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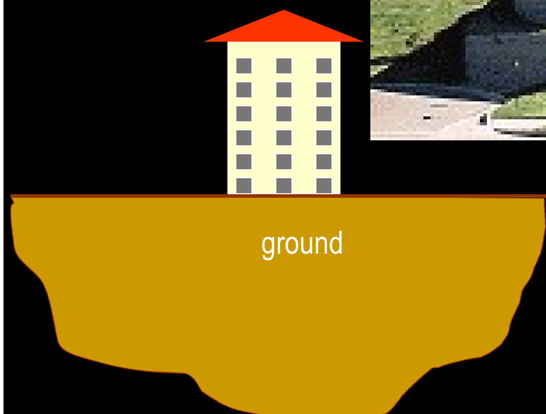
هندسة أساسات

د.عمر حتاملة

Foundation Engineering and Design



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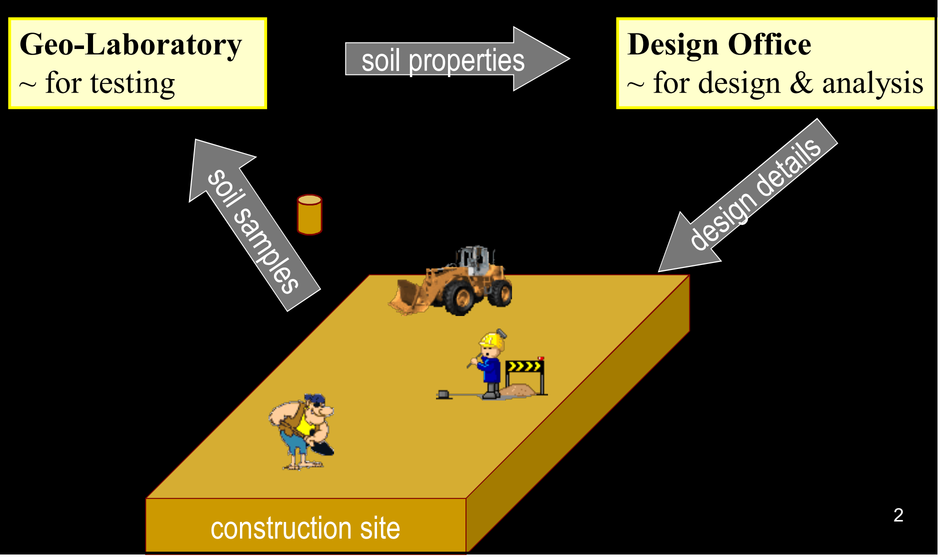
Typical Geotechnical Project



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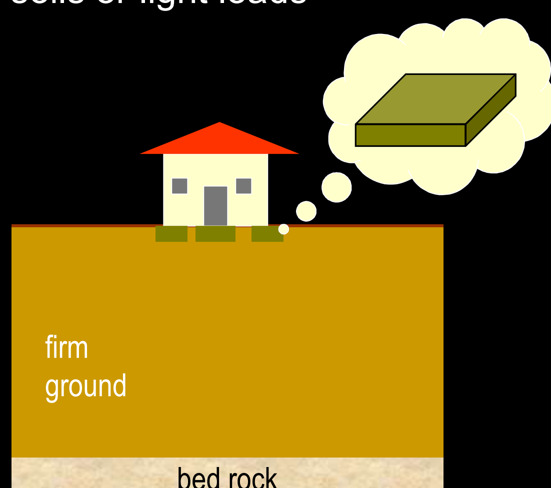
Geotechnical Applications

Shallow Foundations

~ for transferring building loads to underlying ground

~ mostly for firm soils or light loads

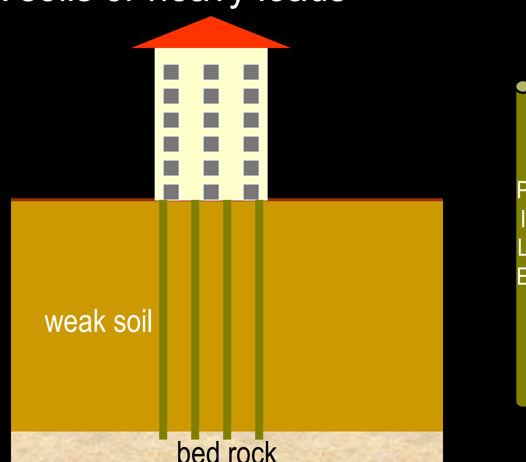


Shallow Foundations

Deep Foundations

~ for transferring building loads to underlying ground

~ mostly for weak soils or heavy loads



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Pile driving rig – Ross River Dam

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Pile driving rig – Ross River Dam

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Deep Foundations

Driven timber piles, Pacific Highway

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Pier Foundations for Bridges

Millau Viaduct in France (2005)

- Cable-stayed bridge
- Supported on 7 piers, 342 m apart
- Longest pier (336) in the world

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Pier Foundations for Bridges

Millau Viaduct in France (2005)

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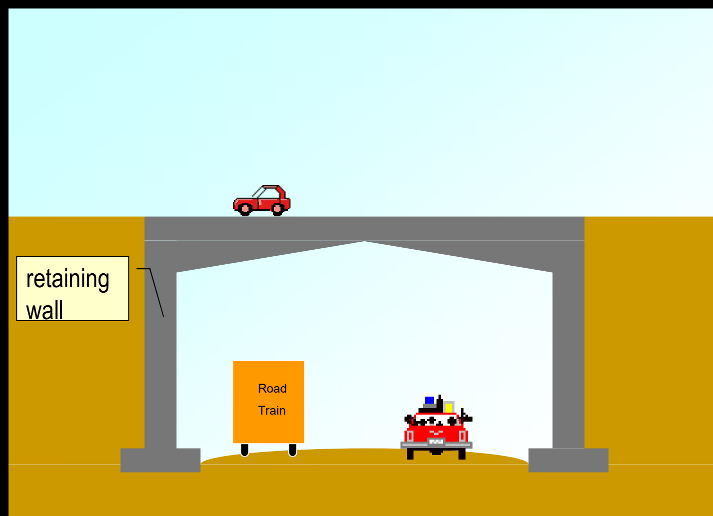
The new link between Copenhagen, (Denmark) and Malmo (Sweden) includes the causeway and its tunnel seen in this photograph, plus one of the world's longest cable-stayed bridges (not seen here).

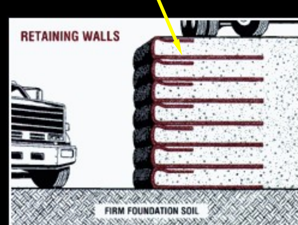
Retaining Walls

~ for retaining soils from spreading laterally



Reinforced Earth Walls

~ using geofabrics to strengthen the soil



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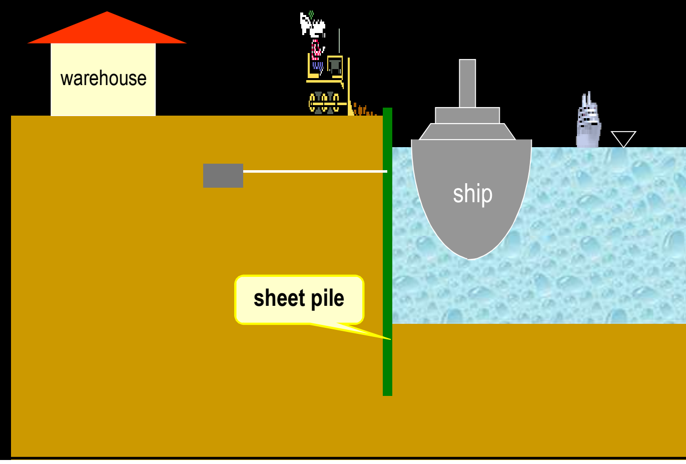
Sheet Piles

~ sheets of interlocking-steel or timber driven into the ground, forming a continuous sheet



Sheet Piles

~ resist lateral earth pressures

~ used in excavations, waterfront structures, ..

Sheet Piles

~ used in temporary works

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Cofferdam

~ sheet pile walls enclosing an area, to prevent water seeping in

18

Cofferdam

~ sheet pile walls enclosing an area, to prevent water seeping in

19

Shoring

propping and supporting the exposed walls to resist lateral earth pressures

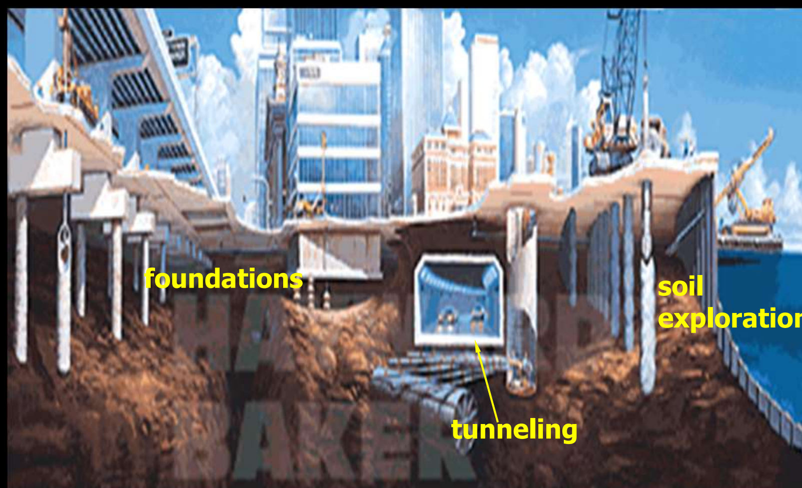
20

Excavations

Typical Safety Factors

Type of Design	Safety Factor	Probability of Failure
Earthworks	1.3-1.5	1/500
Retaining structures	1.5-2.0	1/1500
Foundations	2.0-3.0	1/5000

Some Civil Engineering marvels ….



… buried right under your feet.

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Hall of Fame

Great Contributors to the Developments in Geotechnical Engineering

Karl Terzaghi
1883-1963

C.A.Coulomb
1736-1806

M. Rankine
1820-1872

Challenges

Geotechnical Engineering Landmarks

Leaning Tower of Pisa

Our blunders become monuments!

FOUNDATION ENGINEERING

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Importance & Purpose

- All engineered construction resting on the earth must be carried by some kind of interfacing element called *a foundation*
- Foundation is the part of an engineered system that transmits to, and into, the underlying soil or rock the loads supported by the foundation and its self-weight
- The term *superstructure* is commonly used to describe the engineered part of the system bringing load to the foundation, or substructure. The term *superstructure* has particular significance for buildings and bridges; however, foundations also may carry only machinery, support industrial equipment (pipes, towers, tanks), act as sign bases, and the like.
- The Foundation as that part of the engineered system that interfaces the load-carrying components to the ground.
  - It is evident on the basis of this definition that a foundation is the most important part of the engineering system.

Minimum Required For Designing A Foundation

1. Locate the site and the position of load. A rough estimate of the foundation load(s) is usually provided by the client or made in-house. Depending on the site or load system complexity, a literature survey may be started to see how others have approached similar problems.
2. Physically inspect the site for any geological or other evidence that may indicate a potential design problem that will have to be taken into account when making the design or giving a design recommendation. Supplement this inspection with any previously obtained soil data.
3. Establish the field exploration program and, on the basis of discovery (or what is found in the initial phase), set up the necessary supplemental field testing and any laboratory test program.

Minimum Required For Designing A Foundation Cont’

4. Determine the necessary soil design parameters based on integration of test data, scientific principles, and engineering judgment. Simple or complex computer analyses may be involved.

5. For complex problems, compare the recommended data with published literature or engage another geotechnical consultant to give an outside perspective to the results.

6. Design the foundation using the soil parameters from step 4. The foundation should be economical and be able to be built by the available construction personnel. Take into account practical construction tolerances and local construction practices. Interact closely with all concerned (client, engineers, architect, contractor) so that the substructure system is not excessively overdesigned and risk is kept within acceptable levels. A computer may be used extensively (or not at all) in this step.

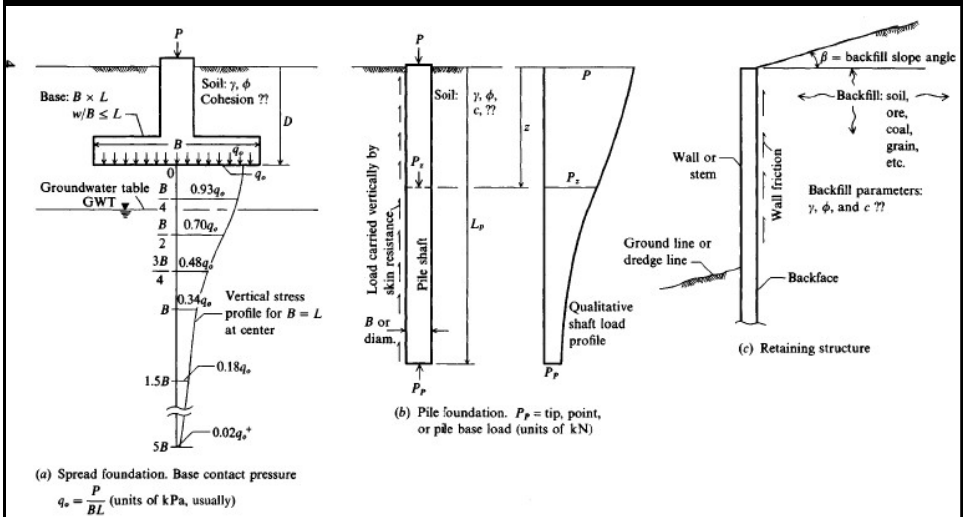
FOUNDATIONS: CLASSIFICATIONS

- Foundations may be classified based on where the load is carried by the ground, producing:

Shallow foundations—termed bases, footings, spread footings, or mats. The depth is generally $D/B < 1$ but may be somewhat more.

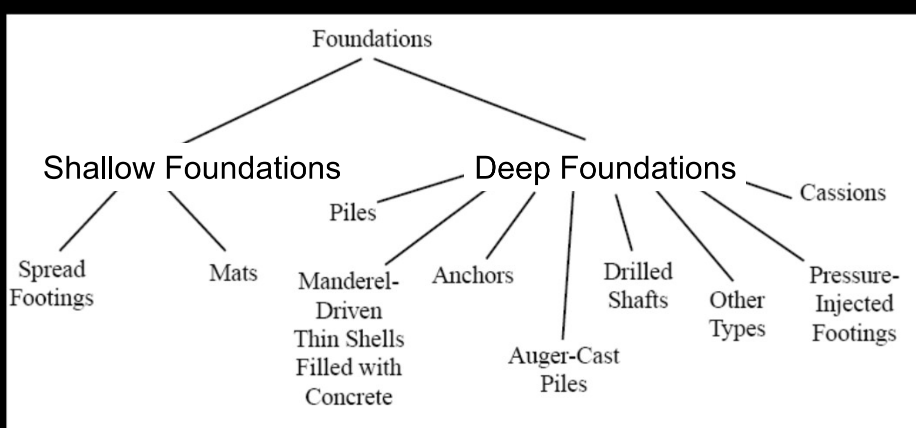
Deep foundations—piles, drilled piers, or drilled caissons. $Lp/B > 4+$

Foundations: Classifications



Definition of select terms used in foundation engineering

Other foundations Type



GENERAL REQUIREMENTS

- Foundation elements must be proportioned both to interface with the soil at a safe stress level and to limit settlements to an acceptable amount
- Excessive settlement problems are fairly common and somewhat concealed
- In summary, a proper design requires the following:
  1. Determining the building purpose, probable service-life loading, type of framing, soil profile, construction methods, and construction costs
  2. Determining the client/owner's needs
  3. Making the design, but ensuring that it does not excessively degrade the environment, and provides a margin of safety that produces a tolerable risk level to all parties: the public, the owner, and the engineer

ADDITIONAL CONSIDERATIONS
That May Have To Be Taken Into Account At Specific Sites

1. Depth must be adequate to avoid lateral squeezing of material from beneath the foundation for footings and mats. Similarly, excavation for the foundation must take into account that this can happen to existing building footings on adjacent sites and requires that suitable precautions be taken. The number of settlement cracks that are found by owners of existing buildings when excavations for adjacent structures begin is truly amazing.

2. Depth of foundation must be below the zone of seasonal volume changes caused by freezing, thawing, and plant growth. Most local building codes will contain minimum depth requirements.

3. The foundation scheme may have to consider expansive soil conditions. Here the building tends to capture upward-migrating soil water vapor, which condenses and saturates the soil in the interior zone, even as normal perimeter evaporation takes place. The soil in a distressingly large number of geographic areas tends to swell in the presence of substantial moisture and carry the foundation up with it.

ADDITIONAL CONSIDERATIONS
That May Have To Be Taken
Into Account At Specific Sites
Cont'

4. In addition to compressive strength considerations, the foundation system must be safe against overturning, sliding, and any uplift (flotation).

5. System must be protected against corrosion or deterioration due to harmful materials present in the soil. Safety is a particular concern in reclaiming sanitary landfills but has application for marine and other situations where chemical agents that are present can corrode metal pilings, destroy wood sheeting/piling, cause adverse reactions with Portland cement in concrete footings or piles, and so forth.

6. Foundation system should be adequate to sustain some later changes in site or construction geometry and be easily modified should changes in the superstructure and loading become necessary.

ADDITIONAL CONSIDERATIONS
That May Have To Be Taken
Into Account At Specific Sites
Cont'

4. The foundation should be buildable with available construction personnel. For one-of-akind projects there may be no previous experience. In this case, it is necessary that all concerned parties carefully work together to achieve the desired result.

- The foundation and site development must meet local environmental standards, including determining if the building is or has the potential for being contaminated with hazardous materials from ground contact (for example, radon or methane gas). Adequate air circulation and ventilation within the building are the responsibility of the mechanical engineering group of the design team.

Selection Of Type

Foundation type	Use	Applicable soil conditions
Shallow foundations (generally $D/B \leq 1$)		
Spread footings, wall footings	Individual columns, walls	Any conditions where bearing capacity is adequate for applied load. May use on a single stratum; firm layer over soft layer or soft layer over firm layer. Check settlements from any source.
Combined footings	Two to four columns on footing and/or space is limited	Same as for spread footings above.
Mat foundations	Several rows of parallel columns; heavy column loads; use to reduce differential settlements	Soil bearing capacity is generally less than for spread footings, and over half the plan area would be covered by spread footings. Check settlements from any source.

Selection Of Type

Deep foundations (generally $L_p/B \geq 4^+$)		
Floating pile	In groups of 2^+ supporting a cap that interfaces with column(s)	Surface and near-surface soils have low bearing capacity and competent soil is at great depth. Sufficient skin resistance can be developed by soil-to-pile perimeter to carry anticipated loads.
Bearing pile	Same as for floating pile	Surface and near-surface soils not relied on for skin resistance; competent soil for point load is at a practical depth (8–20 m).
Drilled piers or caissons	Same as for piles; use fewer; For large column loads	Same as for piles. May be floating or point-bearing (or combination). Depends on depth to competent bearing stratum.

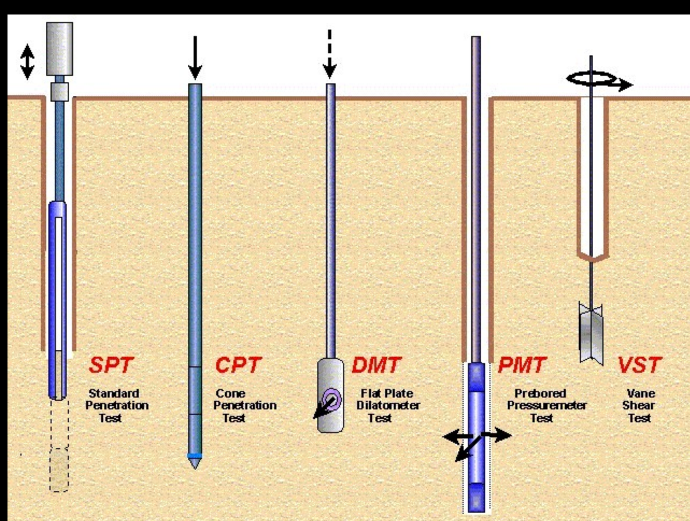
Selection Of Type

Retaining structures		
Retaining walls, bridge abutments	Permanent material retention	Any type of soil but a specified zone (Chaps. 11, 12) in backfill is usually of controlled fill.
Sheeting structures (sheet pile, wood sheeting, etc.)	Temporary or permanent for excavations, marine cofferdams for river work	Retain any soil or water. Backfill for waterfront and cofferdam systems is usually granular for greater drainage.

- Where the groundwater table (GWT) is present, it is common to lower it below the construction zone either permanently or for the duration of the construction work.

- If the GWT later rises above the footing level, the footing will be subject to uplift or flotation, which would have to be taken into account.

Soil Testing



Variety of Field Testing Devices

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Exploration, Sampling, and In Situ Soil Measurements

Dr. Omar Al-Hattamleh

outlines

- **Purpose of Soil Exploration**
- **Planning The Exploration Program**
  - **Collection of primary data**
  - **Reconnaissance of the area**
  - **Site investigation**
    - **Methods of exploration**
    - **Soil boring**
    - **Soil sampling**
    - **Soil testing**
    - **Rock sampling**
    - **Groundwater table (gwt) location**
    - **Number and depth of borings**
    - **Approximate spacing of boreholes (number)**
  - **Example**

Purpose of Soil Exploration

- Information to determine the type of foundation required (shallow or deep).
- Information to allow the geotechnical consultant to make a recommendation on the allowable load capacity of the foundation.
- Sufficient data/laboratory tests to make settlement predictions.
- Location of the groundwater table (or determination of whether it is in the construction zone). For certain projects, groundwater table fluctuations may be required. These can require installation of piezometers and monitoring of the water level in them over a period of time.

Purpose of Soil Exploration Cont’

- Information so that the identification and solution of construction problems (sheeting and dewatering or rock excavation) can be made.
- Identification of potential problems (settlements, existing damage, etc.) concerning adjacent property.
- Identification of environmental problems and their solution.

Planning The Exploration Program

- The subsurface exploration program in general comprises of three steps
  - Collection of primary data
  - Reconnaissance
  - Site Investigation

Collection of primary data

- *Assembly of all available information* on:
  - dimensions,
  - column spacing,
  - type and use of the structure,
  - basement requirements,
  - any special architectural considerations of the proposed building,
  - and tentative location on the proposed site.
- Foundation regulations in the local building code should be consulted for any special requirements.

Reconnaissance of the area

- This may be in the form of:
  - a field trip to the site, which can reveal information on the type and behavior of adjacent structures such as
    - cracks,
    - noticeable sags,
    - and possibly sticking doors and windows.
- The type of local existing structures may influence to a considerable extent the exploration program and the best type of foundation for the proposed adjacent structure.
- Since nearby existing structures must be maintained in their "as is" condition,
  - excavations or construction vibrations will have to be carefully controlled, and this can have considerable influence on the "type" of foundation that can be used.

Reconnaissance of the area Cont'

- Study of the various sources of information available, some of which include the following:
  - *Geological maps.*
  - *Agronomy maps.*
  - *Aerial photographs.* Investigator may require special training to interpret soil data, but the nonspecialist cannot easily recognize terrain features. *Water and/or oil well logs.*
  - *Hydrological data.* Data collected on streamflow data, tide elevations, and flood levels.
  - *Soil manuals by state departments of transportation.*
  - *State (or local) university publications.* These are usually engineering experiment station publications.

Site Investigation

- *A preliminary site investigation.* In this phase a few borings (one to about four) are made or a test pit is opened to establish in a general manner the stratification, types of soil to be expected, and possibly the location of the groundwater table. If the initial borings indicate that the upper soil is loose or highly compressible, one or more borings should be taken to rock or competent strata. This amount of exploration **is usually the extent of the site investigation for small structures**.

- *A detailed site investigation.* Where the preliminary site investigation has established the feasibility and overall project economics, a more detailed exploration program is undertaken. The preliminary borings and data are used as a basis for locating additional borings, which should be confirmatory in nature, and determining the additional samples required.

Methods Of Exploration

- The most widely used method of subsurface investigation for compact sites as well as for most extended sites is boring holes into the ground, from which samples may be collected for either visual inspection or laboratory testing.
- **The several exploration methods for sample recovery**
  - **Disturbed samples taken**
  - **Undisturbed samples taken**

Disturbed samples taken

Disturbed samples taken

Method	Depths	Applicability
Auger boring†	Depends on equipment and time available, practical depths being up to about 35 m	All soils. Some difficulty may be encountered in gravelly soils. Rock requires special bits, and wash boring is not applicable. *Penetration testing* is used in conjunction with these methods, and disturbed samples are recovered in the split spoon. Penetration counts are usually taken at 1- to 1.5 m increments of depth
Rotary drilling Wash boring Percussion drilling	Depends on equipment, most equipment can drill to depths of 70 m or more	
Test pits and open cuts	As required, usually less than 6 m; use power equipment	All soils

† Most common method currently used.

Undisturbed samples taken

Undisturbed samples taken

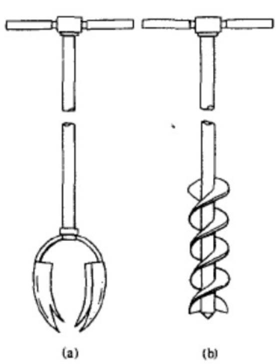
Auger drilling, rotary drilling, percussion drilling, wash boring	Depends on equipment, as for disturbed sample recovery	Thin-walled tube samplers and various piston samplers are used to recover samples from holes advanced by these methods. Commonly, samples of 50- to 100-mm diameter can be recovered
Test pits	Same as for disturbed samples	Hand-trimmed samples. Careful trimming of sample should yield the least sample disturbance of any method

Soil Boring

- Exploratory holes into the soil may be made by
  - hand tools,
  - but more commonly truck- or trailer-mounted power tools are used

Hand Tools For Soil Exploration,

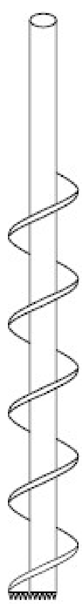
- Hand augers

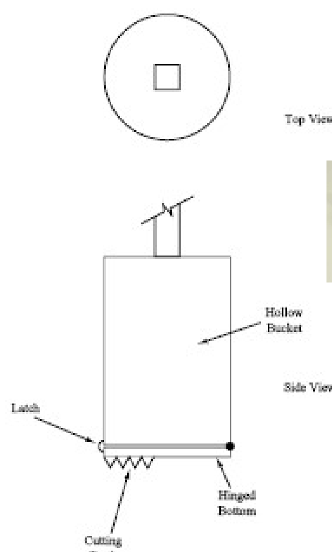


Hand tools:
(a) posthole auger;
(b) helical auger

Gasoline-engine-powered Hand Auger

Gasoline/ electric-engine-powered hand auger with additional auger flights in the foreground together with hand-driven sample tube.





Flight Auger

Bucket Auger.

Mounted Power Drills

Go To Movie For Better Quality

Soil Sampling

- The most important engineering properties for foundation design are strength, compressibility, and permeability.
  - Needs undisturbed sample
    1) The sample is always unloaded from the in situ confining pressures, with some unknown resulting expansion. Lateral expansion occurs into the sides of the borehole, so in situ tests using the hole diameter as a reference are "disturbed" an unknown amount. This is the reason K_0 field tests are so difficult.
    2) Samples collected from other than test pits are disturbed by volume displacement of the tube or other collection device. The presence of gravel greatly aggravates sample disturbance.
    3) Sample friction on the sides of the collection device tends to compress the sample during recovery. Most sample tubes are (or should be) swaged so that the cutting edge is slightly smaller than the inside tube diameter to reduce the side friction.

Soil Sampling Continued

4. There are unknown changes in water content depending on recovery method and the presence or absence of water in the ground or borehole.
5. Loss of hydrostatic pressure may cause gas bubble voids to form in the sample.
6. Handling and transporting a sample from the site to the laboratory and transferring the sample from sampler to testing machine disturb the sample more or less by definition.
7. The quality or attitude of drilling crew, laboratory technicians, and the supervising engineer may be poor.
8. On very hot or cold days, samples may dehydrate or freeze if not protected on-site. Furthermore, worker attitudes may deteriorate in temperature extremes.

sample disturbance

- Sample disturbance depends on factors such as
  - rate of penetration,
  - whether the cutting force is obtained by pushing or driving,
  - and presence of gravel,
  - it also depends on the ratio of the volume of soil displaced to the volume of collected sample, expressed as an *area ratio*

$$A_r = \frac{D_o^2 - D_i^2}{D_i^2} \times 100$$

where D_0 *— outside diameter of tube*
D_i *= inside diameter of cutting edge of tube*

Well-designed sample tubes should have an area ratio of less than about 10 percent.

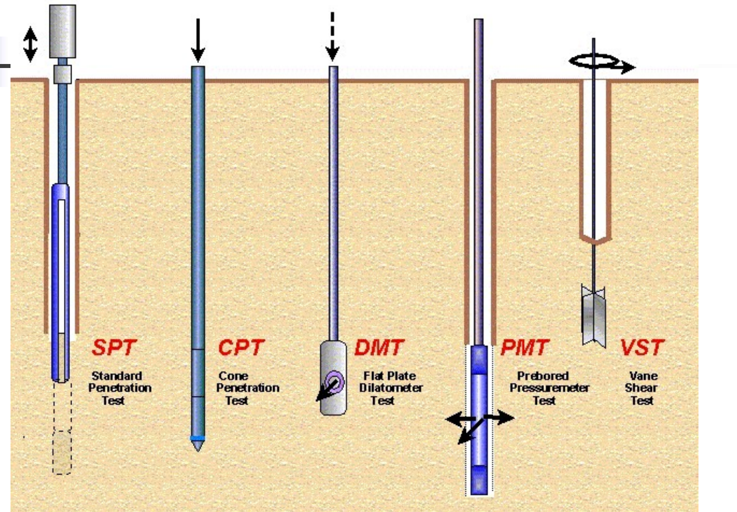
sample disturbance

- Another term used in estimating the degree of disturbance of a cohesive or rock core sample is the recovery ratio L_r

$$L_r = \frac{\text{Actual length of recovered sample}}{\text{Theoretical length of recovered sample}}$$

A recovery ratio of 1 (recovered length of the sample = the length sampler was forced into the stratum) indicates that, theoretically, the sample did not become compressed from friction on the tube. A recovery ratio greater than 1.0 would indicate a loosening of the sample from rearrangement of stones, roots, removal of preload, or other factors.

Soil Testing



Variety of Field Testing Devices

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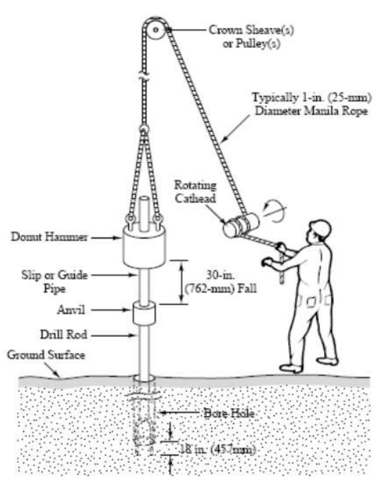
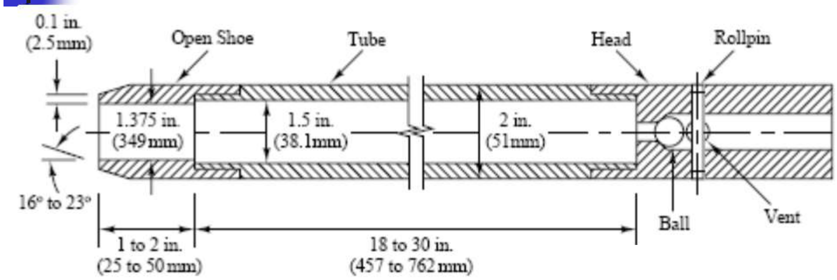
Standard Penetration Test

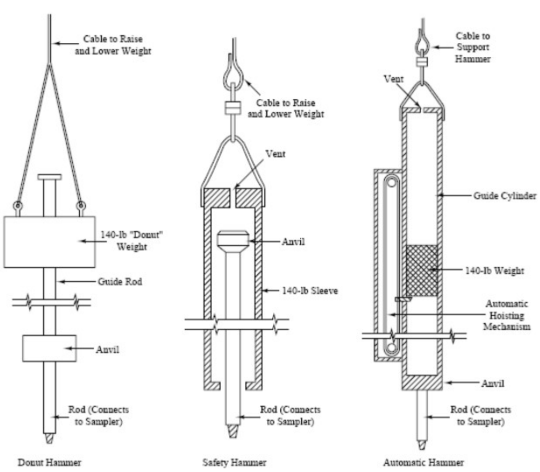


Standard Penetration Test

SPT sampler (Adapted from ASTM D1586)



Types of SPT hammers



SPT discrepancies (correction)

- Equipment from different manufacturers. A large variety of drilling rigs are in current use; however, the rotary auger with the safety hammer of is the most common in North American practice (**type of hammer**).
- **Drive hammer configurations**. The anvil also seems to have some influence on the amount of energy input to the sampler.
- Whether a liner is used inside the split barrel sampler. Side friction increases the driving resistance (and AO and is less without the liner. It is common practice not to use a liner. Also it would appear that *N* values should be larger for soils with OCR > 1 (and larger relative density *Dr)* than for normally consolidated soils. borehole size)
- **Overburden pressure**. Soils of the same density will give smaller TV values if *p'o* is smaller (as near the ground surface). Oversize boreholes on the order of 150 to 200 mm will also reduce N unless a rotary hollow-stem auger is used with the auger left in close contact with the soil in the hole bottom. Degree of cementation may also be significant in giving higher N counts in cemented zones that may have little overburden pressure.
- **Length of drill rod**.

Correction SPT, N_{60}

$$N_{60} = \frac{N\eta_H\eta_B\eta_S\eta_R}{60}$$

N60= SPT values corrected for field procedure

η_H= hammer efficiency (table 2.2 Das)

η_B= borehole diameter correction (table 2.2 Das)

η_S= Sampler correction (table 2.2 Das)

η_R= Rod length correction (table 2.2 Das)

N= Measured SPT N values (table 2.2 Das)

Correction Tables

Table 2.3 Variations of η_H, η_B, η_S, and η_R [Eq. (2.8)]

1. Variation of η_H

Country	Hammer type	Hammer release	η_H (%)
Japan	Donut	Free fall	78
	Donut	Rope and pulley	67
United States	Safety	Rope and pulley	60
	Donut	Rope and pulley	45
Argentina	Donut	Rope and pulley	45
China	Donut	Free fall	60
	Donut	Rope and pulley	50

2. Variation of η_B

Diameter		
mm	in.	η_B
60–120	2.4–4.7	1
150	6	1.05
200	8	1.15

3. Variation of η_S

Variable	η_S
Standard sampler	1.0
With liner for dense sand and clay	0.8
With liner for loose sand	0.9

4. Variation of η_R

Rod length		
m	ft	η_R
>10	>30	1.0
6–10	20–30	0.95
4–6	12–20	0.85
0–4	0–12	0.75

Correction SPT, $(N_1)_{60}$

$$(N_1)_{60} = N_{60}\sqrt{\frac{2000\text{lb}/\text{ft}^2}{\sigma'_o}}$$

$$(N_1)_{60} = C_N N_{60} = \left[\frac{1}{\frac{\sigma'_o}{P_o}}\right]^{0.5} = N_{60}\sqrt{\frac{100\text{kPa}}{\sigma'o}}$$

N_{60}: N corrected for field procedure

$(N_1)_{60}$: N corrected for field procedure and overburden pressure

Consistency of clay

Table 2.4 Approximate Correlation between CI, N_{60}, and q_u

Standard penetration number, N_{60}	Consistency	CI	Unconfined compression strength, q_u (kN/m²)	(lb/ft²)
<2	Very soft	<0.5	<25	500
2–8	Soft to medium	0.5–0.75	25–80	500–1700
8–15	Stiff	0.75–1.0	80–150	1700–3100
15–30	Very stiff	1.0–1.5	150–400	3100–8400
>30	Hard	>1.5	>400	8400

$$CI = \frac{LL - w}{LL - PL}$$

the consistency index (CI)

w = natural moisture content

LL = liquid limit

PL = plastic limit

$$\frac{c_u}{p_a} = 0.29 N_{60}^{0.72}$$

p_a = atmospheric pressure

($\approx$ 100 kN/m²; $\approx$ 2000 lb/in²)

The overconsolidation ratio, OCR,

$$OCR = 0.193\left(\frac{N_{60}}{\sigma'_o}\right)^{0.689}$$

σ'_o = effective vertical stress in MN/m²

Consistency of saturated cohesive soils

$$C_u = KN_{60}$$

K= (3.5-6.5 kN/m²)

Consistency of saturated cohesive soils*

Consistency			(N_{60})	q_u, kPa	Remarks
Very soft	NC	Young clay	0–2	<25	Squishes between fingers when squeezed
Soft			3–5	25–50	Very easily deformed by squeezing
Medium			6–9	50–100	? ?
Stiff	Increasing OCR	Aged/cemented	10–16	100–200	Hard to deform by hand squeezing
Very stiff			17–30	200–400	Very hard to deform by hand squeezing
Hard			>30	>400	Nearly impossible to deform by hand

* Blow counts and OCR division are for a guide—in clay "exceptions to the rule" are very common.

Water Table correction (drained vs undrained)

$$N' = N + \tfrac{1}{2}(N - 15) \text{ for } N > 15;\ N' = N \text{ for } N \le 15.$$

SPT Correlations

Relative Density and Internal Friction Angle

$$D_r(\%) = \left[\frac{N_{60}\left(0.23 + \frac{0.06}{D_{50}}\right)^{1.7}}{9}\left(\frac{1}{\frac{\sigma'_o}{p_a}}\right)\right]^{0.5}(100)$$

$$\phi'(\text{deg}) = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$$

$$\phi = \tan^{-1}\left[\frac{(N_{60})}{12.2 + 20.3(\frac{\sigma'_o}{pa})}\right]^{0.34}$$

$$\phi' = \sqrt{20(N_1)_{60}} + 20$$

Table 2.5 Relation between the Corrected $(N_1)_{60}$ Values and the Relative Density in Sands

Standard penetration number, $(N_1)_{60}$	Approximate relative density, D_r (%)
0–5	0–5
5–10	5–30
10–30	30–60
30–50	60–95

SPT Correlations

Empirical values for ϕ Dr and unit weight of granular soils based on the SPT at about 6 m depth and normally consolidated

$\phi = 28° + 15° D_r\ (\pm 2°)]$

Description		Very loose	Loose	Medium	Dense	Very dense
Relative density D_r		0	0.15	0.35	0.65	0.85
SPT $(N_1)_{60}$	fine	1–2	3–6	7–15	16–30	?
	medium	2–3	4–7	8–20	21–40	> 40
	coarse	3–6	5–9	10–25	26–45	> 45
ϕ:	fine	26–28	28–30	30–34	33–38	
	medium	27–28	30–32	32–36	36–42	< 50
	coarse	28–30	30–34	33–40	40–50	
γ_{wet}, kN/m^3		11–16*	14–18	17–20	17–22	20–23

* Excavated soil or material dumped from a truck has a unit weight of 11 to 14 kN/m^3 and must be quite dense to weigh much over 21 kN/m^3. No existing soil has a D_r = 0.00 nor a value of 1.00. Common ranges are from 0.3 to 0.7.

