Site Investigations 1

1.1 INTRODUCTION

Site investigation is the process by which geological, geotechnical, and other relevant information which might affect the construction or performance of a civil engineering or building project is acquired.

Site investigation will often be carried out by specialists in the field of soil mechanics. Soil, in the engineering sense,

1.2 OBJECTIVES OF THE SITE INVESTIGATIONS

- *Site selection*
- *Foundation and earthworks design*
- *Temporary works design*
- *The effects of the proposed project on its environment*
- *Investigation of existing construction*
- *The design of remedial works*
- *Safety checks*

1.3 PROPOSED SEQUENCES OF THE SITE INVESTIGATIONS

- *Preliminary desk study, or fact-finding survey;*
- *Air photograph interpretation;*
- *Site walk-over survey;*
- *Preliminary subsurface exploration;*
- *Soil classification by description and simple testing;*
- *Detailed subsurface exploration and field testing;*
- *The physical survey (laboratory testing);*
- *Evaluation of data;*
- *Geotechnical design;*
- *Field trials; and*
- *Contact by a geotechnical engineer with site staff during project construction.*

1.4 METHODS OF EXPLORATION

- *There are several methods for exploration*
- *Each method depends on the earth material properties and surrounding circumstances.*
- *Reliability of each method must be elaborated prior to making a final decision on exploration method.*
- *Cost!!!!*

However, Table 1 shows several methods for sample recovery and testing methods.

Table 1: The several exploration methods for sample recovery*

Method	Depths	Applicability
Disturbed samples taken		
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
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

* Marine sampling methods not shown.
† Most common method currently used.

1.5 SOIL BORING

Hand Tools is the simplest method of making exploratory boreholes (Figure 1).

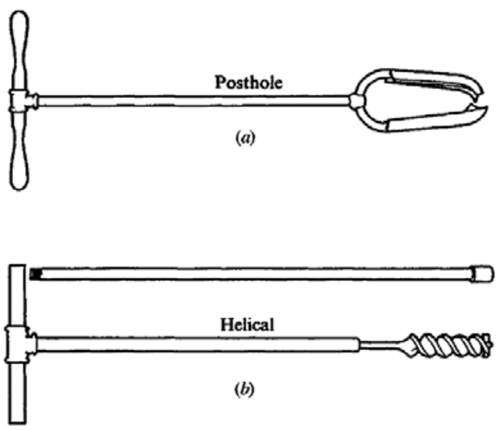


Figure 1: Hand tools for soil exploration

Mounted Power Drills

Rotary wash boring drilling: a casing about (2-3) m long (Figure 2) is driven into the ground. The soil inside the casing is then removed by means of a chopping bit attached to a drilling rod. Water is forced through the drilling rod and exits at a very high velocity through the holes at the bottom of the chopping bit.

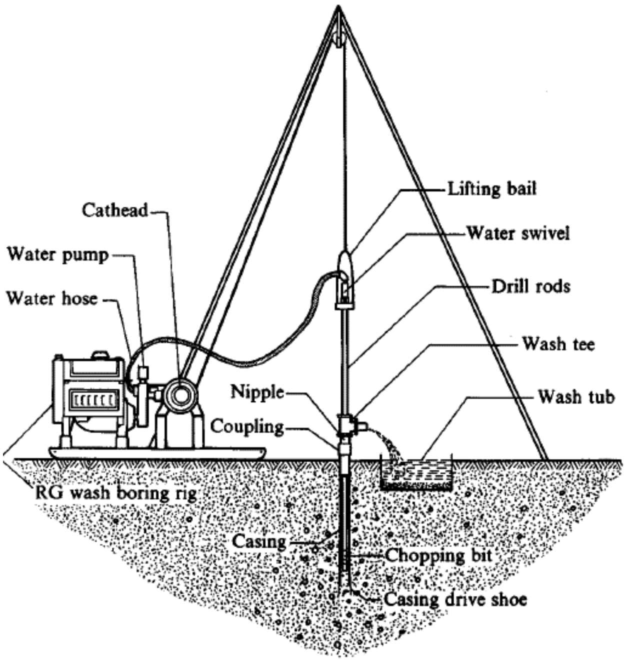


Figure 2: Schematic of wash-boring operations

Continuous-flight augers are probably the most common method used for advancing a borehole as shown in Figure 3. The power for drilling is delivered by truck or tractor mounted drilling rigs. Boreholes up to about **(60-70) m** can be easily made by this method.

Figure 3: Drilling with continuous flight augers

1.6 SOIL SAMPLING

Reasonably good estimates of these properties for cohesive soils can be made by laboratory tests on undisturbed samples. There are some factors that make obtaining of undisturbed samples to be hard. Some of the theses factors are:

1. *The sample is always unloaded from the in situ confining pressures, with some unknown resulting expansion.*
2. *Samples collected from other than test pits are disturbed by volume displacement of the tube or another 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.*
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 the 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.*

1.6.1 UNDISTURBED SOIL SAMPLES

Generally, the undisturbed soil samples are used for consolidation, permeability and shear strength tests of soils.

1.6.2 DISTURBED SOIL SAMPLES

1. *Texture and visual soil classification.*
2. *Grain-size analysis*
3. *Determination of liquid and plastic limits*
4. *Specific gravity of soil solids*
5. *Organic content determination*
6. *Engineering classification of soil*
7. *Chemical tests*

1.7 NUMBER AND DEPTH OF BORINGS

1. There are no clear-cut criteria for determining directly the number and depth of borings (or probing) required on a project in advance of some subsurface exploration.

2. There are no binding rules on either the number or the depth of exploratory soil borings. Each site must be carefully considered with engineering judgment in combination with site discovery to finalize the program and to provide an adequate margin of safety.

General Guides

- *For buildings a minimum of three borings, where the surface is level and the first two borings indicate regular stratification, may be adequate.*
- *Five borings are generally preferable (at building corners and center), especially if the site is not level.*
- *On the other hand, a single boring may be sufficient for an antenna or industrial process tower base in a fixed location with the hole made at the point.*

1.7.1 NUMBER AND SPACING OF BORINGS

There are no hard and fast rules for borehole spacing. Table 2 below gives some general guidelines. Spacing can he increased or decreased, depending on the subsoil condition. If various soil strata are more or less uniform and predictable, fewer boreholes are needed than in nonhomogeneous soil strata.

The Iraqi specification for sampling procedures published in 2015, specifies the number of boring according to the type of the project as shown in Table 3.

Table 2: Approximate Spacing of Boreholes

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

Table 3: Number of Borings (2015-دليل المهندس المقيم للمشاريع الانشائية-الباب السابع)

ت	نوع المنشأ	عدد الجسات المطلوبة
1	المنشات المنعزلة	تحتاج الى نقطتين على الاقل في الأركان المتقابلة في حالة كون مساحة المنشأ اقل من 250م2
2	المخازن الخفيفة والجملونات	تحتاج الى أربع نقاط موزعة على المحيط وواحدة في المركز
3	منشات عامة صغيرة ، مدارس وجوامع	أربع نقاط موزعة بحيث تغطي مساحة المنشأ وواحدة إضافية في مكان تجمع المياه القذرة
4	منشات عالية منفصلة (خزان ماء عالي ، أبراج)	2-4 نقاط معتمدة على عدد وحجم الأسس على ان لاتقل المسافة بين النقاط على 15م
5	الطرق ومسارات الأنابيب	حفرة اختبارية كل 500 م بالإضافة الى إجراء تحريات جيوفيزيائية
6	منشات ذات أسس حصيرية منعزلة وبمساحة اكبر من 250 م2	ثلاث حفر حول المحيط وواحدة في المركز
7	منشات بحرية كبيرة	المسافة بين حفرة وأخرى لا تزيد عن 30م مع إضافة حفر أخرى في المناطق الحساسة
8	الجسور على الأنهر والفضاءات فوق الطرق	حفرة في موقع كل دعامة أو عمود

1.7.2 DEPTH OF BORINGS

The approximate required minimum depth of the borings should be predetermined. The depth can be changed during the drilling operation, depending on the subsoil encountered. The following approaches can be adopted to determine the depth of boring.

1. To determine the approximate minimum depth of boring, engineers may use the rules established by the ASCE (1972):

   a. *Determine the net increase of effective stress, $\Delta\sigma'$, under a foundation with a depth as shown in Figure below.*
   b. *Estimate the variation of the vertical effective overburden pressure ($\sigma o'$) with depth.*
   c. *Determine the depth, D= D1, at which the stress increase $\Delta\sigma'$ is equal to q/10 (q = estimated net stress on the foundation).*
   d. *Determine the depth, D =D2, at which $\Delta\sigma'/\sigma o'$ =0.05.*
   e. *Choose the smaller of the two depths, unless bedrock is encountered.*

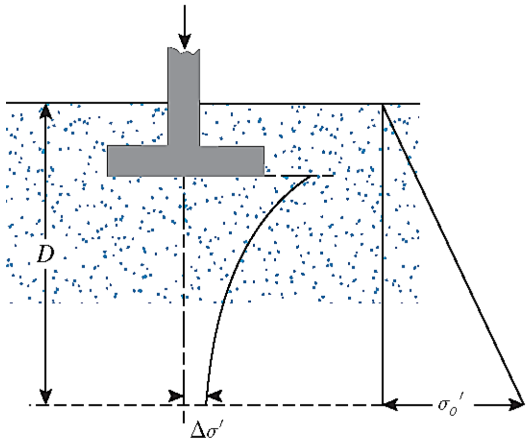


Figure 4: Determination of the minimum depth of boring

2. For Hospitals and office buildings, the minimum boring depth can be estimated according to the following equation:

a) For light steel or narrow concrete buildings:

$$D_b = 3\,S^{0.7}$$

b) For heavy steel or wide concrete buildings:

$$D_b = 6\,S^{0.7}$$

where
D_b = depth of boring, in meters.
S = number of stories.

3. As a general guide, the depth of boring can be taken as 1.5 -2.0 B.

4. The Iraqi specification for sampling procedures published in 2015, specifies the depth of boring according to the type of the project as shown in Table 4.

Table 4: Depth of Borings (دليل المهندس المقيم للمشاريع الانشائية-الباب السابع-2015)

ت	نوع المنشأ	عدد الجسات المطلوبة
1	المنشات الصغيرة والمتعددة الطوابق	يستمر الحفر الى عمق يساوي ضعف عرض الأساس المتوقع مع أجراء حفرة عميقة 20م للتعرف على طبقات التربة
2	المواقع المتكونة من تربة دفن	يتم اختراق تربة الدفن ويستمر الحفر في التربة لحين الوصول الى تربة قوية ملائمة
3	الطرق	يستمر الحفر الى 1-1.5 متر تحت مستوى التبليط في مناطق القطع و 1.5-2 متر لمناطق الدفن
4	المطارات	يستمر الحفر الى 3م تحت مستوى التبليط عند الحفر و 3م تحت الأرض الطبيعية من الدفن الضحل
5	الخزانات الأرضية	يستمر التحري الى عمق الطبقة غير النفاذة أو الى عمق لايقل عن (2×أقصى ارتفاع هيدروليكي متوقع)
6	السدود	عمق التحريات للمنشات الترابية بصورة عامة نصف عرض قاعدة المنشأ ويجب ان تكون نقاط الحفر ليس فقط للتربة الطرية أو غير المستقرة ولكن للطبقات الصخرية النفاذة أيضا والى العمق الذي يمكننا من معالجة التسرب . أو 1-1.5 ارتفاع السد الخرساني في الترب المتجانسة ويستمر الى 3-6م في الطبقة الصلبة الغير نفاذة .
7	الجدران الساندة	يستمر التحري الى عمق (0.75 – 1.5)× ارتفاع الجدار تحت قاعدة الجدار وزيادة في الأمان يستمر الى (2×ارتفاع الجدار)
8	التعليات الترابية	التحري يستمر الى عمق 1.25 × نصف عرض قاعدة التعلية في التحري العادي والى عمق يعادل 2-3× نصف عرض قاعدة التعلية عند التحري عن الهبوط الحاصل فيها
9	استقرارية المنحدرات	يمتد الحفر الى طبقة لا تتأثر بسطح الانزلاق أو الى طبقة قوية جدا
10	القطع العميق	يستمر الحفر الى (0.75-1)× عرض المقطع عندما يكون القطع تحت المياه الجوفية يجب ان يستمر الحفر الى الطبقة غير النفاذة تحت القطع

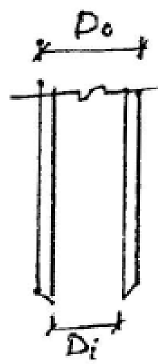
1.8 SAMPLING AND SAMPLES

1.8.1 TYPES

- **Disturbed Samples**
    1. -Taken during boring in plastic bags.
    2. -Used mainly for classification purposes.
- **Undisturbed Samples**
    1. - Special types of tubes are used such that the structure of the grains is approximately the same as that in the site.
    2. Very difficult to obtain for various reasons which will be discussed in the article (C).
    3. Used to determine the mechanical properties of soil (i) Shear strength (c and ϕ) (ii) Consolidation Characteristics (Cc, Cv, …) (iii) Permeability (k) (iv) Stress-Strain relationship (Young's modulus and Poisson's ratio)
- **Remolded Samples**
    1. Disturbed Samples compacted in special molds
    2. For research purposes

1.8.2 REQUIREMENTS FOR OBTAINING UNDISTURBED SAMPLES

1- **Diameter**: D>75 mm
2- **Length**: L≥ intended length + 100mm
3- **Area Ratio** $A_r = \frac{D_o^2 - D_i^2}{D_i^2} \times 100$



Where:
D_o = outside diameter of the tube
D_i = inside diameter of the tube

Ar < 12% for 50 mm sample
Ar < 15% for 75 mm sample
Ar < 20% for 100 mm sample

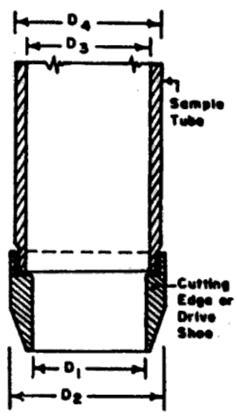
Well-designed sample tubes should have an area ratio of less than about 10 percent. The widely used 51-mm thin walled tube has an A_r of about 13.

4- Recovery Ratio:

L_r = (actual length of recovered sample)/ (theoretical length of sample i.e. length of tube)

Lr = 1 → very good sample

5- Inside Clearance Ratio



$$C_r = \frac{(D_3 - D_1)}{D_1} \times 100$$

$$C_r > 0.5\ for\ Sand\ and\ C_r < 3.0\ for\ Clay$$

Where:

D_3: inside diameter of the sample tube, D_1: Diameter at the sample cutting tube

1.8.3 CAUSES OF DISTURBANCE

1. **Due to advancing the borehole (during boring)**
   a- Sample friction on the sides due to auger rotation.
   b- Samples below water table may drain during covering process.
   c- Volume displacement of the tube.
2. **Due to changes in prevailing condition**
   a- Loss of hydrostatic pressure may cause gas bubble voids to form in the sample.
   b- Changes in water and effective stresses during drilling.
   c- Samples are always unloaded of the in-situ confining pressure with some unknown resulting expansion.
   d- Working environment (temperature).
   e- Handling and transporting a sample from the site to the lab and transferring the sample from sampler to the testing machine.

1.8.4 DEGREE OF DISTURBANCE

- *The way by which the sampler is driven into the soil (pushing or driving), [Pushing is better].*
- *Rate of penetration: high rate causes developing excess pore water pressure.*
- *Dimensions of the sampler.*

1.8.5 SOIL SAMPLERS

Please see Bowles (1997) Chapter 3 for more details

- *Open Drive Sampler*
- *Thin 'Walled Samplers*
- *Split Barrel Samplers*
- *Stationary Piston Sampler*
- *Continuous Sampler*
- *Compressed Air Sampler*
- *Liners*
- *Sand Bailer*

1.9 IN-SITU TESTING (FIELD TESTS)

1.9.1 STANDARD PENETRATION TEST (SPT)

1.9.1.1 DESCRIPTION OF THE (SPT)

- The standard penetration test, developed around 1927, is currently the most popular and economical means to obtain subsurface information (both on land and offshore). It is estimated that 85 to 90 percent of conventional foundation design in North and South America is made using the SPT. This test is also widely used in Iraq and other geographic regions.

- The method has been standardized as ASTM D 1586 since 1958 with periodic revisions to date.

- Good for cohesionless soils and give a rough result for cohesive soils. It is used as an estimate of the shear strength of soils. For cohesive soil, Table 5 can be used as an empirical relationship between the corrected SPT values and the corresponding unconfined compressive strength of cohesive soil.

  For cohesionless soil, Table 6 can be used as an empirical relationship angle of internal friction, relative density and unit weight of cohesionless soil.

Table 5: Approximate correlation between N_{60} and q_u for cohesive soil (Das, 2011)

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

Table 6: Approximate correlation between SPT values and ϕ, Dr and γ for cohesionless soil (Bowles, 2011)

Description	Very loose	Loose	Medium	Dense	Very dense
Relative density D_r	0	0.15	0.35	0.65	0.85
SPT N'_{70}: 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

1.9.1.2 DESCRIPTION OF THE SPT PROCEDURES

The test consists of the following procedures:

1. *Driving the standard split-barrel sampler to a distance of 460 mm into the soil at the bottom of the boring.*
2. *Counting the number of blows to drive the sampler the last two 150 mm distances (total = 300 mm) to obtain the N number.*
3. *Using a 63.5-kg driving mass (or hammer) falling free''from a height of 760 mm. The different types of hammer configurations id given in Figure 5.*
4. *The blows for the first 150 mm is not used.*

The boring log shows *refusal* and the test is halted if:

1. *50 blows are required for any 150-mm increment.*
2. *100 blows are obtained (to drive the required 300 mm).*
3. *10 successive blows produce no advance.*

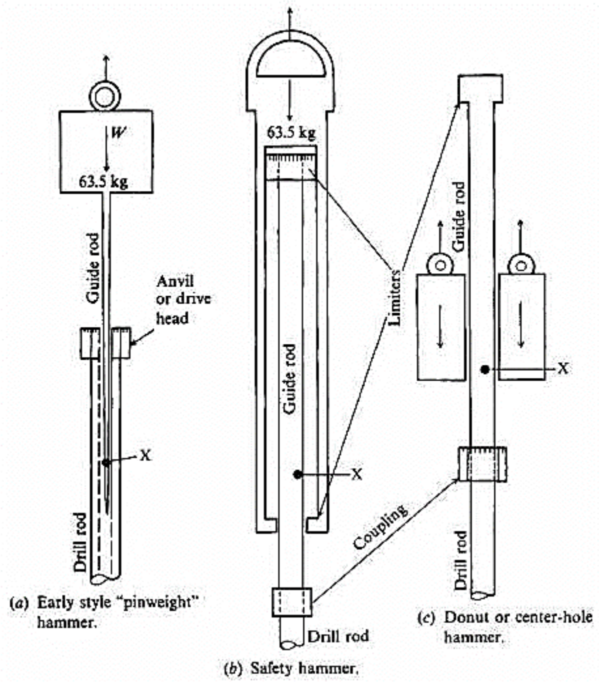


Figure 5: Schematic diagrams of the three commonly used hammers (Bowles. 1997)

1.9.1.3 CORRECTION OF THE SPT DATA

1.9.1.3.1 Correction for Energy

In the field, the magnitude of hammer efficiencies can vary from 30 to 90%. The standard practice now expresses the N-value to an average energy ratio of 60% Thus, correcting for field procedures and on the basis of field observations, it appears reasonable to standardize the field penetration number as a function of the input driving energy and its dissipation around the sampler into the surrounding soil as given below:

$$N_{60} = \frac{N\,\eta_H\,\eta_B\,\eta_S\,\eta_R}{0.60}$$

Where:

N: the measured SPT from the field.

η_H*: the hammer efficiency (%)*

η_B *: the correction for rod length*

η_S *: the sampler correction*

η_R*: the correction for borehole diameter*

N_{60} *Standard penetration number, corrected for field conditions*

The values of different parameters of above equation presented in **Table 7**.

Table 7: Variations of η_H, η_B, η_S and η_R (Das, 2011)

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	η_B
60–120	1
150	1.05
200	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	η_R
>10	1.0
6–10	0.95
4–6	0.85
0–4	0.75

1.9.1.3.2 Uplift Pressure

For soils consisting very fine or silty sand below water table, a correction is made when N> 15 because excess pore water pressure set up during drilling the sampler cannot dissipate. One of the following equations is used:

$$N' = 15 + 0.5 \times (N - 15) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. by\ Terzaghi\ and\ Peck\ (1943)\ [Most\ Popular]$$

$$N' = 0.6 \times N \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. by\ \ Bazaraa\ (1967)$$

1.9.1.3.3 Correction for Overburden Pressure

The SPT (N) values for a depth corresponding to an effective overburden pressure of 110 kPa is considered to be a standard. For effective overburden pressure Po' > 25 kPa, a correction factor (C_N) should be used:

$$C_N = 0.77 \log \frac{2000}{P'_o}$$

Then $N_{corrected} = N_{measured} \times C_N$ or $N_c = N_m \times C_N$

1.9.1.3.4 Correction for Dilatancy

For dilation effect: In saturated fine or silty dense or very dense sands which tends to dilate during shear, N-values may be great. Approximately reduce 5 blows for each 40 blows. [not commonly used unless stated clearly]

1.9.2 DYNAMIC CONE PENETRATION TEST (DCPT)

- Used for hard deposits.
- The cone is driven into the soil by a drop of a standard hammer falling a standard distance as illustrated in Figure 6.
- Record number of blows for each 0.3 meter (Nc)
- Friction on sides increases with depth, hence the diameter of the cone must be greater than the outside diameter of the pipe.
- If depth of investigation is more than 6 m, use bentonite to facilitate penetration.
- Fast and economical (no borehole is required)
- Gives continuous penetration of strata penetrated. It often reveals the presence of strata, which are not recovered or observed in sampling.
- Nc values must be corrected for overburden pressure.

$$Nc' = C1 \times Nc$$

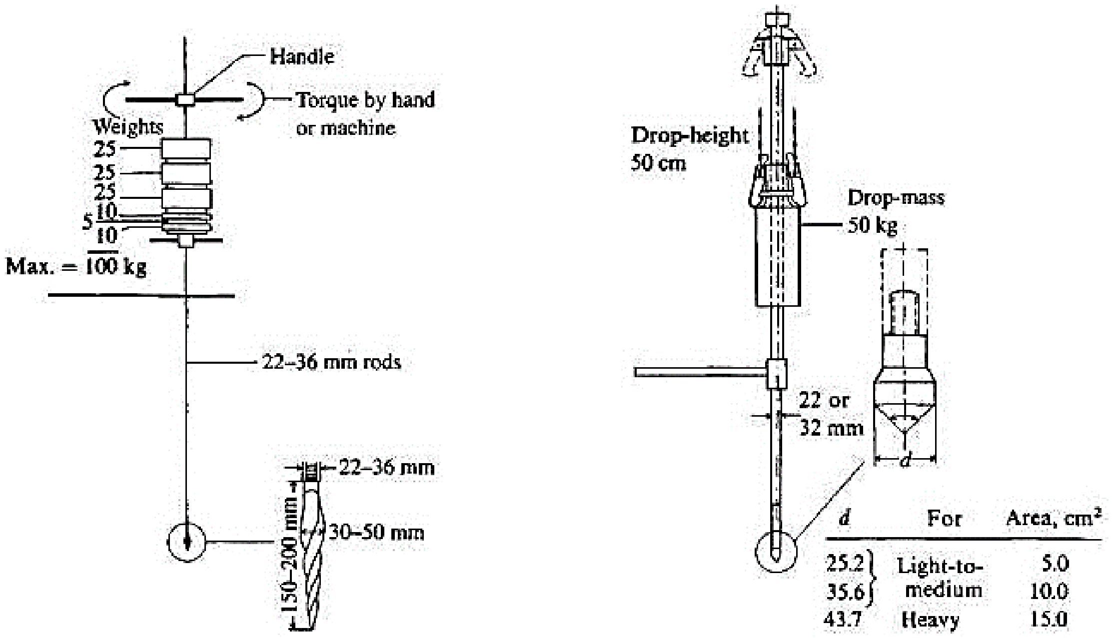
  where C1 equals:

  = 0.8 to 1.2 when bentonite is used:

  = 1.5 up to depth 3m and bentonite is not used

  = 1.75 for depth 3 to 6m and bentonite is not used.

- Limitations:
  - *No samples are taken*
  - *Misleading results in gravel or boulder strata*



(a) (b)

Figure 6: Types of DCPT (a): A type of Swedish weight sounding equipment (b): One type of dynamic cone penetration test

1.9.3 STATIC (DUTCH) CONE PENETRATION TEST (CPT)

- The test is performed according to BS 1377: Part 9:1990 and ASTM D-3441 (mechanical systems) and ASTM D 5778 (electric systems).
- CPT's can be performed to depths exceeding 100m in soft soils and with large capacity pushing equipment.
- Preferable for soft cohesive deposits (fine sands, silty fine sands and clay).
- Pushing hydraulically a steel cone (diameter= 35.7 mm and apex angle 60) at a rate of 10 to 20 mm ***/*** sec and recording the required force and hence the stress (q_c) can be calculated using the following equation (see Figure 7):

  q_c = (force required)/ (base area≈ 1000 mm^2).
- The outer rod is pushed and the force required for pushing the cone and sleeve is recorded and the stress (q_t) can be calculated
- Friction stress =$q_f = q_t - q_c$
- The data obtained are used for bearing capacity and settlement analysis and static pile capacity.
- The data obtained are used for bearing capacity and settlement analysis and static pile capacity.
- As the SPT test, the CPT test data can be correlated to the shear strength parameters for cohesion and cohesionless soils. Moreover, there is good correlation between SPT and CPT data for soils.

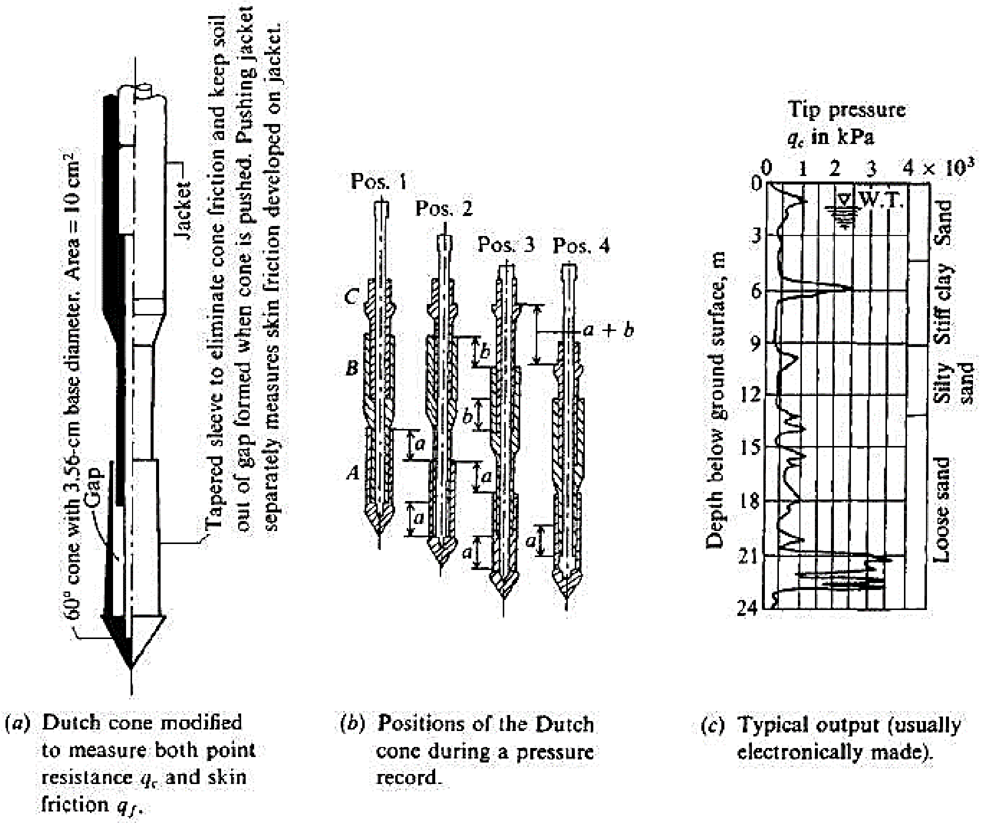
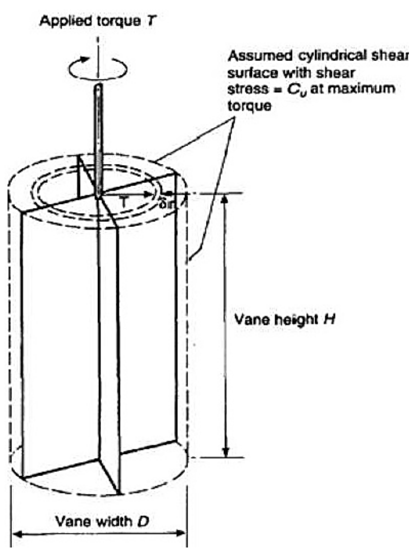


Figure 7: Mechanical (or Dutch) cone, operations sequence, and tip resistance data (Bowles, 1997)

1.9.4 FIELD VANE TEST (VST)

- Used for determination of undrained cohesion of soft to medium stiff clay
- Valuable in sensitive clays where it is difficult to obtain truly undisturbed samples [Sensitivity= (strength in undisturbed state)/ (strength in remolded state)].
- The vane shear apparatus consists of four blades on the end of a rod with H/D ≈2.
- Inserting a four bladed vane in the undisturbed or remolded soil at required depth and rotating it from the surface through link rods to determine the torsional force required (T) to cause a cylindrical surface to be sheared by the vane:

$$c_u = \frac{T}{\pi\, dD^2\left(\frac{H}{2}+\frac{D}{6}\right)}$$



where

T = Torque (F.L)

c_u = Undrained shear strength = cohesion for clayey soils (F/L^2)

H = Height of blade (L)

D = Diameter of blade (L)

For actual design purposes, the undrained shear strength values obtained from field vane shear tests are too high, and it is recommended that they be corrected according to the equation:

$$c_{u(corrected)} = \lambda\, c_{u(VST)}$$

Where λ is a correction factor of the VST. Several correlations are available for the correction factor (λ), among them is the following equation:

$$\lambda = 1.7 - 0.54 \log[PI\,\%]$$

Or it can be obtained from charts as shown in Figure 8 which was performed by Bjerrum 1977.

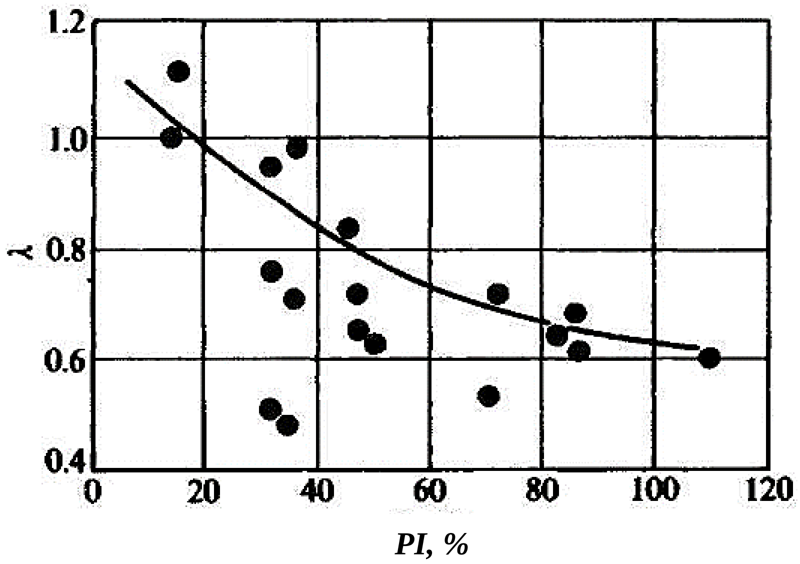


Figure 8: Correction factor for vane shear test [λ vs PI] (Bowles, 1997)

1.9.5 PLATE LOAD TEST

- The tests were performed according to (ASTM D1194).
- The ultimate load-bearing capacity of a foundation, as well as the allowable bearing capacity based on tolerable settlement considerations, can be determined from the field load test such as plate loading test.
- Used to measure the load necessary to induce a given amount of settlement of a model footing.
- Suitable for sandy soils.
- Usually of short duration, hence consolidation settlement does not fully occur during this test.
- Zone of stressed soil beneath the plate is much smaller than that beneath the larger foundation so will be unaffected by deeper strata whose load bearing and settlement characteristic may affect the behavior of the foundation.
- A circular (diameter= 0.3m or 0.45 m) plate is seated on the stratum to be tested, usually at the bottom of a trial pit and loaded (as shown in Figure 9). The load is maintained until full consolidation settlement has taken place. The test is continued with further increments of load. The settlement is plotted against the load. Occasionally, square plates that are 305 mm in length are also used.

- Bearing capacity prediction can be performed from:

$$q_{u(F)} = q_{u(P)} \frac{B_F}{B_P}$$ **for sandy soil**

$$q_{u(F)} = q_{u(P)}$$ **for Clayey soil**

Where:
$q_{u(F)}$: *the ultimate bearing capacity of footing*
$q_{u(P)}$: *the ultimate bearing capacity of plate*
B_F: *width of foundation*
B_P: *width of plate*

- The allowable bearing capacity of a foundation, based on settlement considerations and for a given intensity of load, q_o, is:

$$S_F = S_P \frac{B_F}{B_P}$$ **for clayey soil**

$$S_F = S_P \left(\frac{2\, B_F}{B_F + B_P}\right)^2$$ **for sandy soil**

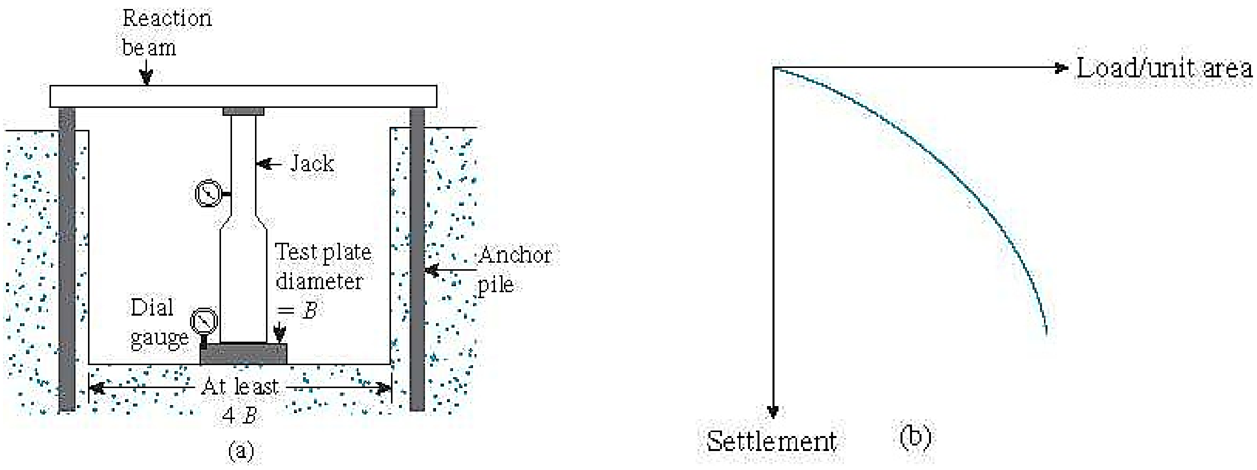


Figure 9: Plate load test: (a) test arrangement; (b) nature of load– settlement curve (Das, 2011)

1.9.6 WATER TABLE LOCATION

- Ground water affects many important phases of foundation design (Bearing capacity and settlement). The GWT is generally determined by directly measuring the stabilized water level in the borehole after a suitable time lapse, often 24 to 48 hr later.

- 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.

- In soils with low permeability such as silts, fine silty sands, and clays, it may take several days to several weeks (or longer) for the GWT to stabilize. In this case, an alternative is to install a **piezometer** (small vertical pipe) with a porous base and a removable top cap in the borehole. Backfill is then carefully placed around the piezometer so that surface water cannot enter the boring.

1.10 ILLUSTRATIVE EXAMPLES

1.10.1 EXAMPLE NO. 1

A three-story building is to be constructed on the campus of Baghdad University. The preliminary data of the soil profile at the site shows that the soil profile may consist mainly of medium to dense silty sand to sandy silt with an average total unit weight of 20 kN/m^3 and the water table level is about 1.0 m below ground level. The area of the site is about 200 m^2 (10 m×20 m) and the estimated net applied pressure is about 90.0 kPa. Estimate the number of boreholes and depth of boring.

1.10.2 EXAMPLE NO. 2

A four-story building (20 mx30 m) with a basement (depth= 3 m below ground surface) is proposed. The net pressure (Δq) of the building at the basement level is 75 kPa. The soil is silty clay with a dry and submerged unit weight equal to 16 kN/m^3 and 9 kN/m^3 respectively. The water table was found at elevation 1 m below ground surface. Determine, for a detailed soil investigation the number, layout and depth of the boreholes.

1.10.3 EXAMPLE NO. 3

The dimensions of two samples tubes are as follows:

Sample tube No.1, outside diameter=76.2 mm, inside diameter=73.0 mm

Sample tube No.2, outside diameter=51.0 mm, inside diameter=38.0 mm

what kind of sample disturbance might you expect using either tube size?

Solution

$$Ar_1 = \frac{76.2^2 - 73^2}{73^2} = 8.96\ \%\ \text{undisturbed}$$

$$Ar_2 = \frac{51^2 - 38^2}{38^2} = 80.12\ \%\ \text{disturbed.}$$

1.10.4 EXAMPLE NO. 4

A vane is 100 mm in diameter and has blades 150 mm height, was pushed into undisturbed clay at the bottom of a borehole, the torque required to rotate the vane is 190 N.m. What is the shear strength of clay? [Note that the PI of the clayey soil is 30%].

Sol.

$$C_u = \frac{T}{\pi d^2 \left(\frac{h}{2} + \frac{d}{6}\right)} = \frac{190\,E{-3}}{\pi (.1)^2 \left(\frac{.15}{2} + \frac{.1}{6}\right)} = 66\ kN/m^2$$

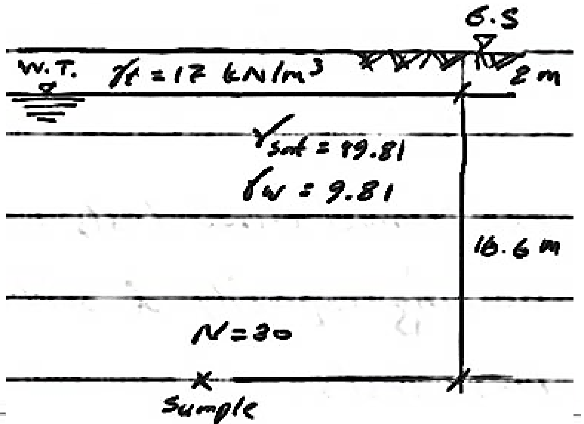
$$\lambda = 1.7 - 0.54 \log[PI\ \%] = 1.7 - 0.54 \times log\ [30] = 0.90$$

$$c_{u(corrected)} = \lambda\, c_{u(VST)} = 0.90 \times 66 = 59.5\ kN/m^2$$

1.10.5 EXAMPLE NO. 5

If the measured SPT number equals to 30 blows at a depth shown in figure, calculate the corrected number of blows if the soil:

- Coarse grain sand
- Fine silt



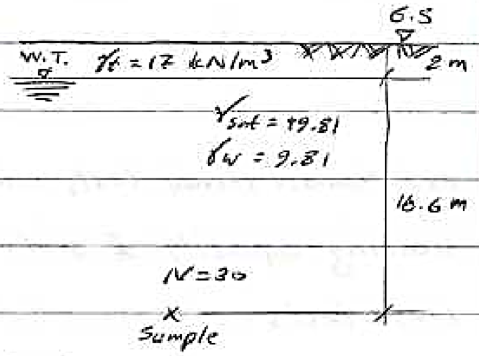
① Coarse Sand :

$C_N = 0.77 \log \frac{2000}{\bar{P}}$

* effective

$\bar{P} = 17 * 2 + 10 * 16.6 = 200 \ kN/m^2$

$\bar{\sigma}^* = \gamma_{sat} - \gamma_w$



$C_N = .77 \log \frac{2000}{200} = .77 \rightarrow N' = C_N \cdot N$

$= .77 * 30 = 23$ Ans.

② Fine Silt :

$N_c' = 15 + .5(N' - 15) = 15 + .5(23 - 15) = 19$ Ans.

Lateral Earth Pressure 2

2.1 INTRODUCTION

Vertical or near-vertical slopes of soil are supported by retaining walls, cantilever sheet-pile walls, sheet-pile bulkheads, braced cuts, earth or rock contacting tunnel walls and other underground structures and other, similar structures. The proper design of those structures requires an estimation of lateral earth pressure, which is a function of several factors, such as

(a) The type and amount of wall movement,

(b) The shear strength parameters of the soil,

(c) The adjacent applied loads,

(d) The topography of the backfilling,

(e) The unit weight of the soil, and

(f) The drainage conditions in the backfill.

Figure 10 and **Figure 11** shows the nature of variation of the lateral pressure for different retaining structures.

