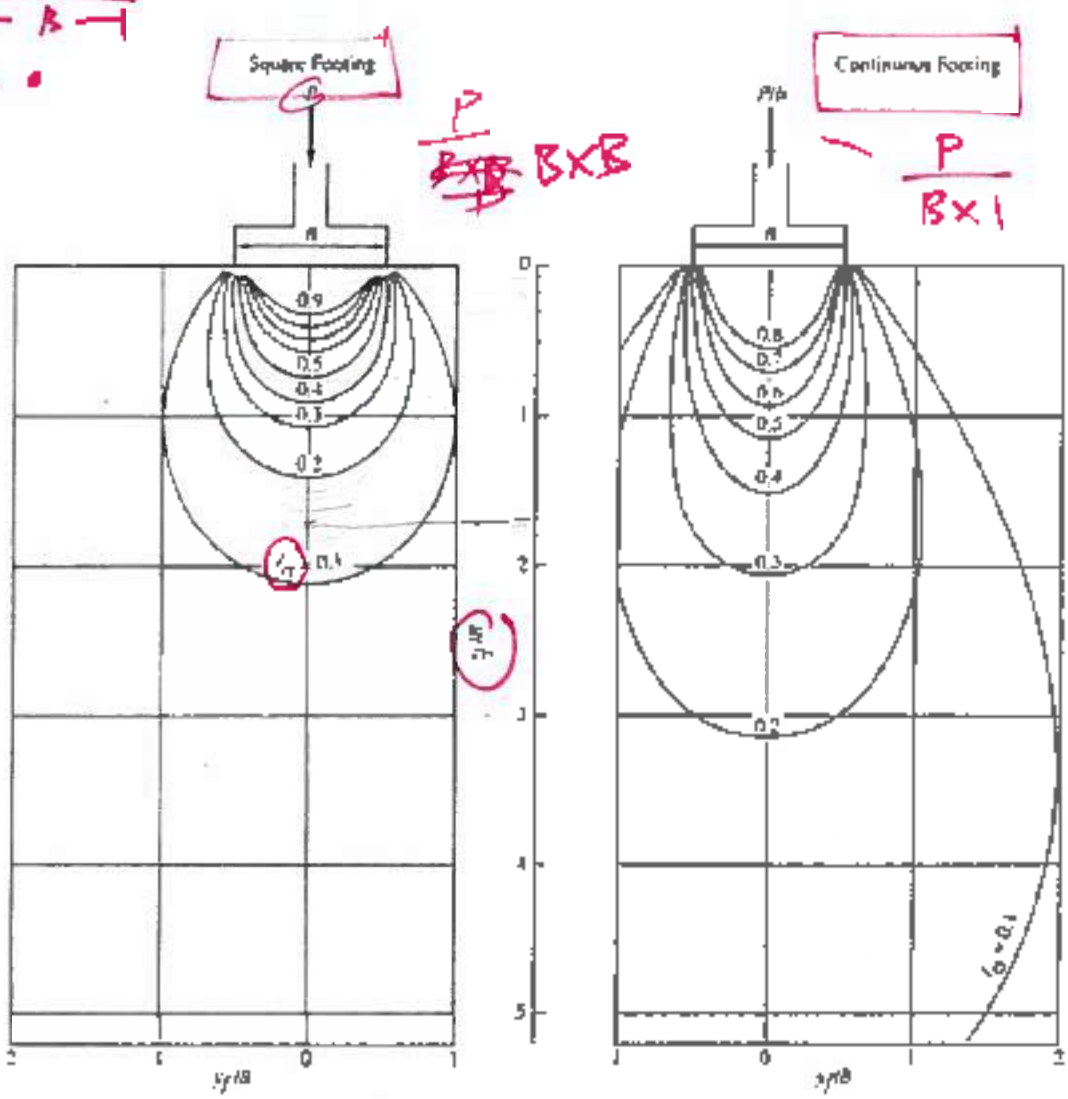


3B

$\frac{Z_f}{B}$

$\frac{P}{B \times B}$ $B \times B$

$\frac{P}{B \times 1}$



x_f = horizontal distance from centerline of footing
z_f = depth below bottom of footing
B = footing width
I_σ = stress influence factor

Figure 7.1 Stress bulbs based on Newmark's solution of Boussinesq's equation for square and continuous footings.

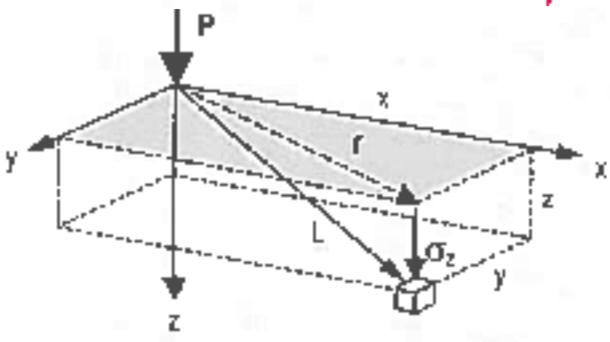
3c

Vertical Stress Caused by a Point Load

Boussinesq Equation:

$$\Delta\sigma_z = \frac{3P}{2\pi}\,\frac{z^3}{(r^2+z^2)^{5/2}} = C_r \cdot \frac{P}{z^2}$$

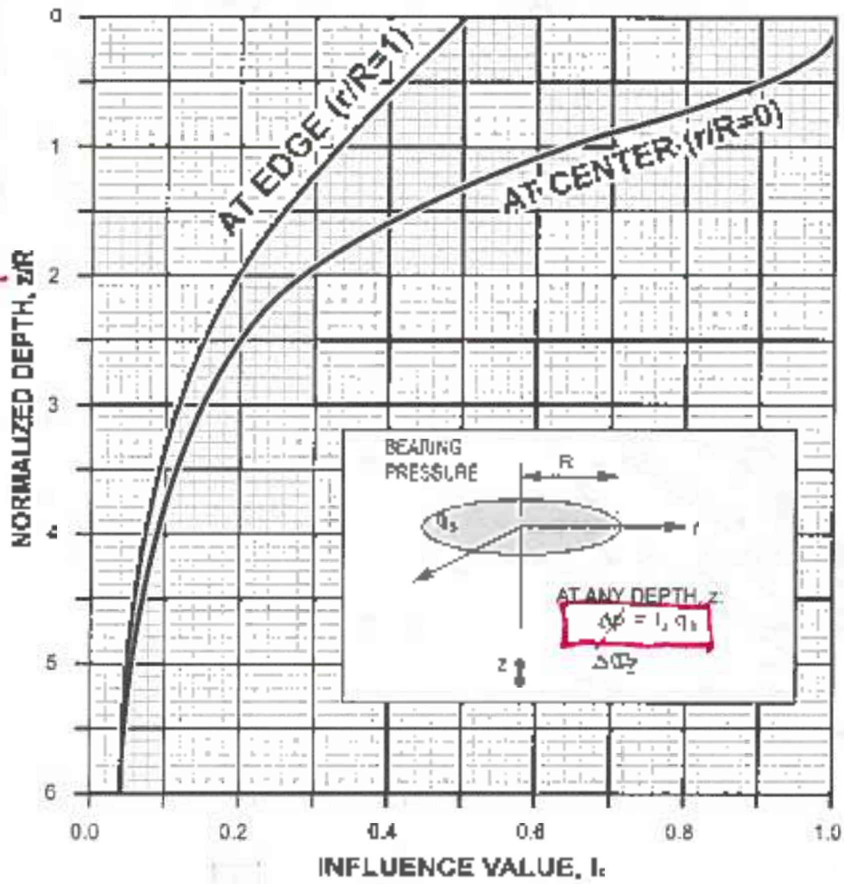
z_f



$\frac{r}{z}$	C_r	$\frac{r}{z}$	C_r	$\frac{r}{z}$	C_r
0.0	0.4775	0.32	0.3742	1.00	0.0844
0.02	0.4770	0.34	0.3632	1.20	0.0513
0.04	0.4765	0.36	0.3521	1.40	0.0317
0.06	0.4723	0.38	0.3408	1.60	0.0200
0.08	0.4699	0.40	0.3294	1.80	0.0129
0.10	0.4657	0.45	0.3011	2.00	0.0085
0.12	0.4607	0.50	0.2733	2.20	0.0058
0.14	0.4548	0.55	0.2466	2.40	0.0040
0.16	0.4482	0.60	0.2214	2.60	0.0029
0.18	0.4409	0.65	0.1978	2.80	0.0021
0.20	0.4329	0.70	0.1762	3.00	0.0015
0.22	0.4242	0.75	0.1565	3.20	0.0011
0.24	0.4151	0.80	0.1386	3.40	0.00085
0.26	0.4050	0.85	0.1226	3.60	0.00066
0.28	0.3954	0.90	0.1083	3.80	0.00051
0.30	0.3849	0.95	0.0956	4.00	0.00040

Vertical Stress Beneath a Uniformly Loaded Circular Area

z_f



$\Delta\sigma_z = I_z q_s$

– 2 –

After the first Day

① Elastic Solution For Settlement:

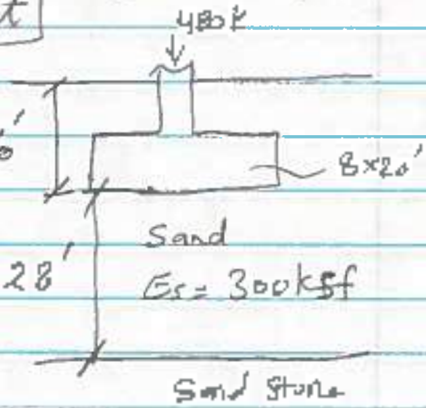
Using Elastic Theory

$$\delta = I_0 I_1 \frac{qB}{E_s} \qquad \text{Janbu et al. (1956)}$$

q = Average bearing Stress
B = Footing width
E_s = Average modulus of compressible soil
I_0 = Influence factor accounting for footing depth
I_1 = Influence factor accounting for footing shape. [Fig 8-2]

$\boxed{\text{is applied only when } E_s \text{ constant}}$

Example:



$\frac{D}{B} = \frac{6}{8} = 0.75$

from fig 8-2 $\rightarrow I_0 = 0.96$

$\frac{L}{B} = \frac{20}{8} = 2.5, \quad \frac{Z_f}{B} = \frac{28}{8} = 3.5$

from fig $I_1 = 0.8$

$$S = I_0 I_1 \frac{qB}{E_s} = (0.96)(3.5) \frac{\frac{480k}{(8 \times 20)\,ft^2} \times 8}{300\,k/ft^2}$$

$= 0.0608\,ft \simeq 0.73\,in$

6a

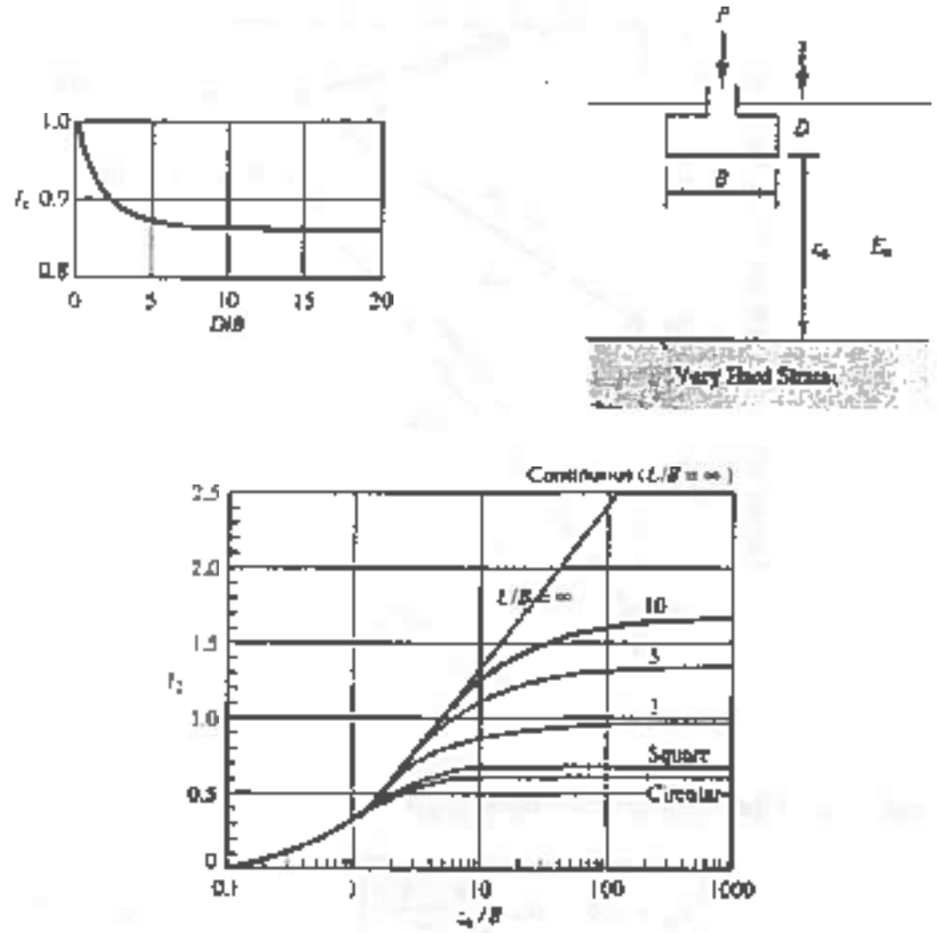


Figure (8.2) Influence factors for Janbu et al. Equation [Coduto, 2014].

