Concrete retaining walls

Types of retaining walls:-

1. Gravity retaining walls.
2. Semi-gravity retaining walls.
3. Cantilever retaining walls.
4. Counterfort retaining walls.

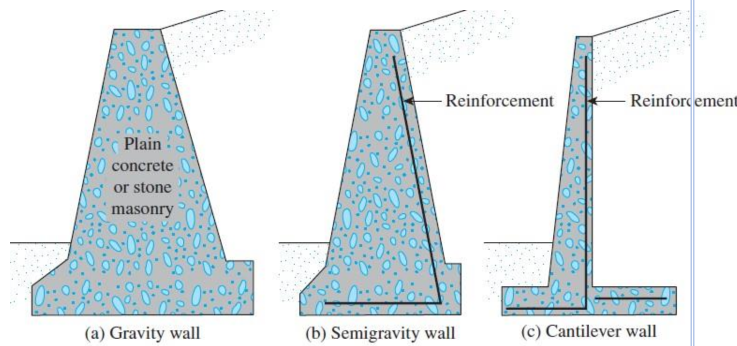
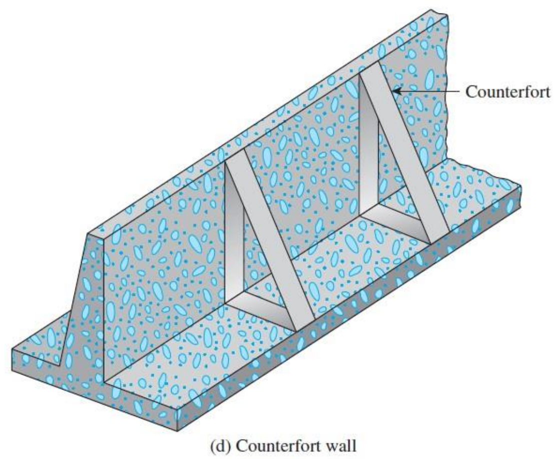


Fig.1 Types of retaining walls



Design considerations

There are two phases in the design of a conventional retaining wall.

1- 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.

2- Each component of the structure is checked for *strength*, and the *steel reinforcement* of each component is determined.

هناك مرحلتان في تصميم الجدار الساندة التقليدية.

1- مع معرفة الضغط الأرضي الجانبي، يتم فحص الهيكل ككل للتأكد من ثباته بحثًا عن حالات الانقلاب والانزلاق وفشل قوة تحمل التربة المحتملة.

2- يتم فحص قوة كل جزء من اجزاء الهيكل، وتصميم حديد التسليح .

Gravity and Cantilever Walls

Proportioning retaining walls

تناسب الجدران الساندة

In designing retaining walls, an engineer must assume some of their dimensions. 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. Fig.2 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 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.*

عند تصميم الجدران الساندة يجب على المهندس أن يفرض بعض أبعادها. تسمى هذه الافتراضات بالتناسب، وهي تسمح للمهندس بفحص الأجزاء المفروضة من الجدران للتأكد من ثباتها. إذا أسفرت عمليات التحقق للاستقرار عن نتائج غير مرغوب فيها، فيمكن تغيير الابعاد وإعادة فحصها. يوضح الشكل 2 النسب العامة لمكونات الجدار الساند المختلفة التي يمكن استخدامها في الفحوصات الأولية. لاحظ أن الجزء العلوي من جذع أي جدار ساند يجب ألا يقل عن حوالي 0.3 متر لوضع الخرسانة بشكل صحيح. يجب أن يكون العمق، D، إلى أسفل القاعدة كحد أدنى، ومع ذلك، يجب وضع الجزء السفلي من لوح القاعدة أسفل خط الصقيع الموسمي. بالنسبة للجدران الساندة المضادة، فإن النسبة العامة للساق والقاعدة هي نفسها بالنسبة للجدران الكابولية. ومع ذلك، قد يبلغ سمك الألواح المضادة حوالي 0.3 مترًا ومتباعدة على مسافات من المركز إلى المركز تتراوح من 0.3 إلى 0.7H.