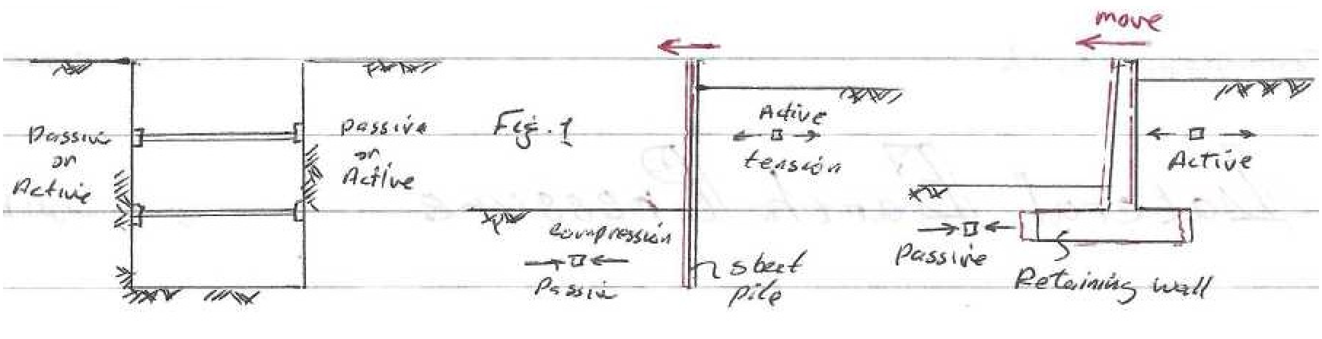


Figure 10: Distribution of the lateral earth pressure for different retaining structures

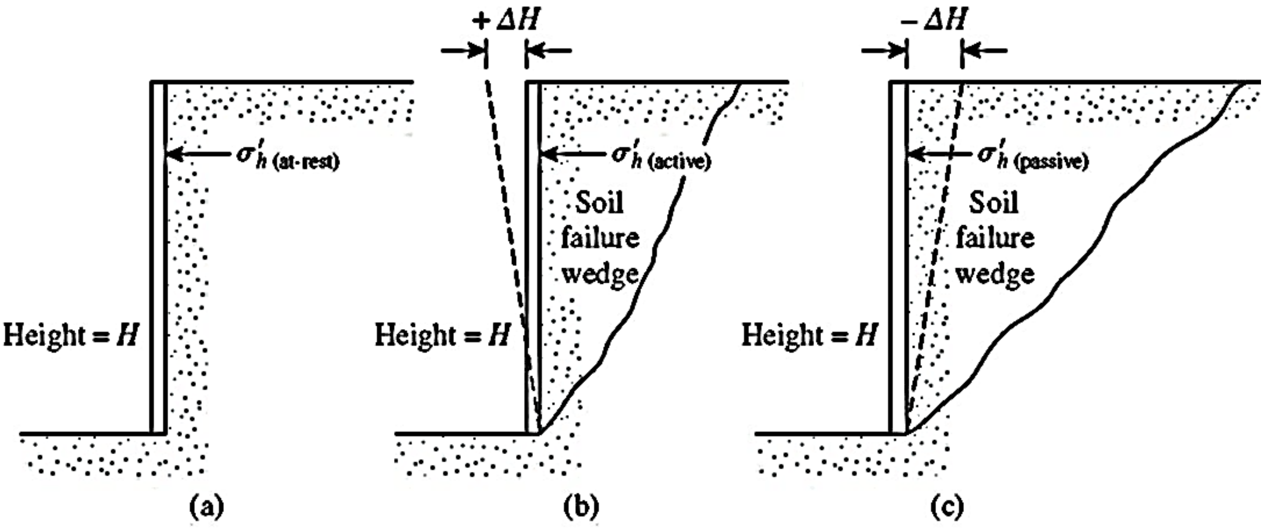


Figure 11: Nature of lateral earth pressure on a retaining wall

Figure 12 and **Figure 13**, illustrates the variation of the lateral force at certain depth. From this figure, it can be seen that $E_a < E_o < E_p$ and $\Delta_p \gg \Delta_a$.

The design of the retaining structure requires the determination of the **magnitude** and **line of action of the lateral earth pressure**.

The magnitude of the lateral earth pressure depends upon a number of factors, such as

1. *Mode of the movement of the wall,*
2. *Flexibility of the wall,*
3. *Properties of the soil.*
4. *Drainage conditions.*

It is a soil-structure interaction problem, as the earth pressure depends upon the flexibility of wall.

The earth pressure theories which consider soil-structure interaction are complicated and require a computer. For convenience, the retaining wall is assumed to be **rigid** and the soil-structure interaction effect is neglected.

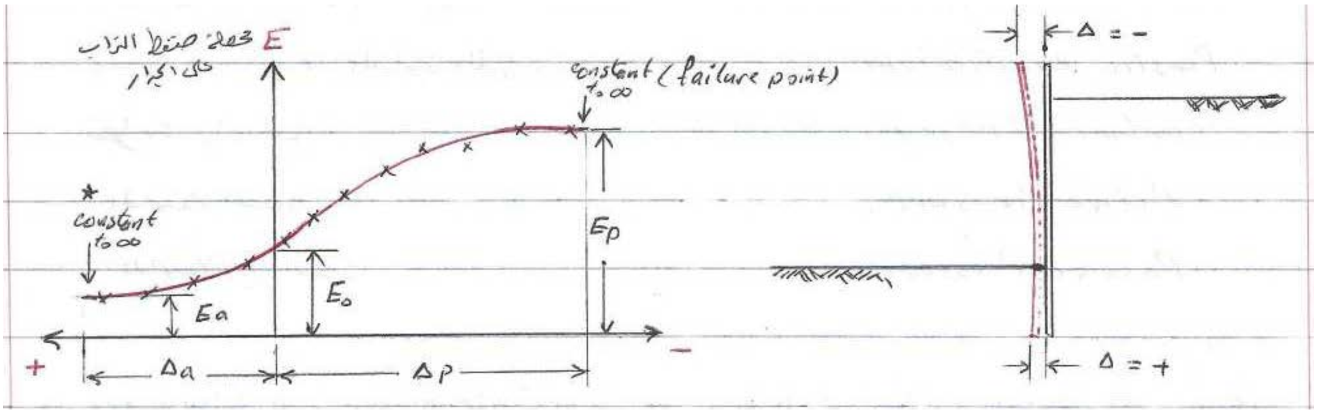


Figure 12: Nature of variation of lateral earth pressure at a certain depth

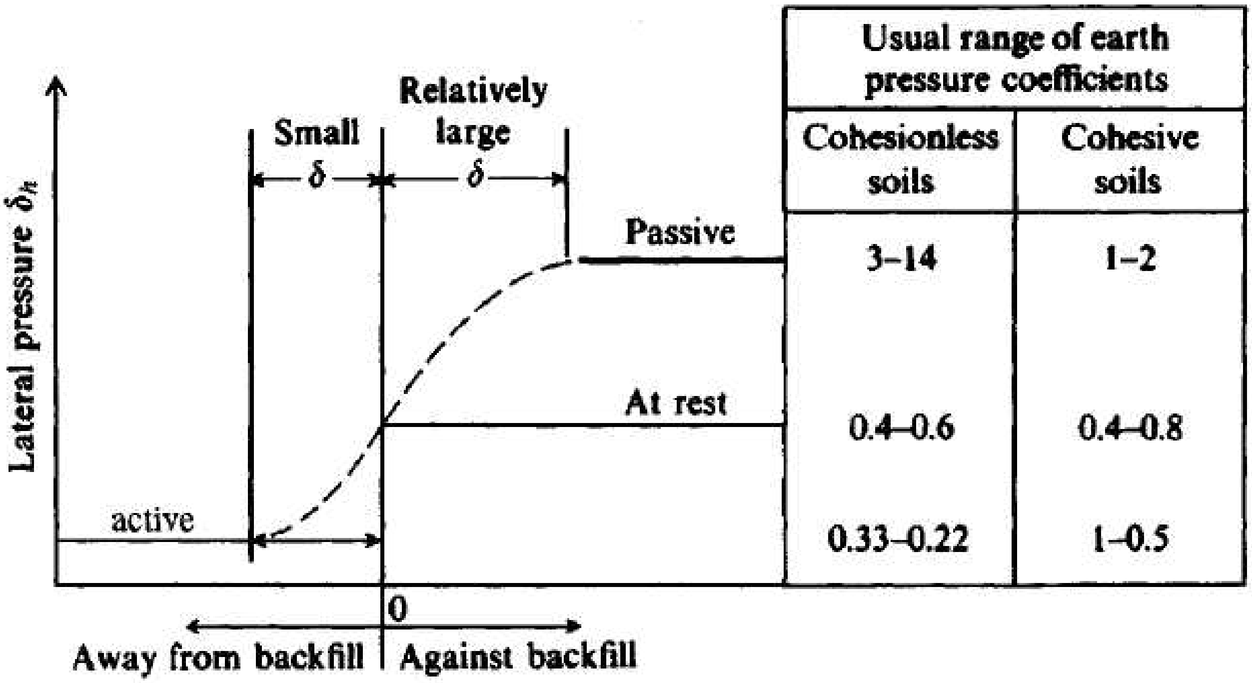


Figure 13: Illustration of active and passive pressures with usual range of values for cohesionless and cohesive soil.

2.2 LATERAL EARTH PRESSURE AT REST (ELASTIC EQUILIBRIUM)

2.2.1 WITHOUT GROUNDWATER

Consider a vertical wall of height H, as shown in **Figure 14**, retaining a soil having a unit weight of A uniformly distributed load, is also applied at the ground surface. The shear strength of the soil is:

$$\boldsymbol{\tau = c' + \sigma_n{}' \tan \emptyset'}$$

where
c' = cohesion
ϕ' = effective angle of friction
σ' = effective normal stress

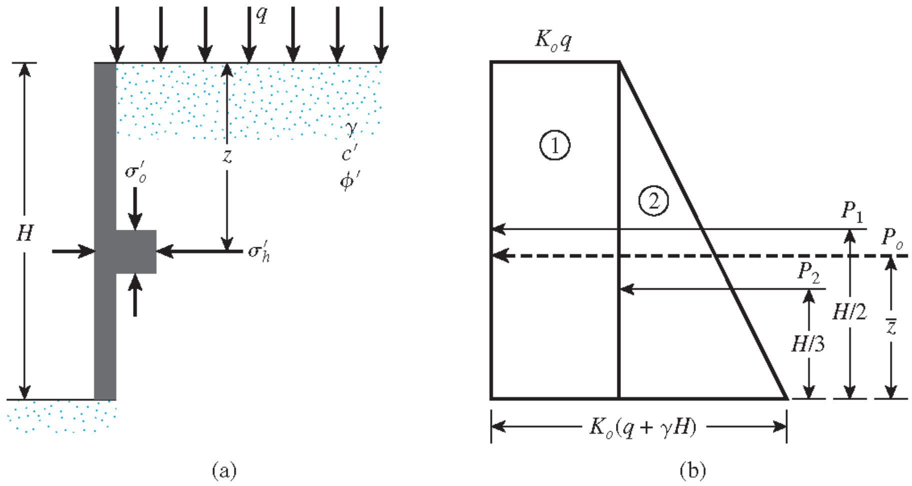


Figure 14: At rest, lateral earth pressure without groundwater

At any depth, z, the vertical effective stress is:

$$\boldsymbol{\sigma'_{vo} = q + \gamma z}$$

And the horizontal effective stress is (k_o is the lateral earth pressure at rest):

$$\boldsymbol{\sigma'_{ho} = k_o\, \sigma'_{vo}}$$

For normally consolidated soil, the $\boldsymbol{k_o}$ relation for (Jaky, 1944)

$$\boldsymbol{k_o} = 1 - \sin \emptyset'$$

For overconsolidated soil, the at-rest earth pressure coefficient may be expressed as (Mayne and Kulhawy, 1982)

$$\boldsymbol{k_o} = (1 - \sin \emptyset')\ OCR^{\sin \emptyset'}$$

The location of the line of action of the resultant force, P_o, can be obtained by taking the moment about the bottom of the wall. Thus,

$$\bar{z} = \frac{P_1\left(\frac{H}{2}\right) + P_2\left(\frac{H}{3}\right)}{P_o}$$

2.2.2 WITH GROUNDWATER

If the water table is located at a depth $z < H$, the at-rest pressure diagram shown is **Figure 15**:

$$\text{at } z = 0, \quad \sigma'_h = K_o\sigma'_o = K_o q$$
$$\text{at } z = H_1, \quad \sigma'_h = K_o\sigma'_o = K_o(q + \gamma H_1)$$

and

$$\text{at } z = H_2, \quad \sigma'_h = K_o\sigma'_o = K_o(q + \gamma H_1 + \gamma' H_2)$$

The value of the total lateral force is

$$P_o = K_o q H_1 + \tfrac{1}{2}K_o\gamma H_1^2 + K_o(q + \gamma H_1)H_2 + \tfrac{1}{2}K_o\gamma' H_2^2 + \tfrac{1}{2}\gamma_w H_2^2$$

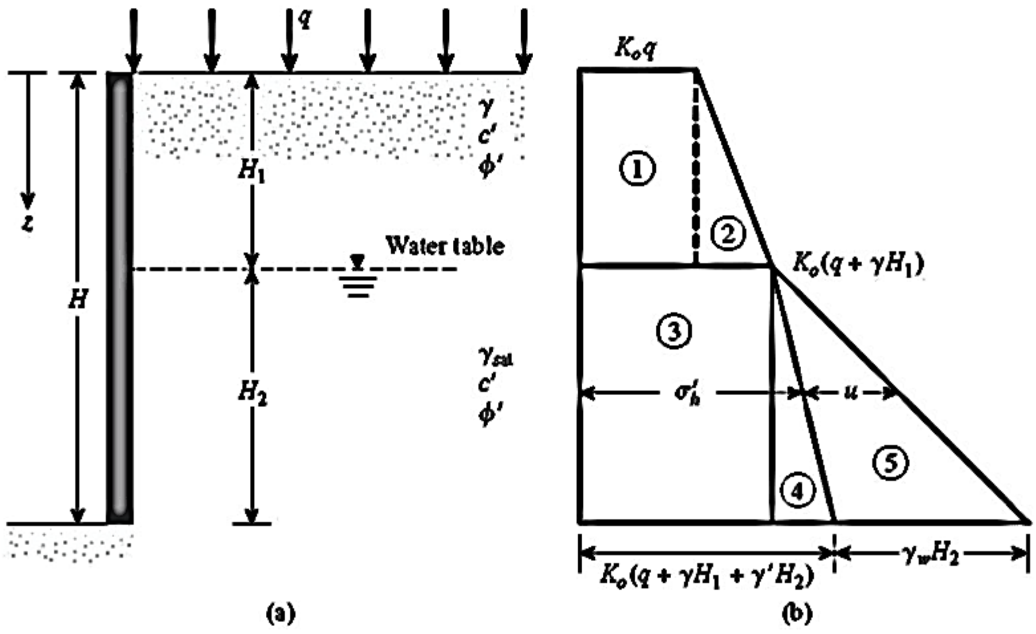


Figure 15: At rest, lateral earth pressure with groundwater

2.3 LATERAL EARTH PRESSURE (PLASTIC EQUILIBRIUM)

2.3.1 RANKINE THEORY (COHESIONLESS SOIL)

A. Active state

A. Passive State

2.3.2 RANKINE THEORY (COHESIVE SOIL)

A. Active state

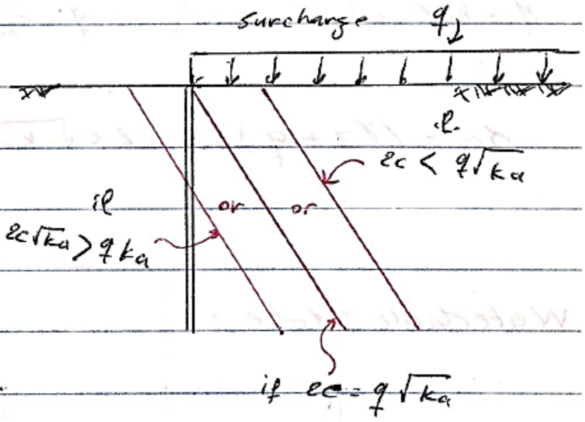
B. Passive State

2.3.3 PRESENCE OF THE SURCHARGE (RANKINE THEORY)

If a surcharge placed at any side of the retaining wall the distribution of lateral stresses will be as shown below:

Surcharge state;

$$\sigma_a = (\gamma z + q) K_a - 2c\sqrt{K_a}$$



2.3.4 PRESENCE OF MULTI-LAYER SOIL PROFILE (RANKINE THEORY)

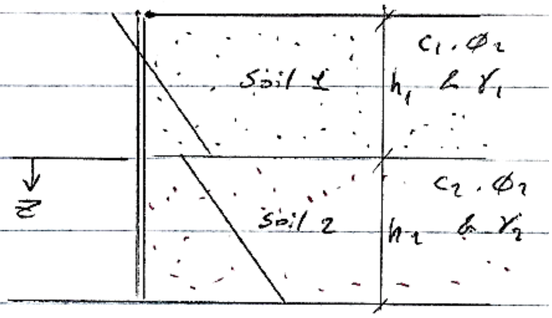
If the soil profile is not homogenous and more than one layer is presence, the distribution of lateral stresses will be as shown below:

Two strata state;

change soil 1 to surcharge;

$q = h_1 \gamma_1$, and start z from the 2nd strata;

$$\sigma_a = (\gamma z + q) K_a - 2c\sqrt{K_a}$$

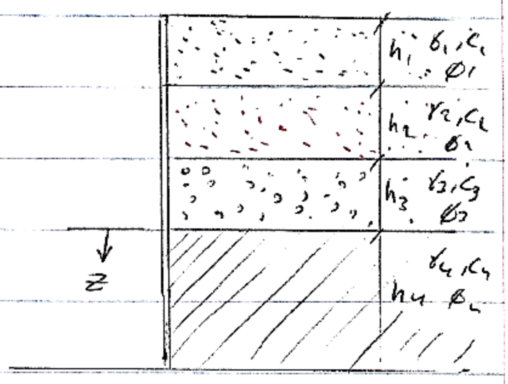


Multi-strata state (stratum);

change soil (1, 2 and 3) to surcharge;

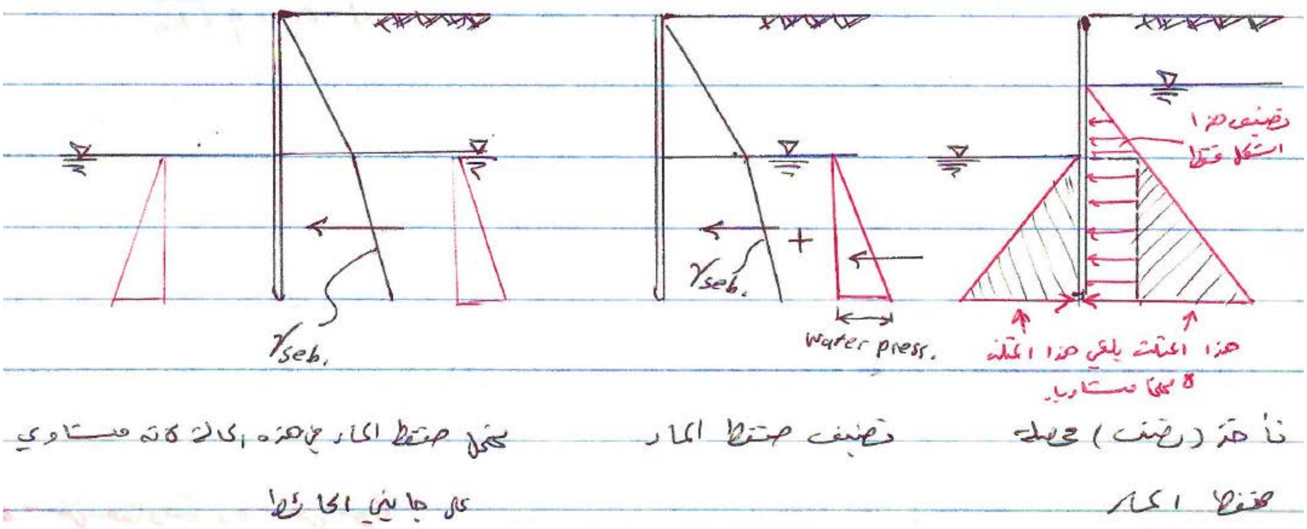
$q = h_1\gamma_1 + h_2\gamma_2 + h_3\gamma_3$, $q = \sum_{n=1}^{k} h_n \gamma_n$

$$\sigma_a = (\gamma z + q) K_a - 2c\sqrt{K_a}$$



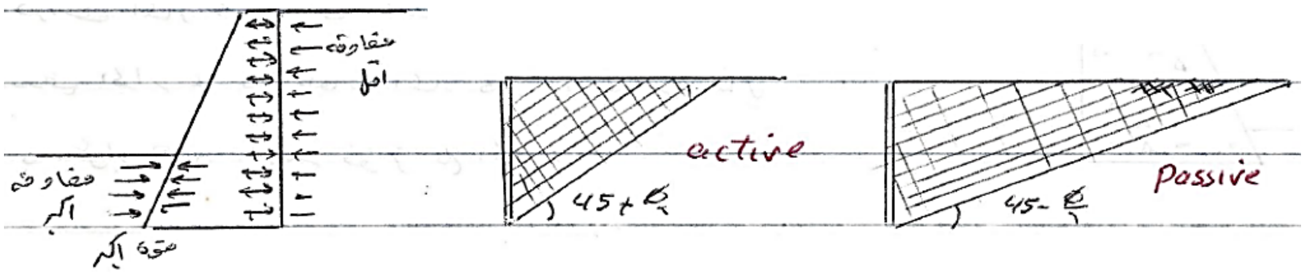
2.3.5 PRESENCE OF WATER TABLE LEVEL (GROUNDWATER)

If the soil profile contains groundwater, the distribution of lateral stresses will be as shown below:



2.3.6 APPLICATION OF THE LATERAL EARTH PRESSURE TO RETAINING WALLS

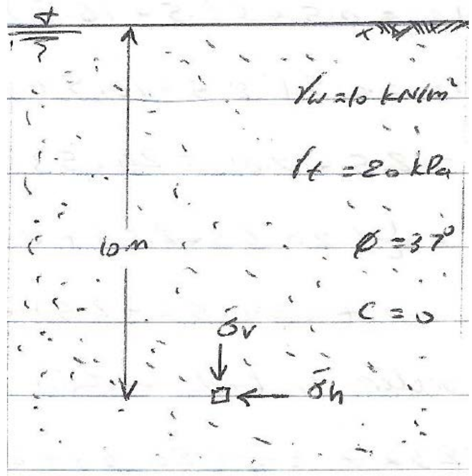
the lateral earth pressure failure surface can be shown in the following figure. As well as, the lateral earth pressure concept can be applied to retaining structures such as retaining walls.



2.4 ILLUSTRATIVE EXAMPLES AND HOMEWORK

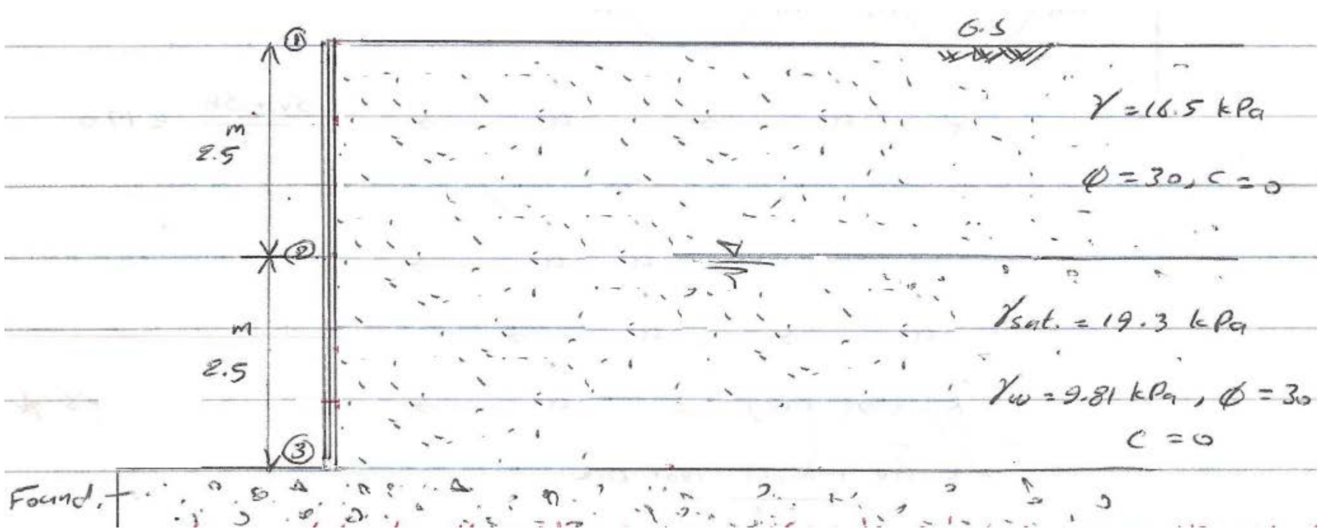
2.4.1 EXAMPLE NO. 1

Find the lateral earth pressure at rest condition for point (a), shown in figure. Also, plot the Mohr's circle for the effective and total stress of this point.



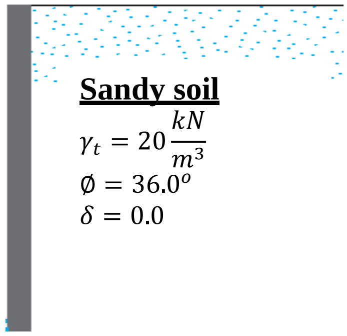
2.4.2 HOMEWORK NO. 1

Find the resultant of the lateral earth pressure on the retaining wall shown in the figure, at rest condition. Also, find the location of the resultant.



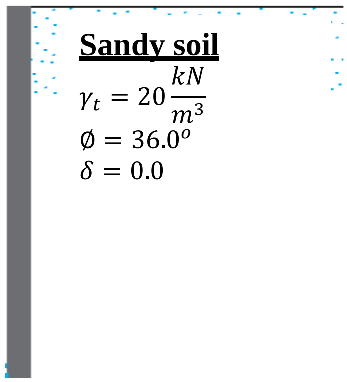
2.4.3 EXAMPLE NO. 2

Determine the lateral earth pressure force on the wall shown in the figure. Draw the stress distribution and locate the location of the resultant force.



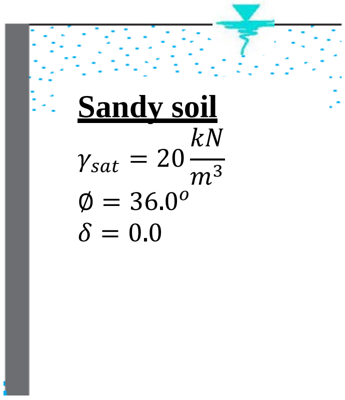
2.5 HOMEWORK NO. 2:

Solve Example No. 2 by assuming the condition to be a passive state.



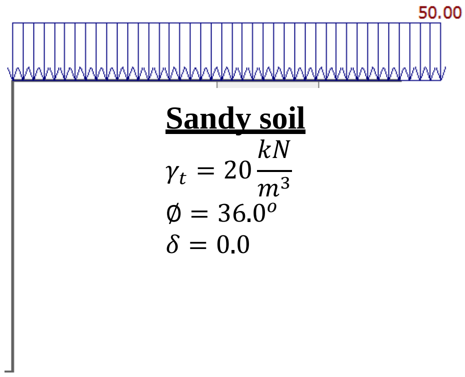
2.6 EXAMPLE NO. 3:

For the following soil profile and retaining wall condition, find the active earth force on the wall and its location.



2.7 EXAMPLE NO. 4:

Resolve Example 2 with a surcharge of 50.0 kPa is placed to the right side of the wall.

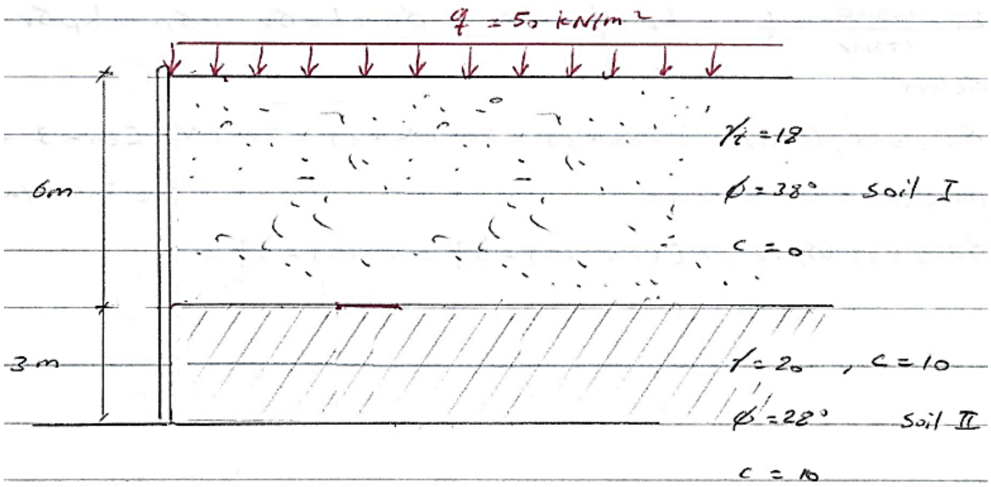


2.8 HOMEWORK NO. 3:

Resolve Example 4 with a groundwater at the surface and at 2.0 m below the ground surface (G. S.).

2.9 EXAMPLE NO. 5:

Plot the active earth pressure distribution on the wall shown in figure below.



2.10 HOMEWORK NO. 4:

Resolve Example 5 if soil I is clayey soil with $c_u = 50.0\ kPa$ moreover, find the resultant of the force and its location.

Retaining Walls 3

3.1 GENERAL

In general, there are different retaining structures that can be found in the design of foundations. Theses retaining structures are:

1. Retaining walls,
2. Flexible retaining structures (Sheet pile walls)
3. Caissons
4. Diaphragm walls

The uses of the retaining walls cover different structures and purposes, the uses of structures are summarized in Figure 16. The present chapter deals with the retaining walls. In general, retaining walls are classified into two major groups that can be explained herein.

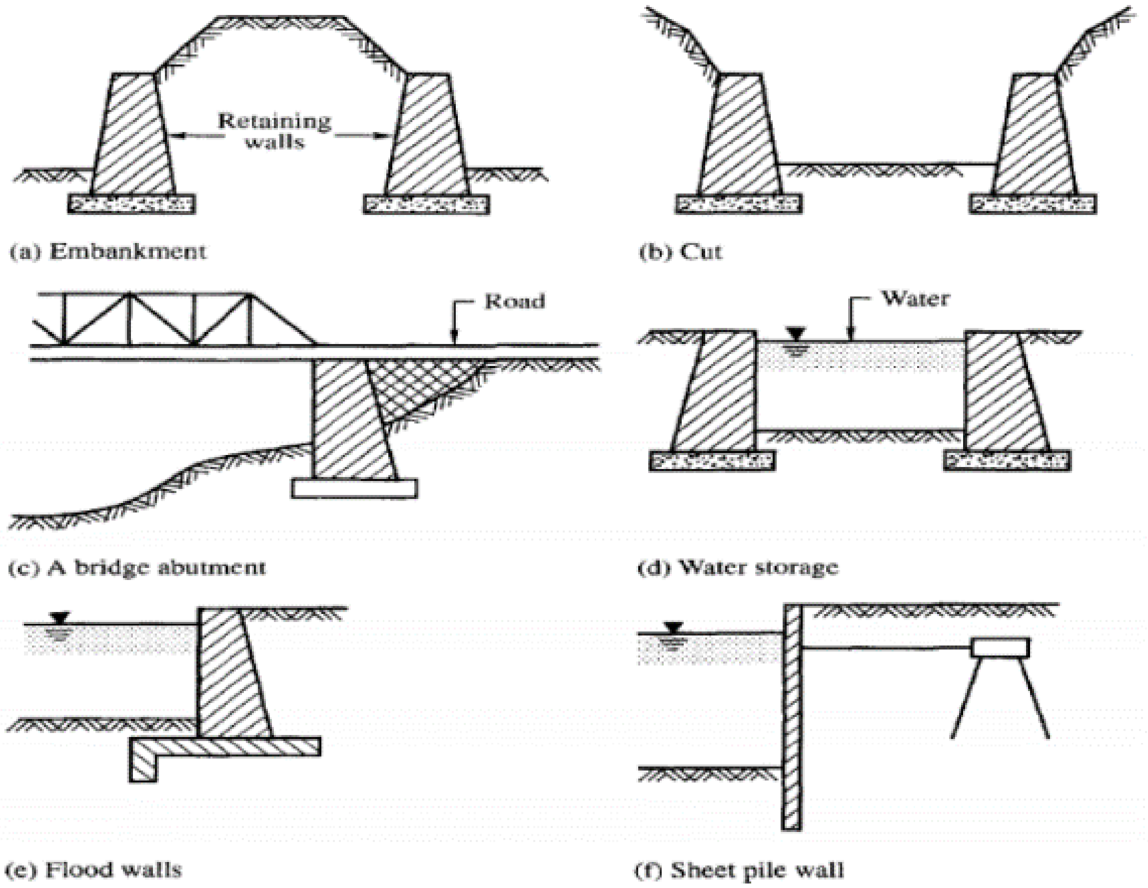


Figure 16: Applications of retaining structures

3.1.1 CONVENTIONAL RETAINING WALL

The main types of this category are shown in Figure 17 and summarized below:

A. Gravity retaining walls

- Constructed with plain concrete or stone masonry,
- Stability of this retaining wall depends mainly on its own weight,
- Not economical for long and high retaining wall.

B. Semi-gravity retaining walls

- Same as gravity retaining walls
- Small amount of steel is added to reduce the section of the wall

C. Cantilever retaining walls

- Made of reinforced concrete that consists of a thin stem and a base slab
- Economical to a height of about 8 m

D. Counterfort retaining walls

- Similar to cantilever walls
- At regular intervals, a thin vertical concrete slab known as counterforts is added. It ties the wall and the base slab together to reduce the shear and the bending moments.

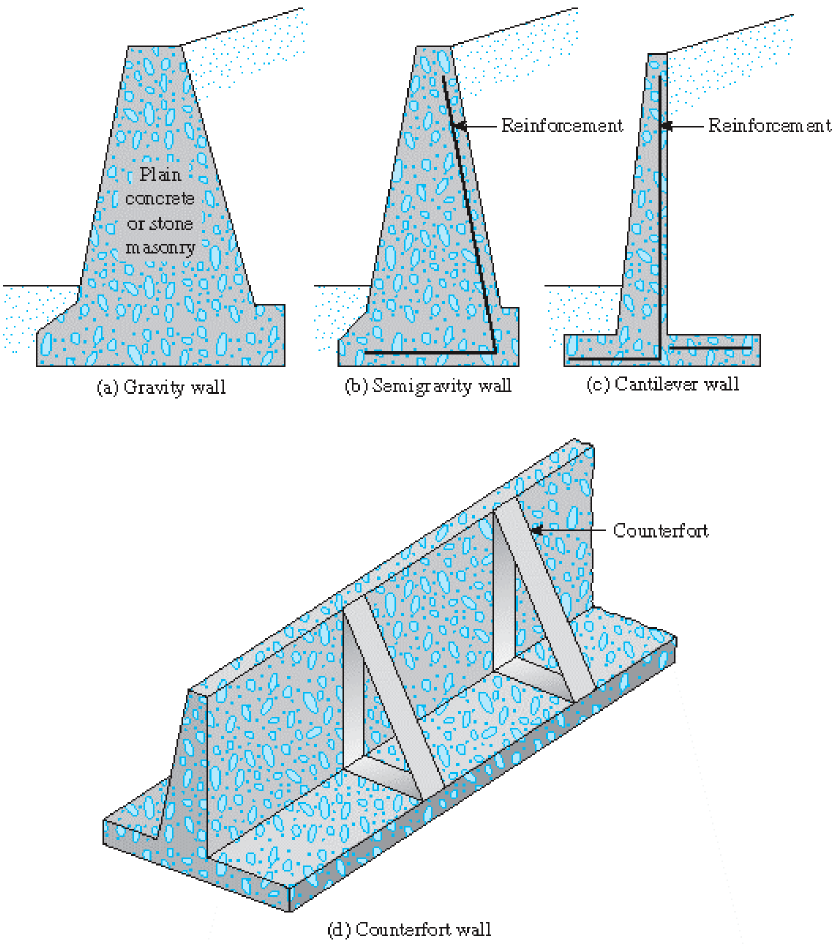
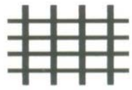
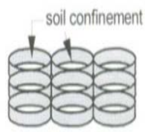


Figure 17: Types of retaining walls

3.1.2 MECHANICALLY STABILIZED EARTH WALLS

- More recently, soil reinforcement has been used in the construction and design of foundations, retaining walls, embankment slopes, and other structures.
- Depending on the type of construction, the reinforcements may be galvanized metal strips, geotextiles, geogrids, or geo-composites (see Figure 18).
- Reinforcement materials such as metallic strips, geotextiles, and geogrids are now being used to reinforce the backfill of retaining walls, which are generally referred to as *mechanically stabilized retaining walls*. Table 8 gives a summary for different types of geosynthetic reinforcement's materials.
- Reinforced earth (see Figure 19) is a construction material made from soil that has been strengthened by tensile elements such as metal rods or strips, geotextiles, geogrids, and the like.

Table 8: Types of geosynthetic reinforcements

Type	Definition	Uses	Form
Geotextiles	Continuous sheets of woven, nonwoven bounded fibers or yarns.	The constructional material in conjunction with other material such as soil & rock.	
Geogrids	Geosynthetic materials that have an open grid.	Reinforcement	
Geomembranes	Continuous flexible sheets manufactured from one or more synthetic materials. They are relatively impermeable.	Liners for fluid or gas containment and as vapor barriers.	
Geopipes	Perforated or solid polymeric pipes. In some cases, the perforated pipe is wrapped with geotextile filter.	Drainage of liquid or gas, in highway-railway edge drains and leachate removal systems	
Geocells	Relatively thick, three-dimensional networks constructed from strips of polymeric sheet. The strips are joined together to form interconnected cells that are filled with soil & sometimes concrete.	Reinforcement	
Geocomposites	This material is made from a combination of two or more geosynthetic types. Examples: geotextile-geonet; geotextile-geogrid; or a geosynthetic clay liner. Prefabricated.	Some application of Gocomposites is blanket drains which used for enhancing road base drainage. Panel drain, which reducing hydrostatic pressure and edge drain to collect and	

Type	Definition	Uses	Form
	Geocomposite drains or prefabricated vertical drains are formed by a plastic drainage core surrounded by geotextile filter.	remove lateral seepage from the road base.	

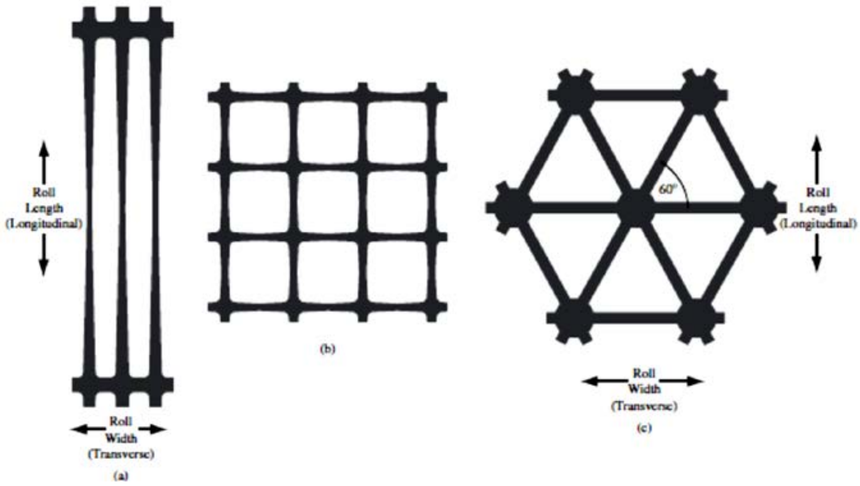
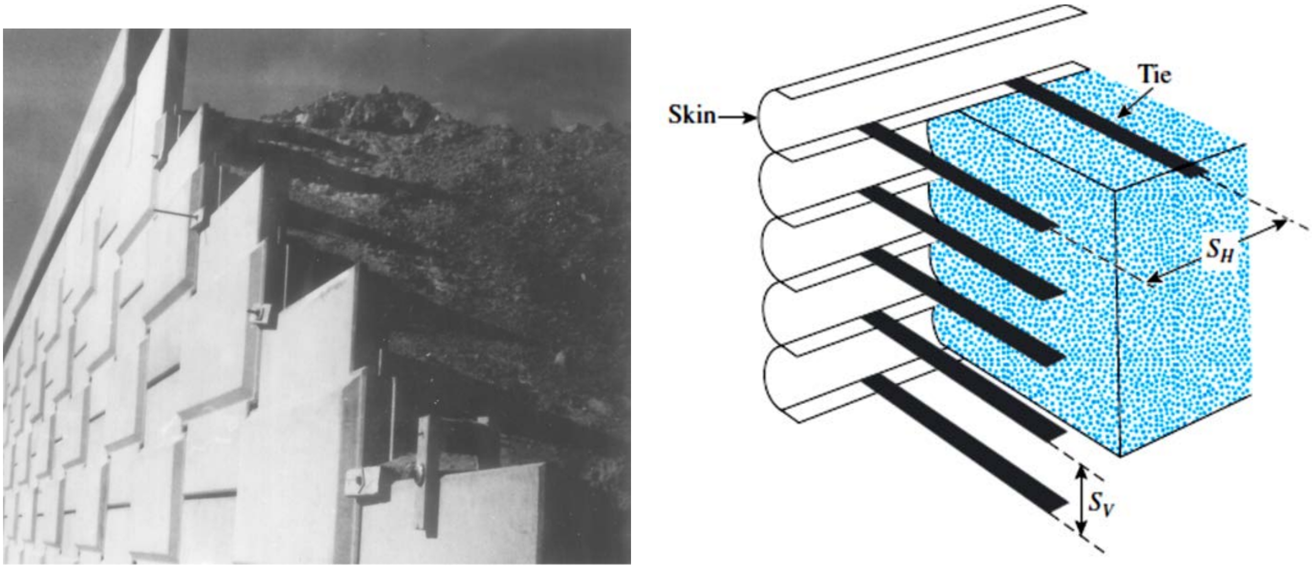


Figure 18: Geogrid: (a) uniaxial; (b) biaxial; (c) with triangular apertures



(a): Picture for real reinforced earth retaining wall (b): Illustration of the reinforced earth

Figure 19: Reinforced – Earth retaining wall

3.2 STABILITY OF RETAINING WALLS

There are two phases in the design of a conventional retaining wall. However, the current course presents the procedures for determining the *stability of the retaining wall*. Checks for *strength* can be found in any textbook on reinforced concrete.

a) With the lateral earth pressure known, the structure as a whole is checked for stability. The structure is examined for possible ***overturning, sliding, and bearing capacity failures***.
b) Each component of the structure is checked for strength, and the steel reinforcement of each component is determined.

