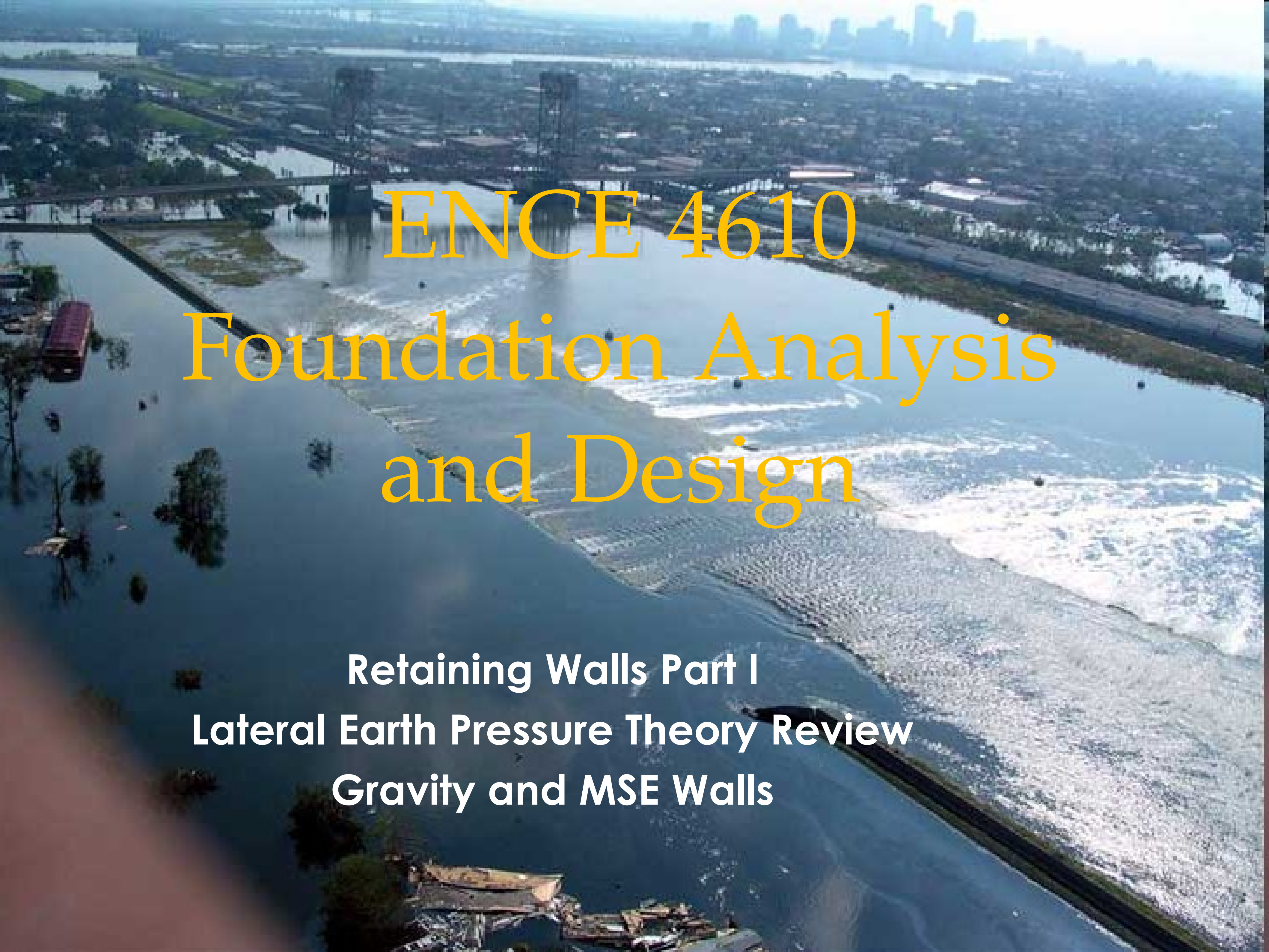


ENCE 4610 Foundation Analysis and Design

Retaining Walls Part I

Lateral Earth Pressure Theory Review

Gravity and MSE Walls

Outline of Retaining Wall Material

- Part I
  - Review of Lateral Earth Pressures
  - Concrete Gravity Retaining Walls
  - MSE (Mechanically Stabilised Earth) Walls
- Part II: Sheet Pile Walls
  - Cantilever Walls
  - Anchored Walls
  - Braced Cuts
- Reference Materials
  - Verruijt Chapters 32-38
  - FHWA NHI-06-089 Chapter 10

Retaining Walls

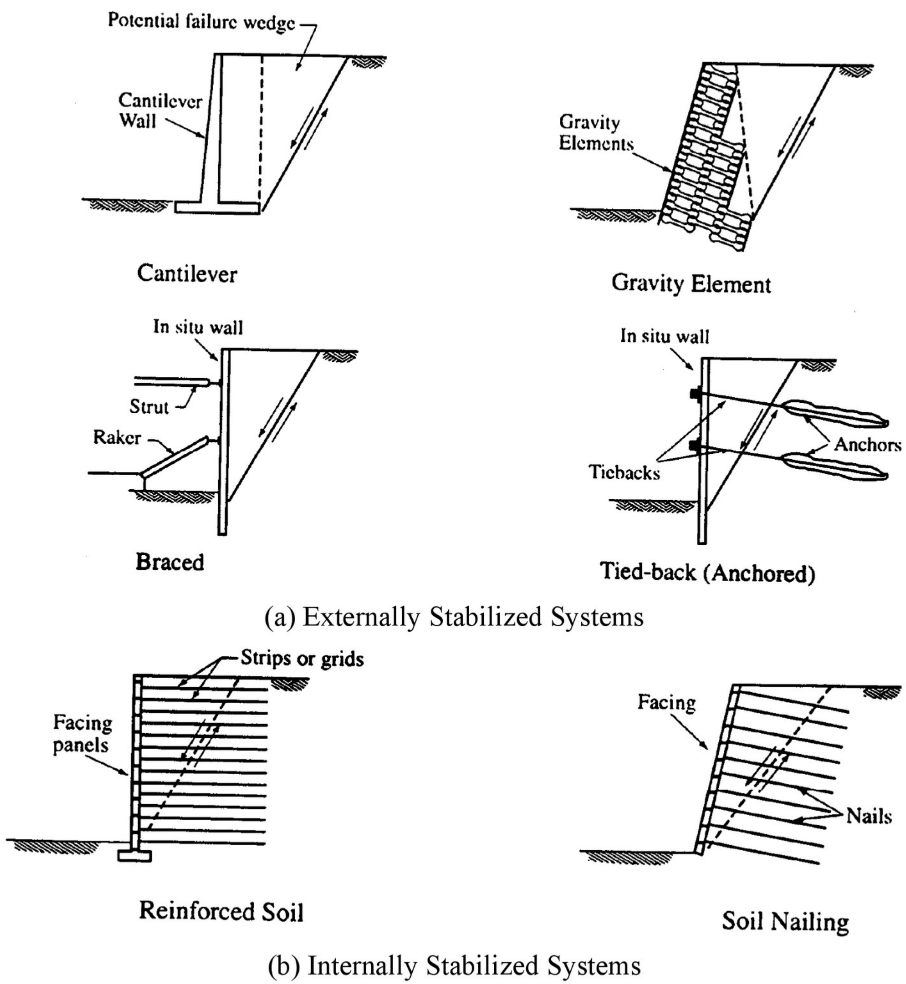
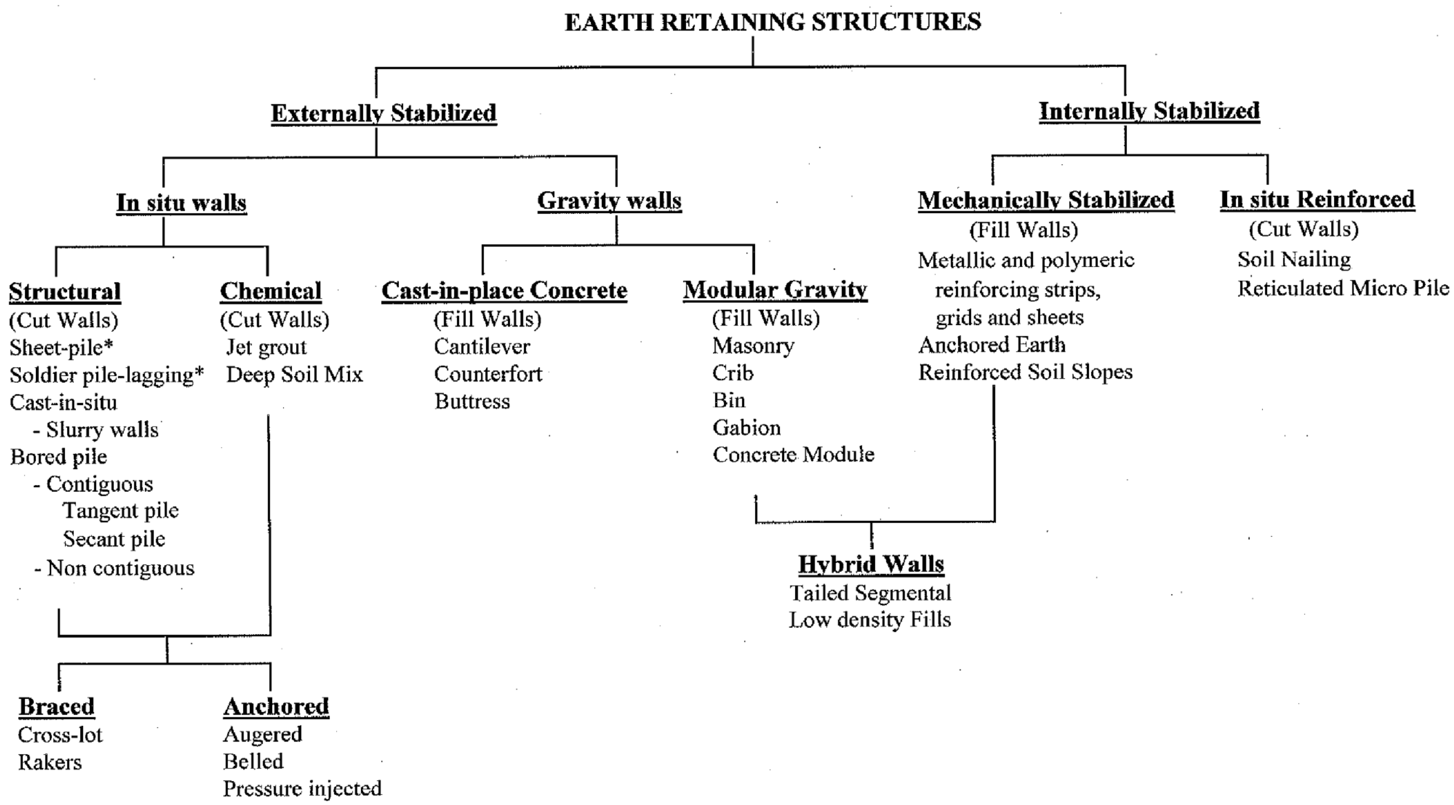


Figure 10-2. Variety of retaining walls (after O'Rourke and Jones, 1990)

- Necessary in situations where gradual transitions either take up too much space or are impractical for other reasons
- Retaining walls are analysed for both resistance to overturning and structural integrity
- Two categories of retaining walls
  - Externally Stabilized
  - Internally Stabilized

Types of Retaining Walls



* can also be used in fill conditions

Figure 10-3. Classification of earth retaining systems (after O'Rourke and Jones, 1990).