The modulus of elasticity of granular soils (Es)

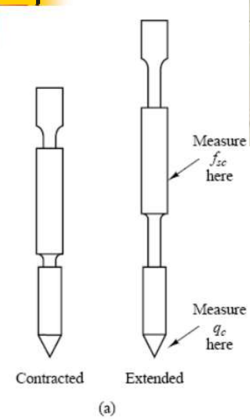
$$\frac{E_s}{p_a} = \alpha N_{60}$$

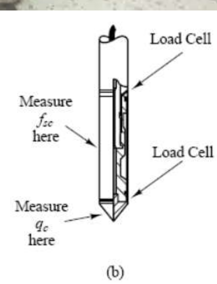
where

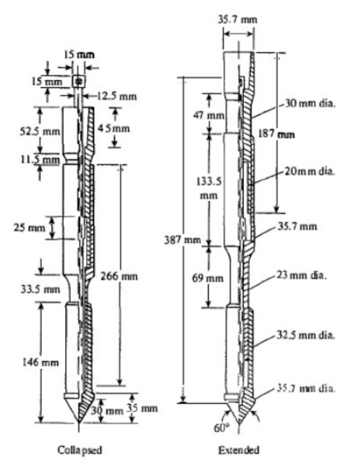
$p_a =$ atmospheric pressure (same unit as E_s)

$$\alpha = \begin{cases} 5 \text{ for sands with fines} \\ 10 \text{ for clean normally consolidated sand} \\ 15 \text{ for clean overconsolidated sand} \end{cases}$$

Cone Penetration Test



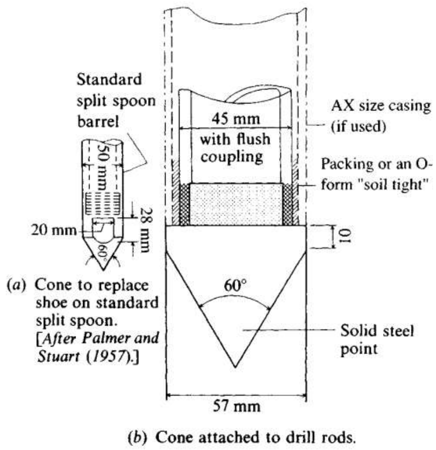


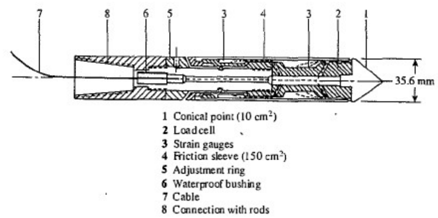


Types of cones (Most common): (a) A mechanical cone (also known as a Begemann Cone); and (b) An electric cone

[see Figure 2.12 & 2.13 page 69 Das]

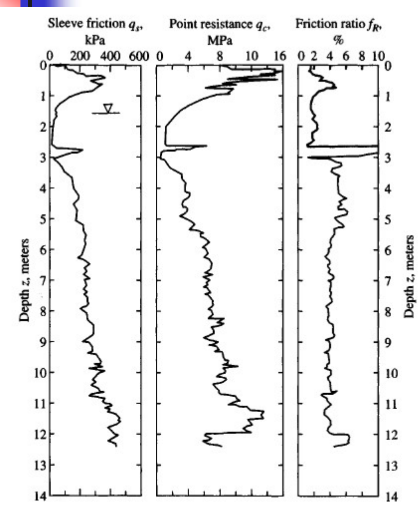
CPT





- Driving rate 10 to 20 mm/s
- The tip (or cone) usually has a projected cross-sectional area of 10 cm^2,
- Friction sleeve area 150 cm^2

Typical measurements



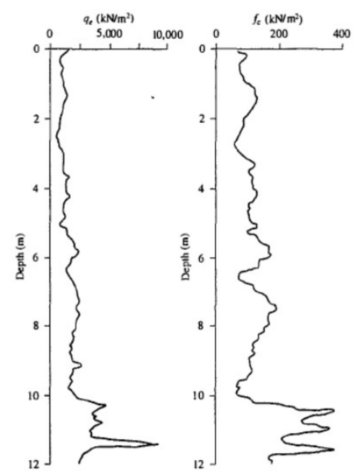
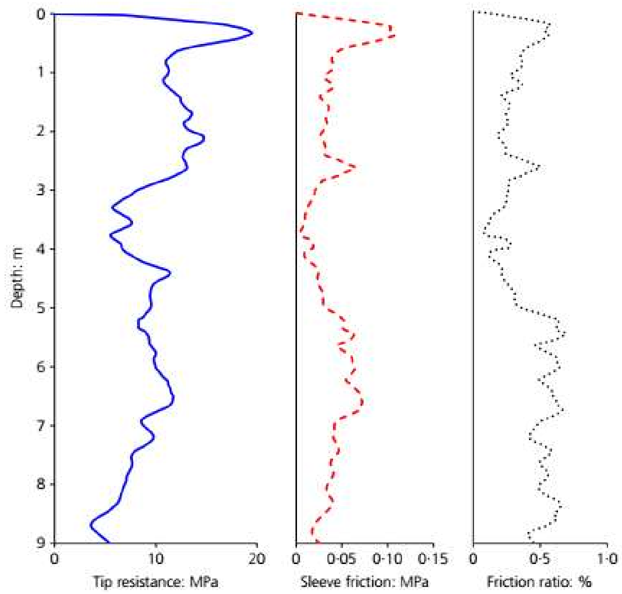
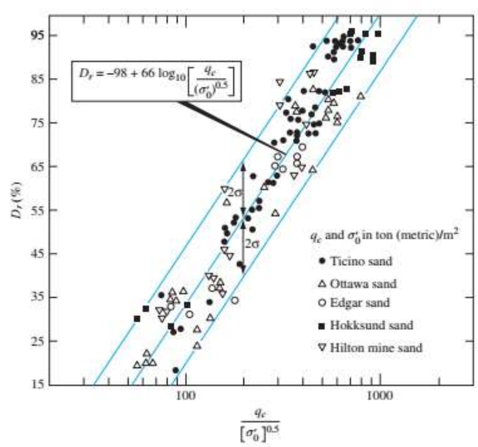


Figure 2.20 Cone penetrometer test with friction measurement



Correlation

With Relative density as



$$D_r(\%) = 68\left[\log\left(\frac{q_c}{\sqrt{p_a \cdot \sigma_0'}}\right) - 1\right]$$

$$Dr = \sqrt{\left[\frac{1}{305 Q_c OCR^{1.8}}\right]\frac{\frac{q_c}{P_a}}{\left(\frac{\sigma_o'}{P_a}\right)^{0.5}}}$$

Qc : compressibility factor **ranges between 0.91 for High compressible sand (loose) to 1.09 law compressible sand (dense to very dense)**

Correlation

- **Internal Friction angle** $\phi' = \tan^{-1}\left[0.1 + 0.38\log\left(\frac{qc}{\sigma'_o}\right)\right]$

- **Undrained Shear Strength, Cu ;**

$$\frac{C_u}{\sigma'_o} = \left(\frac{q_c - \sigma_o}{\sigma'_o}\right)\frac{1}{N_K}$$

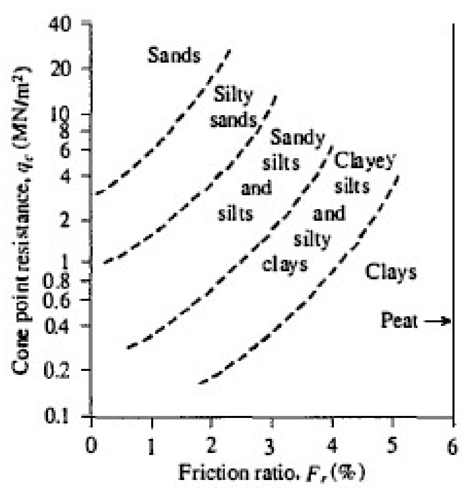
N_K Bearing capacity factor 15 for mechanical and 20 for electrical

- **Maximum past pressure and OCR as**

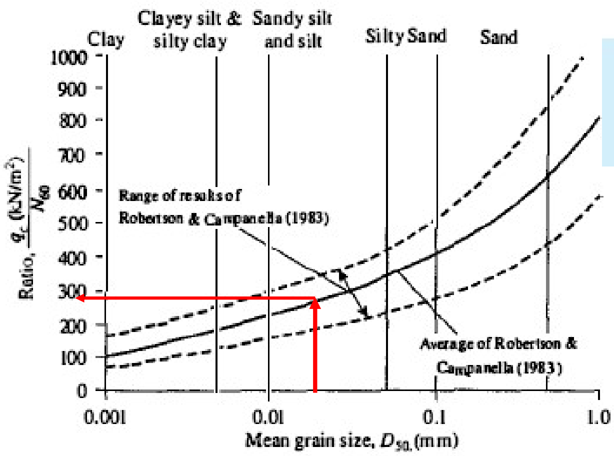
$$OCR = 0.37\left(\frac{q_c - \sigma_o}{\sigma'_o}\right)^{1.01}$$

$$\sigma'_c = 0.243(q_c)^{0.96} \quad (\text{MN/m}^2)$$

Classification of soil based on CPT test results

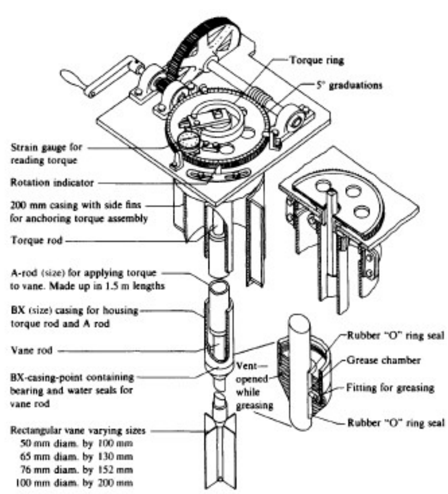


Correlation between q_c/N_{60} and the mean grain size, D50.



$$\frac{\left(\frac{q_c}{p_a}\right)}{N_{60}} = 7.6429 D_{50}^{0.26}$$

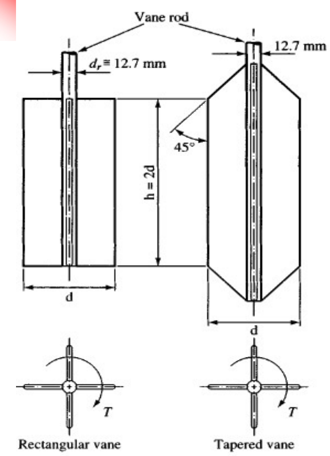
FIELD VANE SHEAR TESTING (FVST)



(a) The Bureau of Reclamation vane shear test apparatus. *Gibbs et al. (1960), courtesy of Gibbs and Holtz of the USBR.*]

Vane Shear Test

Data Reduction
undrained shear strength



(a) usual shapes

$$T = f(c_u, H, \text{and } D)$$

Cu=T/K

$$K = \left(\frac{\pi}{10^6}\right)\left(\frac{D^2H}{2}\right)\left(1 + \frac{D}{3H}\right)$$

$K=366 \times 10^{-8} D^3$; D(cm) if H/D=2.0

$Cu = \lambda C_{u(VST)}$

$\lambda = 1.7 - 0.54\log(PI\%)$

or use Figure 2.11 in text page 67

$$\sigma'_c = 7.04[c_{u(\text{field})}]^{0.83}$$

$$\text{OCR} = \beta \frac{c_{u(\text{field})}}{\sigma'_o}$$

$$\beta = 22[\text{PI}(\%)]^{-0.48}$$

ROCK SAMPLING

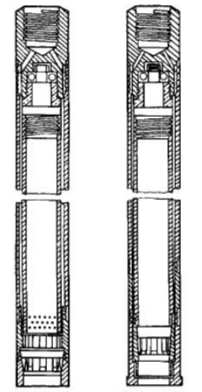
- blow counts are at the refusal level *(N > 100)*→Use Rock cores

Typical standard designation and sizes for rock drill casing (barrel) and bits*

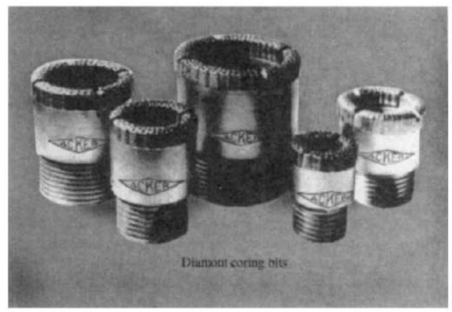
Casing OD, mm		Core bit OD, mm		Bit ID, mm
RW	29	EWT	37	23
EW	46	AWT	48	32
AW	57	BWT	60	44
BW	73	NWT	75	59
NW	89	HWT	100	81
PW	140		194	152

* See ASTM D 2113 for the complete range in core bit, casing, and drill rod sizes in current use. Sizes are nominal—use actual diameter of recovered core.

Rock coring equipment



(a) Core barrels to collect rock cores



(b) Coring bits to attach to core barrel. (The Acker Drill Company)

Rock quality designation

- Rock quality designation (RQD) is an index or measure of the quality of a rock mass used by many engineers. RQD is computed from recovered core samples as

$$\text{RQD} = \frac{\sum \text{Lengths of intact pieces of core} > 100\text{ mm}}{\text{Length of core advance}}$$

Table 2.8 Relation between *in situ* Rock Quality and RQD

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

Depth of Rock Cores

- There are no fast rules for rock core depths. Generally one should core approximately as follows:

1. A depth sufficient to locate sound rock or to ascertain that it is fractured and jointed to a very great depth.

2. For heavily loaded members such as piles or drilled piers, a depth of approximately 3 to 4 m below the location of the base. The purpose is to check that the "sound" rock does not have discontinuities at a lower depth in the stress influence zone and is not a large suspended boulder.

GROUNDWATER TABLE (GWT) LOCATION

- The GWT is generally determined by directly measuring to the stabilized water level in the borehole after a suitable time lapse, often 24 to 48 hr later. **This measurement is done by lowering a weighted tape down the hole until water contact is made**. In soils with a high permeability, such as sands and gravels, 24 hr is usually a sufficient time for the water level to stabilize unless the hole wall has been somewhat sealed with drilling mud.

NUMBER AND DEPTH OF BORINGS

- Depth of Boring for a building of 30.5 m wide
  - $D_b=3S^{0.7}$ **for light steel and narrow concrete building**
  - $D_b=6S^{0.7}$ **for heavy steel or wide concrete structure**
- **For deep excavation at least 1.5 timers of the depth of excavation**
- **In bed rock at least 3m**
- **Approximate spacing of Boreholes (Number)**

Approximate spacing of Boreholes (number)

Type of Project	Spacing (m)
Multistory building	10-30
One store industrial plants	20-60
Highways	250-500
Dams and Dukes	40-80

Minimum Depths Requirements For Boring for Shallow Foundation

AASHTO Standard Specifications for Design of Highway Bridges

- For isolated footings of breadth Lf and width Bf, where Lf < 2Bf, borings shall extend a minimum of two footing widths below the bearing level.

- For isolated footings where Lf >5Bf, borings shall extend a minimum of four footing widths below the bearing level.

- For 2Bf < Lf < 5Bf, minimum boring length shall be determined by linear interpolation between depths of 2Bf and 5Bf below the bearing level.

Minimum Depths Requirements For Boring for Deep Foundation

AASHTO Standard Specifications for Design of Highway Bridges

- In soil, borings shall extend below the anticipated pile or shaft tip elevation a minimum of 6 m, or a minimum of two times the maximum pile group dimension, whichever is deeper.
- For piles bearing on rock, a minimum of 3 m of rock core shall be obtained at each boring location to verify that the boring has not terminated on a boulder.
- For shafts supported on or extending into rock, a minimum of 3 m of rock core, or a length of rock core equal to at least three times the shaft diameter for isolated shafts or two times the maximum shaft group dimension, whichever is greater, shall be extended below the anticipated shaft tip elevation to determine the physical characteristics of rock within the zone of foundation influence.

Minimum Depths Requirements For Boring for Retaining Walls
AASHTO Standard Specifications for Design of Highway Bridges

- Extend borings to depth below final ground line between 0.75 and 1.5 times the height of the wall. Where stratification indicates possible deep stability or settlement problem, borings should extend to hard stratum.

Minimum Depths Requirements For Boring for Roadways
AASHTO Standard Specifications for Design of Highway Bridges

- Extend borings a minimum of 2 m below the proposed subgrade level.

Guidelines For Boring Layout
FHWA Geotechnical Checklist and Guidelines; FHWA-ED-88-053

Bridge Foundations	▪For piers or abutments over 30 m wide, provide a minimum of two borings. ▪For piers or abutments less than 30 m wide, provide a minimum of one boring. ▪Additional borings should be provided in areas of erratic subsurface conditions.
Retaining Walls	▪A minimum of one boring should be performed for each retaining wall. ▪For retaining walls more than 30 m in length, the spacing between borings should be no greater than 60 m. ▪Additional borings inboard and outboard of the wall line to define conditions at the toe of the wall and in the zone behind the wall to estimate lateral loads and anchorage capacities should be considered.

Boring Log

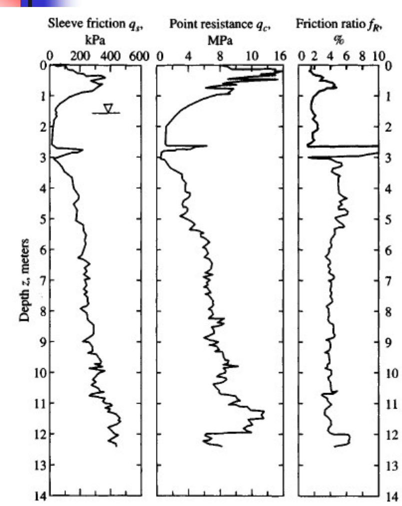
Name of the Project: Two-story apartment building

Location: Johnson & Olive St. Date of Boring: March 2, 2005

Boring No. 3 Type of Boring: Hollow-stem auger Ground Elevation: 60.8 m

Soil description	Depth (m)	Soil sample type and number	N_{60}	w_n (%)	Comments
Light brown clay (fill)					
	1				
Silty sand (SM)	2	SS-1	9	8.2	
°G.W.T. 3.5 m	3	SS-2	12	17.6	LL = 38 PI = 11
	4				
Light gray clayey silt (ML)	5	ST-1		20.4	LL = 36 q_u = 112 kN/m²
	6	SS-3	11	20.6	
Sand with some gravel (SP)	7				
End of boring @ 8 m	8	SS-4	27	9	

EXAMPLE

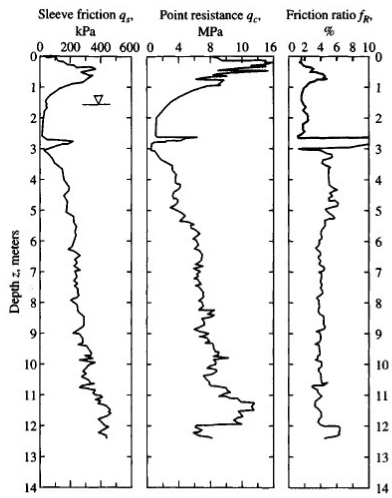


Soil Unit weight γsat=18 kN/m3, γd=16kN/m3; electrical CPT

Determine the following

1. Effective average internal angle of friction at depth 3.5m
2. Undrained shear strength at depth 3.5 m
3. Maximum past pressure at depth 3.5 m
4. Over consolidation ratio at depth 3.5 m
5. the consistency of the soil layer at depth 3.5 m
6. N_{60}, if the main grain size diameter was 0.02mm

EXAMPLE

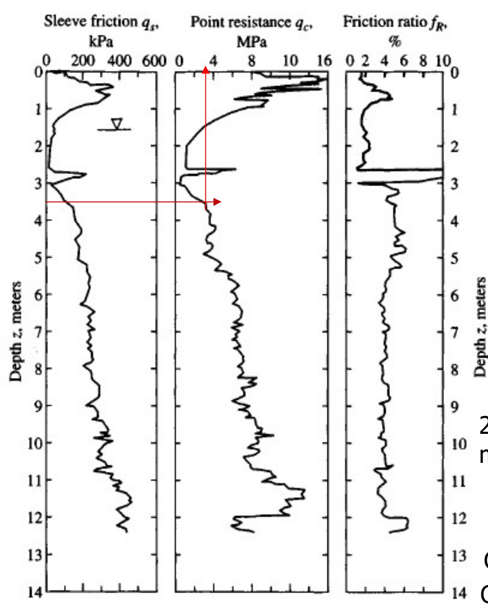


γt=18kN/m3; γd=16kN/m3

Determine the following, electrical CPT

1. Effective average internal angle of friction at depth 3.5m
2. Undrained shear strength at depth 3.5 m
3. Maximum past pressure at depth 3.5 m
4. Over consolidation ratio at depth 3.5 m
5. the consistency of the soil layer at depth 3.5 m
6. N_{60}, if the main grain size diameter was 0.02mm

1. Effective average internal angle of friction at depth 3.5m



qc=3.5 MPa=3500kPa

$$\phi' = \tan^{-1}\left[0.1 + 0.38\log\left(\frac{qc}{\sigma'_o}\right)\right]$$

φ′=tan-1(0.1+0.38log(3500/σ′))

σ′= 1.5*16+2(18-9.81)=40.38kPa

σ=1.5*16+2(18)=60 kPa

φ′=tan-1(0.1+0.38log(3500/40.38

φ′=40°

2.0Undrained shear strength at depth 3 m

$$\frac{C_u}{\sigma'_o} = \left(\frac{q_c - \sigma_o}{\sigma'_o}\right)\frac{1}{N_K}$$

Cu=40.38((3500-60)/40.38)(1/20)

Cu=172 kPa)

Maximum past pressure at depth 3.5 m

$$\sigma'_c = 0.243(q_c)^{0.96}$$

σc;'=0.243(3.5)^0.96=0.81MPa=810 kPa

1. Over consolidation ratio at depth 3.5 m

$$OCR = 0.37\left(\frac{q_c - \sigma_o}{\sigma'_o}\right)^{1.01}$$

OCR=0.37*((3500-60)/40.38)^1.01=33

1. the consistency of the soil layer at depth 3.5 m
2. N_{60}, if the main grain size diameter was 0.02mm

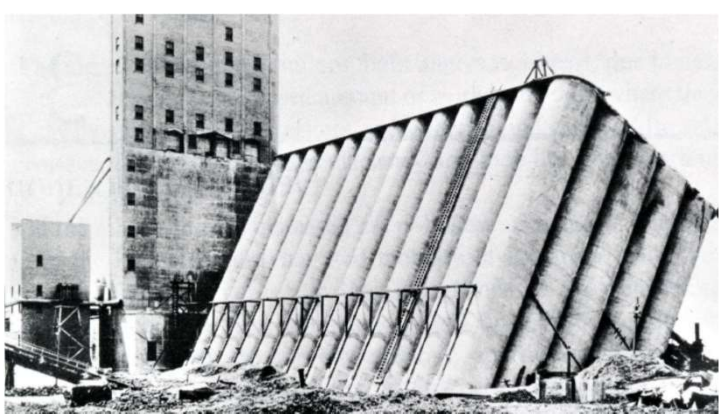
Go to chart convert qc to N60, qc/N60=280   N60=3500/280=12.5
Say N60=12 since below water table N'=12 since N'<15

1. since N'=12 soil from table soil consistency is stiff *to very Stiff (Cu=170 kPa)*

The Bearing Capacity of Soils

Dr Omar Al Hattamleh

Example of Bearing Capacity Failure



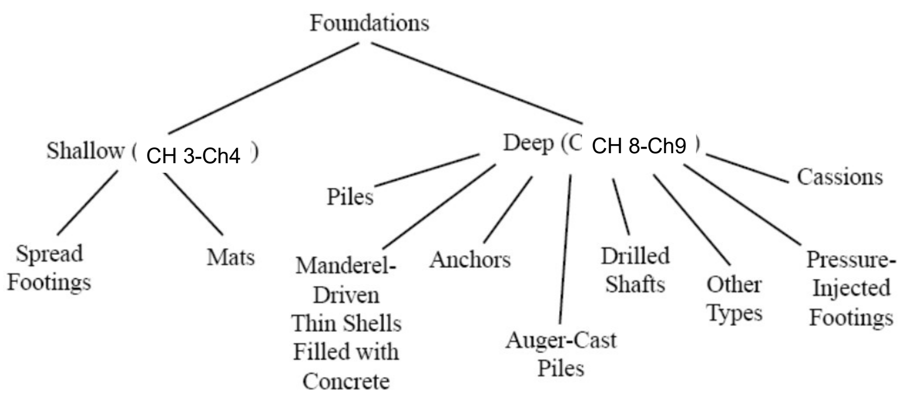
Transcona Grain Silos Failure - Canada

Modern bearing Capacity failure

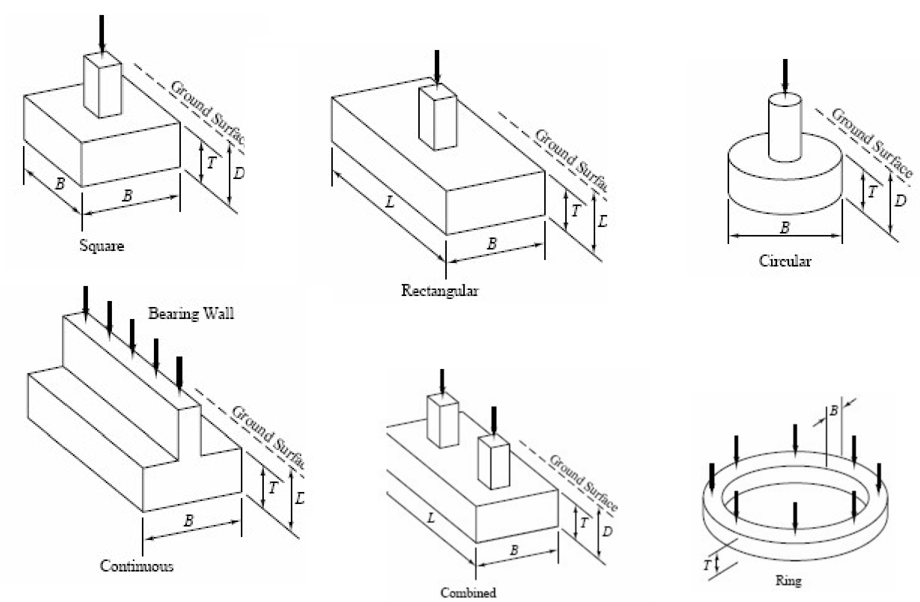
The Bearing Capacity of Soils

- -***Terzaghi's Ultimate Bearing Capacity***
- -***Meyerhof's Method***
- -***Brinch Hansen' Method***
- -***Vesic's Method***
- ***- General Ultimate Bearing Capacity***

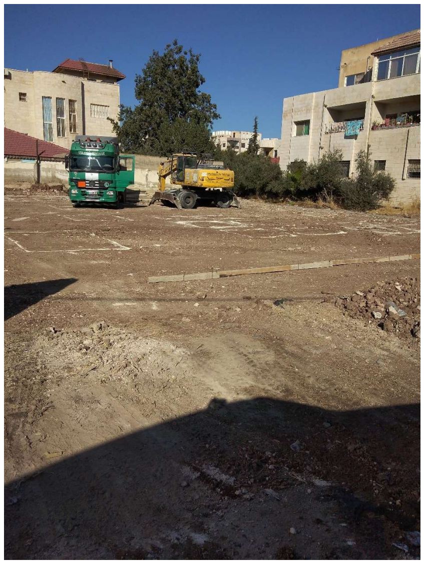
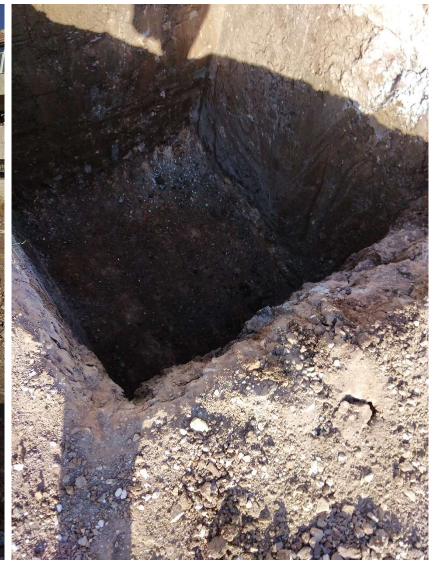
Classification of foundations



Spread footing Shapes & Dimensions

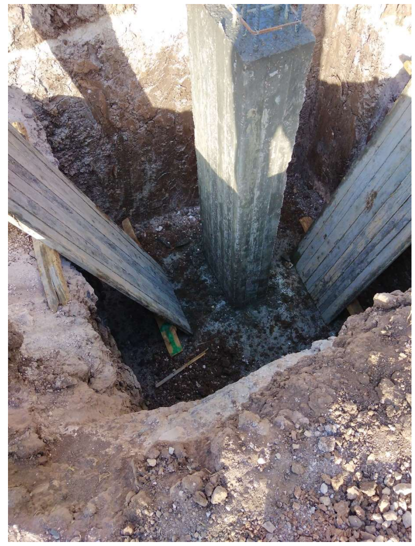


Shallow Foundation in Plan

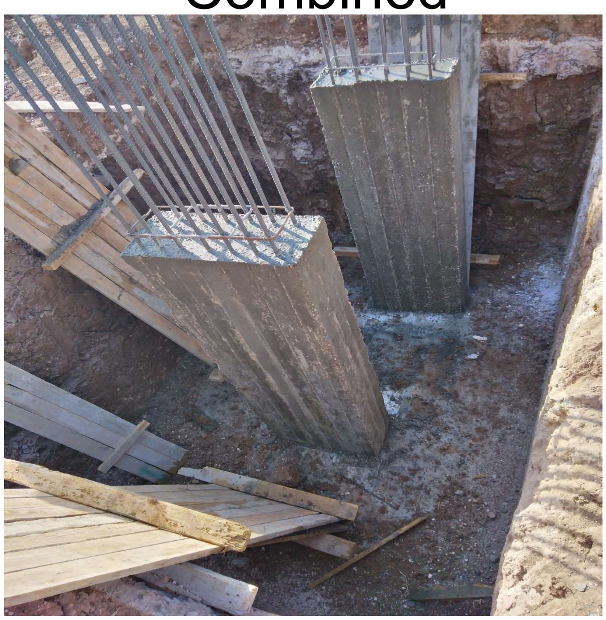



Shallow Foundation in Plan


Shallow Foundation in Plan



Combined

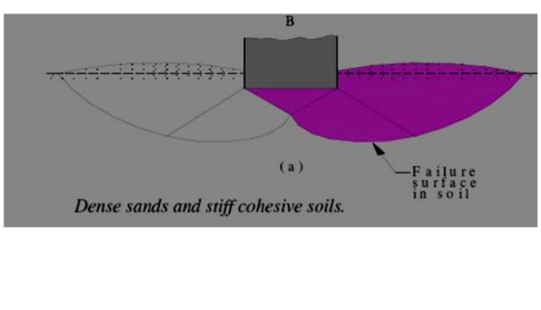


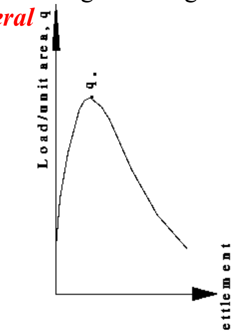
Mode of Failure

1. ***General Shear Failure***
2. ***Local Shear Failure***
3. ***Punching Shear Failure***

Mode of Failure

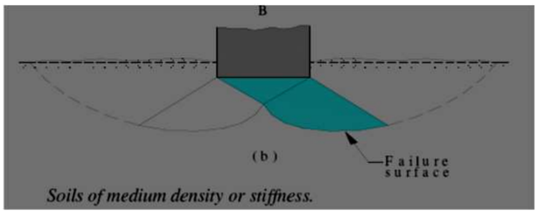
- A continuous footing resting on the surface of a ***dense sand*** or a ***stiff cohesive*** soil is shown in Figure 2a with a width of B. If a load is gradually applied to the footing, its settlement will increase. When the load per unit area equals q_{ult} a sudden failure in the soil supporting the foundation will take place, with the failure surface in the soil extending to the ground surface. This type of ***sudden*** failure is called a ***general***



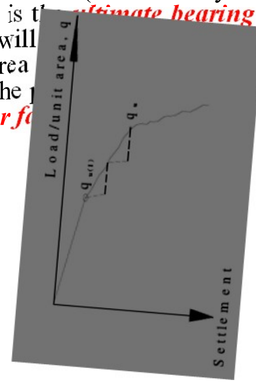


Mode of Failure

If the foundation rests on send or clayey soil of ***medium compaction*** (Figure 2b), an increase of load on the foundation will increase the settlement and the failure surface will ***gradually*** extend outward from the foundation (as shown by the solid line). When the load per unit area on the foundation equals q_{ult}, the foundation movement will be like sudden jerks. A considerable movement of the foundation is required for the failure surface in soil to extend to the ground surface (as shown by the broken lines). The load per unit area at which this happens is t ***ultimate bearing capacity qult***. Beyond this point, an increase of the load will a large increase of footing's settlement. The load per unit area referred to as the ***first failure load*** (Vesic 1963). Note that the realized in this type of failure, which is called the ***local shear f***

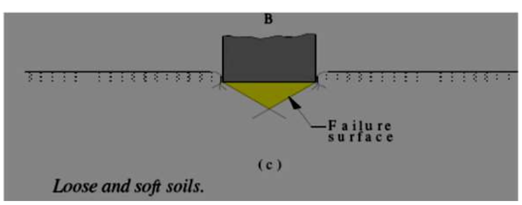


Soils of medium density or stiffness.

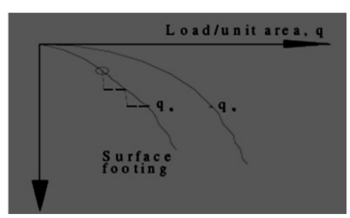


Mode of Failure

If the foundation is supported by a fairly ***loose*** soil, the load-settlement plot will be like the one in Figure 2c. In this case, the failure surface in soil will not extend to the ground surface. Past the value ***qult***, the load-to-settlement plot will be steep and practically linear. This type of failure is called the ***punching shear failure***.

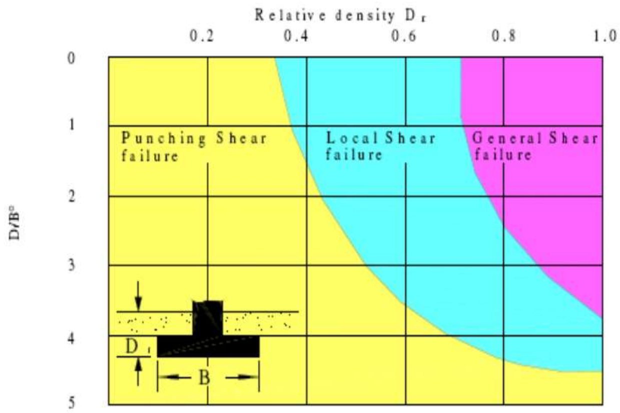


Loose and soft soils.



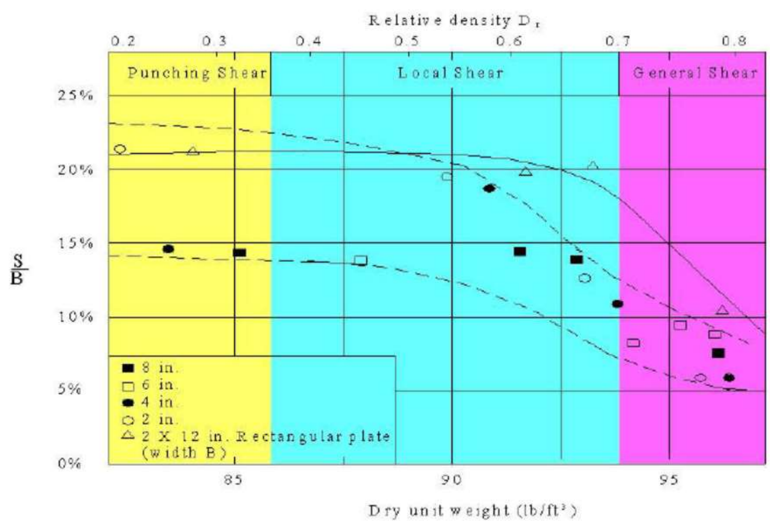
Modes of failure

Based on experimental results from Vesic (1963), a relation for the mode of bearing capacity failure of foundations can be proposed (Figure 4), where
D_r is the relative density in sand,
D_f is the depth of the footing measured from the ground surface,
B is the width and L is the length of the footing (Note: L is always greater than B)



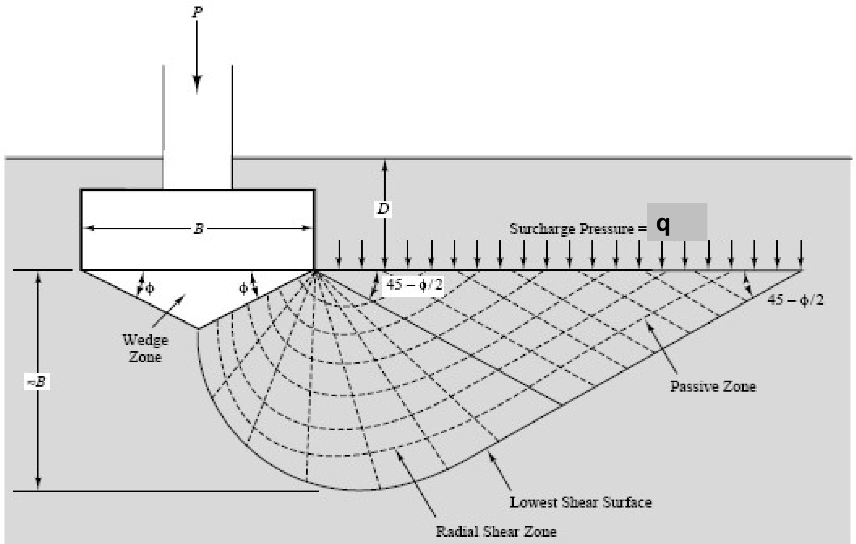
$$B^* = \frac{2BL}{B+L}$$

Mode of Failure and Settlement



Range of settlement of circular and rectangular plates at ultimate loads for *Df* / *B* = 0 in sand (after Vesic, 1973).

Terzaghi's Bearing Capacity Formulas



Terzaghi's Ultimate Bearing Capacity Theory

Using an equilibrium analysis, *Karl Terzaghi* expressed in 1943 the ultimate bearing capacity q_u of a particular soil to be of the form,

$q_u = c N_c + \bar{q} N_q + 0.5 \gamma B N_\gamma$ **(for strip footings, such as wall foundations)**

$q_u = 1.3c N_c + \bar{q} N_q + 0.4 \gamma B N_\gamma$ **(for square footings, typical of interior columns)**

$\boxed{q_u = 1.3c' N_c + q N_q + 0.3 \gamma B N_\gamma}$ **for circular footings, such as towers, chimneys)**

Where,

q□ =q = γD_f is the removed pressure from the soil to place the footing

N_c, N_γ, and N_q are the soil-bearing capacity factors, dimensionless terms, whose values relate to the angle of internal friction . These values can be calculated when is known or they can be looked up in Terzaghi's Bearing Capacity Factor Table 3.1 page 87.

c' = cohesion of soil

γ= unit weight of soil

Terzaghi's Ultimate Bearing Capacity Factors

The bearing capacity factors Nc Nq, and Nγ are defined by

$$N_c = \cot\phi'\left[\frac{e^{2(3\pi/4-\phi'/2)\tan\phi'}}{2\cos^2\left(\frac{\pi}{4}+\frac{\phi'}{2}\right)}-1\right] = \cot\phi'(N_q-1)$$

$$N_q = \frac{e^{2(3\pi/4-\phi'/2)\tan\phi'}}{2\cos^2\left(45+\frac{\phi'}{2}\right)}$$

$$N_\gamma = \frac{1}{2}\left(\frac{K_{p\gamma}}{\cos^2\phi'}-1\right)\tan\phi'$$

$K_{p\gamma}=\tan^2(45+\phi'/2)$

Table 3.1 Terzaghi's Bearing Capacity Factors—Eqs. (3.4), (3.5), and (3.6)

ϕ'	N_c	N_q	N_γ *	ϕ'	N_c	N_q	N_γ *
0	5.70	1.00	0.00	26	27.09	14.21	9.84
1	6.00	1.10	0.01	27	29.24	15.90	11.60
2	6.30	1.22	0.04	28	31.61	17.81	13.70
3	6.62	1.35	0.06	29	34.24	19.98	16.18
4	6.97	1.49	0.10	30	37.16	22.46	19.13
5	7.34	1.64	0.14	~~31~~	~~40.41~~	~~25.28~~	~~22.65~~
6	7.73	1.81	0.20	32	44.04	28.52	26.87
7	8.15	2.00	0.27	33	48.09	32.23	31.94
8	8.60	2.21	0.35	34	52.64	36.50	38.04
9	9.09	2.44	0.44	35	57.75	41.44	45.41
10	9.61	2.69	0.56	36	63.53	47.16	54.36
11	10.16	2.98	0.69	37	70.01	53.80	65.27
12	10.76	3.29	0.85	38	77.50	61.55	78.61
13	11.41	3.63	1.04	39	85.97	70.61	95.03
14	12.11	4.02	1.26	40	95.66	81.27	115.31
15	12.86	4.45	1.52	41	106.81	93.85	140.51
16	13.68	4.92	1.82	42	119.67	108.75	171.99
17	14.60	5.45	2.18	43	134.58	126.50	211.56
18	15.12	6.04	2.59	44	151.95	147.74	261.60
19	16.56	6.70	3.07	45	172.28	173.28	325.34
20	17.69	7.44	3.64	46	196.22	204.19	407.11
21	18.92	8.26	4.31	47	224.55	241.80	512.84
22	20.27	9.19	5.09	48	258.28	287.85	650.67
23	21.75	10.23	6.00	49	298.71	344.63	831.99
24	23.36	11.40	7.08	50	347.50	415.14	1072.80
25	25.13	12.72	8.34				

*From Kumbhojkar (1993)

B.C. Factor of Safety

The factor of safety *FS* against a bearing capacity failure defined

$$q_{all} = \frac{q_{ult}}{FS}$$

Use q_{net} instead of q_{ult}

where *qall is the gross allowable load-bearing capacity* and *qnet is the net ultimate bearing capacity*.

The factor of safety is chosen according the function of the structure, but never less than 3 in all cases.

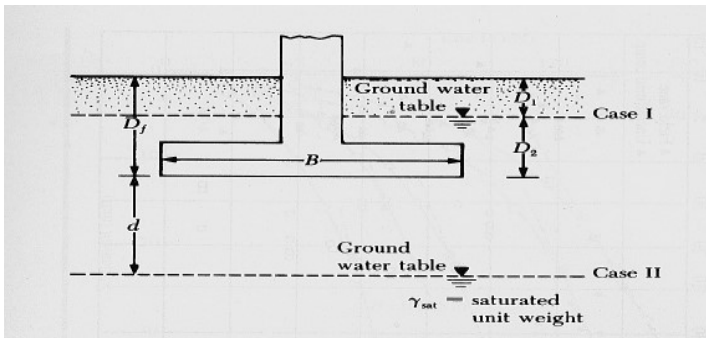
The net ultimate bearing capacity is defined as the ultimate pressure per unit area of the footing that can be supported by the soil in excess of the pressure caused by the surrounding soil at the foundation level.

$$q_{net} = q_{ult} - \bar{q} = q_{ult} - \gamma D_f$$

A footing will obviously not settle at all if the footing is placed at a depth where the weight of the soil removed is equal to the weight of the column's load plus the footing's weight.

Modification of the Bearing Capacity Equations for the Water Table

Case I: When $0 < D_1 < D_f$.



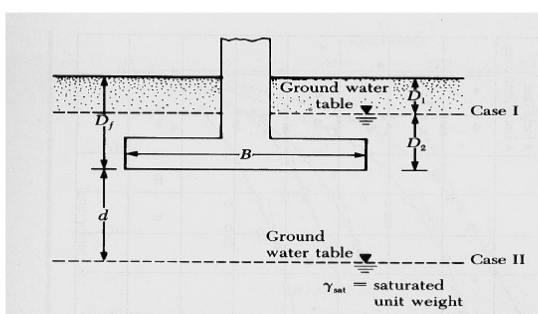
$$\boxed{q_e = D_1\gamma + D_2\left(\gamma_{sat} - \gamma_w\right)}$$

In term 2 of BC equation

Use γ’ in term 3 of BC equation

Modification of the Bearing Capacity Equations for the Water Table

Case II: When 0 ≤ d ≤ B



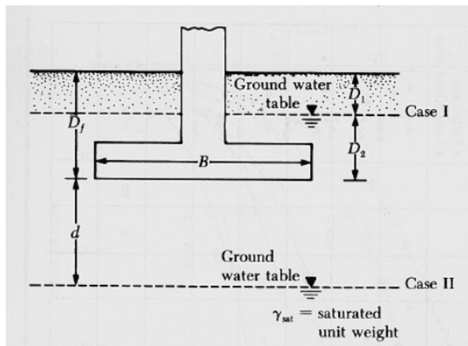
$$\boxed{\bar{\gamma} = \gamma' + (\gamma - \gamma')\frac{d}{B}}$$

In term 3 of BC equation

Use γ in term 2 of BC equation

Modification of the Bearing Capacity Equations for the Water Table

Case III. When $d \geq B$, the water table will have no effect on the ultimate bearing capacity.



The Bearing Capacity for Local or Punching Shear failure

For the local shear failure Terzaghi proposed reducing the *cohesion* and *internal friction angle* as

$$\boxed{\begin{aligned} c'' &= 0.67c \\ \phi'' &= \tan^{-1}(0.67 \tan \phi) \end{aligned}}$$

Examples (1)

A square foundation is 1.5m x 1.5m in plan. The soil supporting the foundation has a friction angle of $\phi' = 20°$ and $c' = 65kPa$. The unit. weight of soil is 19kN/m3. Determine the allowable gross load on the foundation with a Factor safety (FS) of 4: Assume that the depth of the foundation (D_f) is 1.25 m and that general shear failure -occurs in the soil.

-square footing BxB=1.5m x 1.5 m -ϕ'=20° c'=65 kPa γ=19kN/m^3

-FS=4 -DF=1.25m -General shear Failure!

$qu=1.3CNc+qNq+0.4\gamma BN\gamma$

–Assume the ground water table is very deep d>>>B

-ϕ'=20° Nc=17.69 Nq= 7.44 Nγ =3.64

–$q=\gamma Df=19*1.25=23.75kPa$; γ=19kN/m3

qu=1.3(65)(17.69)+(23.75)(7.44)+0.4(19)(1.5)(3.64)=1713.0kPa

qall=qu/FS=1713.0/4=428.25kPa

Qall (load)=qall*(area)=428.25*(1.5*1.5)=963.6kN

Examples (2)

A square foundation is 1.5m x 1.5m in plan. The soil supporting the foundation has a friction angle of $\phi = 20°$ and c' = 65kPa'. The unit. Saturated unit weight of soil is 19.81kN/m3. Determine the allowable gross load on the foundation with a Factor safety (FS) of 4: Assume that the depth of the foundation (D_f) is 1.25 m and that general shear failure -occurs in the soil.

-square footing BxB=1.5m x 1.5 m -ϕ'=20° c'=65 kPa γ_{sat}=19.81kN/m^3

-FS=4 -DF=1.25m -General shear Failure!

$qu=1.3CNc+qNq+0.4\gamma BN\gamma$

–Assume the ground water table at the ground surface D1=0.0

-ϕ'=20° Nc=17.69 Nq= 7.44 Nγ =3.64

–$q=\gamma' Df=10*1.25=12.50kPa$; γ'=19.81-9.81=10.0 kN/m3

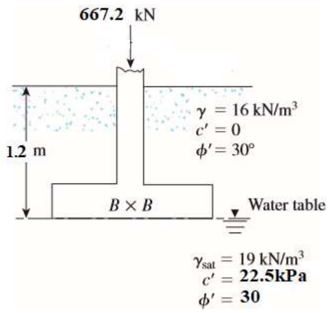
qu=1.3(65)(17.69)+(12.5)(7.44)+0.4(10)(1.5)(3.64)=????.0kPa

qall=qu/FS=?????/4=?????kPa

Qall (load)=qall*(area)=?????*(1.5*1.5)=******kN

Examples (3)

A square footing is shown in Figure below. Use Factor of Safety = 3 and Terzaghi bearing capacity equation **determine the size of the footing if punching shear failure occur**.



Solution:

-Punching shear Failure

C"=0.67(c')=0.67*22.5=15.0kPa

ϕ"=tan^{-1}(0.67tanϕ')=tan^{-1}(0.67tan30)=21.1^o ≅21^o

From Table ϕ'=21^o Nc=18.92; Nq=8.26; Nγ=4.31

$qu = \frac{Qult}{Area} = \frac{Qall(FS)}{area}$=667.2*3/(BXB)=2001.6/B^2 (1)

qu= $1.3(CNc) + qNq + 0.4\gamma BN\gamma$

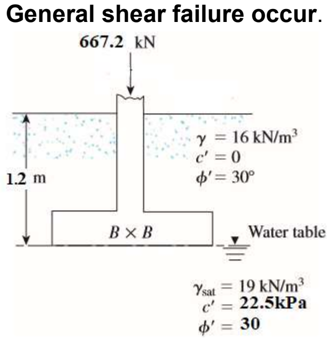
q=γD`f =16*1.2=

qu=1.3(15)(18.92)+(16*1.2)(8.26)+0.4(19-9.81)B(4.31) (2)

Eq (1)=Eq(2) solve B=1.8947m=1.9m

Examples (4)

A Circular footing is shown in Figure below. Use Factor of Safety = 3 and Terzaghi bearing capacity equation **determine the size of the footing if General shear failure occur**.