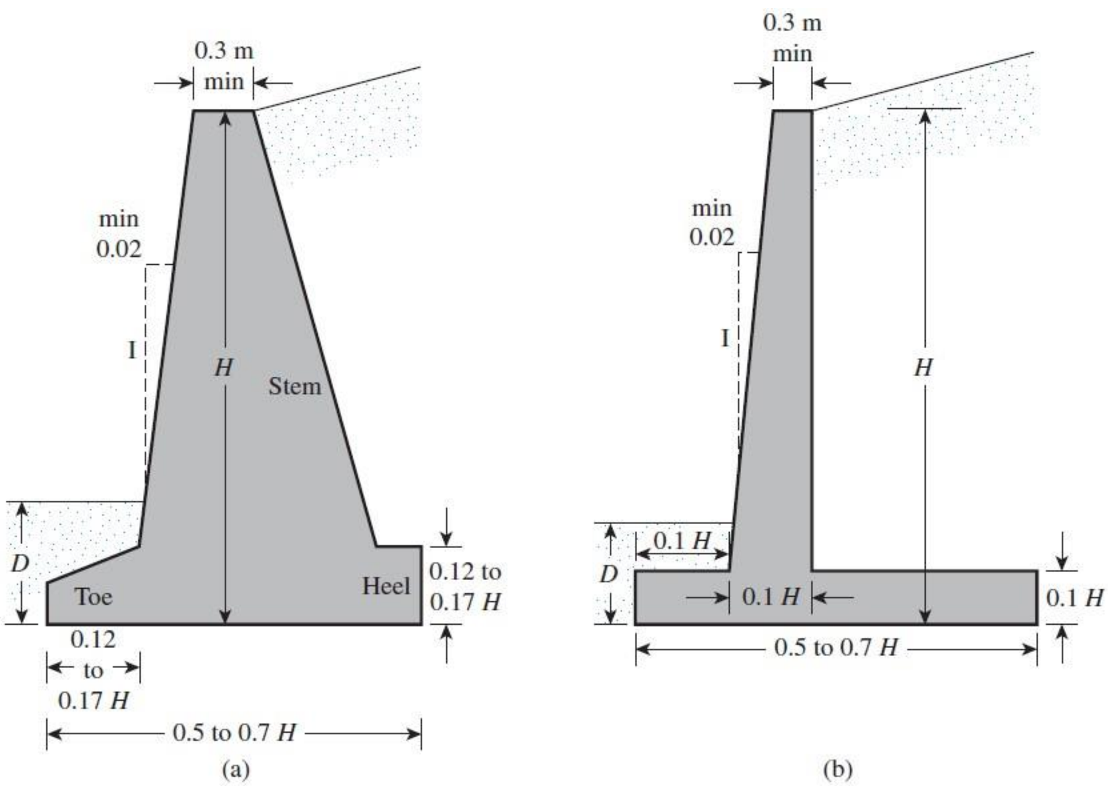


Fig.2 Approximate dimensions for various components of retaining wall for initial stability checks:
(a) gravity wall; (b) cantilever wall

If Coulomb's theory is used, it will be necessary to know the range of the wall friction angle δ with various types of backfill material. Following are some ranges of wall friction angle for masonry or mass concrete walls:

Backfill material	Range of δ' (deg)
Gravel	27–30
Coarse sand	20–28
Fine sand	15–25
Stiff clay	15–20
Silty clay	12–16

In the case of ordinary retaining walls, water table problems and hence hydrostatic pressure are not encountered. Facilities for drainage from the soils that are retained are always provided.

Design criteria

1- Check for overturning.

Figure 8.7 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. P_p is the Rankine passive pressure; recall that its magnitude is [from Eq. (7.63)].

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

where

γ_2 = unit weight of soil in front of the heel and under the base slab
K_p = Rankine passive earth pressure coefficient = $\tan^2(45 + \phi_2'/2)$
c_2', ϕ_2' = cohesion and effective soil friction angle, respectively

The factor of safety against overturning about the toe—that is, about point C in Figure 8.7—may be expressed as

$$FS_{(\text{overturning})} = \frac{\Sigma M_R}{\Sigma M_o} \tag{8.2}$$

where

ΣM_o = sum of the moments of forces tending to overturn about point C
ΣM_R = sum of the moments of forces tending to resist overturning about point C

The overturning moment is

$$\Sigma M_o = P_h\left(\frac{H'}{3}\right) \tag{8.3}$$

where $P_h = P_a\cos\alpha$.

where B = width of the base slab.

Once ΣM_R is known, the factor of safety can be calculated as

$$FS_{(overturning)} = \frac{M_1 + M_2 + M_3 + M_4 + M_5 + M_6 + M_v}{P_a \cos\alpha (H'/3)} \tag{8.5}$$

The usual minimum desirable value of the factor of safety with respect to overturning is 2 to 3.