Lateral Earth Pressure Topics: States and Theories

- States of Lateral Earth Pressure
  - At-Rest (Neutral) Earth Pressure
  - Active Earth Pressure
  - Passive Earth Pressure
  - It's possible to have in-between states, but we usually don't discuss these in elementary retaining wall design

- Theories of Lateral Earth Pressure
  - Jaky (At-Rest Pressures)
  - Planar Failure Surface
    - Rankine (Active, Passive)
    - Coulomb (Active, Passive)
  - Non-Planar Failure Surface
    - Log-Spiral Theory (Active, Passive)

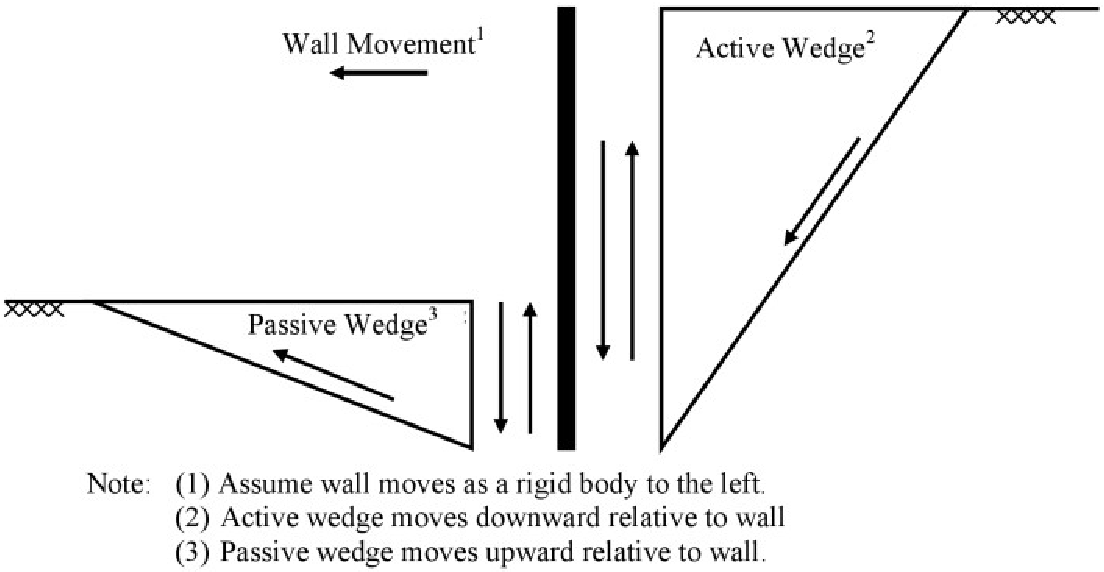


Figure 10-7. Wall friction on soil wedges (after Padfield and Mair, 1984)

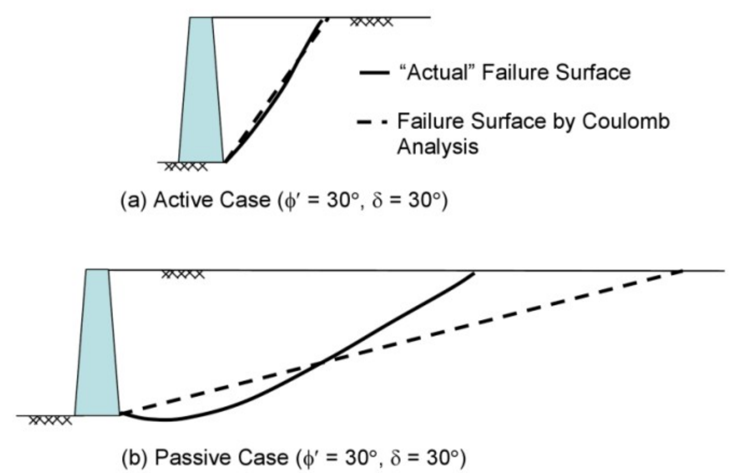


Figure 10-8. Comparison of plane and log-spiral failure surfaces (a) Active case and (b) Passive case (after Sokolovski, 1954) – Note: Depiction of gravity wall is for illustration purpose only.

Lateral Earth Pressure Coefficient

32.1 Coefficient of lateral earth pressure

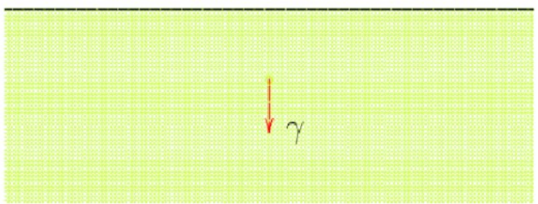


Figure 32.1: Half space.

As stated before, see Chapter 5, even in the simplest case of a semi-infinite soil body, without surface loading, see Figure 32.1, it is impossible to determine all stresses caused by the weight of the soil. It seems reasonable to assume that in a homogeneous soil body with a horizontal top surface the shear stresses σ_{zx}, σ_{zy} and σ_{xy} are zero, and it also seems natural to assume that the vertical normal stress σ_{zz} increases linearly with depth, $\sigma_{zz} = \gamma z$. These assumptions ensure that the condition of vertical equilibrium is satisfied. The horizontal stresses σ_{xx} and σ_{yy}, however, can not be determined unequivocally from the equilibrium conditions.

The stress state described by equations (32.7) – (32.11) can be made somewhat more practical by writing $f(z) = K\sigma_{zz}$, where K is an unknown coefficient, that may depend upon the vertical coordinate z. The horizontal stresses then are

$$\sigma_{xx} = \sigma_{yy} = K\sigma_{zz} = K\gamma z, \tag{32.12}$$

where K is the *coefficient of lateral earth pressure*. It gives the ratio of the lateral normal (effective) stress to the vertical (effective) stress. Theoretically speaking, the problem has not be cleared, because the value of K is still unknown, but it seems to make sense to assume that the horizontal stresses will also increase with depth, if the vertical stresses do so. Thus, it can be assumed that the coefficient K will not vary too much, at least compared to the original function $f(z)$.

A possible approach to the behavior of soils is to consider it as an elastic material. In such a material the stresses and strains satisfy Hooke's law. In a situation in which there can be no lateral deformation, the stresses must satisfy the condition

$$\varepsilon_{xx} = -\frac{1}{E}[\sigma_{xx} - \nu(\sigma_{yy} + \sigma_{zz})] = 0,$$

$$\varepsilon_{yy} = -\frac{1}{E}[\sigma_{yy} - \nu(\sigma_{zz} + \sigma_{xx})] = 0,$$

if the z-direction is vertical. In a medium of large horizontal extent it can be expected that $\sigma_{xx} = \sigma_{yy}$. Then

$$\varepsilon_{xx} = \varepsilon_{yy} = 0 : \quad \sigma_{xx} = \sigma_{yy} = \frac{\nu}{1-\nu}\sigma_{zz}, \tag{32.18}$$

or

$$K = \frac{\nu}{1-\nu}. \tag{32.19}$$

If Poisson's ratio varies between 0 and 0.5, the value of K varies from 0 to 1.

At-Rest (Neutral) Earth Pressures

33.4 Neutral earth pressure

It has been found that the possible states of stress in a soil may vary between fairly wide limits, especially if the friction angle is large. For a normal sand, with $\phi = 30^\circ$, the smallest value of the horizontal stress is $\frac{1}{3}$ of the vertical stress (which usually is known from the surcharge and the weight of the overlying soil), and the largest value is 3 times the vertical stress. In case of a rigid retaining wall, the lateral stress against the wall is unknown, at least from a strictly scientific viewpoint. In reality there may be some additional information that may help to determine the probable range of the horizontal stress. If the horizontal displacements of the wall are practically zero, the ratio of the horizontal stress to the vertical stress is denoted as the *neutral earth pressure coefficient* K_0. In an elastic material this value would be $K_0 = \nu/(1-\nu)$, but this is not a very good estimate, as soil is not an elastic material, and the history of the development of stresses in the soil may be more important than the condition of zero lateral deformation. Nevertheless, in many cases it is unlikely that the coefficient K_0 would be larger than 1, as this would require some form of motion of the wall towards the soil mass. Also, the active state of stress (say $\frac{1}{3}$) may also be unlikely, if the wall is rather stiff and strong. All this suggests that the neutral stress coefficient may perhaps vary in the range from 0.5 to 1.0. For soft clay the value may be close to 1, and for sands values of about 0.6 or 0.7 have been found to give reasonable results. Lacking any better information the value may be estimated by the formula proposed in 1938 by the German engineer J. Jaky,

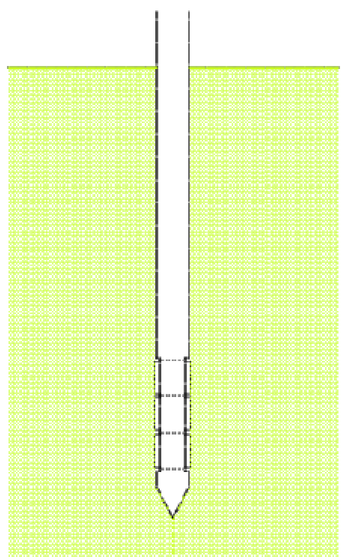


Figure 33.8: CAMKO-meter.

$$K_0 = 1 - \sin\phi, \tag{33.16}$$

but there is no real basis for this formula, except that it gives values between 0 and 1, with the value being close to 1 if the friction angle ϕ is very small (as it is for soft clays).