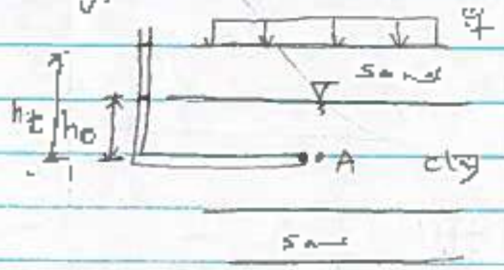
Consolidation Settlement

Cons. sett is time dependant (Why)

- initial cond without loading

$\sigma'_{v0} = \Sigma \gamma z$, $u = \gamma_w h_0$

$\Delta u = 0$



- Loading

Stress in Soil $= \sigma'_{v0} + \Delta\sigma$

$u = h_t \gamma_w$

$\Delta u = (h_t - h_0)\gamma_w$

- @ the end of cons.

$u = h_t \gamma_w$

$\Delta u = 0$

	Load	$\Delta\sigma_z$	σ'_{vA}	Δu
initial	0	0	σ'_{v0}	[illegible]
Loading	q	$\Delta\sigma_{zA}$	$\sigma_{v0}+\Delta\sigma_z$	$(h_t-h_0)\gamma_w$
End of cons	q	$\Delta\sigma_{zA}$	$\sigma_{v0}+\Delta\sigma_z$	0

Δu

total conso. Sett.

one-dim

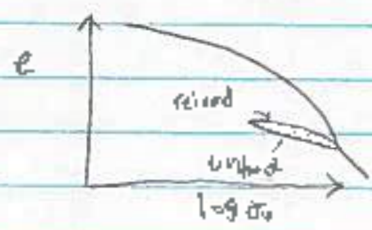
$$S_c = \left(\frac{H_0}{1+e_0}\right)\Delta e$$ (Ter Zaghi's Theory of consolidation)

H_0 = Initial layer thickness

e_0 = Initial void ratio $(Se = G_s w) \Rightarrow e = \frac{G_s w}{S}$

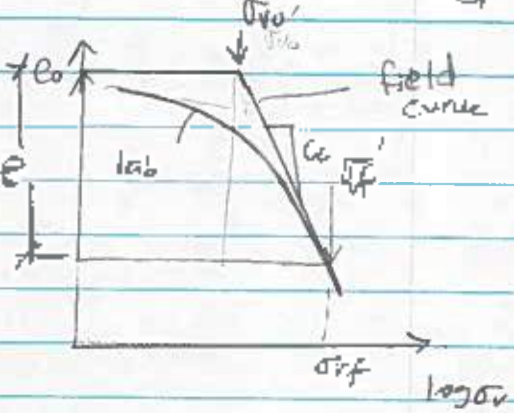
Δe = Change in void ratio

↳ e-log σv curve from consolidometer Test

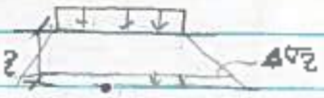


How to calculate Δe?

* NC clay $\sigma_{v0}' = \sigma_c'$



σ_{v0}' = vertical effective stress
σ_c' = Max. past effective stress
(Pre-consolidation Stress)
C_c = Compression index
$\sigma_{vf}' = \sigma_{v0}' + \Delta\sigma_z$

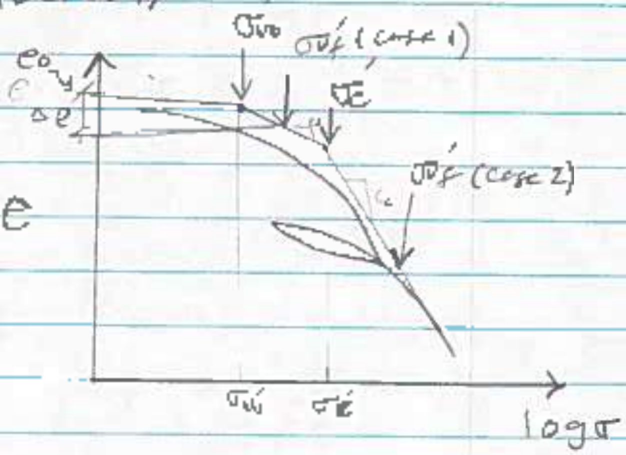


$\sigma_c' = \sigma_{v0}' + \sigma_m'$
σ_m' = Overconsolidation margin

$$\Delta e = C_c (\log \sigma_{vf}' - \log \sigma_{v0}') = C_c \log \frac{\sigma_f'}{\sigma_{v0}'}$$

$$S_c = \frac{H \, C_c}{1+e_0} \log \frac{\sigma_{vf}'}{\sigma_{v0}}$$

* OC clay $\sigma_c' > \sigma_{v0}'$ (OCR > 1)



Case(1) $\sigma_{vf}' < \sigma_c'$

$$S_c = \frac{H \, C_r}{1+e_0} \log \frac{\sigma_{vf}'}{\sigma_{v0}'}$$

C_r = Swelling index (recompression)
or C_s

Case(2) $\sigma_{vf} > \sigma_{v0}' + \sigma_c'$

$$S_c = \frac{H}{1+e_0} \left[C_r \log \frac{\sigma_c}{\sigma_{v0}'} + C_c \log \frac{\sigma_{vf}'}{\sigma_c'} \right]$$

impercal relationships for cons. paramiters

$C_c = 0.009(LL-10)$

$C_r = \frac{1}{5} C_c$ $\quad$ $2\ (\frac{1}{5}-\frac{1}{10})$

LI = liquidity index = $\frac{W_n - PL}{PI}$ $\rightarrow PI = LL - PL$

$LI \geq 0.9$	N.C
$LI < 0.75$	Lightly O.C
$LI < 0$	Heavily O.C

Procedure:

1) Break the consolidating layer into sub layers **
2) Calculate σ'_{vo} and $\Delta\sigma_z$ @ mid-hight of each layer.
   $\sigma'_{vo} = \Sigma(\gamma \text{ or } \gamma')z$ ** w/o any add'tional stress
   $\Delta\sigma_z$ = use Boussinesq
3) Get or estimate N.c, o.c, C_c, C_s, σ'_c
4) Calculate settlement for each layer

$$S_{ct} = \sum_{i=1}^{n} S_{ci}$$

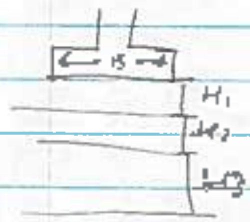
(5) Check $S_{ct} \leq S_a$

a- For computer analysis → use large number of sub layers
a depth where
The last layer at ($\Delta\sigma_z < 0.1\,q$)

b- For manual calculations → use three layers of H_1, H_2, H_3

2nd ed
in 3rd ed. diff. divisi. see the back of the page

Layer	Square and Circular	cont.
H_1	$B/2$	B
H_2	B	$2B$
H_3	$2B$	$4B$



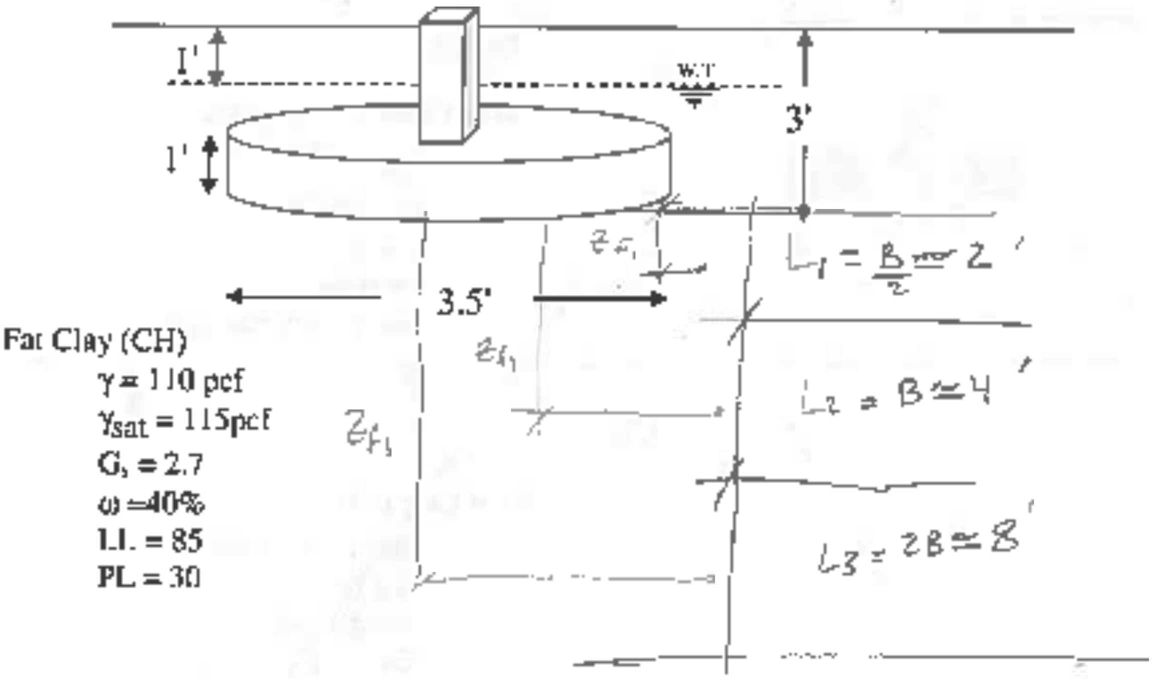
From Coduto (2014)

8.11 A proposed office building will include an 8 ft 6 in square, 3 ft deep spread footing that will support a vertical downward service load of 160 k. The soil below this footing is an overconsolidated clay with the following engineering properties: $C_c/(1 + e_0) = 0.10$, $C_r/(1 + e_0) = 0.022$, $\sigma'_m = 4{,}500\ \text{lb/ft}^2$ and $\gamma = 113\ \text{lb/ft}^3$. This soil strata extends to a great depth and the groundwater table is at a depth of 50 ft below the ground surface. Determine the total settlement of this footing.

8.12 A 1.0 m square, 0.5 m deep footing carries a downward service load of 200 kN. It is underlain by an overconsolidated clay with the following engineering properties: $C_c = 0.20$, $C_r = 0.05$, $e_0 = 0.7$, OCR = 8 and $\gamma = 15.0\ \text{kN/m}^3$ above the groundwater table and $16.0\ \text{kN/m}^3$ below. The groundwater table is at a depth of 1.0 m below the ground surface. Determine the total settlement of this footing.

0.0132.8

You are required to predict the consolidation settlement of the water tank previously deemed dangerously close to a bearing capacity failure (See Bearing Capacity class problem # 3). As you may call the tank weighs approximately 80 tons when full and is supported on four legs. Each leg has a circular foundation as shown below. The soil is fat clay (CH) extends to a great depth. Assume Wf+Ws= 3550 lbs, and $\sigma'_c = 1000 + \sigma'_{vo}$. Is the calculated settlement acceptable?



EARTH RETAINING STRUCTURES (ERS)

Lateral Earth and Water Pressures:

Retaining structures are designed to resist lateral earth pressures and water pressures that develop behind the wall. Earth pressures develop primarily as a result of loads induced by the weight of the backfill and/or retained (in-situ) soil, earthquake ground motions, and various surcharge loads. For purposes of earth retaining system design, three different lateral earth pressures are usually considered:

1) At-rest earth pressure is defined as the lateral pressure that exists in level ground for a condition of no lateral deformation.

2) Active earth pressure is developed as the wall moves away from the backfill or the retained soil. This movement results in a decrease in lateral pressure relative to the at-rest condition. A relatively small amount of lateral movement is necessary to reach the active condition.

3)Passive earth pressure is developed as the wall moves towards the backfill or the retained soil. This movement results in an increase in lateral pressure relative to the at-rest condition. The movements required to reach the passive condition are approximately ten times greater than those required to develop active earth pressure.

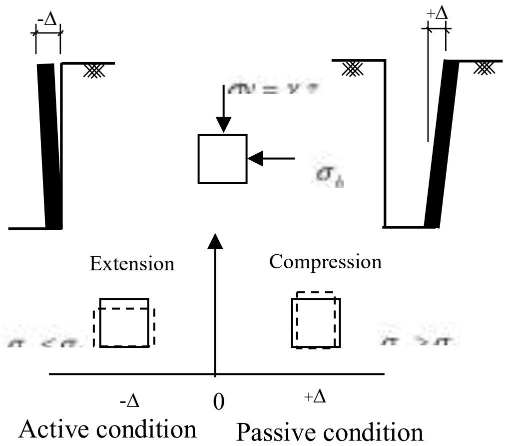
$$K = \frac{\sigma_h}{\sigma_v}$$

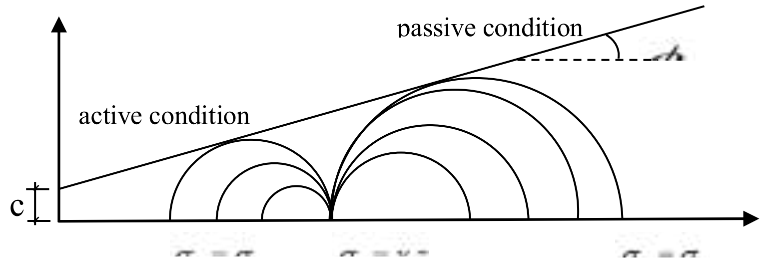
K_o= Coefficient of earth pressure at **rest**.
K_a= Coefficient of **active** earth pressure.
K_p= Coefficient of **passive** earth pressure.