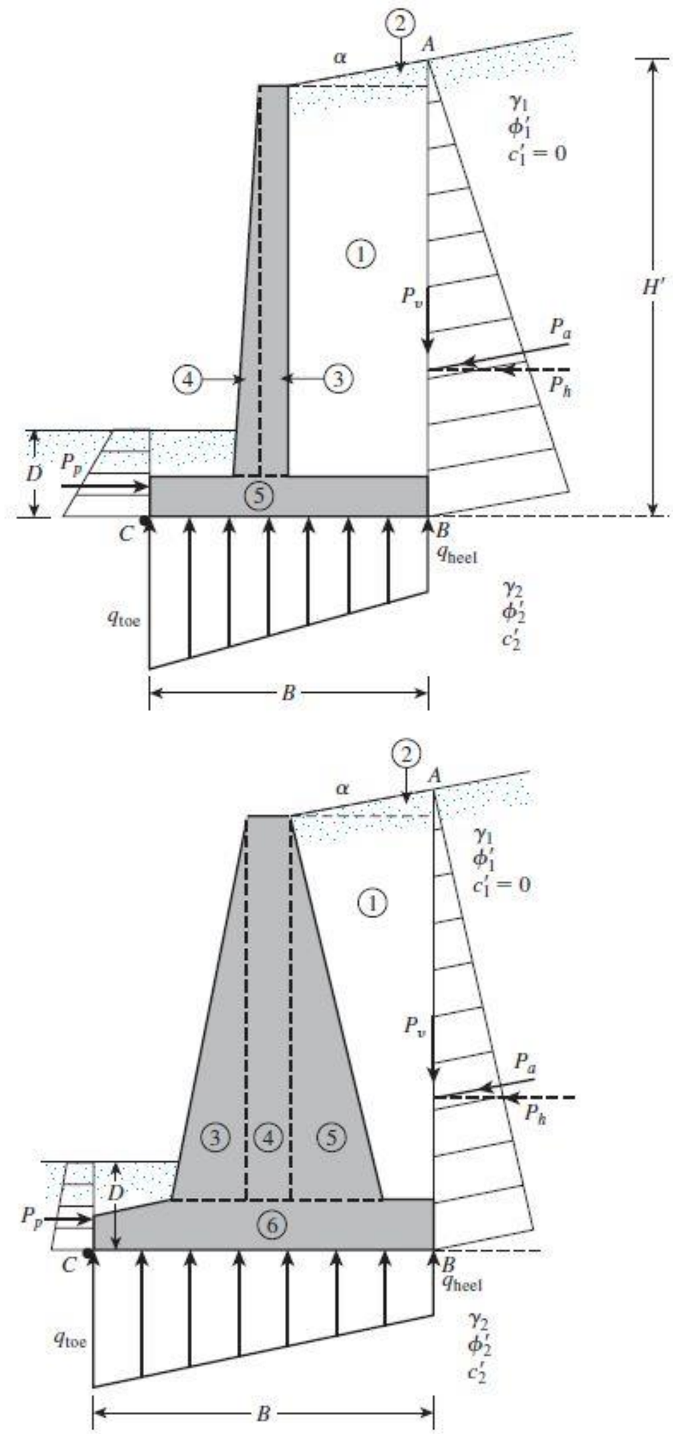


Figure 8.7 Check for overturning, assuming that the Rankine pressure is valid

2- Check for sliding.

$$F_{sliding} = \frac{\sum F_R}{\sum F_d}$$

$F_R = Resisting\ forces, F_d = driving\ forces$

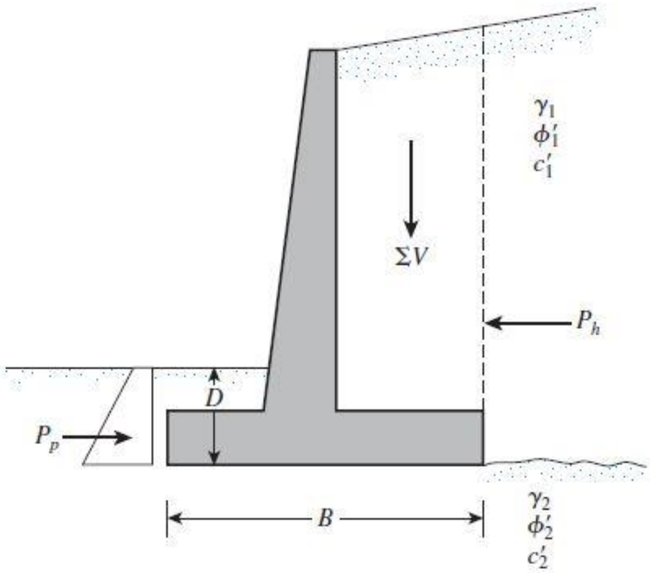


Figure 8.8 Check for sliding along the base

Figure 8.8 shows that the passive force P_p is also a horizontal resisting force. Hence,

$$\Sigma F_{R'} = (\Sigma V)\tan\delta' + Bc'_a + P_p \tag{8.8}$$

The only horizontal force that will tend to cause the wall to slide (a *driving force*) is the horizontal component of the active force P_a, so

$$\Sigma F_d = P_a \cos\alpha \tag{8.9}$$

Combining Eqs. (8.7), (8.8), and (8.9) yields

$$FS_{(\text{sliding})} = \frac{(\Sigma V)\tan\delta' + Bc'_a + P_p}{P_a \cos\alpha} \tag{8.10}$$

A minimum factor of safety of 1.5 against sliding is generally required.