Active Earth Pressures

33.2 Active earth pressure

It can be expected that the smallest value of the horizontal stress occurs in the case of a retaining wall that is moving away from the soil, see Figure 33.3. The Mohr circle for that case is also shown in the figure. The pole for the vectors normal to the planes is the rightmost point of the circle. This means that the critical shear stress acts on planes making an angle $\frac{1}{4}\pi + \frac{1}{2}\phi$ with the horizontal direction, that is an angle of $\frac{1}{4}\pi - \frac{1}{2}\phi$ with the vertical direction. These planes have been indicated in the left half of Figure 33.3. It is sometimes imagined that the soil indeed slides along these planes in case of failure.

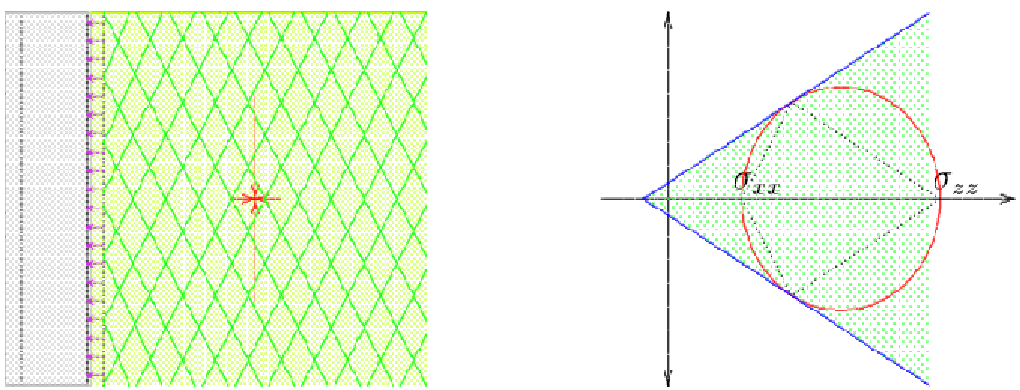


Figure 33.3: Active earth pressure.

The vertical stresses along the wall are

$$\sigma_{zz} = \gamma z, \tag{33.9}$$

in which γ is the volumetric weight of the soil, and z is the depth below soil surface. The horizontal stresses now are, with (33.3),

$$\sigma_{xx} = K_a \gamma z - 2c\sqrt{K_a}. \tag{33.10}$$

The total horizontal force on a wall of height h is obtained by integration from $z = 0$ to $z = h$. This gives

$$Q = \tfrac{1}{2} K_a \gamma h^2 - 2ch\sqrt{K_a}. \tag{33.11}$$

Passive Earth Pressures

33.3 Passive earth pressure

The case of passive earth pressure, in which the horizontal soil stress has its maximum value, can be considered to correspond to a smooth vertical wall that is being pushed in horizontal direction into the soil, see Figure 33.6. Again the Mohr circle has been shown in the figure as well, with the pole in this case being located in the leftmost point of the circle. The critical shear stress $\tau = \tau_f = c + \sigma \tan\phi$ occurs on planes making an angle $\frac{1}{4}\pi - \frac{1}{2}\phi$ with the horizontal direction. These planes have also been indicated in the left half of the figure. In this case the potential sliding planes are shallower than 45°.

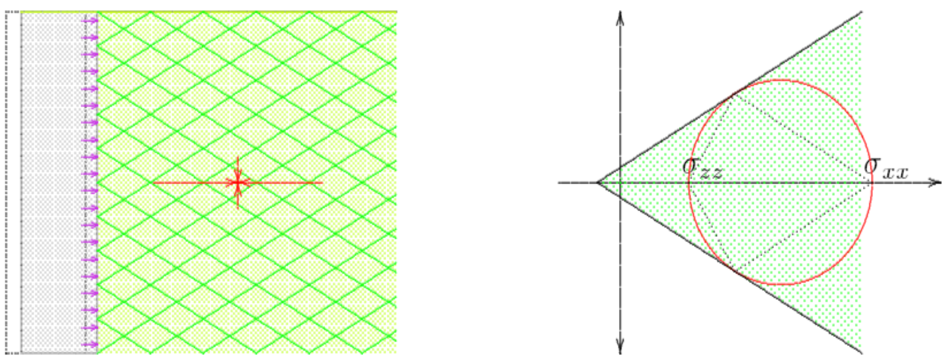


Figure 33.6: Passive earth pressure.

In this case the horizontal stresses are

$$\sigma_{xx} = K_p \gamma z + 2c\sqrt{K_p}. \tag{33.14}$$

The total horizontal force on a wall of height h is obtained by integration of the horizontal stresses from $z = 0$ to $z = h$. This gives

$$Q = \tfrac{1}{2} K_p \gamma h^2 + 2ch\sqrt{K_p}. \tag{33.15}$$

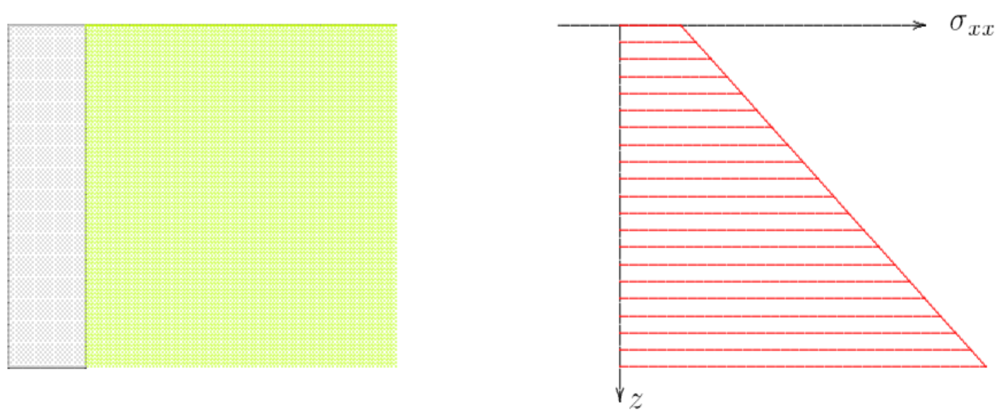


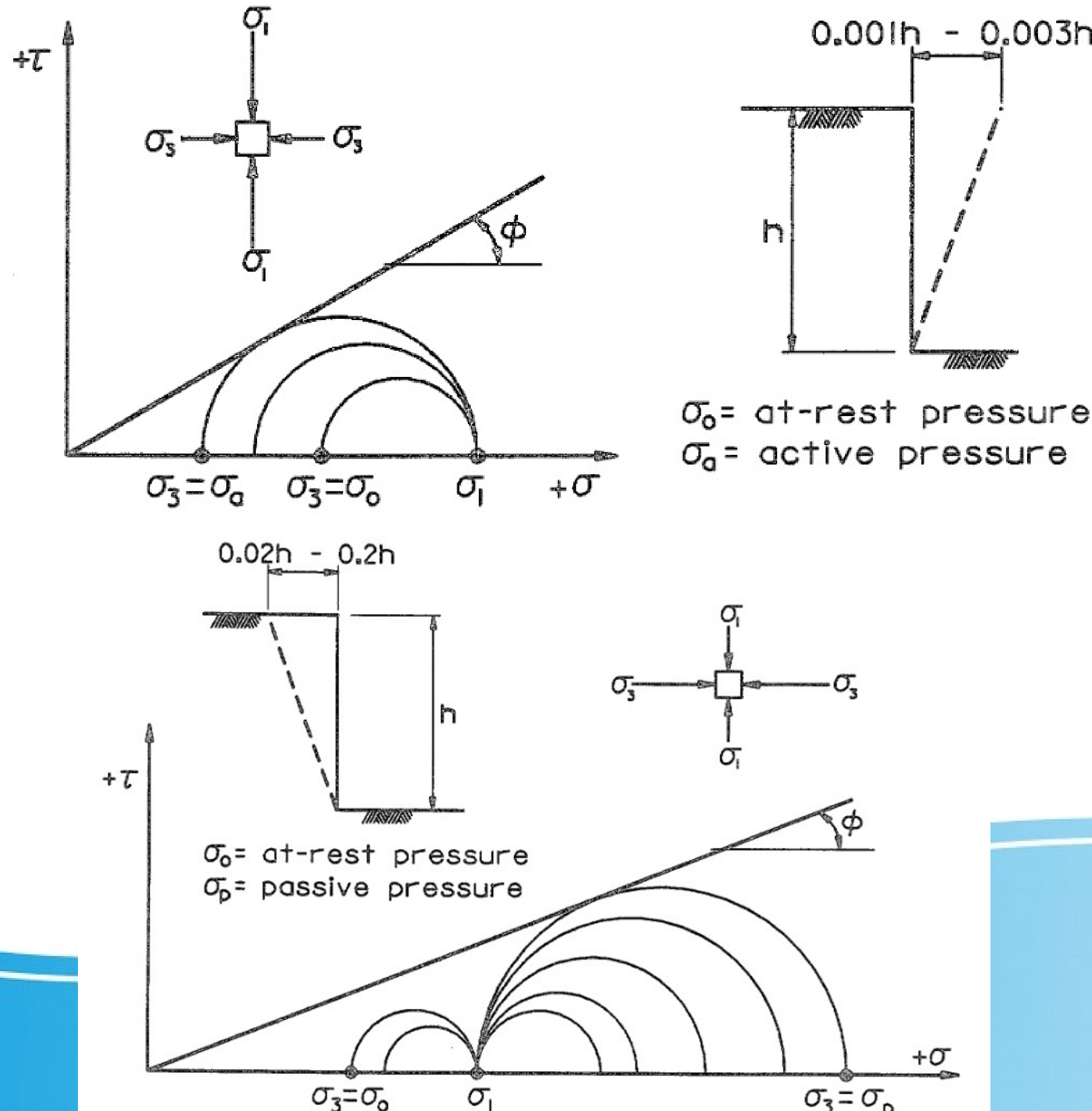
Figure 33.7: Horizontal stresses in case of passive earth pressure.

In the passive case the cohesion c appears to lead to a constant factor in the expression for the horizontal stresses. There is no reason for cracks to appear, as there are no tensile stresses in this case.