$K_p > K_o > K_a$

σ_a= **active** earth pressure.
σ_p= **passive** earth pressure.

P_A= Resultant of **active** earth pressure.
P_P= Resultant of **passive** earth pressure.





Lateral Earth Pressure Theories:

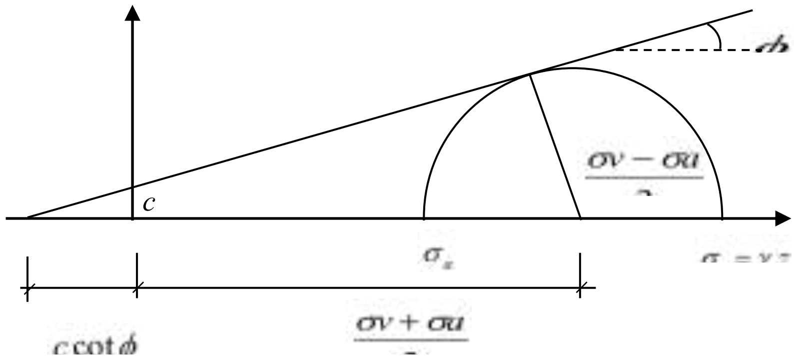
1- Rankine Earth Pressure:

δ= Angle of friction between soil and wall= 0

a- Active condition:

$$\sin\phi = \frac{\dfrac{\sigma v - \sigma a}{2}}{\dfrac{\sigma v + \sigma a}{2} + c\cot\phi}$$

$$\sigma a = \sigma v \frac{1-\sin\phi}{1+\sin\phi} - 2c\sqrt{\frac{1-\sin\phi}{1+\sin\phi}}$$

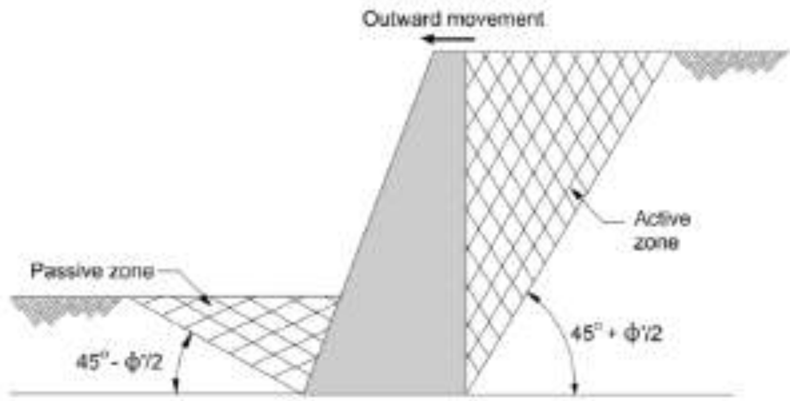


$$Ka = \frac{1-\sin\phi}{1+\sin\phi} = \tan^2(45-\frac{\phi}{2})$$

$$\sigma a = \sigma v Ka - 2c\sqrt{Ka}$$

$$\rho = (45+\frac{\phi}{2})$$

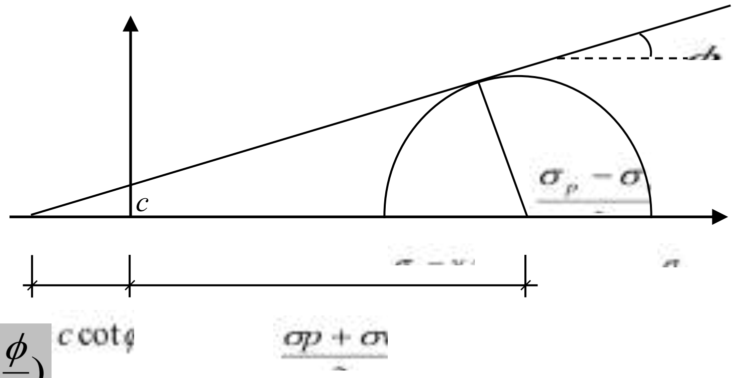
ρ=angle of failure surface



b- Passive condition:

$$\sin\phi = \frac{\dfrac{\sigma_p - \sigma_v}{2}}{\dfrac{\sigma_p + \sigma_v}{2} + c\cot\phi}$$

$$\sigma p = \sigma v \frac{1-\sin\phi}{1+\sin\phi} + 2c\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}$$



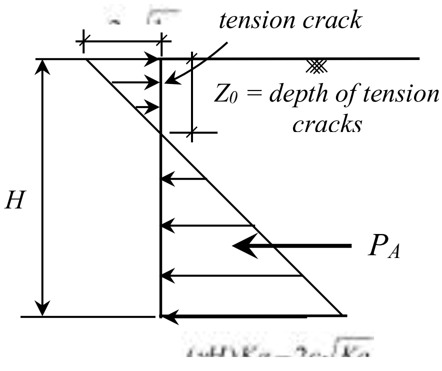
$$Kp = \frac{1+\sin\phi}{1-\sin\phi} = \tan^2(45+\frac{\phi}{2})$$

$$\sigma p = \sigma v Kp + 2c\sqrt{Kp}$$

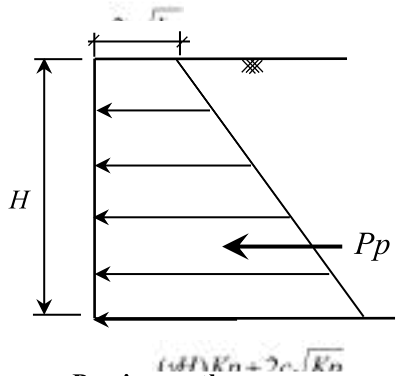
$$\rho = (45-\frac{\phi}{2})$$

Notes:

1- Resultant of earth pressure



Active earth pressure



Passive earth pressure

2- $Ka = \dfrac{1}{Kp}$

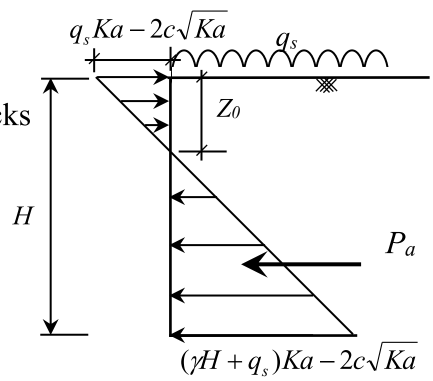
3- For clay $\sigma a = (\gamma z + q_s)Ka - 2c\sqrt{Ka}$

If $(q_s)Ka \geq (2c\sqrt{Ka}) \Rightarrow$ No tension cracks

If σa = 0

$$0 = (\gamma z + q_s)Ka - 2c\sqrt{Ka}$$

$$z_0 = \frac{2c}{\gamma\sqrt{Ka}} - \frac{q_s}{\gamma}$$



active earth pressure

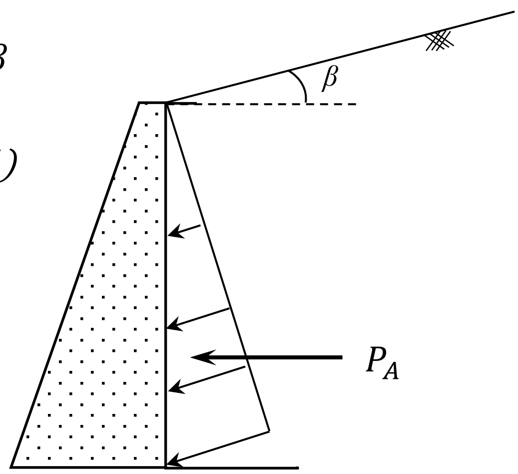
For short term analysis ϕ_u=0, Ka=1, then:

$$z_0 = \frac{2c - q_s}{\gamma}$$

c- Inclined surface (δ=0):

$$P_A = \frac{1}{2}\gamma z^2\ Ka\ cos\beta$$

$$Ka = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\varphi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\varphi}} \text{ .....Eq.(1)}$$



$$P_P = \frac{1}{2}\gamma z^2\ Kp\ cos\beta$$

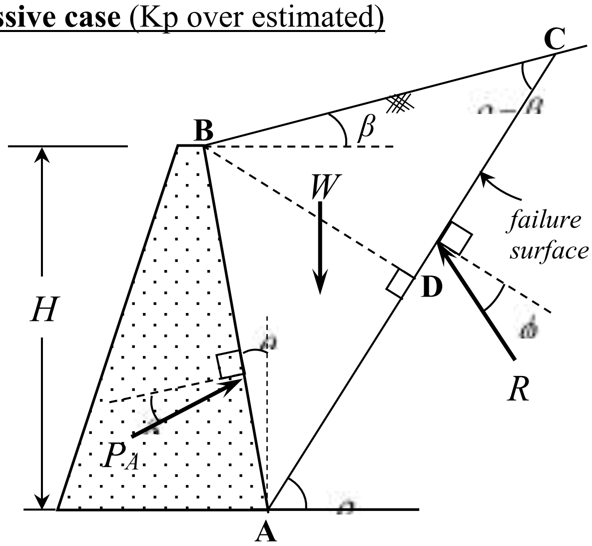
$$Kp = \frac{1}{ka} = \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}} \text{ ....Eq. (2)}$$

2- Coulomb Earth Pressure:

- Friction between soil and wall is considerable (δ≠0)
- In practice, walls are not smooth. Both wall friction and wall adhesion modify the stress distribution near a wall, so wall friction, δ, and wall adhesion, cw, should both be considered as proportions of φ′, and c′ or su, respectively.

a- Cohesionless soil (c=0):

Active case only do not use Coulomb for **passive case** (Kp over estimated)



$$P_A = \frac{1}{2}\gamma H^2 K_{AC}$$

$$K_{AC} = \frac{\cos^2(\phi-\theta)}{\cos^2(\theta)\cos(\delta+\theta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\delta+\theta)\cos(\theta-\beta)}}\right]^2} \quad .....Eq.\ (3)$$

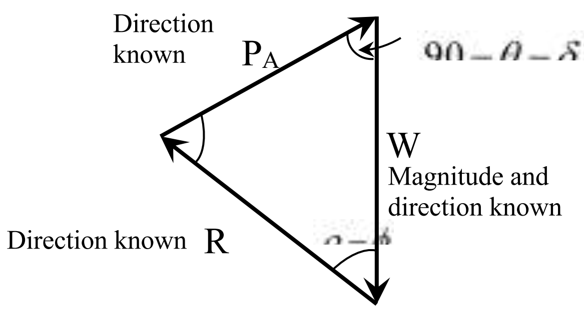
R is the resultant of the perpendicular reaction force and the parallel friction force

If $\beta = 0, \delta = 0, \theta = 0 \Rightarrow Ka = \tan^2(45-\frac{\phi}{2})$

Note here that P_A is NOT horizontal

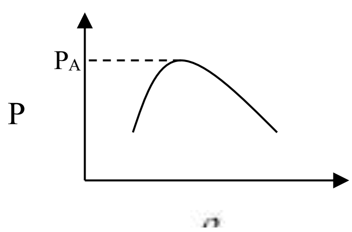
Another method to calculate P_A:

from force diagram



Procedure:

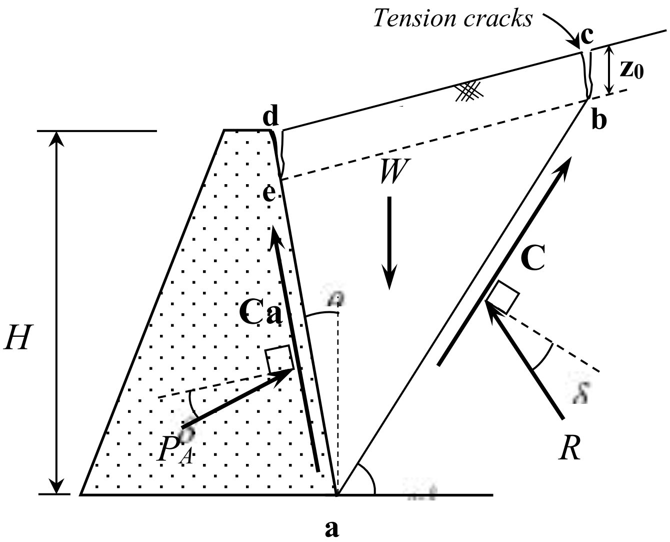
1- Assume different values for ρ.
2- Calculate W
3- Use force diagram to calculate P.
4- Plot ρ vs. P and find P_a (P_{max})



δ= 2/3 ϕ concrete wall
δ= 1/3 ϕ steel

<u>b- Cohesive soil:</u> (c & ϕ soil)

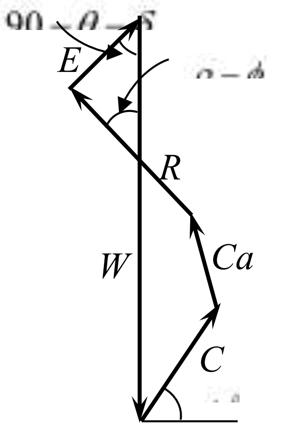
Active Case:



$$\sigma_a = K_{AC}\sigma' v - K_{AC}^{\phi'}.c' \text{.....}Eq.(4)$$

$$K_{AC}^{\phi'} = 2\sqrt{K_{AC}(1+\frac{c_w}{c'})}$$

c_w = soil wall adhesion= 0.5 c_u but $c_w \leq$ 1000 psf for active case
$c_w \leq$ 500 psf for passive case



Force diagram

Total stress analysis (ϕ_u=0) for both active and passive:

- Use σ_v (total stress) to calculate σ_a and σ_p
- $K_{AC}=K_{PC}=1$
- $K_{AC}^{\phi u=0} = K_{PC}^{\phi u=0} = 2\sqrt{(1+\frac{c_w}{c_u})}$

Notes:

1) Equivalent Fluid pressure:

- Sometimes for design purposed, engineers use equivalent fluid density (γ_{eq})

γ_{eq}= equivalent fluid density = Kγ

- According to AASHTO, γ_{eq} shall noly be used when the backfill is free drain (Gravel and Sand)

$$\sigma_a = K\gamma z$$
$$\sigma_a = \gamma_{eq}\, z$$

z = depth below surface of soil, the resultant of the horizontal earth pressure acts at a height of H/3.