A retaining wall may fail in any of the following ways:

- It may overturn about its toe. (See Figure 20 a.)
- It may slide along its base. (See Figure 20 b.)
- It may fail due to the loss of bearing capacity of the soil supporting the base. (See Figure 20 c.)
- It may undergo deep-seated shear failure. (Figure 20 d.)
- It may go through excessive settlement.

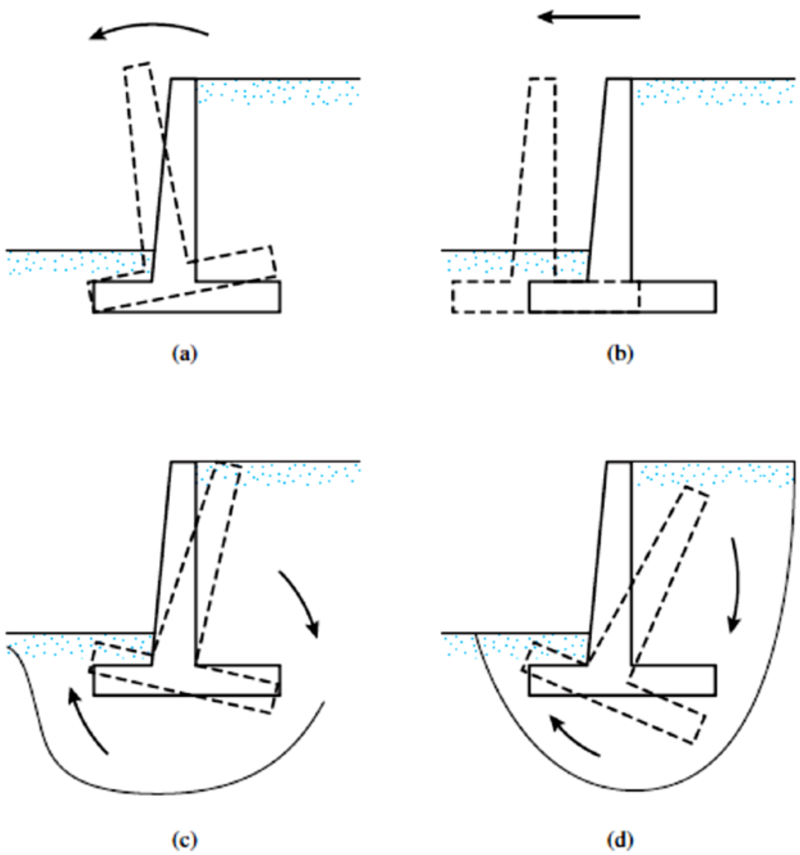


Figure 20: Failure of retaining wall :(a) by overturning; (b) by sliding;(c) by bearing capacity failure and (d) by deep-seated shear failure

3.3 PROPORTIONING RETAINING WALLS

- In designing retaining walls, an engineer must assume some of their dimensions which is called ***proportioning***, such assumptions allow the engineer to check trial sections of the walls for stability.
- If the stability checks yield undesirable results, the sections can be changed and rechecked. Figure 21 shows the general proportions of various retaining-wall components that can be used for initial checks.
- Note that the top of the stem of any retaining wall should not be less than about 0.3 m for proper placement of concrete.
- The depth, D, to the bottom of the base slab should be a minimum of 0.6 m. However, the bottom of the base slab should be positioned below the seasonal frost line.
- For counterfort retaining walls, the general proportion of the stem and the base slab is the same as for cantilever walls. However, the counterfort slabs may be about 0.3 m thick and spaced at center-to-center distances of 0.3 H to 0.7 H.

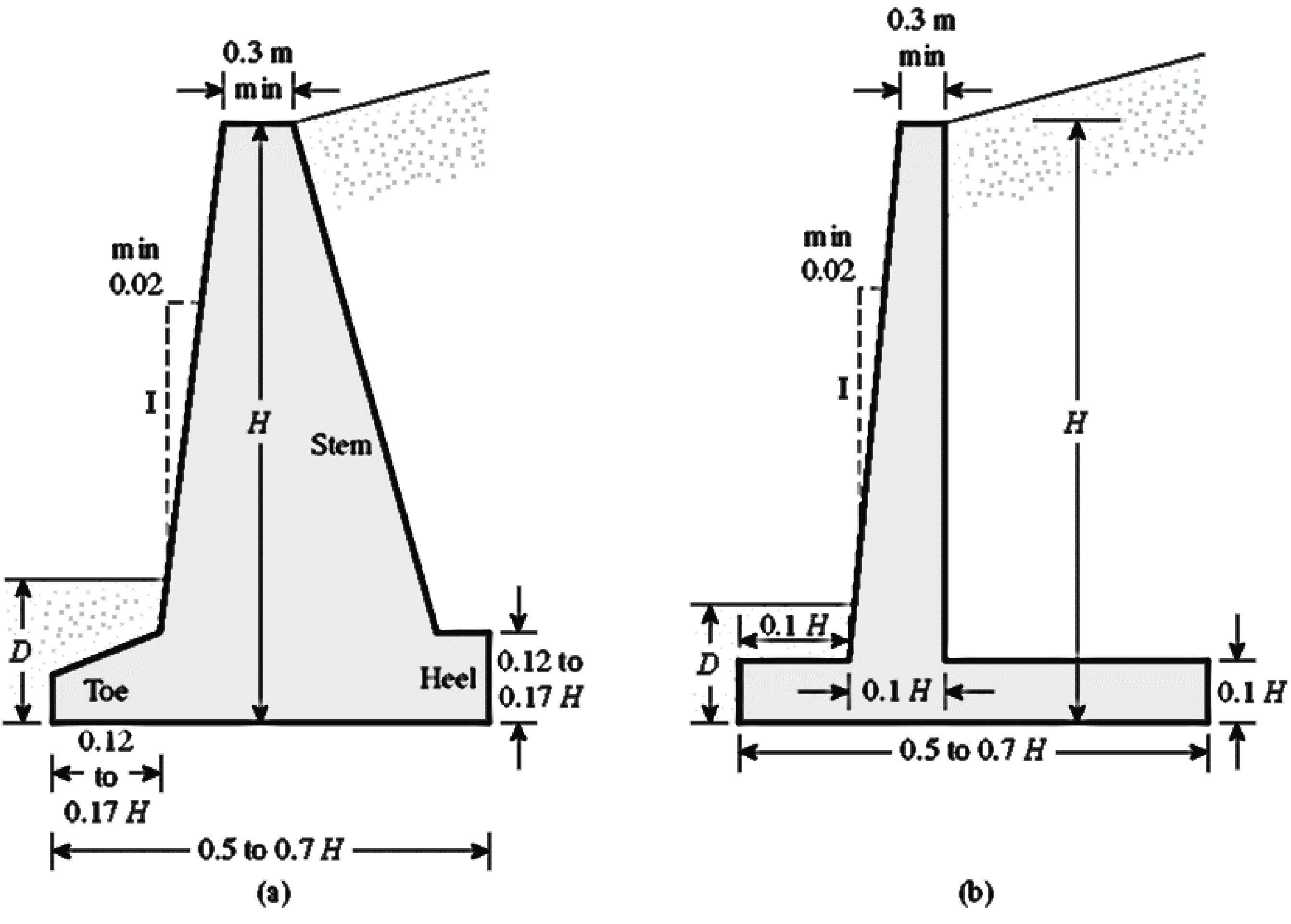


Figure 21: Approximate dimensions for various components of retaining wall for initial stability checks: (a) gravity wall; (b) cantilever wall

3.4 CHECKING THE STABILITY OF THE RETAINING WALLS

- The stability of the retaining wall should be checked against overturning, sliding and bearing capacity. This check can be regarded as an overall checkup.
- The retaining walls must be checked for the section requirements. The latter one is achieved according to reinforced concrete design criteria.

3.4.1 CHECK FOR OVERTURNING

Figure 22 shows the forces acting on a cantilever and a gravity retaining wall, based on the assumption that the Rankine active pressure is acting along a vertical plane AB drawn through the heel of the structure.

$$E_a = \frac{1}{2}\gamma H^2 K_a - 2\,c\,H\sqrt{K_a}$$
$$E_p = \frac{1}{2}\gamma D^2 K_p + 2\,c\,D\sqrt{K_p}$$

The factor of safety against overturning about the toe—that is, about point C in Figure 22—may be expressed as:

$$FS_{overturning} = \frac{Resisting\ Moment}{Overturning\ (Driving)\ Moment} = \frac{\sum M_R}{\sum M_o}$$

In Figure 22, the driving or overturning moment is:

$$\sum M_D = Ea_h\left(\frac{H'}{3}\right)$$
$$Ea_h = E_a\ \cos\alpha$$

To calculate the resisting moment, $\sum M_R$, (neglecting E_p), a table such as Table 9 can be prepared. The weight of the soil above the heel and the weight of the concrete (or masonry) are both forces that contribute to the resisting moment. It is important to note that the force Ea_v also contributes to the resisting moment. Ea_v is the vertical component of the active force or:

$$Ea_v = E_a\ \sin\alpha$$

Once is known, $\sum M_R$ the factor of safety can be calculated as:

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

The allowable factor of safety is always ranging from 2 to 3.

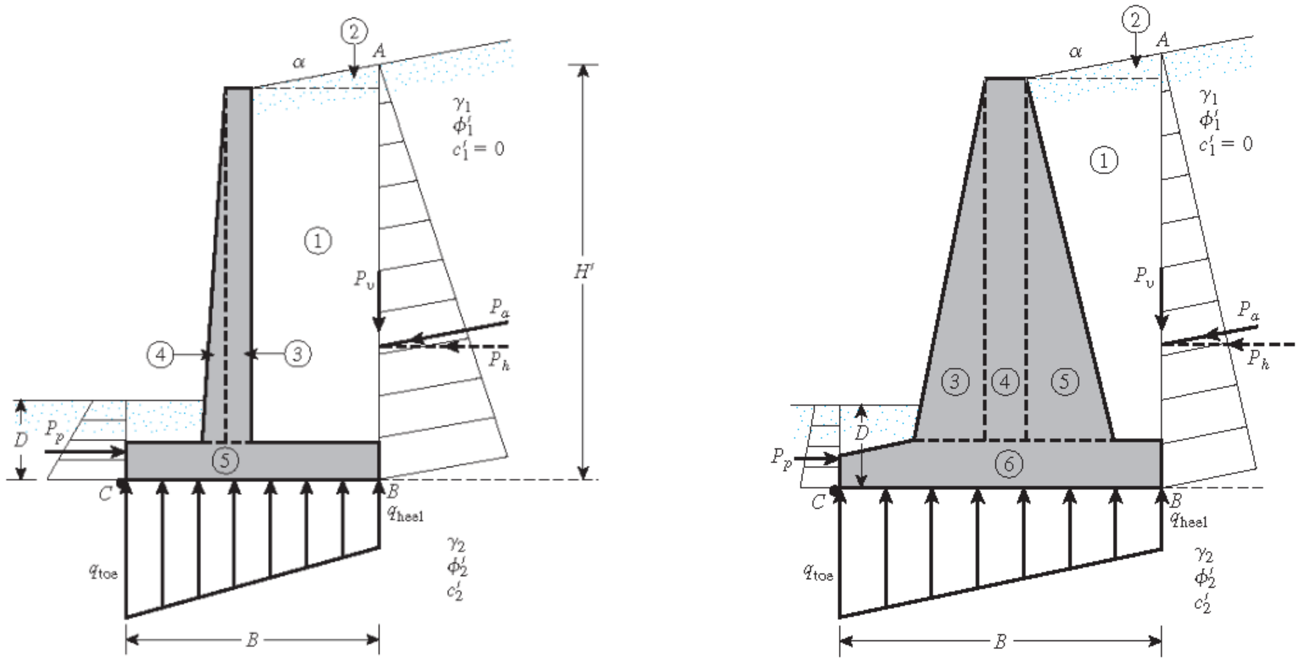


Figure 22: Check for overturning, assuming that the Rankine pressure is valid

Table 9: 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	$W_2 = \gamma_1 \times A_2$	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

(*Note:* γ_l = unit weight of backfill
γ_c = unit weight of concrete)

3.4.2 CHECK FOR SLIDING ALONG THE BASE

Referring to Figure 23, the factor of safety against sliding may be expressed by the equation:

$$FS_{Sliding} = \frac{\textit{horizontal resisting forces}}{\textit{horizontal driving forces}} = \frac{\sum F_R}{\sum F_d}$$

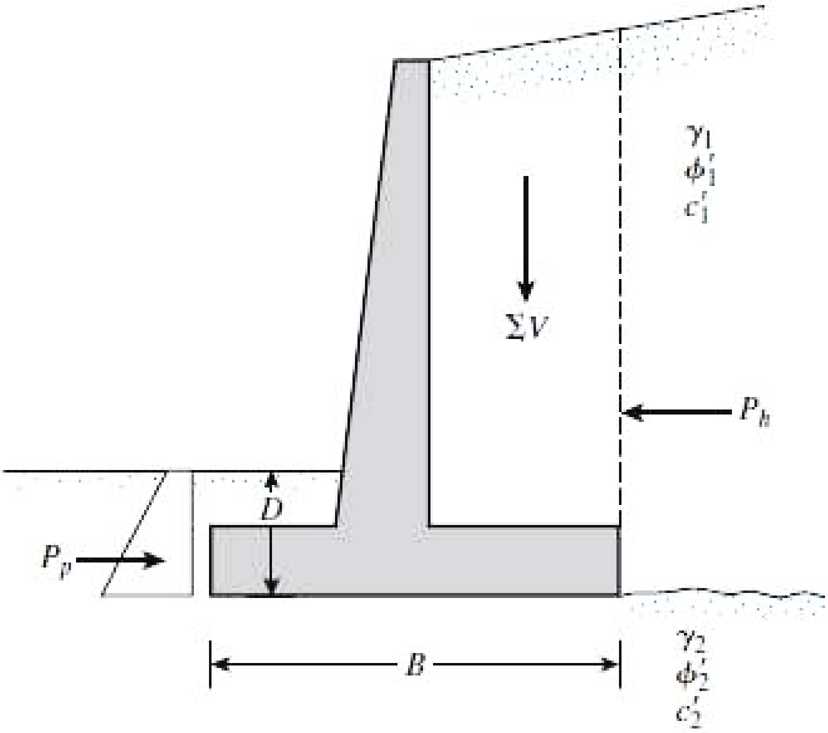


Figure 23: Checking for Sliding along the Base

The strength of the soil (τ) immediately below the base slab may be represented as:

$$\tau = c_a + \sigma_v \tan\delta$$

Where, is the friction angle between foundation materials and soil. Some typical values are listed in Table 10. Now convert the stresses to forces by multiplying the stress with ($B \times 1.0$) to get:

$$S = c_a B + R_v \tan\delta$$

$$\therefore FS_{Sliding} = \frac{S}{Ea_h} \geq 1.5$$

$$or\ FS_{Sliding} = \frac{S + E_p}{Ea_h} \geq 2.0$$

Table 10: Friction angles δ between various foundation materials and soil or rock* (Bowles, 1997)

Interface materials	Friction angle, δ, degrees†
Mass concrete or masonry on the following:	
Clean sound rock	35°
Clean gravel, gravel-sand mixtures, coarse sand	ϕ
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	ϕ
Clean fine sand, silty or clayey fine to medium sand	ϕ
Fine sandy silt, nonplastic silt	ϕ
Very stiff and hard residual or preconsolidated clay	ϕ
Medium stiff and stiff clay and silty clay	ϕ
Steel sheet piles against the following:	
Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22°
Clean sand, silty sand-gravel mixture, single-size hard rock fill	17
Silty sand, gravel, or sand mixed with silt or clay	14
Fine sandy silt, nonplastic silt	11
Formed concrete or concrete sheetpiling against the following:	
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22–26
Clean sand, silty sand-gravel mixture, single-size hard rock fill	17–22
Silty sand, gravel, or sand mixed with silt or clay	17
Fine sandy silt, nonplastic silt	14
Various structural materials	
Masonry on masonry, igneous and metamorphic rocks:	
Dressed soft rock on dressed soft rock	35°
Dressed hard rock on dressed soft rock	33
Dressed hard rock on dressed hard rock	29
Masonry on wood (cross grain)	26
Steel on steel at sheet-pile interlocks	17
Wood on soil	14–16‡

*May be stress-dependent (see text) for sand.

†Single values ±2°. Alternate for concrete poured on soil is $\delta = \phi$.

‡May be higher in dense sand or if sand penetrates wood.

If the factor of safety is not achieved, the following several alternatives may be investigated as shown in Figure 24:

- Increase the width of the base slab (i.e., the heel of the footing).
- Use a key to the base slab. If a key is included, the passive force per unit length of the wall becomes larger.

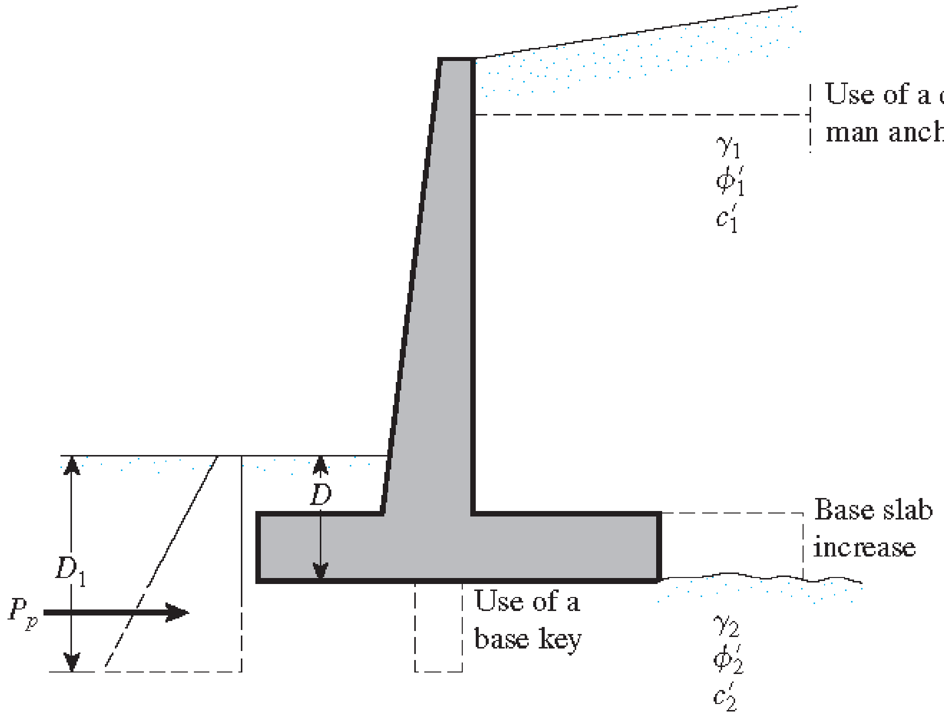


Figure 24: Alternatives for increasing the factor of safety with respect to sliding

3.4.3 CHECK AGAINST BEARING CAPACITY FAILURE

In addition to the overturning and sliding checking, the retaining wall must be checked against bearing failure. This check can be regarded as bearing capacity check that should satisfy the shear strength and settlement of the bearing soil. The illustration of the total loads on the retaining walls is given in Figure 22 and the details of the bearing capacity check of the retaining walls is given in Figure 25.

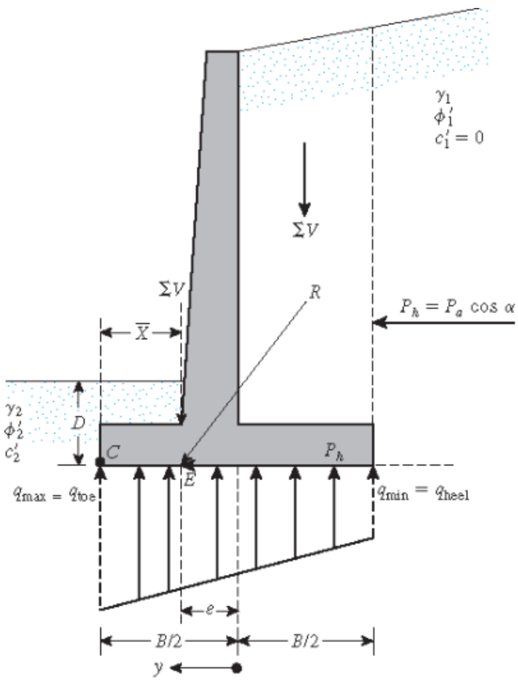
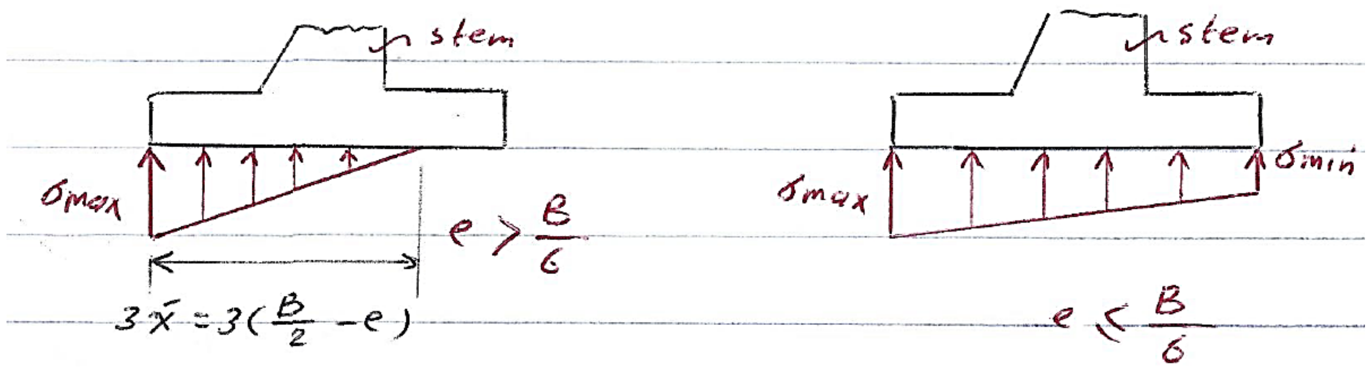


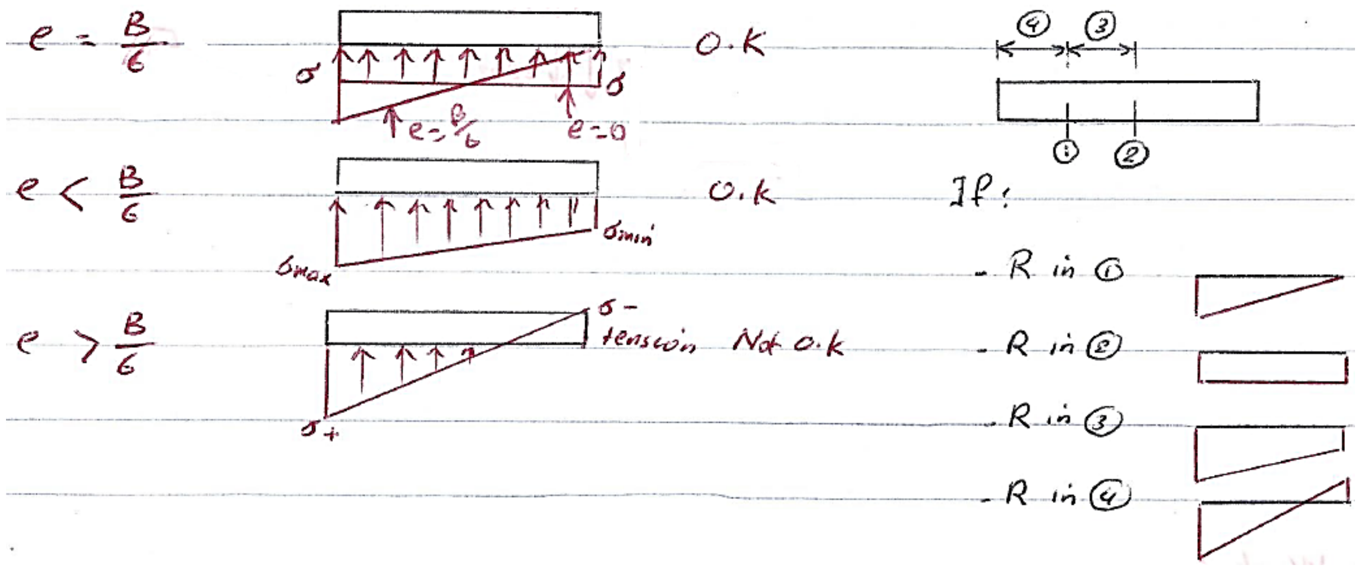
Figure 25: Explanation of the check for the bearing capacity failure.

The maximum and minimum pressure under the base can be obtained from:

$$\sigma_{max} = \frac{4R_v}{3(B-2e)} \qquad \sigma_{max/min} = \frac{R_v}{B}\left[1 \pm \frac{6e}{B}\right]$$



The explanation of the eccentricity (e) is given below:



3.5 APPLICATION OF RANKIN'S THEORY TO SPECIAL CASES

The stability of retaining walls can be checked by Rankin's theory for other cases as shown in Figure 26. For the inclined surface case with granular soil, the active lateral earth force estimated as given below:

$$Ea = \frac{1}{2}\gamma H^2 K_a$$

And the value of K_a is:

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

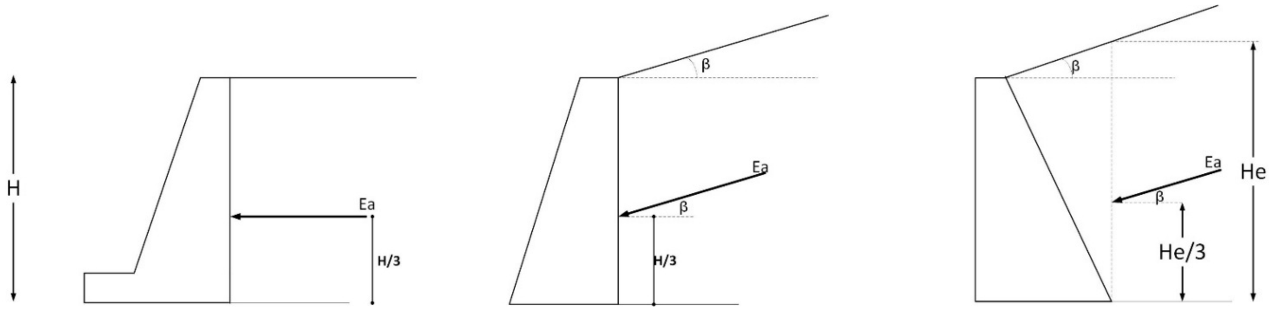
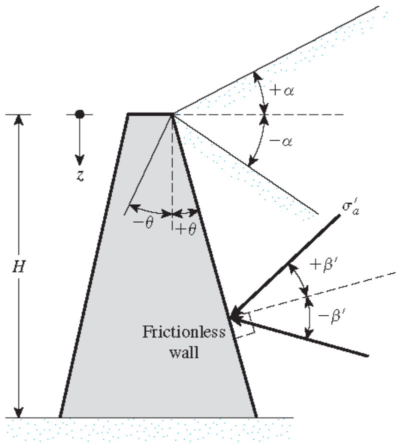


Figure 26: Different cases for retaining walls solved by Rankin's Theory

In the literature, more complicated and generalized case for the retaining walls can be made:



$$Ea = \frac{1}{2}\gamma H^2 K_a$$

$$K_a = \frac{\cos(\alpha - \theta)\sqrt{1 + \sin^2\phi' - 2\sin\phi'\cos\psi_a}}{\cos^2\theta\left(\cos\alpha + \sqrt{\sin^2\phi' - \sin^2\alpha}\right)}$$

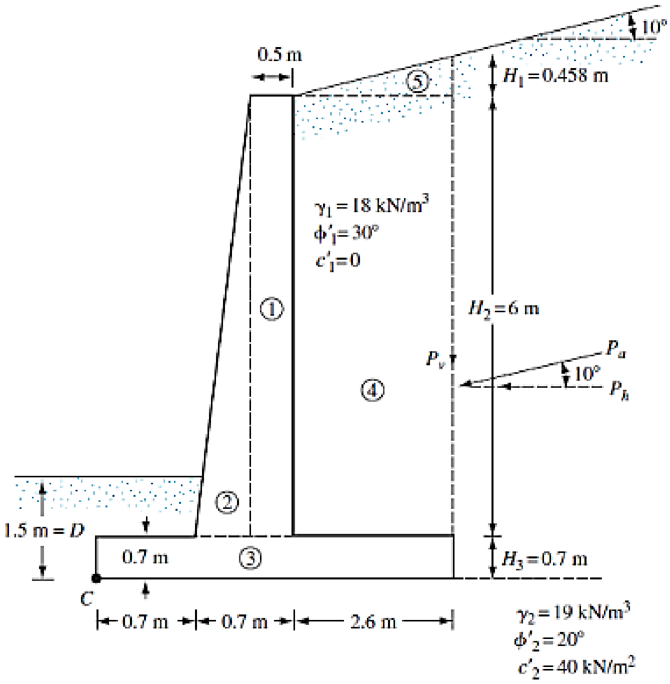
= Rankine active earth-pressure coefficient for generalized case

where $\psi_a = \sin^{-1}\left(\frac{\sin\alpha}{\sin\phi'}\right) - \alpha + 2\theta$ $\qquad$ $\beta' = \tan^{-1}\left(\frac{\sin\phi'\sin\psi_a}{1 - \sin\phi'\cos\psi_a}\right)$

3.6 ILLUSTRATIVE EXAMPLES AND HOMEWORK

3.6.1 EXAMPLE NO. 1

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



Solution:

$H' = 0.7 + 6.0 + 0.458 = 7.158\, m$

Since the soil behind the wall inclined with α=10°, hence, the value of Ka should be calculated from the following equation:

$$K_a = \cos\alpha \, \frac{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\emptyset'}}{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\emptyset'}} = 0.3495$$

$$E_a = \frac{1}{2}\,\gamma\, H^2\, K_a - 2\,c\,\sqrt{K_a} = 0.5 \,\times 18.0 \,\times 7.158^2 \times 0.3495 - 0 = 161.17\; kN/m$$

$$Ea_v = E_a\;\sin\alpha = 161.17\, \sin 10 = 27.98\; kN/m,\; Ea_h = E_a\;\cos\alpha = 161.17\,\cos 10 = 158.72\; kN/m$$

To calculate the FS against overturning and sliding, the following table can be prepared: (use the unit weight of concrete=23.58 kN/m³.

Area No.	Area (m²)	W/unit length	Moment arm, m	Moment, kN.m/m	Type of moment
1 (concrete)	6 ×0.5=3.0	70.74	1.15	81.35	Resisting
2 (concrete)	0.5 ×0.2×6=0.6	14.15	0.83	11.79	Resisting
3 (concrete)	4 × 0.7=2.8	66.02	2.00	132.04	Resisting
4 (soil)	6 × 2.6=15.6	280.80	2.70	758.16	Resisting
5 (soil)	0.5 ×2.6×0.458	10.71	3.13	3.52	Resisting
		$Ea_v = 27.98$	4.0	111.92	Resisting

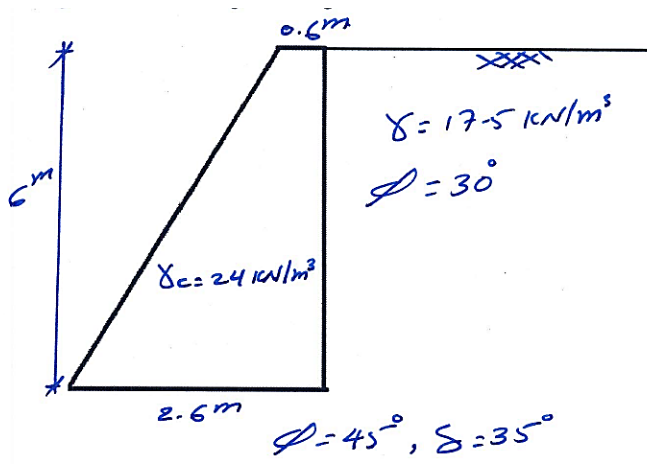
$$FS_{overturning} = \frac{Resisting\ Moment}{Overturning\ (Driving)\ Moment} = \frac{\sum M_R}{\sum M_o} = \frac{\sum M_R}{E_a\ \cos\alpha\ \left(\frac{H'}{3}\right)} = \frac{1128.74}{158.72\ \times \frac{7.158}{3}} = 2.98 > 2\ \therefore OK$$

$$FS_{Sliding} = \frac{S}{Ea_h} = \frac{c_a B + R_v\, tan\,\delta}{Ea_h} = \frac{\frac{2}{3} \times 40\ \times 4.0 + 470.4\ \times \tan\frac{2}{3} \times 20}{158.72} = 1.37 < 1.5\ \therefore N.OK$$

H.W., Please try to find the FS $_{sliding}$ by taking into consideration the passive zone on the left side.

3.6.2 EXAMPLE NO. 2

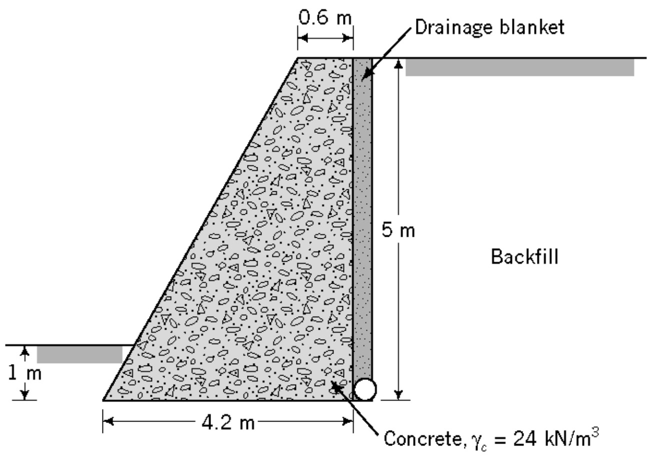
For the following retaining wall, find the factor of safety against sliding.



3.6.3 EXAMPLE NO. 3

A gravity retaining wall, shown in Figure below, is required to retain 5 m of soil. The backfill is a coarse-grained soil with $\gamma_{sat} = 18.0\ \frac{kN}{m^3}$, $\emptyset' = 30.0^{\circ}$. The existing soil (below the base) has the following properties: $\gamma_{sat} = 20.0\ \frac{kN}{m^3}$, $\emptyset' = 36.0^{\circ}$. The wall is embedded 1 m into the existing soil and a drainage system is provided, as shown. The groundwater level is 4.5 m below the base of the wall. Determine the stability of the wall for the following conditions (assume $\delta = 20.0^{\circ}$):

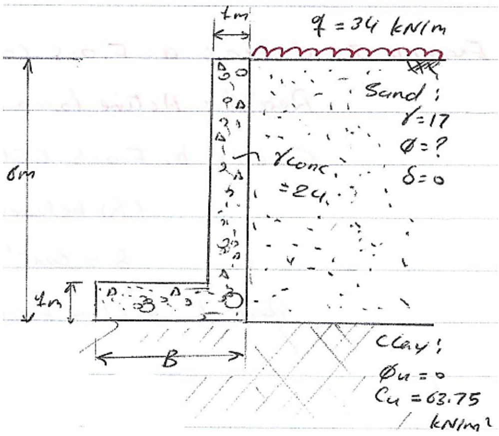
1. Wall friction is zero.
2. The drainage system becomes clogged during several days of a rainstorm and the groundwater rises to the surface. Neglect seepage forces.



3.6.4 EXAMPLE NO. 4

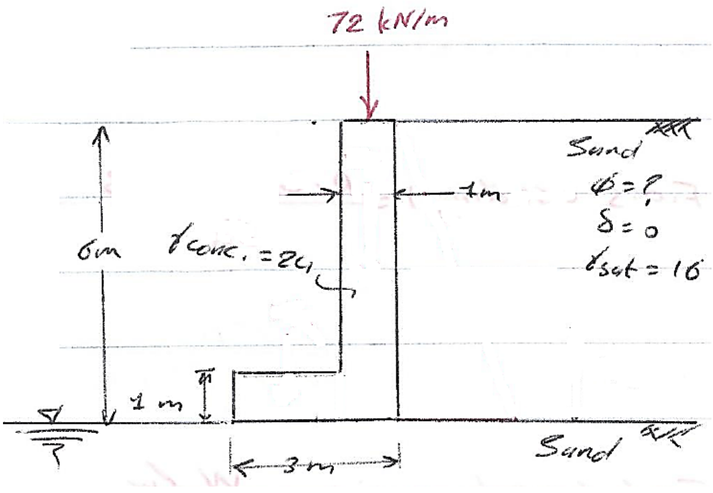
For the cantilever retaining wall shown, if the F.O.S. (Overturning)= F.O.S. (Sliding), find:

A) The width B
B) Least value of ∅ so as to make the wall stable against sliding.



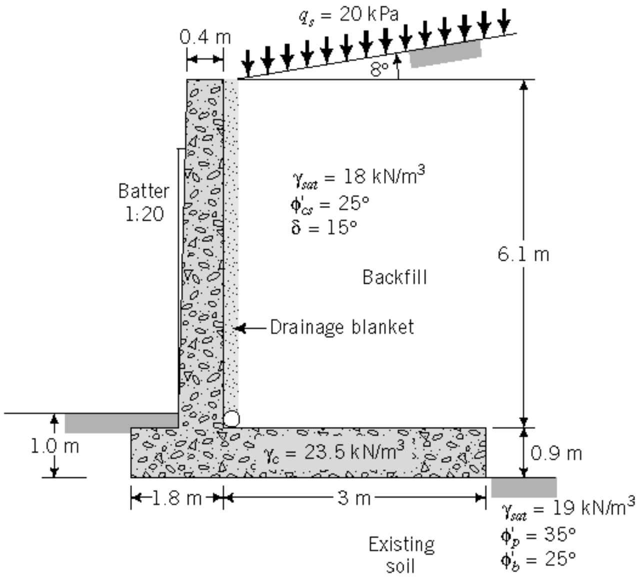
3.6.5 EXAMPLE NO. 5

a) Given: F.O.S. (overturning)=149/16, Required: active force and Ø
b) Given: F.O.S. (sliding)=2.2 and for the angle of friction between soil at the base with the foundation (concrete) is $\delta = \tan^{-1} 0.8$, Required: active force and Ø



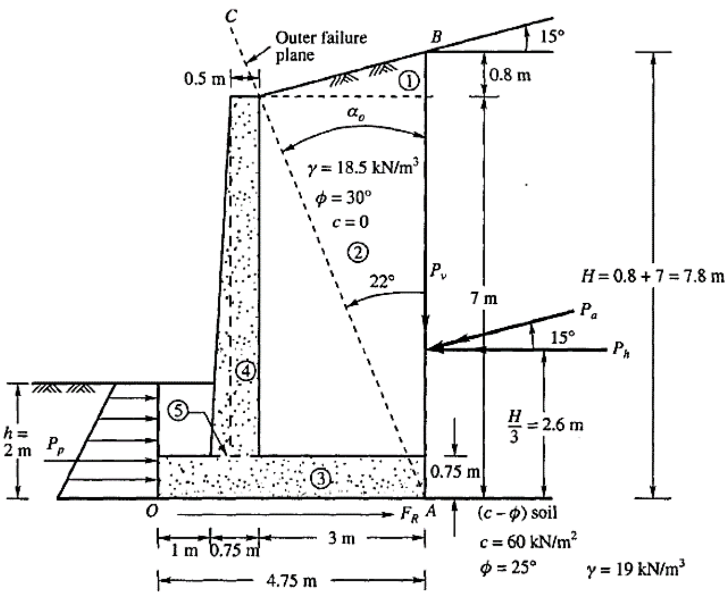
3.6.6 HOMEWORK NO. 1

Determine the stability of the cantilever gravity retaining wall shown in figure below. The existing soil is a clay and the backfill is a coarse-grained soil. The base of the wall will rest on a 50-mm-thick, compacted layer of the backfill. The interface friction between the base and the compacted layer of backfill is 25.0°. Groundwater level is 8 m below the base.