In many cases, the passive force P_p is ignored in calculating the factor of safety with respect to sliding. In general, we can write $\delta' = k_1\phi'_2$ and $c'_a = k_2c'_2$. In most cases, k_1 and k_2 are in the range from $\frac{1}{2}$ to $\frac{2}{3}$. Thus,

$$FS_{(\text{sliding})} = \frac{(\Sigma V)\tan(k_1\phi'_2) + Bk_2c'_2 + P_p}{P_a \cos\alpha} \tag{8.11}$$

If the desired value of $FS_{(sliding)}$ is not achieved, several alternatives may be investigated (see Figure 8.9):

- 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

$$P_p = \frac{1}{2}\gamma_2 D_1^2 K_p + 2c_2' D_1 \sqrt{K_p}$$

where $K_p = \tan^2\left(45 + \frac{\phi_2'}{2}\right)$.

3- Check for bearing capacity.

Check for Bearing Capacity Failure

The vertical pressure transmitted to the soil by the base slab of the retaining wall should be checked against the ultimate bearing capacity of the soil. The nature of variation of the vertical pressure transmitted by the base slab into the soil is shown in Figure 8.11. Note that q_{toe} and q_{heel} are the *maximum* and the *minimum* pressures occurring at the ends of the toe and heel sections, respectively. The magnitudes of q_{toe} and q_{heel} can be determined in the following manner:

The sum of the vertical forces acting on the base slab is Σ **V** (see column 3 of Table 8.1), and the horizontal force $\mathbf{P}_h$ is $P_a \cos\alpha$. Let

$$\mathbf{R} = \Sigma\,\mathbf{V} + \mathbf{P}_h \tag{8.15}$$

be the resultant force. The net moment of these forces about point C in Figure 8.11 is

$$M_{net} = \Sigma M_R - \Sigma M_o \tag{8.16}$$

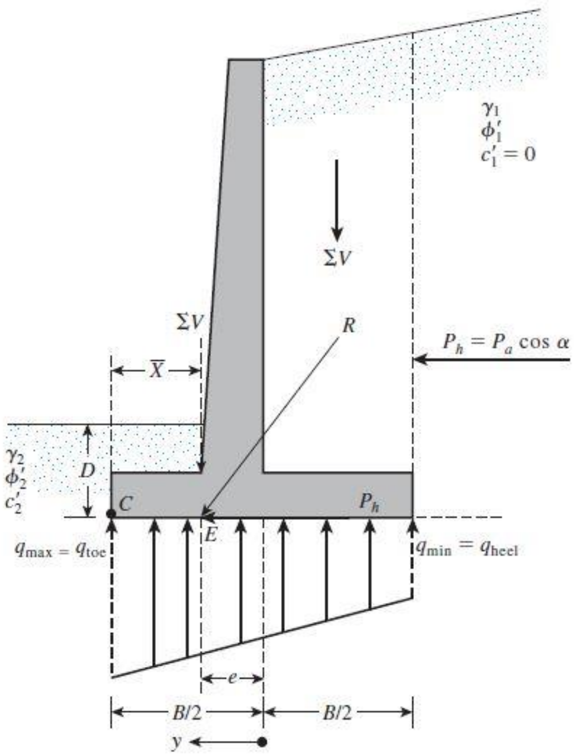


Figure 8.11 Check for bearing capacity failure

Note that the values of ΣM_R and ΣM_o were previously determined. [See Column 5 of Table 8.1 and Eq. (8.3)]. Let the line of action of the resultant R intersect the base slab at E. Then the distance

$$\overline{CE} = \overline{X} = \frac{M_{net}}{\Sigma V} \tag{8.17}$$

Hence, the eccentricity of the resultant R may be expressed as

$$e = \frac{B}{2} - \overline{CE} \tag{8.18}$$

The pressure distribution under the base slab may be determined by using simple principles from the mechanics of materials. First, we have

$$q = \frac{\Sigma V}{A} \pm \frac{M_{net} y}{I} \tag{8.19}$$

where $M_{net} = (\sum V)\ e$
I : moment of inertia per unit length of the base section
$I = (1)B^3/12$

$$q_{\max} = q_{\text{toe}} = \frac{\Sigma V}{(B)(1)} + \frac{e(\Sigma V)\frac{B}{2}}{\left(\frac{1}{12}\right)(B^3)} = \frac{\Sigma V}{B}\left(1 + \frac{6e}{B}\right) \tag{8.20}$$

Similarly,

$$q_{\min} = q_{\text{heel}} = \frac{\Sigma V}{B}\left(1 - \frac{6e}{B}\right) \tag{8.21}$$

Note that ΣV includes the weight of the soil, as shown in Table 8.1, and that when the value of the eccentricity e becomes greater than $B/6$, $q_{\min}$ [Eq. (8.21)] becomes negative. Thus, there will be some tensile stress at the end of the heel section. This stress is not desirable, because the tensile strength of soil is very small. If the analysis of a design shows that $e > B/6$, the design should be reproportioned and calculations redone.