Solution:

-General shear Failure

C=22.5 kPa

ϕ=30^o

From Table ϕ'=30^o Nc=37.16 ; Nq= 22.46; Nγ=19.13

Qu=Qu/Area=Qall*FS/area= 667.2*3*4/(πB2)

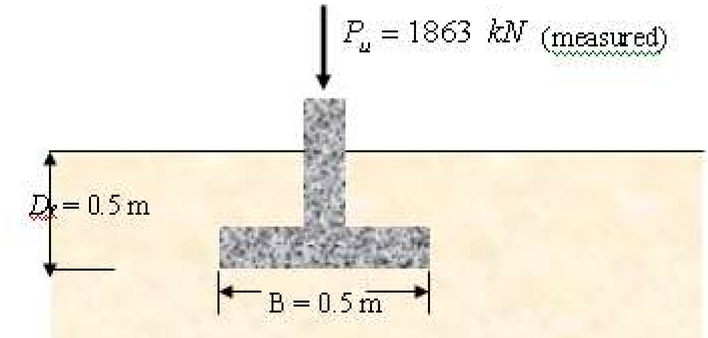
q=γD`f =16*1.2=

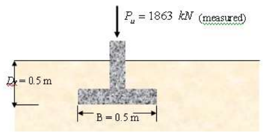
qu=1.3(22.5)(37.16)+(16*1.2)(22.46)+0.3(19-9.81)B(19.13) (2)

Eq (1)=Eq(2) solve B=Diamater=*m=00m**

Example (2)

Compare ***Terzaghi*** bearing capacity equations versus a measured field test that resulted in *qu*., if L= 6.0 m, c=0, φ_t = 42° and γ' = 9.31 kN/m³





Solution

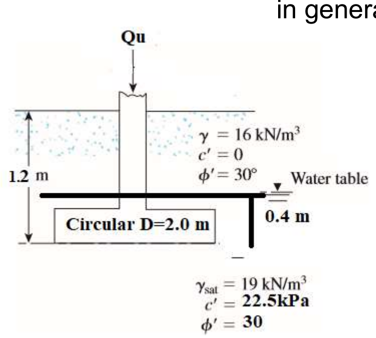
qu=cNc+qNq+0.5γBNγ

φ'=42 Nc=119.67 Nq=108.75 & Nγ =171.99

qu=0+(9.31)(0.5)(108.75)+0.5*9.31*0.5*171.99=906.5kPa
Qu=qu*B=906.5*0.5=453.25kN

Compare calculated load (453.25 kN) with field measurement (1863 kN) Indicate that Terzaghi Equation for bearing capacity calculation is very conservative

Determine the ultimate load that the circular footing shown could carry in general shear failure



qu=1.3CNc+qNq+0.3γBNγ

ϕ'=30°

Nc=37.16 Nq=22.46 Nγ=19.13

Water table 0<D1<Df case # 1
qe=16(1.2-0.4)+(19-9.81)(0.4)=16.476kPa

qu=1.3(22.5)(37.16)+16.476(22.46)+0.3*(19-9.81)*2.0*19.13=1562.5kPa

Qu=qu*area=1562.5*π*2*2/4=4908.74kN

What will happen to the value of ultimate bearing capacity if water table draw down to 0.5 m below the footing invert

qu=1.3(22.5)(37.16)+(1.2*16)(22.46)+0.3γ''*2.0*19.13=1562.5kPa

$\gamma''=\gamma'+(\gamma-\gamma')d/B$=(19-9.81)+((16-(19-9.81))*0.5/2.0=10.89kN/m^3

qu=1.3(22.5)(37.16)+(1.2*16)(22.46)+0.310.89*2.0*19.13=1643.157kPa

General Bearing Capacity Equation

The General Bearing Capacity Equation.

The *Terzaghi* ultimate bearing capacity equations presented previously are for continuous, square, and circular footings only. They do not include rectangular footings (0 < B/L < 1), or take into account the shearing resistance along the failure surface in the soil above the bottom of the foundation, or the inclination of the footing or the load (Hansen, 1970)

$$q_u = c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

Where

c = the cohesion;

q = the excavated soil's pressure at the footing's invert (its bottom);

γ = the unit weight of the soil;

B = width of foundation (equal to the diameter for a circular foundation);

Nc, Nq, Nγ are the bearing capacity factors;

Fcs, Fqs, F γ s are the shape factors;

Fcd, Fqd, Fγd are the depth factors; and

Fci, Fqi, F γ i are the load inclination factors.

bearing capacity factors

$$N_q = e^{\pi \tan\phi} \tan^2(45° + \varphi'/2) \quad N_c = (N_q - 1)\cot\varphi' \quad N_\gamma = 2(N_q + 1)\tan\phi'$$

Table 3.3 Bearing Capacity Factors

ϕ'	N_c	N_q	N_γ	ϕ'	N_c	N_q	N_γ
0	5.14	1.00	0.00	26	22.25	11.85	12.54
1	5.38	1.09	0.07	27	23.94	13.20	14.47
2	5.63	1.20	0.15	28	25.80	14.72	16.72
3	5.90	1.31	0.24	29	27.86	16.44	19.34
4	6.19	1.43	0.34	30	30.14	18.40	22.40
5	6.49	1.57	0.45	31	32.67	20.63	25.99
6	6.81	1.72	0.57	32	35.49	23.18	30.22
7	7.16	1.88	0.71	33	38.64	26.09	35.19
8	7.53	2.06	0.86	34	42.16	29.44	41.06
9	7.92	2.25	1.03	35	46.12	33.30	48.03
10	8.35	2.47	1.22	36	50.59	37.75	56.31
11	8.80	2.71	1.44	37	55.63	42.92	66.19
12	9.28	2.97	1.69	38	61.35	48.93	78.03
13	9.81	3.26	1.97	39	67.87	55.96	92.25
14	10.37	3.59	2.29	40	75.31	64.20	109.41
15	10.98	3.94	2.65	41	83.86	73.90	130.22
16	11.63	4.34	3.06	42	93.71	85.38	155.55
17	12.34	4.77	3.53	43	105.11	99.02	186.54
18	13.10	5.26	4.07	44	118.37	115.31	224.64
19	13.93	5.80	4.68	45	133.88	134.88	271.76
20	14.83	6.40	5.39	46	152.10	158.51	330.35
21	15.82	7.07	6.20	47	173.64	187.21	403.67
22	16.88	7.82	7.13	48	199.26	222.31	496.01
23	18.05	8.66	8.20	49	229.93	265.51	613.16
24	19.32	9.60	9.44	50	266.89	319.07	762.89
25	20.72	10.66	10.88				

Shape and Depth, and Inclination Factors

Shape Factors.

$$F_{cs} = 1 + \frac{B}{L}\frac{N_q}{N_c} \qquad F_{qs} = 1 + \frac{B}{L}\tan\phi \qquad F_{\gamma s} = 1 - 0.4\frac{B}{L}$$

Depth Factors for $D_f / B \le 1$.

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan\phi'} \qquad F_{qd} = 1 + 2\tan\phi(1-\sin\phi)^2\frac{D_f}{B} \qquad F_{\gamma d} = 1$$

Depth Factors for $Df/B > 1$.

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan\phi'} \qquad F_{qd} = 1 + 2\tan\phi(1-\sin\phi)^2 \tan^{-1}\frac{D_f}{B} \qquad F_{\gamma d} = 1$$

For $\phi = 0$ *$D_f / B \le 1$.* $F_{cd} = 1 + 0.4\frac{D_f}{B}$

For $\phi = 0$ *$Df/B > 1$.* $F_{cd} = 1 + 0.4\tan^{-1}\left(\frac{D_f}{B}\right)$

Inclination Factors with β is the inclination of load with respect to the vertical.

$$F_{ci} = F_{qi} = \left(1 - \frac{\beta^o}{90^o}\right)^2 \qquad F_{\gamma i} = \left(1 - \frac{\beta}{\phi}\right)^2$$

Example 2

Rectangular foundation is 1.25 m × 2 m in plan. The soil supporting the foundation has a friction angle of $\phi' = 25°$ and $c' = 20\ \text{kN/m}^2$. The unit weight of soil, γ, is $16.5\ \text{kN/m}^3$. Determine the allowable gross load on the foundation with a factor of safety (FS) of 3. Assume that the depth of the foundation (D_f) is 1.5 m and that general shear failure occurs in the soil. what would be the load if local shear failure occur

Assume water table at depth of 1.0 m below the footing invert and saturated unit weight is 19.81 kN/m3

$$q_u = c'N_cF_{cs}F_{cd}F_{ci} + qN_qF_{qs}F_{qd}F_{qi} + \tfrac{1}{2}\gamma BN_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i}$$

$\phi' = 25°$ go to table of general bearing capacity factors

Nc=20.72 Nq=10.66 and Nγ=10.88

q=γDf=16.5*1.5=24.75kPa

γ=? Location of water table d=1.0m but B=1.25m 0<d=1.0<B=1.25m

$\gamma''=\gamma'+(\gamma-\gamma')d/B$=(19.81-9.81)+(16.5-(19.81-9.81))*(1.0/1.25)=15.2kN/m^3

Shape Factors` ϕ'>0.0 and Df/B>1.5m/1.25 **Df/B>1.0**

$$F_{cs} = 1 + \frac{B}{L}\frac{N_q}{N_c} \qquad F_{qs} = 1 + \frac{B}{L}\tan\phi \qquad F_{\gamma s} = 1 - 0.4\frac{B}{L}$$

Fγs=1-0.4*1.25/2.0 =0.75

Fcs=1+(1.25/2.0)*(10.66/20.72)=1.32 Fqs=1+(1.25/2.0)*tan25=1.29

Depth Factors for Df/B >1.

$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan \phi'}$ $F_{qd} = 1 + 2\tan\phi(1-\sin\phi)^2 \tan^{-1}\frac{D_f}{B}$ $F_{\gamma d} = 1$

Fγd=1.0 Fqd=1+2tan25(1-sin25)²tan⁻¹(1.5/1.25)=1.272

Fcd=1.272-(1.0-1.272)/(20.72*tan25)=1.300

$F_{ci} = F_{qi} = \left(1 - \frac{\beta^o}{90^o}\right)^2$ $F_{\gamma i} = \left(1 - \frac{\beta}{\phi}\right)^2$

Since no inclination in load mention,
Therefore, β=0.0 then Fci=Fqi=Fγi=1.0

$$q_u = c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \tfrac{1}{2}\gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

qu=(20)(20.72)(1.32)(1.30)(1.00)+24.75(10.66)(1.29)(1.272)(1.00)
+0.50(15.2)(1.25)(10.88)(0.75)(1.00)(1.00)=1221.55kPa

Qall=qall*area=(qu/FS)(BL)=1221.55*1.25*2.00/3.00=1017.95kN

If Local Shear Failure Occur c"=0.67c'=0.67*20=13.4kPa
ϕ"=tan⁻¹(0.67tanϕ')=tan⁻¹(0.67tan25)=17.35 say ϕ"=17°

Nc=12.34 Nq=4.77 and Nγ=3.53

q=γDf=16.5*1.5=24.75kPa

γ=? Location of water table d=1.0m but B=1.25m 0<d=1.0<B=1.25m

γ"=γ'+(γ–γ')d/B=(19.81-9.81)+(16.5-(19.81-9.81))*(1.0/1.25)=15.2kN/m³

Shape Factors` ϕ'>0.0 and Df/B>1.5m/1.25 Df/B>1.0

Fcs=1+(1.25/2.0)*(4.77/12.34)=1.24
Fqs=1+(1.25/2.0)*tan17=1.19
Fγs=1-0.4*1.25/2.0=0.75

Fqd=1+2tan17(1-sin17)²tan⁻¹(1.5/1.25)=1.268

Fγd=1.0

Since no inclination in load mention,
Therefore, β=0.0 then Fci=Fqi=Fγi=1.0

qu=(13.40)(12.34)(1.24)(1.34)(1.00)+24.75(4.77)(1.19)(1.268)(1.00)
+0.50(15.2)(1.25)(3.53)(0.75)(1.00)(1.00)=478.05kPa
Qall=478.05*1.25*2.0/3.0=398.37 kN

Nc=12.34 Nq=4.77 and Nγ=3.53

q=γDf=16.5*1.5=24.75kPa

γ=? Location of water table d=1.0m but B=1.25m 0<d=1.0<B=1.25m

γ"=γ'+(γ–γ')d/B=(19.81-9.81)+(16.5-(19.81-9.81))*(1.0/1.25)=15.2kN/m³

Shape Factors` ϕ'>0.0 and Df/B>1.5m/1.25 Df/B>1.0

Fcs=1+(1.25/2.0)*(4.77/12.34)=1.24
Fqs=1+(1.25/2.0)*tan17=1.19
Fγs=1-0.4*1.25/2.0=0.75

$Fqd=1+2\tan17(1-\sin17)^2\tan^{-1}(1.5/1.25)=1.268$

Fγd=1.0 Fcd=1.268-(1.0-1.268)/(12.34*tan17)=1.3400

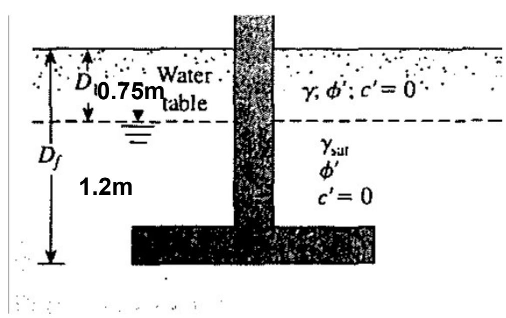
Since no inclination in load mention,
Therefore, β=0.0 then Fci=Fqi=Fγi=1.0

qu=(13.40)(12.34)(1.24)(1.34)(1.00)+24.75(4.77)(1.19)(1.268)(1.00)
+0.50(15.2)(1.25)(3.53)(0.75)(1.00)(1.00)=478.05kPa
Qall=478.05*1.25*2.0/3.0=398.37 kN

Example

A square foundation (B x B) has to be constructed as shown in Figure assume that $\gamma = 17 kN/m^3$, $\gamma_{sat} = 19.5\ kN/m^3$, $D_1 = 0.75m$, and $D_f = 1.2m$. The gross design allowable load, Q_{all}, with FS = 3 is 750 kN. The SPT values are. Determine B?

Depth (m)	N_{60} (Blows/ft)
1.0	4
1.5	6
3.0	10
4.0	5

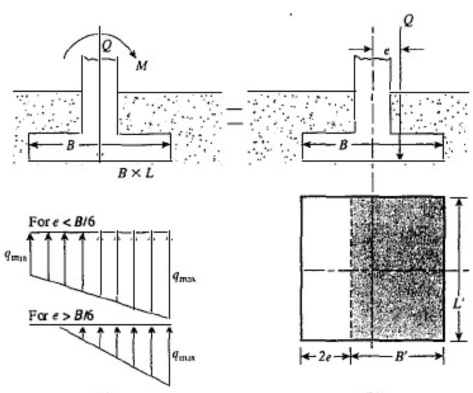


Depth (m)	N_{60} (Blows/ft)	σ' (kPa)	ϕ' (o))
1.0	4	0.75*17+0.25(19.5-9.81)=15.17	4
1.5	6	0.75*17+0.75(19.5-9.81)=20.02	6
3.0	10	0.75*17+2.25(19.5-9.81)=34.55	10
4.0	5	0.75*17+3.25(19.5-9.81)=44.24	5

Bearing Capacity of Soils on Eccentrically Loaded Footings

Foundations with a One-Way Eccentricity.

- In most instances, foundations are subjected to moments in addition to the vertical load as shown below.
- In such cases the distribution of pressure by the foundation upon the soil is not uniform.



The effective width is now,
$B' = B - 2e$
whereas the effective length is Still,
$L' = L$

The distribution of the nominal (contact) pressure

$$q_{max} = \frac{Q}{BL} + \frac{6M}{B^2L}$$

$$q_{min} = \frac{Q}{BL} - \frac{6M}{B^2L}$$

where *Q* is the total vertical load and *M* is the moment on the footing in one axis.

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

Substituting equation in equations above Eqs. yields:

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

Notes

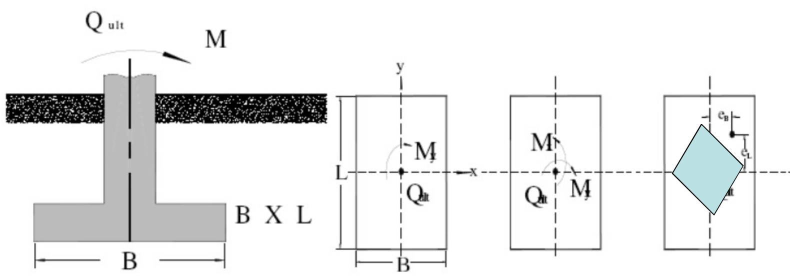
- Note that in these equations,
  - when the eccentricity e becomes B/6, q_{min} is zero.
  - For e >B/6, q_{min} will be negative, which means that tension will develop. Because soils can sustain very little tension, there will be a separation between the footing and the soil under it.
  - The value of q_{max} is then

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

- Also note that the eccentricity tends to decrease the load bearing capacity of a foundation.

Foundations with Two-way Eccentricities

Consider a footing subject to a vertical ultimate load Qult and a moment M as shown in Figures. For this case, the components of the moment M about the x and y axis are Mx and My respectively. This condition is equivalent to a load Q placed eccentrically on the footing with x = eB and y = eL



$$e_B = \frac{M_y}{Q_{ult}} \quad and \quad e_L = \frac{M_x}{Q_{ult}}$$

Modification for General Bearing Capacity

The general bearing capacity equation is therefore modified to,

$$q'_u = c'N_cF_{cs}F_{cd}F_{ci} + qN_qF_{qs}F_{qd}F_{qi} + \tfrac{1}{2}\gamma B'N_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i}$$

$$Q_{ult} = q'_u \overbrace{(B')(L')}^{A'} \qquad \boxed{B' = B - 2e_y} \qquad \boxed{L' = L - 2e_x}$$

- As before, to evaluate Fcs , Fqs , and Fγs , use the effective length (L') and the effective width (B') dimensions instead of L and B, respectively.
- To calculate Fcd , Fqd , and Fγd and ,do not replace B with B'.
- The factor of safety against bearing capacity failure is FS $=Q_{ult}/Q$
- Check the factor of safety against q_{max} or FS = q'_u/q_{max}
- Finally note we confine here our self to $e_L \leq L/6$ or $e_B \leq B/\mathbf{6}$

$q_{max.}$=(Qall/(BL))*(1+6e_B/B) **Eccentricity in B**

$q_{max.}$=(Qall/(BL))*(1+6e_L/L) **Eccentricity in L**

$q_{max.}$=(Qall/(BL))*(1+6e_B/B+6e_L/L) **Eccentricity in Both B & L**

Eccentricity in B

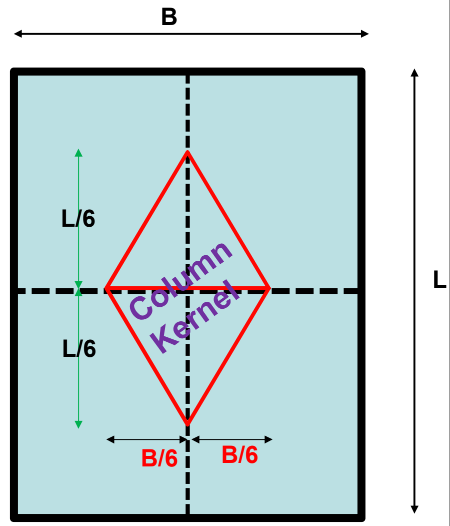
$q_{min.}$=(Qall/(BL))*(1-6e_B/B)

Eccentricity in L

$q_{max.}$=(Qall/(BL))*(1-6e_L/L)

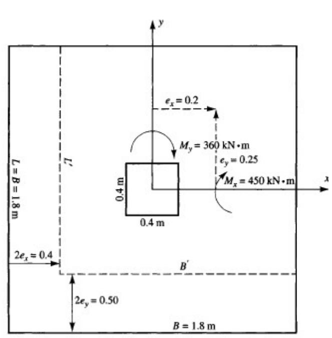
Eccentricity in Both B & L

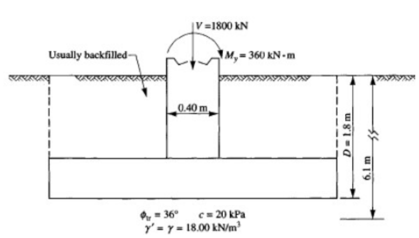
$q_{min.}$=(Qall/(BL))*(1-6e_B/B-6e_L/L)



Example

- A square footing is 1.8 X 1.8 m with a 0.4 X 0.4 m square column. It is loaded with an axial load of 1800 kN and *Mx* = 450 kN • m; *My* = 360 kN • m. Undrained triaxial tests (soil not saturated) give ϕ' = 36° and *c* = 20 kPa. The footing depth *D* = 1.85 m; the soil unit weight γ = 17.00 kN/m3; the water table is at a depth of 6.1 m from the ground surface. Determine the allowable bearing capacity





$$q_u' = c'N_cF_{cs}F_{cd}F_{ci} + qN_qF_{qs}F_{qd}F_{qi} + \frac{1}{2}\gamma B'N_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i}$$

$e_B=M_y$/axial load$=M_y/Q=360/1800=0.20$ m
$e_L= M_x$/axial load$=M_x/Q=450/1800=0.25$ m

$B'=B-2e_B=1.80-2*0.20=1.40$m

$L'=L-2e_L=1.80-2*0.25=1.30$m

But always B'<L', therefore, B'=1.30 m L'=1.40m

$\phi'=36$ $N_c=50.59$; $N_q=37.75$ and $N_\gamma=56.31$

Shape Factors

$$F_{cs} = 1 + \frac{B}{L}\frac{N_q}{N_c} \qquad F_{qs} = 1 + \frac{B}{L}\tan\phi \qquad F_{\gamma s} = 1 - 0.4\frac{B}{L}$$

$F_{cs}=1+(1.30/1.40)*(37.75/50.59)=1.693$

$F_{qs}=1+(1.3/1.4)\tan 36=1.675$

$F_{\gamma s}=1-0.4(1.3/1.4)=0.63$

Depth Factors:

$\phi'>0$ and $D_f=1.85>1.80$m

Depth Factors for Df/B >1.

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan\phi'} \qquad F_{qd} = 1 + 2\tan\phi(1-\sin\phi)^2 \tan^{-1}\frac{D_f}{B} \qquad F_{\gamma d} = 1$$

$F_{qd}=1+2\tan 36(1-\sin 36)^2\tan^{-1}(1.85/1.80)=1.197$

$F_{cd}=1.197-(1-1.197)/(50.59*\tan 36)=1.20$

$F_{\gamma d}=1.00$

NO inclination in the load therefore, $F_{ci}=F_{qi}=F_{\gamma i}=1.00$

GWT depth =6.1m from surface d=(6.1-1.85)> B=1.80m
therefore GWT has no effect.

$q=\gamma Df=17.00(1.85)=31.45$kPa

qu'=(20)(50.59)(1.693)(1.20)(1.00)+31.45(37.75)(1.675)(1.197)(1.00)
+0.5(17.00)(1.300)(56.31)(0.63)(1.00)(1.00)=4827.96kPa

Qu'=qu'(B'L')=4827.96(1.30*1.40)=8786.89kN

$q_{max.}$=(Qall/(BL))*(1+6e_B/B+6e_L/L)
=(1800/(1.80*1.80))(1+6*0.25/1.8+6*0.2/1.80)=1388.9kPa

$q_{min.}$=(Qall/(BL))*(1-6e_B/B-6e_L/L)
=(1800/(1.80*1.80))(1-6*0.25/1.8-6*0.2/1.80)=-277.8kPa

Required allowable bearing capacity

F.S=Qu'/Q design=8786.89/1800=4.9

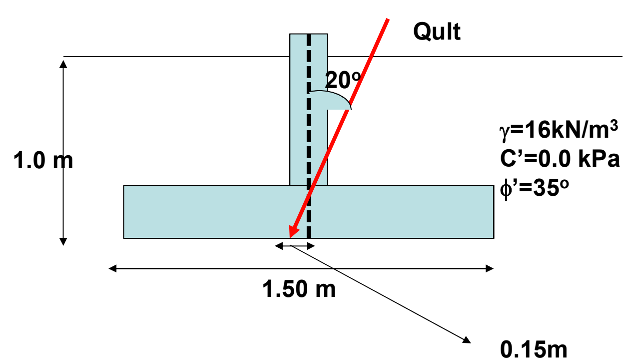
F.S=qu'/qmax=4827.96kPa/1388.9kPa=3.50

qmin<0.0 at least qmin=0.0

1=6e_B/B+6e_L/L since footing is square and Neither e_L nor e_B function of dimension! Thus B=L

1=6(0.2/B+0.25/B)=(0.45)*6/B B=2.7 m!

A continuous (wall footing) is shown below.
Estimate the ultimate load per unit length of the foundation



$$q_u' = c'N_cF_{cs}F_{cd}F_{ci} + qN_qF_{qs}F_{qd}F_{qi} + \frac{1}{2}\gamma B'N_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i}$$

Sol.

$C_{,}=0.0$; $\phi'=35^o$ Nc=46.12; Nq=33.30; $N\gamma=48.03$

$Q=\gamma Df=16$ kPa

Fcs=Fqs=Fγs=1.0 since it is a wall footing!!!!

Depth Factors for $D_f / B \le 1$.

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan \phi'}$$ $$F_{qd} = 1 + 2\tan\phi(1-\sin\phi)^2 \frac{D_f}{B}$$ $$F_{\gamma d} = 1$$

$Fqd=1+2\tan35(1-\sin35)^2(1.00/1.50)=1.1697$

$F\gamma d=1.0$

Inclination Factors with β is the inclination of load with respect to the vertical.

$$F_{ci} = F_{qi} = \left(1-\frac{\beta^o}{90^o}\right)^2$$ $$F_{\gamma i} = \left(1-\frac{\beta}{\phi}\right)^2$$

$Fqi=(1-20/90)^2=0.6049$

$F\gamma i=(1-20/35)^2=0.18367$

$B'=B-2e_B=1.5-2*0.15=1.20m$

$q_{ult}=0.0+(16)(33.30)(1.00)(1.1697)(0.6049)$
$+0.5(16.00)(1.20)(48.03)(1.00)(1.00)(0.18367)=461.67kPa$

$Q_{ult}=q_{ult}(area)=461.67(1.200)=554.00$ kN/m

Bearing Capacity For Footings On Layered Soils

- There are three general cases of the footing on a layered soil as follows:

Case 1. Footing on layered clays (all $\phi = 0$) as in Fig..

a. Top layer weaker than lower layer *($c1 < c2$)*

b. Top layer stronger than lower layer *($c1 > c2$)*

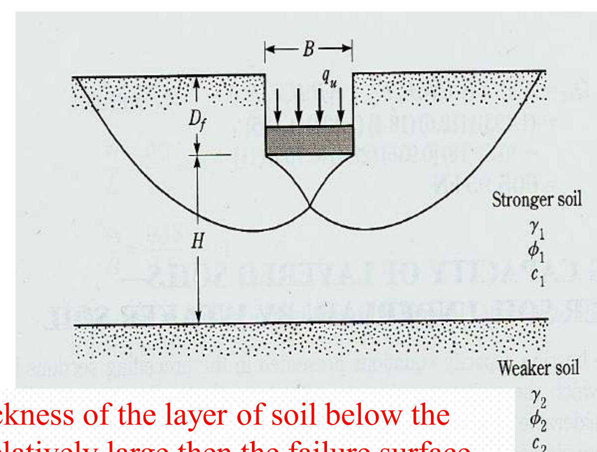
Case 2. Footing on layered ϕ-c soils with *a, b* same as case 1.

Case 3. Footing on layered sand and clay soils as in Fig.

a. Sand overlying clay

b. Clay overlying sand

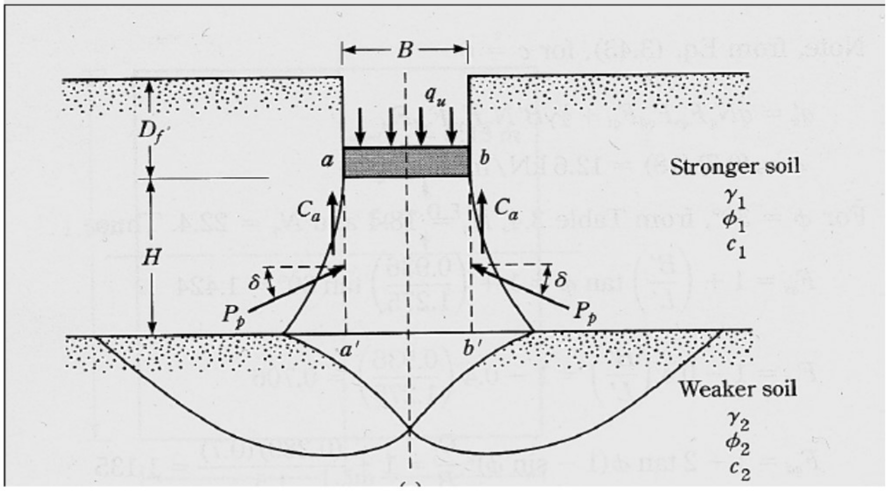
Stronger Soil Is Underlain By A Weaker Soil -1



If H, the thickness of the layer of soil below the footing, is relatively large then the failure surface will be completely located in the top soil layer, which is the upper limit for the ultimate bearing capacity.

Stronger Soil Is Underlain By A Weaker Soil -II

If H is small compared to the foundation width B, a punching shear failure will occur in the top soil layer followed by a general shear failure in the bottom soil layer.



In this condition, where the stronger surface soil is underlain by a weaker stratum, the ***general Bearing capacity*** equation is modified to,

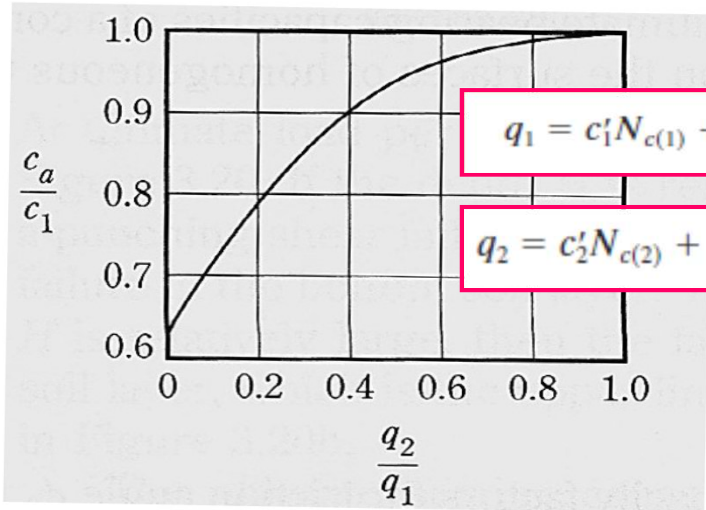
$$q_u = q_b + \left(1+\frac{B}{L}\right)\left(\frac{2c_aH}{B}\right) + \gamma_1 H^2\left(1+\frac{B}{L}\right)\left(1+\frac{2D_f}{H}\right)\left(\frac{K_s\tan\phi_1}{B}\right) - \gamma_1 H < q_t$$

$$q_b = c_2 N_{c(2)} F_{cs(2)} + \gamma_1 (D_f + H) N_{q(2)} F_{qs(2)} + \frac{1}{2}\gamma_2 B N_{\gamma(2)} F_{\gamma s(2)}$$

$$q_t = c_1 N_{c(1)} F_{cs(1)} + \gamma_1 D_f N_{q(1)} F_{qs(1)} + \frac{1}{2}\gamma_1 B N_{\gamma(1)} F_{\gamma s(1)}$$

where, $\mathbf{c_a}$ is the adhesion, $\mathbf{K_s}$ is the punching shear coefficient, $\mathbf{q_t}$ is the bearing capacity of the top soil layer, $\mathbf{q_b}$ is the bearing capacity of the bottom soil layer, **H** is the height of top layer, ϕ_1 is the angle of internal friction of top soil and ϕ_2 for the bottom soil.

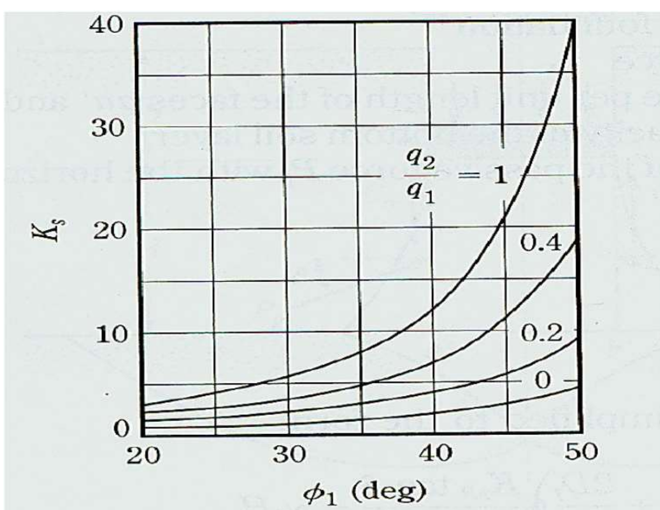
Ca determination



$$q_1 = c_1' N_{c(1)} + \tfrac{1}{2}\gamma_1 B N_{\gamma(1)}$$

$$q_2 = c_2' N_{c(2)} + \tfrac{1}{2}\gamma_2 B N_{\gamma(2)}$$

punching shear coefficient ***Ks***



The Other Cases

1. The top layer is strong, and the bottom layer is a saturated soft clay ($\phi = 0$);

2. The top layer is stronger sand and the bottom layer is a weaker sand ($c1 = 0$) ($c2 = 0$);

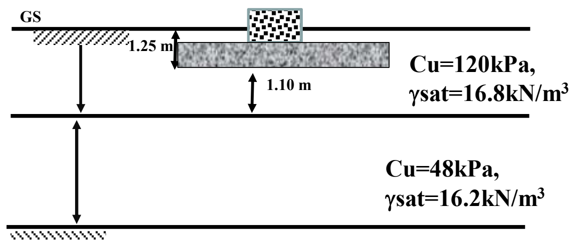
3. The top layer is a stronger saturated clay ($\phi 1 = 0$), and the bottom layer is weaker saturated clay ($\phi 2 = 0$).

Use the same method before
and apply corrections were needed

Example

A foundation 1.5 m by 1 m is placed at a depth of 1 m in a stiff clay. A softer clay layer is located at a depth of 1 m measured from the bottom of the foundation. For the top layer, the un-drained shear strength is 120 kN/m^2, the unit weight is 16.8 kN/m^3, and for the bottom layer the un-drained shear strength is 48 kN/m^2, and the unit weight is 16.2 kN/m^3. Find the allowable bearing capacity for this footing if FS=3.0.

A foundation 1.5 m by 1 m is placed at a depth of 1.25 m in a stiff clay. A softer clay layer is located at a depth of 1.1 m measured from the bottom of the foundation. For the top layer, the un-drained shear strength is 120 kN/m², the unit weight is 16.8 kN/m³, a and for the bottom layer the un-drained shear strength is 48 kN/m³, and the unit weight is 16.2 kN/m². Find the allowable bearing capacity for this footing if FS=3.0.



Solution:

H=1.1m>1.0 m =B, therefore two-layers soil

$$q_t = c_1 N_{c(1)} F_{cs(1)} + \gamma_1 D_f N_{q(1)} F_{qs(1)} + \frac{1}{2}\gamma_1 B N_{\gamma(1)} F_{\gamma s(1)}$$

For top layer soil in undrained condition **ϕu=0.0, Nc=5.14, Nq=1.00, Nγ=0.0**

Shape Factors

$$F_{cs} = 1 + \frac{B}{L}\frac{N_q}{N_c} \qquad F_{qs} = 1 + \frac{B}{L}\tan\phi \qquad F_{\gamma s} = 1 - 0.4\frac{B}{L}$$

Fcs=1+(1.0/1.5)(1.0/5.14)=1.13 **Fqs=1+(1/1.5)tan0.0=1.00** **Fγs=1-0.4*(1/1.5)=0.733**

qt=120.0(5.14)(1.13)+(16.8)(1.25)(1.00)+0.0=717.98kPa

$$q_b = c_2 N_{c(2)} F_{cs(2)} + \gamma_1 (D_f + H) N_{q(2)} F_{qs(2)} + \frac{1}{2}\gamma_2 B N_{\gamma(2)} F_{\gamma s(2)}$$

ϕu=0.0, Nc=5.14,
Nq=1.00, Nγ=0.0

Fcs=1+(1.0/1.5)(1.0/5.14)=1.13
Fqs=1+(1/1.5)tan0.0=1.00
Fγs=1-0.4*(1/1.5)=0.733

qb=(48)(5.14)(1.13)+16.8(1.25+1.1)(1)(1)+0.00=318.27kPa

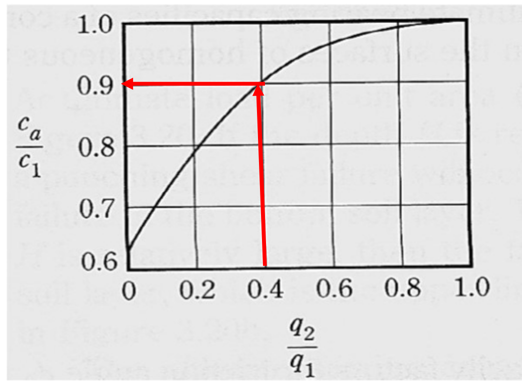
$$q_u = q_b + \left(1+\frac{B}{L}\right)\left(\frac{2c_aH}{B}\right) + \gamma_1 H^2\left(1+\frac{B}{L}\right)\left(1+\frac{2D_f}{H}\right)\left(\frac{K_s \tan\phi_1}{B}\right) - \gamma_1 H < q_t$$

$$q_1 = c_1' N_{c(1)} + \tfrac{1}{2}\gamma_1 B N_{\gamma(1)}$$

$$q_2 = c_2' N_{c(2)} + \tfrac{1}{2}\gamma_2 B N_{\gamma(2)}$$

q1=120(5.14)+0
q2=48(5.14)+0
q2/q1=48/120=0.4

ca/c1=0.90
ca=0.9*120=108kPa

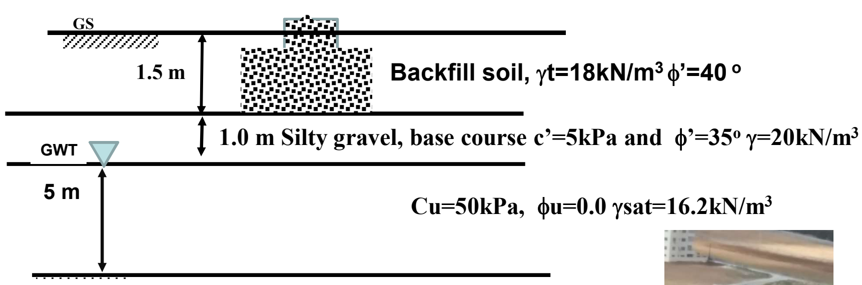


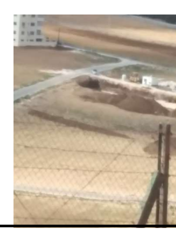
qu=318.27+(1+1/1.5)(2*108*1.1/1.00)
+0.00-16.8(1.1)<717.98
qu=695.8 kPa<717.98kPa

∴ qu=695.8kPa
qall=qu/FS=695.8/3=231.93kPa

Example 2: Layer Soil

1) Determine allowable bearing capacity for original soil if FS=3
2) If a soil replacement was done as in the figure above what is the new allowable bearing capacity for the footing. Note the footing is square and has B=2.0m





Sol.

1) **The original footing assumes to be 2.0 m in a depth of 1.5 m in the soft clay**

ϕu=0.0, Nc=5.14, Nq=1.00, Nγ=0.0

$$q_u = c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

No inclination Fci=Fqi=Fγi

C=Cu=50kPa;

Shape Factors

$$F_{cs} = 1 + \frac{B}{L}\frac{N_q}{N_c} \qquad F_{qs} = 1 + \frac{B}{L}\tan\phi \qquad F_{\gamma s} = 1 - 0.4\frac{B}{L}$$

Fcs=1+(2.0/2.0)(1.0/5.14)=1.195
Fqs=1+(1/1.5)tan0.0=1.00
Fγs=1-0.4*(2/2)=0.60

ϕu=0

For $\phi = 0$

$D_f / B \le 1.$

$$F_{cd} = 1 + 0.4\frac{D_f}{B}$$

Fcd=1+0.4(1.5/2.0)=1.30, Fqd=1.0

qu=(50)(5.14)(1.195)(1.30)(1.00)+(16.2*1.5)(1)(1)(1)(1)+0

qu=423.55kPa **qall=423.55/3=141.18kPa**

-2-If soil replacement is used

H=1.0 <B=2.0m footing in layer soil (note compare with ~1.5B)

$$q_u = q_b + \left(1+\frac{B}{L}\right)\left(\frac{2c_a H}{B}\right) + \gamma_1 H^2\left(1+\frac{B}{L}\right)\left(1+\frac{2D_f}{H}\right)\left(\frac{K_s \tan\phi_1}{B}\right) - \gamma_1 H < q_t$$

qb=same as before without depth factor

C=Cu=50kPa;

Shape Factors

$$F_{cs} = 1 + \frac{B}{L}\frac{N_q}{N_c} \qquad F_{qs} = 1 + \frac{B}{L}\tan\phi \qquad F_{\gamma s} = 1 - 0.4\frac{B}{L}$$

Fcs=1+(2.0/2.0)(1.0/5.14)=1.195
Fqs=1+(1/1.5)tan0.0=1.00

Fγs=1-0.4*(2/2)=0.60

$$q_b = c_2 N_{c(2)} F_{cs(2)} + \gamma_1 (D_f + H) N_{q(2)} F_{qs(2)} + \frac{1}{2}\gamma_2 B N_{\gamma(2)} F_{\gamma s(2)}$$

qb=(50)(5.14)(1.195)+(γf*Df+γ1H)(1)(1)(1)(1)+0

qb=(50)(5.14)(1.195)+(18*1.5+20.0*1.0)(1)(1)(1)(1)+0
qb=354.12kPa

$$q_t = c_1 N_{c(1)} F_{cs(1)} + \gamma_1 D_f N_{q(1)} F_{qs(1)} + \frac{1}{2}\gamma_1 B N_{\gamma(1)} F_{\gamma s(1)}$$

c'=5kPa, ϕ'=35°. γ=20kPa

ϕ'=35° Nc=46.12 Nq=33.3 Nγ=48.03

Shape Factors

$$F_{cs} = 1 + \frac{B}{L}\frac{N_q}{N_c} \qquad F_{qs} = 1 + \frac{B}{L}\tan\phi \qquad F_{\gamma s} = 1 - 0.4\frac{B}{L}$$

Fcs=1+(2/2)(33.3/46.12)=1.722
Fqs=1+(2/2)tan35=1.700
Fγs=1-0.4(2/2)=0.600

$$q_t = c_1 N_{c(1)} F_{cs(1)} + \gamma_1 D_f N_{q(1)} F_{qs(1)} + \frac{1}{2}\gamma_1 B N_{\gamma(1)} F_{\gamma s(1)}$$

qt=(5)(46.12)(1.722)+18(1.5)(33.3)(1.70)+0.5(20.)(2.0)(48.03)(0.6)
=2502 kPa

$$q_u = q_b + \left(1+\frac{B}{L}\right)\left(\frac{2c_a H}{B}\right) + \gamma_1 H^2\left(1+\frac{B}{L}\right)\left(1+\frac{2D_f}{H}\right)\left(\frac{K_s \tan\phi_1}{B}\right) - \gamma_1 H < q_t$$

$$q_1 = c_1' N_{c(1)} + \tfrac{1}{2}\gamma_1 B N_{\gamma(1)}$$

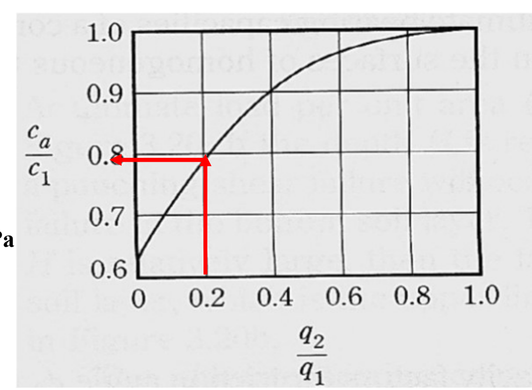
q1=(5)(46.12)+0.5*20*2.0*48.03=1191.2kPa

$$q_2 = c_2' N_{c(2)} + \tfrac{1}{2}\gamma_2 B N_{\gamma(2)}$$

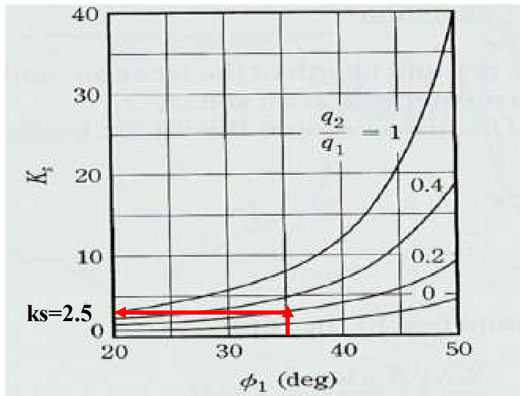
q2=50(5.14)+0=257.0 kPa

q2/q1=257/1191.2=0.216

ca/c1=0.78 ca=0.78*5=3.90 kPa



punching shear coefficient ***Ks***



$$q_u = q_b + \left(1+\frac{B}{L}\right)\left(\frac{2c_a H}{B}\right) + \gamma_1 H^2 \left(1+\frac{B}{L}\right)\left(1+\frac{2D_f}{H}\right)\left(\frac{K_s \tan\phi_1}{B}\right) - \gamma_1 H < q_t$$

qu=354.12+(1+2/2)(2*3.9*1)/2)+18(1)^2(1+2/2)(1+2*1.5/1)(2.5*tan35/2.0) -18*1<2502

qu=460.13kPa<2502kPa then qu=460.13kPa qall=460.13.2/3=153.376kPa

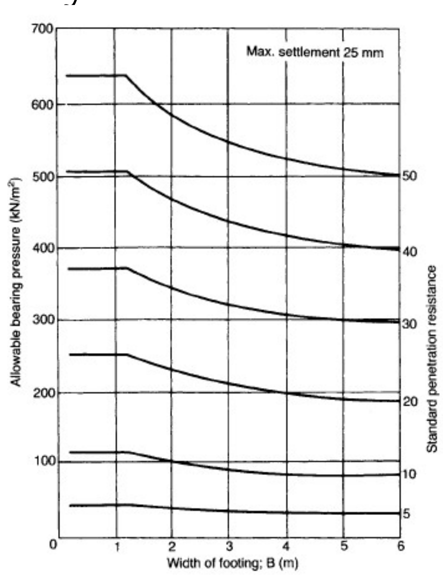
HW (next Tuesday) : Determine the thickness of the replaced layer to achieve 250 kPa allowable bearing capacity.