The two extreme states of stress considered here are often denoted as the *Rankine states*, after the Scottish

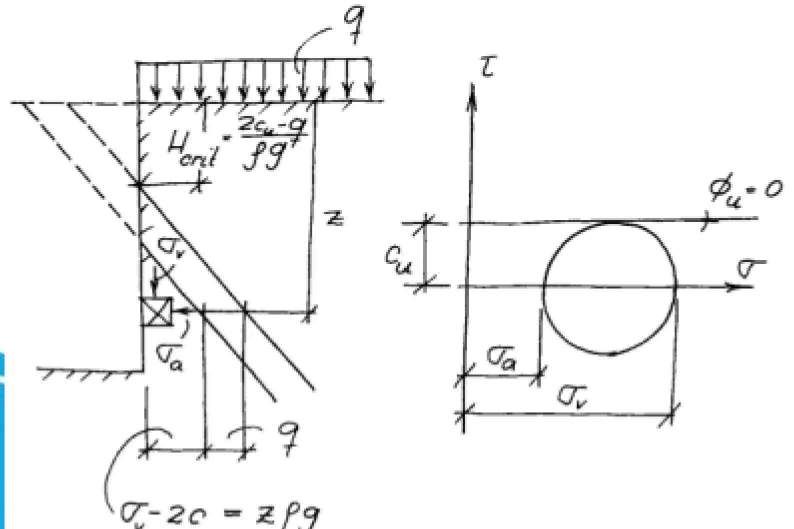
Effect of Wall Movement

Range of Values



Rankine Earth Pressure

- Assumes Mohr-Coulomb failure conditions for the soil
- Does not include effects of wall-soil friction
- Can be extended to include sloping backfill
- Also includes provision for cohesive soils (see chart at right, below for purely cohesive soil with surcharge)



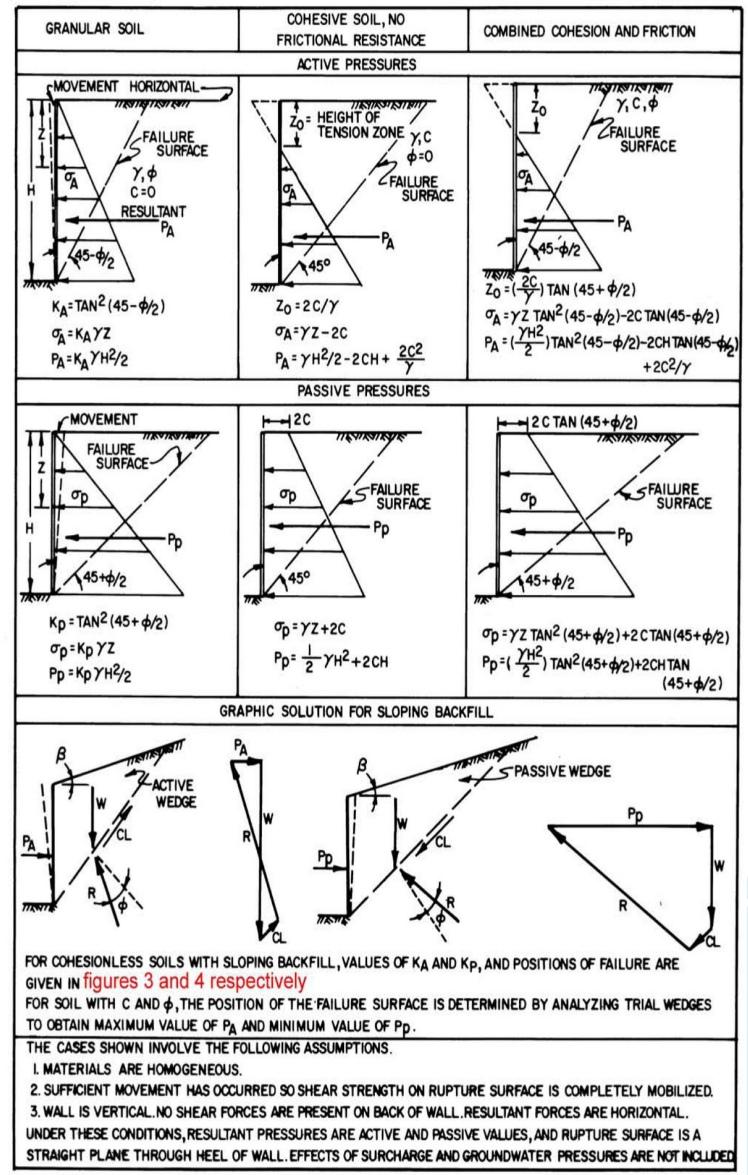
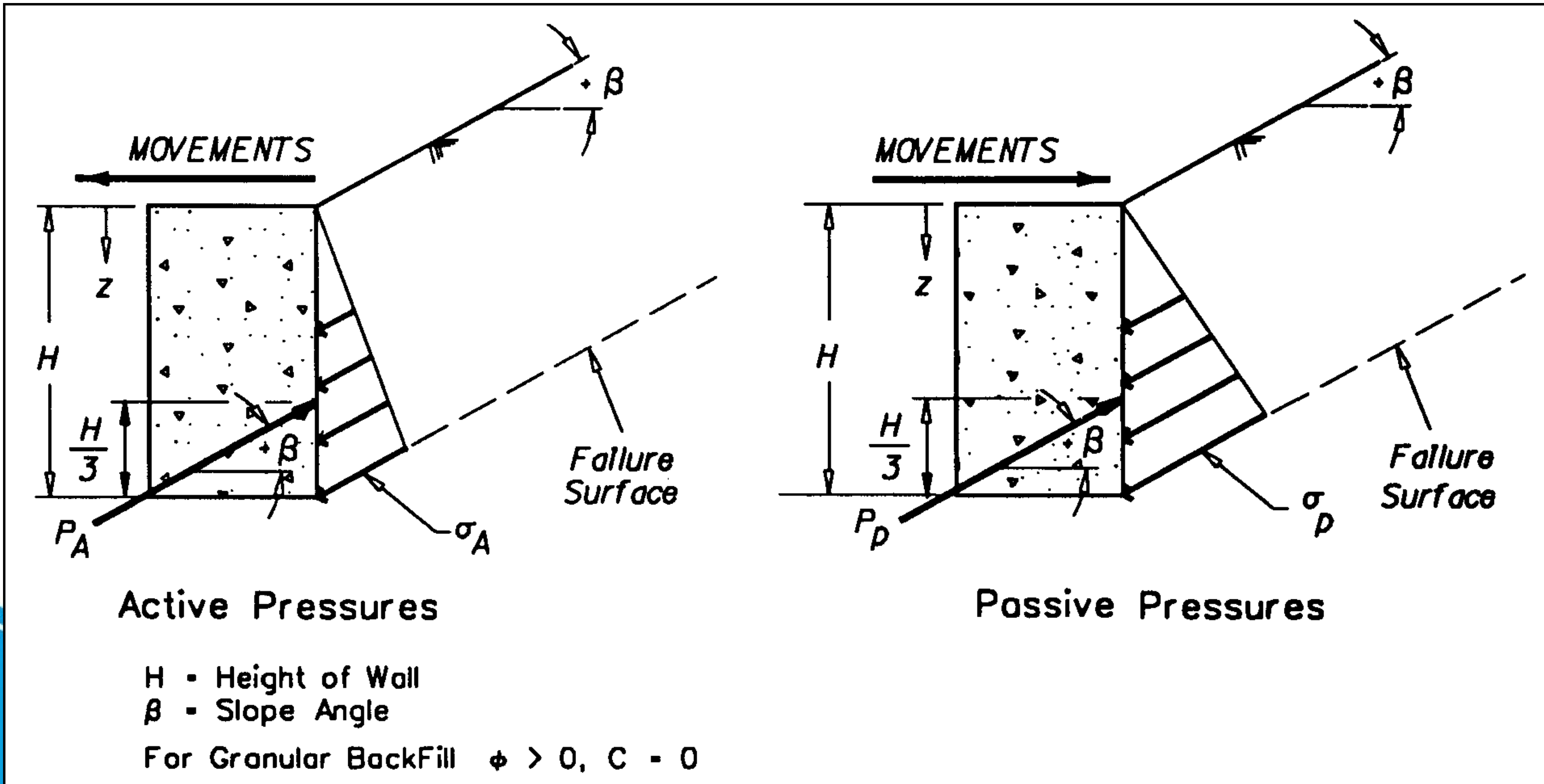


FIGURE 2
Computation of Simple Active and Passive Pressures

Rankine Theory with Inclined Backfills



Rankine Coefficients with Inclined Backfills and Vertical Walls

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

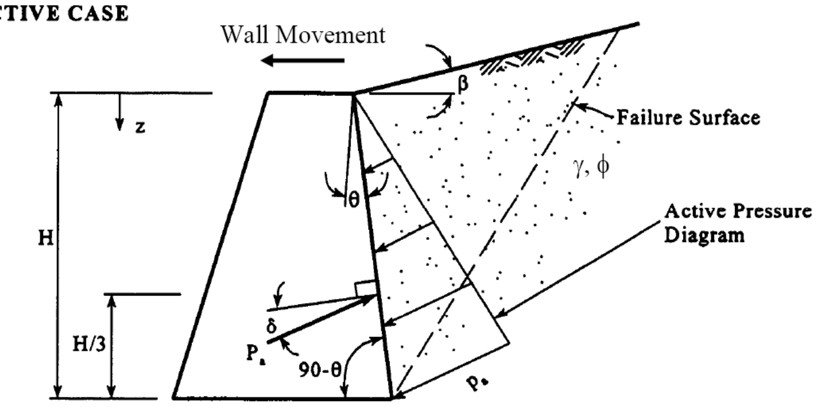
$$K_p = (\cos(\phi))^2 \cos(\beta) \left(\cos(\beta) - 2\sqrt{\frac{\sin(\phi)\sin(\phi+\beta)}{\cos(\beta)}} \cos(\beta) + \sin(\phi)\sin(\phi+\beta) \right)^{-1}$$

These are derived from Coulomb wedge theory; for a more thorough treatment of this topic, click here