Typical values of equivalent fluid density for wall heights not exceeding 6m for sand or gravel are provided in Table 1. The values are presented for at- rest conditions and for walls that can tolerate movements of 25 mm in 6 m (i.e., Δ/H = 1/240).

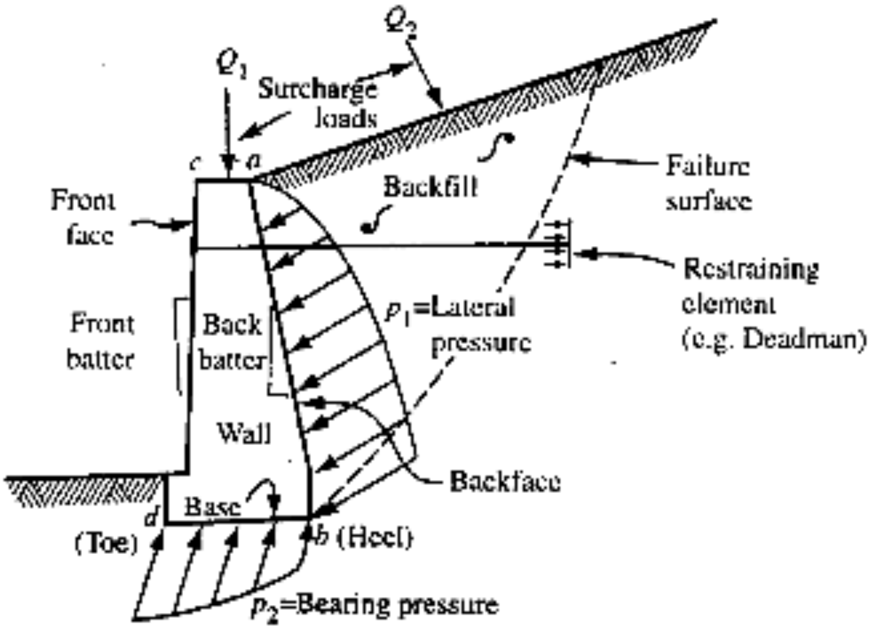
Table 1. Typical Values for Equivalent Fluid Unit Weight of Soils (after AASHTO, 2007)

Type of soil	Level Backfill		Backfill with β=25°	
	At-rest γ_{eq} (kN/m3)	Active Δ/H = 1/240 γ_{eq} (kN/m3)	At-rest γ_{eq} (kN/m3)	Active Δ/H = 1/240 γ_{eq} (kN/m3)
Loose sand or gravel	8.6	6.2	10.2	7.8
Medium sand or gravel	7.8	5.4	9.4	7
Dense sand or gravel	7	4.7	8.6	6.2

2) The Rankine method cannot take account of wall friction, and accordingly K_a is overestimated slightly, and K_p is under estimated, thereby making the Rankine method conservative for most applications.

Design of Retaining Structures

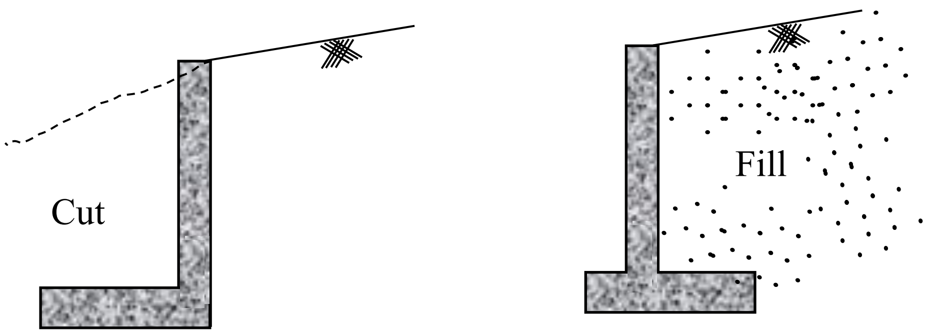
Earth retaining systems (or retaining walls) are used to hold back earth and maintain a difference in the elevation of the ground surface as shown in Figure 1-1. The retaining wall is designed to withstand the forces exerted by the retained ground or "backfill", and to transmit these forces safely to foundation and/or to the portion of restraining elements located beyond the failure surface (FHWA Reference Manual, 2007).



CLASSIFICATION OF EARTH RETAINING STRUCTURES

In this manual, earth retaining systems may be classified according to:

- Load support mechanism, i.e., externally or internally stabilized walls;
- Construction concept, i.e., fill or cut walls; and
- System rigidity, i.e., rigid or flexible walls.

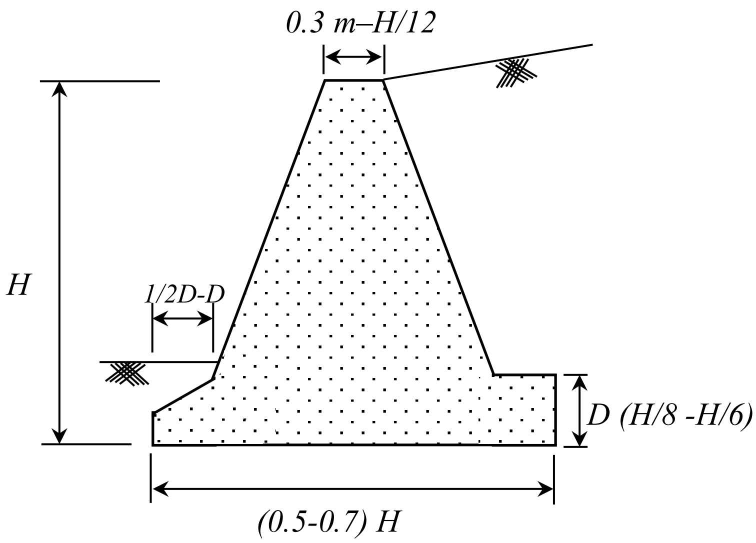


Types of retaining walls

Retaining walls are usually classified by their stability mechanism

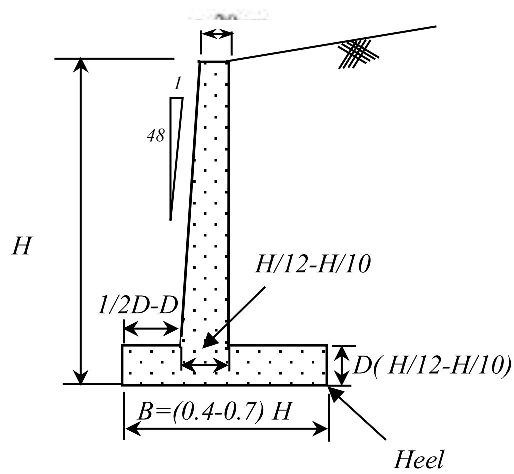
1- Gravity walls:

- $H \leq 3m$
- Stability due to self-weight aided by passive resistance.



2- Cantilever walls:

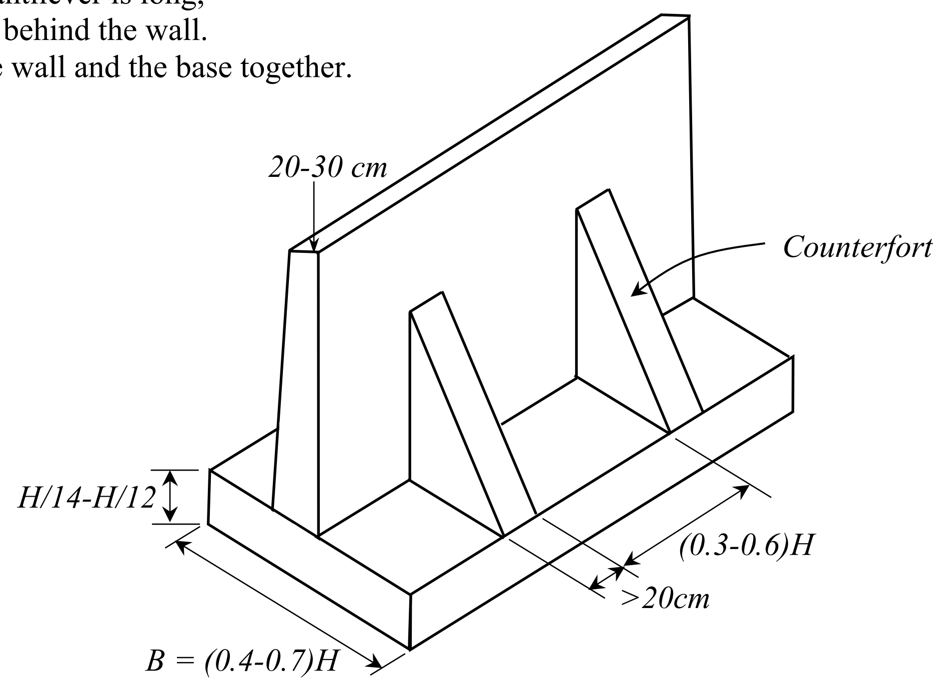
- H from 3 to 7.5 m
- Reinforced concrete walls
- Utilize cantilever action to retain the soil mass
- Stability partially achieved from weight of soil on the heel portion of base slab.



3- Counterfort walls:

- H >7.5 m

- Is used when the cantilever is long, or for high pressure behind the wall.
- Counterforts tie the wall and the base together.



Stability of Retaining Walls:

(1) Assume a direction of possible unacceptable movement (sliding, overturning).
(2) Draw a FBD (free body diagram) of the wall.
(3) Determine the forces acting on the wall which tend to cause sliding or overturning.
(4) Determine the forces which resist the failure mode.
(5) Compute the Factor of Safety (FS) for each failure mode.
(6) Compare the FS with the accepted minimum (design criteria).

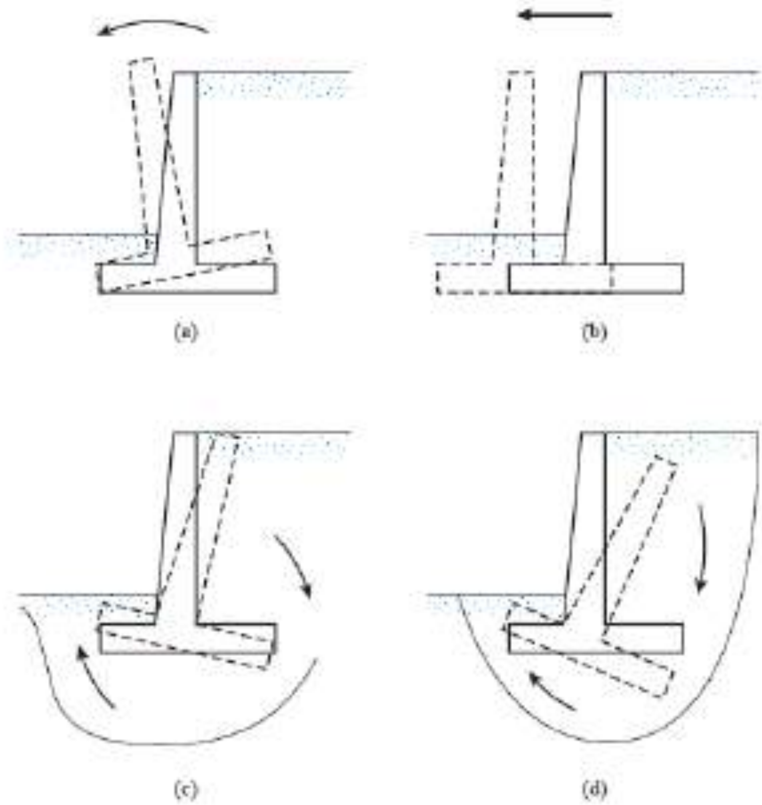
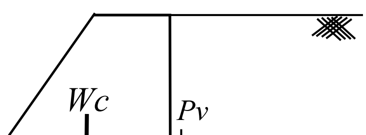


Figure 8.5 Failure of retaining wall:
(a) by overturning; (b) by sliding;
(c) by bearing capacity failure;
(d) by deep-seated shear failure

From Das (2011)

1- **Sliding Stability**. Factor of safety against sliding (FS_{SL}), is calculated as follows:



$$FS_{SL} = \frac{\sum resisting \cdot forces}{\sum driving.force}$$

Example: $FS_{SL} = \dfrac{S + Pp}{Pa} \geq 2$

If Pp neglected, (FS) sliding $\geq$ *1.5*

S = adhesion + friction

friction (F)= $\mu N = N \tan\delta$

$S = c_w.B + N \tan\delta$

N= The vertical component of the resultant (R).

Pa is the horizontal component of the lateral earth pressure force (P_A) when P_A acts at an angle.