3.6.7 HOMEWORK NO. 2

Figure below shows a section of a cantilever wall with dimensions and forces acting thereon. Check the stability of the wall with respect to (a) overturning, (b) sliding, and (c) bearing capacity.



Ans: FS $_{overturning}$=3.78, and FS $_{sliding}$=5.7

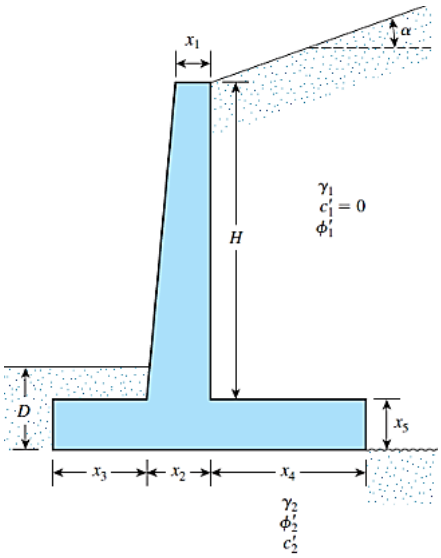
Assume adhesion factor=0.55, and wall friction=angle of internal friction, unit weight of concrete=24.0 kN/m^3

3.6.8 HOMEWORK NO. 3

For the cantilever retaining wall shown below, calculate the factor of safety with respect to overturning, sliding, and bearing capacity. The required data are listed below:

Wall dimensions: H=8.0m, x1=0.4m, x2=0.6m, x3=1.5m, x4=3.5m, x5=0.96m, D=1.75, α=10.0°

Soil properties:γ_1=16.5 kN/m^3, ϕ'_1=32°,:γ_2=17.6 kN/m^3, ϕ'_2=28°, c'_2=30 kPa.



3.6.9 HOMEWORK NO. 4

Repeat Homework 3 with the following properties:

Wall dimensions: H=6.5m, x1=0.3m, x2=0.6m, x3=0.8m, x4=2.0m, x5=0.80m, D=1.5, α=0.0°

Soil properties:γ_1=18.08 kN/m^3, ϕ'_1=36°,:γ_2=19.65 kN/m^3, ϕ'_2=15°, c'_2=30 kPa.

Sheet Pile Walls 4

4.1 DEFINITION AND USES

The sheet piles are one of the most well-known retaining structures that can be used for different applications. It is characterized as a flexible retaining walls. Connected or semi-connected sheet piles are often used to:

1. Build continuous walls for waterfront structures that range from small waterfront pleasure boat launching facilities to large dock facilities.
2. Sheet piles are also used for some temporary structures, such as braced cuts.
3. Sheet piling is also used for beach erosion protection;
4. Stabilizing ground slopes, particularly for roads
5. Shoring walls of trenches and other excavations; and for cofferdams

In contrast to the construction of other types of retaining wall, the building of sheet pile walls does not usually require dewatering of the site.

4.2 TYPES AND MATERIALS USED FOR SHEET PILING

4.2.1 TIMBER SHEET PILING

Timber piling is sometimes used for free-standing walls of $H < 3$ m. It is more often used for temporarily braced sheeting to prevent trench cave during installation of deep water and sewer lines. If timber sheeting is used in permanent structures above water level, preservative treatment is necessary, and even so the useful life is seldom over 10 to 15 years.

4.2.2 REINFORCED CONCRETE SHEET PILING

These sheet piles are precast concrete members, usually with a tongue-and-groove joint. Even though their cross section is considerably dated.

They are designed for service stresses, but because of their mass, both handling and driving stresses must also be taken into account. The points are usually cast with a bevel, which tends to wedge the pile being driven against the previously driven pile.

4.2.3 COMPOSITE SHEET-PILE WALLS

Walls may be constructed using composite construction. (for more details, please see Bowles, 1997)

4.2.4 STEEL SHEET PILING

Steel sheet piling is the most common type used for walls because of several advantages over other materials:

1. It is resistant to the high driving stresses developed in hard or rocky material.

2. It is relatively lightweight.
3. It may be reused several times.
4. It has a long service life either above or below water if it is provided with modest protection
5. It is easy to increase the pile length by either welding or bolting. If the full design length cannot be driven, it is easy to cut the excess length using a cutting torch.
6. Joints are less apt to deform when wedged full with soil and small stones during driving.
7. A nearly impervious wall can be constructed by driving the sheeting with a removable plug in the open thumb-and-finger joint.

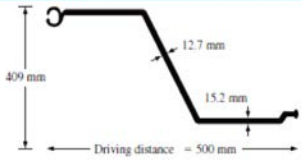
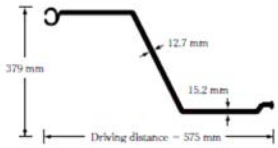
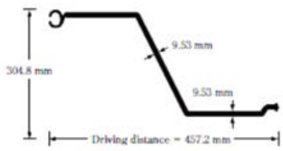
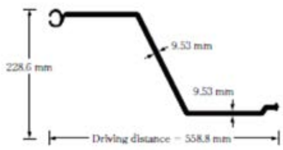
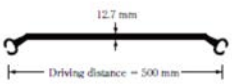
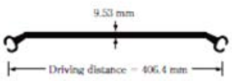
Table 4.3-1 lists the properties of the steel sheet pile sections produced by the Bethlehem Steel Corporation.

4.3 CONSTRUCTION METHODS OF SHEET PILES

The construction methods generally can be divided into two categories:

1. Backfilled structure (See Figure 4.3-1)
2. Dredged structure

Table 4.3-1: Properties of Some Sheet-Pile Sections Produced by Bethlehem Steel Corporation

Section designation	Sketch of section	Section modulus m^3/m of wall	Moment of inertia m^4/m of wall
PZ-40		326.4×10^{-5}	670.5×10^{-6}
PZ-35		260.5×10^{-5}	493.4×10^{-6}
PZ-27		162.3×10^{-5}	251.5×10^{-6}
PZ-22		97×10^{-5}	115.2×10^{-6}
PSA-31		10.8×10^{-5}	4.41×10^{-6}
PSA-23		12.8×10^{-5}	5.63×10^{-6}

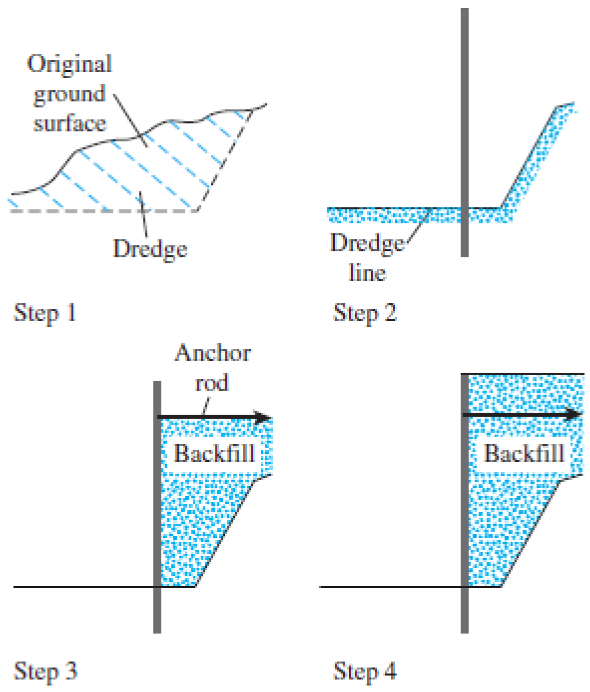


Figure 4.3-1: Sequence of construction for a backfilled structure

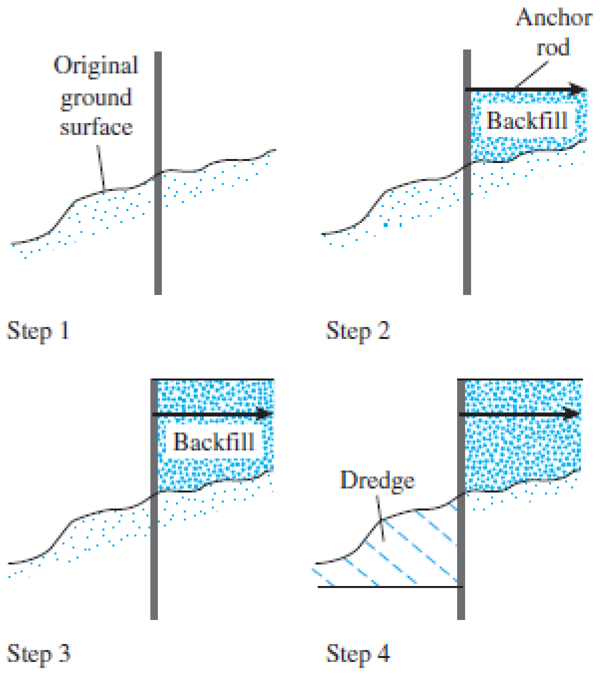


Figure 4.3-2: Sequence of construction for a dredged structure

4.4 ANALYSIS OF SHEET PILES

- The sheet piles are analyzed and design to adjust the design requirements. The requirements always classified into two main objectives. The first one is the section requirements of sheet piles and the second one is the overall stability of the sheet pile within the penetrated soil.
- The analysis depends mainly on the type and condition of the soil type at the site. The other additional requirements such as amount of the applied loads can be enrolled later.
- The sheet piles are mainly classified into two types: cantilever and anchored sheet piles.

4.4.1 CANTILEVER SHEET PILE WALLS

- Cantilever sheet pile walls are usually recommended for walls of ***moderate height— about 6 m*** or less, measured above the dredge line.
- In such walls, the sheet piles act as a ***wide cantilever beam*** above the dredge line.
- The basic principles for estimating net lateral pressure distribution on a cantilever sheet-pile wall can be explained with the aid of Figure 4.4-1. The figure shows the nature of lateral yielding of a cantilever wall penetrating a sand layer below the dredge line.
- Care should be taken in determining the water level that will affect the net pressure diagram.

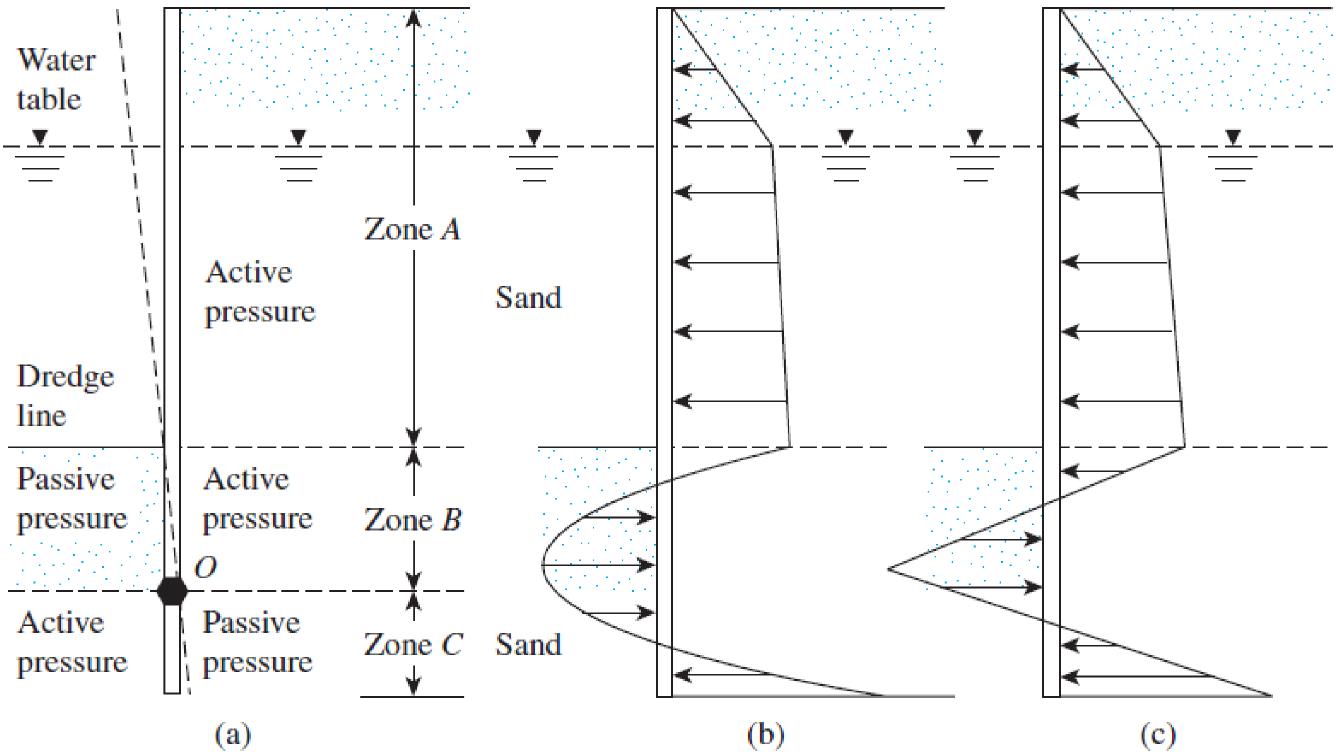


Figure 4.4-1: Cantilever sheet pile penetrating sand

4.4.1.1 CANTILEVER SHEET PILING PENETRATING SANDY SOILS

- The soil retained by the sheet piling above the dredge line is sand. The water table is at a depth L1 below the top of the wall.
- At the level of the dredge line, the hydrostatic pressures from both sides of the wall are the same magnitude and cancel each other.

Detailed solution:

Refereeing to Figure 4.4-2, the variation of lateral earth pressure is shown in addition to the resulting bending moment in the sheet pile.

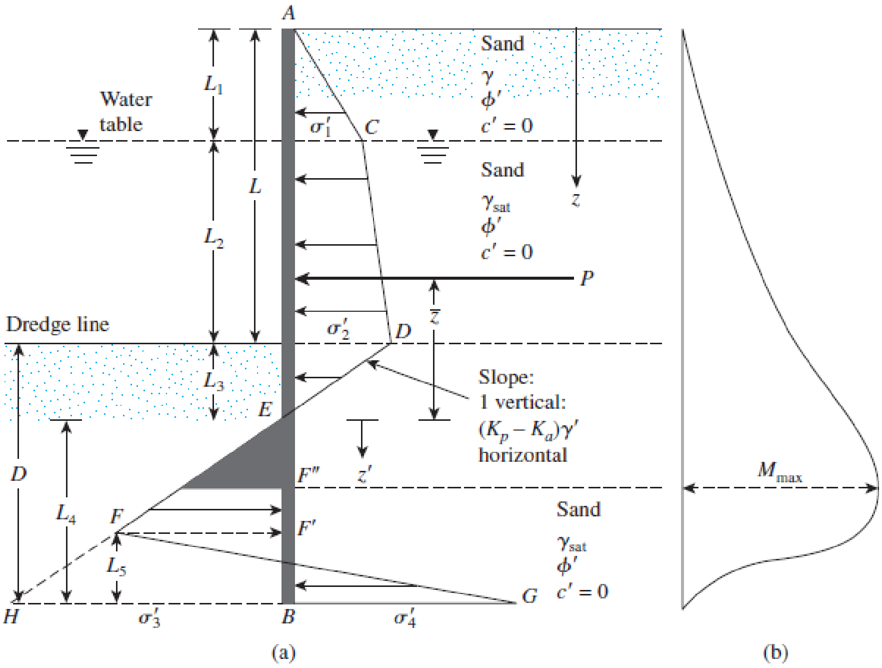


Figure 4.4-2: Cantilever sheet pile penetrating sand: (a) variation of net pressure diagram; (b) variation of moment

Following these illustration, the final results of derivation gives:

$$L_4^4 + A_1 L_4^3 - A_2 L_4^2 - A_3 L_4 - A_4 = 0$$

$$A_1 = \frac{\sigma_5'}{\gamma'(K_p - K_a)}$$

$$A_2 = \frac{8\,P}{\gamma'(K_p - K_a)}$$

$$A_3 = \frac{6\,P\left[2\,\bar{z}\,\gamma'\left(K_p - K_a\right) + \sigma_5'\right]}{\gamma'^2(K_p - K_a)^2}$$

$$A_4 = \frac{P\,[6\,\bar{z}\,\sigma_5' + 4\,P]}{\gamma'^2(K_p - K_a)^2}$$

$$\sigma_5' = (\gamma\,L_1 + \gamma'\,L_2)K_p + \gamma' L_3(K_p - K_a)$$

And penetration depth $D = L_3 + L_4$

To find the bending moment

$$P = \frac{1}{2}(z')^2\,(K_p - K_a)\gamma'$$

$$\text{Or } z' = \sqrt{\frac{2\,P}{(K_p - K_a)\gamma'}}$$

$$M_{max} = P\,(\bar{z} - z') - \left[\frac{1}{2}\gamma' z'^2 (K_p - K_a)\right]\left(\frac{1}{3}\right)\,z'$$

4.4.1.2 CANTILEVER SHEET PILING PENETRATING CLAYEY SOILS

- To show the difference in the analysis between sheet piles penetrated in sandy soil and those penetrated in cohesive soil,

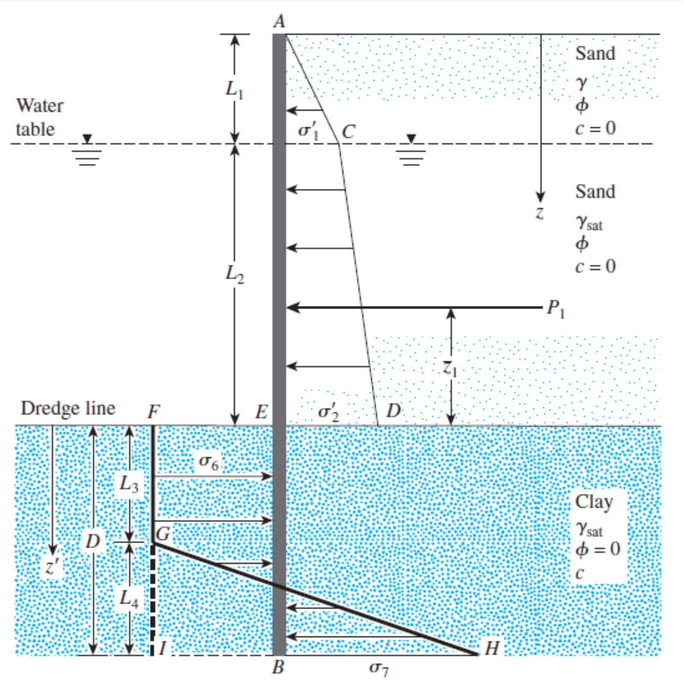


Figure 4.4-3: Cantilever sheet pile penetrating clay

Detailed solution:

Refereeing to Figure 4.4-3, the variation of lateral earth pressure is shown in addition to the resulting bending moment in the sheet pile. Following this illustration, the final results of derivation gives:

The active pressure, from right to left, is

$$\sigma_a = [\gamma\,L_1 + \gamma' L_2 + \gamma_{sat}(z - L_1 - L_2] - 2c$$

The passive pressure from left to right may be expressed as

$$\sigma_p = \gamma_{sat}(z - L_1 - L_2) + 2c$$

$\therefore\ \sigma_6 = \sigma_p - \sigma_a = 4c - (\gamma L_1 - \gamma' L_2)$

At the bottom of the sheet pile, the passive pressure from right to left is

$$\sigma_p = (\gamma L_1 + \gamma' L_2 + \gamma_{sat} D) + 2c$$

the active pressure from left to right is

$$\sigma_a = \gamma_{sat} D - 2c$$

$$\sigma_7 = \sigma_p - \sigma_a = 4c + (\gamma L_1 + \gamma' L_2)$$

From equilibrium equation in the horizontal direction, it can be get that:

$$L_4 = \frac{D\,[4c - (\gamma\, L_1 + \gamma' L_2)] - P_1}{4c}$$

taking the moment about point B,

$$D^2[4c - (\gamma L_1 + \gamma' L_2)] - 2DP_1 - \frac{P_1(P_1 + 12\, c\, \bar{z}_1}{(\gamma L_1 + \gamma' L_2) + 2c} = 0$$

The latter equation may be solved to obtain *D*, the theoretical depth of penetration of the clay layer by the sheet pile.

4.4.2 ANCHORED SHEET-PILE WALLS

When the height of the backfill material behind a cantilever sheet-pile wall exceeds about 6.0 m, tying the wall near the top to anchor plates, anchor walls, or anchor piles becomes more economical. This type of construction is referred to as *anchored sheet pile wall* or an *anchored bulkhead.* Anchors minimize the depth of penetration required by the sheet piles and also reduce the cross-sectional area and weight of the sheet piles needed for construction. However, the tie rods and anchors must be carefully designed.

The two basic methods of designing anchored sheet-pile walls are (a) the *free earth support* method and (b) the *fixed earth support* method. Figure 4.4-4 shows the assumed nature of deflection of the sheet piles for the two methods.

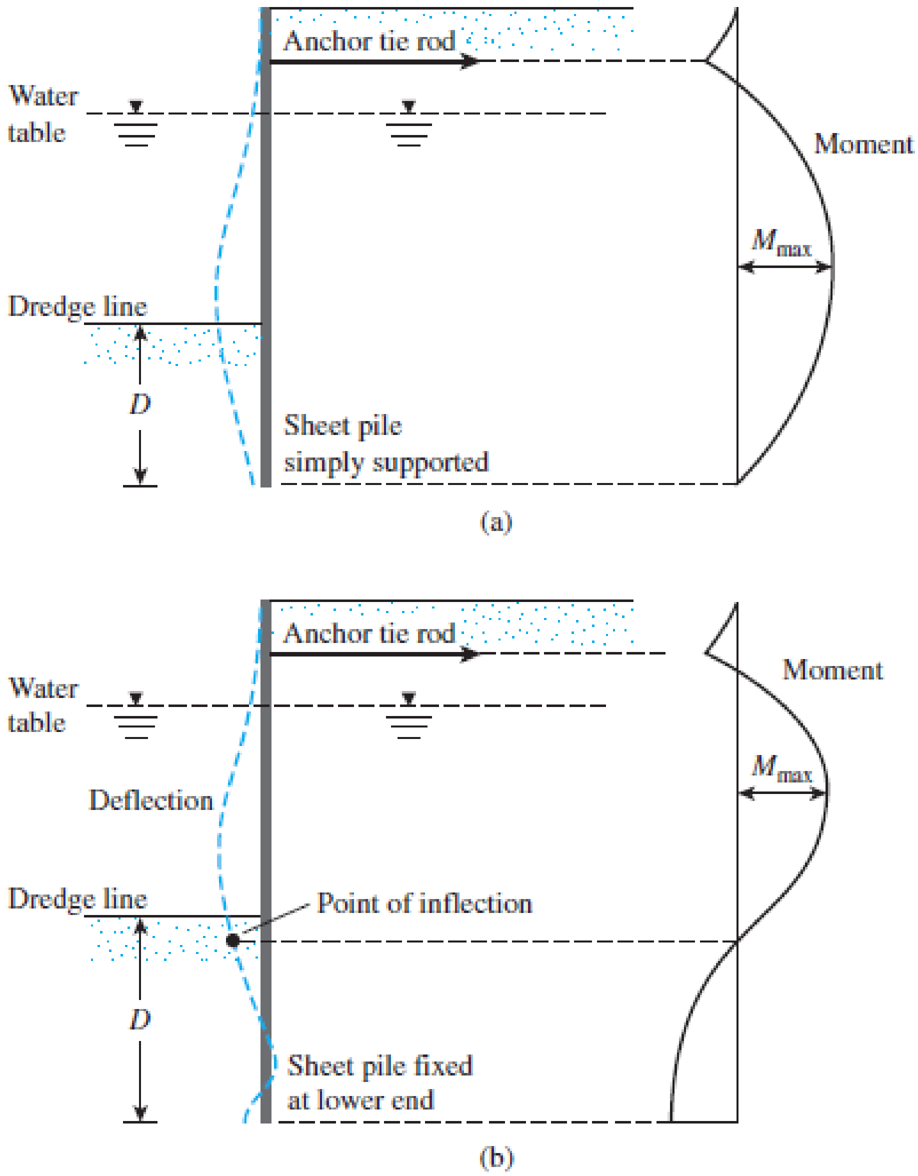


Figure 4.4-4: Nature of variation of deflection and moment for anchored sheet piles: (a) free earth support method; (b) fixed earth support method

4.4.3 TYPES OF ANCHORS

The general types of anchor used in sheet-pile walls are as follows:

1. Anchor plates and beams (Deadman)
2. Tie backs
3. Vertical anchor piles
4. Anchor beams supported by batter (compression and tension) piles

Anchor plates and beams are generally made of **cast concrete blocks**. (See Figure 4.4-5.) The anchors are attached to the sheet pile by *tie-rods*. A *wale* is placed at the front or back face of a sheet pile for the purpose of conveniently attaching the tie-rod to the wall. To protect the tie rod from corrosion, it is generally coated with paint or asphaltic materials.

In the construction of *tiebacks*, bars or cables are placed in predrilled holes (see Figure 4.4-5b) with concrete grout (cables are commonly high-strength, prestressed steel tendons).

Note: It is required to have an idea for the method of placement and construction of the anchors.

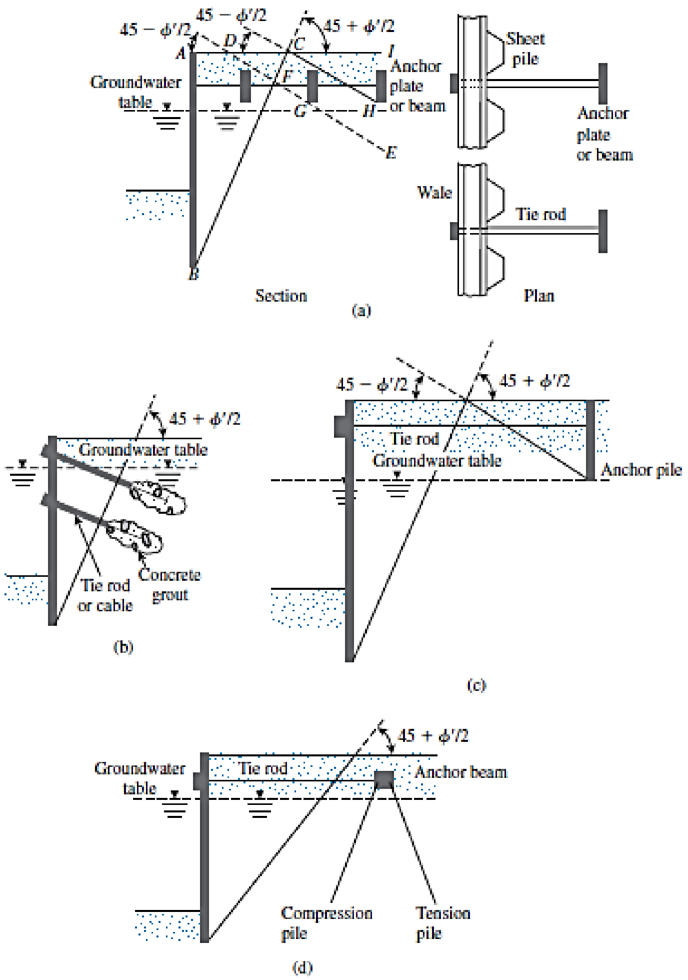


Figure 4.4-5: Various types of anchoring for sheet-pile walls: (a) anchor plate or beam; (b) tieback; (c) vertical anchor pile; (d) anchor beam with batter piles

4.4.4 SIMPLIFIED SOLUTION FOR THE ANALYSIS AND DESIGN OF SHEET PILES

4.4.4.1 CANTILEVER SHEET PILING PENETRATING COHESIONLESS SOIL

The solution for the cantilever sheet pile walls is shown in Figure 4.4-6.

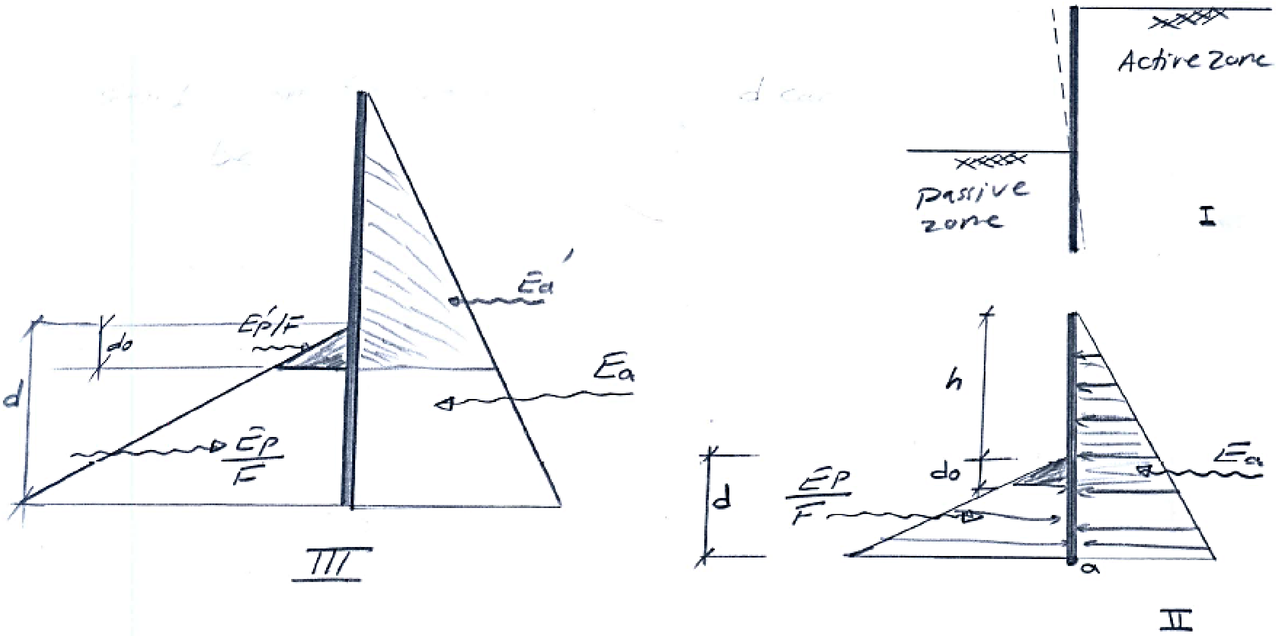


Figure 4.4-6: Free body diagram of the simplified solution for the free earth support sheet pile

The steps can be summarized as:

1. The soil within the passive side is divided by suitable safety factor (usually taken as 2.0).
2. Find the depth of penetration (d) by taking moment at point a [as shown in Figure 4.4-6 (II)]:

$$\sum M_a = 0 \rightarrow \; E_a \times \frac{h+d}{3} = \frac{E_p}{F} \times \frac{d}{3}$$

Where:

$$E_a = \frac{1}{2}\gamma k_a (h+d)^2 \;\; and \; E_p = \frac{1}{2}\gamma k_p (d)^2$$

$$F = factor\ of\ safety\ (taken\ as\ 2.0)$$

3. Increase the total depth of penetration (D) by (20-40) % of the depth (d); or:

$$\boldsymbol{D = (1.2 + 1.4)\, d} \;\; , usually\ \boldsymbol{D = 1.2\, d}$$

4. Find the depth of zero shear (do in Figure 4.4-6 III) that gives the maximum bending moment. This can be achieved by:

$$\sum F_x = 0 \rightarrow \; E_a{}' = \frac{E_p{}'}{F}$$

Where:

$$E_a{}' = \frac{1}{2}\gamma k_a (h+d_o)^2 \;\; and \; E_p{}' = \frac{1}{2}\gamma k_p (d_o)^2$$

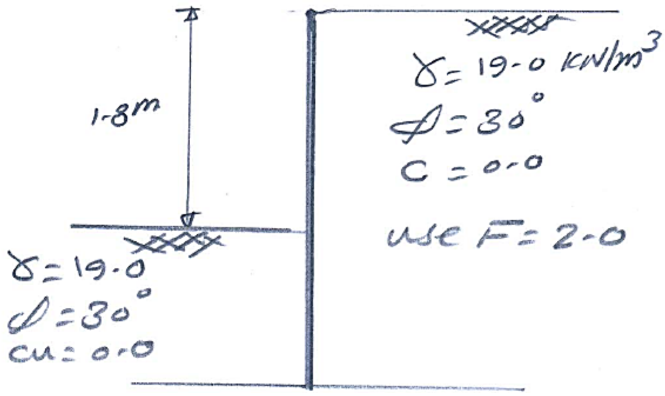
5. Calculate the maximum bending moment at depth (do).
6. Calculate the section modulus of the sheet pile (S) from:

$$S\left(m^3/_m\right) = \frac{Maximum\ Moment}{allawable\ flextural\ stress\ in\ sheet\ pile} = \frac{M_{max}}{\sigma_{all}}$$

4.4.4.1.1 EXAMPLE NO. 1:

For sheet pile shown that penetrated in sandy soil, find:

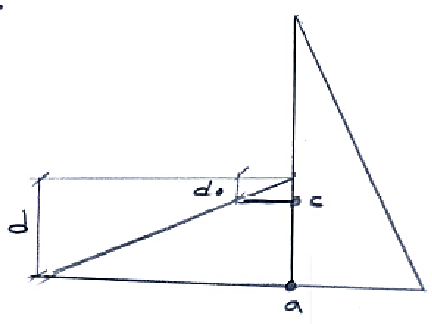
1. Total depth of penetration
2. Maximum moment



solution:-

① $Ka = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30}{1+\sin 30} = \frac{1}{3}$

$Kp = 3.0$



$\Sigma M_a = 0 \Rightarrow \frac{E_P}{F} * \frac{d}{3} = E_a * \frac{(1.8+d)}{3}$

$\Rightarrow \frac{E_P}{2} * d = E_a * (1.8+d)$

$\Rightarrow \frac{1}{2} * d * \left[\frac{1}{2} * 19 * 3 * d^2\right] = \frac{1}{2} * 19 * \frac{1}{3} * (1.8+d)^2 * (1.8+d)$

solving and get $d = 2.76\ m$

$\therefore D = 1.2 \times 2.76 = 3.312m$ $\quad \therefore \boxed{D = 3.312^m}$

$\therefore$ Total depth of sheet pile $= 1.8 + 3.312 = 5.112^m$

② To find Mmax, it is required to find (d_o)

To find d_o, $\sum F_x = 0 \Rightarrow \frac{E_p'}{F} = E_a'$

$$E_p' = \frac{1}{2} * 19 * 3 * d_o^2$$

$$E_a' = \frac{1}{2} * 19 * \frac{1}{3} * (1.8 + d_o)^2$$

$$\therefore \frac{1}{2}\left[\frac{1}{2} * 19 * 3 * d_o^2\right] = \frac{1}{2} * 19 * \frac{1}{3} * (1.8 + d_o)^2$$

solving and get, $\boxed{d_o = 1.61\,m}$

Now the max. moment can be computed by taking moment at point c (distance d_o)

$$\therefore M_{max} = \frac{1}{2} * 19 * (1.61)^2 * \left(\frac{3}{2}\right) * \frac{1.61}{3} - \frac{1}{2} * 19 * \frac{1}{3} * (1.8 + 1.61)^2 * \frac{(1.8 + 1.61)}{3}$$

$$= 22.05 \text{ kN.m/m}$$

4.4.4.2 *FREE EARTH SUPPORT METHOD FOR ANCHORED SHEET PILE IN COHESIONLESS SOIL*

The solution for the anchored sheet pile is shown in Figure 4.4-7.

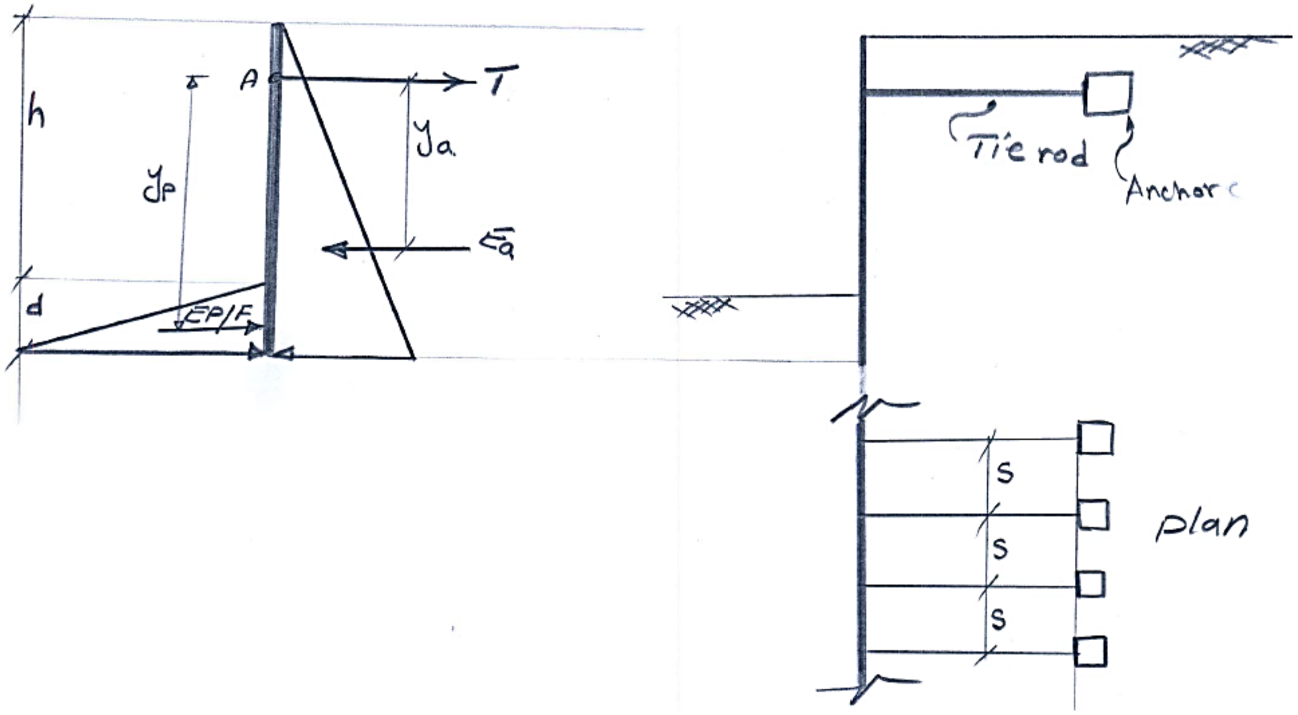


Figure 4.4-7: Free body diagram of the simplified solution for the free earth support anchored sheet pile in cohesionless soil (sandy soil)

From the free body diagram shown above, ⇒

1. To find the value of d, the moment at point A can be taken

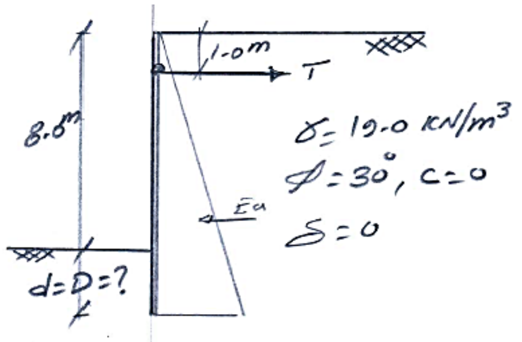
$$\sum M_A = 0 \Rightarrow E_a * y_a = \frac{E_P}{F} * y_P$$

2. The value of the force in the tie rod (T) can be obtained by:-

$$\sum F_H = 0 \Rightarrow T + \frac{E_P}{F} = E_a$$

4.4.4.2.1 EXAMPLE NO. 2:

For the anchored sheet pile shown below, find d and T.



Solution:-

$K_a = \frac{1}{3}$ & $K_P = 3.0$ and $\frac{K_P}{F} = 1.5$

$$E_a = \frac{1}{2}\gamma H^2 K_a = \frac{1}{2} * 19 * (8+d)^2 * \frac{1}{3}$$

$$\frac{E_P}{F} = \frac{1}{2}\gamma H^2 \frac{K_P}{F} = \frac{1}{2} * 19 * d^2 * 1.5$$

$$+\sum M_A = 0 \Rightarrow E_a * \left[(8+d)*\frac{2}{3} - 1\right] = \frac{E_P}{F} * \left(7 + \frac{2}{3}d\right)$$

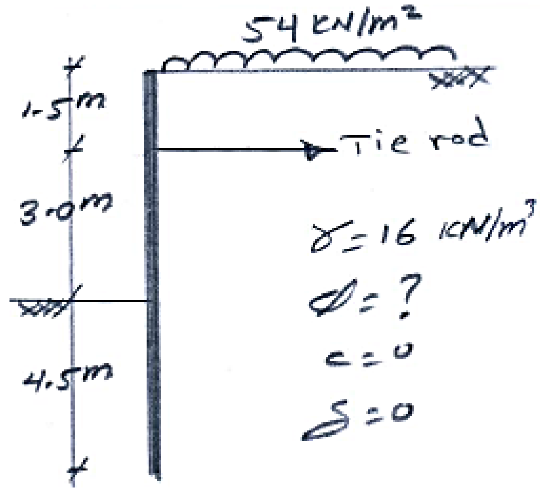
Solving and get $d = 5.5\,m$

$$+\sum F_H = 0 \Rightarrow \frac{1}{2} * 19 * (5.5)^2 * 1.5 + T = \frac{1}{2} * 19 * (8+5.5)^2 * \frac{1}{3}$$

$$\Rightarrow T = 146.1\ kN/m$$

4.4.4.2.2 EXAMPLE NO. 3:

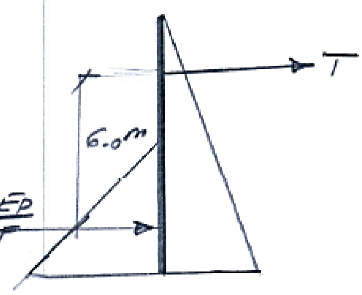
An anchored sheet pile is shown in figure below, for a factor of safety of 2.0 for the passive earth pressure, determine the force in the tie rods if these are spaced at 2.4 m, centers.