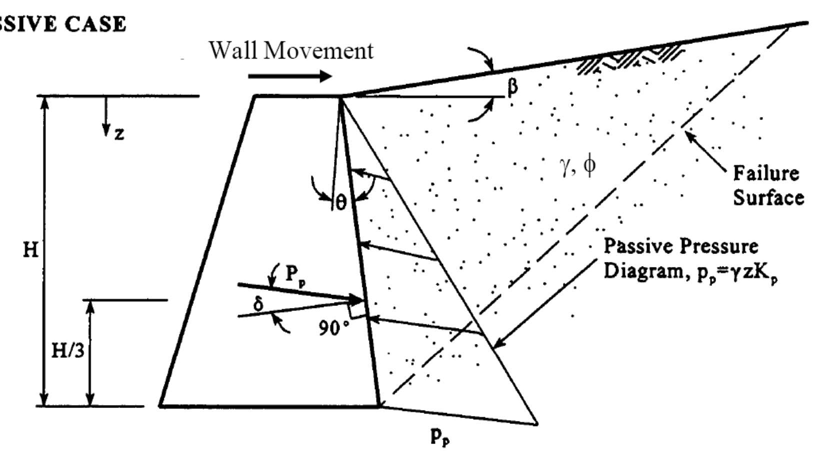
Inclined and level backfill equations are identical when $\beta = 0$ for both this and "Rankine extended" theory

Coulomb Failure Theory

- Includes effects of wall-soil friction through use of failure wedge theory
- Can also be used with sloping backfill walls
- Is very UNCONSERVATIVE for passive pressures with high values of wall friction
- Formulas to the right assume that δ is positive for both active and passive pressures



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



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

Figure 10-5. Coulomb coefficients K_a and K_p for sloping wall with wall friction and sloping cohesionless backfill (after NAVFAC, 1986b).

Coulomb Earth Pressure Theory

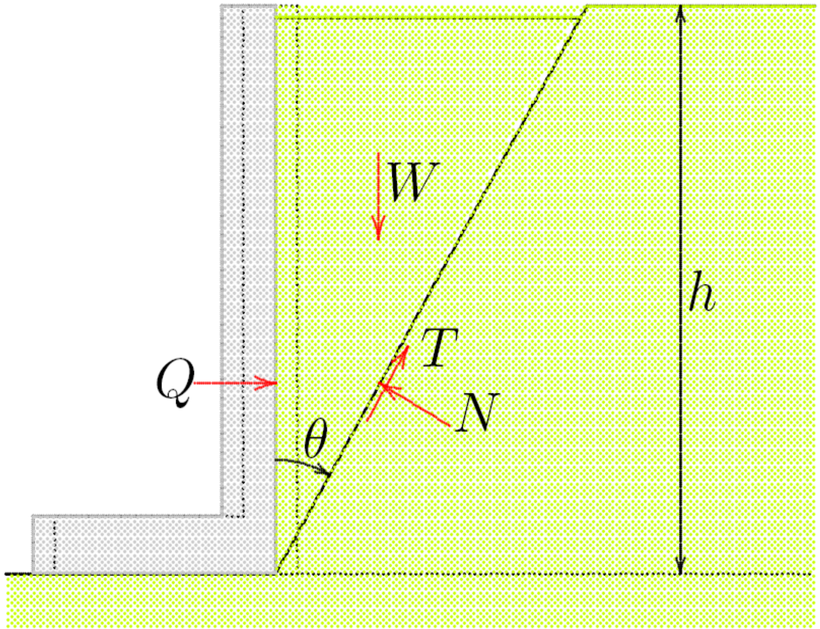


Figure 34.1: Active earth pressure.

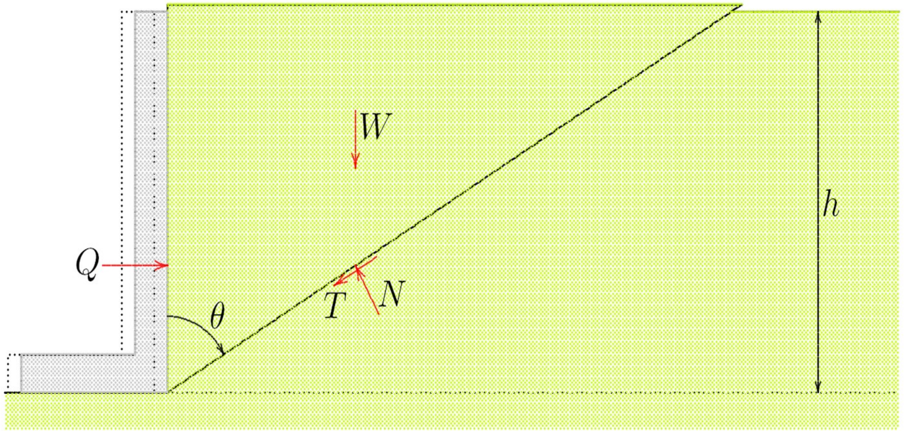
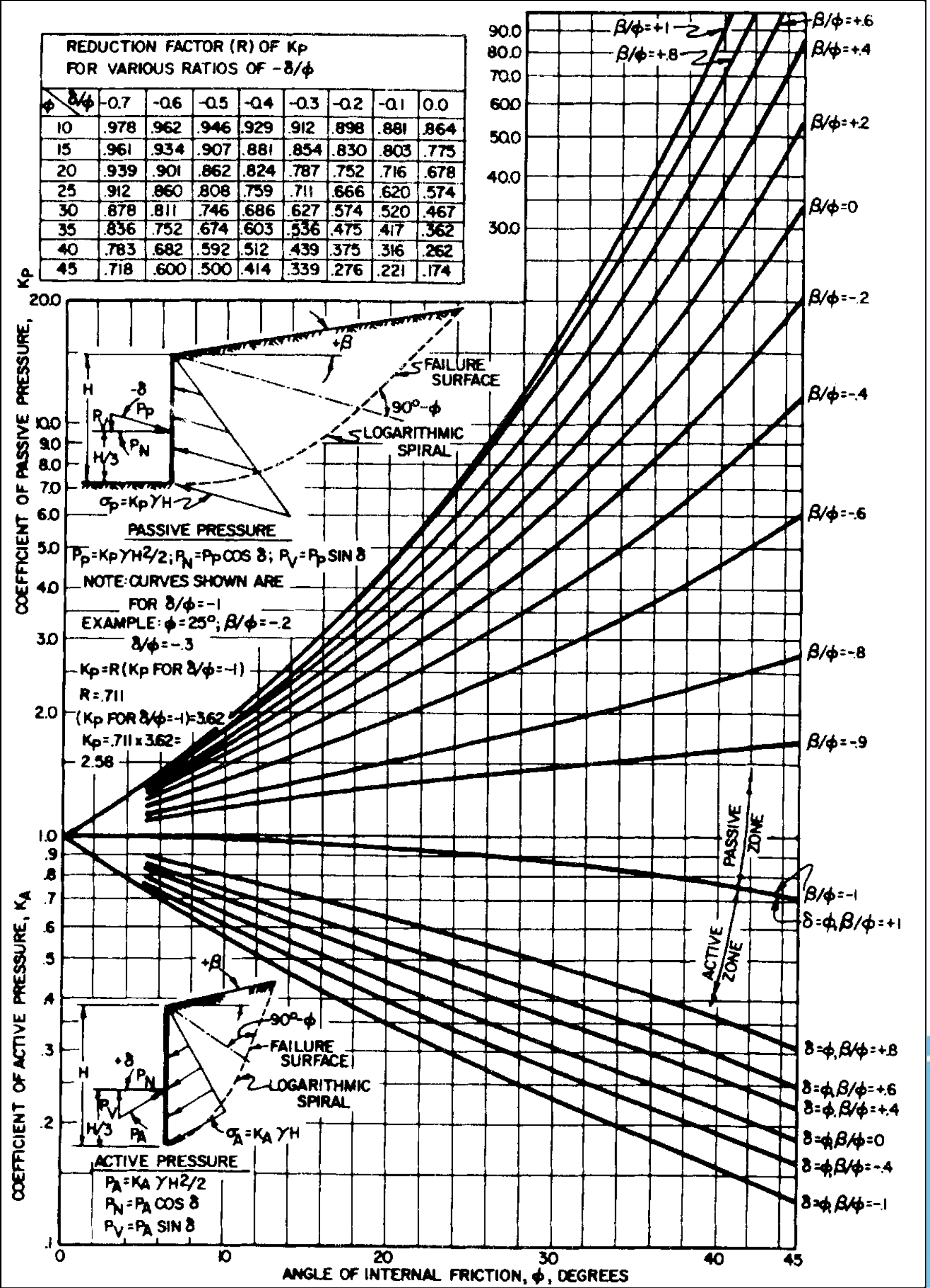


Figure 34.2: Passive earth pressure.

http://vulcanhammer.net/2021/09/28/rankine-and-coulomb-earth-pressure-coefficients/

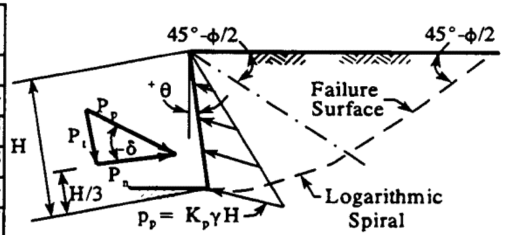


Log-Spiral Model

- Assumes log spiral failure surface behind wall
- Requires use of suitable chart for K_A and K_P
- Requires different chart for vertical wall and horizontal backfill (vertical wall shown at left)
- With AASHTO specifications, only log-spiral charts for K_P are necessary

REDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF $-\delta/\phi$								
ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