Sometimes soils of foundation and fill behind the wall are different.

2- **Overturning Stability**. The FS_{OT} is calculated as follows.

$$FS_{OT} = \frac{\sum M_R}{\sum M_{OT}} = \frac{\sum \text{Resisting Moments about the Toe}}{\sum \text{Overturning Moments about the Toe}} \geq 2$$

Example: $(FS)_{overturning} = \dfrac{Wc.lc}{Pa.la - Pv.B} \geq 2$

3- Kern: The normal component of the resultant, **N**, should be located within the middle 1/3 (the "kern") of the wall's base **($e \leq B/6$)**. The magnitude of **N** is found by summing the vertical forces on the free body diagram of the wall and adjacent soil masses. The point of application of **N** is found by summing moments of all appropriate forces acting on the free body diagram about some convenient point and solving for d.

$e = B/2 - d$

$\sum M_{toe}=0$

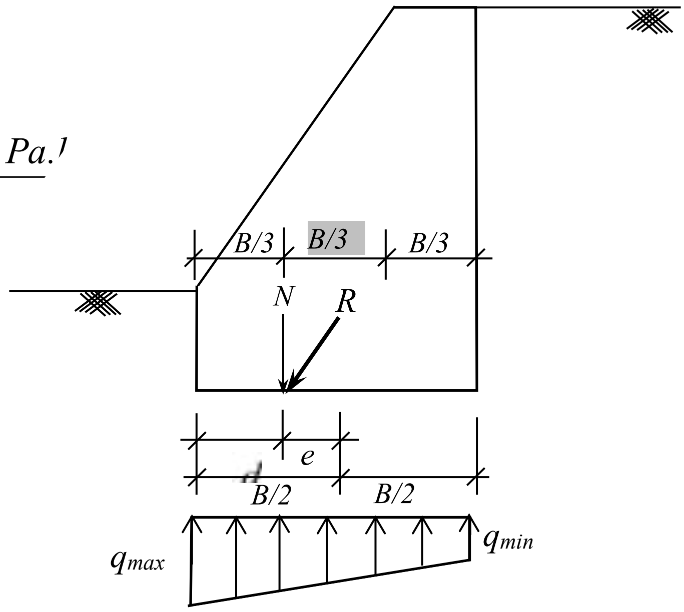
$$\sum M_R - \sum M_{OT} = N \times d$$

$$\therefore d = \frac{\sum M_R - \sum M_{OT}}{N}$$

Example:

$N=Wc + Pv$

$$d = \frac{Wc.\ lw\ +\ Pv.B\ +\ Pp.lp\ -\ Pa.l}{Wc\ +\ Pv}$$



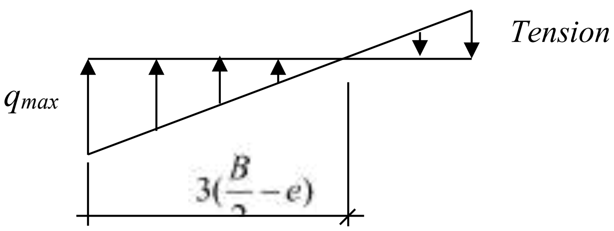
When $e \leq B/6$

$$q_{\max} = \frac{N}{B}(1 + \frac{6e}{B})$$

When $e > B/6$

$$q_{\max} = \frac{2N}{3(\frac{B}{2} - e)}$$

Not preferred because of tension



A wall for which **N** falls outside of the middle 1/3 should be redesigned. If **N** is found to be forward of the middle third of the base (towards the toe), the analysis indicates the soil under the rear of the base must be placed in tension to maintain stability. If **N** is found to be to the rear of the middle third of the base (towards the heel), the structure is over designed and could be designed/built more economically.

4- Bearing Failure. The bearing stability of a retaining wall depends upon a comparison of the bearing capacity of the supporting soil (q_u) and the maximum bearing pressure applied to the soil (q_{max}). Bearing stability analysis of a retaining wall is an adaptation of the bearing capacity analysis for continuous or strip footings. Two important differences exist:

a. The resultant normal force applied by the wall base on the soil is not located at the center of the wall base as it is with footings. The wall's applied normal force, **N**, is eccentrically located a distance **d from** the toe of the wall (usually within the middle third of the wall).

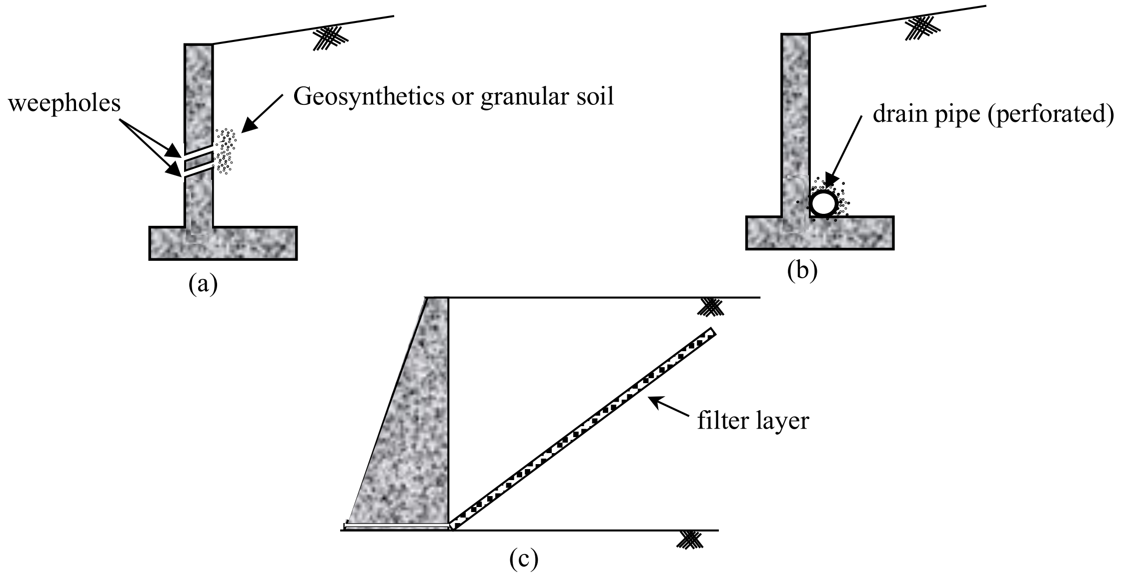
b. The depth of surcharge for the wall's base is always much less on the toe side of the wall than the heel side (see the typical shape of cantilever walls). This results in a different pressure distribution than a strip footing where the depth of surcharge is often the same on both sides

$$FS_B = \frac{BearingCapacity}{Bearing\,\mathrm{Pr}\,essure} = \frac{Ultimate}{Applied} = \frac{q_u}{q_a} \rangle 3.0$$

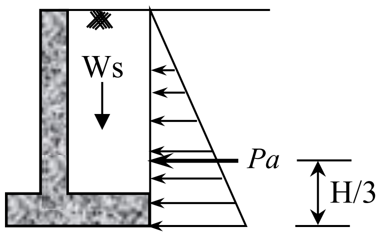
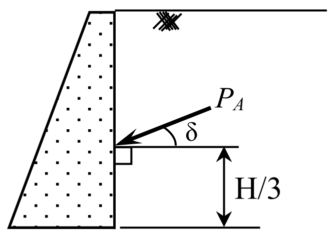
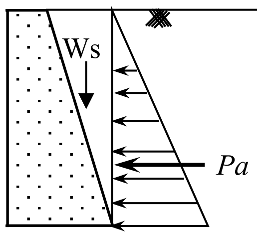
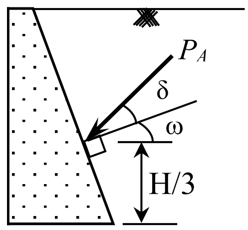
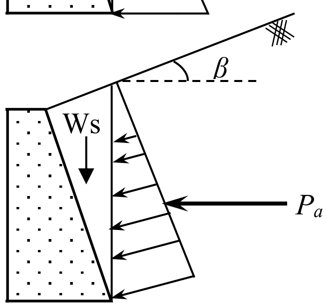
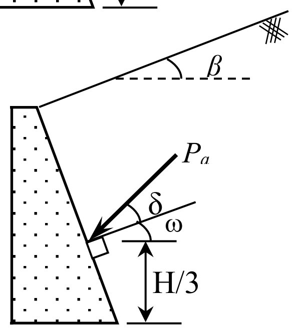
4- Other considerations:

A. Settlement and General Stability: Thorough analysis of a retaining wall should include settlement and general stability. Settlement refers to the change in height of compressible layers associated with a change in effective stress on the soil. It is not typically checked for walls of modest size since it usually replaces excavated soil. General stability refers to the overall slope stability of the surrounding soil. It is possible for the entire wall to be within the failure zone of a larger stability failure. This topic will be addressed in the Earth Structures course as part of advanced slope stability.

B. Drainage: As seen in example problems and class problems during the lesson block on lateral earth pressure, a saturated backfill exerts a tremendous amount of pressure on the retaining structure. Therefore, it is essential to provide adequate drainage of the soil behind the retaining wall. The drainage often requires more than just weepholes at the base of the wall. For example, a select backfill of gravel or sand may be necessary if the in situ soil is a compressible clay. Additionally, filter material (often geosynthetics) or a properly designed aggregate filter is required to prevent the weephole or drainage pipes from clogging



Use Rankine or Coulomb to calculate lateral earth pressure as follows:

Rankine ($\delta = 0$)	Coulomb ($\delta \neq 0$)
	
	
	

Class Problem (4): Retaining Wall Stability

For the wall shown in Figure 2 determine the stability of the retaining wall for sliding, overturning, and kern.

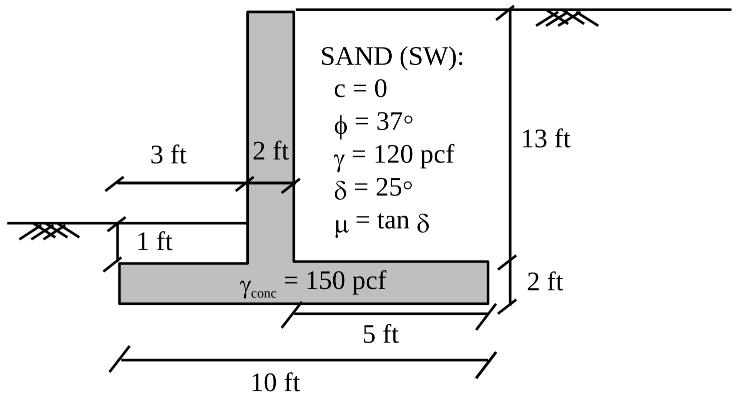


Figure 1. Cantilevered Retaining Wall with Well-Graded Sand Backfill (SW)

1. Draw the FBD.

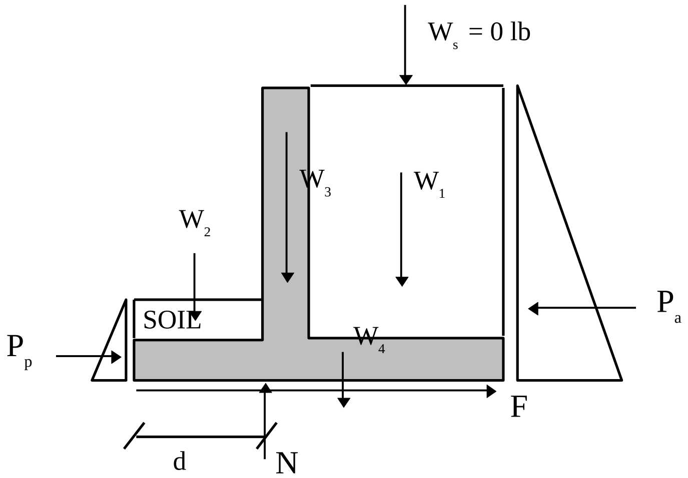


Figure 2. FBD of Example Problem Cantilevered Retaining Wall

2. Determine Rankine active and passive earth pressure coefficients.

$K_p = \tan^2(45 + \phi/2) = \tan^2(45 + 37/2) = 4.0$
$K_a = 1/K_p = \tan^2(45 - \phi/2) = \tan^2(45 - 37/2) = 0.25$

3. Determine Rankine Soil Forces (Note: No GWT or surcharge).

$$P_a = \frac{1}{2}\gamma H^2 K_a = \frac{1}{2}120)(15)^2(0.25) = 3380 lbs$$

$M_{OT} = P_a * 5\text{ ft} = 16900$ ft-lb

$$P_p = \frac{1}{2}\gamma\ H^2 K_p = \frac{1}{2}(120)(3)^2(4) = 2160 lb$$

$\gamma = 120$ pcf $\gamma_{conc} = 150$ pcf

Weights:		Moment Arm (about toe)	M_R
$\mathbf{W_1} = \gamma\ (5)\ (13) =$	7800 lb	7.5 ft	= 58500 ft-lb
$\mathbf{W_2} = \gamma\ (3)\ (1) =$	360 lb	1.5 ft	= 540 ft-lb
$\mathbf{W_3} = \gamma_c\ (2)\ (13) =$	3900 lb	4.0 ft	= 15600 ft-lb
$\mathbf{W_4} = \gamma_c\ (2)\ (10) =$	3000 lb	5.0 ft	= 15000 ft-lb
	N = 15060 lb		M_R = 89640 ft-lb