Solution:- $K_a = \frac{1}{K_P}$

$$\sum M_A = 0 \Rightarrow \frac{1}{2} * 16 * (4.5)^2 * \frac{K_P}{F} * 6 = \frac{1}{2} * 16 * (9)^2 * \frac{1}{K_P} * 4.5 + 54 * 9 * 3 * \frac{1}{K_P}$$

$$\Rightarrow K_P = 3.0 \quad \therefore \boxed{\phi = 30^\circ}$$

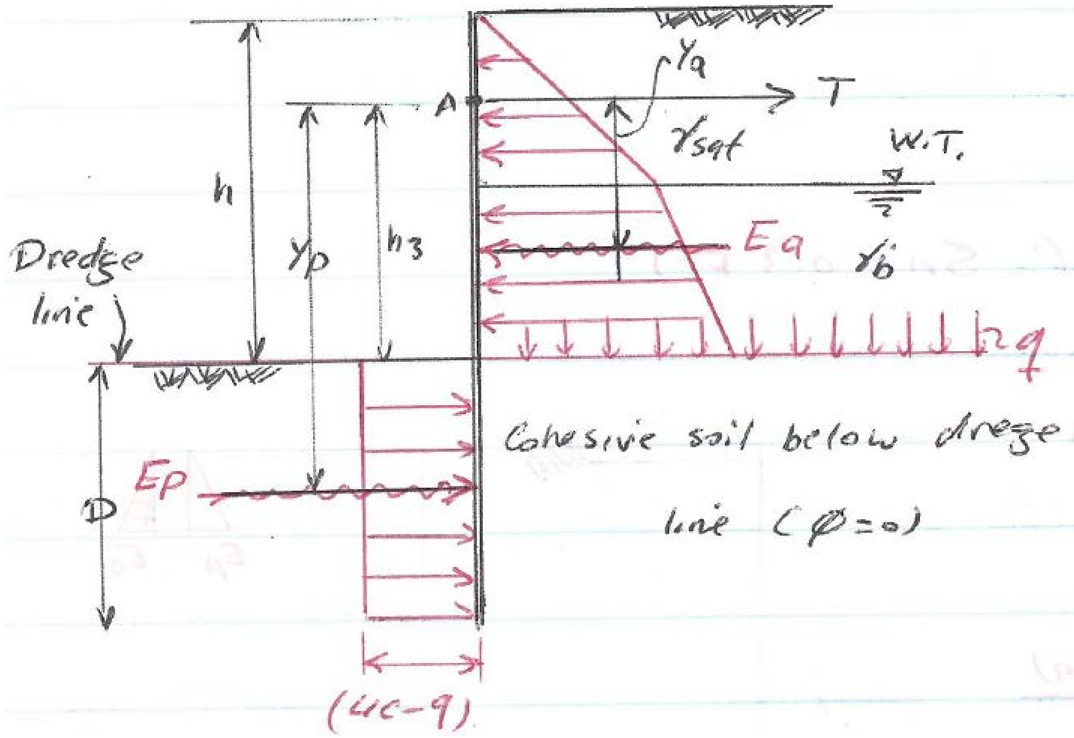


$$\sum F_x = 0 \Rightarrow \frac{1}{2} * 16 * (4.5)^2 * \frac{3}{2} + T = \frac{1}{2} * 16 * 9^2 * \frac{1}{3} + 54 * 9 * \frac{1}{3}$$

$$\therefore T = 135 \text{ kN/m}$$

$$\therefore \text{Total force} = 135 * 2.4 = 324 \text{ kN}$$

4.4.4.3 FREE EARTH SUPPORT METHOD FOR THE ANCHORED SHEET PILE IN COHESIVE SOIL

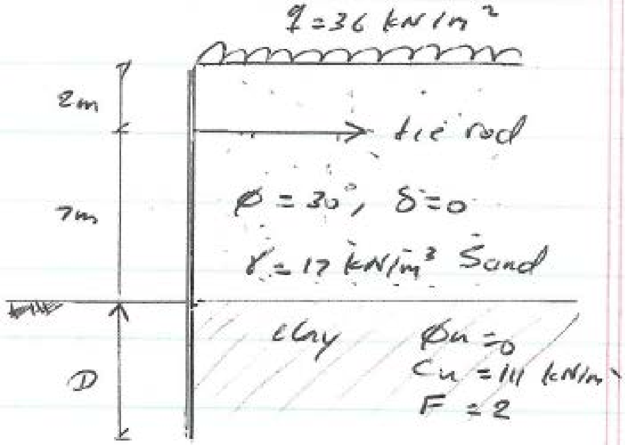


4.4.4.3.1 *EXAMPLE NO. 4:*

For the anchored sheet pile shown, find:

1. The depth of embedment, D.
2. The force in the tie rods, if these are spaced at 2.0m centers.
3. The cross-sectional area of the rod, if the allowable stress is 164400 kPa.

Solution:



$E_{a1} = 36 * \frac{1}{3} * 9 = 108$ kN/m

$E_{a2} = \frac{1}{2}(17)(9)^2 \frac{1}{3} = 229.5$ kN/m

$\frac{4c}{F} - q = \frac{(4)(111)}{2} - (17*9+36) = 33$

$\Sigma M_A = 0$

$33D(7 + \frac{D}{2}) = 108 * 2.5 + 229.5 * 4$

$231D + 16.5D^2 = 1188$

$D^2 + 14D - 72 = 0 \Rightarrow (D-4)(D+18) = 0$

$\therefore D = 4m$

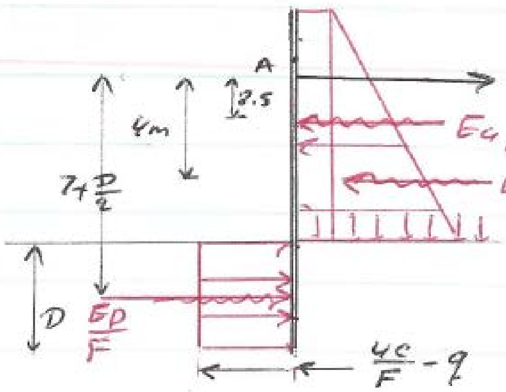
b) $\Sigma F_x = 0$

$T = 108 + 229.5 - 33 * 4$

$= 205.5$ kN/m

∴ The force $= 205.5 * 2$

$= 411$ kN



c) The cross-sectional area $= \frac{T}{F_{all}}$

$\therefore A = \frac{411}{164400} = 2.5E-3\ m^2$

$= 25\ cm^2$

SLOPE STABILITY 5

5.1 INTRODUCTION

An exposed ground surface that stands at an angle with the horizontal is called an unrestrained slope. The slope can be natural or man-made. It can fail in various modes.

5.2 CLASSIFICATION OF SLOPE FAILURE

1. **Fall.** This is the detachment of soil and/or rock fragments that fall down a slope (Figure 5.2-1).
2. **Topple.** This is a forward rotation of soil and/or rock mass about an axis below the center of gravity of mass being displaced (Figure 5.2-2).
3. **Slide.** This is the downward movement of a soil mass occurring on a surface of rupture (**Figure 5.2-3**
4. **Spread.** This is a form of slide (**Figure 5.2-4**) by translation. It occurs by "sudden movement of water-bearing seams of sands or silts overlain by clays or loaded by fills".
5. **Flow.** This is a downward movement of soil mass similar to a viscous fluid (**Figure 5.2-5**).

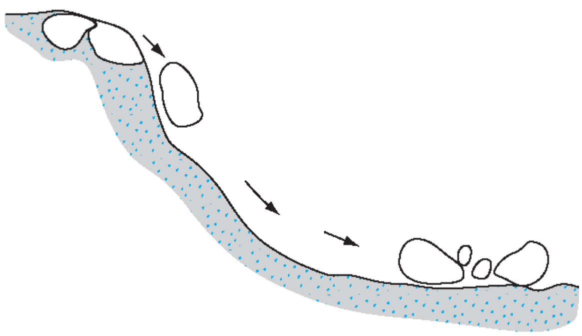


Figure 5.2-1: "Fall" type of landslide

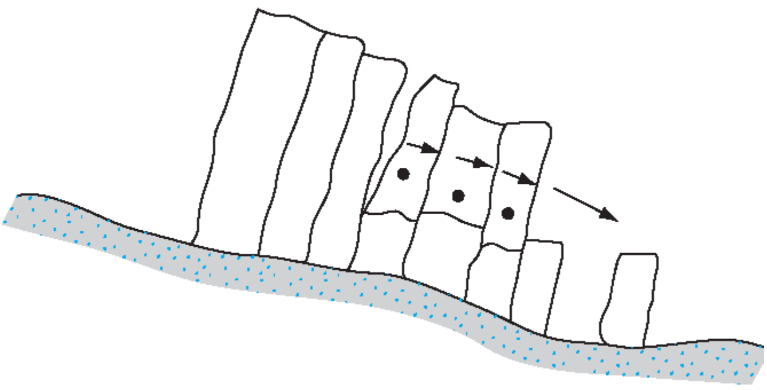


Figure 5.2-2: Slope failure by "toppling"

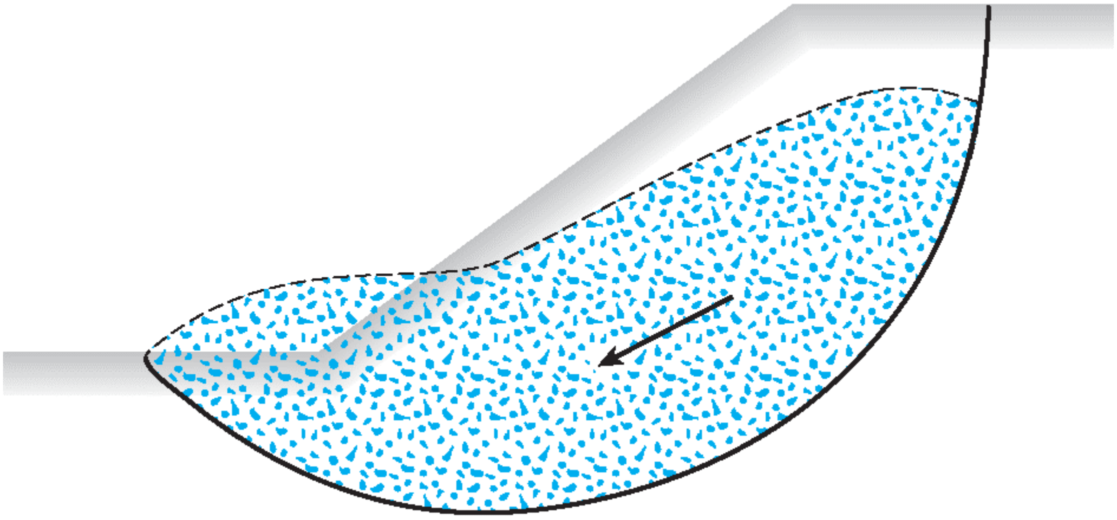


Figure 5.2-3: Slope stability failure by Sliding

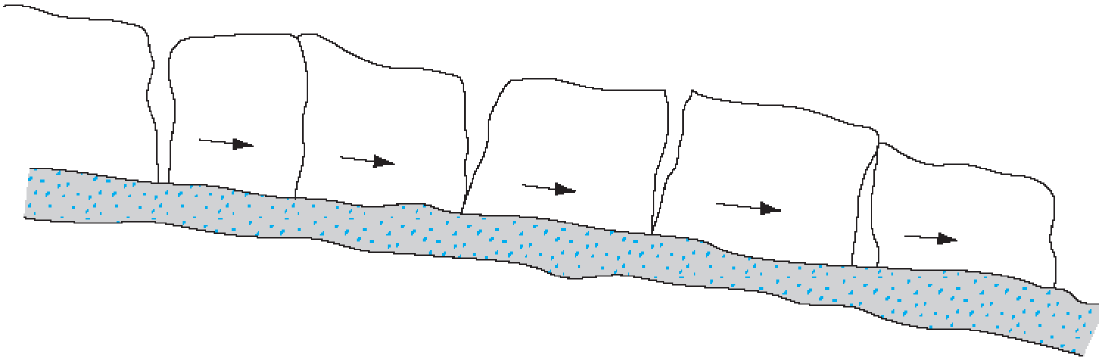


Figure 5.2-4: Slope stability failure by Spreading

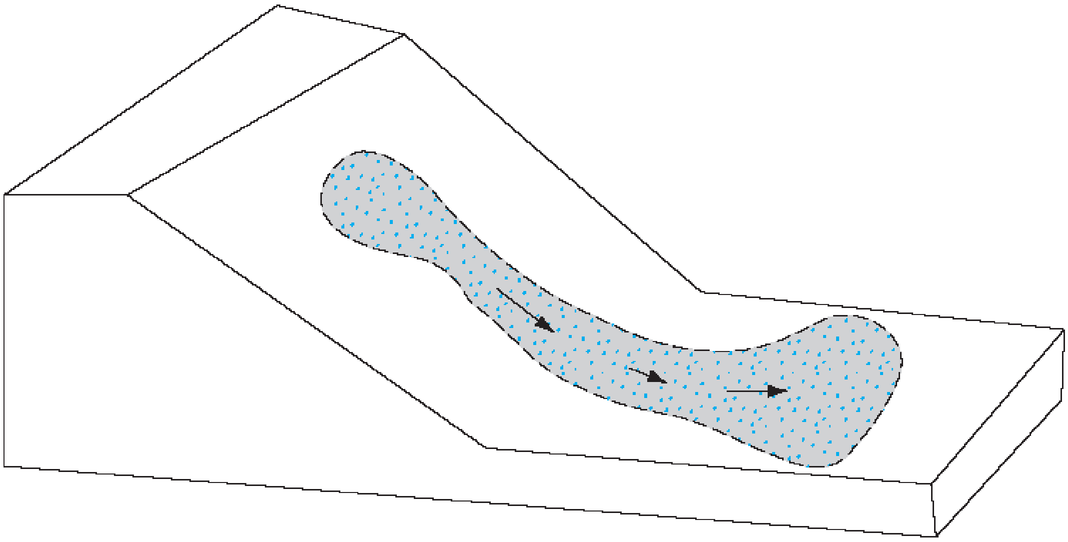


Figure 5.2-5: Slope stability failure by Flowing

5.3 FACTOR OF SAFETY CONCEPT

Safety Factor:

$$F_s = \frac{\text{shear strength}}{\text{shear stress}} = \frac{S}{\tau} = \boxed{\frac{\bar{c} + \bar{\sigma}\tan\bar{\phi}}{c^* + \bar{\sigma}\tan\phi^*} = F_s}$$

$$F_c = \frac{\bar{c}}{c^*} \quad (\text{cohesive}) \quad , \quad F_\phi = \frac{\tan\bar{\phi}}{\tan\phi^*} \quad (\text{friction})$$

ملاحظة: في حالة تساوي (أي منهما) أي اثنين من F_c او F_ϕ او F_s ، فان ثالثهما يساويهما.

proof:

$$F_s = \frac{\bar{c} + \bar{\sigma}\tan\bar{\phi}}{c^* + \bar{\sigma}\tan\phi^*} \quad , \quad F_s c^* + F_s \bar{\sigma}\tan\phi^* = \bar{c} + \bar{\sigma}\tan\bar{\phi}$$

$$\frac{F_s c^*}{\bar{\sigma}\tan\phi^*} + F_s = \frac{\bar{c}}{\bar{\sigma}\tan\phi^*} + \frac{\bar{\sigma}}{\bar{\sigma}}\underbrace{\left(\frac{\tan\bar{\phi}}{\tan\phi^*}\right)}_{F_\phi} \quad , \text{ and if } F_s = F_\phi$$

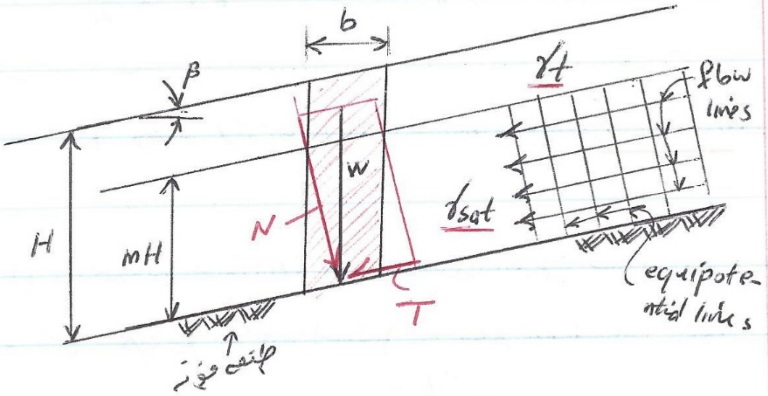
$$\therefore \frac{F_s c^*}{\bar{\sigma}\tan\phi^*} = \frac{\bar{c}}{\bar{\sigma}\tan\phi^*} \quad , \text{ or } \quad F_s = \frac{\bar{c}}{c^*} = F_c$$

5.4 INFINITE SLOPE

Infinite Slopes: المنحدرات غير المحدودة :

$$F_s = \frac{\bar{c} + \bar{\sigma}\tan\bar{\phi}}{c^* + \bar{\sigma}\tan\phi^*}$$

$0 \leq m \leq 1$



$$F_s = \frac{\bar{c} + (\sigma - u)\tan\bar{\phi}}{\tau} \quad \text{--- (1)}$$

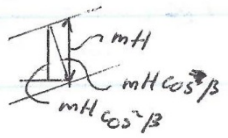
$$\sigma = \frac{N}{area} = \frac{W\cos\beta}{b/\cos\beta}$$

$$W = (H - mH)\gamma_t\, b + mH\gamma_{sat}\cdot b$$

$$\sigma = [(1-m)\gamma_t + m\gamma_{sat}]\, H\cos^2\beta \quad \text{--- (2)}$$

$$\tau = \frac{T}{area} = \frac{W\sin\beta}{b/\cos\beta}, \quad \tau = [(1-m)\gamma_t + m\gamma_{sat}]\, H\cos\beta\sin\beta \quad \text{--- (3)}$$

$$u = h_w\gamma_w = mH\cos^2\beta\,\gamma_w \quad \text{--- (4)}$$



للحصول على عامل الامان نعوض عن σ من معادلة رقم ② بمعادلة رقم ① ، ونجد قيمة τ من معادلة رقم ③ بمعادلة رقم ① ، ونجد قيمة u من معادلة رقم ④ ونعوض بمعادلة رقم ① ،

If $m = 0$ (no seepage)

$$F_s = \frac{\bar{c} + (\gamma_t H\cos^2\beta)\tan\phi}{\gamma_t H\cos\beta\sin\beta} \quad \text{------ (5)}$$

In Eq. 5, if $\phi = 0$ (clay) $\Rightarrow$

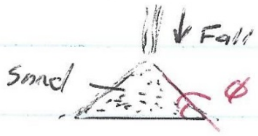
$$F_s = F_c = \frac{c}{\gamma_t H \cos\beta \sin\beta} \quad \text{---} \quad (6)$$

Conclusion (important) $F_s \propto \frac{1}{H}$, $F_s \frac{1}{\propto} H$

In Eq. 5, if $c = 0$ (sand) $\Rightarrow$

$$F_s = F_\phi = \frac{\tan\phi}{\tan\beta} \quad \text{---} \quad (7)$$

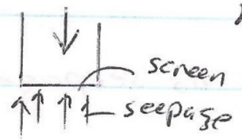
Conclusion : الـ (Fs) مستقل ، اي لا يعتمد على قيمة (H) لكن النقطة المهمة هو ان (β)
لا يمكن ان تكون اكبر من (ϕ) ، لانه عندما تصل β=ϕ يحصل الفشل . ولعل صورة جزءا اكثر
ان نعرف (ϕ) بصورة تقريبية من ترك الرمل ينهمر ، الآتي :



In Eq. 1, if $m = 1$ and $c = 0$ (Sand, seepage in all the layer)

$$F_\phi = \frac{\bar{\gamma}}{\gamma_{sat}} \frac{\tan\phi}{\tan\beta} \quad \text{-----} \quad (8)$$

بما ان $\bar{\gamma}$ تقريباً مساوي لـ $(\frac{1}{2}\gamma_{sat}) \Leftarrow (\bar{\gamma} \approx \frac{1}{2}\gamma_{sat})$ معنى ذلك ، عندما يكون هناك
(فاذا فرضنا مثلا) $(F_s = 2)$ ↑ويكون هناك seepage ، يقل الى $(F_s = 1)$ ويحصل الفشل .
نلاحظ ان تأثير الماء لم يدخل بحساب الـ Fs وجاء ذلك في المعادلات السابقة
γ_t , γ_b , seep. force



Examples about infinite slopes:

Ex 1; For the Taylor's Stability number for infinite slope

$$\frac{c}{F_c \gamma H} = \cos^2\beta\left(\tan\beta - \frac{\tan\phi}{F_\phi}\right)$$, find Fs, if c = 20 kN/m²

and $\beta = \phi = 45°$, $\gamma = 20$ kN/m³, H = 2m

Sol.

$$\frac{20}{F_c * 20 * 2} = \cos^2(45)\left(\tan 45 - \frac{\tan 45}{F_\phi}\right)$$

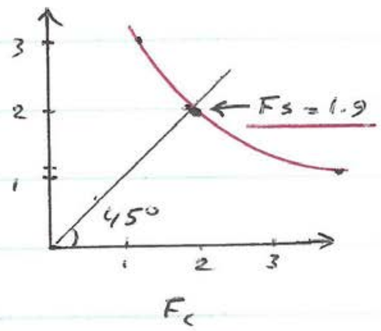
$$\frac{1}{2F_c} = \frac{1}{2}\left(1 - \frac{1}{F_\phi}\right) \Rightarrow \frac{1}{F_c} = 1 - \frac{1}{F_\phi}$$, for $F_c = F_\phi$, $F_c = F_s = 2$

Ex 2: $\gamma_t = 20$ kN/m³, $\phi = 25°$, c = 30 kN/m², H = 6 m and $\beta = 25°$. Determine the safety factor for infinite slope

Solution:

$$\frac{c}{F_c \gamma H} = \cos^2\beta\left(\tan\beta - \frac{\tan\phi}{F_\phi}\right)$$

$$\frac{1}{F_c} = 1.5321 - \frac{1.532}{F_\phi} \quad \text{eq.}$$



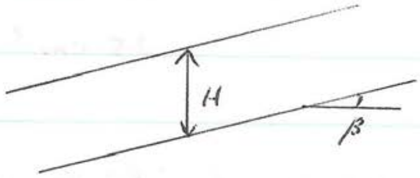
or $$F_s = \frac{s}{\tau} = \frac{c + \sigma' \tan\phi}{\gamma_t H \cos\beta \sin\beta} = \frac{c + \gamma_t H \cos^2\beta \tan\phi}{\gamma_t H \cos\beta \sin\beta} = 1.89$$

Exampe 3:

$\phi = 30°$, $c = 50\ kN/m^2$, $\beta = 30°$, $\gamma_{sat} = 19.81\ kN/m^3$

$\gamma_w = 9.81\ kN/m^3$, $H = 4m$

If $F\phi = 1.86$, determini F_c



Sol.

$\tau = c^* + (\sigma - u)\tan\phi^*$

$\tau = \frac{c}{F_c} + (\sigma - u)\frac{\tan\phi}{\phi}$ ... ①, $\sigma = \gamma_{sat} H \cos^2\beta$

$= 19.81 * 4 \cos^2 30 = 59.43\ kN/m^2$

and $u = \gamma_w H \cos^2\beta = 9.81 * 4 \cos^2 30$

$= 29.43\ kN/m^2$

$\tau = \gamma_{sat} H \cos\beta \sin\beta$

$= 19.81 * 4 \cos 30 \sin 30 = 34.31\ kN/m^2$

sub in eq. ①

$34.31 = \frac{50}{F_c} + (59.43 - 29.43)\frac{\tan 30}{F\phi}$

@ $F\phi = 1.86 \Rightarrow F_c = \frac{50}{25} = 2$

5.5 FINITE SLOPE

($\phi_u = 0$ - Method)

المنحدرات في التربة الطينية ،
(في حالة التربة عير المبزولة)

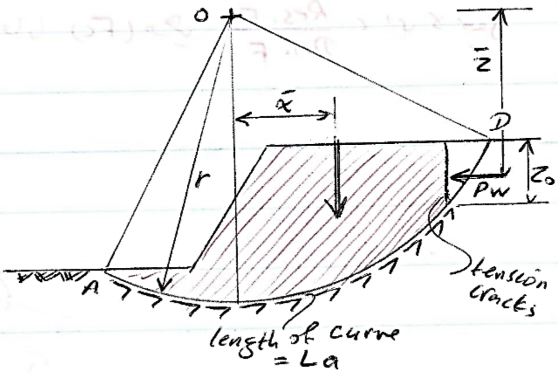
Driving Moment $= W \cdot \bar{x}$

Resisting M. $= c^* L_a \cdot r$

$c^* L_a = W\bar{x} + P_w \bar{z}$,

$\frac{c}{F_c} L_a = W\bar{x} + P_w \bar{z}$

$$F_c = \frac{c L_a r}{W\bar{x} + P_w \cdot \bar{z}}$$



if c varies ;

$$F_c = \frac{r \sum c_i l_i}{W\bar{x} + P_w \bar{z}}$$

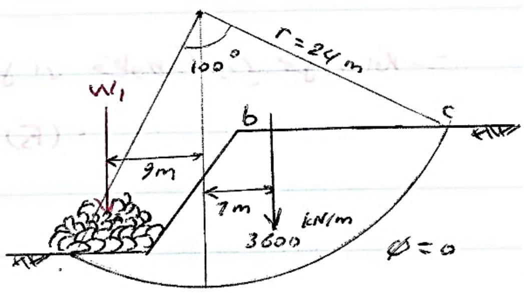
مثال :- F = 1.2 ، المطلوب زيادة معامل الامان الى (1.5) باضافة صخر او حجر
وزنه (W1) ، جد W1 .

Sol,

$$F_c = \frac{c\, L_a . r}{W\, \bar{x}} = 1.2$$

$$\frac{c * 41.89^{*} * 24}{3600 * 7} = 1.2$$

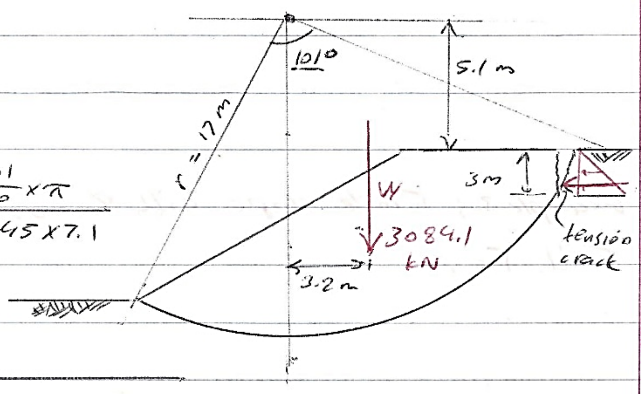
$*\ L_a = \frac{100}{360}\, 2\pi * 24 = 41.89$ m



$c = 30$ kN/m², $F_c = \frac{c\, L_a\, r}{W\bar{x} - W_1 \bar{x}_1}$ *

$$1.5 = \frac{41.89 * 30 * 24}{3600 * 7 - W_1 * 9} \Rightarrow W_1 = 565.8 \text{ kN}$$

Example; Finite slope with tension crack; filled with water; $\phi_u = 0$, $C_u = 30\ kN/m^2$, and $\gamma_w = 10\ kN/m^3$. Find the Factor of Safety.



Solution;

$P_W = \frac{1}{2}(10)(3)^2 = 45\ kN/m$

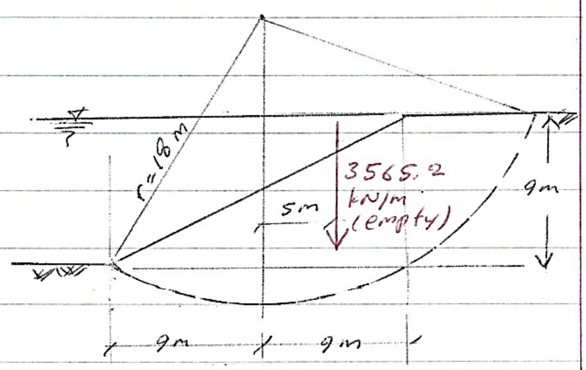
$$F_S = \frac{C \cdot L_a \cdot r}{W\bar{x} + P_W\bar{z}} = \frac{30 \times 17 \times 17 \times \frac{101}{180} \times \pi}{3084.1 \times 3.2 + 45 \times 7.1}$$

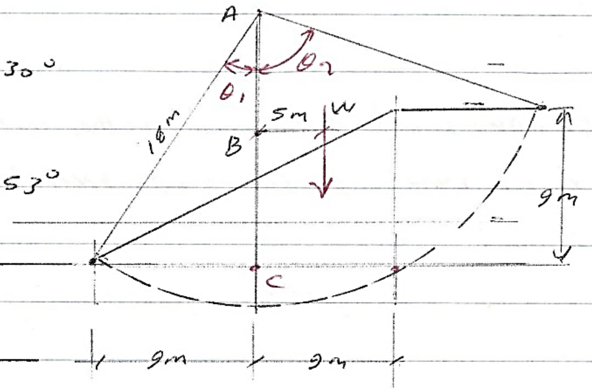
$= 1.5$ Ans.

Example 2; Bank of a canal; $\phi_u = 0°$, $C_u = 48\ kN/m^2$ and $\gamma_{sat} = 2\gamma_w$ ($\gamma_{sat} = 2\gamma_w$). Find:

a- Fs if the water in the canal is level with the top of the bank, and.

b- Fs if the canal is empty.





$\theta_1 = \sin^{-1}\frac{9}{18} = 30°$, $\overline{AC} = 18\cos 30°$

and $\overline{AB} = 18\cos(30°) - 9$

$\cos\theta_2 = \frac{18\cos 30° - 9}{18}$, $\theta_2 = 68.53°$

$\theta = \theta_1 + \theta_2 \Rightarrow \theta = 98.53°$

$$F_S = \frac{C \cdot L_a \cdot r}{W\bar{x}} = \frac{48 \times 18 \times 18 \times \frac{98.53}{180}\pi}{3565.2 \times 5}$$

$\therefore F_S = 1.5$ the canal is empty,

since $\gamma_b = 2\gamma_w - \gamma_w = \gamma_w = \frac{1}{2}\gamma_{sat}$, Sub.

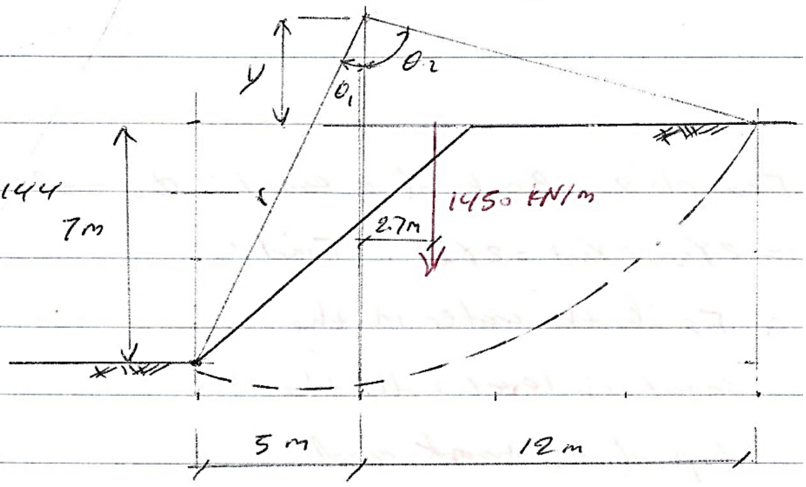
$\therefore F_S = 3$ if the canal is filled with water*,

$\because F_{S_{em.}} < F_{S_{fill}}$, $\therefore$ empty is the dangerous state

Example 3; Finite slope with $\phi_u = 0$, and $C_u = 22.13$ kN/m²

Find Fs,

Solution;



$(y+7)^2 + (5)^2 = y^2 + (12)^2$

$y^2 + 14y + 49 + 25 = y^2 + 144$

$14y = 144 - 74 = 70$

$\therefore y = 5m$

$r^2 = (5+7)^2 + (5)^2$

$\therefore r = 13m$

$\theta_1 = \sin^{-1}\frac{5}{13}$, $\theta_2 = \cos^{-1}\frac{5}{13}$, $\theta = \theta_1 + \theta_2 = 90°$

[illegible]

$$Fs = \frac{22.13 \times 13 \times \frac{1}{2}\pi}{1450 * 2.7} = 1.5 \quad \text{Ans.}$$

Bearing Capacity of Foundations

6

6.1 INTRODUCTION

- The term bearing capacity is related to the calculation of the potential strength of soil to the applied loads from the structure. It is related to the shallow foundations where $B \geq D$. In general, the foundations for any structure must satisfy two requirements:
    - They must have adequate shear strength to support the structure.
    - The settlement of the structure should be kept within allowable limits.

- Bearing capacity is the ability of foundation soil to hold the forces from the superstructure without undergoing shear failure or excessive settlement. In other words, the limiting shear resistance beyond which the soil collapses or becomes unstable is called the ultimate bearing capacity or the load per unit area of the foundation at which shear failure in soil occurs.

- Foundation soil is that portion of ground which is subjected to additional stresses when foundation and superstructure are constructed on the ground. Figure 6.1-1 illustrates the main terms related to the bearing capacity of shallow footings.

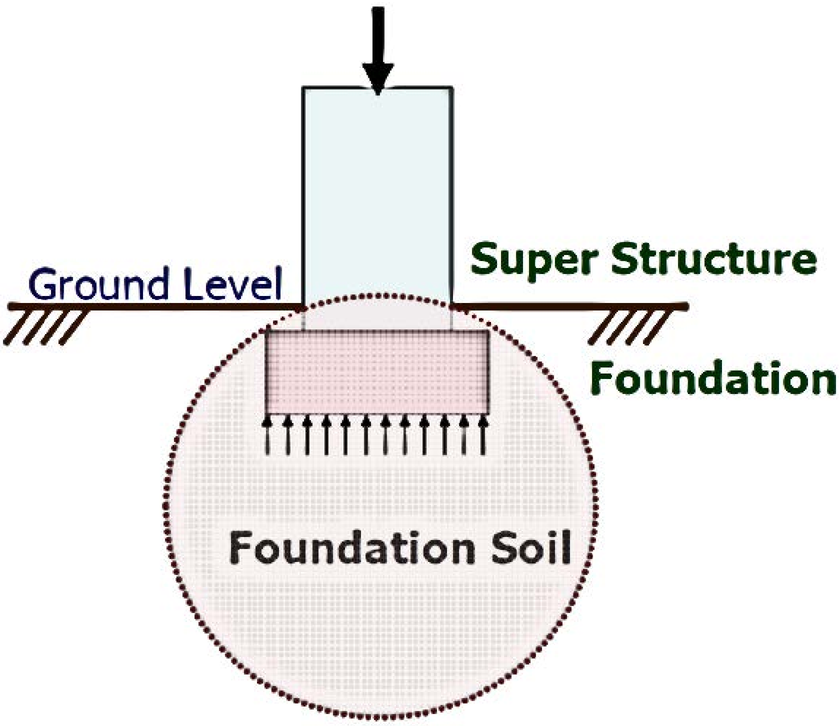


Figure 6.1-1: Main components of a structure including soil

- The procedures related to the design and analysis of shallow foundation form geotechnical and structural point of view can be illustrated in Figure 6.1-2.

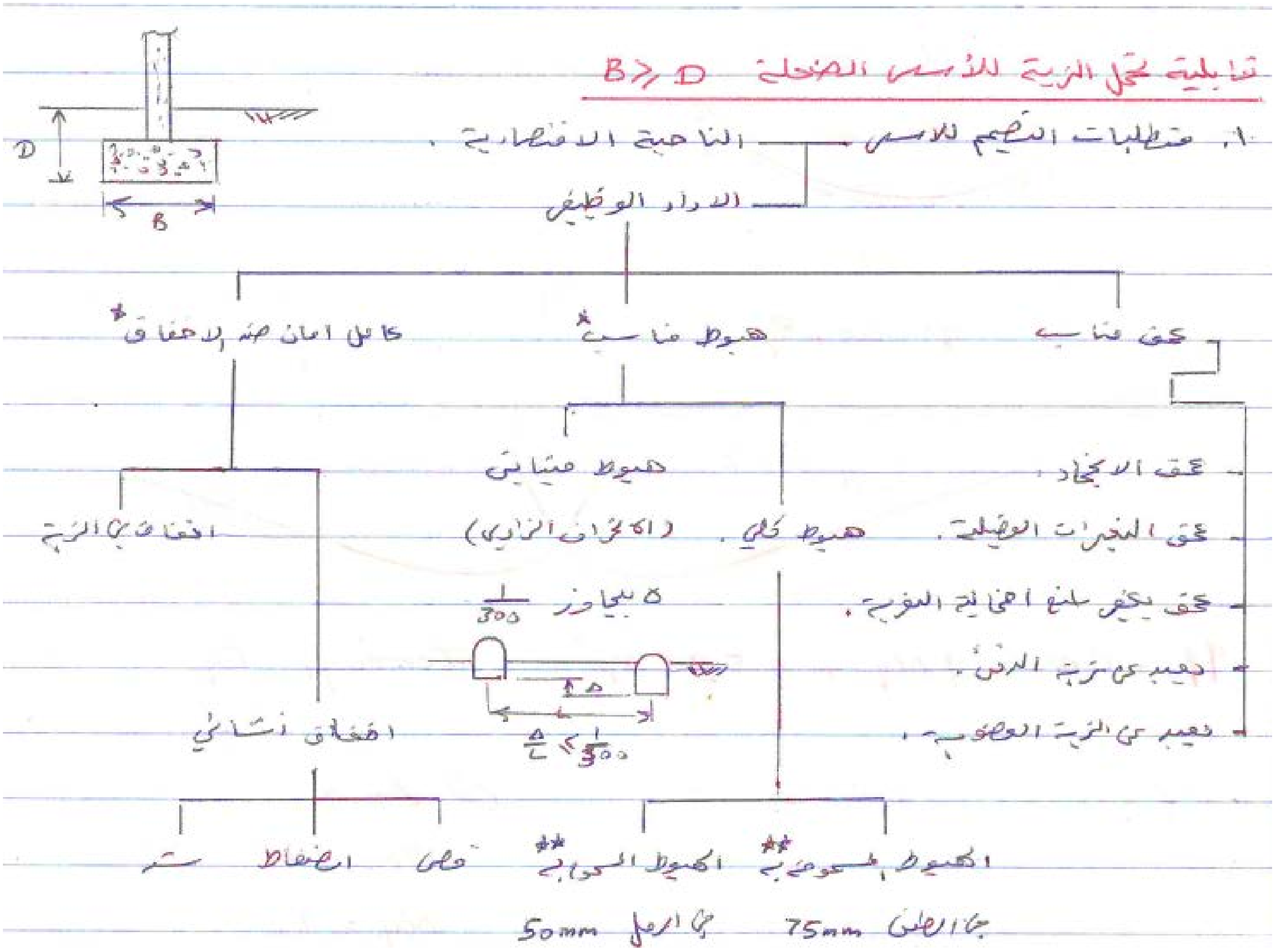


Figure 6.1-2: Flow chart explains the requirements for the analysis and design of shallow foundations

6.2 MODES OF SHEAR FAILURE IN FOUNDATIONS

- ***General shear failure***: If the soil is generally dense or stiff, the settlement of the footing that precedes the ultimate shear failure is relatively small. It is called general shear failure as shown in Figure 6.2-1 (a) and curve 1 of the load-settlement curves.

- ***Local shear failure***: If the soil is loose or soft, then a large settlement precedes the shear failure as shown in Figure 6.2-1(b) and curve 2, of the load-settlement curve.

- ***Punching shear failure***: In some structures like liquid storage tanks and rafts supported on loose or soft soils, there could be a base shear failure in which the base/foundation undergoes a punching failure as shown in Figure 6.2-1 (c) and curve 3 of the load-settlement curve.