The relationships pertaining to the ultimate bearing capacity of a shallow foundation were discussed in Chapter 3. Recall that [Eq. (3.40)].

$$q_u = c_2' N_c F_{cd} F_{ci} + q N_q F_{qd} F_{qi} + \tfrac{1}{2}\gamma_2 B' N_\gamma F_{\gamma d} F_{\gamma i} \tag{8.22}$$

where

$$q = \gamma_2 D$$

$$B' = B - 2e$$

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan \phi_2'}$$

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

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

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

$$F_{\gamma i} = \left(1 - \frac{\psi^\circ}{\phi_2'^\circ}\right)^2$$

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

Note that the shape factors F_{cs}, F_{qs}, and $F_{\gamma s}$ given in Chapter 3 are all equal to unity, because they can be treated as a continuous foundation. For this reason, the shape factors are not shown in Eq. (8.22).

Once the ultimate bearing capacity of the soil has been calculated by using Eq. (8.22), the factor of safety against bearing capacity failure can be determined:

$$\text{FS}_{(\text{bearing capacity})} = \frac{q_u}{q_{\max}} \tag{8.23}$$

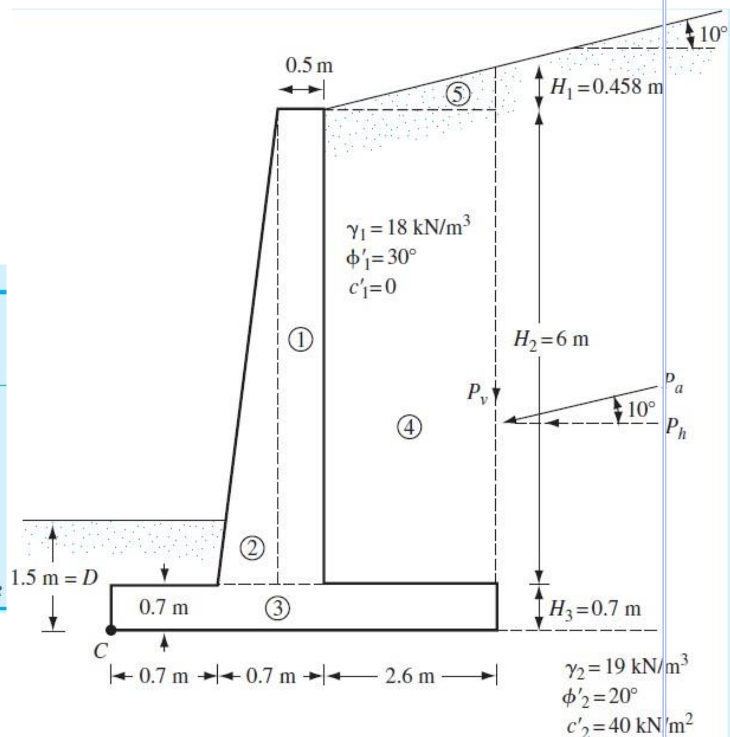
Generally, a factor of safety of 3 is required. In Chapter 3, we noted that the ultimate bearing capacity of shallow foundations occurs at a settlement of about 10% of the foundation width.

4- Check for total settlement.

Design Example:

A full design of the retaining wall is required.

Factor safety for overturning moment



$H = 0.7 + 6 + 2.6tan10 = 7.158m$

For $\emptyset = 30\ and\ \alpha = 10,\ ka = 0.3495$

$P_a = k_a \frac{1}{2} \gamma h^2 = 0.3495 * \frac{1}{2} * 18 * 7.158^2 = 162.9KN/m$

$P_h = 162.9\cos(10) = 160.4\ KN/m$

$P_V = 160.9\sin(10) = 28.29KN/m$

The following table can now be prepared for determining the resisting moment:

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

$\gamma_{cr} = 23.58$ kN/m

$$Overturning\ moment = 160.4 * \frac{7.158}{3} = 382.7\ KN.m/m$$

$$F_s(overturning) = \frac{1130.02}{382.7} = 2.95$$

Factor of safety for sliding

$$k_p = \tan^2(45 + \frac{\emptyset}{2}) = 2.04$$

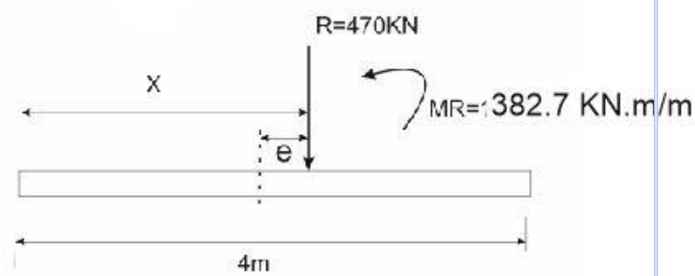
$$P_p = \frac{1}{2} k_p \gamma D^2 + 2C\sqrt{k_p}D = \frac{1}{2} * 2.04 * 19 * 1.5^2 + 2 * 40 * \sqrt{2.04} * 1.5 = 215KN/m$$