4. Compute Factors of Safety:

a. FS_{SL}

$\mathbf{F} = \mu\ \mathbf{N}$ = tan 25° (15060 lb) = .466 (15060 lb) $\mathbf{F}$ = 7018lb.

$$FS_{SL} = \frac{F + P_p}{P_a} = \frac{7018 + 2160}{3380} = \frac{9180}{3380}$$
$$FS_{SL} = 2.72 \rangle 1.5 \quad \therefore OK$$

b. FS_{OT}

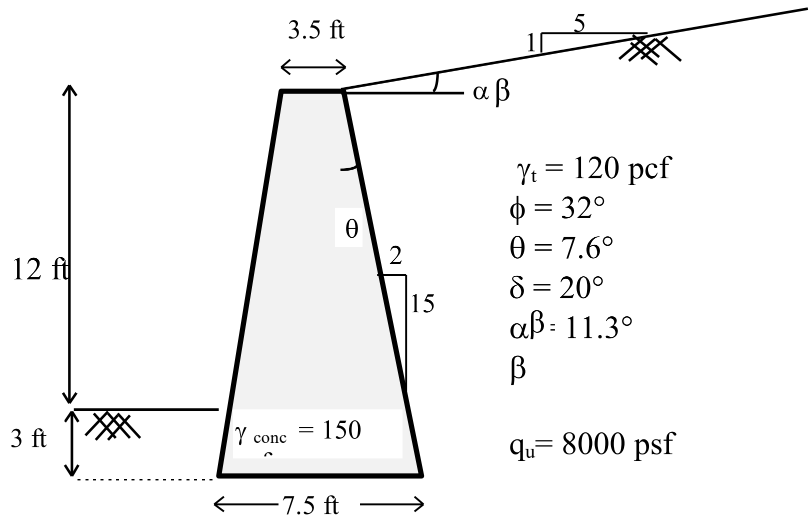
$$FS_{OT} = \frac{M_R}{M_{OT}} = \frac{89640\,ftlb}{16900\,ftlb} = 5.3 \rangle 2.0 \quad \therefore OK$$

c. Kern

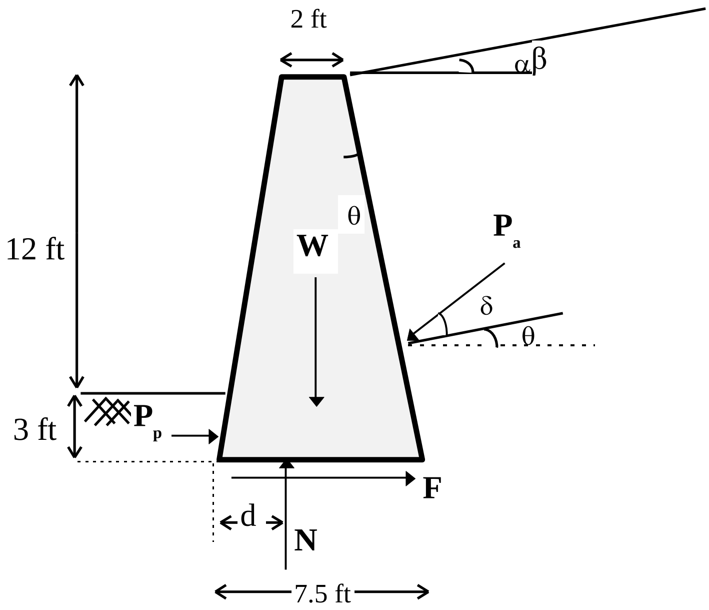
$$d = \frac{M_R - M_{OT}}{N} = \frac{89640 - 16900\,ftlb}{15060\,lb} = \frac{72740\,ftlb}{15060\,lb}$$
$$d = 4.83\,ft$$
$$e = B/2 - d = 10/2 - 4.83 = 0.17\,ft$$
$$B/6 = 1.66\,ft$$
$$e < \mathrm{B/6} \quad \therefore OK$$

Class Problem (5): Retaining Wall Stability (Gravity Wall)

Given the wall shown below. Evaluate the stability of the wall. The wall shown below is the same wall analyzed in class problem 3 of Lateral Earth Pressure.



1. Draw the FBD.



2. Determine Active and Passive Earth Pressure Coefficients

a. Active Earth Pressure (Coulomb).

b. Passive Earth Pressure (Rankine).

3. Determine forces and moments

a. P_a

b. P_p

c. W_{wall}

d. Friction

e. Table

Forces	Magnitude (lbf)	POA (ft)	Moment (about toe)

3. Factors of Safety.

a. FS$_{sl}$

$$FS_{SL} = \frac{\sum F_R}{\sum F_d} = \frac{\text{Resisting Forces}}{\text{Driving Forces}}$$

b. FS$_{OT}$

$$FS_{OT} = \frac{\sum M_R}{\sum M_{OT}} = \frac{\sum \text{ Resisting Moments about the Toe}}{\sum \text{ Overturning Moments about the Toe}}$$

c. Kern $e < B/6$

d. Bearing

$$FS_B = \frac{BearingCapacity}{Bearing\,\mathrm{Pr}\,essure} = \frac{Ultimate}{Applied} = \frac{q_u}{q_a} \rangle 3.0$$

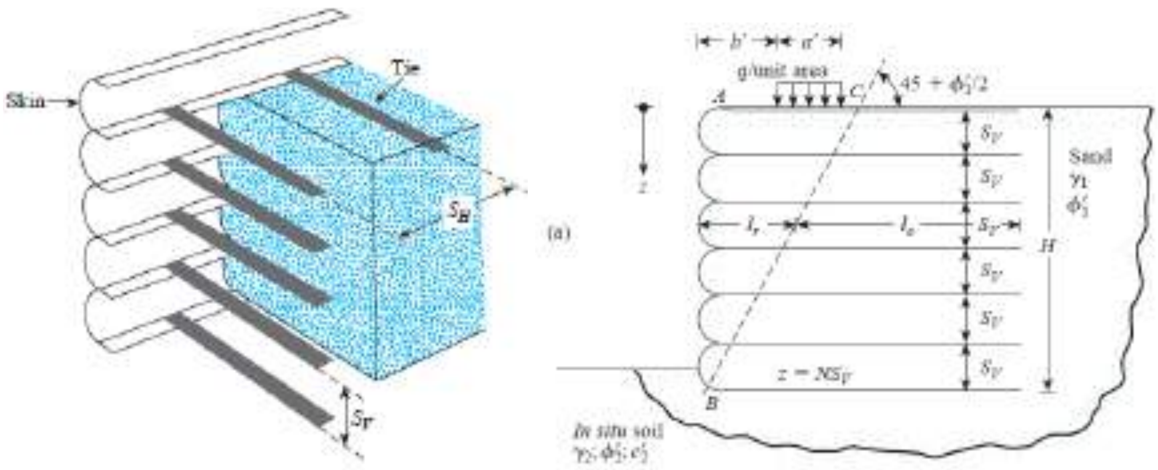
4. Conclusion.

Check	Result	Goal	Conclusion	Course of Action
FS $_{SL}$		**FS $_{SL}$ ≥ 1.5**		
FS $_{OT}$		**FS $_{SL}$ ≥ 2.0**		
Kern		**1/3 ≤ d/B ≤ 2/3**		
FS $_{Bearing}$		**FS $_{BEARING}$ ≥ 3.0**		

Mechanically stabilized Earth (MSE)

- Soil reinforcement has been used in the construction of foundations, retaining walls, and embankments.

- Metal strips , geotextile, or geogrids may be utilized to lock soil mass together through shearing stresses
- Facing panels are used to o maintain appearance and to avoid soil erosion between the reinforcements. The facing units may be curved or flat metal plates or precast concrete strips or plates.
- Typically free draining granular backfill (with less than 15 percent passing the No. 200 sieve) is used in reinforced zoon.



From Das (2011)

MSE wall stability:

- Tests with experimental walls indicate that the Rankine wedge (of angle ρ = 45° + ϕ/2) adequately defines the "soil wedge".
- Assume all the tension stresses are in the reinforcement outside the assumed soil wedge zone—typically the distance Le.

The wall failure may occur in one of three ways:

a. Tension in the reinforcements (breakage or pullout)
b. Bearing-capacity failure of the base soil supporting the wall.
c. Sliding of the full-wall block along base AB.

Design of Deep Foundations

Deep Foundation

A deep foundation is used to carry and transfer the applied load to the bearing ground located at some depth below ground surface. The main components of the deep foundation are the **pile cap** and the **piles**.

- Piles are long and slender members which transfer the load to a deeper soil or rock of high bearing capacity avoiding shallow soils of low bearing capacity.
- The main types of materials used for piles are wood, steel and concrete. Piles made from these materials are driven, drilled or jacked into the ground and connected to the pile cap.

When to use pile foundations:

1- The soil immediately beneath the structure is weak or unstable i.e.:
   - the soil does not have adequate bearing capacity,
   - the magnitude of the estimated settlement is not acceptable
   - expansive or collapsible soils.
2- When a cost estimate indicates that a pile foundation is cheaper than any other compared foundation or ground improvement.
3- Piles are a convenient method when foundation must penetrate through water such as those for a pier or when the soil is subjected to scour.
4- Piles are sometimes used to resist horizontal loads. This type of situation is generally encountered in the construction of earth-retaining structures and foundations of tall structures that are subjected to high wind or to earthquake forces.
5- Piles can also be used to resist uplift forces. The foundations of some structures such as offshore platforms, transmission towers, and basement mats below the water table are subjected to uplift forces.

Classification of piles

Piles are classified according to pile material, their effect on the soil, and load transmission

1- Type of material:

• Timber:

- Have been used for thousands of years and still used for many applications.
- Cannot withstand high driving stress
- Can stay undamaged indefinitely if they are surrounded by saturated soils. However, they are subjected to attack from marine organisms and insects.

• Steel:

- Usually pipe piles or steel H-section piles.

- Steel piles can penetrate hard layers such as dense gravel and soft rock because of their small cross-sectional area combined with their high strength.

- They can be easily cut off or joined by welding.

- If the pile is driven into a soil with low pH value, then there is a risk of corrosion. Tar coating, epoxy coating, or cathodic protection can be employed against corrosion. The speed of corrosion is 0.2-0.5 mm/year and, in design, this value can be taken as 1mm/year.

- In many cases, the pipe piles are filled with concrete after they have been driven.

- Disadvantages: high level of noise during driving, corrosion, may be damaged or deflected during driving

• Concrete:

- Precast or cast in place (CIP) piles

- Reinforcement is needed to resist the vertical load, the bending moment developed during pickup and transportation, and the bending moment due to a lateral load.

2- Effect on the soil:

- Driven piles:

- Driven piles are considered to be **<u>displacement</u>** piles. In the process of driving the pile into the ground, soil is moved radially as the pile shaft enters the ground. There may also be a component of movement of the soil in the vertical direction.
- Precast concrete piles usually of square, triangle, circle or octagonal section.
- Reinforcement is necessary within the pile to help withstand both handling and driving stresses.
- Precast piles can be prestressed and are becoming more popular than the ordinary pre cast as less reinforcement is required.

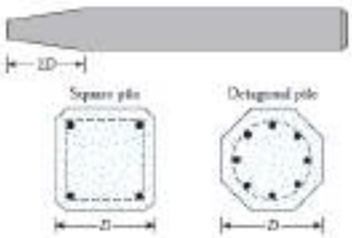


Figure 11.3 Precast piles with ordinary reinforcement

From Das (2011)

- Drilled shafts:

- Drilled shafts are generally considered to be **<u>non-displacement</u>** piles (**Replacement piles**).
- A hole is formed by boring or excavation and then it filled with concrete. A reinforcement cage is placed prior to concreting.
- The diameter of the shaft can be as high as 15 ft.
- Other commonly used names to identify drilled shafts are:
  A. Bored piles
  B. Cast in place (CIP) piles
  C. Drilled piers
  D. Caissons
- A bell can be constructed at the bottom to increase the capacity. Piles with a bell at the bottom are known as belled piers or underreamed piers.

-CIP piles can be cased or uncased. Sidewall failure and difficulty of keeping the hole open are very common problems. Steel casing would prevent the sidewalls from falling. Slurry can also be used during drilling. Basically, two types of slurry are used.