Bearing Capacity From SPT

- Two Ways:
  1. Using the correlation to find ϕ'and using the general bearing capacity equation
  2. Using the following chart (for surface footing)

Bearing Capacity From SPT

Allowable bearing capacity for **surface-loaded** footings with settlement limited to approximately 25 mm.



Bearing Capacity From SPT

$$q_{net(all)} = 19.16 N_{60} F_d \left(\frac{S_a}{25.4}\right)$$ For B≤1.22 m

$$q_{net(all)} = 11.98 N_{60} \left(\frac{3.28B + 1}{3.28B}\right)^2 F_d \left(\frac{S_a}{25.4}\right)$$ For B≥1.22 m

Where

$q_{net(all)} = q_{all} - \gamma D_f$ kN/m^2

Sa: tolerable settlement in mm

Fd=depth factor=1+0.33(Df/B)≤1.33

Bearing Capacity Using The Cone Penetration Test (CPT)

$$q_{net(all)} = \left(\frac{q_c}{15}\right)$$ For B≤1.22 m

$$q_{net(all)} = \left(\frac{q_c}{25}\right)\left(\frac{3.28B + 1}{3.28B}\right)^2$$ For B≥1.22 m

Where

$q_{net(all)} = q_{all} - \gamma D_f$ kN/m^2

The Bearing Capacity of Mat Foundations

Mat foundations must be designed to limit their settlements to a tolerable amount.

The ultimate bearing capacity of a soil supporting a mat foundation can be computed from,

$$q_u = c\,N_c\;F_{cs}\,F_{cd}\,F_{ci} + \gamma D_f N_q\;F_{qs}\,F_{qd}\,F_{qi} + \gamma B\,N_\gamma\,F_{\gamma s}\,F_{\gamma d}\,F_{\gamma i}$$

When $\phi = 0$ use,

$$q_u = 5.14 c_u\left(1 + 0.195\frac{B}{L}\right)\left(1 + 0.4\frac{D_f}{B}\right)$$

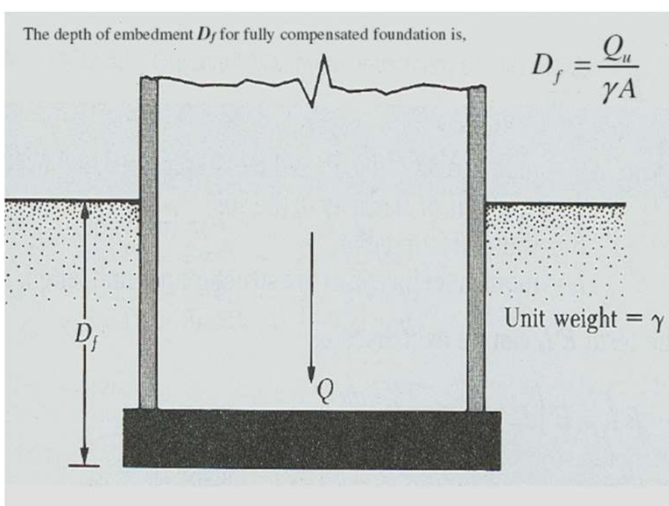
where c_u is the un-drained cohesion. When using corrected SPT values, the ***allowable bearing capacity*** may be calculated by,

$$q_{all} = 11.98\,N\left(1 + 0.33\frac{D_f}{B}\right)\left(\frac{s}{25.4}\right) < 15.93\left(\frac{s}{25.4}\right)$$

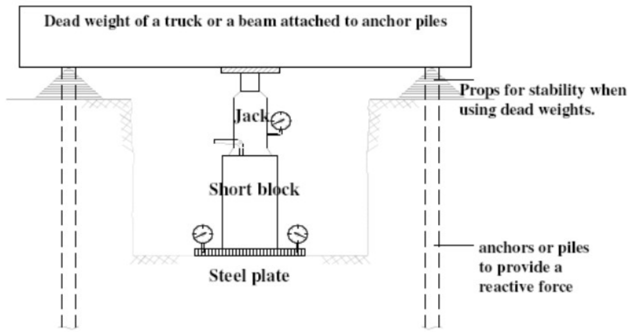
where N is the corrected standard penetration resistance, and s is the settlement in millimeters.

$$q_{all}\,(in\ kN/m^2) \simeq 36N(1 + 0.33D_f)\left(\frac{\Delta}{25.4}\right)$$

Compensation Mat Foundation



Bearing Capacity for Field Load Tests



Several dial gauges attached to an independent suspension system to record plate settlements with each increment of the jack load.

Bearing Capacity Based On Building Codes (Presumptive Pressure)

Presumptive bearing capacities from indicated building codes, kPa

Soil descriptions vary widely between codes. The following represents author's interpretations.

Soil description	Chicago, 1995	Natl. Board of Fire Underwriters, 1976	BOCA,* 1993	Uniform Bldg. Code, 1991†
Clay, very soft	25			
Clay, soft	75	100	100	100
Clay, ordinary	125			
Clay, medium stiff	175	100		100
Clay, stiff	210		140	
Clay, hard	300			
Sand, compact and clean	240		140	200
Sand, compact and silty	100			
Inorganic silt, compact	125			
Sand, loose and fine			140	210
Sand, loose and coarse, or sand-gravel mixture, or compact and fine		140 to 400	240	300
Gravel, loose and compact coarse sand	300		240	300
Sand-gravel, compact			240	300
Hardpan, cemented sand, cemented gravel	600	950	340	
Soft rock				
Sedimentary layered rock (hard shale, sandstone, siltstone)			6000	1400
Bedrock	9600	9600	6000	9600

Note: Values converted from psf to kPa and rounded.

*Building Officials and Code Administrators International, Inc.

Safety Factors In Foundation Design

There are more uncertainties in determining the allowable strength of the soil than in the superstructure elements. These may be summarized as follows:

- Complexity of soil behavior
- Lack of control over environmental changes after construction
- Incomplete knowledge of subsurface conditions
- Inability to develop a good mathematical model for the foundation
- Inability to determine the soil parameters accurately

Safety Factors In Foundation Design

These uncertainties and resulting approximations have to be evaluated for each site and a suitable safety factor directly (or indirectly) assigned that is not overly conservative but that takes into account at least the following:

1. Magnitude of damages (loss of life, property damage, and lawsuits) if a failure results
2. Relative cost of increasing or decreasing SF
3. Relative change in probability of failure by changing SF
4. Reliability of soil data
5. Changes in soil properties from construction operations, and later from any other causes
6. Accuracy of currently used design/analysis methods

Safety Factors Usually Used

- **Values of stability numbers (or safety factors) usually used**
- It is customary to use overall safety factors on the order of those shown in Table. Shear should be interpreted as bearing capacity for footings.

Failure mode	Foundation type	SF
Shear	Earthworks	
	Dams, fills, etc.	1.2–1.6
Shear	Retaining structure	
	Walls	1.5–2.0
Shear	Sheetpiling cofferdams	1.2–1.6
	Temporary braced excavations	1.2–1.5
Shear	Footings	
	Spread	2–3
	Mat	1.7–2.5
	Uplift	1.7–2.5
Seepage	Uplift, heaving	1.5–2.5
	Piping	3–5

Bearing Capacity Of Rock

Range of properties for selected rock groups; data from several sources

Type of rock	Typical unit wt., kN/m³	Modulus of elasticity E, MPa × 10³	Poisson's ratio, μ	Compressive strength, MPa
Basalt	28	17–103	0.27–0.32	170–415
Granite	26.4	14–83	0.26–0.30	70–276
Schist	26	7–83	0.18–0.22	35–105
Limestone	26	21–103	0.24–0.45	35–170
Porous limestone		3–83	0.35–0.45	7–35
Sandstone	22.8–23.6	3–42	0.20–0.45	28–138
Shale	15.7–22	3–21	0.25–0.45	7–40
Concrete	15.7–23.6	Variable	0.15	15–40

*Depends heavily on confining pressure and how determined; E = tangent modulus at approximately 50 percent of ultimate compression strength.

the bearing-capacity factors for sound rock are approximately

$$N_q = \tan^6\left(45° + \frac{\phi}{2}\right) \qquad N_c = 5\tan^4\left(45° + \frac{\phi}{2}\right) \qquad N_\gamma = N_q + 1$$

$$q'_{ult} = q_{ult}(\text{RQD})^2$$

Bearing Capacity Of Rock

ROCK SAMPLING

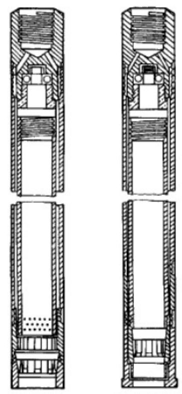
- blow counts are at the refusal level *(N >* 100)→Use Rock cores

Typical standard designation and sizes for rock drill casing (barrel) and bits*

Casing OD, mm		Core bit OD, mm		Bit ID, mm
RW	29	EWT	37	23
EW	46	AWT	48	32
AW	57	BWT	60	44
BW	73	NWT	75	59
NW	89	HWT	100	81
PW	140		194	152

* See ASTM D 2113 for the complete range in core bit, casing, and drill rod sizes in current use. Sizes are nominal—use actual diameter of recovered core.

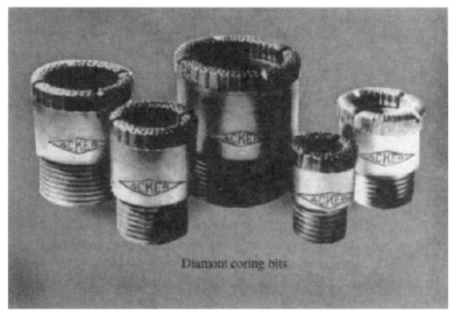
Rock coring equipment



Standard double-tube core barrel

Series "M" double-tube core barrel

(*a*) Core barrels to collect rock cores



(*b*) Coring bits to attach to core barrel. (The Acker Drill Company)

Rock quality designation

- Rock quality designation (RQD) is an index or measure of the quality of a rock mass used by many engineers. RQD is computed from recovered core samples as

$$\boxed{\mathrm{RQD} = \frac{\sum \text{Lengths of intact pieces of core} > 100\text{ mm}}{\text{Length of core advance}}}$$

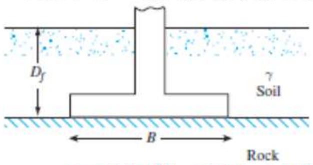
Table 2.8 Relation between *in situ* Rock Quality and RQD

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

A square column foundation B xB = 2.5 m x 2.5 m is to be constructed in a depth of 2.0 m over sandstone has c'= 32 MN/m^2 , ϕ'= 30° γ = 25 kN/m^3 and RDQ = 50%. Back fill soil has γ = 17 kN/m3

Estimate the allowable load-bearing capacity.

Use FS = 4. Also, for concrete, use fc= 30 MN/m2.



Sol.

In Footing on Rock use Terzaghi's B.C Equations

$$q_u = 1.3c\,N_c + \bar{q}\,N_q + 0.4\,\gamma B\,N_\gamma$$ **(for square footings, typical of interior columns)**

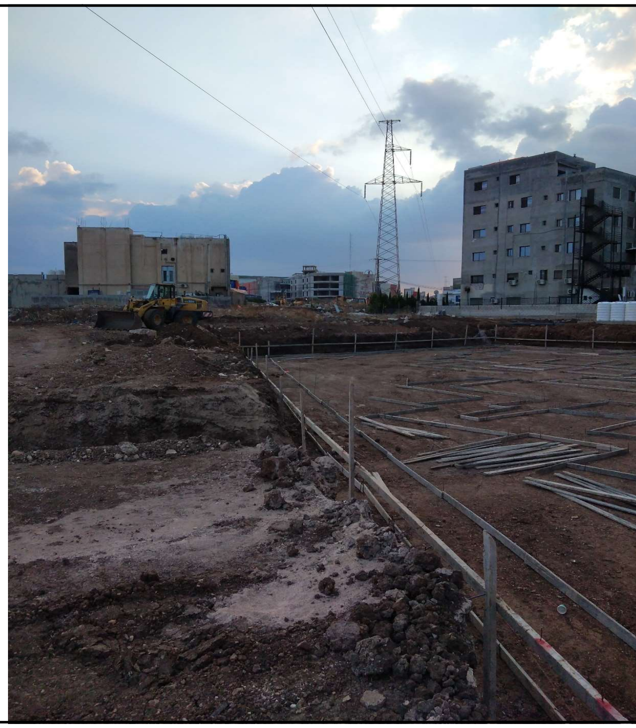
the bearing-capacity factors for sound rock are approximately

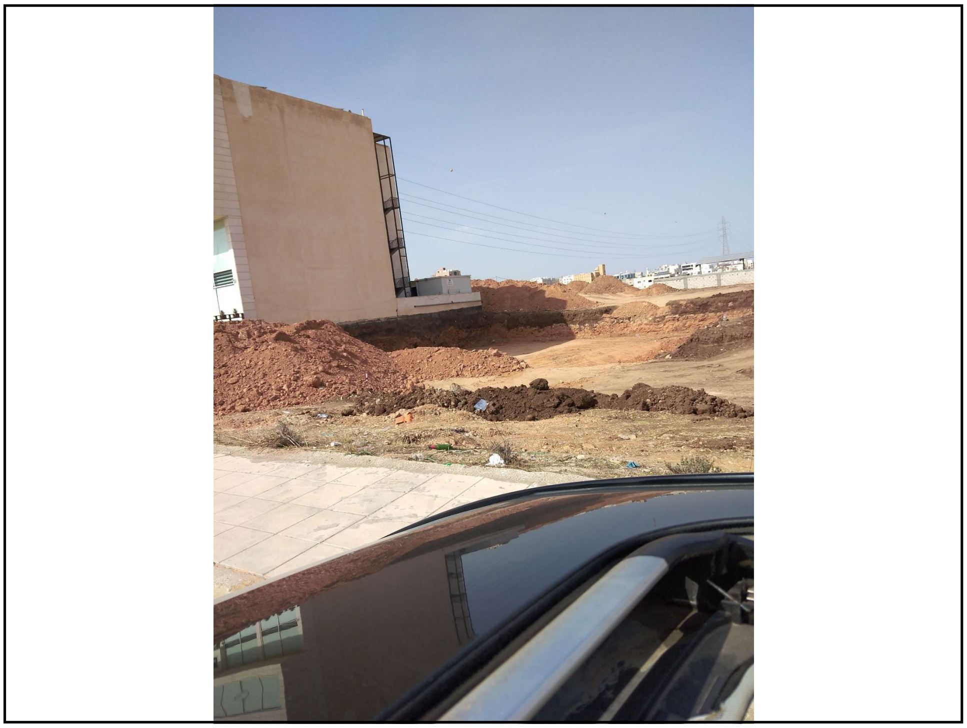
$$N_q = \tan^6\left(45° + \frac{\phi}{2}\right) \qquad N_c = 5\tan^4\left(45° + \frac{\phi}{2}\right) \qquad N_\gamma = N_q + 1$$

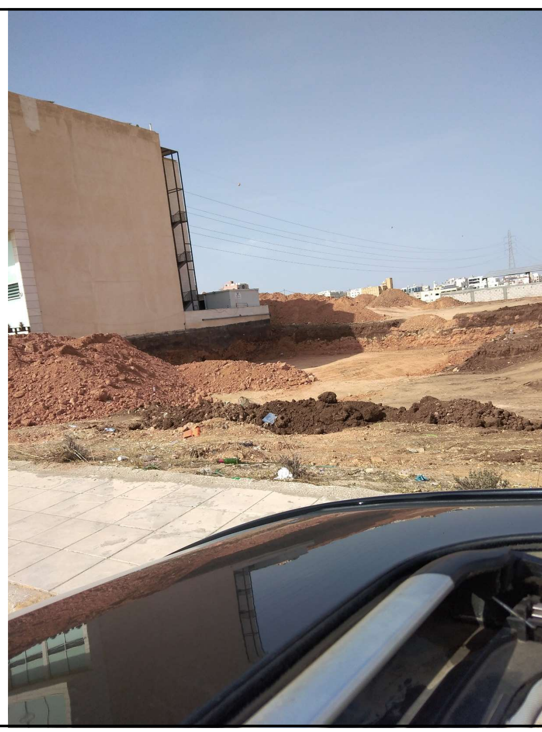
$N_q=\tan^6(45+30/2)=27.0$ $N_c=5\tan^4(45+30/2)=45.0$ $N_\gamma=28.0$

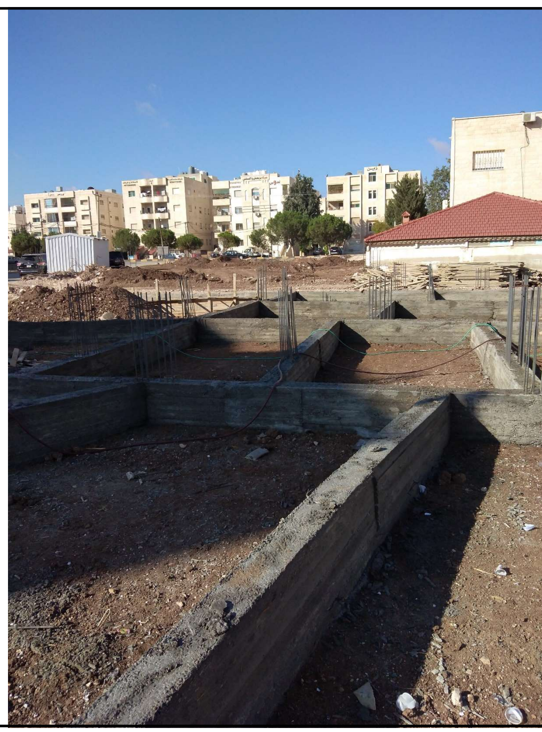
qu=1.3*32*1000*45.0+(17*2)*27.0+0.4*25*2.5*28.0=1873618 kPa

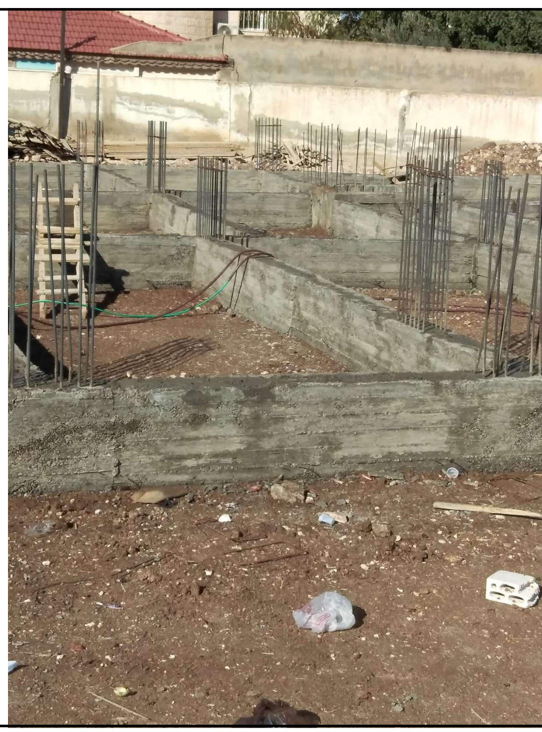
qu'=qu*(RQD)^2=1873618*0.5^2=468.4MPa **qall=468.4/4=117.1MPA**











References

1. J. Bowles, “Foundation Analysis and Design”, McGraw-Hill;
2. B. Das, “Principles of foundation Engineering”, Thompson;
3. Coduto, “Foundations Design”, Prentice Hall;

Foundation Settlements

Dr. Omar Al Hattamleh

Foundation settlements

- Foundation settlements must be estimated with great care for buildings, bridges, towers, power plants, and similar high-cost structures.
- For structures such as fills, earth dams, levees, braced sheeting, and retaining walls a greater margin of error in the settlements can usually be tolerated

problems with soil settlement analyses

There are two major problems with soil settlement analyses:

1. *Obtaining reliable values of the "elastic" parameters*. Problems of recovering "undisturbed" soil samples mean that laboratory values are often in error by 50 percent or more. There is now a greater tendency to use in situ tests, but a major drawback is they tend to obtain horizontal values. *Anisotropy* is a common occurrence, making vertical elastic values (usually needed) different from horizontal ones. **Often the difference is substantial**. Because of these problems, correlations are commonly used, particularly for preliminary design studies. More than one set of elastic parameters must be obtained (or estimated) if there is *stratification* in the zone of influence *H*.
2. *Obtaining a reliable stress profile from the applied load*. We have the problem of computing both the correct numerical values and the effective depth *H* of the influence zone. Theory of Elasticity equations are usually used for the stress computations, with the influence depth *H* below the loaded area taken from $H = 0$ to $H \rightarrow \infty$ (but more correctly from 0 to about *4B* or *5B)*. Since the Theory of Elasticity usually assumes an isotropic, homogeneous soil, agreement between computations and reality is often a happy coincidence.

Settlements are usually classification

1. *Immediate,* or those that take place as the load is applied or within a time period of about 7 days.
2. *Consolidation,* or those that are time-dependent and take months to years to develop. The Leaning Tower of Pisa in Italy has been undergoing consolidation settlement for over 700 years. The lean is caused by the consolidation settlement being greater on one side. This, however, is an extreme case with the principal settlements for most projects occurring in 3 to 10 years.

Stresses Distribution

A) Approximate method
- 2:1 Method

B) Elasticity theory
- a) Under point Load
- b) Under rectangular (square) area
- c) Under circular area
- d) Under embankment
- e) Under wedge
- f) From any uniform shape

Approximate Method 2:1 method

Strip Load Width B

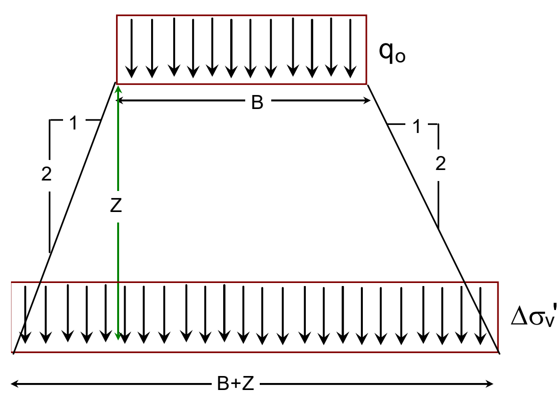
$$\Delta\sigma'_v = \frac{q_o(Bx1)}{(B+Z)}$$

Square Load Width B

$$\Delta\sigma'_v = \frac{q_o(BxB)}{(B+Z)(B+Z)}$$

Rectangular Load B X L

$$\Delta\sigma'_v = \frac{q_o(BxL)}{(B+Z)(L+Z)}$$

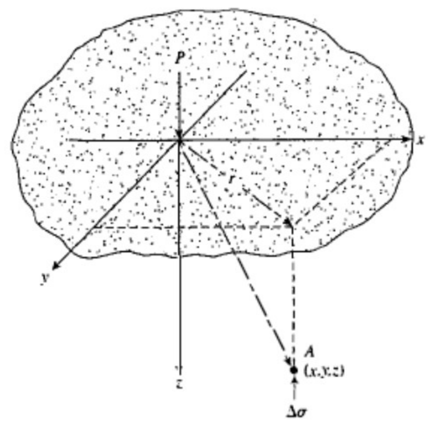


Point Load

the vertical stress increase at point A caused by a point load of magnitude P is given by

$$\Delta\sigma = \frac{3P}{2\pi z^2\left[1+\left(\frac{r}{z}\right)^2\right]^{5/2}}$$

$r = \sqrt{x^2 + y^2}$
x, y, z = coordinates of the point A



Stress Due to a Circularly Loaded Area

$$\Delta\sigma = \int d\sigma = \int_{\theta=0}^{\theta=2\pi}\int_{r=0}^{r=B/2} \frac{3(q_o r\, d\theta\, dr)}{2\pi z^2\left[1+\left(\frac{r}{z}\right)^2\right]^{5/2}}$$

$$= q_o\left\{1 - \frac{1}{\left[1+\left(\frac{B}{2z}\right)^2\right]^{3/2}}\right\}$$

Table 5.1 Variation of $\Delta\sigma/q_o$ for a Uniformly Loaded Flexible Circular Area

	$r/(B/2)$					
$z/(B/2)$	**0**	**0.2**	**0.4**	**0.6**	**0.8**	**1.0**
0	1.000	1.000	1.000	1.000	1.000	1.000
0.1	0.999	0.999	0.998	0.996	0.976	0.484
0.2	0.992	0.991	0.987	0.970	0.890	0.468
0.3	0.976	0.973	0.963	0.922	0.793	0.451
0.4	0.949	0.943	0.920	0.860	0.712	0.435
0.5	0.911	0.902	0.869	0.796	0.646	0.417
0.6	0.864	0.852	0.814	0.732	0.591	0.400
0.7	0.811	0.798	0.756	0.674	0.545	0.367
0.8	0.756	0.743	0.699	0.619	0.504	0.366
0.9	0.701	0.688	0.644	0.570	0.467	0.348
1.0	0.646	0.633	0.591	0.525	0.434	0.332
1.2	0.546	0.535	0.501	0.447	0.377	0.300
1.5	0.424	0.416	0.392	0.355	0.308	0.256
2.0	0.286	0.286	0.268	0.248	0.224	0.196
2.5	0.200	0.197	0.191	0.180	0.167	0.151
3.0	0.146	0.145	0.141	0.135	0.127	0.118
4.0	0.087	0.086	0.085	0.082	0.080	0.075

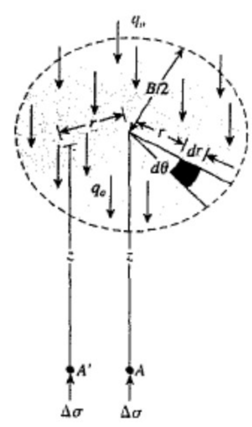


Figure 3.25 page 226

Vertical Stress under uniformly loaded circular area

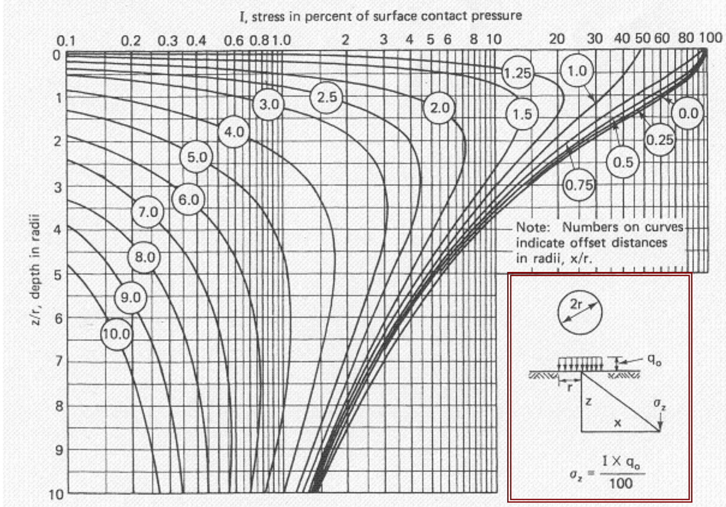


Fig. 8.22 Influence values, expressed in percentage of surface contact pressure, q_o, for vertical stress under uniformly loaded circular area (after Foster and Ahlvin, 1954, as cited by U.S. Navy, 1971).

Stress Under Corner of Uniformly Loaded Rectangular (table 5.2 page 228)

$$I = \text{influence factor} = \frac{1}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\cdot\frac{m^2+n^2+2}{m^2+n^2+1} + \tan^{-1}\left(\pi - \frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1-m^2n^2}\right)\right]$$

$$m = \frac{B}{z} \qquad n = \frac{L}{z}$$

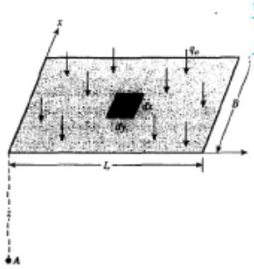
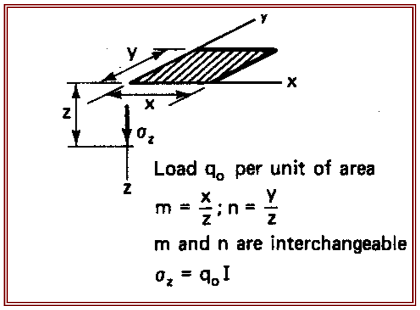


Table 5.2 Variation of Influence Value I [Eq. (5.6)][a]

	n											
m	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4
0.1	0.00470	0.00917	0.01323	0.01678	0.01978	0.02223	0.02420	0.02576	0.02698	0.02794	0.02926	0.03007
0.2	0.00917	0.01790	0.02585	0.03280	0.03866	0.04348	0.04735	0.05042	0.05283	0.05471	0.05733	0.05894
0.3	0.01323	0.02585	0.03735	0.04742	0.05593	0.06294	0.06858	0.07308	0.07661	0.07938	0.08323	0.08561
0.4	0.01678	0.03280	0.04742	0.06024	0.07111	0.08009	0.08734	0.09314	0.09770	0.10129	0.10631	0.10941
0.5	0.01978	0.03866	0.05593	0.07111	0.08403	0.09473	0.10340	0.11035	0.11584	0.12018	0.12626	0.13003
0.6	0.02223	0.04348	0.06294	0.08009	0.09473	0.10688	0.11679	0.12474	0.13105	0.13605	0.14309	0.14749
0.7	0.02420	0.04735	0.06858	0.08734	0.10340	0.11679	0.12772	0.13653	0.14356	0.14914	0.15703	0.16199
0.8	0.02576	0.05042	0.07308	0.09314	0.11035	0.12474	0.13653	0.14607	0.15371	0.15978	0.16843	0.17389
0.9	0.02698	0.05283	0.07661	0.09770	0.11584	0.13105	0.14356	0.15371	0.16185	0.16835	0.17766	0.18357
1.0	0.02794	0.05471	0.07938	0.10129	0.12018	0.13605	0.14914	0.15978	0.16835	0.17522	0.18508	0.19139
1.2	0.02926	0.05733	0.08323	0.10631	0.12626	0.14309	0.15703	0.16843	0.17766	0.18508	0.19584	0.20278
1.4	0.03007	0.05894	0.08561	0.10941	0.13003	0.14749	0.16199	0.17389	0.18357	0.19139	0.20278	0.21020
1.6	0.03058	0.05994	0.08709	0.11135	0.13241	0.15028	0.16515	0.17739	0.18737	0.19546	0.20731	0.21510
1.8	0.03090	0.06058	0.08804	0.11260	0.13395	0.15207	0.16720	0.17967	0.18986	0.19814	0.21032	0.21836
2.0	0.03111	0.06100	0.08867	0.11342	0.13496	0.15326	0.16856	0.18119	0.19152	0.19994	0.21235	0.22058
2.5	0.03138	0.06155	0.08948	0.11450	0.13628	0.15483	0.17036	0.18321	0.19375	0.20236	0.21512	0.22364
3.0	0.03150	0.06178	0.08982	0.11495	0.13684	0.15550	0.17113	0.18407	0.19470	0.20341	0.21633	0.22499
4.0	0.03158	0.06194	0.09007	0.11527	0.13724	0.15598	0.17168	0.18469	0.19540	0.20417	0.21722	0.22600
5.0	0.03160	0.06199	0.09014	0.11537	0.13737	0.15612	0.17185	0.18488	0.19561	0.20440	0.21749	0.22632
6.0	0.03161	0.06201	0.09017	0.11541	0.13741	0.15617	0.17191	0.18496	0.19569	0.20449	0.21760	0.22644
8.0	0.03162	0.06202	0.09018	0.11543	0.13744	0.15621	0.17195	0.18500	0.19574	0.20455	0.21767	0.22652
10.0	0.03162	0.06202	0.09019	0.11544	0.13745	0.15622	0.17196	0.18502	0.19576	0.20457	0.21769	0.22654
∞	0.03162	0.06202	0.09019	0.11544	0.13745	0.15623	0.17197	0.18502	0.19577	0.20458	0.21770	0.22656

Stress Under Corner of Uniformly Loaded Rectangular



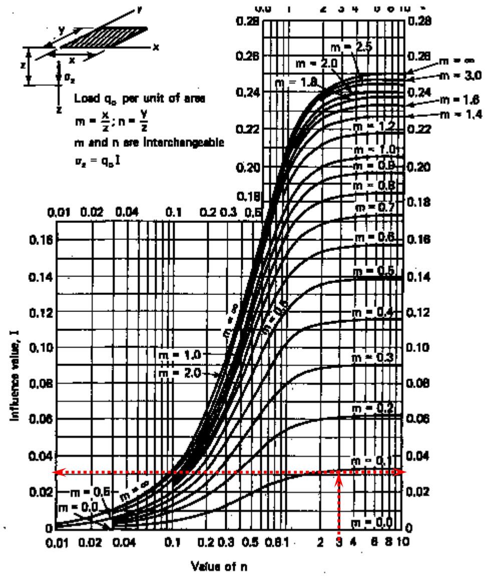


Fig. 8.21 Influence value for vertical stress under corner of a uniformly loaded rectangular area (after U.S. Navy, 1971).

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

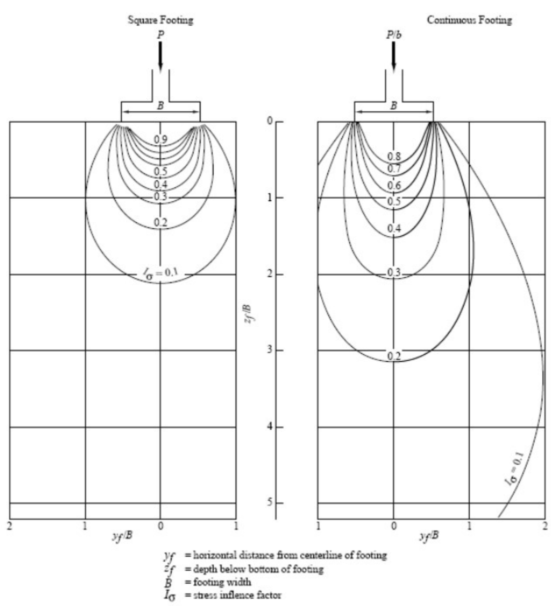
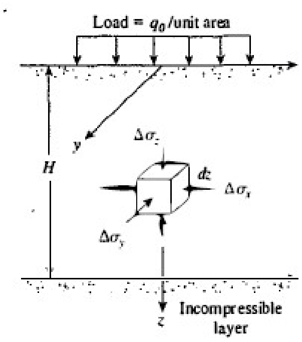


Figure 7.2 Stress bulbs based on Newmark's solution of Boussinesq's equation for square and continuous footings.

Elastic Settlement

- The elastic settlement of a shallow foundation can be estimated by using the theory of elasticity

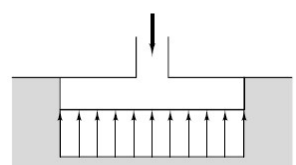
$$S_e = \int_0^H \varepsilon_z dz = \frac{1}{E_s}\int_0^H (\Delta\sigma_z - \mu_s\Delta\sigma_x - \mu_s\Delta\sigma_y)d$$



S_e = elastic settlement
E_s = modulus of elasticity of soil
H = thickness of the soil layer
μs = Poisson's ratio of the soil
$\Delta\sigma_x$, $\Delta\sigma_y$, $\Delta\sigma_z$ = stress increase due to the net applied foundation load in the x, y, and z directions, respectively

Settlement On Clay

$$S_T = S_d + S_c + S_t$$



S_T=total settlement
S_d=distortion (elastic) settlement F(P, E, B, D)
S_c=Primary consolidation Settlement f(P,Cc, Cr, eo)
S_t= secondary consolidation settlement (creep settlement) f(p,t)

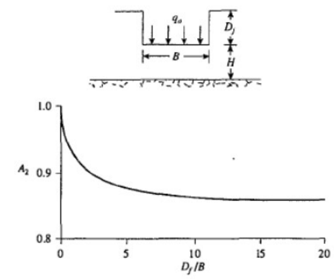
Elastic Settlement of Foundations on Saturated Clay

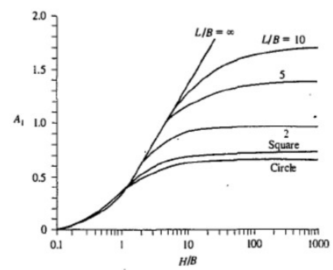
the average settlement of flexible foundations on saturated clay soils (μ_s= 0.5).

$$S_d = A_1 A_2 \frac{qo}{E_u} B$$

qo: net applied contact pressure
E_u= undrained young modulus
B=width of footing
A_2=Depth factors
A_1=Shape factors

Note: wherever elastic parameters exist the principle of superposition always valid



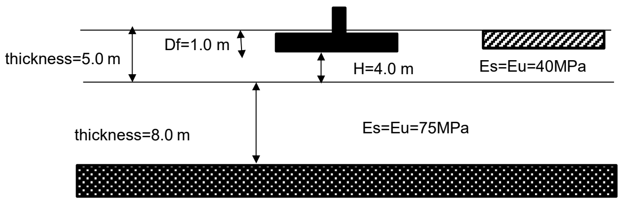


Example

- A foundation 4x2 m, carrying a net uniform pressure of 150 kN/m^2, is located at a depth of 1m in a layer of clay 5m thick for which the value of Eu is 40MN/m^2. The layer is underlain by a second clay layer 8m thick for which the value of Eu is 75MN/m^2. A hard stratum lies below the second layer. Determine the average immediate settlement under the foundation.

Solution:

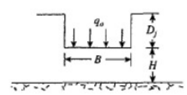
$$S_d = A_1 A_2 \frac{qo}{E_u} B$$



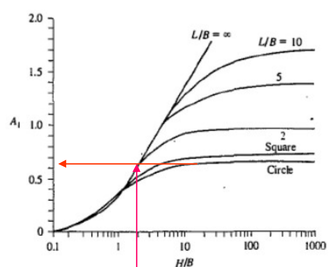
Since the footing in two layer soil, a superposition will be used

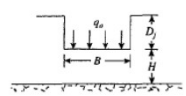
1. Consider the footing only in a first layer and the second layer is a hard stratum
   H=4.0 m, Df=1.0m , Eu=40MPa, B=2.0m

H/B=4/2=2.0; L/B=2.0 from Chart A_1=0.65

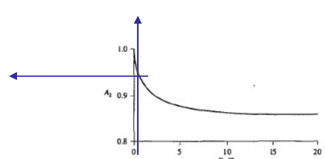


H/B=4/2=2.0; L/B=2.0





from Chart A_1=0.65



Df/B=1/2=0.5 A_2=0.94

Sd_1=(0.65)(0.94)(150/40x10^3)(2)=**4.5825x10^{-3}m**

2.0 Consider the bottom layer is extended to the footing depth and the first layer is not existing
H=12.0 m, Df=1.0m , Eu=75MPa, B=2.0m

H/B=12/2=6.0; L/B=2.0 from Chart A_1=0.85
Df/B=1/2=0.5 A_2=0.94
Sd_2=(0.85)(0.94)(150/75x10^3)(2)=**3.196x10^{-3}m**

3.0 Consider the bottom layer is only located between the footing and the original depth of it to the footing depth and the first layer is not existing (must be subtracted)

H=4.0 m, Df=1.0m , Eu=75MPa, B=2.0m

H/B=4/2=2.0; L/B=2.0 from Chart A_1=0.65

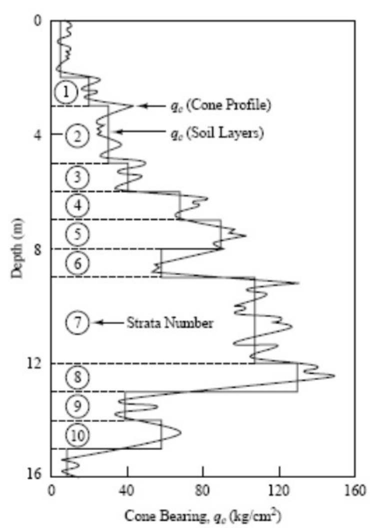
Df/B=1/2=0.5 A_2=0.94

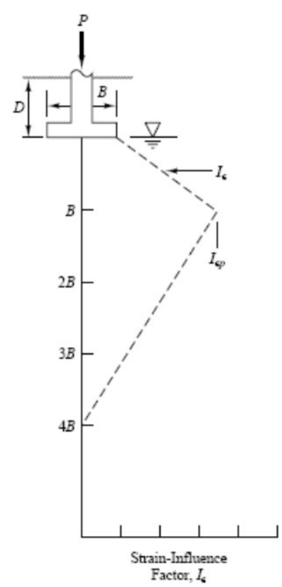
$Sd_3=(0.65)(0.94)(150/75x10^3)(2)=$**$2.444x10^{-3}$m**

Therefore, the total average settlement below the footing will be

$Sd_T=Sd_1+Sd_2-Sd_3=$**$4.5825x10^{-3}$m+$3.196x10^{-3}$m-$2.444x10^{-3}$m=$5.335x10^{-3}$ m=5.34mm**

Settlement Based on Field Test





Settlement on Sand

$$S_e = C_1 C_2 (\bar{q} - q) \sum_0^{z_2} \frac{I_z}{E_s} \Delta z$$

$$I_{z(m)} = 0.5 + 0.1\sqrt{\frac{\bar{q} - q}{q'_{z(1)}}}$$

$\bar{q}$ = stress at the level of the foundation

$$C_1 = 1 - 0.5\left(\frac{q}{q' - q}\right)$$

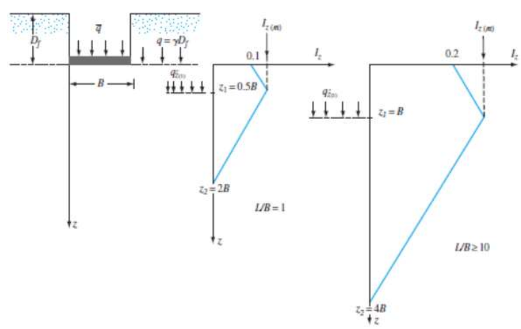
$$C_2 = 1 + 0.2\log\left(\frac{t}{0.1}\right)$$

I_z = strain influence factor

C_1 = a correction factor for the depth of foundation embedment

C_2 = a correction factor to account for creep in soil

$\bar{q}$ = stress at the level of the foundation
$q = \gamma D_f$



- I_z at $z = 0$ $\quad I_z = 0.1 + 0.0111\left(\frac{L}{B} - 1\right) \leq 0.2$
- Variation of z_1/B for $I_{z(m)}$ $\quad \frac{z_1}{B} = 0.5 + 0.0555\left(\frac{L}{B} - 1\right) \leq 1$
- Variation of z_2/B $\quad \frac{z_2}{B} = 2 + 0.222\left(\frac{L}{B} - 1\right) \leq 4$

Strain Influence Factors

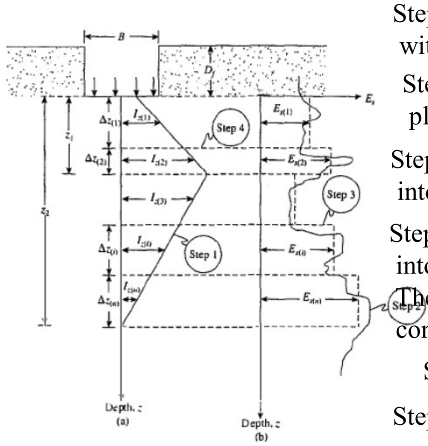


Figure 5.23 Procedure for calculation of S_e using the strain influence factor

Step 1: Plot the foundation and the variation of i_z, with depth to scale

Step 2: Using the correlation from (N60) or (qc), plot the actual variation of E with depth

Step 3. Approximate the actual variation of E, into a number of layers of soil having a constant E

Step 4. Divide the soil layer from z = 0 to Z = Z2 into a number of layers by drawing horizontal lines. The number of layers will depend on the break in continuity in the iz, and E, diagrams.