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

$$Fs(sliding) = \frac{470.71\tan\left(\frac{2}{3}*20\right) + 4*\left(\frac{2}{3}\right)40 + 215}{162.4} = 2.7 > 2\ ok$$

Factor of Safety against Bearing Capacity Failure

$X = \frac{\sum M}{R} = \frac{1130}{470} = 2.4m$

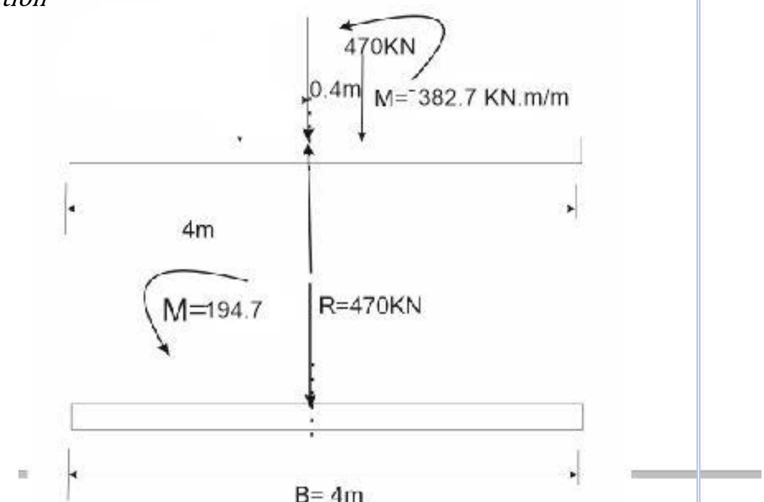


Eccentricity analysis

$\sum M_{o} = 0$

$RX = M_{net}, \quad 470X = 1130, \quad X = 2.4, \;\; e = \frac{B}{2} - X$

$= \frac{4}{2} - 2.4 = 0.4 < \frac{B}{6} = \frac{4}{6} = 0.66m$ *good no negative soil reaction*



$q = \frac{\sum V}{A} \mp \frac{My}{I}$, Let $\sum V = R$, or

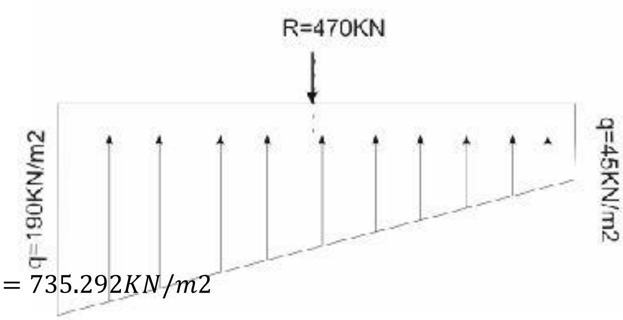
$$q = \frac{\sum V}{BL} \mp \frac{My}{I}$$

$$I = \frac{1 * 4^3}{12} = 5.33m4$$

$$= \frac{470}{4*1} + \frac{194.7*2}{5.33}$$

$$= 117.5 \mp 72.79$$

$q1 = 190KN/m2 \, , q2 = 44.7KN/m2$



Bearing capacity by Terzaghi

$q_u = 1.3CN_C + qN_q + 0.4\gamma BN_\gamma$

For $\emptyset = 20, \;\; N_c = 11.85, \quad N_q = 3.88, \quad N_\gamma = 1.12$

$q_u = 1.3 * 40 * 11.85 + 19 * 1.5 * 3.88 + 0.4 * 19 * 1 * 1.12 = 735.292KN/m2$

$F_s = \frac{735.292}{190} = 3.8$ oK

Consolidation settlement

Let $H = 1.5\,B = 1.5 * 4m = 6m$

$$S = \frac{C_C H}{1+e_O} log \frac{\sigma+\Delta\sigma}{\sigma}$$

H = 6m, $C_C = 0.2$, $e_o = 0.95$,

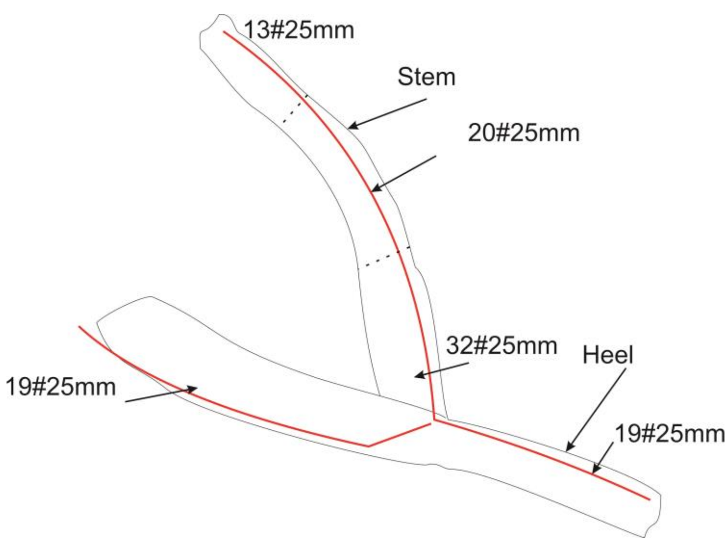
$$\sigma = 7.158m * 19 + 3 * 18 = 190KN/m2$$