• Polymer-based fluids
• Bentonite-based fluids

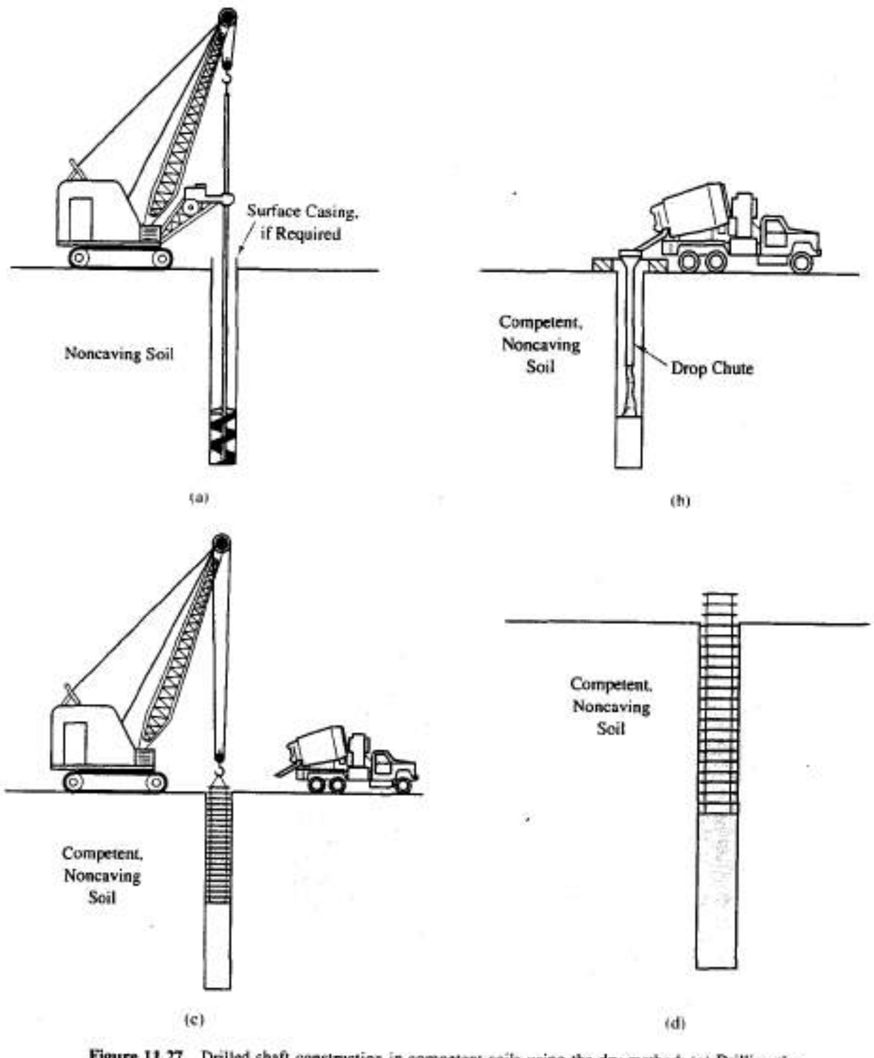
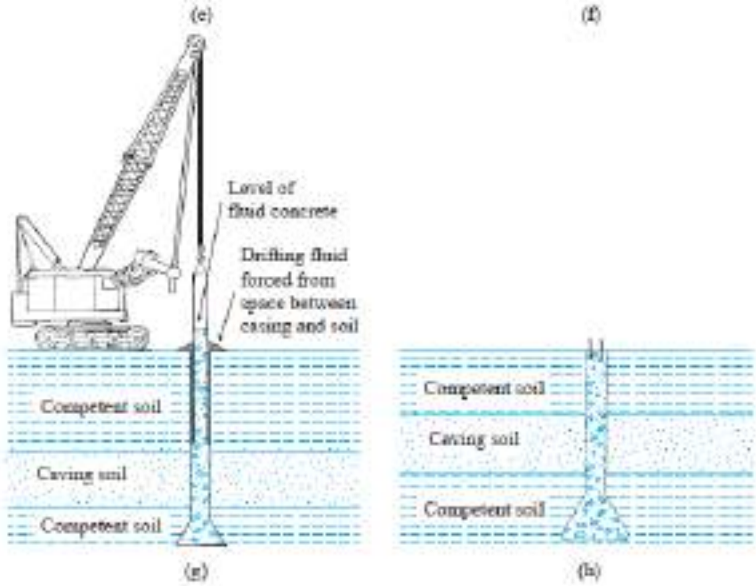


Figure 11.27 Drilled shaft construction in competent soils using the dry method: (a) Drilling the shaft; (b) Starting to place the concrete; (c) Placing the reinforcing steel cage; and (d) Finishing the concrete placement (Reese and O'Neill, 1988).

From Coduto (2001)



Belled piers (from Das, 2011)

3- Load transmission and functional behavior:

- **Point bearing piles**
  - These piles transfer their load to a firm stratum located at a considerable depth below the base of the structure and they derive most of their carrying capacity from the penetration resistance of the soil at the toe of the pile.
  - The pile behaves as an ordinary column and should be designed as such.
  - Even in weak soil a pile will not fail by buckling and this effect needs only be considered if part of the pile is unsupported, i.e. if it is in either air or water.
  - The depth of the pile is influenced by the results of the site investigation and soil tests.

- **Friction piles**
  - Carrying capacity is derived mainly from skin friction of the soil in contact with the shaft of the pile when no layer of rock or strong soil is present at a reasonable depth at a site.
  - The lengths of friction piles depend on the applied load, the pile size, and the shear strength of the soil.

Sometimes, the soil surrounding the pile may adhere to the surface of the pile and causes "Negative Skin Friction" on the pile. This, sometimes have considerable effect on the capacity of the pile. Negative skin friction is caused by the drainage of the ground water and consolidation of the soil.

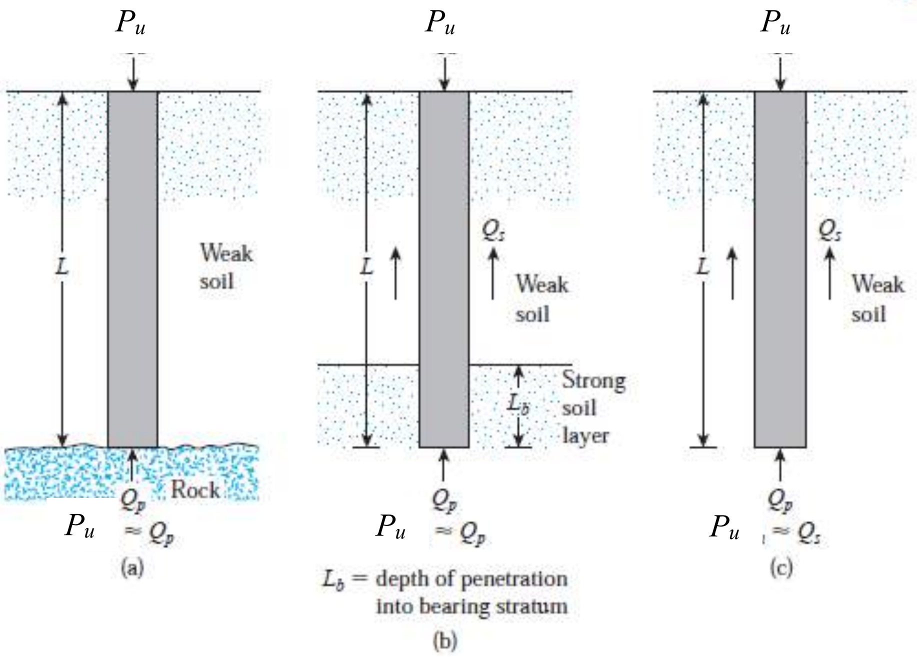


Figure 11.6 (a) and (b) Point bearing piles; (c) friction piles

From Das (2011)

$$P_u = Q_P + Q_S$$

$$P_u = q_P.A + \sum_{i=1}^{n}(f_{Si}.A_{si})$$

Q_P= load carried at the pile tip
Q_S= load carried by skin friction developed at the side of the pile (caused by shearing resistance between the soil and the pile)
q_p= unit tip resistance (point resistance)
A= area of pile tip
f_S= unit skin friction (friction or adhesion resistance). In clay, skin friction is also used to refer to adhesion.
A_s= surface area of the pile= perimeter * layer thickness =p*Hi

$$P_{all} = \frac{P_u}{FS} = \frac{q_P.A + \sum_{i=1}^{n}(f_{Si}.A_{si})}{FS}$$

P_{all} = allowable load capacity of the pile
FS= factor of safety=3

Piles in Sand

$$P_u = Q_P + Q_S$$

1- Tip (point) Resistance - Sand

A modified version of the Terzaghi bearing capacity equation is widely used for pile design. The third term or the density term in the Terzaghi bearing capacity equation is negligible in piles and hence usually ignored. The lateral earth pressure coefficient (K) is introduced to compute the skin friction of piles.

$$Q_P = q_p A$$

$$q_p = \sigma'_t Nq$$

σ'_t = effective stress at the tip of the pile
N_q = bearing factor coefficient
A = cross-sectional area of the pile at the tip

A number of methods are available for computing Nq:

- From Table 1.

Table 1 Friction angle vs. Nq

ϕ	26	28	30	31	32	33	34	35	36	37	38	39	40
N_q (for driven piles)	10	15	21	24	29	35	42	50	62	77	86	120	145
N_q (for bored piles)	5	8	10	12	14	17	21	25	30	38	43	60	72

(Source: NAVFAC DM 7.2)

- The American Petroleum Institute (API) method, 2000:

$$N_q = 40 + 60 \log N$$

$$q_{lim} = 3.4 + 38 \log N \quad \text{MPa}$$

q_{lim}= limit tip resistance ($q_p \le q_{lim}$)

If SPT data is not available, see Table 2. for Nq and q_{lim}

2- Skin Friction- Sand

$$Q_s = \sum_{i=1}^{n} (f_{Si} . A_{si})$$

Numerous techniques have been proposed to compute the skin friction in sandy soils.

2.1 API Method, 2000

$$f_s = K\sigma'_v tan\delta \leq f_{lim}$$

σ'_v = effective stress at the midpoint of the pile
K = lateral earth pressure coefficient
δ = pile skin friction angle

- $K = 0.16 + 0.15\overline{N}$ non-displacement piles
- $K = 0.70 + 0.15\overline{N}$ displacement piles
- $\delta = 20 + 8\log\overline{N}$ $f_{lim} = 67\log\overline{N}$ kPa

If SPT data is not available, see Table 2.

Table 2. API Method

Density	**Soil Type**	**δ (deg)**	**Limiting Skin Friction, f_{lim} (ksf)**	**Bearing Capacity Factor , N_q**	**Limiting End Bearing Stress, q_{lim} (ksf)**
Very loose Loose Medium	Sand Sand/Silt Silt	15	1.0 47.9 (kPa)	8	40 1916 (kPa)
Very loose Loose Medium	Sand Sand/Silt Silt	20	1.4 67.0 (kPa)	12	60 2873 (kPa)
Medium Dense	Sand Sand/Silt	25	1.7 81.4 kPa)	20	100 4788 (kPa)
Dense Very Dense	Sand Sand/Silt	30	2.0 95.8 (kPa)	40	200 9576 (kPa)
Dense Very Dense	Gravel Sand	35	2.4 114.9 (kPa)	50	250 11,970 (kPa)

K=0.8 for non-displacement piles K=1.0 for displacement piles

2.2. Meyerhof (1976):

$\beta = K\tan\delta$

$f_s = \beta\sigma'_v$

σ'_v = effective stress at the midpoint of the pile
A_p = perimeter surface area of the pile

$\beta = 0.10$ for $\phi = 33°$
$\beta = 0.20$ for $\phi = 35°$
$\beta = 0.35$ for $\phi = 37°$

3- Correlations with SPT and CPT data- Sand

SPT Data:

Tip Capacity

- Sand and Gravel

$$q_p = \frac{0.4ND_b}{B} \leq 4N$$

- Silt

$$q_p = \frac{0.3ND_b}{B} \leq 3N$$

Side Capacity

$$f_s = \frac{\bar{N}}{50} \quad \text{Displacement}$$

$$f_s = \frac{\bar{N}}{100} \quad \text{Non-Displacement}$$

CPT Data

Tip Capacity

- $q_p = \frac{q_c D_b}{10B} \leq q_{lim}$
- $q_{lim} = 0.5 N_q \tan\phi$

Side Capacity

- $f_S = 2\bar{f_c}$ or $q_c/100$ displacement
- $f_S = \bar{f_c}$ or $q_c/200$ non-displacement

Piles in Clay

$$P_u = Q_P + Q_S$$

1- Tip Resistance- Clay

$$Q_P = q_p A$$

- $q_p = 9c_u$
- $q_p = 2N_{60}$ (tsf)
- $q_p = q'_{ca}/k_c$ (tsf)

Note: c_u and N_{60} averaged 1.5 D below and 3D above pile tip

Bearing Capacity Factors, k_c

Robertson, 2013

After Bustamante and Gianeselli, 1982		*Factors k_c*	
Nature of soil	**q_c (MPa)**	***Group I***	***Group II***
Soft clay and mud	<1	0.4	0.5
Moderately compact clay	1 to 5	0.35	0.45
Silt and loose sand	≤5	0.4	0.5
Compact to stiff clay and compact silt	>5	0.45	0.55
Soft chalk	≤5	0.2	0.3
Moderately compact sand and gravel	5 to 12	0.4	0.5
Weathered to fragmented chalk	>5	0.2	0.4
Compact to very compact sand and gravel	>12	0.3	0.4

Group I: plain bored piles, mud bored piles, micro piles (grouted under low pressure), cased bored piles, hollow auger bored piles, piers, barrettes. **i.e. low displacement piles**

Group II: cast screwed piles, driven pre-cast piles, pre-stressed tubular piles, driven cast piles, jacked metal piles, micropiles (small diameter piles grouted under high pressure with diameter <250mm), driven grouted piles (low pressure grouting), driven metal piles, driven rammed piles, jacket concrete piles, high pressure grouted piles of large diameter , **i.e. high displacement piles**