Step 5. Prepare a table to obtain $\Sigma \frac{I_z}{E_s} \Delta z$.

Step 6 Calculate C1, and C2

Step 7 Calculate S

Calculation of $\boxed{\Sigma \frac{I_z}{E_s} \Delta z.}$

Layer No.	Δz (in.)	E_s (lb/in.^{2})	z to the middle of the layer (in.)	I_z at the middle of the layer	$\frac{I_z}{E_s}\Delta z$ (in^3/lb)
1	48	750	24	0.275	0.0176
2	48	1250	72	0.425	0.016
3	96	1250	144	0.417	0.032
4	48	1000	216	0.292	0.014
5	144	2000	312	0.125	0.009
	$\Sigma 384$ in. $= 4B$				$\Sigma 0.0886$ in^3/lb

Typical Elastic Parameters of Various Soils

Table 5.8 Elastic Parameters of Various Soils

Type of soil	Modulus of elasticity, E_s: MN/m^2	Modulus of elasticity, E_s: lb/in^2	Poisson's ratio, μ_s
Loose sand	10.5–24.0	1500–3500	0.20–0.40
Medium dense sand	17.25–27.60	2500–4000	0.25–0.40
Dense sand	34.50–55.20	5000–8000	0.30–0.45
Silty sand	10.35–17.25	1500–2500	0.20–0.40
Sand and gravel	69.00–172.50	10,000–25,000	0.15–0.35
Soft clay	4.1–20.7	600–3000	
Medium clay	20.7–41.4	3000–6000	0.20–0.50
Stiff clay	41.4–96.6	6000–14,000	

Modulus of elasticity, Es

Sand

$$\frac{E_s}{p_a} = \alpha N_{60}$$

where

p_a = atmospheric pressure ≈ 100 kN/m² (≈ 2000 lb/ft²)

$$\alpha = \begin{cases} 5 \text{ for sands with fines} \\ 10 \text{ for clean normally consolidated sand} \\ 15 \text{ for clean overconsolidated sand} \end{cases}$$

$E_s = 2.5q_c$ for square foundations $(L/B = 1)$

$E_s = 3.5q_c$ for long foundations $(L/B \geq 10)$

$$E_{s(\text{rectangle})} = \left(1 + 0.4 \log\frac{L}{B}\right) E_{s(\text{square})}$$

Clays

$E_s = \beta c_u$ **Cu = undrained shear strength:**

Table 5.9 Range of β for Clay [Eq. (5.45)][a]

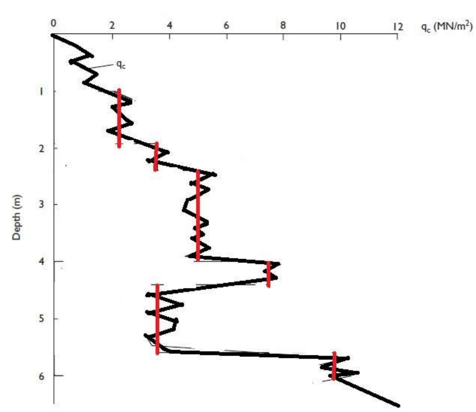
Plasticity index	β OCR = 1	OCR = 2	OCR = 3	OCR = 4	OCR = 5
< 30	1500–600	1380–500	1200–580	950–380	730–300
30 to 50	600–300	550–270	580–220	380–180	300–150
> 50	300–150	270–120	220–100	180–90	150–75

[a]Interpolated from Duncan and Buchignani (1976)

Example

A footing 2.5×2.5m supports a net foundation pressure of 150kN/m² at a depth of 1.0m in a deep deposit of normally consolidated fine sand of unit weight 17kN/m³.

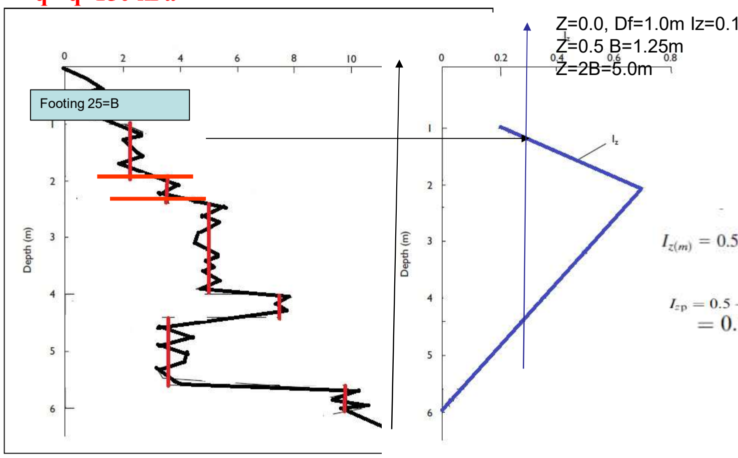
Determine the elastic settlement of this footing after 10 yrs



$$S_e = C_1 C_2(\bar{q} - q)\sum_{0}^{z_2}\frac{I_z}{E_s}\Delta z$$

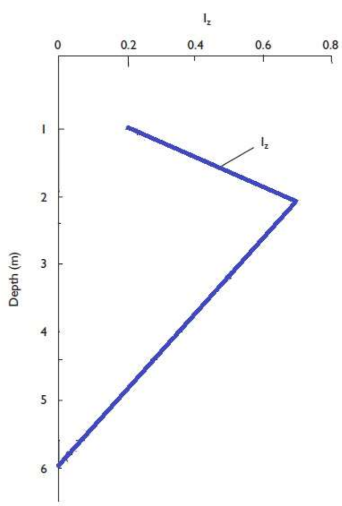
$$C_1 = 1 - 0.5\left(\frac{q}{\bar{q} - q}\right) = 1 - 0.5\left(\frac{17}{150}\right) = 0.943 \qquad C_2 = 1 + 0.2\log\left(\frac{t}{0.1}\right) = 1 + 0.2\log\left(\frac{10}{0.1}\right) = 1.40$$

q"-q=150 kPa



$$I_{z(m)} = 0.5 + 0.1\sqrt{\frac{q - q}{q'_{z(1)}}}$$

$$I_{zp} = 0.5 + 0.1\left(\frac{150}{17 \times 2.25}\right)^{0.5} = 0.70$$



Layer	Δz (m)	q_c (MN/m^2)	E_s (MN/m^2)	I_z	$I_z \Delta z / E_s$ (m^3/MN)
1	0.90	2.3	2.5*2.3 =5.75	0.41	0.064
2	0.50	3.6	9.00	0.68	0.038
3	1.60	5.0	12.50	0.50	0.064
4	0.40	7.5	18.75	0.33	0.007
5	1.20	3.3	8.25	0.18	0.026
6	0.40	9.9	24.75	0.04	0.001
					0.200

Se=0.943*1.40*(150)(0.2x10^{-3})=39.6x10^{-3}m=39.6mm

Simplified Settlement of Foundation on Sand Based on Standard Penetration Resistance

- Elastic Settlement, Se (mm)

$$S_e(\text{mm}) = \frac{1.25 q_{net}(\text{kN/m}^2)}{N_{60} F_d} \quad (\text{for } B \leq 1.22\,\text{m})$$

$$S_e(\text{mm}) = \frac{2 q_{net}(\text{kN/m}^2)}{N_{60} F_d}\left(\frac{B}{B+0.3}\right)^2 \quad (\text{for } B > 1.22\,\text{m})$$

$$q_{net} = \bar{q} - \gamma D_f$$

$$F_d = \text{depth factor} = 1 + 0.33(D_f/B)$$

The N_{60} is the standard penetration resistance between the bottom of the foundation and 2B below the bottom for (square) and 4B for strip footing.

Consolidation Settlement

Primary Consolidation Settlement

- Normally consolidation Soil

$$S_c = \frac{Cc}{1+e_o} H \log\left(\frac{\sigma'_f}{\sigma'_o}\right)$$

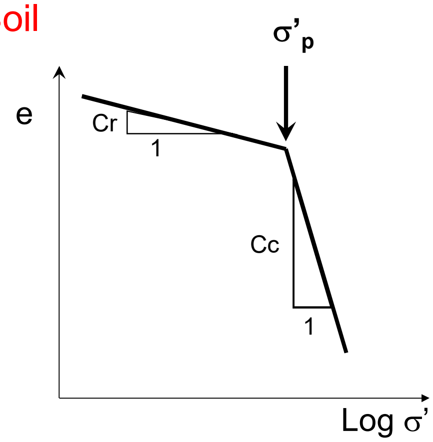
- Over consolidated Soil

I. $\sigma'_o < \sigma'_f < \sigma'_p$

$$S_c = \frac{Cr}{1+e_o} H \log\left(\frac{\sigma'_f}{\sigma'_o}\right)$$

II. $\sigma'_o < \sigma'_p < \sigma'_f$

$$S_c = \frac{Cr}{1+e_o} H \log\left(\frac{\sigma'_p}{\sigma'_o}\right) + \frac{Cc}{1+e_o} H \log\left(\frac{\sigma'_f}{\sigma'_p}\right)$$



Definition of Basic Term

$$S_c = \frac{Cc}{1+e_o} H \log\left(\frac{\sigma'_f}{\sigma'_o}\right)$$

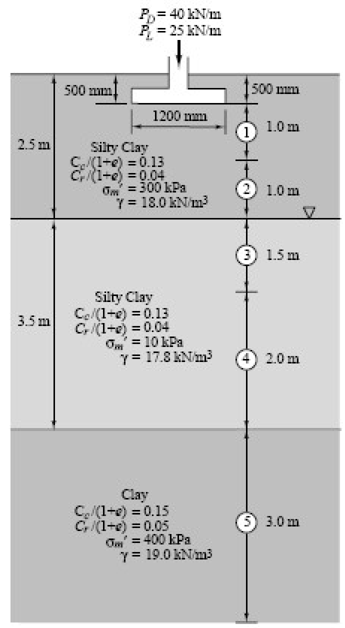
Cc compression index

e_o void ratio

H Height of clay layer

σ'_o initial overburden stress

$\sigma_f' = \sigma_o' + \Delta\sigma$ final effective stress



Determine the primary consolidation settlement under the wall footing shown (consider σvm'=σc'=σp')

Qall=40+25=65kN/m

qo=/1.2-0.5(18-9.81)=50kPa

Sc=Sc for layer1+ Sc for layer2+ Sc for layer3

Consider the GWT at the surface (use γw=10kN/m3)

σo'= at the middle of clay layer beneath the footing

=(0.5*18+1.0*18)-1.5*10=12kPa

Δσ=[Δσtop+ 4Δσmiddle+ Δσbottom]/6

I will use 2:1 approximate method for simplification

Δσtop=qoB/(B+Z)=qo=50 kPa

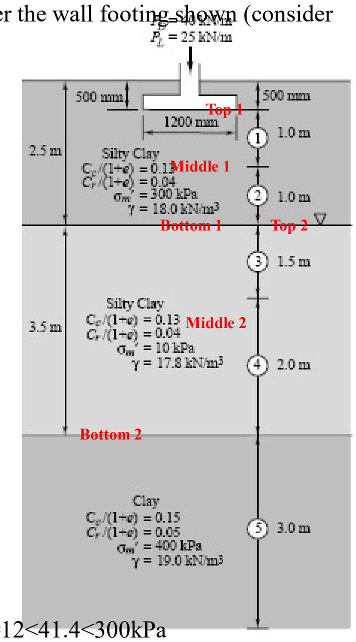
Δσmiddle=qoB/(B+Z)=50*1.2/(1.2+1) =27.27 kPa

Δσbottom=qoB/(B+Z)=50*1.2/(1.2+2) =18.75 kPa

Δσ=[Δσtop+ 4Δσmiddle+ Δσbottom]/6 =29.64kPa

σf'= σo'+ Δσ=12+29.64=41.64 kPa σo'< σf'< σp'=12<41.4<300kPa

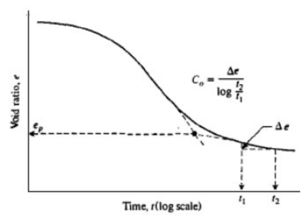
$$S_{c1} = \frac{Cr}{1+e_o} H \log\left(\frac{\sigma'_f}{\sigma'_o}\right) = 0.04*(2.0)\log\frac{44.1}{12} = 0.045m = 45mm$$



Secondary Compression Settlements

$S_t = C_\alpha H \log(t/t_P)$
Where
- t: the time in which secondary settlement needed
- t_p: primary consolidation settlement ended
- H: the height of soil layer
- c_α: *Secondary Compression index*

- For inorganic clays and silts:
  $$C_\alpha / C_c \approx 0.04 \pm 0.01$$
- For organic clays and silts:
  $$C_\alpha / C_c \approx 0.05 \pm 0.01$$
- For peats:
  $$C_\alpha / C_c \approx 0.075 \pm 0.01$$



Example

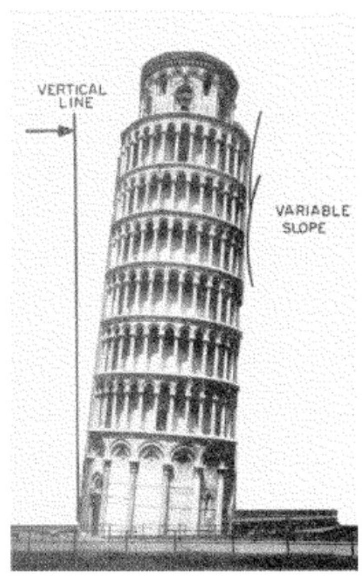
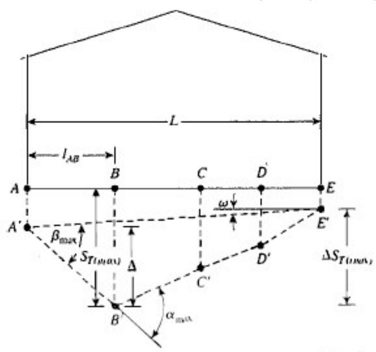


FIGURE 8.4 Leaning Tower of Pisa—variable tilt.

Definition of parameters for differential settlement



S_T = total settlement of a given point
ΔS_T = difference in total settlement between any two points
α = gradient between two successive points
β = angular distortion = $\dfrac{\Delta S_{T(ij)}}{l_{ij}}$
Note: l_{ij} = distance between points i and j)
ω = tilt
Δ = relative deflection (i.e. movement from a straight line joining two reference points)
$\dfrac{\Delta}{L}$ = deflection ratio

- Maximum settlement and maximum angular distortion, to be used for building purposes:

Maximum settlement, $S_{T(max)}$	
In sand	32 mm
In clay	45 mm
Maximum differential settlement, $\Delta S_{T(max)}$	
Isolated foundations in sand	51 mm
Isolated foundations in clay	76 mm
Raft in sand	51–76 mm
Raft in clay	76–127 mm
Maximum angular distortion, β_{max}	1/300

Limiting Angular Distortion

- **recommended the following limiting angular distortion, β_{max} for various structures**

Category of potential damage	β_{max}
Safe limit for flexible brick wall ($L/H > 4$)	1/150
Danger of structural damage to most buildings	1/150
Cracking of panel and brick walls	1/150
Visible tilting of high rigid buildings	1/250
First cracking of panel walls	1/300
Safe limit for no cracking of building	1/500
Danger to frames with diagonals	1/600

European Commitee Recommendation

- limiting values for serviceability and the maximum accepted foundation movements.

Table 5.13 Recommendations of European Committee for Standardization on Differential Settlement Parameters

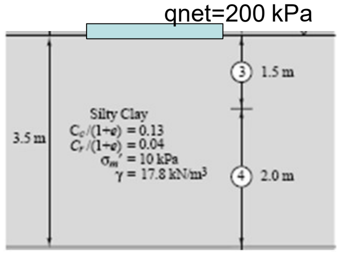
Item	Parameter	Magnitude	Comments
Limiting values for serviceability (European Committee for Standardization. 1994a)	S_T	25 mm	Isolated shallow foundation
		50 mm	Raft foundation
	ΔS_T	5 mm	Frames with rigid cladding
		10 mm	Frames with flexible cladding
		20 mm	Open frames
	β	1/500	—
Maximum acceptable foundation movement (European Committee for Standardization. 1994b)	S_T	50	Isolated shallow foundation
	ΔS_T	20	Isolated shallow foundation
	β	≈1/500	—

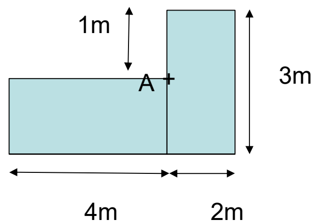
Note on Structural Tolerance To Settlement & Differential Settlements

1. The values in Table above should be adequate most of the time. The values in brackets are recommended for design; others are the range of settlements found for satisfactory structural performance.
2. One must carefully look at the differential movement between two adjacent points in assessing what constitutes an acceptable slope.
3. Residual stresses in the structure may be important, as it has been observed that there is a range of tolerable differential settlements between similar buildings.
4. Construction materials that are more ductile—for example, steel—can tolerate larger movements than either concrete or load-bearing masonry walls.
5. Time interval during which settlement occurs can be important—long time spans allow the structure to adjust and better resist differential movement.

Determine the primary and secondary consolidation settlement L shape footing under point A shown after 25 years of construction.
Consider $c_\alpha/c_c=0.05$; $e_0=0.65$; t_p=5years

qnet=200 kPa



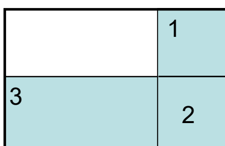


$\sigma o' = \gamma' z = (17.8-10.0)(3.5/2) = 13.65$ kPa $\sigma f' = \sigma o' + \Delta\sigma$

$\Delta\sigma$ use elasticity method

At top of the layer $\Delta\sigma = 200.0$ kPa

At the middle of clay layer



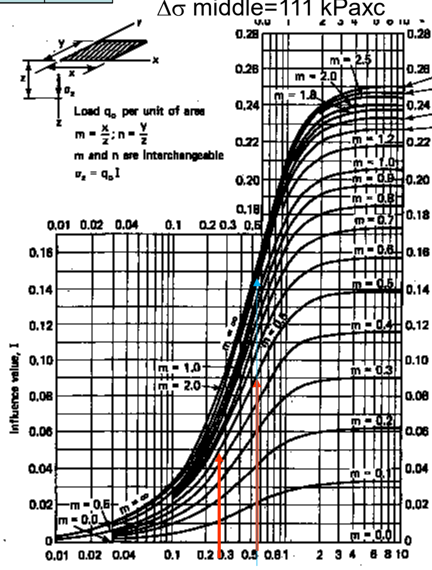
For part 1 B=1; L=2 z=3.5/2 m=B/z=1.0/1.75=0.571 n=2.0/1.75=1.1428;I=0.16

For part 2 B=2; L=2 z=3.5/2 m=2/z=2/1.75=1.1428 n=2/1.75=1.1428; I=0.19

For part 3 B=2; L=4 z=3.5/2 m=2/z=2/1.75=1.1428 n=4/1.75=2.2857; I=0.205

$\Delta\sigma$ middle=$\Delta\sigma1+\Delta\sigma2+\Delta\sigma3=q_0I_1+q_0I_2+q_0I_3$=200(0.16+0.19+0.205)

$\Delta\sigma$ middle=111 kPaxc



For Bottom of layer

For part 1 b=1; L=2 z=3.5
m=B/z=1.0/3.5=0.285 n=2.0/3.5=0.571;I=0.048

For part 2 b=2; L=2 z=3.5
m=2/z=2/3.75=0.5741 n=2/3.5=0.517; I=0.09

For part 3 b=2; L=4 z=3.5
m=2/z=2/3.5=0.571 n=4/3.5=1.1428; I=0.145

$\Delta\sigma$ bottom=$\Delta\sigma1+\Delta\sigma2+\Delta\sigma3=q_0I_1+q_0I_2+q_0I_3$
=200(0.048+0.09+0.145)= $\Delta\sigma$ bottom=56.6 kPa

$\Delta\sigma$=[$\Delta\sigma$ top+ 4$\Delta\sigma$ middle+ $\Delta\sigma$ bottom]/6
=[200+4*111+56.6]/6=116.767kPa

$\sigma f' = \sigma o' + \Delta\sigma = 13.65+116.767 = 130.42$kPa
$\sigma f' > \sigma 0' > \sigma c'$

$$S_c = \frac{Cc}{1+e_o} H \log(\frac{\sigma'_f}{\sigma'_o})$$

Sc=0.13*3.5*log(130.42/13.65)=0.45 m

$S_t=C_\alpha H\log(t/t_P)$

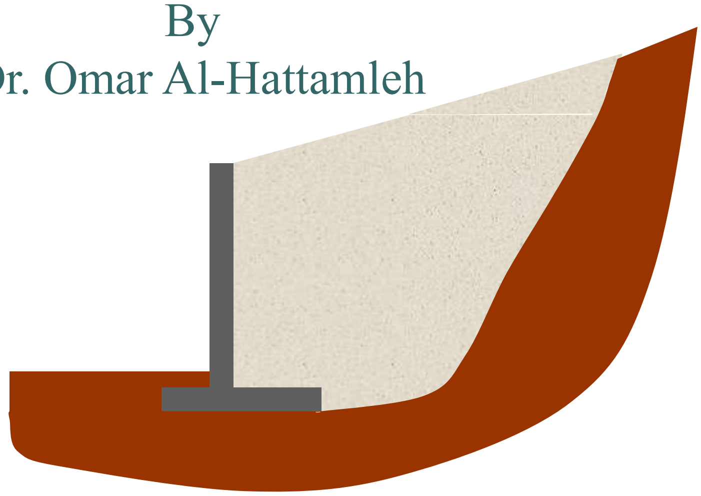
$C\alpha=0.05$ cc

Cc/(1+eo)=0.13=cc/(1+0.65)   cc=0.2145 $c\alpha=0.05*0.2145=0.0107$

$S_t=C_\alpha H\log(t/t_P)=0.0107*3.5*\log((25-5)/5)=0.02256$ m

Lateral Earth Pressures

By
Dr. Omar Al-Hattamleh



Part of these slides are from sviugan

Contents

- Geotechnical applications
- K_0, active & passive states
- Rankine's earth pressure theory
- Design of retaining walls

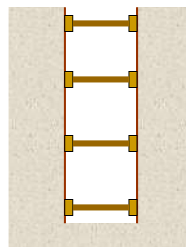
2

Lateral Support

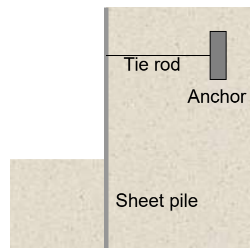
In geotechnical engineering, it is often necessary to prevent lateral soil movements.

We have to estimate the lateral soil pressures acting on these structures, to be able to design them.

Cantilever retaining wall



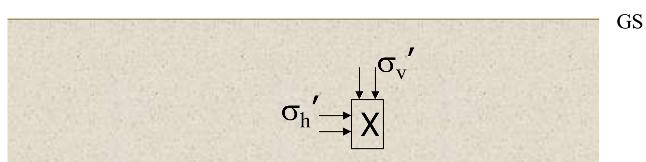
Braced excavation



Anchored sheet pile

Earth Pressure at Rest

In a homogeneous natural soil deposit,



the ratio σ_h'/σ_v' is a constant known as coefficient of earth pressure at rest (K_0).

Importantly, **at K_0 state**, there are no lateral strains.

Estimating K_0

For normally consolidated clays and granular soils,

$$K_0 = 1 - \sin \phi'$$

For overconsolidated clays,

$$K_{0,\text{overconsolidated}} = K_{0,\text{normally consolidated}} (OCR)^{\sin\phi}$$

From elastic analysis,

$$K_0 = \frac{\upsilon}{1-\upsilon}$$

Poisson's ratio

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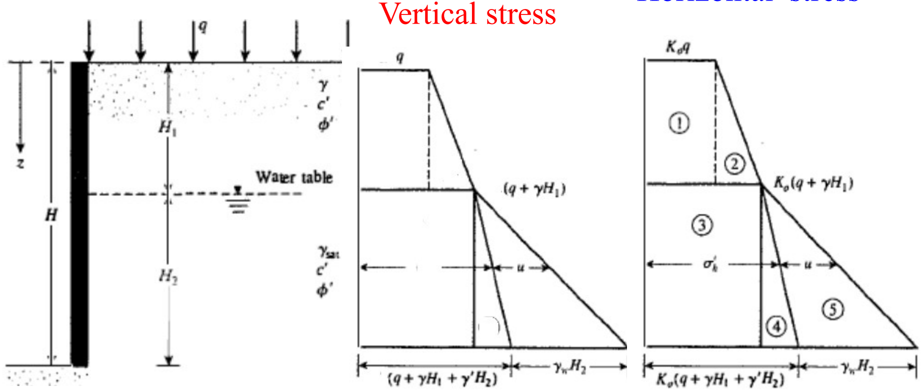
Example

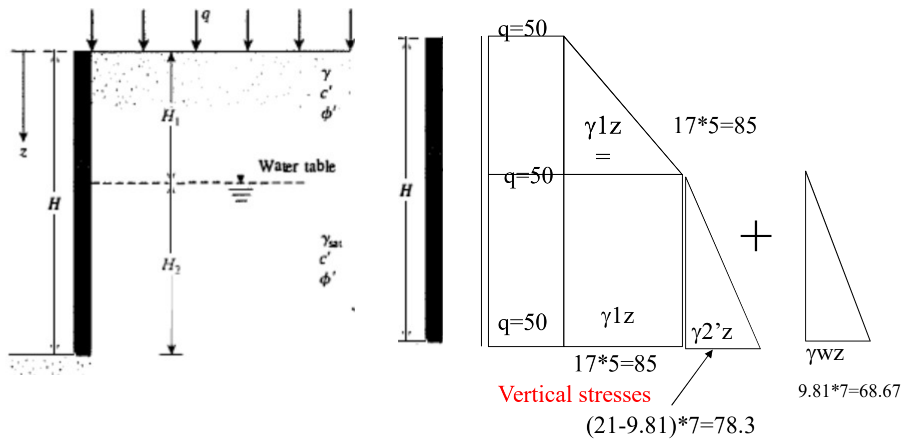
- If the wall is prevented from movement. Determine the total force acting the wall and line of action the total force, q=50kPa, H1=5m and H=12 m , γ_{sat}=21kN/m3 , γ_{dry}=17kN/m^3, c=0kPa and ϕ=35^o

Horizontal stress

Vertical stress



If the wall is prevented from movement. Determine the total force acting the wall and line of action the total force, q=50kPa, H1=5m and H=12 m , γ_{sat}=21kN/m3 , γ_{dry}=17kN/m^3, c=0kPa and ϕ=35°



Ko=1-sinϕ=1–sin35=0.4264

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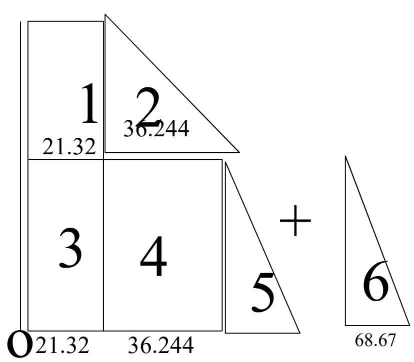
Horizontal s

50*.4264=21.32



Segment 1	Area (kN/m)	Xi (m)	Moment (Area*xi)
1	21.32*5	7+5/2	21.32*(7+5/2)
2	0.50*36.244*5	7+5/3	
3	21.32*7	7/2	
4	36.244*7	7/2	
5	0.5*33.387*7	7/3	
6	0.5*68.68*7	7/3	
total	122.85	188.23	

78.3*.4264=33.387

Total horizontal Force=122.85 kN/m

Location above point o=188.23/122.85=1.53 m

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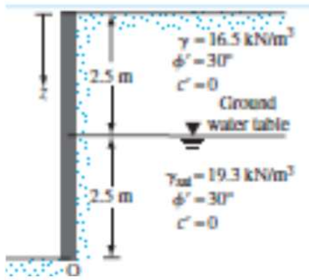
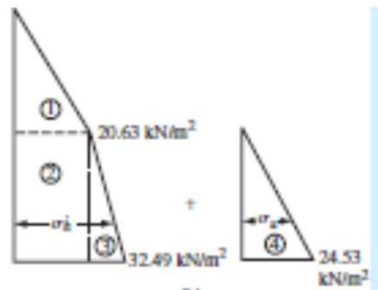
For the retaining wall shown in Figure shown, determine the lateral earth force at rest per unit length of the wall. Also determine the location of the resultant force



$$K_o = 1 - \sin \phi' = 1 - \sin 30° = 0.5$$



$$P_o = \text{Area 1} + \text{Area 2} + \text{Area 3} + \text{Area 4}$$
$$= \tfrac{1}{2}(2.5)(20.63) + (2.5)(20.63) + \tfrac{1}{2}(2.5)(32.49 - 20.63) + \tfrac{1}{2}(2.5)(24.53) = \mathbf{122.85\ kN/m}$$

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$$\bar{z} = \frac{(\text{Area 1})\left(2.5+\frac{2.5}{3}\right)+(\text{Area 2})\left(\frac{2.5}{2}\right)+(\text{Area 3}+\text{Area 4})\left(\frac{2.5}{3}\right)}{P_o}$$
$$= \frac{(25.788)(3.33) + (51.575)(1.25) + (14.825 + 30.663)(0.833)}{122.85}$$
$$= \frac{85.87 + 64.47 + 37.89}{122.85} = \mathbf{1.53\ m}$$

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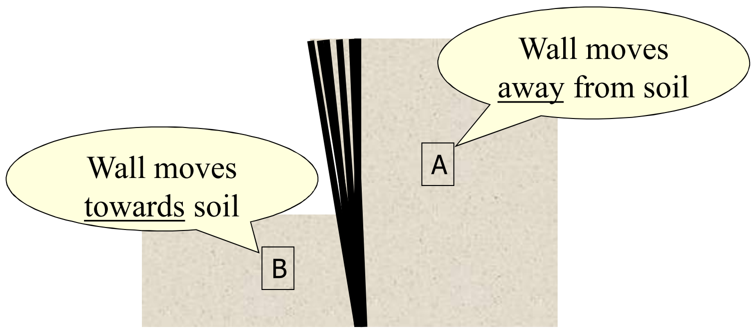
Active/Passive Earth Pressures

- in **granular** soils



smooth wall

Let's look at the soil elements A and B during the wall movement.

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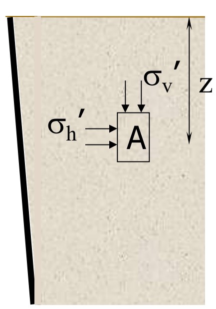
Active Earth Pressure

- in **granular** soils



$\sigma_v' = \gamma z$

Initially, there is no lateral movement.

$\therefore \sigma_h' = K_0 \sigma_v' = K_0 \gamma z$

As the wall moves away from the soil,

σ_v' remains the same; and

σ_h' decreases till failure occurs.

Active state

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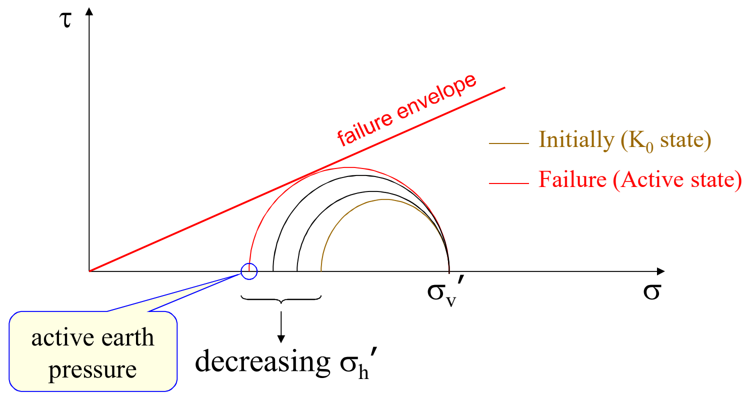
Active Earth Pressure

- in **granular** soils

As the wall moves away from the soil,



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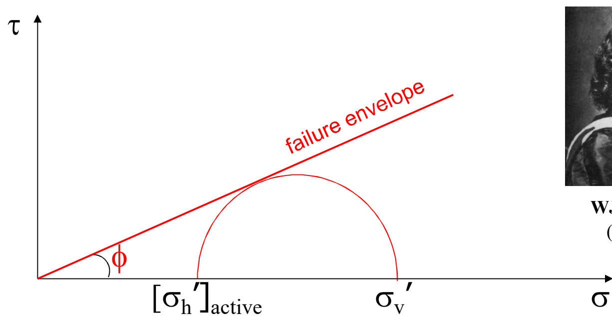
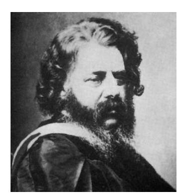
Active Earth Pressure

- in **granular** soils





WJM Rankine
(1820-1872)

$$[\sigma_h']_{active} = K_A \sigma_v'$$

Rankine's coefficient of active earth pressure

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

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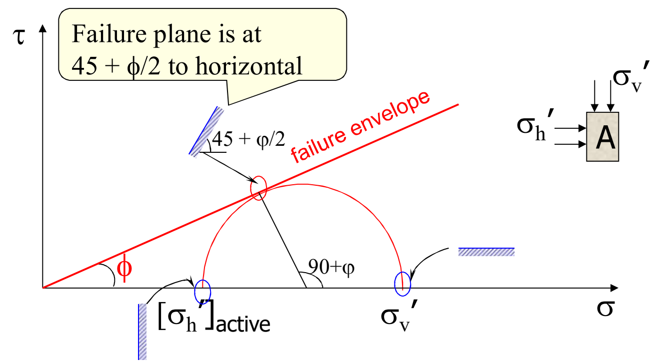
Active Earth Pressure

- in **granular** soils



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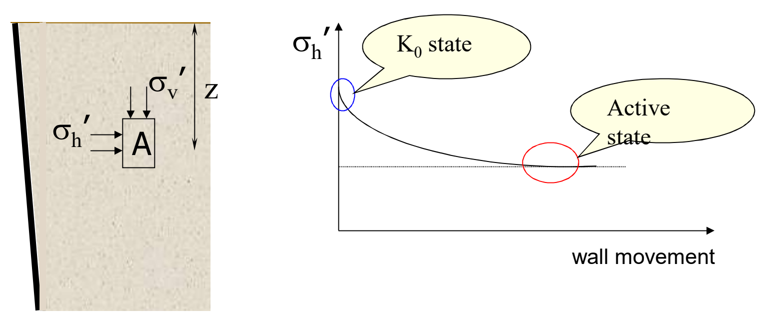
Active Earth Pressure

- in **granular** soils

As the wall moves away from the soil,

σ_h' decreases till failure occurs.



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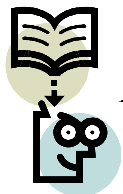
Active Earth Pressure

- in **cohesive** soils



Follow the same steps as for granular soils. Only difference is that c ≠ 0.

$$[\sigma_h']_{active} = K_A \sigma_v' - 2c\sqrt{K_A}$$

Everything else the same as for granular soils.

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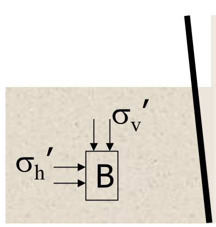
Passive Earth Pressure

- in **granular** soils

Initially, soil is in K_0 state.



As the wall moves towards the soil,

σ_v' remains the same, and

σ_h' increases till failure occurs.

Passive state

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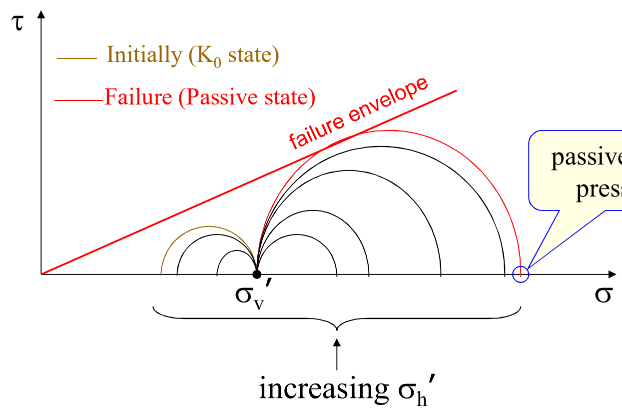
Passive Earth Pressure

- in **granular** soils

As the wall moves towards the soil,



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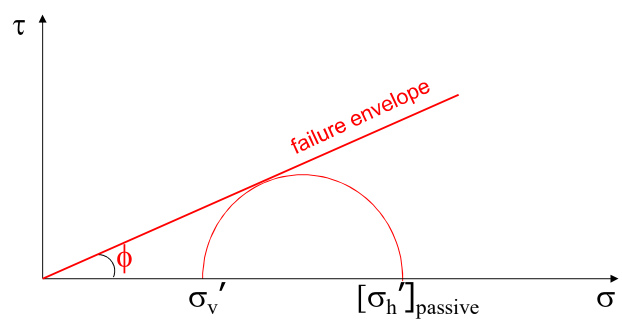
Passive Earth Pressure

- in **granular** soils



$$[\sigma_h']_{passive} = K_P \sigma_v'$$

Rankine's coefficient of passive earth pressure

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

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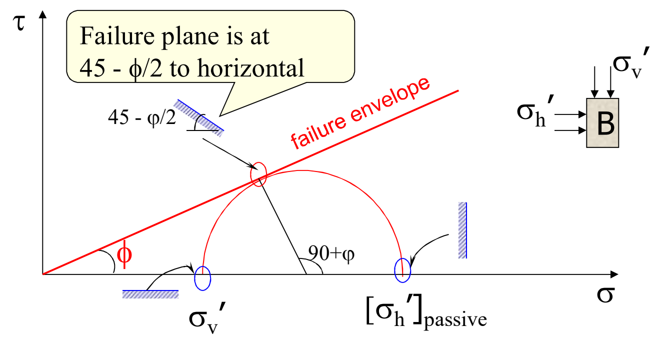
Passive Earth Pressure

- in **granular** soils

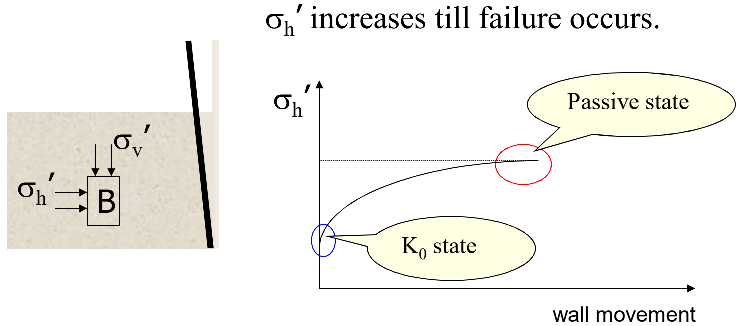


Passive Earth Pressure

- in **granular** soils

As the wall moves towards the soil,

σ_h' increases till failure occurs.



Passive Earth Pressure

- in **cohesive** soils

Follow the same steps as for granular soils. Only difference is that c ≠ 0.

$$[\sigma_h']_{passive} = K_P \sigma_v' + 2c\sqrt{K_P}$$

Everything else the same as for granular soils.

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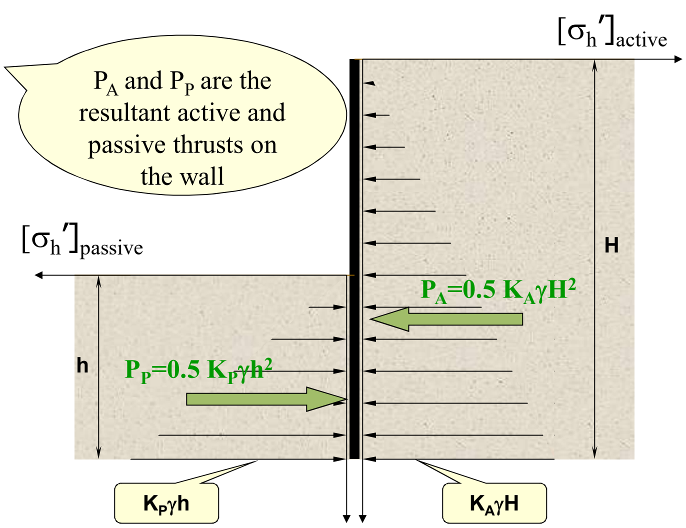
Earth Pressure Distribution

- in **granular** soils

P_A and P_P are the resultant active and passive thrusts on the wall



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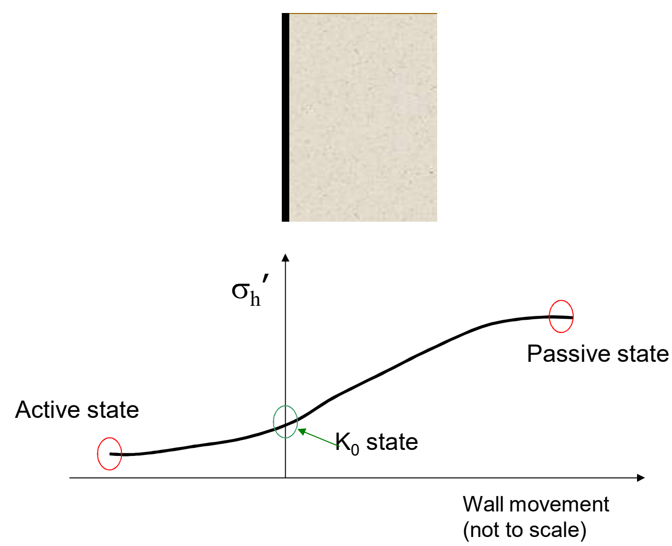
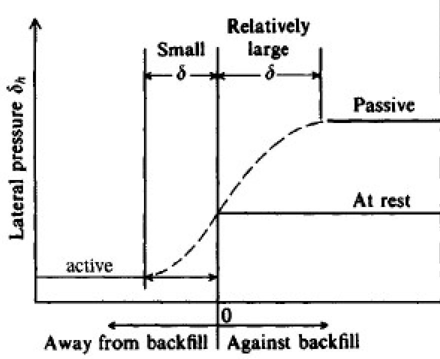


Illustration of active and passive pressures with usual range of values for cohesionless and cohesive soil.



Usual range of earth pressure coefficients	
Cohesionless soils	Cohesive soils
3–14	1–2
0.4–0.6	0.4–0.8
0.33–0.22	1–0.5

Rankine's Earth Pressure Theory

$$[\sigma_h']_{active} = K_A\sigma_v' - 2c\sqrt{K_A}$$

$$[\sigma_h']_{passive} = K_P\sigma_v' + 2c\sqrt{K_P}$$

- ❑ Assumes smooth wall
- ❑ Applicable only on vertical walls

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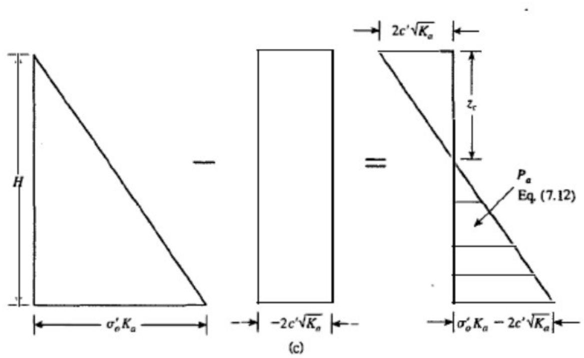
Active Earth Pressure Distribution on Cohesive soil



The depth z_c, is usually referred to as the tensile depth crack

$$z_c = \frac{2c'}{\gamma\sqrt{K_a}}$$

Determination of Active Earth Force

- Before Tension Crack

$$P_a = \int_0^H \sigma'_a dz = \int_0^H \gamma z K_a dz - \int_0^H 2c'\sqrt{K_a} dz$$

$$= \tfrac{1}{2}\gamma H^2 K_a - 2c'H\sqrt{K_a}$$

- After Tension Crack

$$P_a = \tfrac{1}{2}(H - z_c)(\gamma H K_a - 2c'\sqrt{K_a})$$

$$P_a = \frac{1}{2}\left(H - \frac{2c'}{\gamma\sqrt{K_a}}\right)\left(\gamma H K_a - 2c'\sqrt{K_a}\right)$$

$\bar{z} = (H - z_c)/3$ above the bottom of the wall

Example

A 6-m-high retaining wall is to support a soil with unit weight $\gamma = 17.4\ \mathrm{kN/m^3}$, soil friction angle $\phi' = 26°$, and cohesion $c' = 14.36\ \mathrm{kN/m^2}$. Determine the Rankine active force per unit length of the wall both before and after the tensile crack occurs, and determine the line of action of the resultant in both cases.