$$q = \frac{q_1+q_2}{2} = \frac{190+45}{2} = 117.5\ KN/m2$$

$$\Delta\sigma = \frac{117*4*1}{(4+3)(1+3)} = 16.7KN/m$$

$$S = \frac{0.1*6}{1+0.95} log \frac{190+16.7}{190} = 0.0112m = 11mm < 25mm \quad ok.$$

Reinforcement Design



Design of Stem

Mu on the stem $=162.9kN * 2.39m = 389\ KN.M/m$

$$\rho = \frac{A_S}{bd}$$

$$M_U = \emptyset\rho b d^2 f_y (1 - \frac{\rho f_y}{1.7 f_C}),$$

$$M = \frac{162.9(7.158-0.7)}{3} = 350\ KN.m$$

$$1.7 * 350 = 0.85 * \rho * 1 * 1.2^2 * 430 * 1000(1 - \frac{\rho*430}{1.7*30})$$

$0.00113 = \rho(1 - 8.4\rho)$, $\rho = 0.00128$, $0.00128 = \frac{A_S}{1*1.2}$, $A_S = 0.001536m2 = 14254mm2$

As min = 0.0018*1*1.2=0.014254m2 =14254 mm2,

Use 29#25 mm/ 1m $\perp$ paper

Design of Toe

$$M_U = 45 * \frac{0.7^2}{2} + \frac{(190-45)}{4} * 0.7 * \frac{0.7}{2} * \frac{0.7}{3} = 13KN.m/m$$

$$1.7 * 13 = 0.85 * \rho * 1 * 0.7^2 * 428 * 1000(1 - \frac{\rho * 428}{1.7 * 28})$$

$0.00012397 = \rho(1 - 9\rho)$, $\rho = 0.000125$, $0.000125 = \frac{A_S}{1*0.7}$, $A_S = 0.0000874m2 = 87mm2$

AS min=0.013*0.7=0.0091m2=9100mm2, use 19#25

Design of Heel

$$M_U = 45 * \frac{2.6^2}{2} + \left(\frac{145}{4} * 1.4\right) * \frac{2.6^2}{2} + 94.25 * \frac{2.6^2}{2} * \frac{2}{3} = 536KN.m$$

$$M_U = \emptyset \rho b d^2 f_y (1 - \frac{\rho f_y}{1.7 f_C}),$$

$$1.7 * 536 = 0.85 * \rho * 1 * 0.7^2 * 428 * 1000(1 - \frac{\rho * 428}{1.7 * 28})$$

$0.00511 = \rho(1 - 9\rho)$, $\rho = 0.0055$, $0.0055 = \frac{A_S}{1*0.7}$, $A_S = 0.00385m2 = 3850mm2$

As min = 0.013*1*0.7=0.0156m2 =9100 mm2,

Use 19#25 mm/ 1m $\perp$ paper

Home work

Solve Problems, 8.1, 8.2, 8.3 page 433

Table 3.2 Terzaghi's Modified Bearing Capacity Factors N_c', N_q', and N_γ'

ϕ'	N_c'	N_q'	N_γ'	ϕ'	N_c'	N_q'	N_γ'
0	5.70	1.00	0.00	26	15.53	6.05	2.59
1	5.90	1.07	0.005	27	16.30	6.54	2.88
2	6.10	1.14	0.02	28	17.13	7.07	3.29
3	6.30	1.22	0.04	29	18.03	7.66	3.76
4	6.51	1.30	0.055	30	18.99	8.31	4.39
5	6.74	1.39	0.074	31	20.03	9.03	4.83
6	6.97	1.49	0.10	32	21.16	9.82	5.51
7	7.22	1.59	0.128	33	22.39	10.69	6.32
8	7.47	1.70	0.16	34	23.72	11.67	7.22
9	7.74	1.82	0.20	35	25.18	12.75	8.35
10	8.02	1.94	0.24	36	26.77	13.97	9.41
11	8.32	2.08	0.30	37	28.51	15.32	10.90
12	8.63	2.22	0.35	38	30.43	16.85	12.75
13	8.96	2.38	0.42	39	32.53	18.56	14.71
14	9.31	2.55	0.48	40	34.87	20.50	17.22
15	9.67	2.73	0.57	41	37.45	22.70	19.75
16	10.06	2.92	0.67	42	40.33	25.21	22.50
17	10.47	3.13	0.76	43	43.54	28.06	26.25
18	10.90	3.36	0.88	44	47.13	31.34	30.40
19	11.36	3.61	1.03	45	51.17	35.11	36.00
20	11.85	3.88	1.12	46	55.73	39.48	41.70
21	12.37	4.17	1.35	47	60.91	44.45	49.30
22	12.92	4.48	1.55	48	66.80	50.46	59.25
23	13.51	4.82	1.74	49	73.55	57.41	71.45
24	14.14	5.20	1.97	50	81.31	65.60	85.75
25	14.80	5.60	2.25				