Note: Curves shown are for $\delta/\phi = -1$



$P_p = \frac{K_p \gamma H^2}{2}$

$P_n = P_p \cos\delta$

$P_t = P_p \sin\delta$

$p_p = K_p \gamma H$

Example: $\phi = 30°$; $\theta = -10$; $\delta/\phi = -0.6$

$K_p = R(K_p \text{ for } \delta/\phi = -1) = (0.811)(8.2) = 6.65$

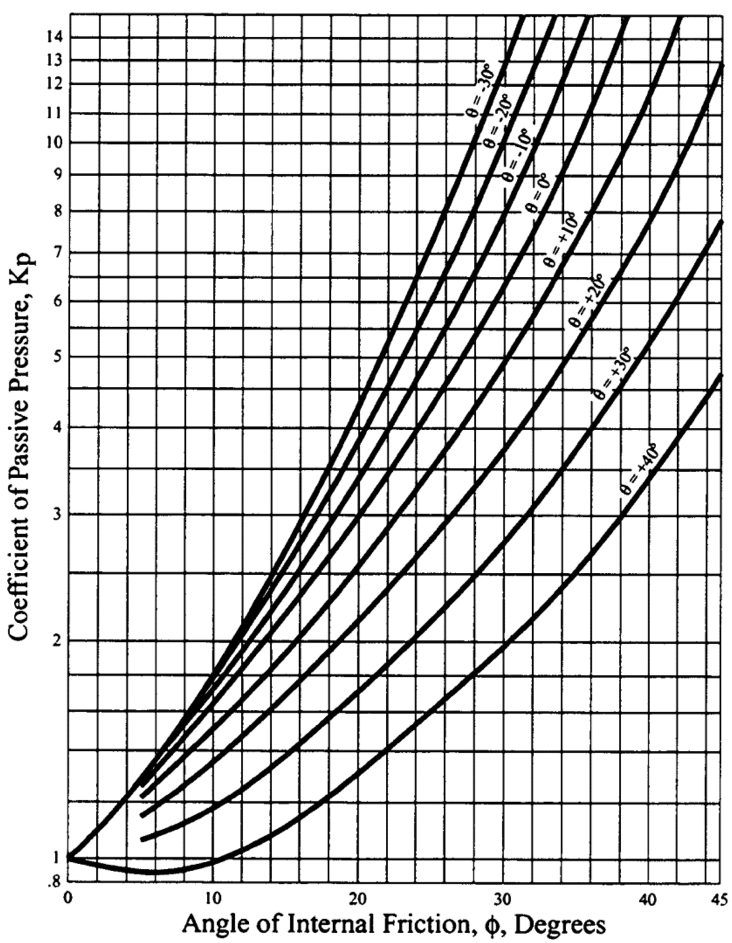
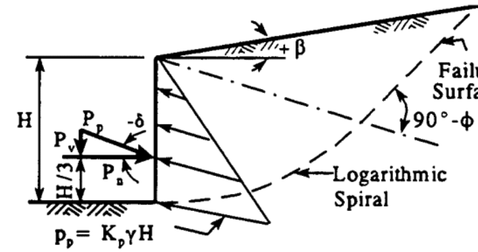


Figure 10-9. Passive coefficients for sloping wall with wall friction and horizontal backfill (Caquot and Kerisel, 1948; NAVFAC, 1986b).

REDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF $-\delta/\phi$								
ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

Note: Curves shown are for $\delta/\phi = -1$



$p_p = K_p \gamma H$

$P_p = K_p \gamma H^2/2$; $P_n = P_p \cos\delta$; $P_v = P_p \sin\delta$

Example: $\phi = 25°$; $\beta/\phi = -0.2$; $\delta/\phi = -0.3$

$K_p = R(K_p \text{ for } \delta/\phi = -1) = (0.711)(3.62) = 2.58$

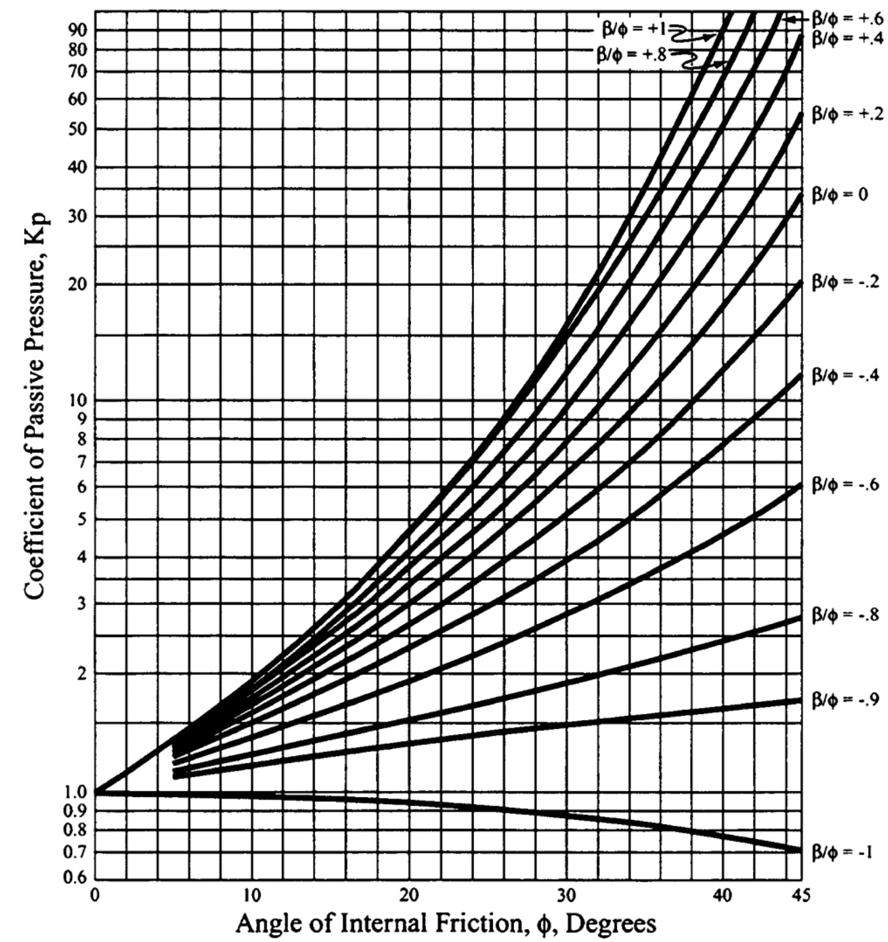


Figure 10-10: Passive coefficients for vertical wall with wall friction and sloping backfill (Caquot and Kerisel, 1948; NAVFAC, 1986b).

Typical Values of Wall Friction

Table 10-1
Wall friction and adhesion for dissimilar materials (after NAVFAC, 1986b)

Interface Materials	Friction Factor, tan δ	Friction angle, δ degrees
Mass concrete on the following foundation materials:		
Clean sound rock	0.70	35
Clean gravel, gravel sand mixtures, coarse sand	0.55 to 0.60	29 to 31
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	0.45 to 0.55	24 to 29
Clean fine sand, silty or clayey fine to medium sand	0.35 to 0.45	19 to 24
Fine sandy silt, nonplastic silt	0.30 to 0.35	17 to 19
Very stiff and hard residual or preconsolidated clay	0.40 to 0.50	22 to 26
Medium stiff and stiff clay and silty clay (Masonry on foundation materials has same friction factor)	0.30 to 0.35	17 to 19
Steel sheet piles against the following soils:		
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	0.40	22
Clean sand, silty sand-gravel mixtures, single size hard rock fill	0.30	17
Silty sand, gravel or sand mixed with silt or clay	0.25	14
Fine sandy silt, nonplastic silt	0.20	11
Formed concrete or concrete sheet piling against the following soils:		
Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	0.40 to 0.50	22 to 26
Clean sand, silty sand-gravel mixture, single size hard rock fill	0.30 to 0.40	17 to 22
Silty sand, gravel or sand mixed with silt or clay	0.30	17
Fine sandy silt, nonplastic silt	0.25	14
Various structural materials:		
Masonry on masonry, igneous and metamorphic rocks:		
Dressed soft rock on dressed soft rock	0.70	35
Dressed hard rock on dressed soft rock	0.65	33
Dressed hard rock on dressed hard rock	0.55	29
Masonry on wood (cross grain)	0.50	26
Steel on steel at sheet pile interlocks	0.30	17

Interface Materials (Cohesion)	Adhesion c_a (kPa)
Very soft cohesive soil (0 - 12 kPa)	0 - 12
Soft cohesive soil (12 - 24 kPa)	12 - 24
Medium stiff cohesive soil (24 - 48 kPa)	24 - 36
Stiff cohesive soil (48 - 96 kPa)	36 - 45
Very stiff cohesive soil (96 - 192 kPa)	45 - 62

Maximum wall friction suggested for design:

Active: $\delta = 2\varphi'/3$

Passive: $\delta = \varphi'/2$

Also: $\delta = \tan^{-1}(\sin(\varphi))$

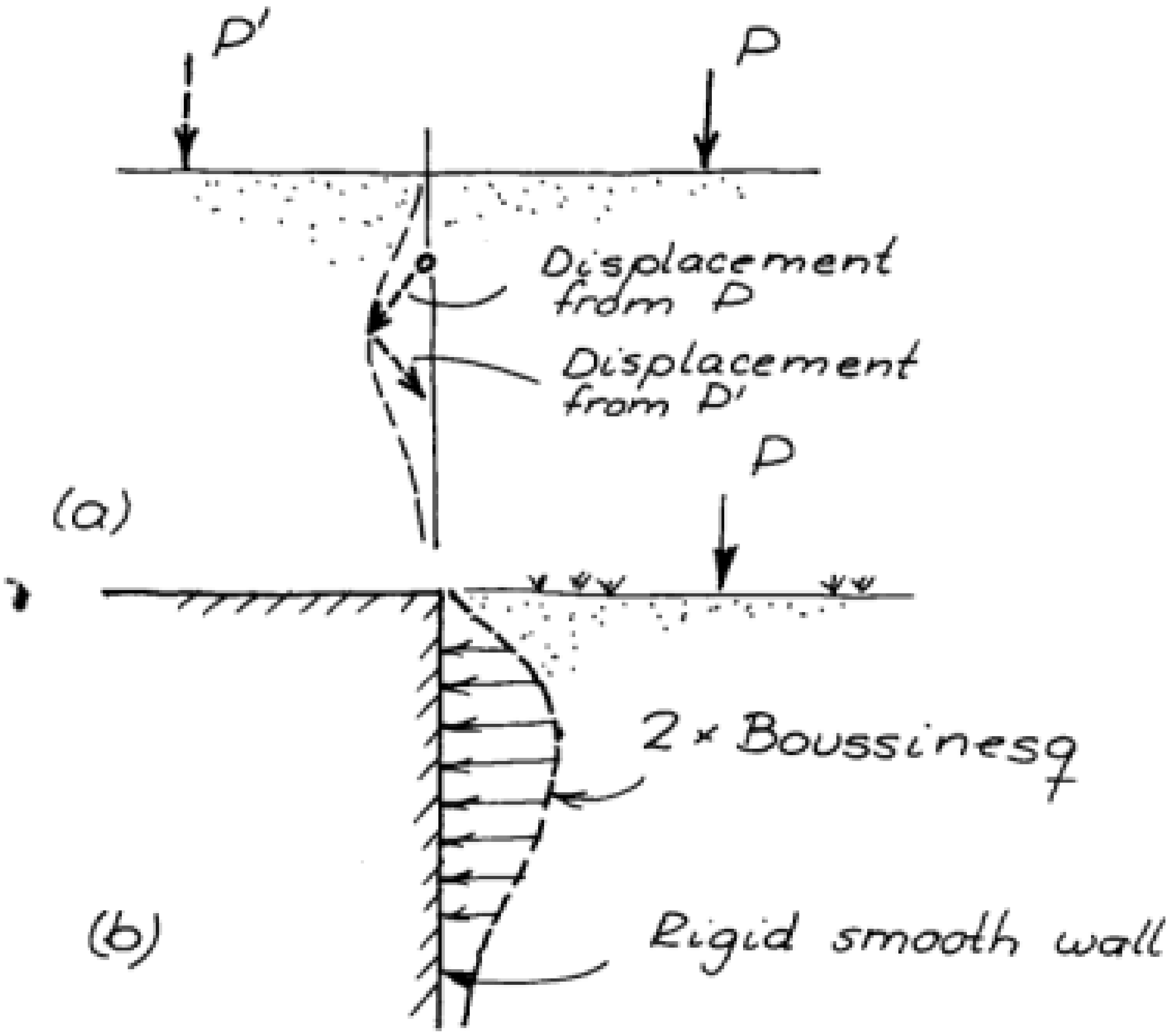
Walls with Cohesive Backfill

- Retaining walls should generally have cohesionless backfill, but in some cases cohesive backfill is unavoidable
  - Cohesive soils present the following weaknesses as backfill:
    - Poor drainage
    - Creep
    - Expansiveness
- Most lateral earth pressure theory was first developed for purely cohesionless soils ($c = 0$) and has been extended to cohesive soils afterward

Table 10-3
Suggested gradation for backfill for cantilever semi-gravity and gravity retaining walls

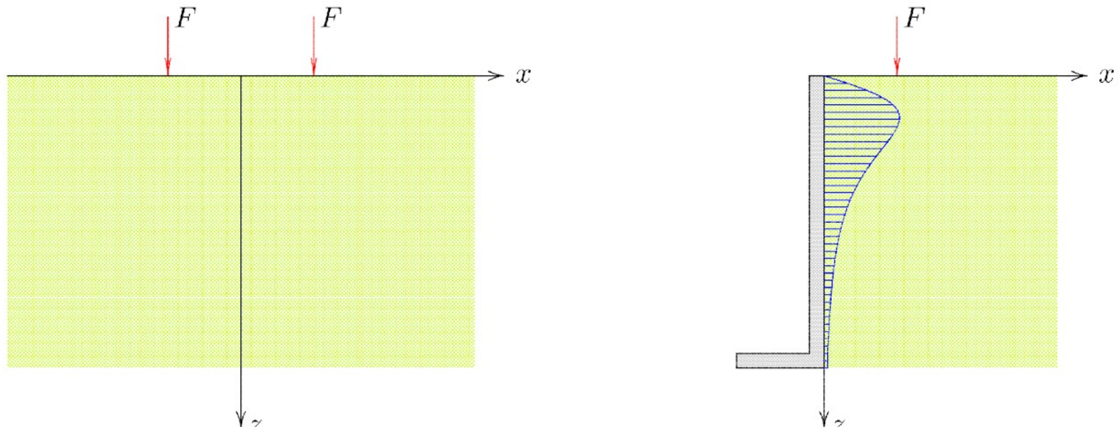
Sieve Size	Percent Passing
3 in. (76.2 mm)	100
No. 4 (4.75 mm)	35 – 100
No. 30 (0.6 mm)	20 – 100
No. 200 (0.075 mm)	0 – 15

Origin of Lateral Pressures from Surface Loading (Line Loading)



Origin of Lateral Pressures from Surface Loading (Line Loading)

Another interesting application of Flamant's solution is shown in Figure 30.5. In the case of a surface load by two parallel line loads it can



be expected that at the axis of symmetry ($x = 0$) the horizontal displacement and the shear stress will be zero, because of symmetry. This means that the solution of that problem can also be used as the solution of the problem of a line load at a certain distance from a smooth rigid wall, because the boundary conditions along the wall are that $u_x = 0$ and $\sigma_{zx} = 0$ for $x = 0$. By the symmetry of the problem shown in the left half of Figure 30.5, these conditions are satisfied by the solution of that problem.

The horizontal stresses against the wall are given by equation (30.2) for Flamant's basic problem, multiplied by 2, because there are two line loads and each gives the same stress. In this formula the value of x should be taken as $x = a$, where a is the distance of the force to the wall. The horizontal stress against the wall in this case is

$$\sigma_{xx} = \frac{4}{\pi}\frac{Fa^2z}{(a^2+z^2)^2}. \tag{30.15}$$

The distribution of the horizontal stresses against the wall are also shown in Figure 30.5. The maximum value occurs for $z = 0.577\,a$, and that maximum stress is

$$\sigma_{xx-\max} = 0.4135\,\frac{F}{a}$$

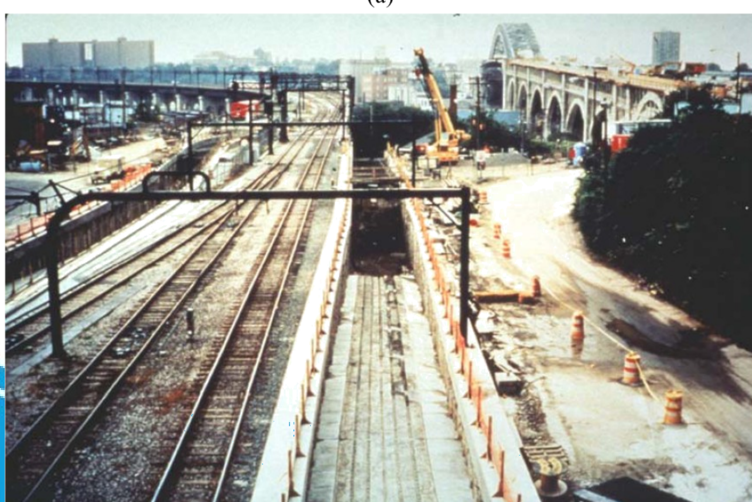
The total force on a wall of depth h can be found again by integration of the horizontal stress over the depth of the wall. This gives

$$Q = \frac{2}{\pi}\frac{F}{1+a^2/h^2}. \tag{30.16}$$

If $a = 0$ this is $Q = 0.637\,F$. If a increases the value of Q will gradually become smaller. A force at a larger distance from the wall will give smaller stresses against the wall, as could be expected.

Effects of Surface Loading (modified from elastic theory per Terzaghi)

(a)



(b)

Figure 10-13: (a) Retaining wall with uniform surcharge load and (b) Retaining wall with line loads (railway tracks) and point loads (catenary support structure).

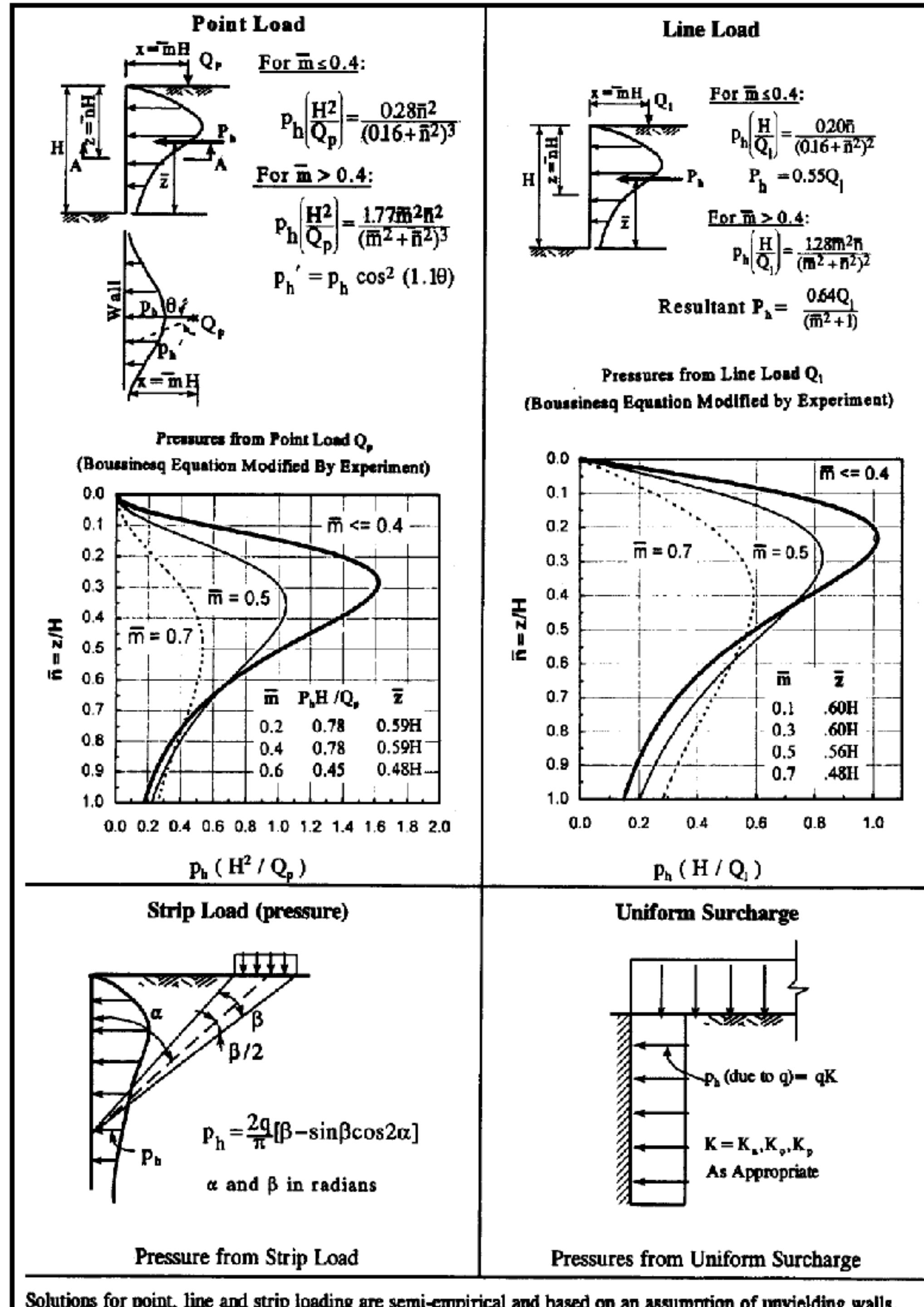
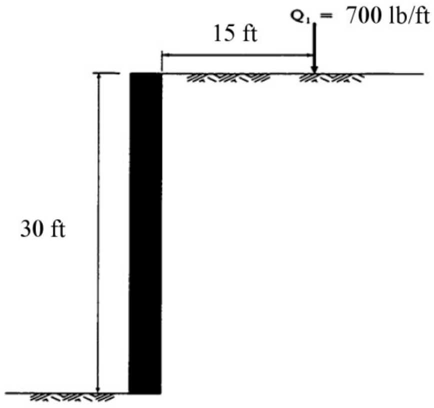


Figure 10-14. Lateral pressure due to surcharge loadings (after USS Steel, 1975)

Example of Surcharge Loading

Example 10-2: Construct the lateral pressure diagram due to a line load of 700 lb/ft located 15 ft behind the top of a 30 ft high unyielding wall shown below.