For $\phi' = 26°$, $K_a = \tan^2\left(45 - \frac{\phi'}{2}\right) = \tan^2(45 - 13) = 0.39$

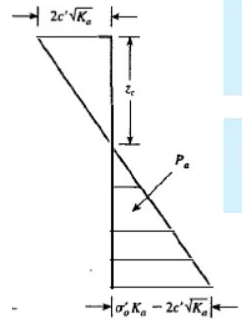
$\sigma'_a = \gamma H K_a - 2c'\sqrt{K_a}$ **(the soil behind the wall has no inclination)**

at $z = 0$ $\quad \sigma'_a = -2c'\sqrt{K_a} = -2(14.36)(0.625) = -17.95\ \mathrm{kN/m^2}$

and at $z = 6$ m $\quad \sigma'_a = (17.4)(6)(0.39) - 2(14.36)(0.625) = 40.72 - 17.95 = 22.77\ \mathrm{kN/m^2}$

$P_a = \frac{1}{2}\gamma H^2 K_a - 2c'H\sqrt{K_a}$ **Before tensile crack)**

$= \frac{1}{2}(6)(40.72) - (6)(17.95) = 122.16 - 107.7 = \mathbf{14.46\ kN/m}$



$$P_a\bar{z} = (122.16)\left(\frac{6}{3}\right) - (107.7)\left(\frac{6}{2}\right)$$

$$\bar{z} = \frac{244.32 - 323.1}{14.46} = -\mathbf{5.45\ m}.$$

Active Force after the Tensile Crack Appeared

$$z_c = \frac{2c'}{\gamma\sqrt{K_a}} = \frac{2(14.36)}{(17.4)(0.625)} = 2.64 \text{ m}$$

$$P_a = \tfrac{1}{2}(H - z_c)(\gamma H K_a - 2c'\sqrt{K_a}) = \tfrac{1}{2}(6 - 2.64)(22.77) = \mathbf{38.25\ kN/m}$$

$\bar{z} = (H - z_c)/3$ $\quad$ $\bar{z} = \dfrac{6 - 2.64}{3} = \mathbf{1.12\ m}$

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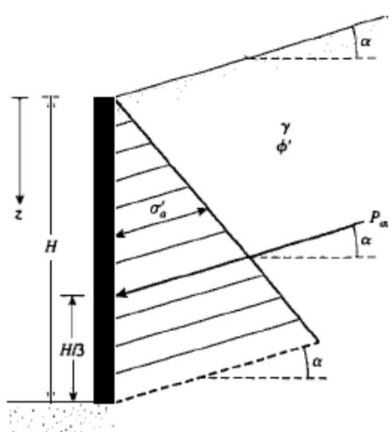
Rankine's Earth Pressure Coefficient for sloped ground

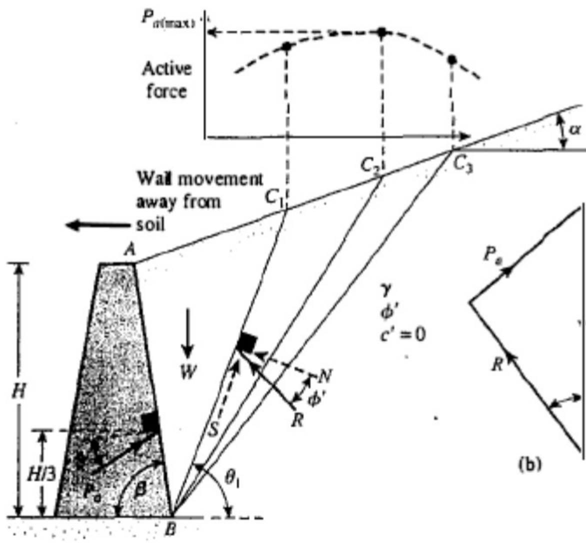


$$K_a = \cos\alpha \frac{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\phi'}}{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi'}}$$

$$K_p = \cos\alpha \frac{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi'}}{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\phi'}}$$

$$z_c = \frac{2c'}{\gamma}\sqrt{\frac{1 + \sin\phi'}{1 - \sin\phi'}}$$

Coulomb’s coefficients of Earth Pressure



Coulomb’s coefficients of Earth Pressure

- Take into consideration
  - ❑ Wall roughness
  - ❑ Inclined walls

K_a = Coulomb’s active earth pressure coefficient

$$= \frac{\sin^2(\beta + \phi')}{\sin^2\beta \sin(\beta - \delta')\left[1 + \sqrt{\frac{\sin(\phi' + \delta')\sin(\phi' - \alpha)}{\sin(\beta - \delta')\sin(\alpha + \beta)}}\right]^2}$$

K_p = Coulomb’s passive pressure coefficient

$$= \frac{\sin^2(\beta - \phi')}{\sin^2\beta \sin(\beta + \delta')\left[1 - \sqrt{\frac{\sin(\phi' + \delta')\sin(\phi' + \alpha)}{\sin(\beta + \delta')\sin(\beta + \alpha)}}\right]^2}$$

Where

β : angle between the wall’s back face and the vertical

δ : angle of friction between the soil and the backside of the wall

α : angle of the slope for the backfill behind the wall

Example # 1

a) Compare the Rankine and Coulomb lateral earth pressure coefficients for a wall that retains a granular backfill soil with $\phi = 35°$, $\delta = 15°$, and $\alpha = 20°$. (Note: δ is the angle of friction between the soil and the backside of the wall, and α is the angle of the slope for the backfill behind the wall).

b) What is the passive earth force on the wall at failure if the wall is 10 m high, $\gamma = 18.1$ kN/m3 and c = 9 kN/m2?

Example # 1

Solution:

a) For the "at rest" condition use Jaky's formula, because we are not given the soil's PI,

Ko = 1 – sin ϕ = 1 – sin (35°) = 1 - 0.57 = 0.43

Rankine's coefficients.

$$K_a = \cos\alpha \frac{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\phi'}}{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi'}} = \underline{0.271}$$

$$K_p = \cos\alpha \frac{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi'}}{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\phi'}} = \underline{3.690}$$

Example # 1

Coulomb's coefficients, (assume $\beta = 90°$, where β is the angle between the wall's back face and the horizontal),

K_a = Coulomb's active earth pressure coefficient

$$= \frac{\sin^2(\beta + \phi')}{\sin^2\beta \sin(\beta - \delta')\left[1 + \sqrt{\frac{\sin(\phi' + \delta')\sin(\phi' - \alpha)}{\sin(\beta - \delta')\sin(\alpha + \beta)}}\right]^2} = \underline{0.323}$$

K_p = Coulomb's passive pressure coefficient

$$= \frac{\sin^2(\beta - \phi')}{\sin^2\beta \sin(\beta + \delta')\left[1 - \sqrt{\frac{\sin(\phi' + \delta')\sin(\phi' + \alpha)}{\sin(\beta + \delta')\sin(\beta + \alpha)}}\right]^2} = \underline{3.517}$$

Note that when $\alpha = 0°$, $\beta = 90°$ and $\delta = 0°$ the Coulomb formula becomes identical to Rankine's.

Example # 1

b) The total passive force Fp on the wall per unit length is, Using Rankine

$$Fp = \tfrac{1}{2}\gamma H^2 Kp + 2cH\sqrt{(Kp)}$$

$$= (0.5)(18.1)(10)^2(3.690) + 2(9)(10)\sqrt{(3.690)} = \underline{3{,}690 \text{ kN/m}}$$

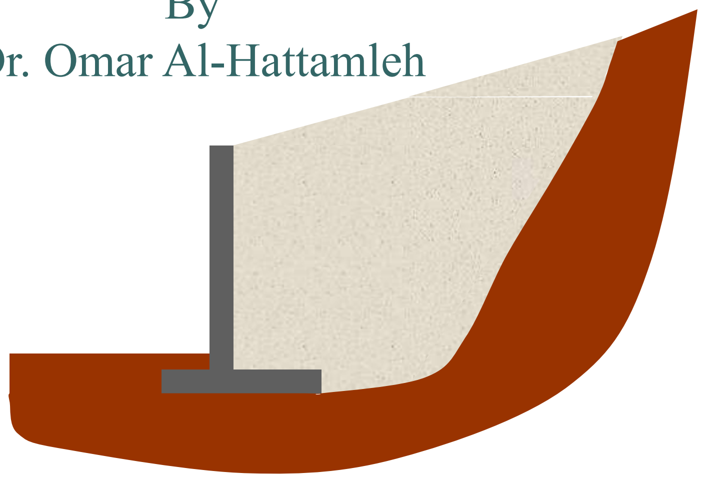
Using Coulomb $Fp = (0.5)(18.1)(10)^2(3.517) + 2(9)(10)\sqrt{(3.517)} = \underline{3{,}520 \text{ kN/m}}$

Lateral Earth Pressures

Retaining Walls - Applications

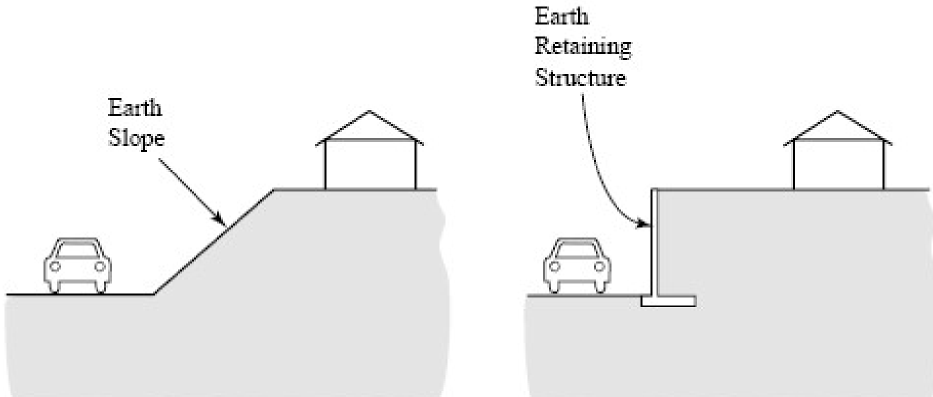
By

Dr. Omar Al-Hattamleh



Part of these slides are from Hattamlehkuga

Earth slopes and earth retaining structures



Used to maintain two different ground surface elevations

Types of Earth Retaining Structure

Fill Wall Construction

Externally Stabilized

Rigid Gravity and semi gravity Walls
- CIP Concrete Gravity Wall
- CIP concrete cantilever Wall
- Prefabricated Modular Gravity Wall
- Crib Wall
- Bin Wall
- Gabion Wall

Non Gravity Cantilevered Walls
- Sheet pile Wall
- Soldier pile and logging Wall
- Slurry Wall
- Tangent And Secant Wall
- SMW Wall
- Anchored Wall

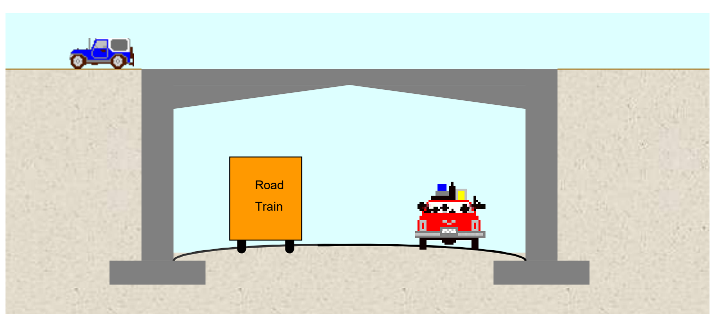
Internally Stabilized

MSE WALLS
- Segmented Pre-cast facing Sys.
- Prefabricated Modular Block MSE
- Geotextile, Geogrid MSE wall
- Reinforced Soil Slope

In Situ Reinforced Wall
- Soil Nail
- Micro-pile

Cut Wall Construction

Retaining Walls - Applications



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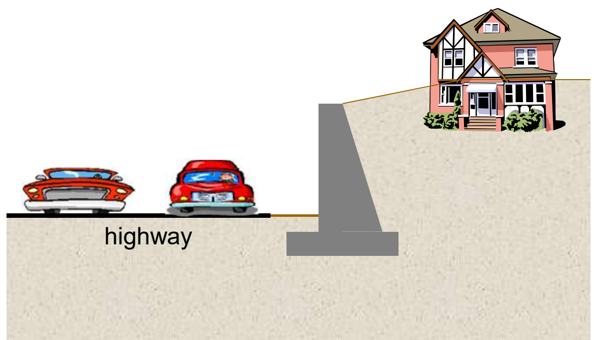
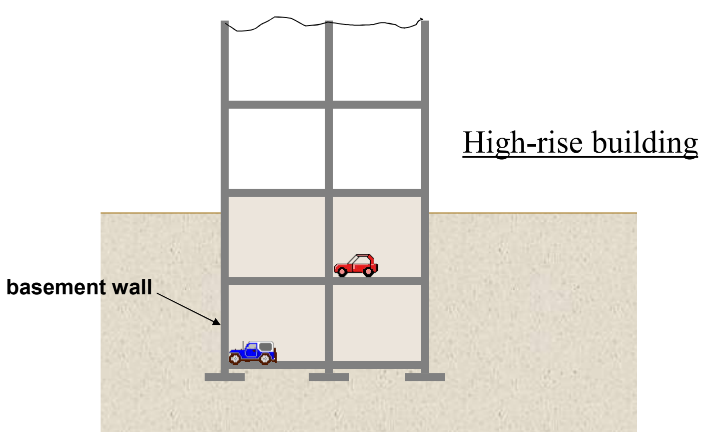
Retaining Walls - Applications



5

Retaining Walls - Applications



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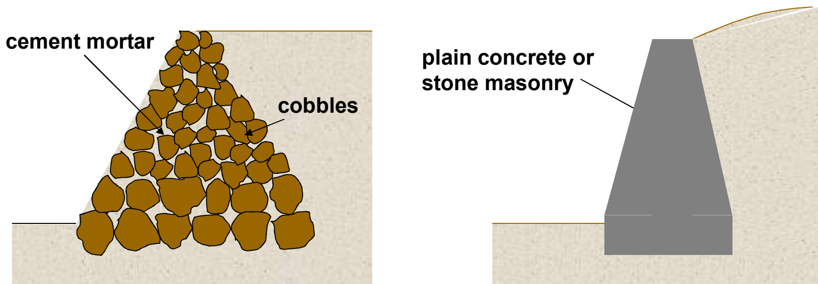
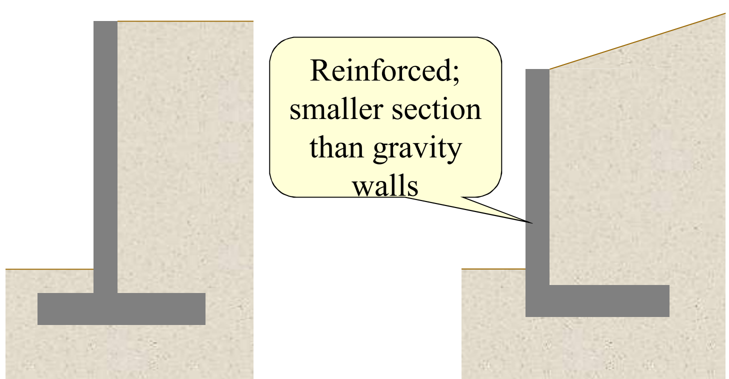
Gravity Retaining Walls



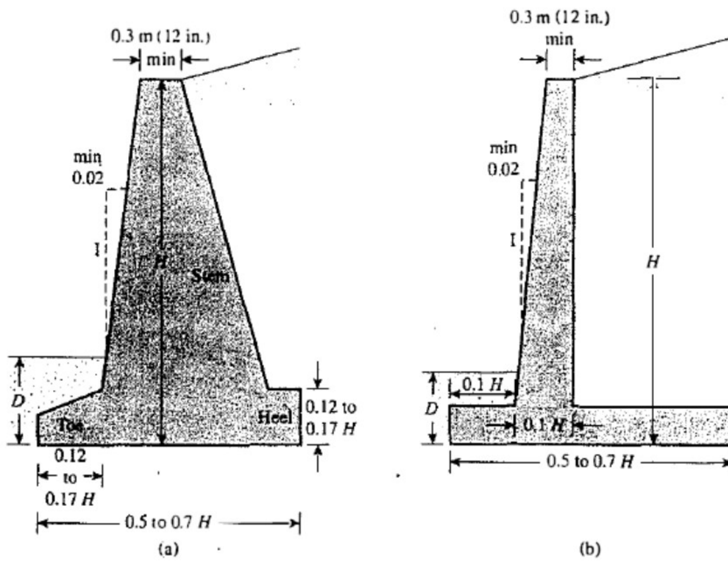
They rely on their self weight to support the backfill

Cantilever Retaining Walls



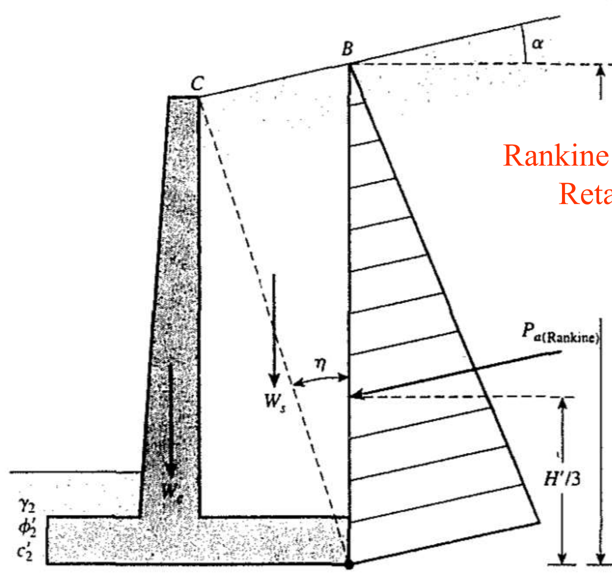
They act like vertical cantilever, fixed to the ground

Conventional Retaining Wall Design



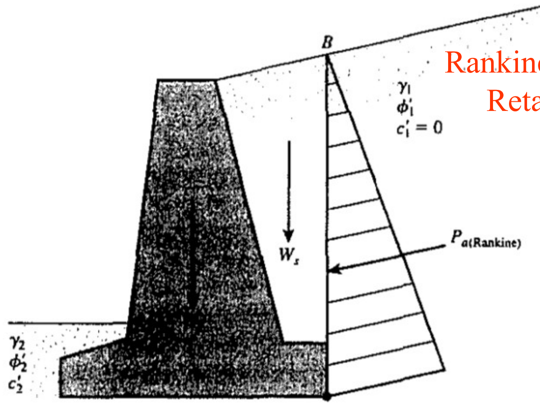
Approximate dimensions for various components of retaining wall for initial stability checks: (a) gravity wall; (b) cantilever wall

Application of Lateral Earth Pressure Theories to Design

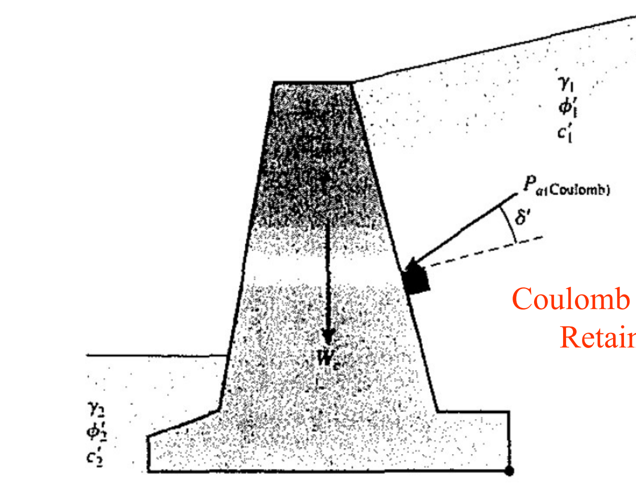


Rankine On Cantilever Retaining Wall

Application of Lateral Earth Pressure Theories to Design



Application of Lateral Earth Pressure Theories to Design

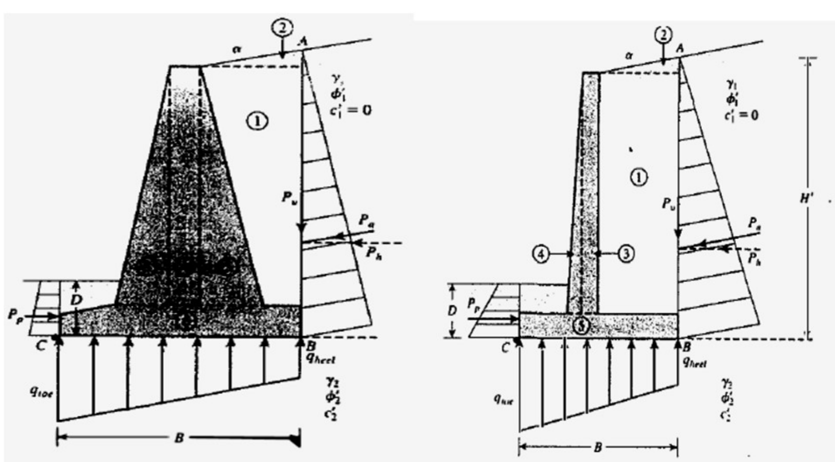


Conventional Retaining Wall Design

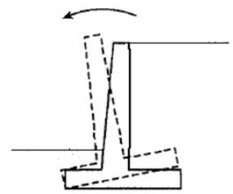
Stability Checks

1. Overturning about toe
2. Sliding along base
3. Bearing Capacity failure of the wall footing
   - q_{max}
   - q_{min}
   - e_{max}
4. Settlement
5. Overall stability (deep seated slide) failure

Conventional Retaining Wall Design



Overturning



$$F_{overturning} = \frac{\sum M_R}{\sum M_O} = \frac{P_P\ h/3 + \sum\{W_i x_i\}}{P_h\ H'/3}$$

$$F_{overturning} \geq 2 - 3$$

$\sum M_o$ = sum of the moment forces tending to Overturn the wall about the toe

$\sum M_R$ = sum of the moment forces resist the Overturn of the wall about the toe

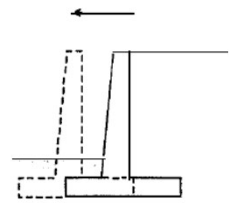
Procedure for Calculating $\sum M_R$

Section (1)	Area (2)	Weight/unit length of wall (3)	Moment arm measured from C (4)	Moment about C (5)
1	A_1	$W_1 = \gamma_1 \times A_1$	X_1	M_1
2	A_2	$W_2 = \gamma_2 \times A_2$	X_2	M_2
3	A_3	$W_3 = \gamma_c \times A_3$	X_3	M_3
4	A_4	$W_4 = \gamma_c \times A_4$	X_4	M_4
5	A_5	$W_5 = \gamma_c \times A_5$	X_5	M_5
6	A_6	$W_6 = \gamma_c \times A_6$	X_6	M_6
		P_v	B	M_v
		ΣV		ΣM_R

$$FS_{(overturning)} = \frac{M_1 + M_2 + M_3 + M_4 + M_5 + M_6 + M_v}{P_a \cos\alpha(H'/3)}$$

$$FS_{(overturning)} = \frac{M_1 + M_2 + M_3 + M_4 + M_5 + M_6}{P_a \cos\alpha(H'/3) - M_v}$$

Sliding along the base



$$FS_{(sliding)} = \frac{(\Sigma V)\tan(k_1\phi_2') + Bk_2c_2' + P_p}{P_a\cos\alpha}$$

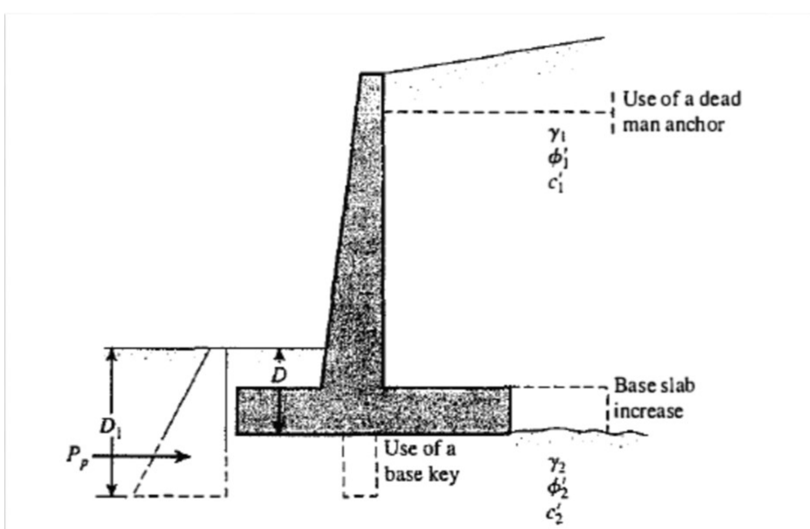
$FS_{sliding} \geq 1.5$

$k_1 = k_2 = 0.5 - 0.67$

If $F_{sliding}$ does not satisfy

1. Use stronger backfill
2. Install Tieback anchor (either on stem or on the base)
3. Extended the heel
4. Provide Key (*economical*)

If $FS_{sliding}$ does not satisfy



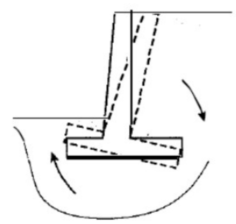
Bearing Capacity

$$e = \frac{B}{2} - \frac{\Sigma M_R - \Sigma M_o}{\Sigma V}$$

$$q_{toe} = q_{max} = \frac{\sum V}{B}(1 + \frac{6e}{B})$$

$$q_{heel} = q_{min} = \frac{\sum V}{B}(1 - \frac{6e}{B}) \succ 0.0$$

$$F_{bearing\ capacity} = \frac{q_{ult}}{q_{max}} \geq 3.0$$



q_{ult} = As before for wall (strip) footing

Bearing Capacity

$$q_u = c_2' N_c F_{cd} F_{ci} + q N_q F_{qd} F_{qi} + \tfrac{1}{2}\gamma_2 B' N_\gamma F_{\gamma d} F_{\gamma i}$$

$$q = \gamma_2 D$$

$$B' = B - 2e$$

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan \phi_2'} \qquad F_{qd} = 1 + 2 \tan \phi_2' (1 - \sin \phi_2')^2 \frac{D}{B'} \qquad F_{\gamma d} = 1$$

$$F_{ci} = F_{qi} = \left(1 - \frac{\psi^\circ}{90^\circ}\right)^2 \quad F_{\gamma i} = \left(1 - \frac{\psi^\circ}{\phi_2'^\circ}\right)^2 \quad \psi^\circ = \tan^{-1}\left(\frac{P_a \cos \alpha}{\Sigma V}\right)$$

$$\text{FS}_{(\text{bearing capacity})} = \frac{q_u}{q_{\max}}$$

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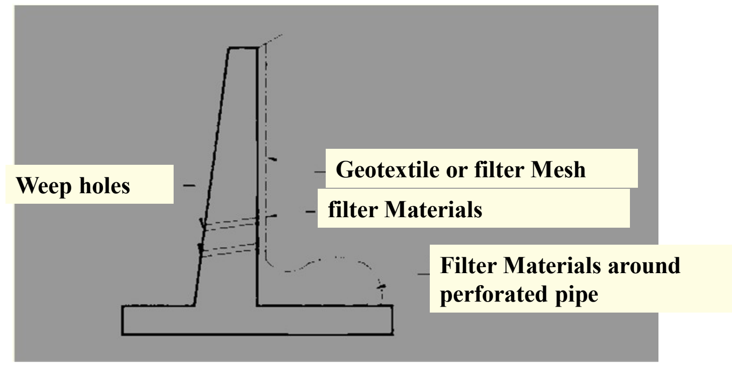
Backfill Drainage

- Build of water on the backfill would impose a sizable increase in the horizontal loading upon the wall. A fail proof system of drainage is very important, either through weepholes or geotextile backwall drainage systems

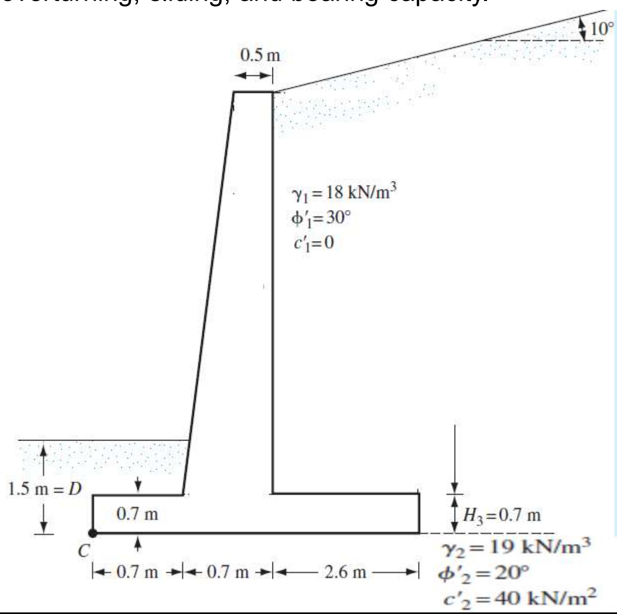


Backfill Drainage

- Weep-holes are holes of at least 0.1 m diameter, spaced every 2-4 m horizontally. In order to prevent weep-hole clogging, or loss of backfill fines, with a consequent settlement of the backfill, a filter blanket is placed between the weep-holes and the backfill.
- When soil filters are used, two factors are used: (1) the backfill fines can not wash into the filter, and (2) an excessive hydrostatic pressure head is not created behind the filtered weep-hole due to the low permeability of the filter soil.
- These two conditions are met when, for
  - (1) $D_{15(F)}/D_{85(B)} < 5$ and
  - (2) $D_{15(F)}/D_{15(B)} > 4$
  - (3) $D_{50(F)}$ should be less than $25D_{50(B)}$
  - (4) $D_{15(F)}$ should be less than $20D_{15(B)}$

Where D15(F) is the diameter of the particles of soil that could pass 15% in the filter fabric, and D85(B) is the diameter of the particles of soil in the backfill. Where F refers to filter soil and B to backfill soil. The D15 and D85 refer to the diameter of the particles of the soil that the percentage of the soil would pass through.

The cross section of a cantilever retaining wall with stem of 6.00 m is shown in Figure below Calculate the factors of safety with respect to overturning, sliding, and bearing capacity.



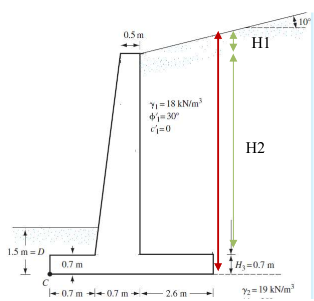
23

$\gamma_{concrete} = 23.58$ kN/m^3

$H' = H_1 + H_2 + H_3 = 2.6 \tan 10° + 6 + 0.7$

$$K_a = \cos\alpha \frac{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\phi'}}{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi'}}$$

α=10 **and** ϕ=30 **ka**=0.3532

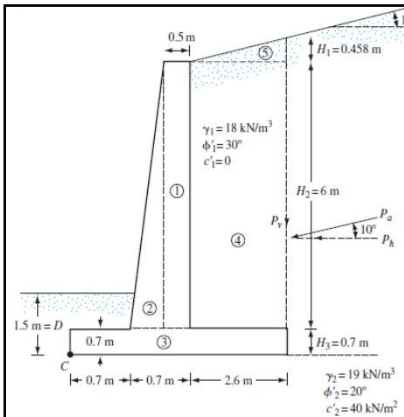


Pa =0.5 (18) $(7.158)^2$(0.3532) =162.9 kN/m

$Ph = Pa \cos 10° = 162.9 (\cos 10°) = 160.43$ kN/m

$Pv = Pa \sin 10° = 162.9 (\sin 10°) = 28.29$ kN/m

24



Section no.*	Area (m²)	Weight/unit length (kN/m)	Moment arm from point C (m)	Moment (kN-m/m)
1	$6 \times 0.5 = 3$	70.74	1.15	81.35
2	$\frac{1}{2}(0.2)6 = 0.6$	14.15	0.833	11.79
3	$4 \times 0.7 = 2.8$	66.02	2.0	132.04
4	$6 \times 2.6 = 15.6$	280.80	2.7	758.16
5	$\frac{1}{2}(2.6)(0.458) = 0.595$	10.71	3.13	33.52
		$P_v = 28.29$	4.0	113.16
		$\Sigma V = 470.71$		$1130.02 = \Sigma M_R$

$$M_o = P_h\left(\frac{H'}{3}\right) = 160.43\left(\frac{7.158}{3}\right) = 382.79 \text{ kN-m/m}$$

$$FS_{\text{(overturning)}} = \frac{\Sigma M_R}{M_o} = \frac{1130.02}{382.79} = \mathbf{2.95 > 2, OK}$$

$$FS_{\text{(sliding)}} = \frac{(\Sigma V)\tan(k_1\phi'_2) + Bk_2c'_2 + P_p}{P_a\cos\alpha}$$

$$P_p = \tfrac{1}{2}K_p\gamma_2 D^2 + 2c'_2\sqrt{K_p}D \qquad K_p = \tan^2\left(45 + \frac{\phi'_2}{2}\right) = \tan^2(45 + 10) = 2.04$$

$$P_p = \tfrac{1}{2}(2.04)(19)(1.5)^2 + 2(40)(\sqrt{2.04})(1.5) = 43.61 + 171.39 = 215 \text{ kN/m}$$

$$FS_{\text{(sliding)}} = \frac{(470.71)\tan\left(\frac{2 \times 20}{3}\right) + (4)\left(\frac{2}{3}\right)(40) + 215}{160.43}$$

$$= \frac{111.56 + 106.67 + 215}{160.43} = \mathbf{2.7 > 1.5, OK}$$

25

$$e = \frac{B}{2} - \frac{\Sigma M_R - \Sigma M_o}{\Sigma V} = \frac{4}{2} - \frac{1130.02 - 382.79}{470.71}$$

$$= 0.411 \text{ m} < \frac{B}{6} = \frac{4}{6} = 0.666 \text{ m}$$

$$q_{\substack{\text{toe}\\\text{heel}}} = \frac{\Sigma V}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{470.71}{4}\left(1 \pm \frac{6 \times 0.411}{4}\right) = 190.2 \text{ kN/m}^2 \text{ (toe)}$$

$$= 45.13 \text{ kN/m}^2 \text{ (heel)}$$

$$q_u = c'_2N_cF_{cd}F_{ci} + qN_qF_{qd}F_{qi} + \frac{1}{2}\gamma_2B'N_\gamma F_{\gamma d}F_{\gamma i}$$

For $\phi'_2 = 20°$ $\quad N_c = 14.83$, $N_q = 6.4$, and $N_\gamma = 5.39$. $\quad q = \gamma_2 D = (19)(1.5) = 28.5 \text{ kN/m}^2$

$$B' = B - 2e = 4 - 2(0.411) = 3.178 \text{ m}$$

$$F_{qd} = 1 + 2\tan\phi'_2(1 - \sin\phi'_2)^2\left(\frac{D}{B'}\right) = 1 + 0.315\left(\frac{1.5}{3.178}\right) = 1.118$$

Use B instead of B'

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c\tan\phi'_2} = 1.148 - \frac{1 - 1.148}{(14.83)(\tan 20)} = 1.1399$$

$$F_{\gamma d} = 1$$

26

$$F_{ci} = F_{qi} = \left(1 - \frac{\psi^\circ}{90^\circ}\right)^2$$

$$\psi = \tan^{-1}\left(\frac{P_a \cos\alpha}{\Sigma V}\right) = \tan^{-1}\left(\frac{160.43}{470.71}\right) = 18.82^\circ$$

$$F_{ci} = F_{qi} = \left(1 - \frac{18.82}{90}\right)^2 = 0.626$$

$$F_{\gamma i} = \left(1 - \frac{\psi}{\phi'_2}\right)^2 = \left(1 - \frac{18.82}{20}\right)^2 \approx 0$$ 0.0034

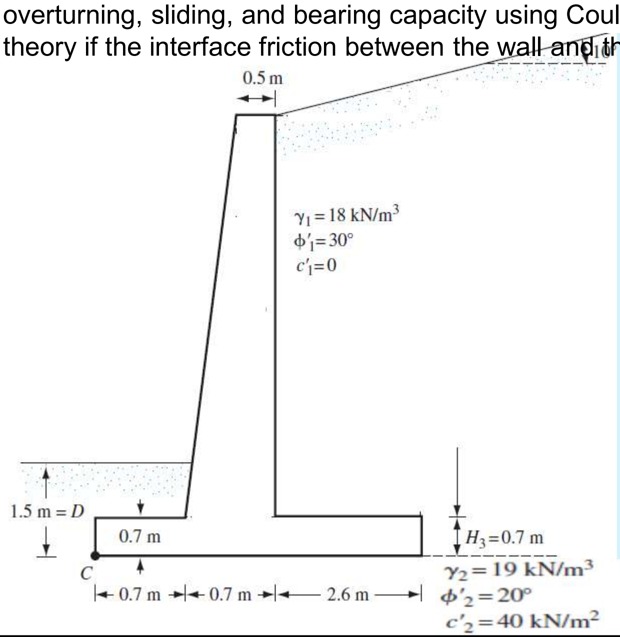
$$q_u = (40)(14.83)(1.1399c)(0.626) + (28.5)(6.4)(1.118)(0.626) + \tfrac{1}{2}(19)(5.93)(3.178)(1)(0.0034)$$
$$= 436.33 + 131.08 + 0 = 567.41 \text{ kN/m}^2$$

423.294+127.656+0.6087=551.56 kPa

$$\text{FS}_{\text{(bearing capacity)}} = \frac{q_u}{q_{\text{toe}}} = \frac{551.56}{190.2} = 2.899 = 2.90$$

27

The cross section of a cantilever retaining wall with stem of 6.00 m is shown in Figure below Calculate the factors of safety with respect to overturning, sliding, and bearing capacity using Coulomb earth pressure theory if the interface friction between the wall and the soil is 0.6667ϕ



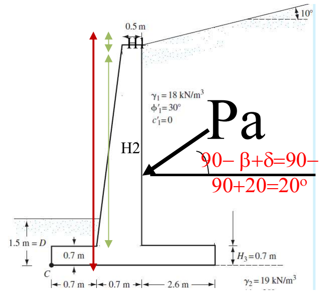
28

$\gamma_{concrete} = 23.58$ kN/m³

$H' = H_1 + H_2 + H_3 = 2.6 \tan 10° + 6 + 0.7$ =6.7 m

K_a = Coulomb's active earth pressure coefficient

$$= \frac{\sin^2(\beta + \phi')}{\sin^2\beta \sin(\beta - \delta')\left[1 + \sqrt{\frac{\sin(\phi' + \delta')\sin(\phi' - \alpha)}{\sin(\beta - \delta')\sin(\alpha + \beta)}}\right]^2}$$



β=90°; α=10°; ϕ'=30°;δ=20°

Ka=0.3400

Pa =0.5 (18) (6.7)²(0.3400) =137.3634 kN/m

Ph = *Pa* cos20° = 137.3634 (cos20°) = 129.08 kN/m

*P*v = *Pa* sin20° = 137.3634 (sin20°) = 46.981 kN/m

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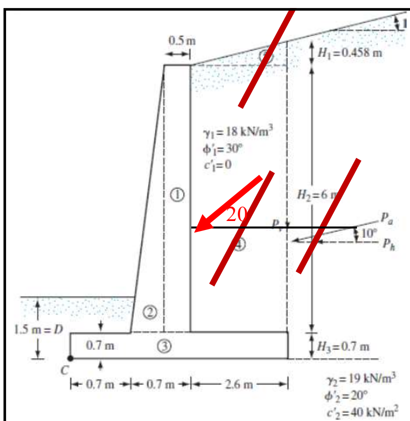


Section no.*	Area (m²)	Weight/unit length (kN/m)	Moment arm from point *C* (m)	Moment (kN-m/m)
1	$6 \times 0.5 = 3$	70.74	1.15	81.35
2	$\frac{1}{2}(0.2)6 = 0.6$	14.15	0.833	11.79
3	$4 \times 0.7 = 2.8$	66.02	2.0	132.04
~~4~~	[illegible]	[illegible]	[illegible]	[illegible]
~~5~~	[illegible]	46.98	1.4	65.772=Mv
		ΣV = 197.891		290.952 = ΣM_R

$M_o = P_h\left(\frac{H'}{3}\right) =$ 129.08 x6.7/3=288.2787 kN.m/m.