α (deg)	ϕ' (deg) →												
↓	28	29	30	31	32	33	34	35	36	37	38	39	40
0	0.3610	0.3470	0.3333	0.3201	0.3073	0.2948	0.2827	0.2710	0.2596	0.2486	0.2379	0.2275	0.2174
1	0.3612	0.3471	0.3335	0.3202	0.3074	0.2949	0.2828	0.2711	0.2597	0.2487	0.2380	0.2276	0.2175
2	0.3618	0.3476	0.3339	0.3207	0.3078	0.2953	0.2832	0.2714	0.2600	0.2489	0.2382	0.2278	0.2177
3	0.3627	0.3485	0.3347	0.3214	0.3084	0.2959	0.2837	0.2719	0.2605	0.2494	0.2386	0.2282	0.2181
4	0.3639	0.3496	0.3358	0.3224	0.3094	0.2967	0.2845	0.2726	0.2611	0.2500	0.2392	0.2287	0.2186
5	0.3656	0.3512	0.3372	0.3237	0.3105	0.2978	0.2855	0.2736	0.2620	0.2508	0.2399	0.2294	0.2192
6	0.3676	0.3531	0.3389	0.3253	0.3120	0.2992	0.2868	0.2747	0.2631	0.2518	0.2409	0.2303	0.2200
7	0.3701	0.3553	0.3410	0.3272	0.3138	0.3008	0.2883	0.2761	0.2644	0.2530	0.2420	0.2313	0.2209
8	0.3730	0.3580	0.3435	0.3294	0.3159	0.3027	0.2900	0.2778	0.2659	0.2544	0.2432	0.2325	0.2220
9	0.3764	0.3611	0.3463	0.3320	0.3182	0.3049	0.2921	0.2796	0.2676	0.2560	0.2447	0.2338	0.2233
10	0.3802	0.3646	0.3495	0.3350	0.3210	0.3074	0.2944	0.2818	0.2696	0.2578	0.2464	0.2354	0.2247
11	0.3846	0.3686	0.3532	0.3383	0.3241	0.3103	0.2970	0.2841	0.2718	0.2598	0.2482	0.2371	0.2263
12	0.3896	0.3731	0.3573	0.3421	0.3275	0.3134	0.2999	0.2868	0.2742	0.2621	0.2503	0.2390	0.2281
13	0.3952	0.3782	0.3620	0.3464	0.3314	0.3170	0.3031	0.2898	0.2770	0.2646	0.2527	0.2412	0.2301
14	0.4015	0.3839	0.3671	0.3511	0.3357	0.3209	0.3068	0.2931	0.2800	0.2674	0.2552	0.2435	0.2322
15	0.4086	0.3903	0.3729	0.3564	0.3405	0.3253	0.3108	0.2968	0.2834	0.2705	0.2581	0.2461	0.2346
16	0.4165	0.3975	0.3794	0.3622	0.3458	0.3302	0.3152	0.3008	0.2871	0.2739	0.2612	0.2490	0.2373
17	0.4255	0.4056	0.3867	0.3688	0.3518	0.3356	0.3201	0.3053	0.2911	0.2776	0.2646	0.2521	0.2401
18	0.4357	0.4146	0.3948	0.3761	0.3584	0.3415	0.3255	0.3102	0.2956	0.2817	0.2683	0.2555	0.2433
19	0.4473	0.4249	0.4039	0.3842	0.3657	0.3481	0.3315	0.3156	0.3006	0.2862	0.2724	0.2593	0.2467
20	0.4605	0.4365	0.4142	0.3934	0.3739	0.3555	0.3381	0.3216	0.3060	0.2911	0.2769	0.2634	0.2504
21	0.4758	0.4498	0.4259	0.4037	0.3830	0.3637	0.3455	0.3283	0.3120	0.2965	0.2818	0.2678	0.2545
22	0.4936	0.4651	0.4392	0.4154	0.3934	0.3729	0.3537	0.3356	0.3186	0.3025	0.2872	0.2727	0.2590
23	0.5147	0.4829	0.4545	0.4287	0.4050	0.3832	0.3628	0.3438	0.3259	0.3091	0.2932	0.2781	0.2638
24	0.5404	0.5041	0.4724	0.4440	0.4183	0.3948	0.3731	0.3529	0.3341	0.3164	0.2997	0.2840	0.2692
25	0.5727	0.5299	0.4936	0.4619	0.4336	0.4081	0.3847	0.3631	0.3431	0.3245	0.3070	0.2905	0.2750