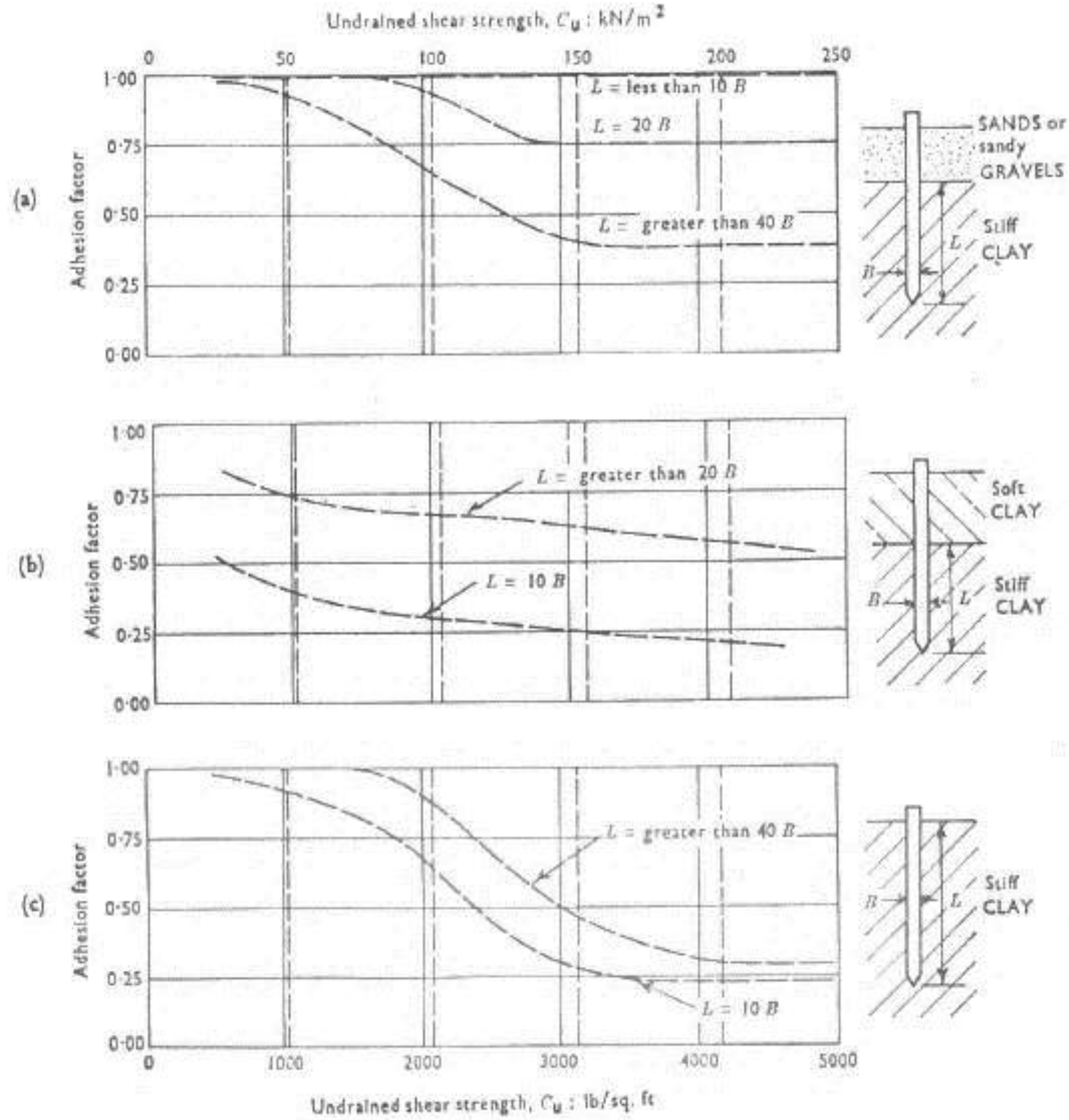
2- Skin Friction – Clay

$$Q_s = \sum_{i=1}^{n} (f_{Si}.A_{si})$$

2.1 Based on Undrained Shear Strength (Cohesion)

$$\boxed{f_s = \alpha c_u}$$

- α Driven piles



Note: 1. Curves not applicable to H or cruciform sections or to bored or driven and cast in place piles
2. Safety factor should not be less than 2·5 except for designs based on adequate loading test data

- α **Drilled shafts**

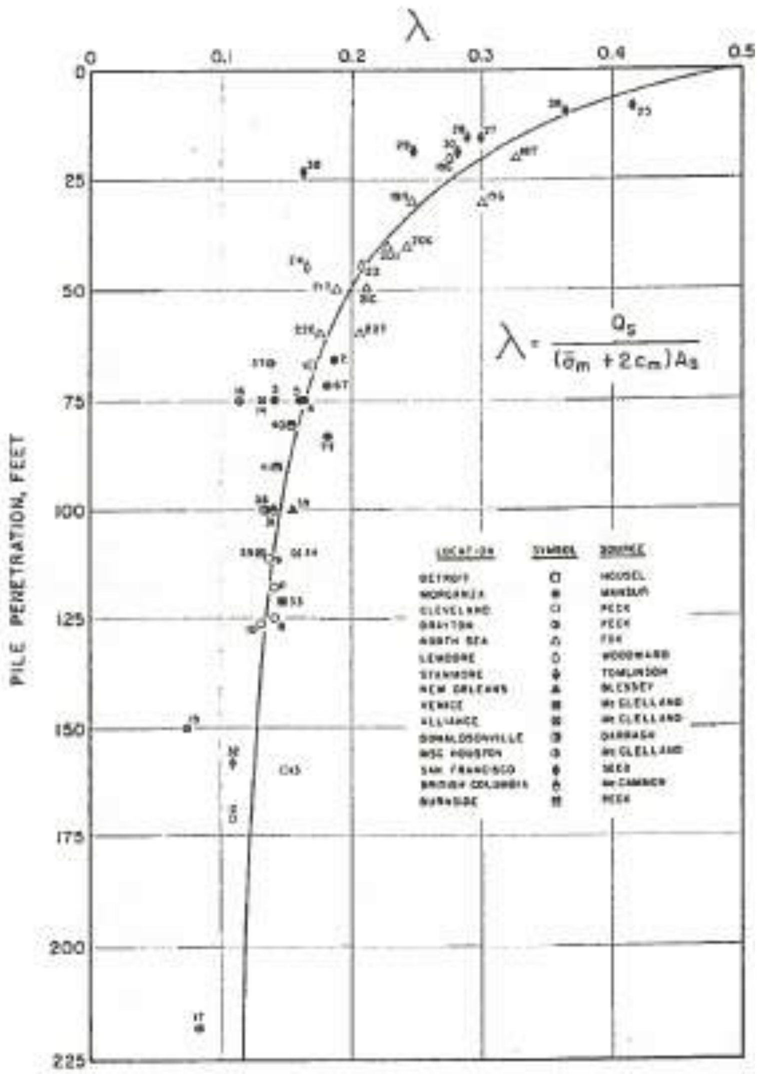
α= 0.55 (AASHTO)

2-2 Based on Both Cohesion and Effective Stress (λ Method)

- $f_s = (\sigma'_m + 2c_m)\lambda$

  σ'_m = mean effective vertical stress along the pile side

  c_m = mean undrained cohesion along pile side



3- Correlations with SPT and CPT data- Clay

-SPT

Meyerhof - 1956

- $f_s = N_{avg}/100$ (tsf) displacement piles
- $f_s = N_{avg}/200$ (tsf) non-displacement piles

- CPT

- $f_s = q_c/30_{(concrete)} = q_c/30_{(steel)}$ $q_c \leq 1MPa$
- $f_s = q_c/40_{(concrete)} = q_c/80_{(steel)}$ $1MPa < q_c \leq 5MPa$
- $f_s = q_c/60_{(concrete)} = q_c/120_{(steel)}$ $q_c > 5MPa$

Specific Considerations for Drilled Shafts:

- Unlike driven piles, drilled shafts relive the stresses in the ground and loosen the soil at the toe which may reduce the shear stress of the soil at the toe and the side friction.
- In cohesive soils, the skin friction within 5 ft (1.5 m) of the ground surface should be neglected because of clay shrinkage and foundation lateral movement.

Upward load capacity

$P_{all} = (W_p + f_s .A_p)/ FS$

FS>= 5 because f_s uplift is 70 to 85 percent of f_s downward.

Pile Groups

- Typically, piles are installed in a group and provided with a pile cap. The column is placed on the pile cap so that the column load is equally distributed among the individual piles in the group.

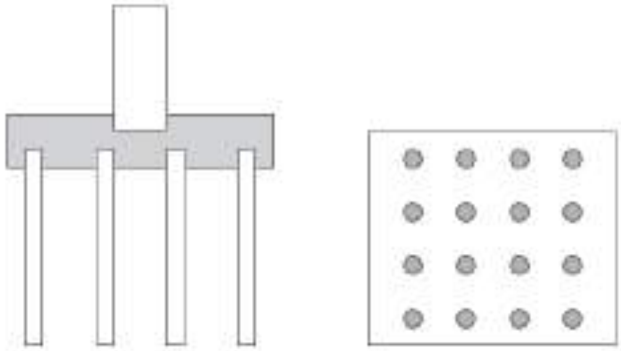


Figure 8.1 Pile groups

- In clay, when piles are driven, the soil surrounding the pile is disturbed. Disturbed soil has less strength than undisturbed soil. Some of the piles in the group are installed in partially disturbed soil causing them to have less capacity than others.

- The capacity of a pile group is obtained by using an efficiency factor.

Pile group capacity = Efficiency of the pile group X Single pile capacity X Number of piles

Example: If the pile group contains 16 piles and capacity of a single pile is 30 tons and the group efficiency is found to be 0.9, the group capacity is 432 tons.

- However, when driving piles in sandy soils, surrounding soil will be compacted. Compacted soil tends to increase the skin friction of piles. Pile group placed in sandy soils may have a larger than one group efficiency.

- Piles that mainly rely on end bearing capacity may not be affected by other piles in the group.

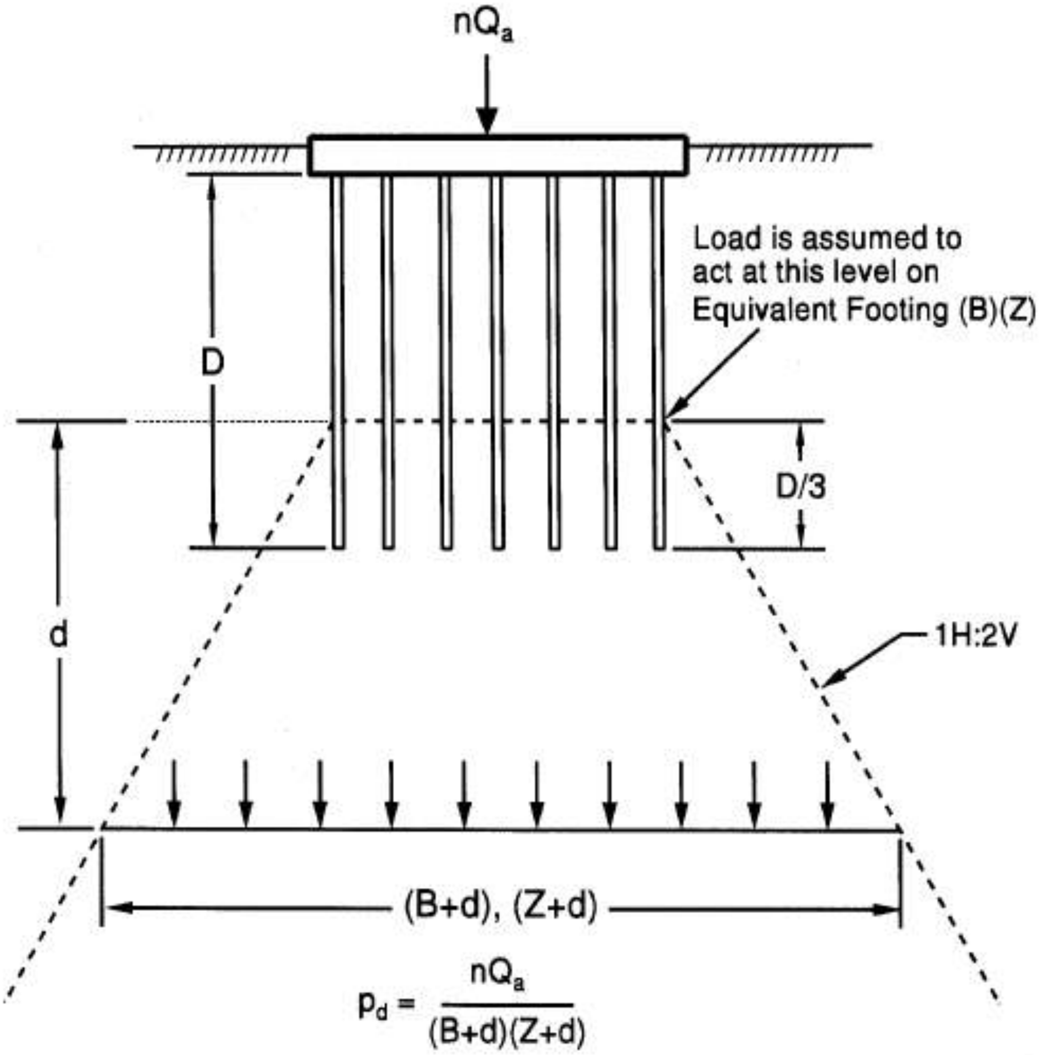
following tables use AASHTO guidelines for pile group effi-
in cohesive soils.

Pile Group Efficiency for Clayey Soils

Pile Spacing (center to center)	Group Efficiency
3 D	0.67
4 D	0.78
5 D	0.89
6 D or more	1.00

D = Diameter of piles

Equivalent Footing to calculate settlement of pile groups



Note: Pile Group has Plan Dimension of B and Z