$FS_{(overturning)} = \frac{\Sigma M_R}{M_o} =$ 290.952/288.2787=1.0093

$FS_{(sliding)} = \frac{(\Sigma V)\tan(k_1\phi'_2) + Bk_2c'_2 + P_p}{P_a\cos\alpha}$

$P_p = \frac{1}{2}K_p\gamma_2D^2 + 2c'_2\sqrt{K_p}D$

Coulomb; α=0.0, δ'=13.33°;
β=tan⁻¹(6.0/0.2)=71.565°; ϕ'=20°; kp≈2.70?

$$P_p = \tfrac{1}{2}(2.70)(19)(1.5)^2 + 2(40)(\sqrt{2.70})(1.5)$$
$$= 43.61 + 171.39 = 254.9 \text{ kN/m}$$

30

$$FS_{\text{(sliding)}} = \frac{197.891 \tan\left(\frac{2\times 20}{3}\right) + (4)\left(\frac{2}{3}\right)(40) + 254.9}{129.08}$$

=3.164

$$e = \frac{B}{2} - \frac{\Sigma M_R - \Sigma M_o}{\Sigma V} = \frac{4}{2} - \frac{1130.02 - 382.79}{470.71}$$

See example 8.2, Text book

=4/2-(290.952-288.2787)/197.89=1.986m>b/6!!! qmin will fail!

$$q_{\substack{\text{toe}\\\text{heel}}} = \frac{\Sigma V}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{470.71}{4}\left(1 \pm \frac{6\times 0.411}{4}\right) = 190.2\ \text{kN/m}^2\ \text{(toe)}$$
$$= 45.1\ \text{kN/m}^2\ \text{(heel)}$$

$$q_u = c_2' N_c F_{cd} F_{ci} + q N_q F_{qd} F_{qi} + \frac{1}{2}\gamma_2 B' N_\gamma F_{\gamma d} F_{\gamma i}$$

For $\phi_2' = 20°$ $N_c = 14.83$, $N_q = 6.4$, and $N_\gamma = 5.39$. $q = \gamma_2 D = (19)(1.5) = 28.5\ \text{kN/m}^2$

$$B' = B - 2e = 4 - 2(0.411) = 3.178\ \text{m}$$

$$F_{qd} = 1 + 2\tan\phi_2'(1 - \sin\phi_2')^2\left(\frac{D}{B'}\right) = 1 + 0.315\left(\frac{1.5}{3.178}\right) = 1.118$$

Use B instead of B'

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan\phi_2'} = 1.148 - \frac{1 - 1.148}{(14.83)(\tan 20)} = 1.1399$$

31

$$F_{\gamma d} = 1$$

$$F_{ci} = F_{qi} = \left(1 - \frac{\psi°}{90°}\right)^2$$

$$\psi = \tan^{-1}\left(\frac{P_a \cos\alpha}{\Sigma V}\right) = \tan^{-1}\left(\frac{160.43}{470.71}\right) = 18.82°$$

$$F_{ci} = F_{qi} = \left(1 - \frac{18.82}{90}\right)^2 = 0.626$$

$$F_{\gamma i} = \left(1 - \frac{\psi}{\phi_2'}\right)^2 = \left(1 - \frac{18.82}{20}\right)^2 \approx 0$$ 0.0034

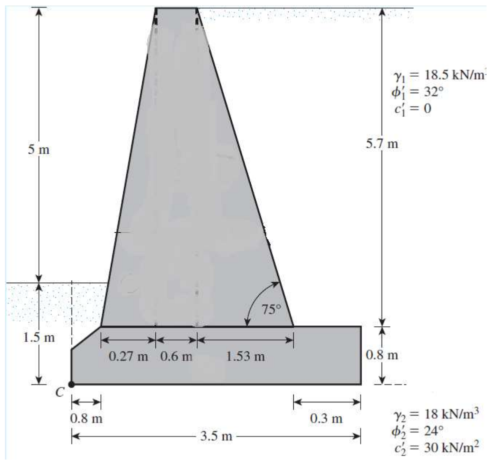
$$q_u = (40)(14.83)(1.1399)(0.626) + (28.5)(6.4)(1.118)(0.626) + \tfrac{1}{2}(19)(5.93)(3.178)(1)(0.0034)$$
$$= 436.33 + 131.08 + 0 = 567.41\ \text{kN/m}^2$$

423.294+127.656+0.6087=551.56 kPa

$$FS_{\text{(bearing capacity)}} = \frac{q_u}{q_{\text{toe}}} = \frac{551.56}{190.2} = 2.899=2.90$$

32

EX: A gravity retaining wall is shown in Figure shown. Use $\delta' = 2/3\phi'_1$ and Coulomb's active earth pressure theory. Determine



a. The factor of safety against overturning
b. The factor of safety against sliding
c. The pressure on the soil at the toe and heel

33

Solution

The height $H' = 5 + 1.5 = 6.5$ m

Coulomb's active force is $P_a = \frac{1}{2}\gamma_1 H'^2 K_a$

With $\alpha = 0°$, $\beta = 75°$, $\delta' = 2/3\phi'_1$, and $\phi'_1 = 32°$

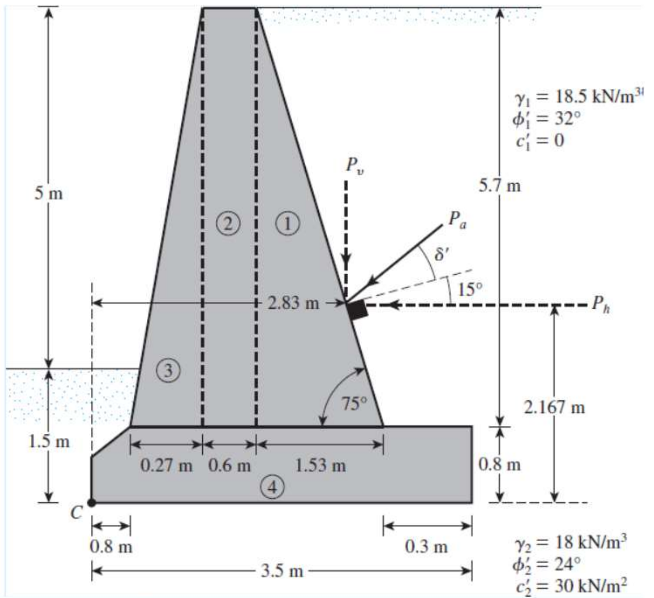
K_a = Coulomb's active earth pressure coefficient

$$= \frac{\sin^2(\beta + \phi')}{\sin^2\beta \sin(\beta - \delta')\left[1 + \sqrt{\frac{\sin(\phi' + \delta')\sin(\phi' - \alpha)}{\sin(\beta - \delta')\sin(\alpha + \beta)}}\right]^2}$$

$K_a = 0.4023$

$P_a = \frac{1}{2}(18.5)(6.5)^2(0.4023) = 157.22$ kN/m

34



$$P_h = P_a \cos(15 + \tfrac{2}{3}\phi'_1) = 157.22 \cos 36.33 = 126.65 \text{ kN/m}$$

35

$$P_v = P_a \sin(15 + \tfrac{2}{3}\phi'_1) = 157.22 \sin 36.33 = 93.14 \text{ kN/m}$$

Part a: Factor of Safety against Overturning

Area no.	Area (m^2)	Weight* (kN/m)	Moment arm from C (m)	Moment (kN-m/m)
1	$\frac{1}{2}(5.7)(1.53) = 4.36$	102.81	2.18	224.13
2	$(0.6)(5.7) = 3.42$	80.64	1.37	110.48
3	$\frac{1}{2}(0.27)(5.7) = 0.77$	18.16	0.98	17.80
4	$\approx (3.5)(0.8) = 2.8$	66.02	1.75	115.54
		$P_v = 93.14$	2.83	263.59
		$\Sigma V = 360.77$ kN/m		$\Sigma M_R = 731.54$ kN-m/m

$$\text{Overturning moment} = M_o = P_h\left(\frac{H'}{3}\right) = 126.65(2.167) = 274.45 \text{ kN-m/m}$$

$$\text{FS}_{(\text{overturning})} = \frac{\Sigma M_R}{\Sigma M_o} = \frac{731.54}{274.45} = \mathbf{2.67 > 2, OK}$$

36

*$\gamma_{concrete} = 23.58$ kN/m³

$$\mathrm{FS}_{(\text{sliding})} = \frac{(\Sigma V)\tan\left(\frac{2}{3}\phi_2'\right) + \frac{2}{3}c_2'B + P_p}{P_h}$$

$$P_p = \tfrac{1}{2}K_p\gamma_2 D^2 + 2c_2'\sqrt{K_p}D$$

$$K_p = \tan^2\left(45 + \frac{24}{2}\right) = 2.37$$

$$P_p = \tfrac{1}{2}(2.37)(18)(1.5)^2 + 2(30)(1.54)(1.5) = 186.59 \text{ kN/m}$$

$$\mathrm{FS}_{(\text{sliding})} = \frac{360.77\tan\left(\frac{2}{3}\times 24\right) + \frac{2}{3}(30)(3.5) + 186.59}{126.65}$$

$$= \frac{103.45 + 70 + 186.59}{126.65} = \mathbf{2.84}$$

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Part c: Pressure on Soil at Toe and Heel

$$e = \frac{B}{2} - \frac{\Sigma M_R - \Sigma M_o}{\Sigma V}$$

$$= \frac{3.5}{2} - \frac{731.54 - 274.45}{360.77} = 0.483 < \frac{B}{6} = 0.583$$

$$q_{\text{toe}} = \frac{\Sigma V}{B}\left[1 + \frac{6e}{B}\right]$$

$$= \frac{360.77}{3.5}\left[1 + \frac{(6)(0.483)}{3.5}\right] = \mathbf{188.43\ kN/m^2}$$

$$q_{\text{heel}} = \frac{V}{B}\left[1 - \frac{6e}{B}\right]$$

$$= \frac{360.77}{3.5}\left[1 - \frac{(6)(0.483)}{3.5}\right] = \mathbf{17.73\ kN/m^2}$$

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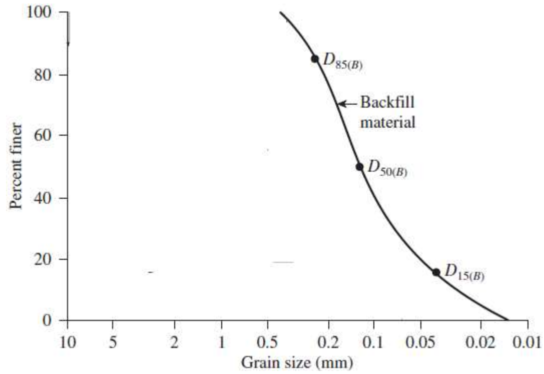
Ex: Figure below shows the grain-size distribution of a backfill material. Using the conditions of filter material requiremnt, determine the range of the grain-size distribution for the filter material.



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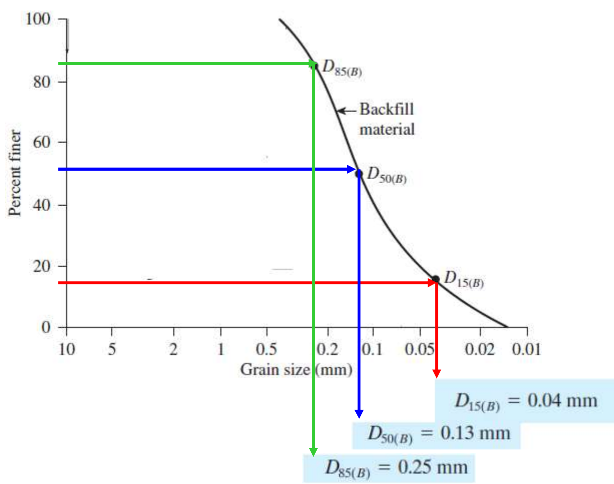
Solution

From the grain-size distribution curve given in the figure, the following values can be
determined:



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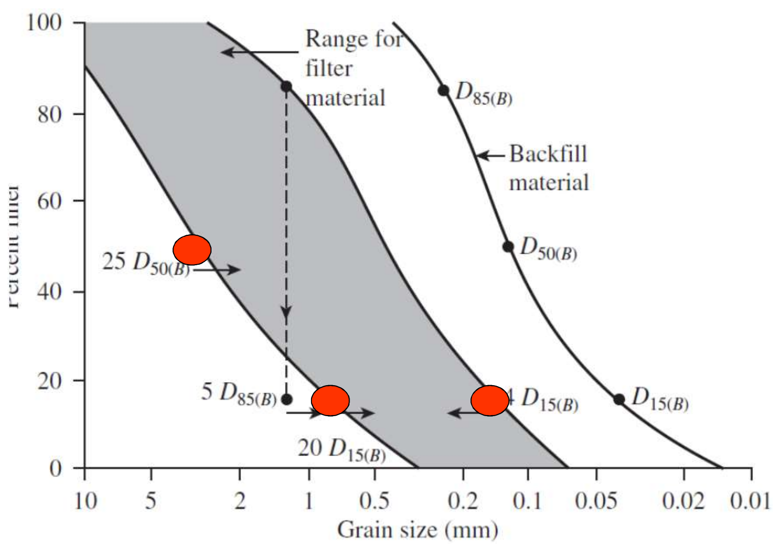
Conditions of Filter

1. $D_{15(F)}$ should be less than $5D_{85(B)}$; that is, $5 \times 0.25 = 1.25$ mm.
2. $D_{15(F)}$ should be greater than $4D_{15(B)}$; that is, $4 \times 0.04 = 0.16$ mm.
3. $D_{50(F)}$ should be less than $25D_{50(B)}$; that is, $25 \times 0.13 = 3.25$ mm.
4. $D_{15(F)}$ should be less than $20D_{15(B)}$; that is, $20 \times 0.04 = 0.8$ mm.

41



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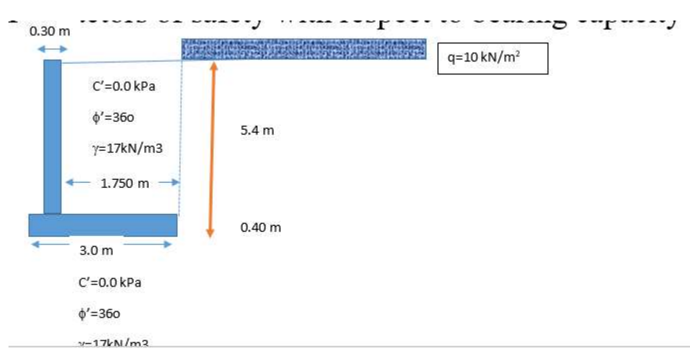
Ex 2: The cress section of the wall is designed with Rankine earth pressure theory. If the ∑V=∑Ai=212.3 kN/m and ∑Mr= ∑Mv=397.3 kN.m/m and there is no soil to the right of the wall.



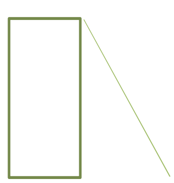
1) Calculate the earth pressure thrust

2) The factor of safety with respect to overturning

3) The factor of safety with respect to sliding,

Solution

Ka=(1-sinϕ)/(1+sinϕ)=(1-sin36)/(1+sin36)=0.2596



Pa=0.5x17x5.4x5.4x0.2596

+10x5.4x0.2596

=64.344+14.019

=78.36 kN/m

1) The factor of safety with respect to overturning

FS=Mr/Mo=397.3/(64.344x5.4/3+14.019x5.4/2)=2.5857

The factor of safety with respect to sliding

$$FS_{(sliding)} = \frac{(\Sigma V)\tan(k_1\phi_2') + Bk_2c_2' + P_p}{P_a\cos\alpha}$$

$FS_{sliding}$=(212.3xtan(2*36/3)+3.000x(2x0/3)+0.0)/78.36=1.213<1.5????

The factors of safety with respect to bearing capacity

e=3/2-(397.3-153.6488)/(212.3)=0.3523m<B/6=3/6=0.5 m

B'=B-2e=3.00-2x0.3523=2.295 m

$$q_u = c_2'N_cF_{cd}F_{ci} + qN_qF_{qd}F_{qi} + \frac{1}{2}\gamma_2B'N_\gamma F_{\gamma d}F_{\gamma i}$$

qu=0.00+0.00+0.5(17)(2.295)(Nγ)(1.000)(1.00)(Fγi)

$F\gamma i=(1-\psi/\phi)^2=(1-20.26/36)^2=0.19118$

$\psi=\tan^{-1}(Ph/\sum V)=\tan^{-1}(78.36/212.3)=20.26^o$

45

ϕ'=36° General Bearing capacity Factors Table

Nγ=56.32

qu=0.00+0.00+0.5(17)(2.295)(56.32)(1.000)(1.00)(0.1911)

=209.9544 kPa

$$q_{toe} = q_{max} = \frac{\sum V}{B}(1+\frac{6e}{B})$$

$$q_{heel} = q_{min} = \frac{\sum V}{B}(1-\frac{6e}{B}) \succ 0.0$$

qmax=(212.3/3)(1+6x0.3523/3)

=120.629 kPa

qmin=(212.3/3)(1-6x0.3523/3)

=20.904 kPa

FS=qult/qmax=209.9544/120.629=1.74>1.0

(note better to have > 3)

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Find the depth of a key below the base to let the footing just safe (FS=1.5)?

$$FS_{(sliding)} = \frac{(\Sigma V)\tan(k_1\phi'_2) + Bk_2c'_2 + P_p}{P_a\cos\alpha}$$

1.50==(212.3xtan(2*36/3)+3.000x(2x0/3)+Pp)/78.36

P_P=23.02kPa

$$P_p = \tfrac{1}{2}K_p\gamma_2 D^2 + 2c'_2\sqrt{K_p}D$$

Kp=(1+sinϕ)/(1-sinϕ)=(1+sin36)/(1-sin36)=3.852

23.02=0.5(3.852)(17)(D^2)+0

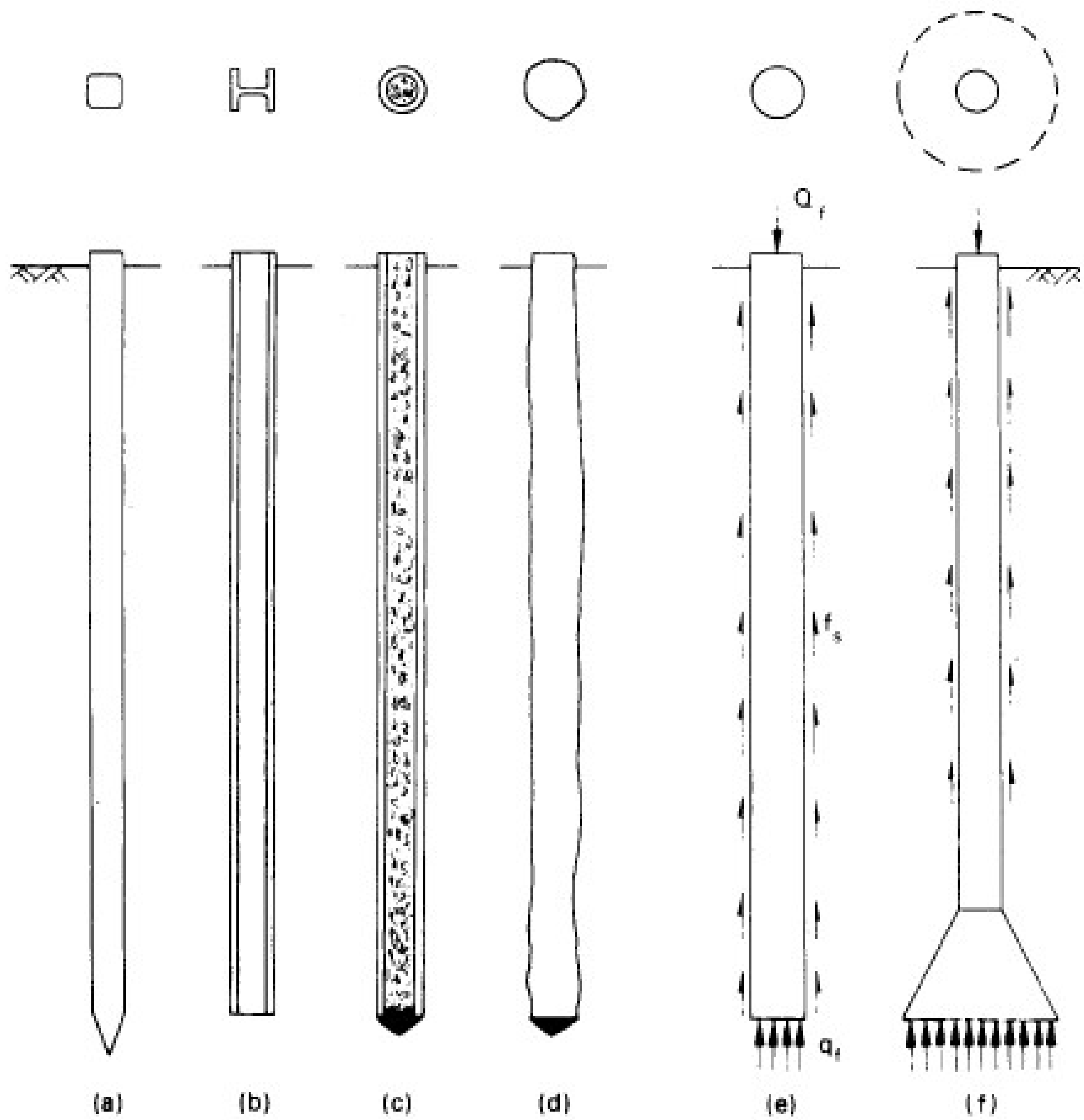
D=0.838 m=0.84 m

Typical pile characteristics and uses

Dr. Omar Hattamleh

Principal types of deep Foundation:

(a)Precast RC Pile,
(b)Steel H Pile,
(c)Shell Pile,
(d)Concrete Pile Cast As Driven Tube Withdrawn
(e)Bored Pile (Cast In Situ) And
(a)Under-reamed Bored Pile (Cast In Situ).



Purpose of a Deep Foundation

The purpose of a deep foundation is to transmit the structural loads to a stratum that is capable of providing both bearing capacity and acceptable settlements. The deep foundation must be also capable of resisting vertical compressive, lateral and uplift loads.

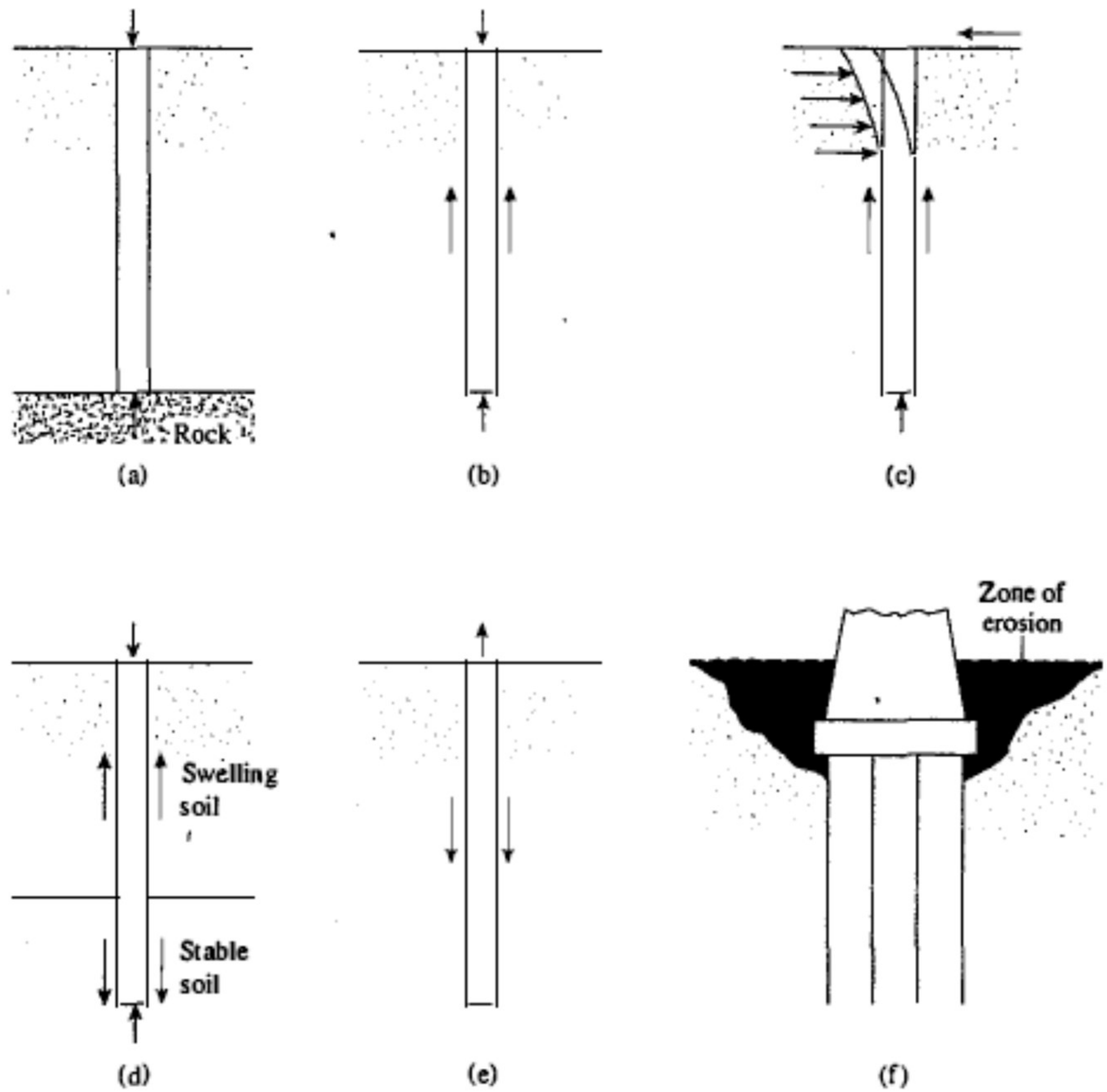
Piles are commonly used for,

1. To carry structure loads into or through a soil stratum.
2. To resist uplift or overturning forces.
3. To control settlements when spread footings are on marginal or highly compressible soil.
4. To control scour problems on bridge abutments or piers.
5. In offshore construction to transmit loads through the water and into the underlying soil.
6. To control earth movements, such as landslides.

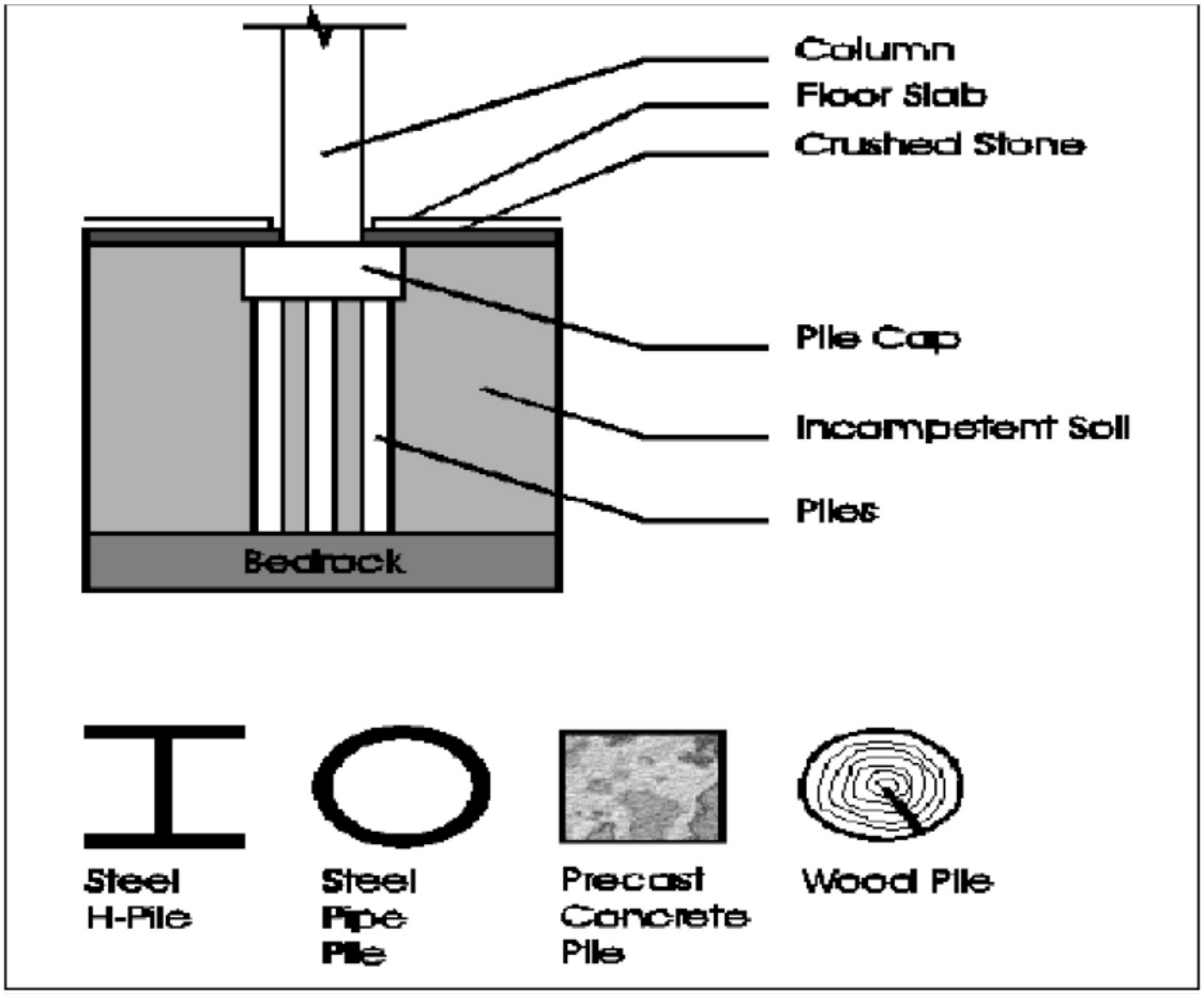
Piles are inserted into the soil by the following methods:

1. Driving using a pile hammer.
2. Driving using a vibratory device.
3. Jacking the pile.
4. Drilling a hole (pre-drilling) and inserting a pile into it.
5. Screwed into the ground and injected with a column of grout (augercast shafts).

Conditions for the pile foundation



Piles' Cross section



common types of Deep Foundations

The most common types of Deep Foundations

Driven:

1. ***Timber piles***
2. ***Steel and composite piles***
3. ***Precast prestressed concrete piles***
4. ***Pressure injected footings***
5. ***Pin piles, geo-piles, soil nailing***

Placed:

1. ***Auger-cast***
2. ***Drilled shafts (with steel casing or with slurry)***
3. ***Under-reamed or belled shafts***
4. ***Pin piles.***

Classification of pile w.r.t their effect on the soil.

Driven

Driven piles are considered to be displacement piles. In the process of driving the pile into the ground, soil is moved radically as the pile shaft enters the ground. There may also be a component of movement of the soil in The vertical direction.

Bored or placed

Generally a non-displacement pile, where a void is formed by boring, and concrete is cast into the void. Stiff clays are particularly amenable, since the bore hole walls do not requires temporary support except cloth to the ground surface. In unstable ground, such as gravel the ground requires temporary support from a steel casing or using a Bentonite slurry

Classification of piles with respect to load transmission and functional behavior

- ***End bearing piles (point bearing piles).*** Transfer their load on to a firm stratum below the base of the structure. 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 Even in weak soil a pile will not fail by buckling and this effect need only be considered if part of the pile is unsupported, i.e. if it is in either air or water. Load is transmitted to the soil through friction or cohesion. But 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. The founding depth of the pile is influenced by the results of the site investigate on and soil test.
- ***Friction piles (cohesion piles).*** Carrying capacity is derived mainly from Carrying capacity is derived mainly from the adhesion or friction of the soil in contact with the shaft of the pile**.** These piles transmit most of their load to the soil through skin friction. This process of driving such piles close to each other in groups greatly reduces the porosity and compressibility of the soil within and around the groups. Therefore piles of this category are some times called compaction piles. During the process of driving the pile into the ground, the soil becomes molded and, as a result loses some of its strength. Therefore the pile is not able to transfer the exact amount of load which it is intended to immediately after it has been driven. Usually, the soil regains some of its strength three to five months after it has been driven.
- ***Combination of friction and cohesion piles.***

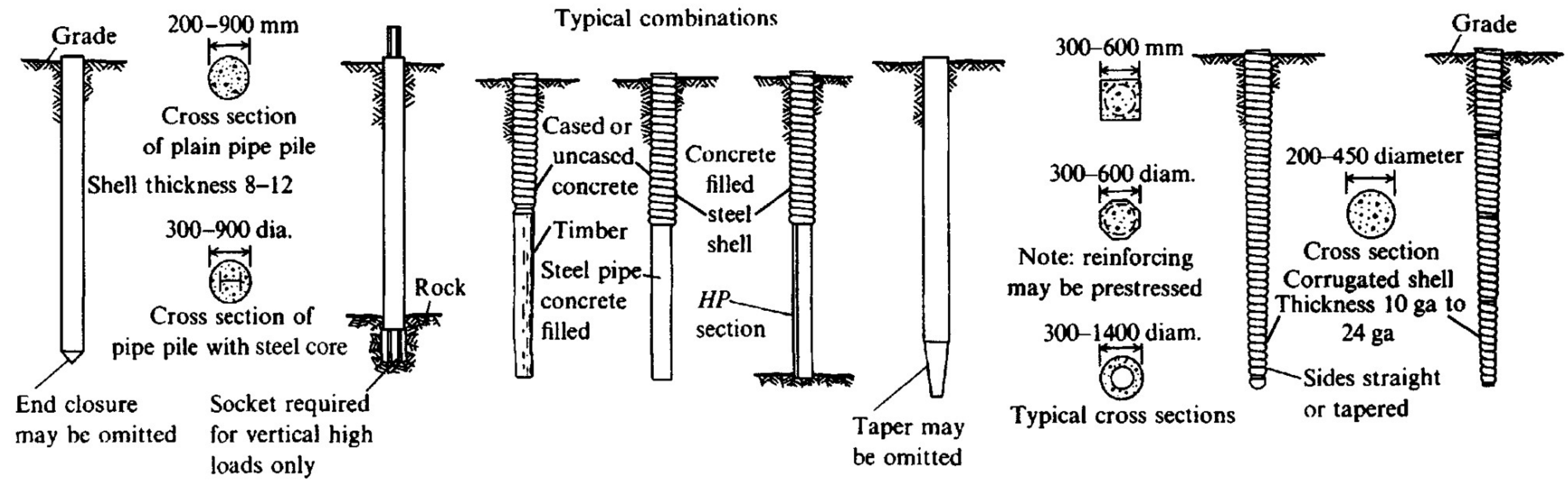
Typical pile characteristics and uses

Pile type	Timber	Steel	Cast-in-place concrete piles (shells driven without mandrel)	Cast-in-place concrete piles (shells withdrawn)
Maximum length	35 m	Practically unlimited	10–25 m	36 m
Optimum length	9–20 m	12–50 m	9–25 m	8–12 m
Applicable material specifications	ASTM-D25 for piles; P1-54 for quality of creosote; C1-60 for creosote treatment (Standards of American Wood Preservers Assoc.)	ASTM-A36, A252, A283, A572, A588 for structural sections ASTM-A1 for rail sections	ACI	ACI†
Recommended maximum stresses	Measured at midpoint of length: 4–6 MPa for cedar, western hemlock, Norway pine, spruce, and depending on Code. 5–8 MPa for southern pine, Douglas fir, oak, cypress, hickory	$f_s = 0.35\text{–}0.5 f_y$	$0.33 f_c'$; $0.4 f_c'$ if shell gauge ≤ 14; shell stress $= 0.35 f_y$ if thickness of shell ≥ 3 mm $f_c' \geq 18$ MPa	$0.25\text{–}0.33 f_c'$
Maximum load for usual conditions	450 kN	Maximum allowable stress × cross section	900 kN	1300 kN
Optimum load range	80–240 kN	350–1050 kN	450–700 kN	350–900 kN
Disadvantages	Difficult to splice Vulnerable to damage in hard driving Vulnerable to decay unless treated Difficult to pull and replace when broken during driving	Vulnerable to corrosion HP section may be damaged or deflected by major obstructions	Hard to splice after concreting Considerable displacement	Concrete should be placed in dry More than average dependence on quality of workmanship

Typical pile characteristics and uses continues

Pile type	Timber	Steel	Cast-in-place concrete piles (shells driven without mandrel)	Cast-in-place concrete piles (shells withdrawn)
Advantages	Comparatively low initial cost Permanently submerged piles are resistant to decay Easy to handle	Easy to splice High capacity Small displacement Able to penetrate through light obstructions	Can be redriven Shell not easily damaged	Initial economy
Remarks	Best suited for friction pile in granular material	Best suited for end bearing on rock Reduce allowable capacity for corrosive locations or provide corrosion protection	Best suited for friction piles of medium length	Allowable load on pedestal pile is controlled by bearing capacity of stratum immediately below pile

Typical illustrations



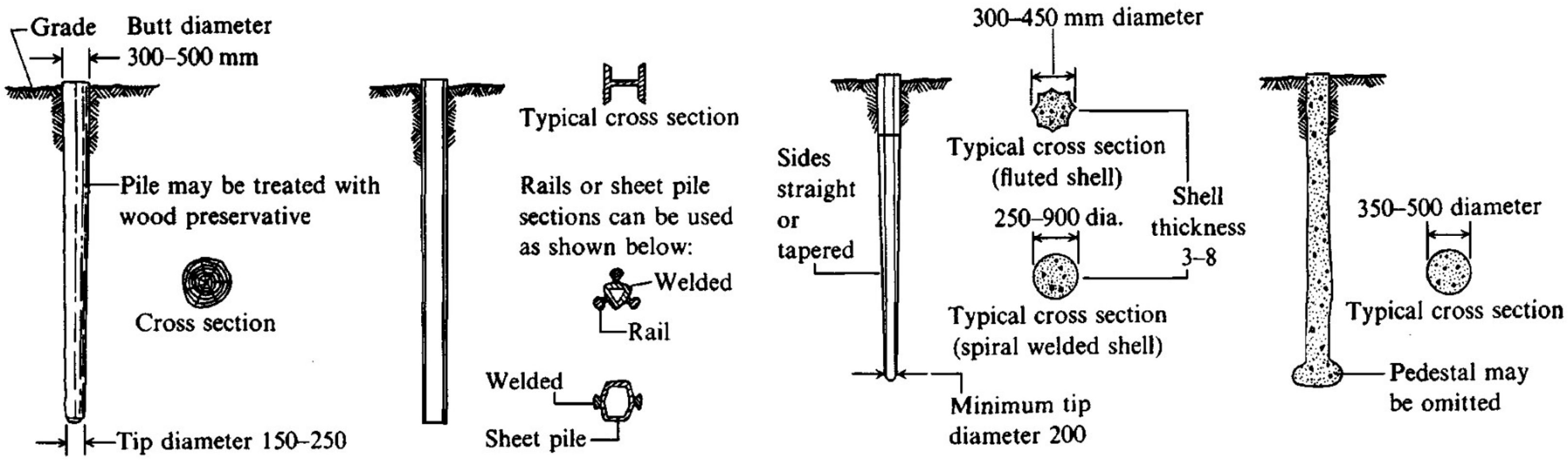
Typical pile characteristics and uses continues

Pile type	Concrete-filled steel pipe piles	Composite piles	Precast concrete (including prestressed)	Cast in place (thin shell driven with mandrel)	Auger-placed pressure-injected concrete (grout) piles
Maximum length	Practically unlimited	55 m	10–15 m for precast 20–30 m for prestressed	6–35 m for straight sections 12 m for tapered sections	5–25 m
Optimum length	12–36 m	18–36 m	10–12 m for precast 18–25 m for prestressed	12–18 m for straight 5–12 m for tapered	10–18 m
Applicable material specifications	ASTM A36 for core ASTM A252, A283 for pipe ACI Code 318 for concrete	ACI Code 318 for concrete ASTM A36 for structural section ASTM A252 for steel pipe ASTM D25 for timber	ASTM A15 reinforcing steel ASTM A82 cold-drawn wire ACI Code 318 for concrete $f'_c \geq 28$ MPa precast $f'_c \geq 35$ MPa prestressed	ACI	See ACI
Recommended maximum stresses	$0.40 f_y$ reinforcement < 205 MPa 0.35–$0.50 f_y$ for shell < 175 MPa $0.33 f'_c$ for concrete	Same as concrete in other piles Same as steel in other piles Same as timber piles for composite	$0.33 f'_c$ unless local building code is less $0.4 f_y$ for reinforced unless prestressed	$0.33 f'_c$; $f_s = 0.4 f_y$ if shell gauge ≤ 14 use $f_y = 0.35 f_y$ if shell thickness ≥ 3 mm	$0.25 f'_c$
Maximum load for usual conditions	1800 kN without cores 18 000 kN for large sections with steel cores	1800 kN	8500 kN for prestressed 900 kN for precast	675 kN	700 kN
Optimum load range	700–110 kN without cores 4500–14 000 kN with cores	250–725 kN	350–3500 kN	250–550 kN	350–900 kN
Disadvantages	High initial cost Displacement for closed-end pipe	Difficult to attain good joint between two materials	Difficult to handle unless prestressed High initial cost Considerable displacement Prestressed difficult to splice	Difficult to splice after concreting Redriving not recommended Thin shell vulnerable during driving Considerable displacement	Dependence on workmanship Not suitable in compressible soil

Typical pile characteristics and uses continues

Pile type	Concrete-filled steel pipe piles	Composite piles	Precast concrete (including prestressed)	Cast in place (thin shell driven with mandrel)	Auger-placed pressure-injected concrete (grout) piles
Advantages	Best control during installation No displacement for open-end installation Open-end pipe best against obstruction High load capacities Easy to splice	Considerable length can be provided at comparatively low cost	High load capacities Corrosion resistance can be attained Hard driving possible	Initial economy Tapered sections provide higher bearing resistance in granular stratum	Freedom from noise and vibration Economy High skin friction No splicing
Remarks	Provides high bending resistance where unsupported length is loaded laterally	The weakest of any material used shall govern allowable stresses and capacity	Cylinder piles in particular are suited for bending resistance	Best suited for medium-load friction piles in granular materials	Patented method

Typical illustrations



*Additional comments in *Practical Guidelines for the Selection, Design and Installation of Piles* by ASCE Committee on Deep Foundations, ASCE, 1984, 105 pages.

†ACI Committee 543, "Recommendations for Design, Manufacture, and Installation of Concrete Piles," *JACI,* August 1973, October 1974; also in ACI MCP 4 (reaffirmed 1980).

A common use of concrete precast piles is in marine sites where the soils are soft and loose.

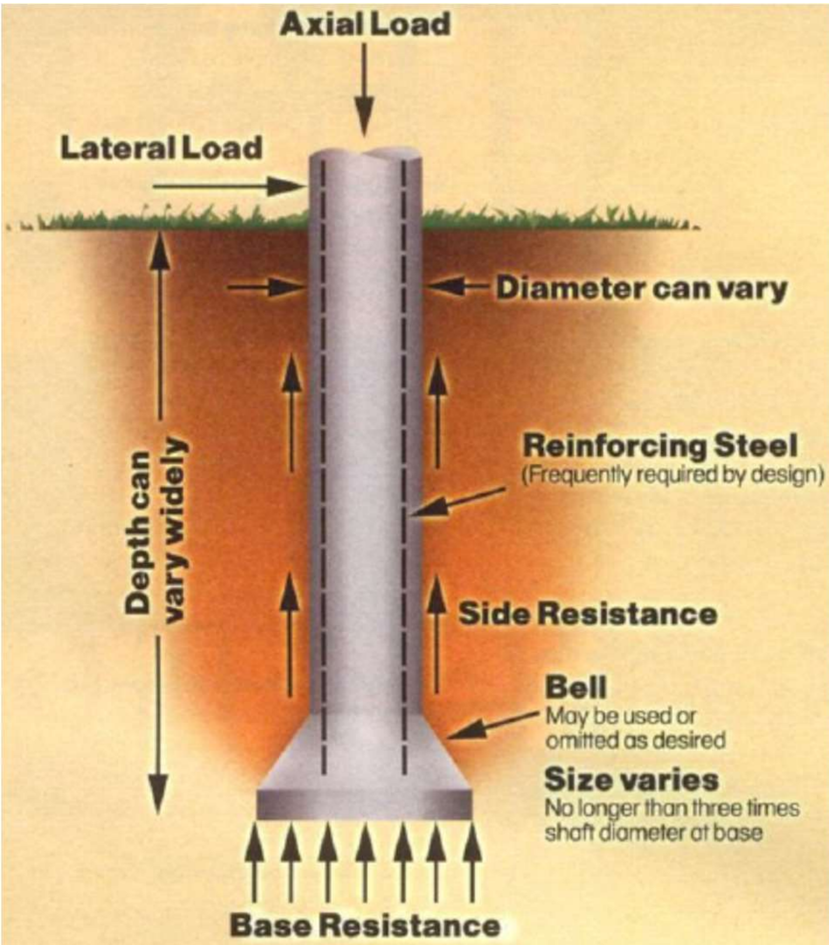
Driving Steel Piles

Dwelling Building – temper pile

Drilled Shafts

- Built by vibrating a steel casing into the ground and then filling it with concrete.
- Casings are removed as the concrete is being placed in the shaft.
- The casings are light, easy to handle, cut, and splice.
- The shafts are clean out and visually inspection before filling with concrete.
- In expansive soils, the shafts are filled as soon as possible to avoid damage due to lateral soil pressures.

Drilled Shafts



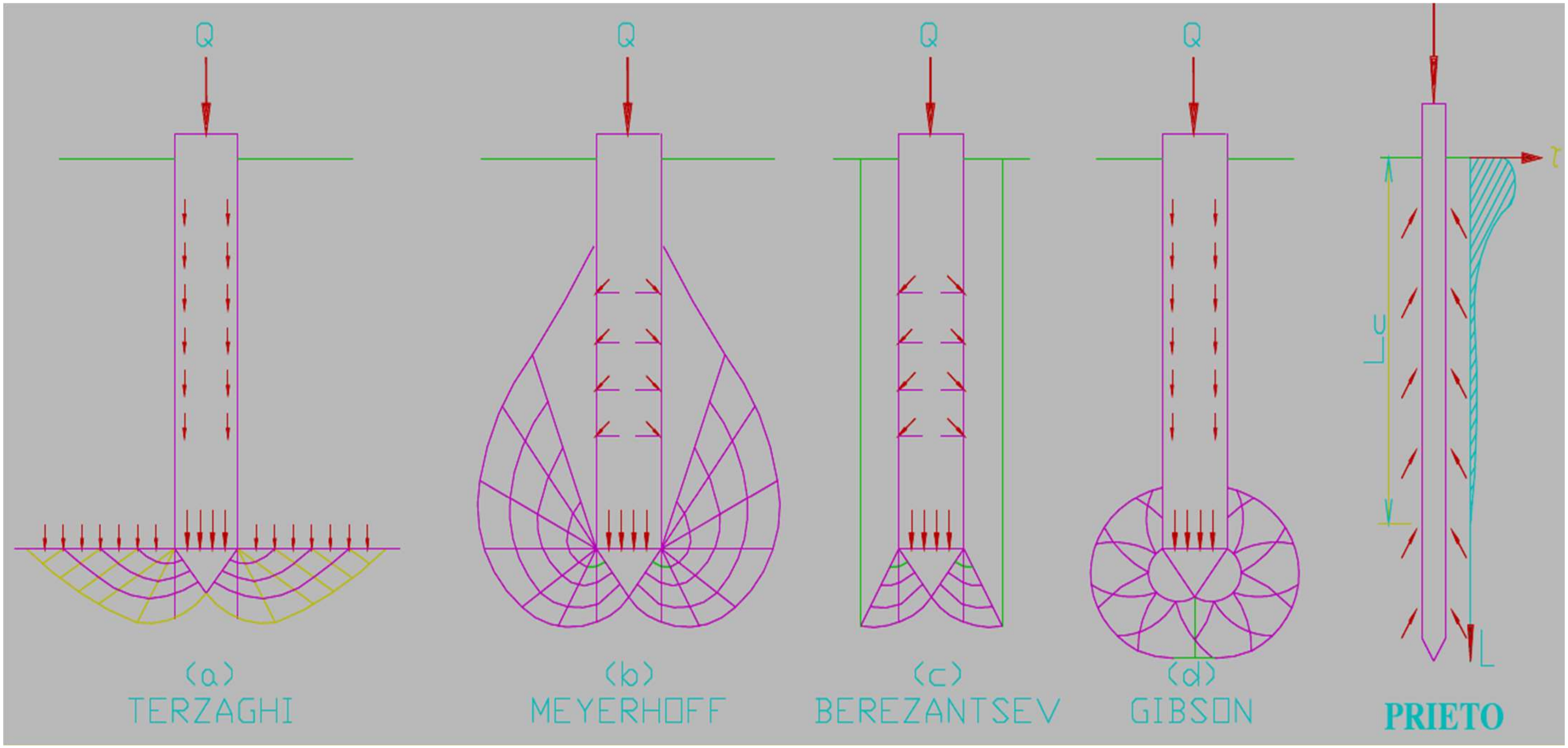
The elements of a drilled shaft. Notice the "bell" at the bottom of the shaft, also called "underreaming". This expanded shaft serves to increase the bearing area by as much as 50% of the shaft's capacity.

auger rig & bit

A 5-foot diameter auger bit,
being centered over the surveyed location of the shaft.