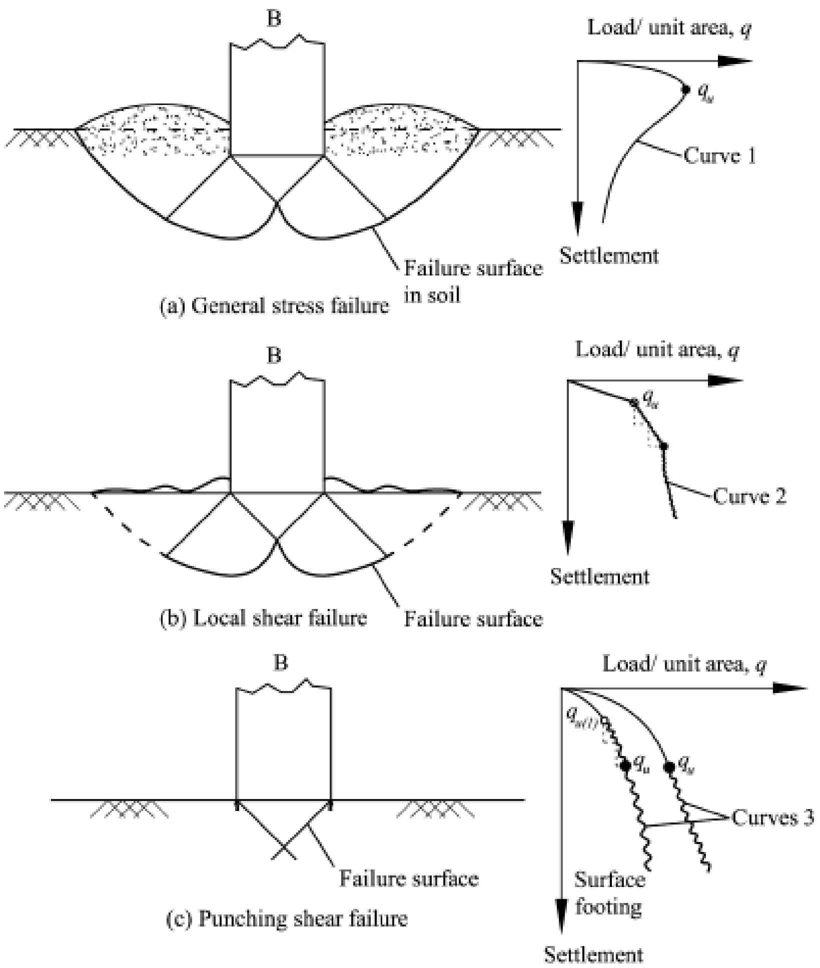


Figure 6.2-1: Sketches of bearing capacity failures in soil

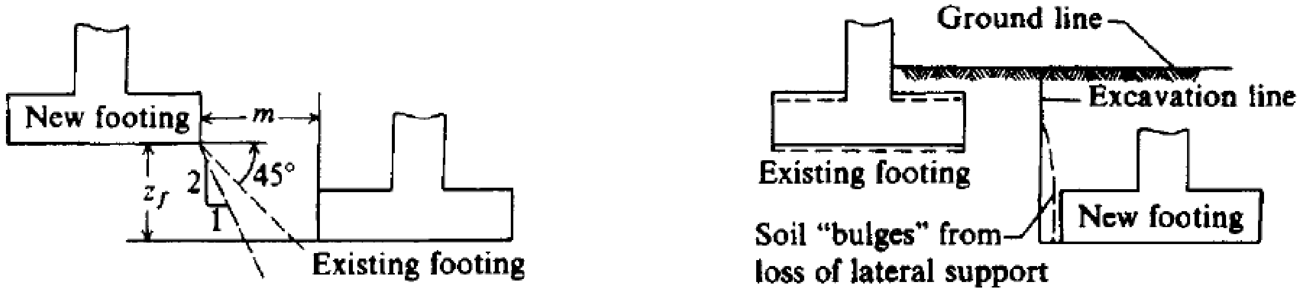
6.3 FACTORS TO CONSIDER IN FOUNDATION DESIGN

6.3.1 FOOTING DEPTH AND SPACING

- The frost line
- Zones of high volume change due to moisture fluctuations
- Topsoil or organic material
- Peat and muck
- Unconsolidated material such as abandoned (or closed) garbage dumps and similar filled in areas.

When footings are to be placed adjacent to an existing structure, as indicated in Figure 6.3-1, the line from the base of the new footing to the bottom edge of the existing footing should be 45° or less with the horizontal plane. From this requirement, it follows that the distance m of Figure 6.3-1 (a) should be greater than the difference in elevation of the two footings, z_f. This approximation should produce very conservative pressures in that zone where there is a contribution from more than one footing.

Conversely, Figure 6.3-1 (b) indicates that if the new footing is lower than the existing footing, there is a possibility that the soil may flow laterally from beneath the existing footing. This may increase the amount of excavation somewhat but, more importantly, may result in settlement cracks in the existing building. This problem is not easy to analyze. More details for this conditions can be followed in Bowles (1996).



(*a*) An approximation for the spacing of footings to avoid interference between old and new footings. If the "new" footing is in the relative position of the "existing" footing of this figure, interchange the words "existing" and "new." Make $m > z_f$.

(*b*) Possible settlement of "existing" footing because of loss of lateral support of soil wedge beneath existing footing.

Figure 6.3-1: Location considerations for spread footings.

6.3.2 OTHER FACTORS AFFECT THE PERFORMANCE AND DESIGN OF SHALLOW FOUNDATION*

- Displaced soil effects
- Net versus gross soil pressure: design soil pressures
- Erosion problems for structures adjacent to flowing water
- Corrosion protection
- Water table fluctuation
- Foundations in sand and silt deposits
- Foundations on loess and other collapsible soils
- Foundations on unsaturated soils subject to volume change with change in water content
- Foundations on clays and clayey silts
- Foundations on residual soils
- Foundations on sanitary landfill sites
- Frost depth and foundations on permafrost
- Environmental considerations

Detailed discussion for each factor can be found in Bowles (1996)

6.4 TYPES OF SHALLOW FOOTINGS

- Foundation is that part of the structure which is in direct contact with soil.
- Foundation transfers the forces and moments from the super structure to the soil below such that the stresses in soil are within permissible limits and it provides stability against sliding and overturning to the super structure.
- Foundation is a transition between the super structure and foundation soil

6.4.1 STRIP FOOTING

- A strip footing (Figure 6.4-1) is provided for a load-bearing wall. A strip footing is also known as continuous footing

- A strip footing is also provided for a row of columns which are so closely spaced that their spread footings overlap or nearly touch each other. In such a case, it is more economical to provide a strip footing than to provide several spread footings in one line.

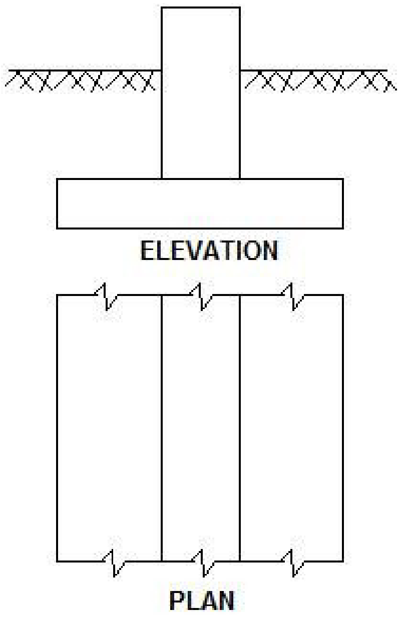


Figure 6.4-1: Strip Footing

6.4.2 SPREAD OR ISOLATED FOOTING:

- A spread footing (or isolated or pad) footing (Figure 6.4-2) is provided to support an individual column.
- A spread footing is circular, square, or rectangular section of uniform thickness. Sometimes, it is stepped to spread the load over a large area.

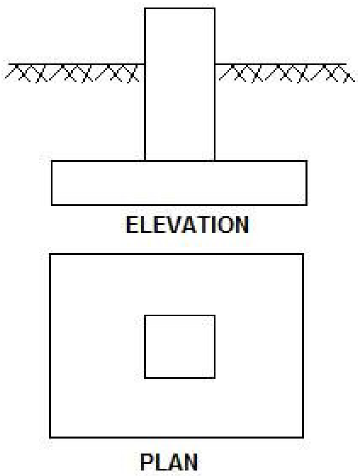


Figure 6.4-2: Isolated Footing

6.4.3 COMBINED FOOTING:

- A combined footing (Figure 6.4-3) supports two or more columns. A combined footing may be rectangular or trapezoidal in plan.
- A combined footing is also provided when the property line is so close to one column that a spread footing would be eccentrically loaded when kept entirely within the property line.

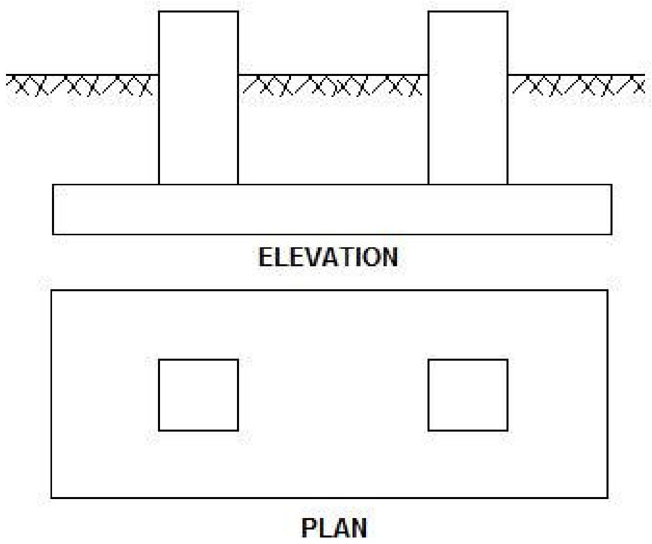


Figure 6.4-3: Combined Footing

6.4.4 STRAP OR CANTILEVER FOOTING:

- A strap (or cantilever) footing (Figure 6.4-4) consists of two isolated footings connected with a structural strap or a lever. The strap connects the two footings such that they behave as one unit.
- The strap is designed as a rigid beam. A strap footing is more economical than a combined footing when the allowable soil pressure is relatively high and the distance between the columns is large.

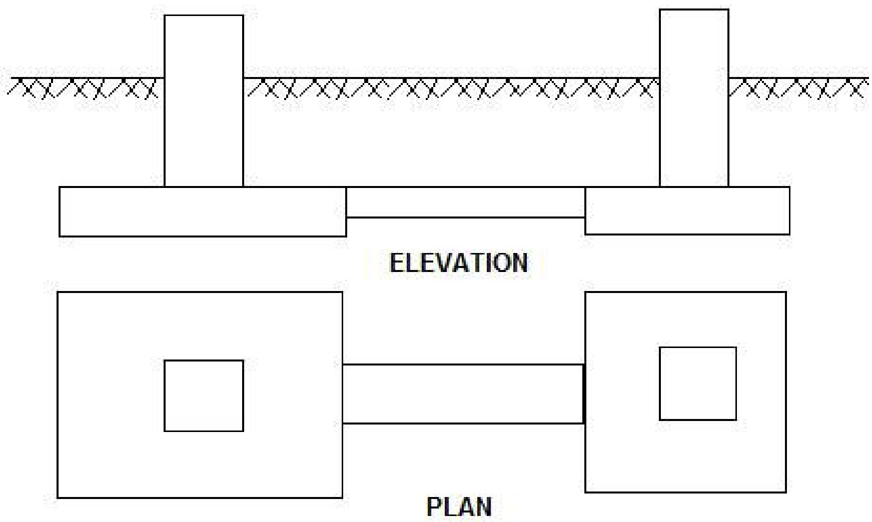


Figure 6.4-4: Strap Footing

6.4.5 MAT OR RAFT FOUNDATIONS:

- A mat or raft foundation (Figure 6.4-5) is a large slab supporting a number of columns and walls under the entire structure or a large part of the structure.
- A mat is required when the allowable soil pressure is low or where the columns and walls are so close that individual footings would overlap or nearly touch each other.
- Mat foundations are useful in reducing the differential settlements on non-homogeneous soils or where there is a large variation in the loads on individual columns.

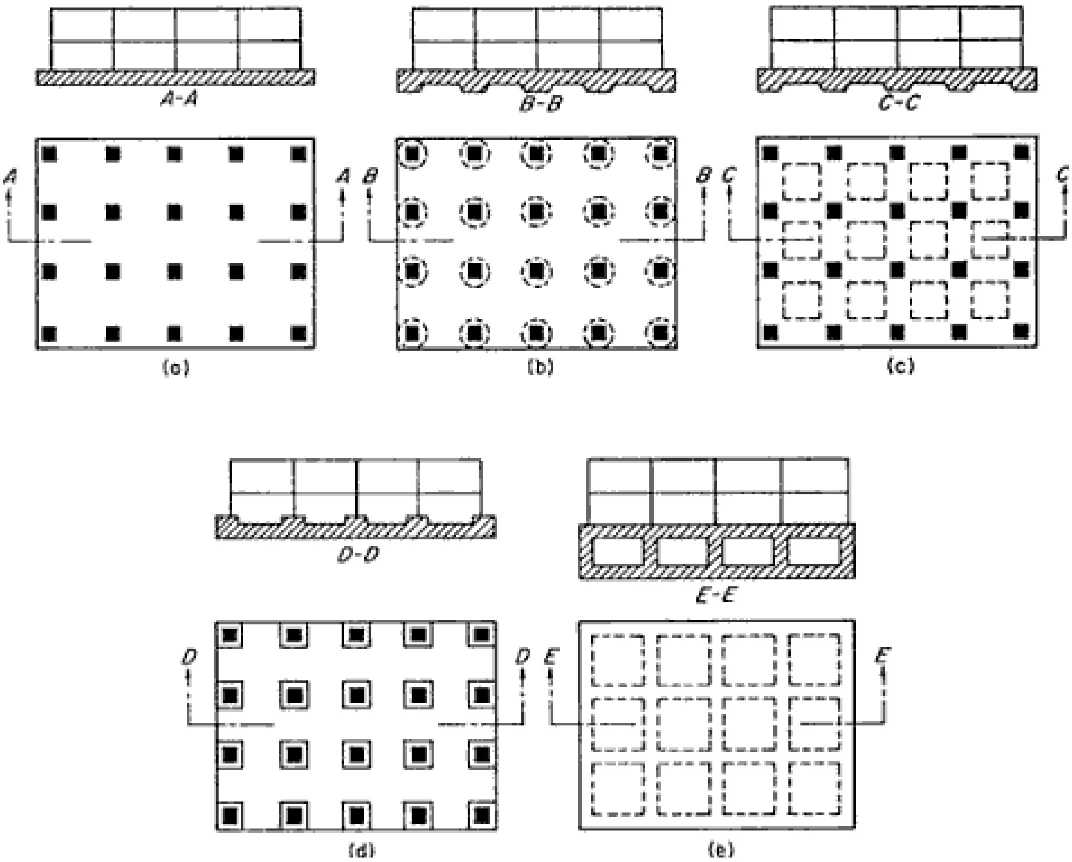


Figure 6.4-5: Types of Mat Foundations

6.5 TERZAGHI'S BEARING CAPACITY THEORY

6.5.1 ASSUMPTIONS AND EQUATION FOR STRIP FOOTING

Terzaghi (1943) was the first to present a comprehensive theory for the evaluation of the ultimate bearing capacity of rough shallow foundations. The assumptions of the bearing capacity are:

1. The soil is homogeneous and isotropic.
2. Mohr Coulombs Criteria represent the shear strength of soil.
3. The footing is of strip footing type with the rough base. It is a two-dimensional plane strain problem.
4. The elastic zone has straight boundaries inclined at an angle with respect to the horizontal.
5. Failure zone is not extended above, beyond the base of the footing. Shear resistance of soil above the base of the footing is neglected.
6. The method of superposition is valid.
7. Footing and ground are horizontal.
8. Limit equilibrium is reached simultaneously at all points.
9. The properties of foundation soil do not change during the shear failure.

Terzaghi suggested that for a continuous, or strip, foundation, the failure surface in soil at ultimate load may be assumed to be like that shown in Figure 6.5-1. The present analysis of the real behavior of soil under vertical load is illustrated in Figure 6.5-2.

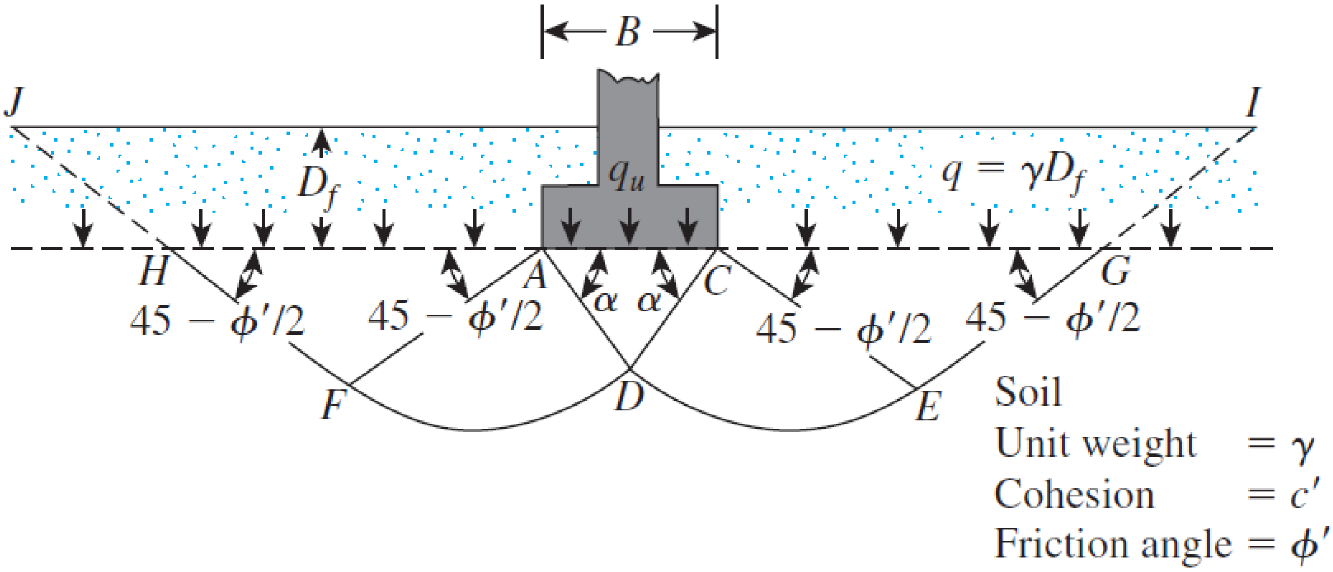


Figure 6.5-1: Bearing capacity failure in soil under a rough rigid continuous (strip) foundation

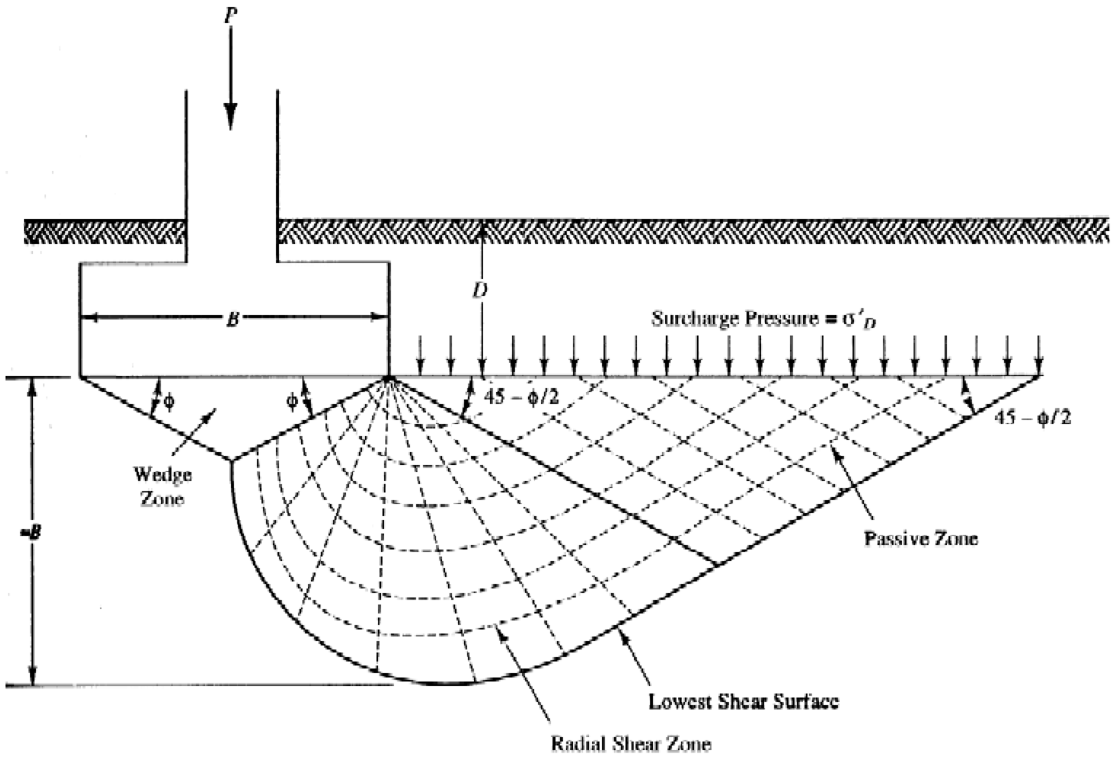


Figure 6.5-2: Terzaghi's concept of Footing with five distinct failure zones in foundation soil

By using equilibrium analysis, Terzaghi expressed the ultimate bearing capacity in the form the following equation for strip footing:

$$q_u = c' N_c + q\, N_q + \frac{1}{2}\gamma B N_\gamma$$

Where:

c':is the cohesion of soil

γ :is the unit weight of soil

q is the overburden pressure at the base of the foundation level=$\gamma\ D_f$

$N_c\ N_q\ and\ N_\gamma$ are the bearing capacity factors that are non-dimensional and are functions only of the soil friction angle $\emptyset'$, and can be expressed as:

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

$$N_c = \cot\emptyset' \,(N_q - 1)$$

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

The variations of the bearing capacity factors are given in Table 6.5-1.

Table 6.5-1: Terzaghi's Bearing Capacity Factors

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				

H.W.: Write a bearing capacity equation for the case of local and general shear failure.

For square and circular footing, Terzaghi gave the following equations:

$q_u = 1.3\, c'N_c + q\, N_q + 0.4\gamma B N_\gamma$ **For square footing:**

$q_u = 1.3\, c'N_c + q\, N_q + 0.3\gamma B N_\gamma$ **For circular footing**

6.5.2 EXAMPLE 6.1:

A strip footing with 2.4 m width is to be constructed at a depth of 3.0 m. Find the ultimate bearing capacity for the footing if $c_u = 65.0\, kPa, \emptyset_u = 0.0, \gamma_t = 17.70\, kN/m^3$. Use $N_c = 10.0, N_q = 4.0\ and\ N_\gamma = 0.0$.

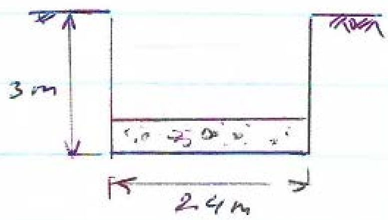
Solution:

$q_f = cN_c + qN_q + 0.5B\gamma N_\gamma$

$= (65)(10) + (17.7)(3)(4)$

$+ (0.5)(2.4)(17.7)(2)$

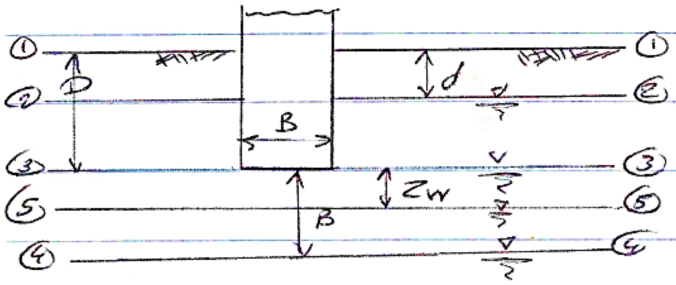
$\therefore q_f = 910\ kN/m^2$



6.5.3 EFFECT OF THE LOCATION OF WATER TABLE LEVEL

The ultimate bearing capacity, based on the assumption that the water table is located well below the foundation. However, if the water table is close to the foundation, some modifications of the bearing capacity equations will be necessary as shown below. It is important to state herein that if $d \geq B$, the water will have no effect on the ultimate bearing capacity.

It is valuable to know that the shear strength parameters for dry and saturated soils are not the same. In general, the soil possesses high shear strength parameters in dry conditions. Hence, from a practical point of view, the designer prefers to assume saturated soil rather than partially saturated soil to be on the safe side.



- الماء بالمستوى ① - ① ، فان:

$$q_f = cN_c + \bar{\gamma} D N_q + 0.5 B \bar{\gamma} N_\gamma$$

- الماء بالمستوى ② - ② فان:

$$q_f = cN_c + [\gamma_t d + \bar{\gamma}(D-d)] N_q + 0.5 B \bar{\gamma} N_\gamma$$

- الماء بالمستوى ⑤ - ⑤ فان:

$$q_f = cN_c + \gamma_t D N_q + 0.5 [Z_w \gamma_t + (B - Z_w) \bar{\gamma}] N_\gamma$$

- الماء بالمستوى ③ - ③ فان:

$$q_f = cN_c + \gamma_t D N_q + 0.5 B \bar{\gamma} N_\gamma$$

- الماء بالمستوى ④ - ④ فاكثر الى الاسفل:

$$q_f = cN_c + \gamma_t D N_q + 0.5 B \gamma_t N_\gamma$$

6.6 THE GENERAL BEARING CAPACITY EQUATION

The ultimate Terzhagi's bearing capacity equation is for continuous, square, and circular foundations only; it does not address the case of rectangular foundations Also, the equations do not consider the shearing resistance along the failure surface in soil above the bottom of the foundation. In addition, the load on the foundation may be inclined. To account for all these shortcomings, Meyerhof (1963) suggested the following form of the general bearing capacity equation:

$$q_u = c\, S_{cs}\, S_{cd}\, S_{ci}\, N_c + q\, S_{qs}\, S_{qd}\, S_{qi}\, N_q + 0.5\, B\, \gamma\, S_{\gamma s}\, S_{\gamma d}\, S_{\gamma i}\, N_\gamma$$

Where:

c=cohesion of soil

B=width of the footing

q is the overburden pressure at the base of the foundation level=$\gamma\ D_f$

$S_{cs}\ S_{cd}\ S_{ci} = shape\ factors$

$S_{qs}\ S_{qd}\ S_{qi} = depth\ factors$

$S_{\gamma s}\ S_{\gamma d}\ S_{\gamma i} = inclination\ factors$

$N_c\ N_q\ and\ N_\gamma = bearing\ capacity\ factors$

For the Meyerhof's Bearing capacity, Table 6.6-1 summarizes all the equation that can be used to calculate different factors.

Table 6.6-1: Shape, Depth, and Inclination Factors for general bearing capacity equation

Factor	Relationship	Reference
Shape	$S_{cs} = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$ $S_{qs} = 1 + \left(\frac{B}{L}\right)\tan\phi'$ $S_{\gamma s} = 1 - 0.4\left(\frac{B}{L}\right)$	DeBeer (1970)
Depth	$\frac{D_f}{B} \le 1$ For $\phi = 0$: $S_{cd} = 1 + 0.4\left(\frac{D_f}{B}\right)$ $S_{qd} = 1$ $S_{\gamma d} = 1$ For $\phi' > 0$: $S_{cd} = S_{qd} - \frac{1 - S_{qd}}{N_c \tan\phi'}$ $S_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2\left(\frac{D_f}{B}\right)$ $S_{\gamma d} = 1$ $\frac{D_f}{B} > 1$ For $\phi = 0$: $S_{cd} = 1 + 0.4 \underbrace{\tan^{-1}\left(\frac{D_f}{B}\right)}_{\text{radians}}$ $S_{qd} = 1$ $S_{\gamma d} = 1$ For $\phi' > 0$: $S_{cd} = S_{qd} - \frac{1 - S_{qd}}{N_c \tan\phi'}$ $S_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2 \underbrace{\tan^{-1}\left(\frac{D_f}{B}\right)}_{\text{radians}}$ $S_{\gamma d} = 1$	Hansen (1970)
Inclination	$S_{ci} = S_{qi} = \left(1 - \frac{\beta^\circ}{90^\circ}\right)^2$ $S_{\gamma i} = \left(1 - \frac{\beta}{\phi'}\right)$ β = inclination of the load on the foundation with respect to the vertical	Meyerhof (1963); Hanna an Meyerhof (1981)

Many researchers followed different approaches, however, the main difference between their approaches are the bearing capacity factors. In addition, the variation of the bearing capacity factors for the general bearing capacity equation is given in Table 6.6-2.

Table 6.6-2: shows the variation of the bearing capacity factors with soil friction angles for the general bearing capacity equation.

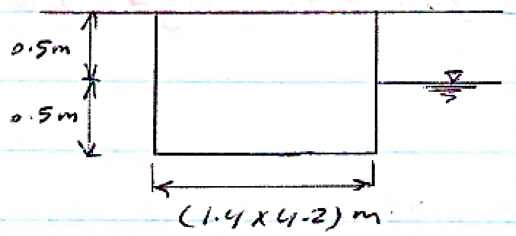
ϕ'	N_c	N_q	N_γ	ϕ'	N_c	N_q	N_γ
0	5.14	1.00	0.00	26	22.25	11.85	12.54
1	5.38	1.09	0.07	27	23.94	13.20	14.47
2	5.63	1.20	0.15	28	25.80	14.72	16.72
3	5.90	1.31	0.24	29	27.86	16.44	19.34
4	6.19	1.43	0.34	30	30.14	18.40	22.40
5	6.49	1.57	0.45	31	32.67	20.63	25.99
6	6.81	1.72	0.57	32	35.49	23.18	30.22
7	7.16	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				

6.6.1 EXAMPLE 6.2:

A rectangular footing shown below (1.4×4.2 m) is to be constructed on soil with $\gamma_t = \gamma_{sat} = 2\gamma_w = 20.0\ kN/m^3$, $c_u = 4.0\ kPa$ and $\emptyset = 25.0^{\circ}$. Find the ultimate bearing capacity of the footing if $N_c = 20.72$, $N_q = 10.66$ and $N_\gamma = 10.88$

Sol.

$q_f = cN_c S_c + q N_q S_q + 0.5 B \gamma N_\gamma S_\gamma$



$\therefore q_f = (4)(20.72)(1.17)$

$+ (0.5*20 + 0.5(10))(10.66)(1.16) + (0.5)(1.4)(10)(10.88)(0.866)$

$q_f = 348.41\ kN/m^2$

6.7 EFFECT OF FOOTING DEPTH ON THE BEARING CAPACITY OF SATURATED CLAY (SKEMPTON'S SOLUTION)

For the fully saturated clay in undrained condition (ϕu=0.0), Skempton proposed the following simplified solution:

$$q_f = cN_c + q\ N_q$$

For rectangular footing, the following equation is used:

$$N_c = \left(1 + 0.2\,\frac{B}{L}\right) N_{c(strip)}$$

Or used the following chart (Figure 6.7-1)

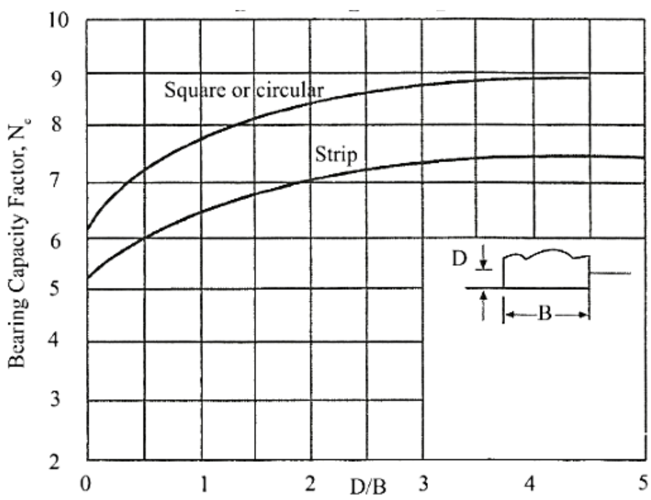


Figure 6.7-1: Skempton's bearing capacity factor, Nc

6.8 FACTOR OF SAFETY FOR BEARING CAPACITY

Calculating the gross allowable load-bearing capacity of shallow foundations requires the application of a factor of safety (FS) to the gross ultimate bearing capacity, or

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

However, some practicing engineers prefer to use a factor of safety such that:

$$Net\ stress\ increase\ on\ soil = \frac{net\ ultimate\ bearing\ capacity}{FS}$$

The net ultimate bearing capacity is defined as the ultimate pressure per unit area of the foundation that can be supported by the soil more than the pressure caused by the surrounding soil at the foundation level. If the difference between the unit weight of concrete used in the foundation and the unit weight of soil surrounding is assumed to be negligible, then:

$$q_{net\ (u)} = q_u - q = q_u - \gamma\ D$$

Where: $q_{net\ (u)} = net\ ultimate\ bearing\ capacity$

The above-mentioned explanation can be summarized and illustrated in the Figure 6.8-1. On the other hand, the factor of safety for different structures and functions are listed in Table 6.8-1)

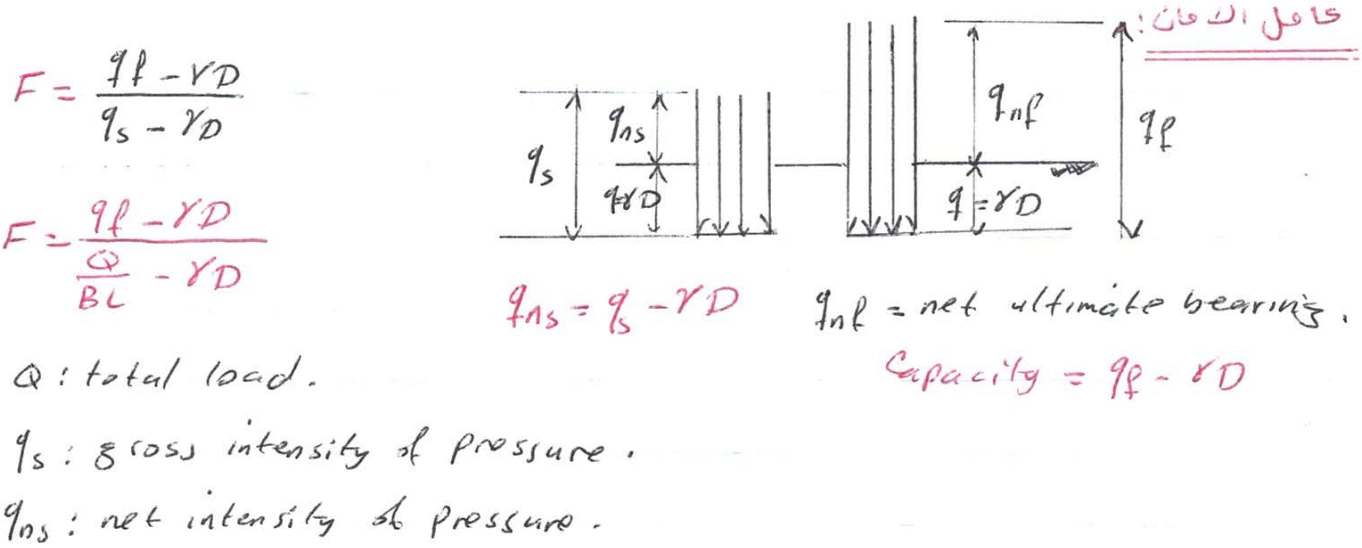


Figure 6.8-1: Net bearing capacity and factor of safety

Table 6.8-1: Minimum value of safety factor for design of shallow foundations (Vesic, 1970)

Category	Typical Structure	Characteristics of the category	Soil Exploration	
			Thorough	Limited
A	Railway bridge, Warehouses, blast furnaces, silos, hydraulic retaining walls	Maximum design load likely to occur often, consequence of failure disastrous	3.0	4.0
B	Highway bridge, light industrial and public buildings	Maximum design load may occur occasionally, consequence of failure serious	2.5	3.5
C	Apartments and office buildings	Maximum design load unlikely to occur	2.0	3.0

6.8.1 EXAMPLE 6.3:

A square foundation is 2 m ×2m in plan. The soil supporting the foundation has a friction angle $\emptyset = 25.0^{\circ}$ and $c' = 20.0\ kPa$. The unit weight, $\gamma_t = 16.5\ kN/m^3$. Determine the allowable gross load on the foundation with a factor of safety of 3.0. Assume that the depth of the foundation is 1.5 m and that general shear failure occurs in the soil (Use Terzaghi Method and Meyerhof Method).

$$q_u = 1.3c'N_c + qN_q + 0.4\gamma BN_\gamma$$

From Table 3.1, for $\phi' = 25°$,

$$N_c = 25.13$$

$$N_q = 12.72$$

$$N_\gamma = 8.34$$

Thus,

$$q_u = (1.3)(20)(25.13) + (1.5 \times 16.5)(12.72) + (0.4)(16.5)(2)(8.34)$$

$$= 653.38 + 314.82 + 110.09 = 1078.29\ \text{kN/m}^2$$

So, the allowable load per unit area of the foundation is

$$q_{\text{all}} = \frac{q_u}{\text{FS}} = \frac{1078.29}{3} \approx 359.5\ \text{kN/m}^2$$

Thus, the total allowable gross load is

$$Q = (359.5)\ B^2 = (359.5)\ (2 \times 2) = \mathbf{1438\ kN}$$

Example 3.2

Solve Example Problem 3.1 using Eq. (3.19).

Solution

From Eq. (3.19),

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

Since the load is vertical, $F_{ci} = F_{qi} = F_{\gamma i} = 1$. From Table 3.3 for $\phi' = 25°$, $N_c = 20.72$, $N_q = 10.66$, and $N_\gamma = 10.88$.

Using Table 3.4,

$$F_{cs} = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right) = 1 + \left(\frac{2}{2}\right)\left(\frac{10.66}{20.72}\right) = 1.514$$

$$F_{qs} = 1 + \left(\frac{B}{L}\right)\tan\phi' = 1 + \left(\frac{2}{2}\right)\tan 25 = 1.466$$

$$F_{\gamma s} = 1 - 0.4\left(\frac{B}{L}\right) = 1 - 0.4\left(\frac{2}{2}\right) = 0.6$$

$$F_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2\left(\frac{D_f}{B}\right)$$

$$= 1 + (2)(\tan 25)(1 - \sin 25)^2\left(\frac{1.5}{2}\right) = 1.233$$

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c\tan\phi'} = 1.233 - \left[\frac{1 - 1.233}{(20.72)(\tan 25)}\right] = 1.257$$

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

Hence,

$$q_u = (20)(20.72)(1.514)(1.257)(1)$$
$$+(1.5 \times 16.5)(10.66)(1.466)(1.233)(1)$$
$$+\frac{1}{2}(16.5)(2)(10.88)(0.6)(1)(1)$$
$$= 788.6 + 476.9 + 107.7 = 1373.2 \text{ kN/m}^2$$

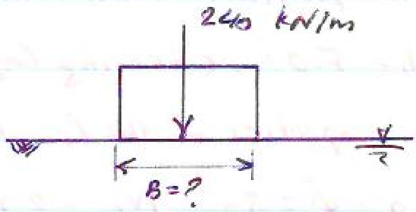
$$q_{all} = \frac{q_u}{FS} = \frac{1373.2}{3} = 457.7 \text{ kN/m}^2$$

$$Q = (457.7)(2 \times 2) = 1830.8 \text{ kN}$$

6.8.2 EXAMPLE 6.4:

Find the dimension b of a strip footing resting at the ground surface and carrying a load of 240 kN/m as shown in figure. The shear strength parameters of the soil are $\emptyset = 30.0^\circ$ and $c' = 5.0\ kPa$

Sol.



$N_c = 30,\ N_q = 18.4,\ N_\gamma = 21$

$\gamma_{sat} = 19.81\ kN/m^3,\ \gamma_w = 9.81\ kN/m^3$

, use $F = 3$

$q_f = cN_c + q N_q + 0.5 B \gamma N_\gamma$ (q → 0)

$= 5 \times 30 + 0.5B(19.81 - 9.81) \times 21$

$= 150 + 105B$

$F = \dfrac{q_f - \gamma D}{\frac{Q}{Area} - \gamma D} \Leftrightarrow 3 = \dfrac{150 + 105B}{\frac{240}{B}},\quad 7B^2 + 10B - 48 = 0$

$(7B + 24)(B - 2) = 0,\quad \therefore B = 2\ m$

6.8.3 EXAMPLE 6.5:

A square footing of (1×1m) was placed in a saturated clayey soil. The undrained cohesion was 40 kPa, find:

1. The least value of D beyond which there is no increase in the net ultimate capacity
2. The ultimate bearing capacity (gross) if the γ=20.0 kN/m^3.

Solution:

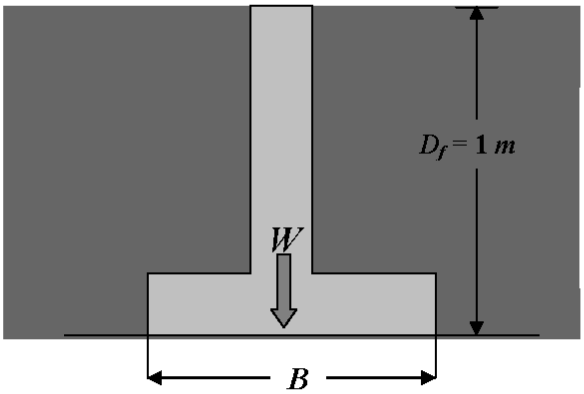
@ $\frac{D}{B} = 4.5$, N_c is constant, so q_f doesn't increase.

$\therefore D = 4.5 \times 1 = 4.5\ m$

$q_f = (40)(9) + (20)(4.5) = 450\ kN/m^2.$

6.8.4 EXAMPLE 6.6:

The square footing shown below must be designed to carry a 294 kN load. Use Terzaghi's bearing capacity formula to determine B of the square footing with a Factor of Safety =3.



Solution:

Terzaghi's formula for the ultimate bearing capacity q_{ult} of a square footing is,

$$q_{ult} = 1.3c'N_c + \bar{q}N_q + 0.4\gamma BN_\gamma \quad where \quad \bar{q} = D_f\gamma$$

The allowable bearing capacity q_{all} with the factor of safety of 3 is,

$$q_{all} = \frac{q_{ult}}{3} = \frac{1}{3}\left(1.3c'N_c + \bar{q}N_q + 0.4\gamma BN_\gamma\right) \quad and \quad q_{all} = \frac{W}{B^2} = \frac{294\ kN}{B^2}$$

$$or \quad \frac{294}{B^2} = \frac{1}{3}\left(1.3c'N_c + \bar{q}N_q + 0.4\gamma BN_\gamma\right)$$

For ϕ=35°, N_c=57.8, N_q=41.4, and N_γ=41.1.

Substituting these values into Terzaghi's equation, we get

$$\frac{294}{B^2} = \frac{1}{3}\left[(0) + (18.15)(1)(41.4) + (0.4)(18.15)B(41.1)\right]$$

$$\frac{294}{B^2} = 250.5 + 99.5B$$

$$B^3 + 2.52B^2 - 2.96 = 0 \quad \therefore \quad B = 0.90\ m$$

6.8.5 EXAMPLE 6.7:

Use Meyerhof's bearing capacity formula to determine B with a factor of safety =3 for the previous example.

Solution:

Meyerhof's formula for the ultimate bearing capacity q_{ult} of a square footing is,

$$q_{ult} = c' N_c F_{sc} F_{dc} F_{ic} + \bar{q} N_q F_{sq} F_{dq} F_{iq} + 0.4 \gamma B N_\gamma F_{s\gamma} F_{d\gamma} F_{i\gamma} \quad where \quad \bar{q} = D_f \gamma$$

Since the load is vertical, all three inclination factors $F_{ic} = F_{iq} = F_{i\gamma} = 1$.

$$F_{sq} = 1 + \left(\frac{B}{L}\right)\tan\phi = 1 + \left(\frac{1}{1}\right)\tan 35° = 1.70 \quad and \quad F_{s\gamma} = 1 - 0.4\left(\frac{B}{L}\right) = 1 - 0.4(1) = 0.6$$

$$F_{dq} = 1 + 2\tan\phi(1-\sin\phi)^2\left(\frac{D_f}{B}\right) = 1 + 2(\tan 35°)(1-\sin 35°)^2\left(\frac{1}{B}\right) \approx 1.25 \quad and \; F_{d\gamma} = 1$$

The allowable bearing capacity q_{all} with the factor of safety of 3 is,

$$q_{all} = \frac{q_{ult}}{3} = \frac{1}{3}\left(c' N_c F_{sc} F_{dc} + \bar{q} N_q F_{sq} F_{dq} + 0.4 \gamma B N_\gamma F_{s\gamma} F_{d\gamma}\right) \quad and \quad q_{all} = \frac{W}{B^2} = \frac{294\, kN}{B^2}$$

$$or \quad \frac{294}{B^2} = \frac{1}{3}\left(c' N_c F_{sc} F_{dc} + \bar{q} N_q F_{sq} F_{dq} + 0.4 \gamma B N_\gamma F_{s\gamma} F_{d\gamma}\right)$$

For $\phi = 35°$, $N_c = 46.12$, $N_q = 33.30$, and $N_\gamma = 37.15$.

Substituting these values into Meyerhof"s equation, we get

$$\frac{294}{B^2} = \frac{1}{3}\left[(0) + (18.15)(1)(33.3)(1.7)(1.25) + (0.4)(18.15)B(37.15)(0.6)(1)\right]$$

$$\frac{294}{B^2} = 428.1 + 53.94B \quad or \quad B^3 + 7.94B - 5.45 = 0 \quad \therefore B = 0.65\ m$$

6.8.6 EXAMPLE 6.8:

A) A square footing (2.0 ×2.0 m) is placed at 4.0 m depth in a very stiff clay soil of γ_{sat}=21.0 kN/m^3 and undrained cohesion at 4.0m equals to 120.0 kPa and ϕu=0.0. If the factor of safety against shear failure is 3.0, find the total load that can be applied on the footing.

B) For the same data in (A), what would be the value of total load if a rectangular footing of (1.0 ×4.0 m) used instead of the square one.

Sol.

A) $\frac{D}{B} = 2.0$, $N_c = 8.4$ (from chart)

$q_f = C_u N_c + q N_q = (120)(8.4) + (21)(9)$: 1092 kN/m^2

$F = (q_f - \gamma D) / (\frac{Q_s}{2 \times 2} - \gamma D) = 3$

$3 = \dfrac{1092 - 21 \times 4}{\frac{Q_s}{4} - 21 \times 4} \Rightarrow Q_s =$ 1680 kN

B) From chart, input the D/B=4 and get Nc $_{(strip)}$=7.5, Then:

$$N_c = \left(1 + 0.2\,\frac{B}{L}\right) N_{c(strip)} = \left(1 + 0.2\,\frac{1}{4}\right) \times 7.5 = 7.875$$

$\therefore qf = 120 \times 7.875 + 21 \times 4.0 =$1029 kN/m^2

$$FS = \frac{q_f - \gamma D}{\frac{Q_s}{BL} - \gamma D} \rightarrow 3 = \frac{1029 - 21 \times 4}{\frac{Q_s}{1 \times 4} - 21 \times 4}$$

Simplifying and get Qs=1596 kN

6.8.7 EXAMPLE 6.9:

A circular footing paled at a depth of 1.0 in unsaturated silty clayey soil if γ_t=18.0 kN/m3 and undrained cohesion (cu) equals 40.0 kPa and ϕu=10.0, calculate the diameter of the footing (Use FS=3.0).

Sol.

$$q_f = cN_c S_c + \gamma D N_q S_q + 0.5 B \gamma N_\gamma S_\gamma$$

From Table 4.6-2, input the value of ϕ=10.0 and get:

$N_c = 8.35$, $N_q = 2.47$ $N_\gamma = 1.22$

$S_c = 1 + \frac{B}{L}\frac{N_q}{N_c}$, $S_q = 1 + \frac{B}{L}\tan\phi$, $S_\gamma = 1 - 0.4\frac{B}{L}$

@ $B = L$

$\therefore S_c = 1 + \frac{N_q}{N_c}$, $S_q = 1 + \tan\phi$, $S_\gamma = 1 - 0.4$

$\therefore S_c = 1.3$, $S_q = 1.18$, $S_\gamma = 0.6$

$q_f = 40 * 8.35 * 1.3 + 18 * 1 * 2.47 * 1.18 + 0.5 B * 18 * 1.22 * 0.6$

$= 486.7 + 6.6B$, $\therefore q_{nf} = 486.7 + 6.6B - 18*1$

$$q_{nf} = 468.7 + 6.6B$$

$F = \frac{q_{nf}}{q_{ns}} = \frac{468.7 + 6.6B}{\frac{850}{\pi B^2/4} - 18*1} = 3$, $B^3 + 79.2B^2 - 492 = 0$

$\therefore B \approx 2.45$ m **Ans.**

6.8.8 EXAMPLE 6.10:

Example: For the retaining wall shown in Fig. estimate:

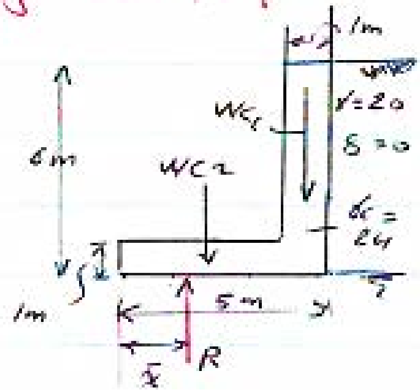
a) The pressure under the base

b) The F.O.S. (Bearing Capacity).

The properties of the foundation soil are:

$\bar{c} = 0$, $\bar{\phi} = 30^\circ$, $N_c = 29$, $N_q = 20$, $N_\gamma = 16$

$\gamma_{sat} = 21.81\ kN/m^3$, $\gamma_w = 9.81\ kN/m^3$



Sol.

$E_a = \frac{1}{2}\gamma H^2 k_a$, $k_a = \tan^2(45 - \frac{\phi}{2}) = \tan^2(45 - \frac{30}{2}) = \frac{1}{3}$

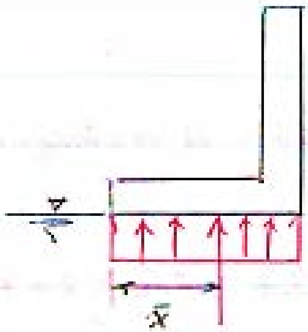
$E_a = \frac{1}{2}(20)(6)^2 \times \frac{1}{3} = 120\ kN/m$

$W_{c1} = 24 \times 1 \times 5 = 120\ kN/m$

$W_{c2} = 24 \times 1 \times 5 = 120\ kN/m$

$\bar{x} = \frac{120 \times 4.5 + 120 \times 2.5 - 120 \times 2}{120 + 120} = 2.5 m$

$\therefore e = 0$



The pressure under the base is uniform

$= \frac{240}{5} = \underline{48}\ kN/m^2$

$q_f = c N_c + q N_q + 0.5 B \gamma N_\gamma$

$= 0.5 \times 5 (21.81 - 9.81) \times 16 = 480\ kN/m^2$

$F.O.S|_{B.C} = \frac{480}{48} = \underline{10}$

6.8.9 EXAMPLE 6.11:

A square footing is to be constructed on a deep deposit of sand at a depth of 0.9 m to carry a design load of 300 kN with a factor of safety of 2.5. The ground water table may rise to the ground level during the rainy season. Design the plan dimension of footing given γ_{sat} = 20.8 kN/m3, N_c = 25, N_q = 34 and N_γ =32.

6.8.10 EXAMPLE 6.12:

A square footing 2.5 m × 2.5 m is built on a homogeneous bed of sand of density 19 kN/m³ having an angle of shearing resistance of 36° The depth of foundation is 1.5 m below the ground surface. Calculate the safe load that can be applied on the footing with a factor of safety of 3. Take bearing capacity factors as N_c= 27, N_q = 30, N_γ= 35.

6.8.11 EXAMPLE 6.13:

What will be the net ultimate bearing capacity of sand having ϕ = 36°and γ_d = 19 kN/m³ for (i) 1.5 m strip foundation and (ii) 1.5 m × 1.5 m square footing. The footings are placed at a depth of 1.5 m below ground level. Assume F = 2.5. Use Terzaghi' s equations.

ϕ	N_c	N_q	N_γ
35°	57.8	41.4	42.4
40°	95.7	81.3	100.4

6.8.12 EXAMPLE 6.14:

A strip footing 2 m wide carries a load intensity of 400 kPa at a depth of 1.2 m in sand. The saturated unit weight of sand is 19.5 kN/m³ and unit weight above the water table is 16.8 kN/m³. If c = 0 and ϕ = 35°, determine the factor of safety with respect to shear failure for the following locations of water table.

a) Water table is 4 m below Ground Level
b) Water table is 1.2 m below Ground Level
c) Water table is 2.5 m below Ground Level
d) The water table is at Ground Level.

Using Terzaghi' s equation, take Nq = 41.4 and Nγ = 42.4.

6.8.13 EXAMPLE 6.15:

A square footing located at a depth of 1.3 m below ground should carry a safe load of 800 kN. Find the size of footing if the desired factor of safety is 3. Use Terzaghi' s analysis for general shear failure. Take c = 8 kPa, N_c = 37.2, N_q = 22.5, N_γ = 19.7.

6.9 ECCENTRICALLY LOADED FOUNDATIONS

In several instances, as with the base of a retaining wall, foundations are subjected to moments in addition to the vertical load, as shown in Figure 6.9-1 and 6.9-2. In such cases, the distribution of pressure by the foundation on the soil is not uniform.

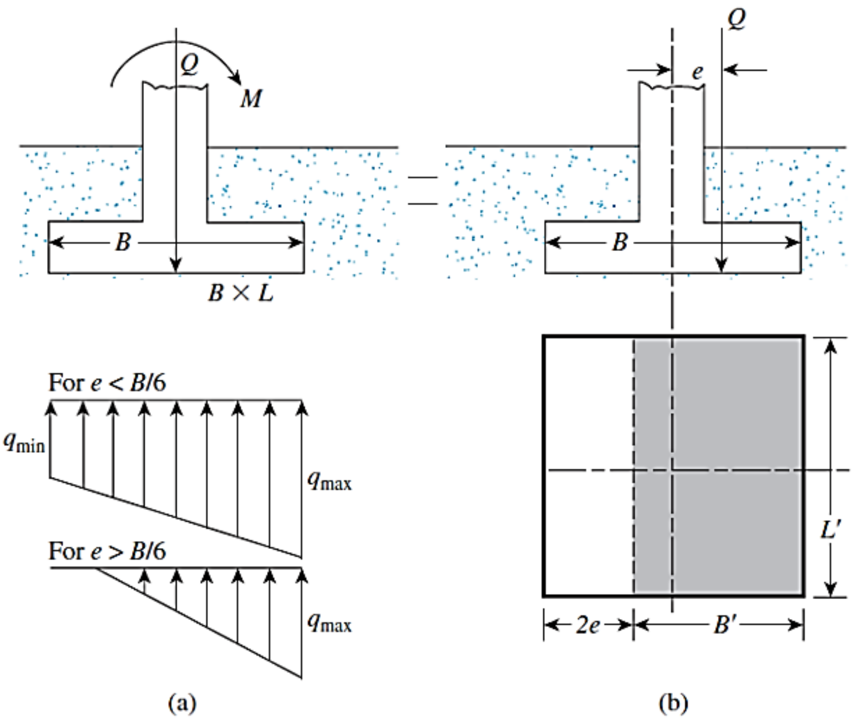


Figure 6.9-1: Eccentrically loaded foundations

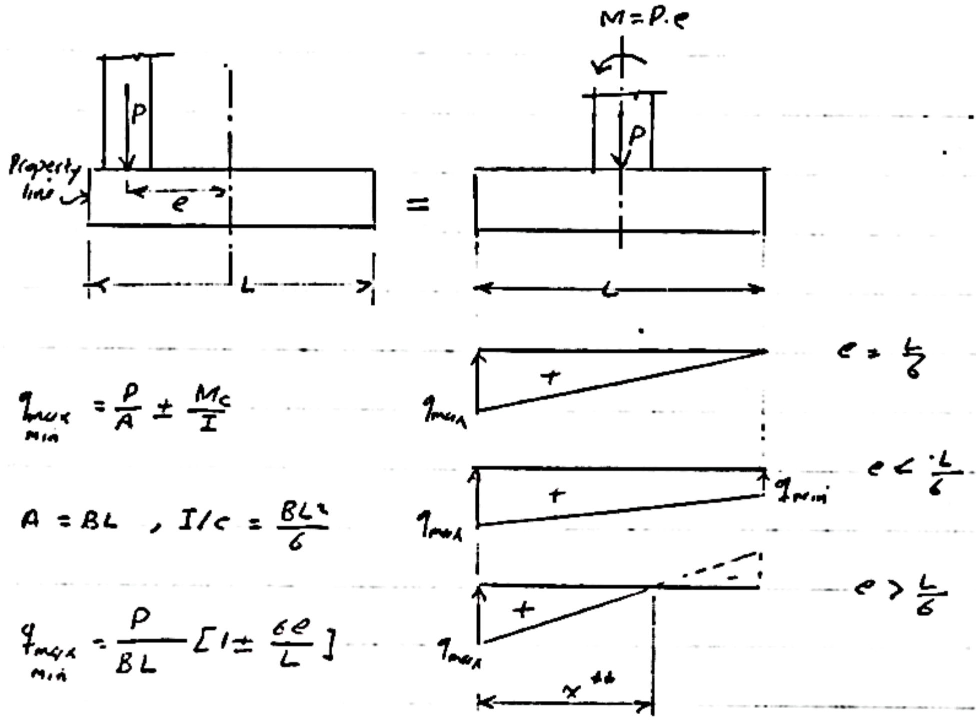


Figure 6.9-2: Eccentrically loaded foundations due to property line

The pressure under the base will not be uniform if there is an eccentricity of load application. In this case, the bearing pressure can be calculated as given below:

$q_{max} = \frac{Q}{B\,L} + \frac{6\,M}{B^2L}$ and $q_{min} = \frac{Q}{B\,L} - \frac{6\,M}{B^2L}$

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

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

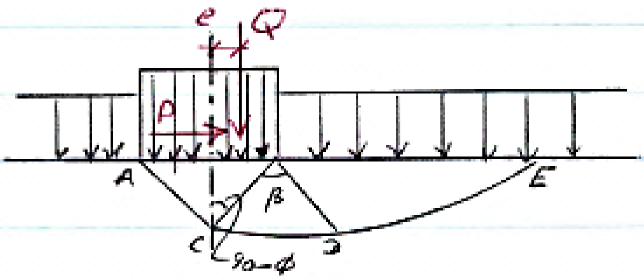
Where: Q: total vertical load

M: moment on the foundation

e: eccentricity

On the other hand, the ultimate bearing capacity of the footing can be found as illustrated herein.

$\bar{B} = B - 2e$, e: eccentricity



ويتم ادخال معاملات الميل :

(inclination factors)

$S_{\gamma i}$, S_{qi} , S_{ci}

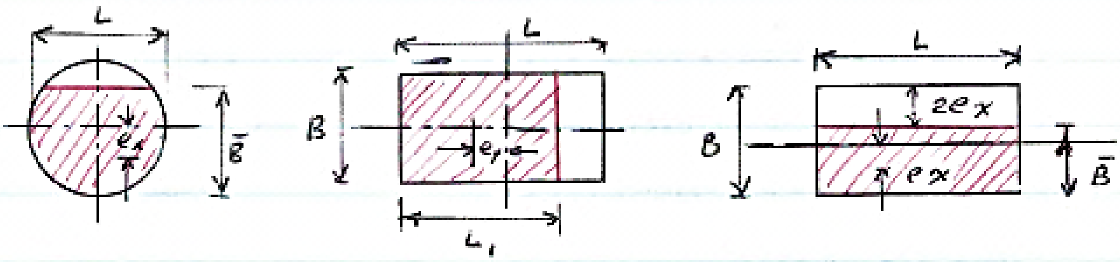
يجوز اي معقول لوجود مركبة افقية P

$$q_u = cN_cS_cS_{ci} + qN_qS_qS_{qi} + \frac{1}{2}\gamma\bar{B}N_\gamma S_\gamma S_{\gamma i}$$

$$S_{qi} = \left[1 - \frac{P}{Q + \bar{B}Lc\cot\phi}\right]^m$$

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

$$S_{\gamma i} = \left[1 - \frac{P}{Q + \bar{B}Lc\cot\phi}\right]^{m+1}$$



تأثير الحمل غير المركزي على مساحة التحميل اللازمة.

$$S_{ci} = S_{qi} - \frac{1 - S_{qi}}{N_c\tan\phi}$$

6.9.1 EXAMPLE 6.16:

A rectangular footing (1.4m ×4.2m) was placed at a depth of 1.0m from the natural ground level of fully saturated clayey soil [$\gamma_t = 21.0 \frac{kN}{m^3}, \phi_u = 0.0, c_u = 22.0\ kN/m^2$]. The ground water table level was at 0.8m from the natural ground level. The footing was subjected to eccentric load (e_B=0.2m). the horizontal and vertical component of the load was 46.2 and 220 kN, respectively. Estimated the maximum bearing capacity of the footing.

6.9.2 EXAMPLE 6.17:

Example: The rectangular footing (1.4 x 4.2) m. The soil properties are:

$\bar{\phi} = 25^\circ$, $\bar{c} = 4\ kN/m^2$

$\gamma_t = 21\ kN/m^3$, $\gamma_w = 10\ kN/m^3$

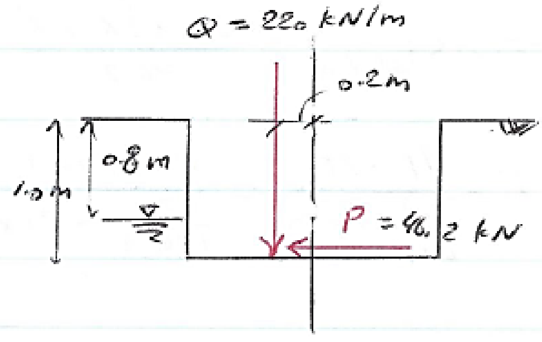
Find: the ultimate bearing capacity.

Use: $N_c = 20.72$, $N_q = 10.66$

$N_\gamma = 10.88$, $S_{ci}' = 0.67$

$S_{qi}' = .7$, $S_{\gamma i}' = 0.58$

$S_c = 1.17$, $S_q = 1.16$, $S_\gamma = 0.87$



Sol.

$q_f = c N_c S_c S_{ci}' + q N_q S_q S_{qi}' + 0.5 \bar{B} \gamma N_\gamma S_\gamma S_{\gamma i}'$

$= 4 \times 20.72 \times 1.17 \times 0.67 + (0.8 \times 21 + 0.2 \times 11) \times 10.66 \times 1.16 \times 0.7$

$+ 0.5 \underline{(1.4 - 2 \times 0.2)}_{B'} (21-10) \times 10.88 \times 0.87 \times 0.58$

$= 225\ kN/m^2$

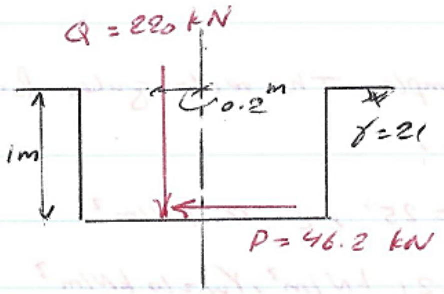
6.9.3 EXAMPLE 6.18:

Example: Rectangular footing (1.4 x 4.2) m. The soil is saturated clay. $\phi_u = 0$, $C_u = 22\ kN/m^2$, $N_c = 5.14$, $N_q = 1$, $S_c = 1.085$, $S_q = 1$. Find:

1- Ultimate bearing capacity if $S_{qi} = 1$, and $S_{ci} = 0.83$.

2- F.O.S. against sliding if C_a (adhesion) = 22 kN/m².

3- F.O.S against bearing capacity failure.



1- $B' = B - 2e = 1.4 - 2 \times 0.2 = 1.0\ m$

$q_f = c N_c S_c S_{ci} + q N_q S_q S_{qi} + 0.5 B \gamma N_\gamma N_{\gamma i}$

$= 22 \times 5.14 \times 1.065 \times 0.83 + 21 \times 1 \times 1 \times 1 \times 1 = 121\ kN/m^2$

2- $F.O.S|_{sliding} = \dfrac{22 \times 1 \times 4.2}{46.2} = 2$

3- $F = \dfrac{q_f - \gamma D}{\dfrac{Q}{L \times B'} - \gamma D} = \dfrac{121 - 21 \times 1}{\dfrac{220}{1 \times 4.2} - 21 \times 1} = 3.71$

SETTLEMENT OF SHALLOW FOOTINGS 7

7.1 INTRODUCTION

The settlement of a foundation can be divided into two major categories:

(a) elastic, or immediate, settlement and

(b) consolidation settlement.

Immediate, or elastic, settlement of a foundation takes place during or immediately after the construction of the structure. Consolidation settlement occurs over time. The total settlement of a foundation is the sum of the elastic settlement and the consolidation settlement.

Consolidation settlement comprises two phases: ***primary and secondary***. The fundamentals of primary consolidation settlement were explained in detail in the basic of the consolidation theory. Secondary consolidation settlement occurs after the completion of primary consolidation caused by ***slippage and reorientation of soil particles under a sustained load***. Primary consolidation settlement is more significant than secondary settlement in inorganic clays and silty soils. However, in organic soils, secondary consolidation settlement is more significant.

For the calculation of foundation settlement (both elastic and consolidation), it is required to estimate the vertical stress increase in the soil mass due to the net load applied on the foundation. Hence, this chapter is divided into the following three parts:

1. Procedure for calculation of vertical stress increase
2. Elastic settlement calculation
3. Consolidation settlement calculation

7.2 VERTICAL STRESS IN A SOIL MASS CAUSED BY FOUNDATION LOAD

The calculation of the stresses into the soil mass is dependent on the nature and type of the applied load and the properties and nature of the soil mass. In this section, a brief description for each loading type will be illustrated.

7.2.1 STRESS DUE TO A CONCENTRATED LOAD

As per the Boussinesq's solution (Figure 7.2-1), the stresses scan be estimated from the following relation:

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

Where:

$r = \sqrt{x^2 + y^2}$

x, y and z is the coordinates of the point A

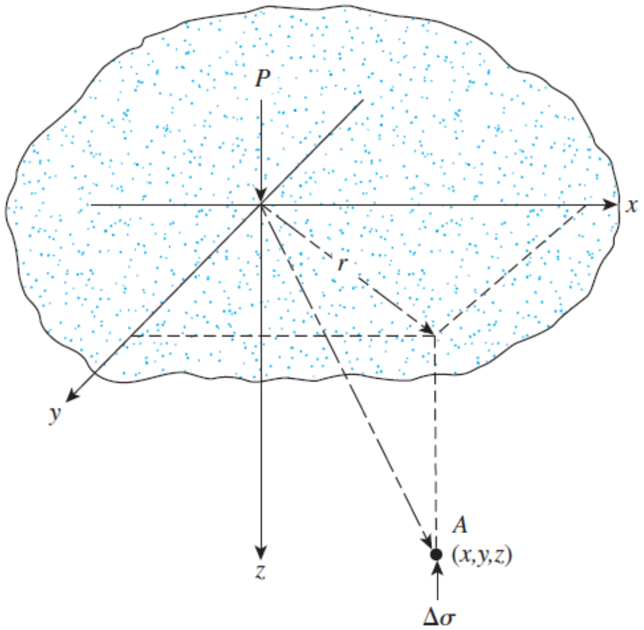


Figure 7.2-1: Vertical stress at a point A caused by a point load on the surface

7.2.2 STRESS DUE TO A CIRCULARLY LOADED AREA

The Boussinesq equation for point load can be can also be used to determine the vertical stress below the center of a flexible circularly loaded area, as shown in Figure 7.2-2.

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

The solution can be normalized as given in Table 7.2-1.

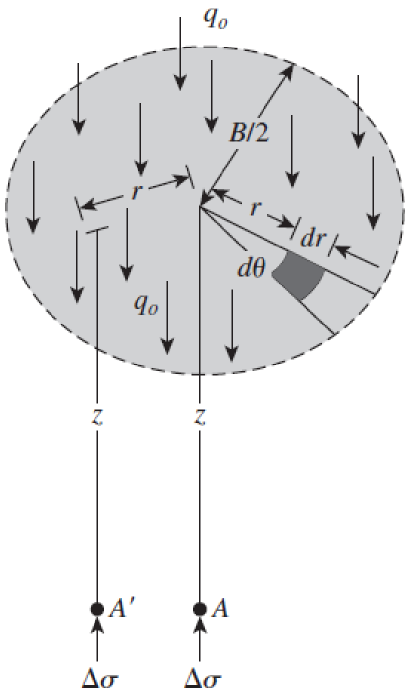


Figure 7.2-2: Increase in pressure under a uniformly loaded flexible circular area

Table 7.2-1: Variation of $\frac{\Delta\sigma}{q_o}$ 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

7.2.3 STRESS BELOW A RECTANGULAR AREA

The solution of such problem can be summarized as illustrated in Figure 7.2-3. The solution assumes that the stresses would be calculated under the corner of the rectangular footing. The required equations and parameters are listed below.

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

Where: $\boldsymbol{m} = \frac{\boldsymbol{B}}{\boldsymbol{z}}$ *and* $\boldsymbol{n} = \frac{\boldsymbol{L}}{\boldsymbol{z}}$

The normalized solution of the influence factor is given in Figure 7.2-4

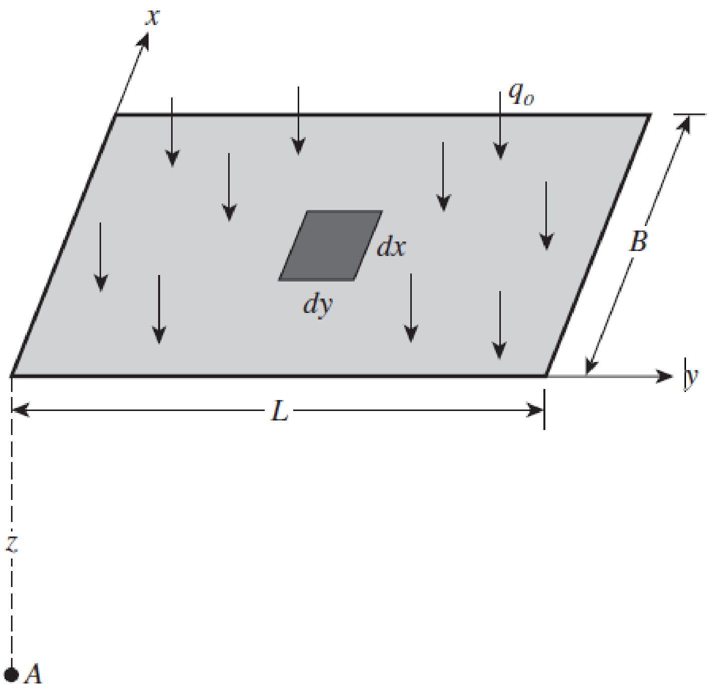


Figure 7.2-3: Determination of stress below the corner of a flexible rectangular loaded area

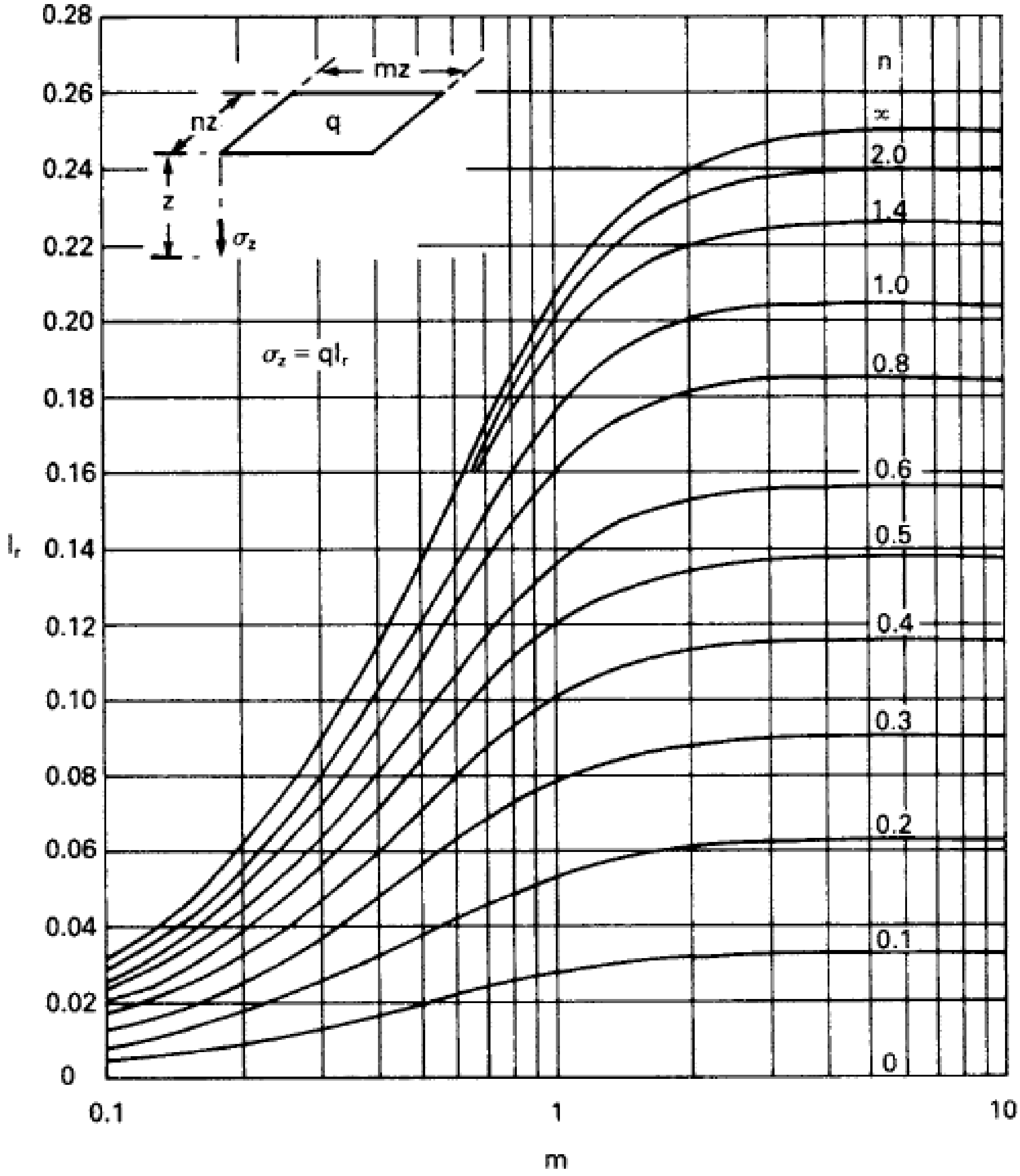


Figure 7.2-4: Values of the influence factor for rectangular loads area

For any arbitrary point within the foundation area, the influence factor can be calculated as illustrated in Figure 7.2-5. The stress increment will be:

$$\Delta\sigma = q_o(I_1 + I_2 + I_3 + I_4)$$

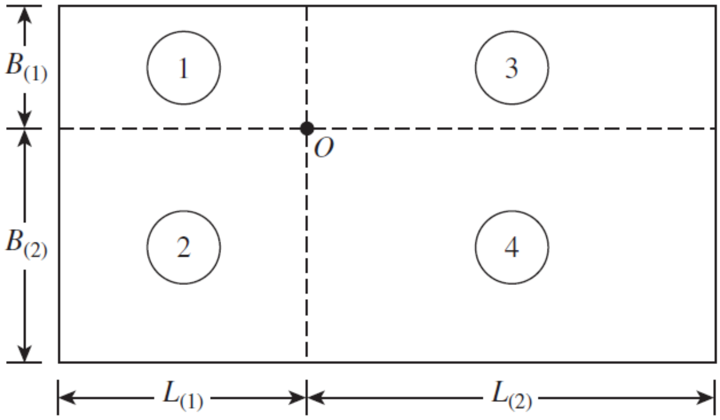


Figure 7.2-5: Stress below any point of a loaded flexible rectangular area

7.2.4 SIMPLIFIED METHOD FOR STRESS BELOW A SURFACE LOADED AREA

Foundation engineers often use an approximate method to determine the increase in stress with depth caused by the construction of a foundation. The method is referred to as the *2:1 method.* (See Figure 7.2-6.) According to this method, the increase in stress at depth z is:

$$\Delta\sigma = \frac{q_o \times B \times L}{(B+z)(L+z)}$$

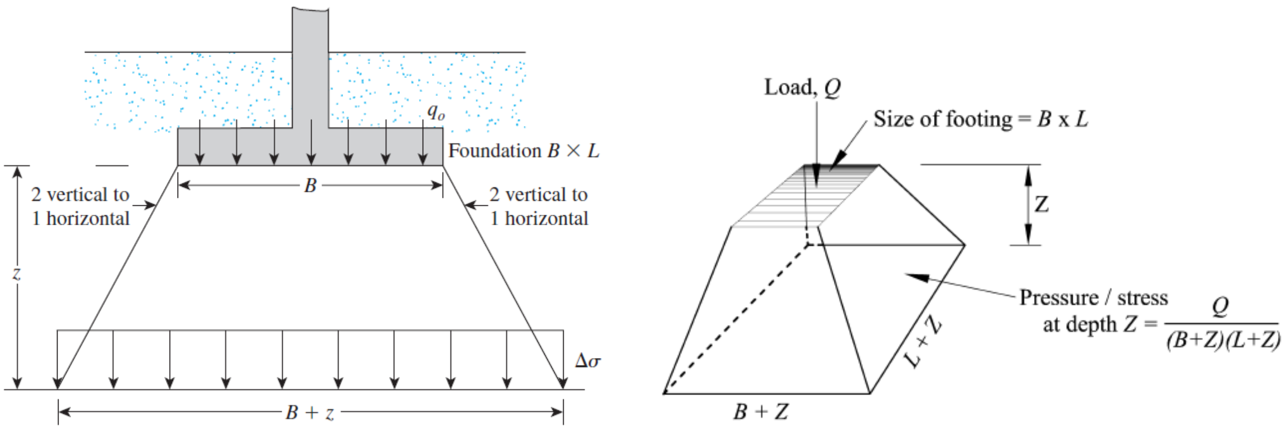


Figure 7.2-6: The 2:1 method of finding stress increase under a foundation

7.3 TYPES OF SETTLEMENT

Generally, the settlement can be classified as follows:

1. According to the shape or style of the settlement to:
   a. Uniform settlement
   b. Differential settlement
2. According to the general and engineering aspects:
   a. Immediate settlement (Si)
   b. Consolidation settlement (Sc)
   c. Secondary compression settlement (Ss)

Thus, the total settlement of any footing equals to:

$$S_t = S_i + S_c + S_s$$

The scope of the present course is to estimate the immediate and consolidation settlement. Moreover, the rate of consolidation settlement will be discussed herein. The concept of the settlement and contact pressure under the footing for rigid and flexible footing is presented in Figure 7.3-1and Figure 7.3-2.

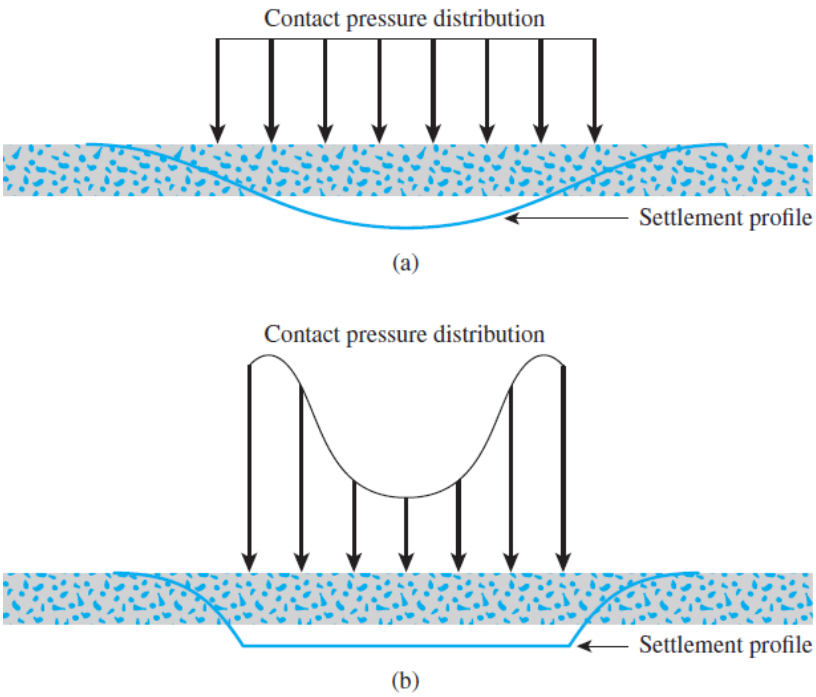


Figure 7.3-1:Elastic settlement profile and contact pressure in clay: (a) flexible foundation; (b) rigid foundation

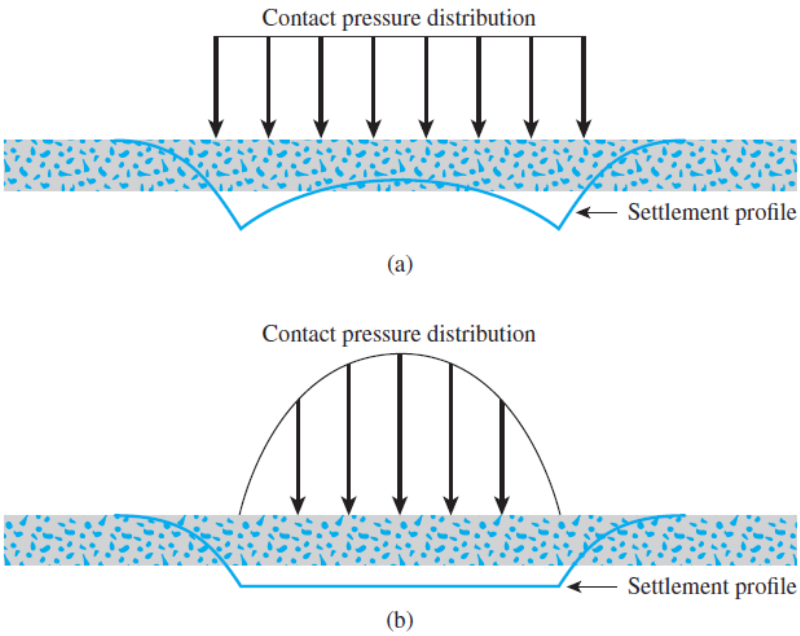


Figure 7.3-2: Elastic settlement profile and contact pressure in sand: (a) flexible foundation; (b) rigid foundation

7.3.1 IMMEDIATE (ELASTIC) SETTLEMENT

Figure 7.3-3 shows a shallow foundation subjected to a net force per unit area equal to Δq_s. Let the Poisson's ratio and the modulus of elasticity of the soil supporting it be u and E_s, respectively. Theoretically, if the foundation is perfectly flexible, the settlement may be expressed as:

$$S_i = \frac{\Delta q_s \, B \, (1 - \mu^2)}{E_s} \, I_p$$

Where:

Δq_s is the uniform stress applied to the foundation (net).

B is the dimeter of circular footing, width of square and the least dimension of the rectangular footing.

E_s is the modulus of elasticity (deformation) of soil

μ is the Poisson's ration of soil, and

I_p is the influence factor

The value of the Ip is estimated at the corner the rectangular footing and some of the values are given in Table 7.3-1. The values of other shapes can be estimated from basic geometry as shown in Figure 7.3-4.