Transfer of Load

The way that the load from a column transfers into the soil through the pile has evolved during the past fifty years, from Terzaghi at the extreme left figure, through to Prieto (1978) on the extreme right.



The Behavior of Soils Around a Driven Pile.

The effect of pile is reflected in remolding the soil around the pile. Sands and clays respond to pile driving differently. First, we describe the behavior of clays and then the behavior of sands.

Clays.

The effects of pile driving in clays are listed in four major categories:

1. Remolding or disturbance to structure of the soil surrounding the pile
2. Changes of the state of stress in the soil in the vicinity of the pile
3. Dissipation of the excess pore pressure developed around the pile
4. Long term phenomena of strength regain in the soil

The essential difference between the actions of piles under dynamic and static loading is the fact that clays show pronounced time effects, and hence the show the greatest difference between dynamic and static action. These effects may be mechanistically described as follow.

Let us consider piles driven into a deep deposit of a soft impervious saturated clay. Since a pile has a volume of many cubic feet, an equal volume of clay must be displaced when the pile is driven.

The Behavior of Soils Around a Driven Pile

Sands

A pile in sand is usually installed by driving. The vibrations from driving a pile in sand have two effects:

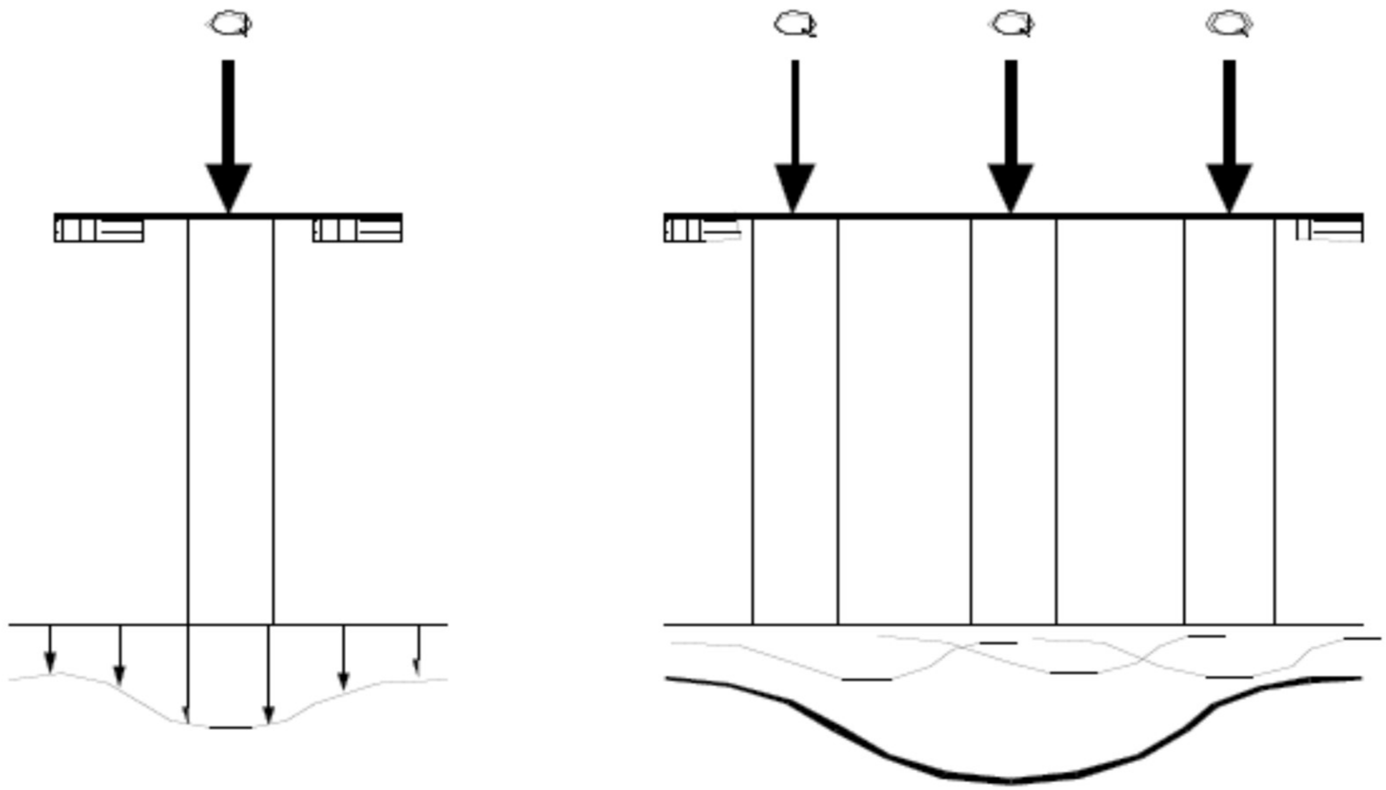
1. Densify the sand, and

2. Increase the value of lateral pressure around the pile.

- Penetration tests results in a sand prior to pile driving and after pile driving indicate significant densification of the sand for distances as large as eight diameters away from the center of the pile.
- Increasing the density results in an increase in friction angle.
- Driving of a pile displaces soil laterally and thus increases the horizontal stress acting on the pile.

Group Action of Piles

Piles are driven in groups at a spacing ranging from 3 to 4*B* where *B* is the diameter or side of a pile. The behavior of piles in a group may be quite different than that of a single pile if the piles are friction piles. This difference may not be so marked in bearing piles.



(a) Single pile

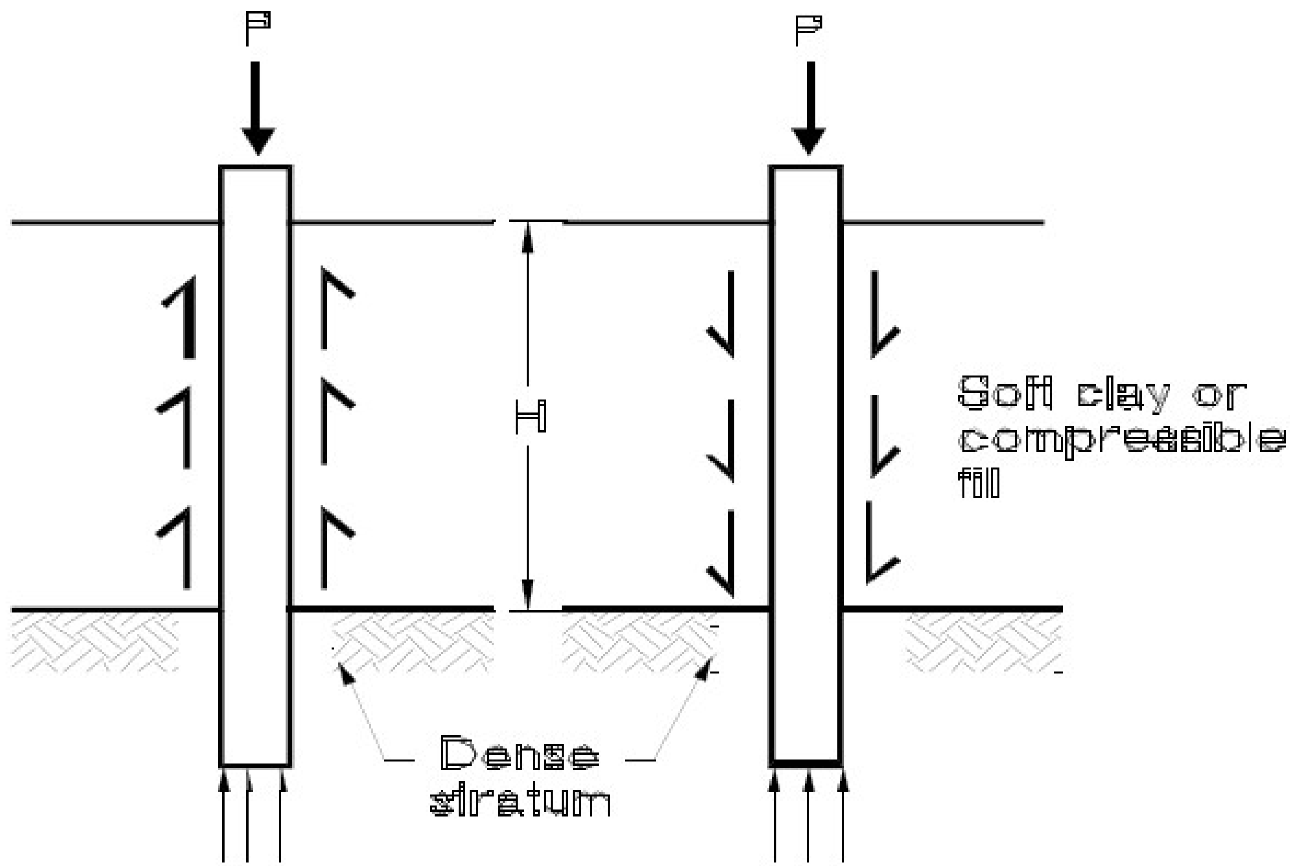
(b) Group of piles

Negative Skin Friction

If a pile is driven in a soft clay or recently placed fill and has its tip resting in a dense stratum , the settlement of both the pile and the soft clay or fill is taking place after the pile has been driven and loaded. During and immediately after driving, a portion of the load is resisted by adhesion of soft soil with pile. But, as consolidation of the soft clay proceeds, it transmits all the load onto the tip of the pile.

In case of a fill, the settlement of the fill may be greater than that of the pile. In the initial stages of consolidation of the fill, it transmit all the load resisted by adhesion onto the tip of the pile. A further settlement results in a downward drag on the pile. It is known as *negative skin friction*. Both these cases should be recognized in the field in the design of bearing piles. When this condition occurs, the pile must be capable of supporting the soil weight as well as all other loads that the pile is designed to carry. Also, if fill is to be placed around an existing pile foundation, the ability of the piles to carry the added load should be thoroughly investigated.

Piles in a soft soil overlying a dense strata



(a) Skin friction immediately and during pile driving,
(b) negative skin friction afterwards.

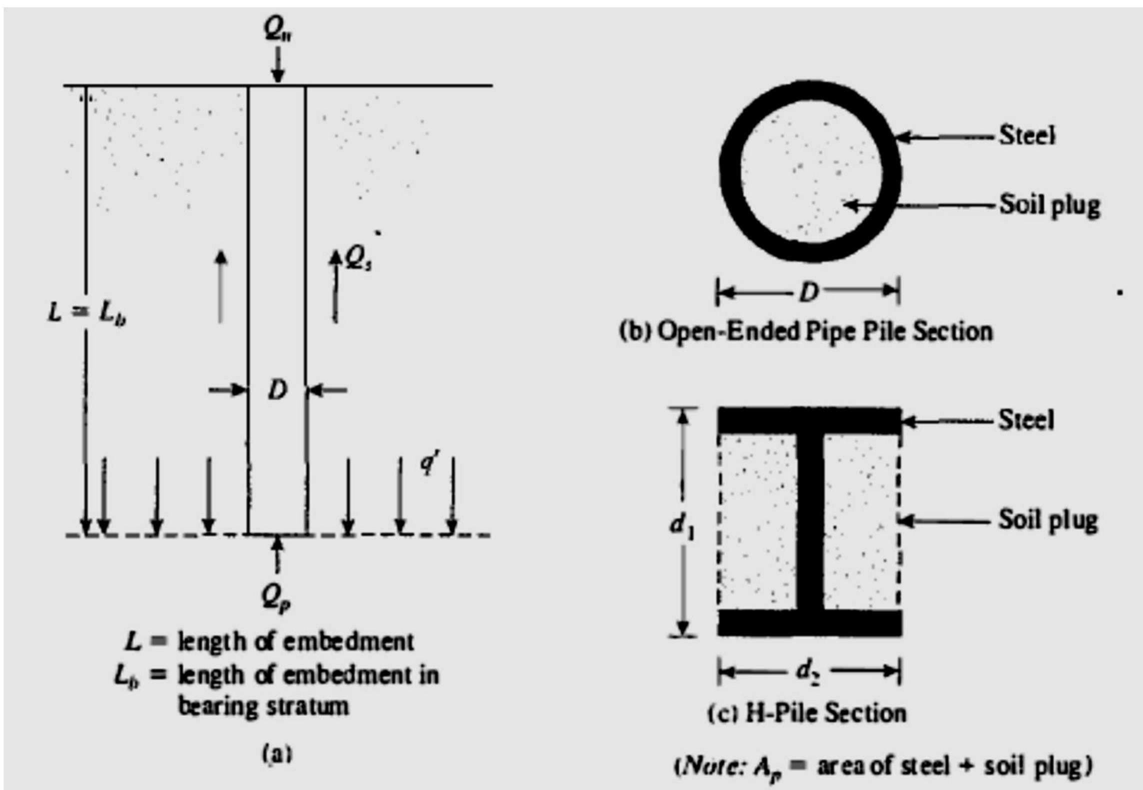
Static Pile Capacity

Equations for Estimating Pile Capacity

- The ultimate load-carrying capacity Q_u of a pile is given by the equation

$$Q_u = Q_p + Q_s$$

Q_p = load-carrying capacity of the pile point
Q_s = Frictional resistance (skin friction) derived from the soil-pile interface



Point Bearing Capacity

- Remember General Bearing Capacity given by

$$q_u = c'N_cF_{cs}F_{cd} + qN_qF_{qs}F_{qd} + \frac{1}{2}\gamma BN_\gamma F_{\gamma s}F_{\gamma d}$$

- It can be written as

$$q_u = c'N_c^* + qN_q^* + \gamma BN_\gamma^*$$

- For pile with width , D can be written as

$$q_u = q_p = c'N_c^* + qN_q^* + \gamma DN_\gamma^*$$

- Since D is small, third term become negligible

$$q_p = c'N_c^* + q'N_q^*$$

- load-carrying capacity of the pile point is given as

$$Q_p = A_pq_p = A_p(c'N_c^* + q'N_q^*)$$

where Ap = area of pile tip
c' = cohesion of the soil supporting the pile tip
q_p = unit point resistance
q' = effective vertical stress at the level of the pile tip
N^*_C Nq^* = the bearing capacity factors

The Frictional Resistance

- The Frictional, skin, or shaft resistance of a pile may be written as

$$Q_s = \Sigma\, p\, \Delta L\, f$$

where

p = perimeter of the pile section
ΔL = incremental pile length over which p and f are taken to be constant
f = unit friction resistance at any depth z

- Allowable Load, Q_{all}

$$Q_{all} = \frac{Q_u}{FS}$$

where
Q_{all} = Allowable load-carrying capacity for each pile
FS= Factor of Safety

Static Pile Capacity

- All static pile capacities can be computed by the following equations:

$$\left.\begin{aligned} P_u &= P_{\mathrm{pu}} + \sum P_{si} \\ &= P_p + \sum P_{si,u} \end{aligned}\right\} \quad \text{(compression)}$$

$$T_u = \sum P_{si,u} + W_p \qquad \text{(tension)}$$

where *Pu* = ultimate (maximum) pile capacity in compression
Tu = ultimate pullout capacity
Ppu = ultimate pile tip capacity
Psi,u =ultimate skin resistance capacity
Pp = tip capacity
W = weight of pile being pulled
Σ= summation process over I soil layers making up the soil profile over length of pile shaft embedment

Static Pile Capacity

$$P_a = \frac{P_u}{\mathrm{SF}} \qquad \text{or} \qquad T_a = \frac{T_u}{\mathrm{SF}}$$

This value of *Pa* or *Ta* should be compatible with the capacity based on the pile material (timber, concrete, or steel) considered earlier; and SF/ represents the safety factors, which commonly range from 2.0 to 4 or more, depending on designer uncertainties.

Meyerhof's Method for Estimating Qp

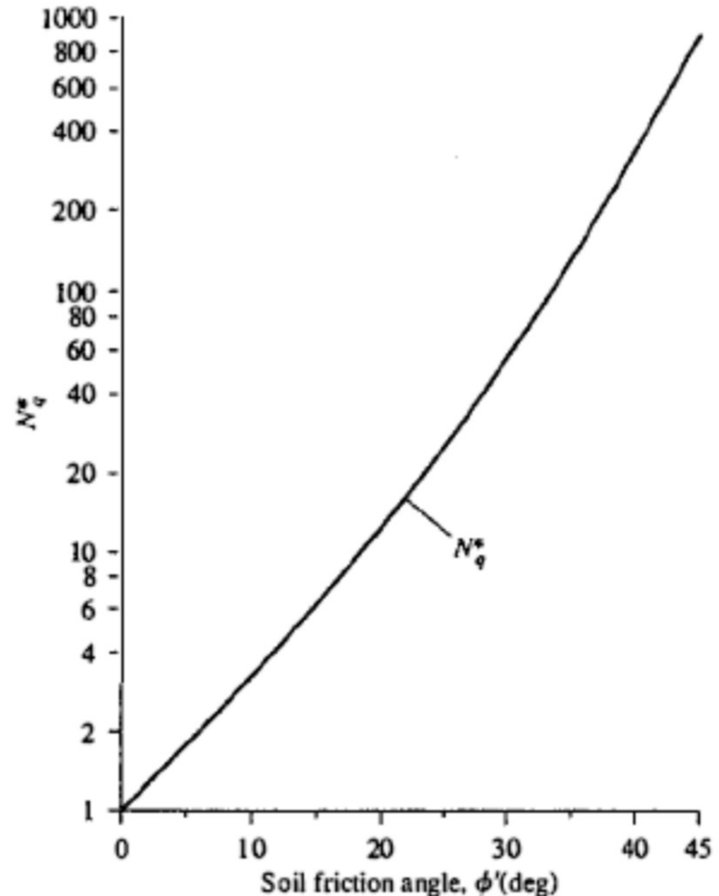
❖ For tip Pile rest on Sand

$$Q_p = A_p q_p = A_p q' N_q^*$$

Shall not exceed

$$q_l = 0.5\, p_a N_q^* \tan \phi'$$

where p, = atmospheric pressure
(=100 kN/m2 or 2000 Ib/ft^2)
ϕ' = effective soil friction angle of the bearing stratum



❖Clay ($\phi_T = 0$)

For piles in saturated clays under undrained conditions ($\phi^T = 0$),

$$Q_p = N_c^* c_u A_p = 9 c_u A_p$$

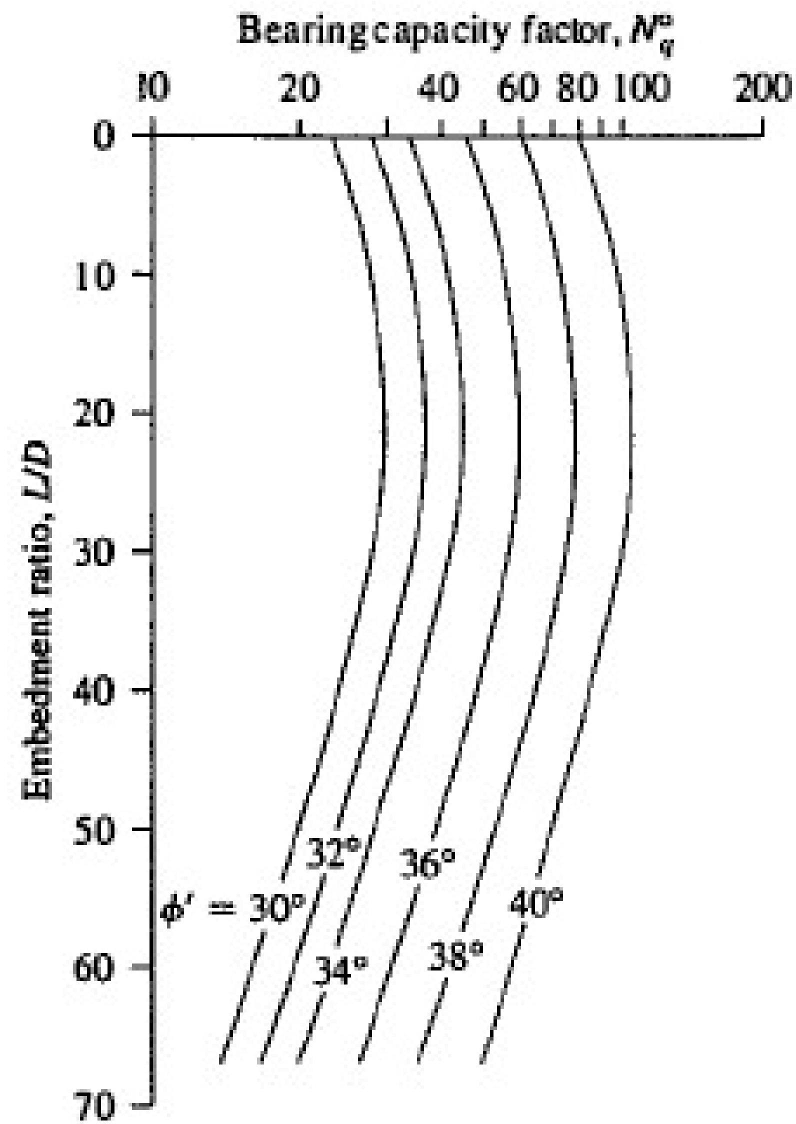
where c_u = undrained cohesion of the soil below the tip of the pile.

Coyle and Castello's Method

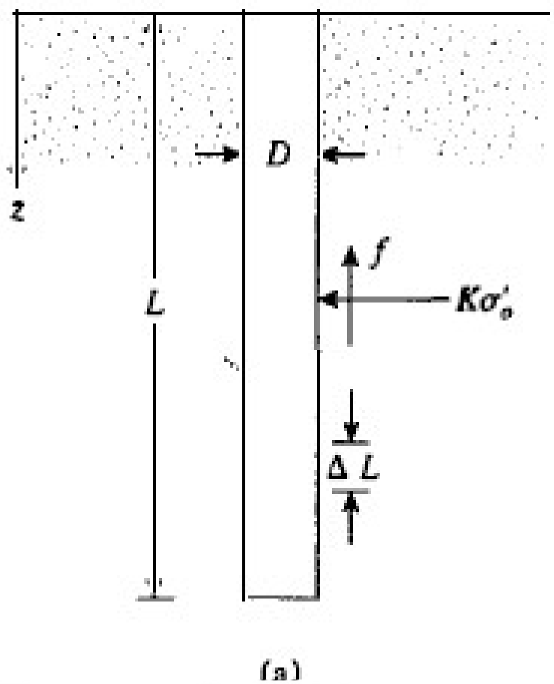
- Coyle and Castello's Method Applied for Estimating Qp in Driven Pile in Sand

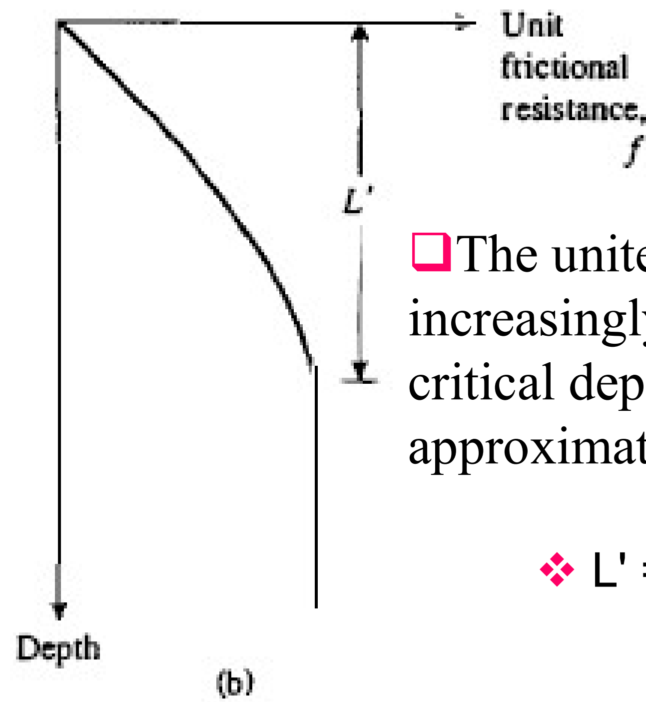
$$Q_p = q' N_q^* A_p$$

where
q' = effective vertical stress at the pile tip
N*q = bearing capacity factor



Frictional Resistance (Qs) in Sand





- The unite skin friction increasingly with depth until critical depth L' which approximately 15 to 20 D

  - L' = 15D

For $z = 0$ to L',

$$f = K\sigma'_o \tan \delta'$$

and for $z = L'$ to L,

$$f = f_{z=L'}$$

Where $\sigma'o$ is the effective stress at each level considered

δ′ is the friction angle between the soil and the pile or the shaft: δ′ ranges between 0.5ϕ′ to 0.80ϕ′

Pile type	K
Bored or jetted	$\approx K_o = 1 - \sin\phi'$
Low-displacement driven	$\approx K_o = 1 - \sin\phi'$ to $1.4K_o = 1.4(1 - \sin\phi')$
High-displacement driven	$\approx K_o = 1 - \sin\phi'$ to $1.8K_o = 1.8(1 - \sin\phi')$

Coyle and Castello's Method

- Coyle and Castello's Method Applied for Estimating Qs in Driven Pile in Sand

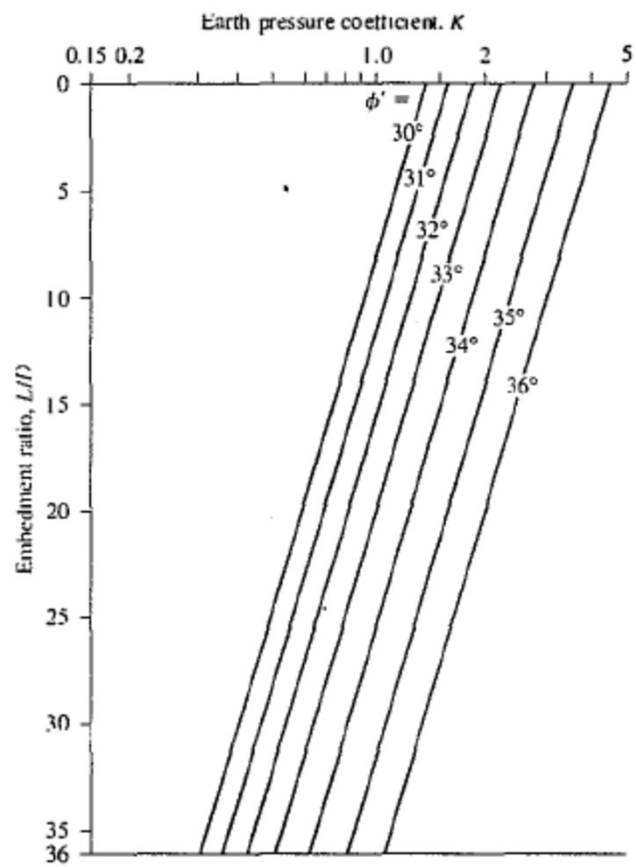
$$Q_s = f_{av} p L = (K \bar{\sigma}'_o \tan \delta') p L$$

where

$\bar{\sigma}'_o$ = average effective overburden pressure

δ'= soil-pile friction angle = $0.8\phi'$

K from the chart



Frictional (Skin) Resistance in Clay

- Three method exist

1. α method 2. β method 3. λ method

- λ method

$$f_{av} = \lambda(\bar{\sigma}'_o + 2c_u)$$

$\bar{\sigma}'_o$ = mean effective vertical stress for the entire embedment length

c_u= mean (weighted average) undrained shear strength (ϕ_u = 0)

Table 11.7 Variation of λ with pile embedment length, L

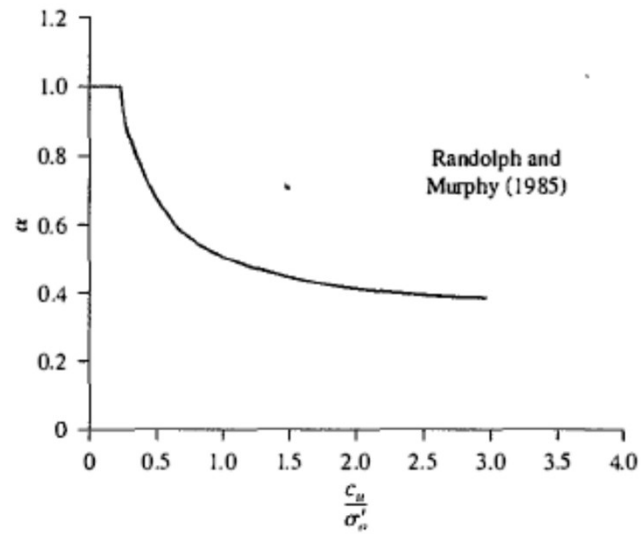
Embedment Length, L (m)	λ
0	0.5
5	0.336
10	0.245
15	0.200
20	0.173
25	0.150
30	0.136
35	0.132
40	0.127
50	0.118
60	0.113
70	0.110
80	0.110
90	0.110

Frictional (Skin) Resistance in Clay

- α **Method (total stresses)**

$$f = \alpha c_u$$

$$Q_s = \Sigma f p \Delta L = \Sigma \alpha c_u p \Delta L$$



- β **Method (effective stress)**

$$f = \beta \sigma'_o$$

$$f = (1 - \sin \phi'_R) \tan \phi'_R \sqrt{OCR}\, \sigma'_o$$

σ'_o = vertical effective stress

$\beta = K \tan \phi'_R$

ϕ'_R = drained friction angle of remolded clay

K = earth pressure coefficient

OCR = overconsolidation ratio.

Point Bearing Capacity of Piles Resting on Rock

$$q_p = q_u(N_\phi + 1)$$

where

$N_\phi = \tan^2(45 + \phi'/2)$
q_u = unconfined compression strength of rock
ϕ' = drained angle of friction

$$q_{u(\text{design})} = \frac{q_{u(\text{lab})}}{5}$$

$$Q_{p(\text{all})} = \frac{[q_{u(\text{design})}(N_\phi + 1)]A_p}{\text{FS}}$$

FS shall be ≥3

Table 11.8 Typical Unconfined Compressive Strength of Rocks

Type of rock	q_u MN/m²	q_u lb/in²
Sandstone	70–140	10,000–20,000
Limestone	105–210	15,000–30,000
Shale	35–70	5000–10,000
Granite	140–210	20,000–30,000
Marble	60–70	8500–10,000

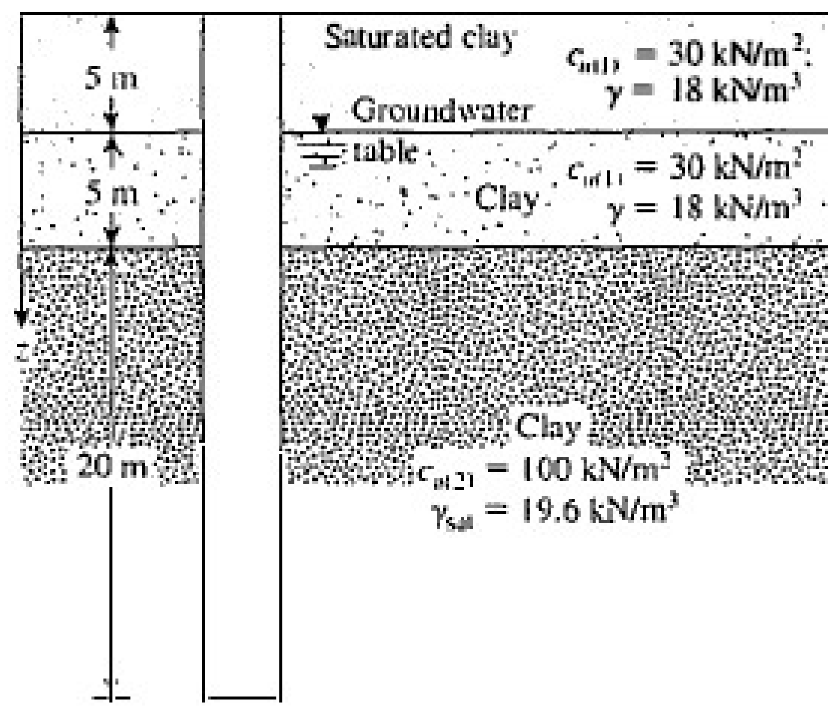
Table 11.9 Typical Values of Angle of Friction ϕ' of Rocks

Type of rock	Angle of friction, ϕ' (deg)
Sandstone	27–45
Limestone	30–40
Shale	10–20
Granite	40–50
Marble	25–30

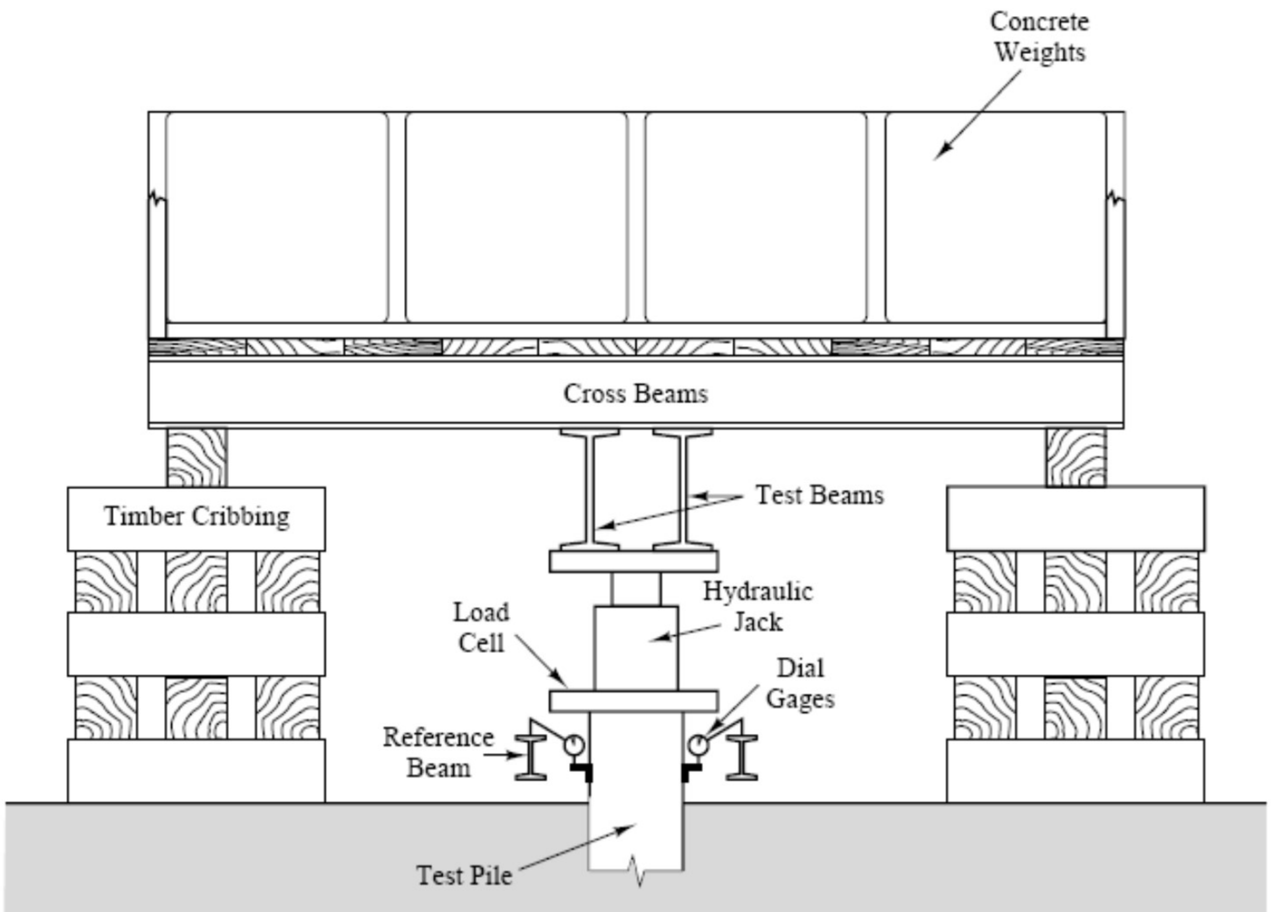
Example 1

1. Estimation of the load bearing capacity of a driven-pipe pile with a diameter of 406mm using α, β and λ
2. Point resistance
3. Allowable net carrying capacity

Have Example 2 on sand Deposits

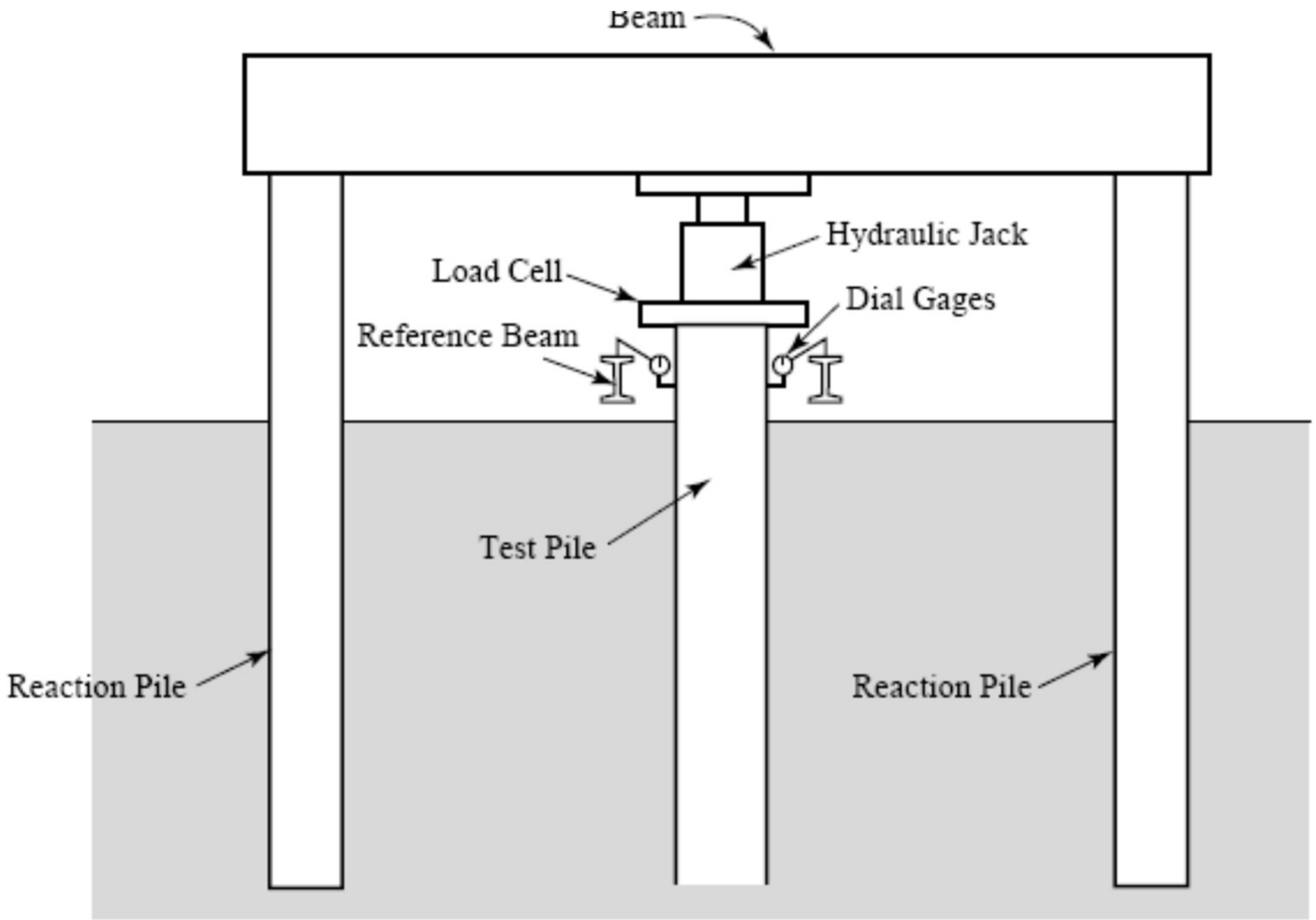


Pile-load Tests

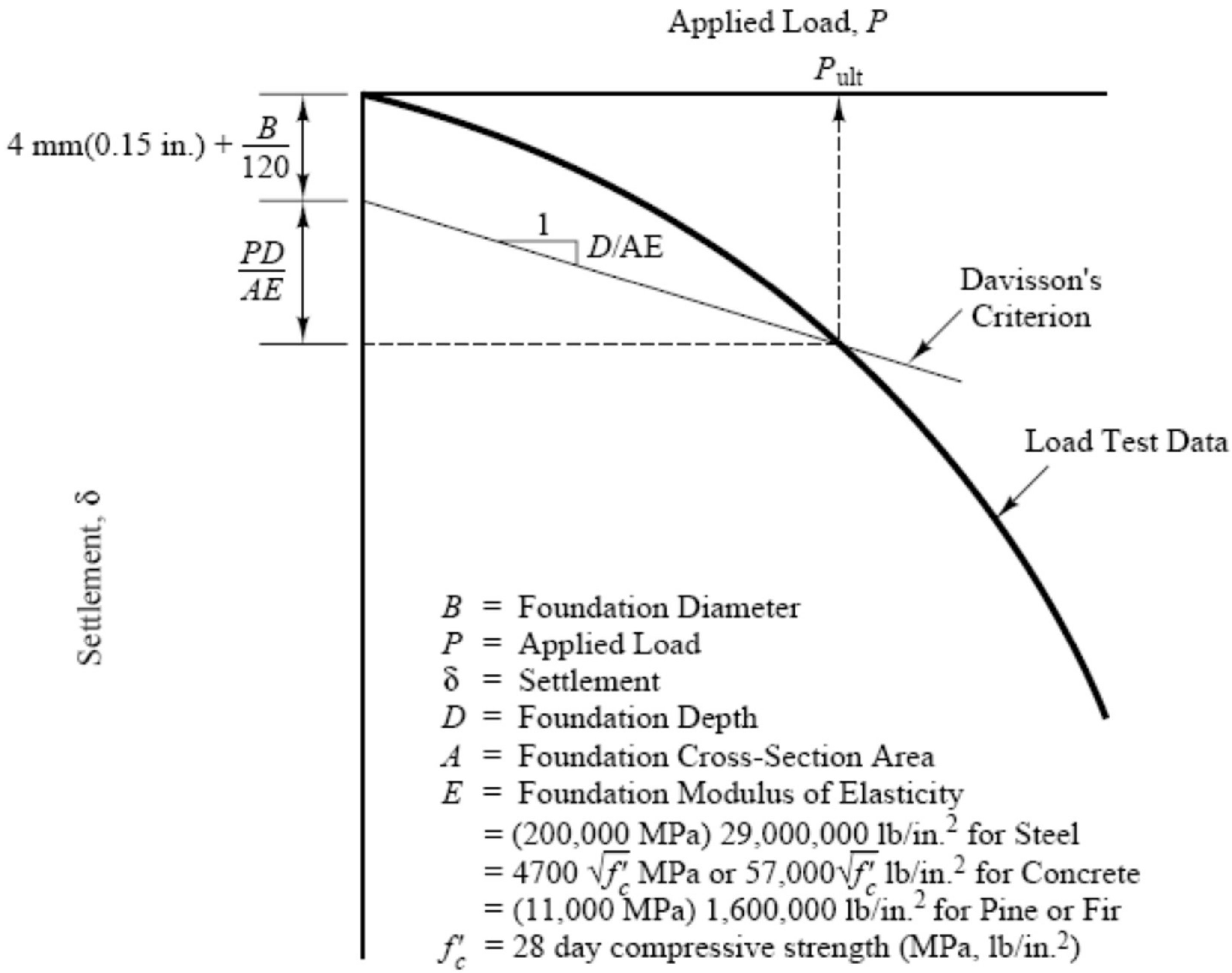


Use of a hydraulic jack reacting against dead weight to develop the test load in a static load test

Use of a hydraulic jack reacting against a beam and reaction piles to develop the test load in a static load test



Estimate pile capacity



Actual Pile Test