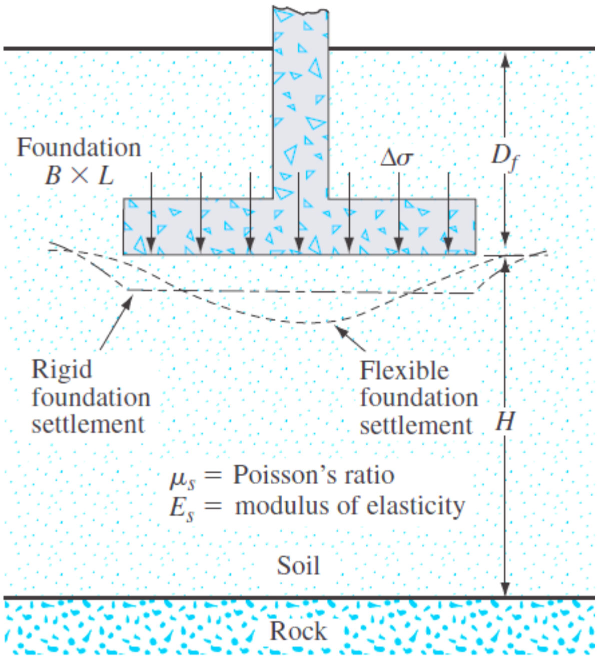


Figure 7.3-3: Elastic settlement of flexible and rigid foundations

Table 7.3-1: Values of Ip

Shape of the area (L × B) $m = B/L$	Ip values			
	Corner	Center	Average	Rigid foundation
Square, $m = 1$	0.56	1.12	0.95	0.88
Circular with diameter, B	0.64	1.00	0.85	0.79
Rectangle				
$m = 1.5$	0.68	1.36	1.15	1.08
$m = 2.0$	0.77	1.53	1.30	1.22
$m = 5$	1.05	2.10	1.83	1.72
$m = 10$	1.27	2.53	2.25	2.12
$m = 100$	2.00	4.00	3.69	—

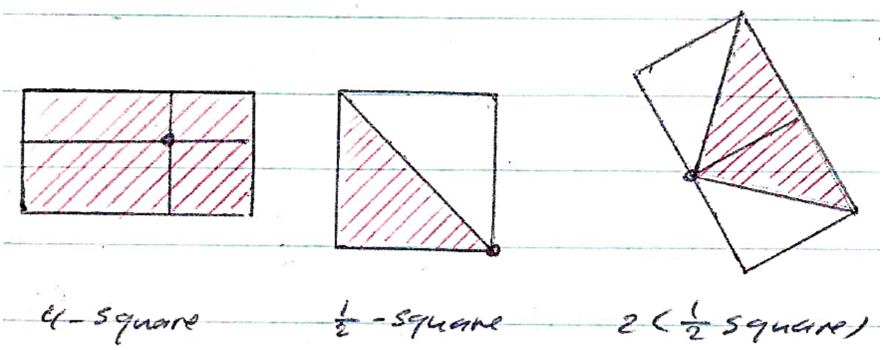


Figure 7.3-4: effect of footing shape on the calculation of Ip

7.3.2 SPECIAL CASES IN THE ESTIMATION OF IMMEDIATE SETTLEMENT

In the estimation and calculation of settlement, it is well known that there are some special cases and conditions that may be encountered in the real soil profile. Two cases will be discussed herein. The first one is the stratified soil profile and the second one is the fully saturated undrained cohesive soil layers.

7.3.2.1 NON-HOMOGENOUS SOIL LAYERS

If a non-homogenous (stratified) soil layers (Figure 7.3-5) encountered in the site, different methods may evaluate the soil parameters such as:

1. Adopt the parameters that gives the worst condition.
2. Take into consideration the parameters at depth of influence
3. Make intensive parametric study by using professional statistical tools
4. The method of superposition may be applied
5. Average the required parameters

However, weighted average could be considered as the reliable approach for most practical problems. Hence, for example if the value of the modulus of elasticity (Es) is to be used for settlement calculations, the weighted average value can be calculated as:

$$E_{avg} = \frac{E_{s1}d_1 + E_{s2}d_2 + E_{s3}d_3 + \cdots}{d_1 + d_2 + d_3 + \cdots}$$

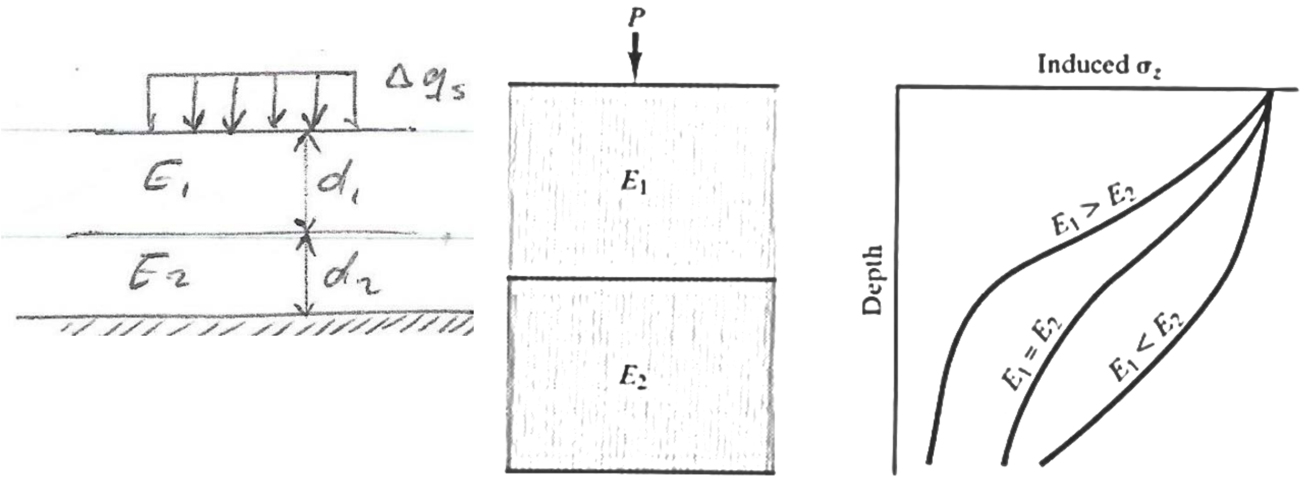


Figure 7.3-5: Induced stresses in stratified soil layers

7.3.2.2 IMMEDIATE SETTLEMENT OF SATURATED COHESIVE SOIL

Janbu et al. (1956) proposed a generalized equation for estimating the average elastic settlement of a uniformly loaded flexible foundation located on saturated clay (μ = 0.5). This relationship incorporates (a) the effect of embedment D_f, and (b) the possible existence of a rigid layer at a shallow depth under the foundation as shown in Figure 7.3-6, or,

$$S_i = \mu_o \mu_1 \frac{\Delta q_s B}{E_s}$$

Where:

μ_o is $f\left(\frac{D_f}{B}\right)$ this factor can be obtained from Table 7.3-2 or Figure 7.3-7

μ_1 is $f\left(\frac{H}{B},\frac{L}{B}\right)$ this factor can be obtained from Table 7.3-3 or Figure 7.3-7

The superposition method may be used for the stratified soil.

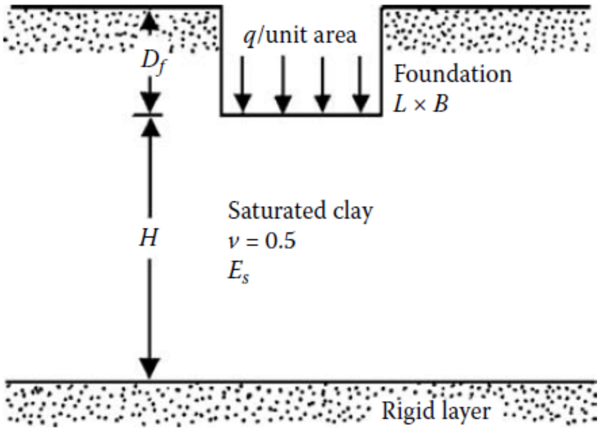


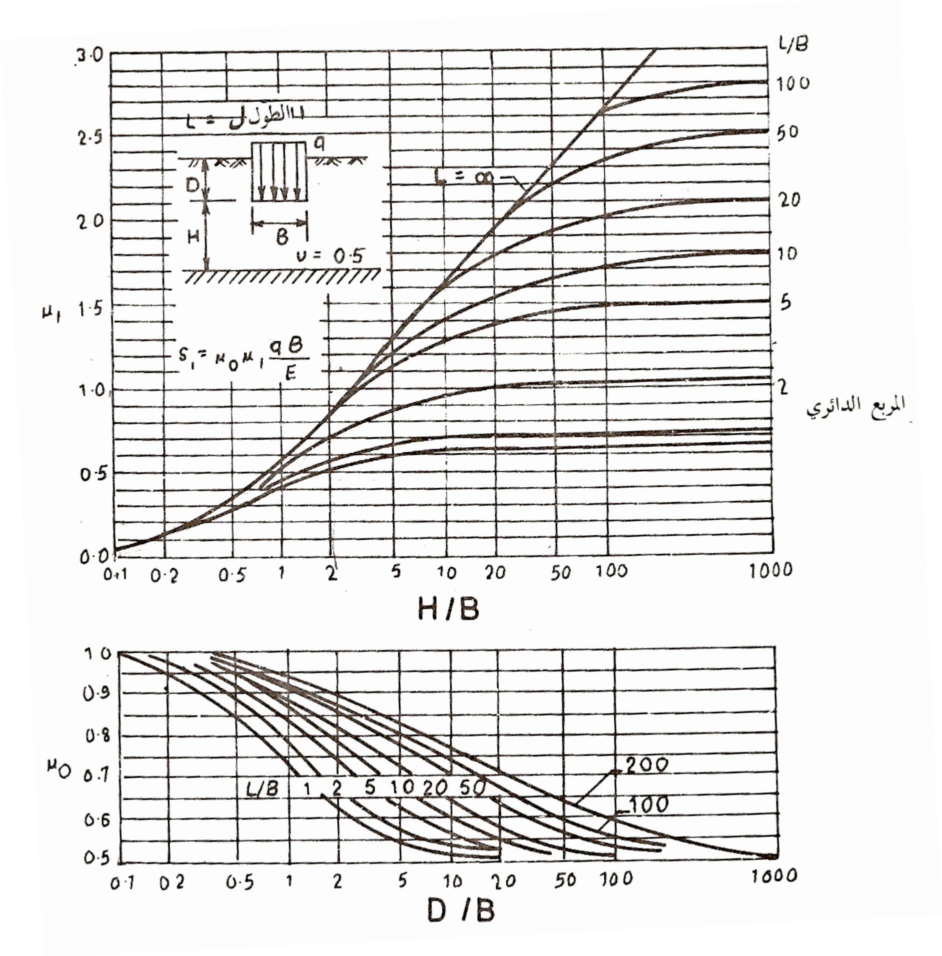
Figure 7.3-6: Settlement of foundation on saturated clay

Table 7.3-2: Variation of μ_o with $\left(\frac{D_f}{B}\right)$

D_f/B	μ_o
0	1.0
2	0.9
4	0.88
6	0.875
8	0.87
10	0.865
12	0.863
14	0.860
16	0.856
18	0.854
20	0.850

Table 7.3-3: Variation of μ_1 with $\left(\frac{H}{B}\right)$ and $\left(\frac{L}{B}\right)$

	L/B					
H/B	Circle	1	2	5	10	∞
1	0.36	0.36	0.36	0.36	0.36	0.36
2	0.47	0.53	0.63	0.64	0.64	0.64
4	0.58	0.63	0.82	0.94	0.94	0.94
6	0.61	0.67	0.88	1.08	1.14	1.16
8	0.62	0.68	0.90	1.13	1.22	1.26
10	0.63	0.70	0.92	1.18	1.30	1.42
20	0.64	0.71	0.93	1.26	1.47	1.74
30	0.66	0.73	0.95	1.29	1.54	1.84



(a)

Figure 7.3-7: Values of μ_o and μ_1 for elastic settlement calculation

7.4 CONSOLIDATION SETTLEMENT

As mentioned before, consolidation settlement occurs over time in saturated clayey soils subjected to an increased load caused by construction of the foundation. Based on the one-dimensional consolidation settlement equations presented in the soil mechanics, the consolidation settlement (Sc) of the compressible cohesive soil layers, can be estimated by different methods as presented below:

$$S_c = \frac{e_o - e_f}{1 + e_o} = \frac{\Delta e}{1 + e_o} \dots\dots\dots$$

$$S_c = m_v\, \Delta\sigma_v'\, H \dots\dots\dots\dots.$$

Equation (3) depends on the overconsolidation ratio (OCR)$= \frac{\sigma_p'}{\sigma_{vo}'}$, and the settlement can be estimated as follows:

- ***For Normally Consolidated Clay with OCR=1.0, the consolidation settlement will:***

$$S_c = \frac{c_c H}{1 + e_o} log\left(\frac{\sigma_{vo}' + \Delta\sigma_z}{\sigma_{vo}'}\right) \dots\dots\dots\dots\dots\dots\dots\dots\dots.\dots.$$

- ***For Over Consolidated Clay, the consolidation settlement depends on the stresses increments and two conditions must be checked:***
  - if $\sigma_{vo}' + \Delta\sigma_z > \sigma_p'$; the consolidation settlement could be calculated using the following equations:

$$S_c = \frac{c_s H}{1+e_o} log\left(\frac{\sigma_p'}{\sigma_{vo}'}\right) + \frac{c_c H}{1+e_o} log\left(\frac{\sigma_{vo}' + \Delta\sigma_z}{\sigma_p'}\right) \ldots\ldots..$$

  - For $\boldsymbol{\sigma_{vo}' + \Delta\sigma_z \leq \sigma_p'}$ the consolidation settlement can be estimated from the following equation:

$$S_c = \frac{c_s H}{1+e_o} log\left(\frac{\sigma_{vo}' + \Delta\sigma_z}{\sigma_{vo}'}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots$$

- ***For Under Consolidated Clay with OCR<1.0, the consolidation settlement will:***

$$S_c = \frac{c_c H}{1+e_o} log\left(\frac{\sigma_{vo}' + \Delta\sigma_z}{\sigma_p'}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots$$

Where:

S_c: Consolidation settlement of clay layer.
H: Thickness of compressible layer.
c_c: Compression index.
c_s: Swelling index.
$\Delta\sigma_z$: Stress increment at certain depth.
σ_{vo}': Effective overburden stress.
σ_p': Maximum Preconsolidation stress
e_o: Initial void ratio
e_f: Final void ratio
m_v: coefficient of volume change
OCR: over consolidation ratio

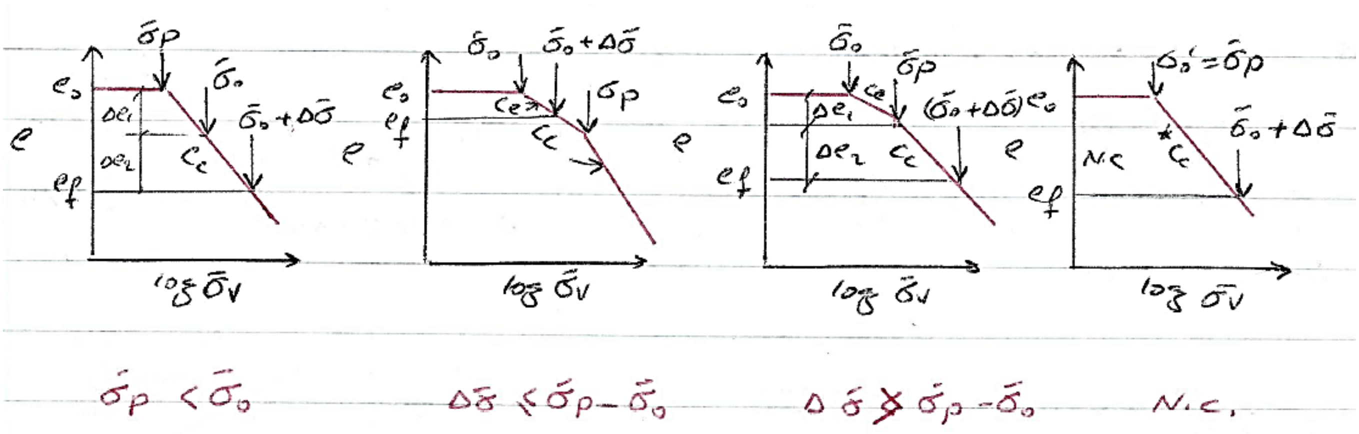


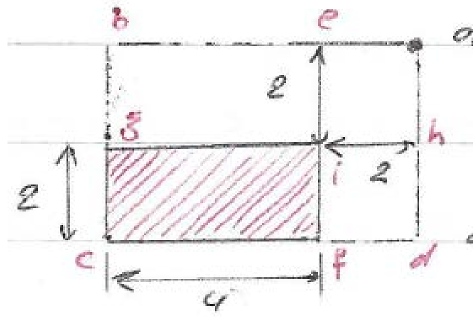
Figure 7.4-1: Different States for the e-log σ_v curves

7.4.1.1 Example 7.1

A rectangular area measures 2.5 m ×3 5 m in plan. It supports a load of 150 kN/m^2. Determine the vertical stress increase due to the load at a depth of 2.0, 6.25 and 10.0 m below the center of the rectangular area. Use theory of elasticity method and 2:1 method.

7.4.1.2 Example 7.2:

For a flexible footing shown, find the immediate settlement at point (a) if E=20000 kN/m^2, Δq_s=200 kN/m^2 and μ=0.5



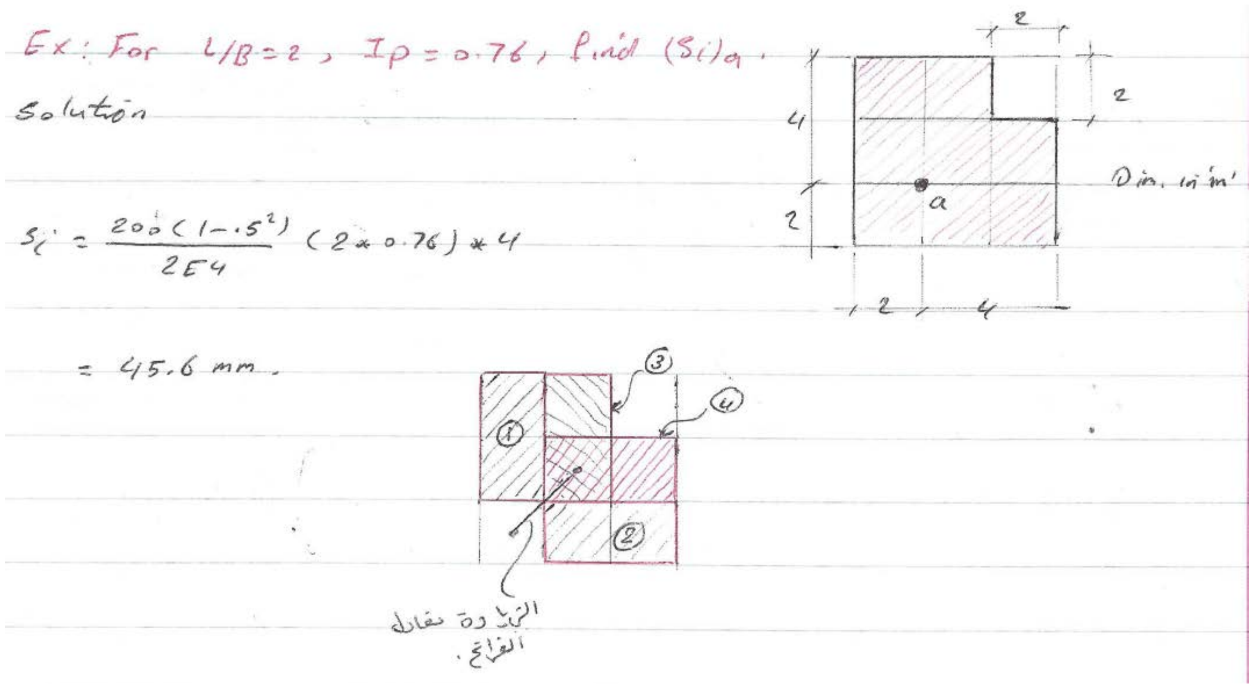
Solution:

The solution is illustrated in table below

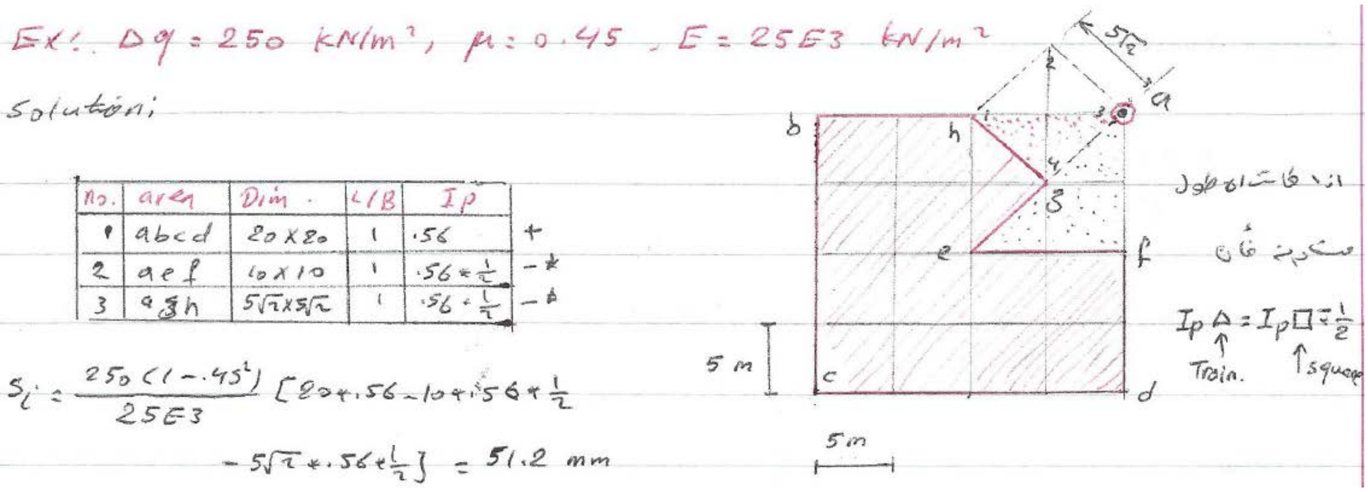
shape	Dimension	L/B	Ip	Area No.	B Ip with sign
a b c d	6 ×4	1.5	0.68	1	+ 4 ×0.68
a e f d	4 ×2	2.0	0.77	2	- 2 ×0.77
a b g h	6 ×2	3.0	0.88	3	- 2 ×0.88
a e i h	2 ×2	1.0	0.56	4	+ 2 ×0.56
					$\sum = 0.54$

Hence, $S_i = \frac{\Delta q_s \, B \, (1-\mu^2)}{E_s} I_p = \frac{200 \times (1-0.5^2)}{20000} \times 0.54 = 4.05 \times 10^{-3} m = 4.05 \, mm$

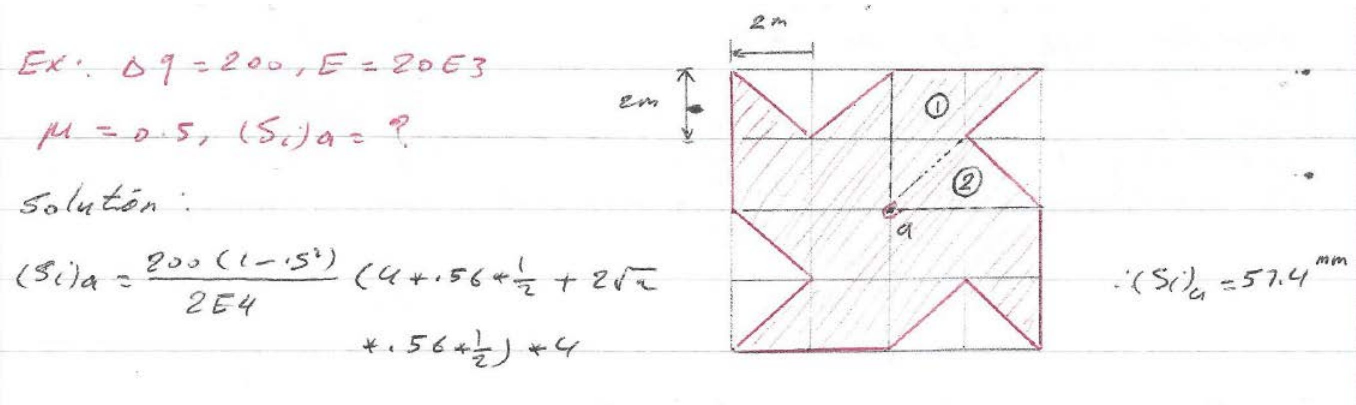
7.4.1.3 Example 7.3:



7.4.1.4 *Example 7.4:*

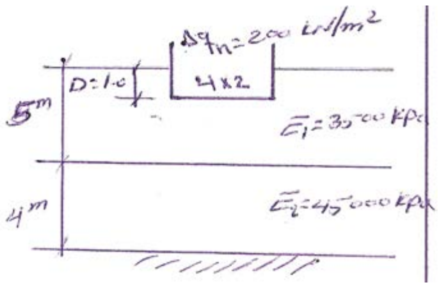


7.4.1.5 *Example 7.5:*



7.4.1.6 *Example 7.6:*

For the soil profile shown, find the immediate settlement of the footing at the corner. Assume ($\mu = 0.5$)



Solution:

Method 1: (use the average values)

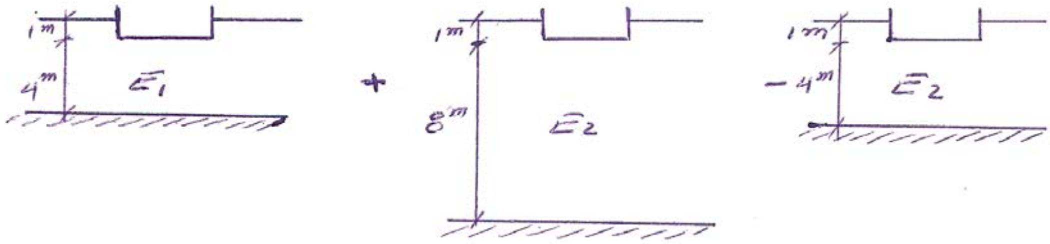
$$E_{avg} = \frac{E_{s1}d_1 + E_{s2}d_2}{d_1 + d_2} = \frac{35000 \times 4.0 + 45000 \times 4.0}{4.0 + 4.0} = 40000\ kPa$$

$$for\frac{L}{B}=\frac{4}{2}=2.0\ \rightarrow I_p=0.77\ (Table\ 5.3-1)$$

$$\therefore S_i=\frac{\Delta q_s\ B\ (1-\mu^2)}{E_s}\ I_p=\frac{200\times 2.0\ (1-0.5^2)}{40000}\times 0.77\times 1000=5.78\ mm$$

Method 2: (use superposition and Janbu et. al method)

The settlement can be estimated as follows:



$\frac{D_f}{B}$	$\frac{1}{2}=0.5$	0.5	0.5
$\frac{H}{B}$	$\frac{4}{2}=2.0$	$\frac{8}{2}=4.0$	2.0
$\frac{L}{B}$	$\frac{4}{2}=2.0$	2.0	2.0
μ_o Table 7.3-2	0.97	0.97	0.97
μ_1 Table 7.3-3	0.63	0.82	0.63

$$\therefore S_i=S_{i1}+S_{i2}-S_{i3}$$

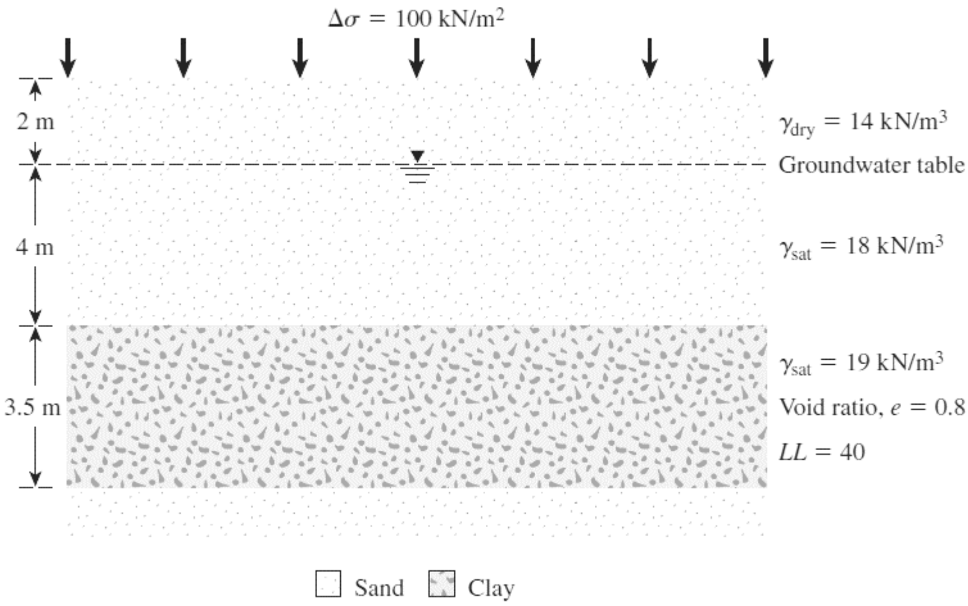
$$\rightarrow S_i=\mu_o\mu_1\frac{\Delta q_s B}{E_s}=200\times 2.0\left[\frac{0.97\times 0.63}{35000}+\frac{0.97\times 0.82}{45000}+\frac{0.97\times 0.63}{45000}\right]\times 1000=8.62\ mm$$

7.4.1.7 Example 7.6:

A soil profile is shown in figure below. If a uniformly distributed load, $\Delta\sigma$, is applied at the ground surface, what is the settlement of the clay layer caused by primary consolidation if:

1. The clay is normally consolidated
2. The preconsolidation pressure is 200 kPa
3. The preconsolidation pressure is 150 kPa

Use $c_s = \frac{1}{5} c_c$



7.4.1.8 Example 7.7:

Given:- square footing 2 m x 2 m

Required :- consolidation settlement in mm.

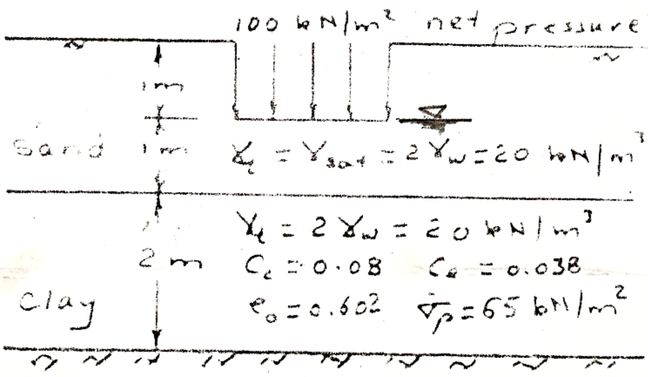
Note: use 2:1 for stress distribution

Properties at the center of clay layer are:-

$C_c = 0.08$ $C_e = 0.038$

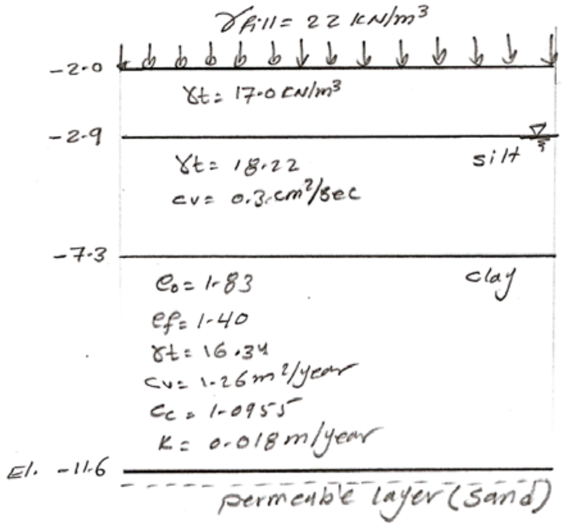
$e_o = 0.602$

$\bar{\sigma}_p$ = max. preconsolidation stress $= 65\ kN/m^2$



7.4.1.9 EXAMPLE 7.8

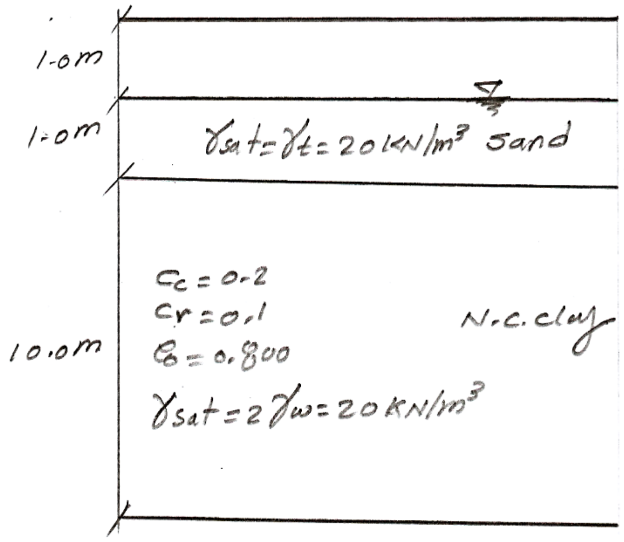
A 4.5 m height of fill is placed over large area having the properties shown. Calculate the final settlement of the clay layer using three methods.



7.4.1.10 EXAMPLE 7.9

For the soil profile shown, find the consolidation settlement for the following cases:-

1. Square footing B = 4.0 m with 800 kN applied column load.
2. Circular footing with D = 4.0 m carrying 800 kN load.
3. Same as ① above with footing placed at 1.0 m from ground surface.
4. Fill of 50 kN/m² at ground surface.

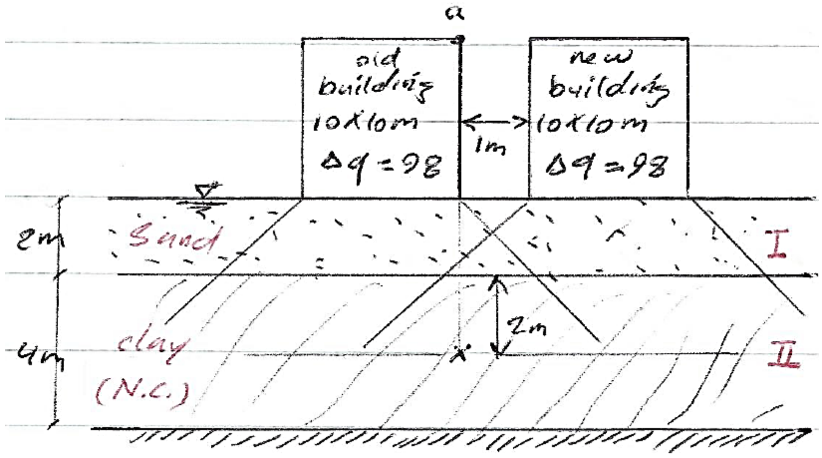


7.4.1.11 Example 7.10:

For the soil profile shown, find the consolidation settlement below point a resulting from the construction of the new building shown.

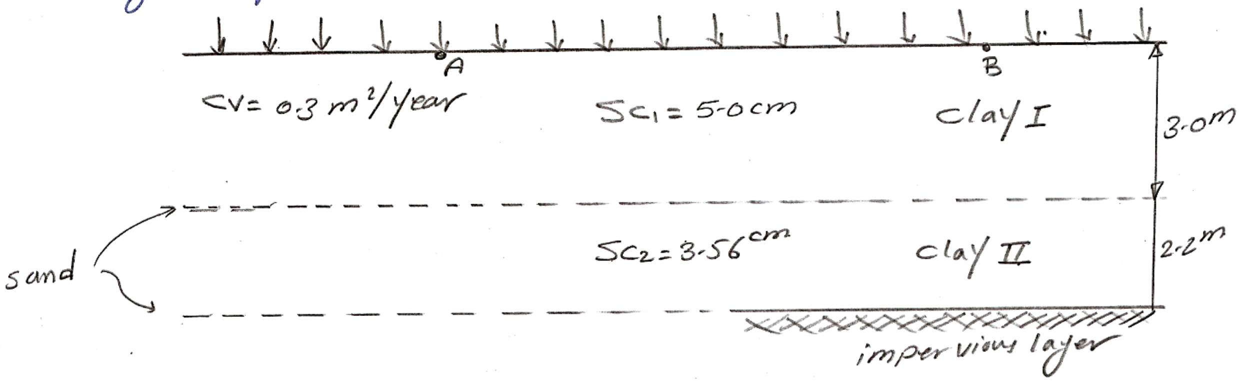
For soil I: $\gamma_t = 19.81\frac{kN}{m^3}$ and $\gamma_w = 9.81\frac{kN}{m^3}$

For soil II: $\gamma_t = 19.81\frac{kN}{m^3}, \gamma_w = 9.81\frac{kN}{m^3}, e_o = 0.765$ and $c_c = 0.230$



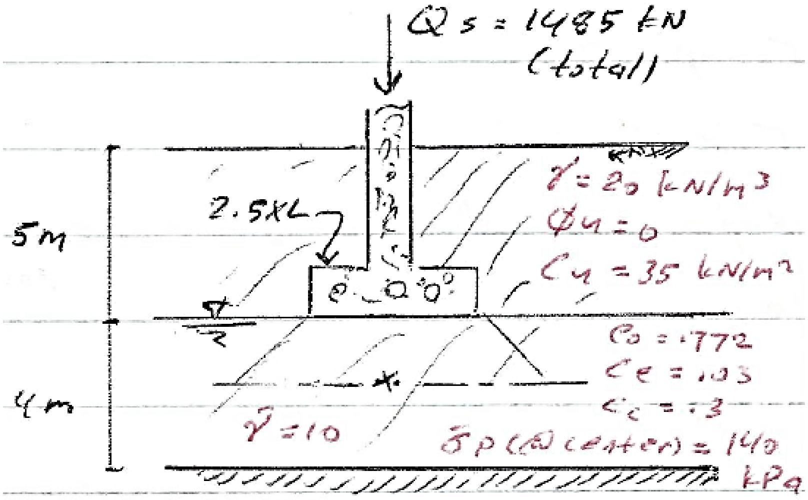
7.4.1.12 Example 7.11:

If the relationship between the time factor and degree of consolidation is $T = \frac{\pi}{4}U^2$, Find the differential settlement between point A and B after one year of construction.



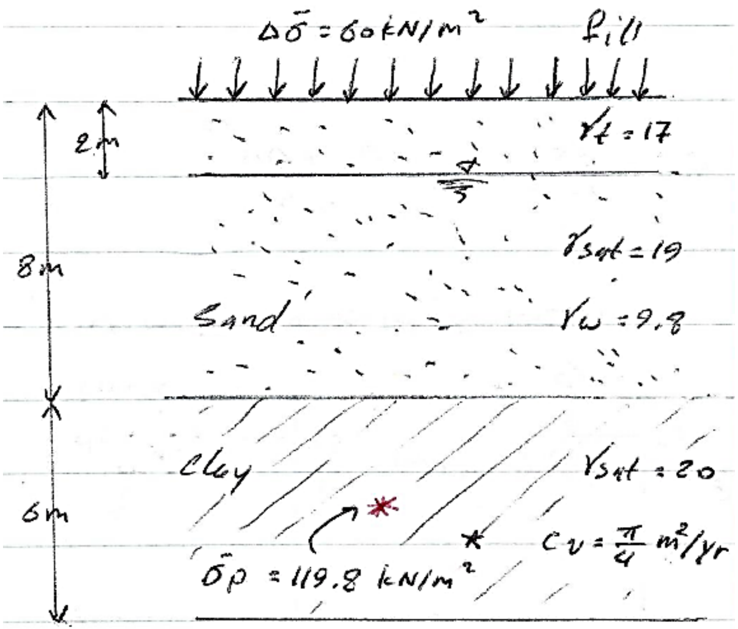
7.4.1.13 Example 7.12:

Find the consolidation settlement of the footing shown in the figure. The factor of safety against shear failure is 3 and the Nc=8.4.



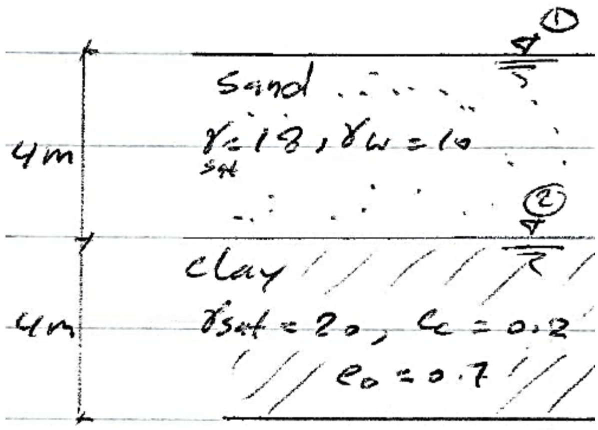
7.4.1.14 Example 7.13:

For the soil profile shown, find the consolidation settlement if $e = 0.88 - 0.32\ \log \frac{\sigma_v{}'}{100}$



7.4.1.15 Example 7.14:

For the soil profile shown, if the water table level lowered from level 1 to level 2, what would be the consolidation settlement due to this process?



7.4.1.16 Example 7.15:

For the square footing shown, find the dimension (B) if the allowable consolidation settlement is 21.0 mm