Geometry of the Example Problem 10-2

Solution:

The procedure to calculate the lateral pressures due to a line load is given in Figure 10-14. From this figure the lateral pressure can be found as follows:

$$\overline{m} = \frac{15\,\text{ft}}{30\,\text{ft}} = 0.5 > 0.4$$

For $\overline{m} > 0.4$, the lateral pressure is given by:

$$P_h = 1.28\left(\frac{Q_1}{H}\right)\left[\frac{\overline{m}^2\overline{n}}{\left(\overline{m}^2 + \overline{n}^2\right)^2}\right]$$

For $\overline{m} = 0.5$, Q_l=700 lb/ft and H = 30 ft, the lateral pressure is given by:

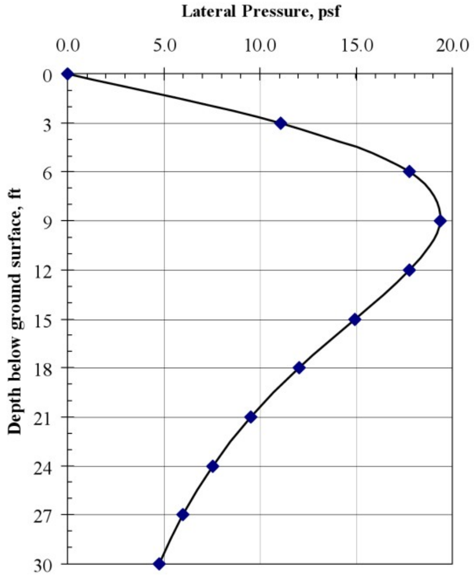
$$P_h = 1.28\left(\frac{700\,\text{lb/ft}}{30\,\text{ft}}\right)\left[\frac{0.5^2\overline{n}}{\left(0.5^2 + \overline{n}^2\right)^2}\right] \quad \rightarrow \quad P_h = 29.9\left[\frac{0.25\overline{n}}{\left(0.25 + \overline{n}^2\right)^2}\right]$$

Lateral pressures computed at various depths by using the above formula and the chart for line loads in Figure 10-14 are tabulated below.

Computation of Lateral Earth Pressures Due To Line Load

$\overline{n} = z/H$	Depth below top of wall (ft)	P_h (psf)
0	0	0.00
0.1	3	11.0
0.2	6	17.8
0.3	9	19.4
0.4	12	17.8
0.5	15	14.9
0.6	18	12.0
0.7	21	9.5
0.8	24	7.5
0.9	27	6.0
1.0	30	4.8

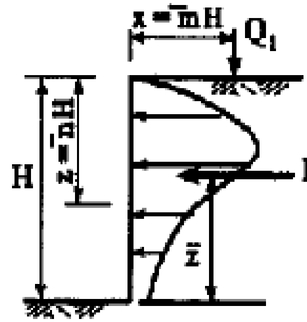
The information in the table is used to construct the curve of depth vs. lateral pressure shown below.



Example of Surcharge Loading: Location and Magnitude of Resultant

- From problem $\bar{m} = 0.5$
- Resultant force $P_h = (0.64)(700 \text{ lb/ft})/(1+0.5^2) = 358.4$ lb.
- From chart at right $\bar{z} = z/H$, for $\bar{m} = 0.5$, $z = 0.56H = (0.56)(30') = 16.8'$ from dredge line
- Resultants are more typically used in hand calculations; the distributed load (which varies with depth) is usually input into a computer program

Line Load



For $\bar{m} \le 0.4$:

$$p_h\left(\frac{H}{Q_l}\right) = \frac{0.20\bar{n}}{(0.16+\bar{n}^2)^2}$$

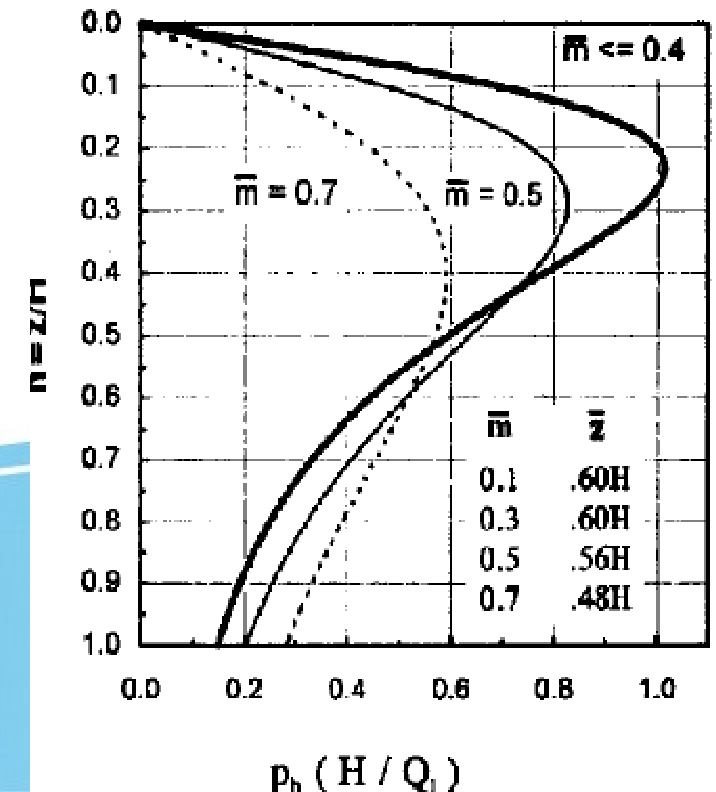
$$P_h = 0.55Q_l$$

For $\bar{m} > 0.4$:

$$p_h\left(\frac{H}{Q_l}\right) = \frac{1.28\bar{m}^2\bar{n}}{(\bar{m}^2+\bar{n}^2)^2}$$

$$\text{Resultant } P_h = \frac{0.64Q_l}{(\bar{m}^2+1)}$$

Pressures from Line Load Q_l
(Boussinesq Equation Modified by Experiment)



Groundwater Effects

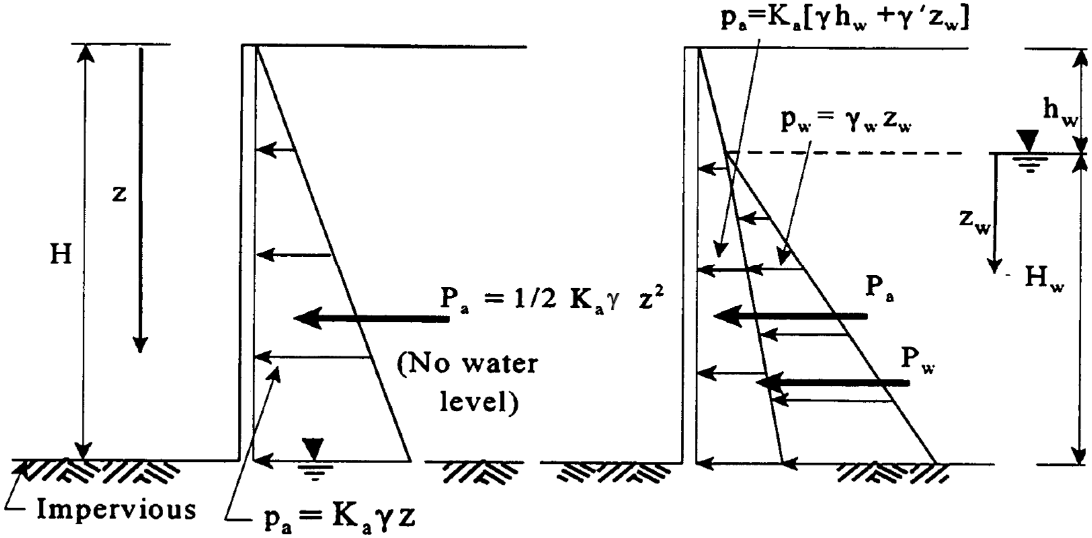
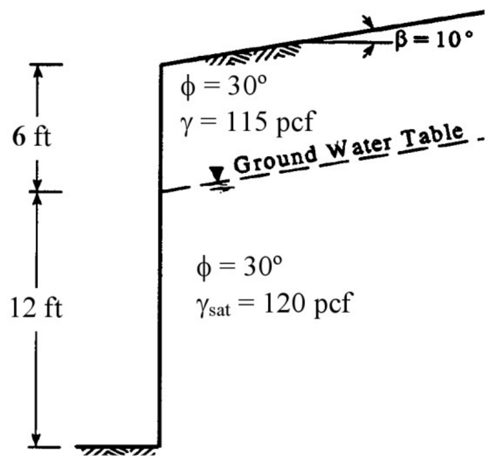


Figure 10-12. Computation of lateral pressures for static groundwater case.

- Steps to properly compute horizontal stresses including groundwater effects:
  - Compute total vertical stress
  - Compute effective vertical stress by removing groundwater effect through submerged unit weight; plot on P_o diagram
  - Compute effective horizontal stress by multiplying effective vertical stress by K
  - Compute total horizontal stress by directly adding effect of groundwater unit weight to effective horizontal stress

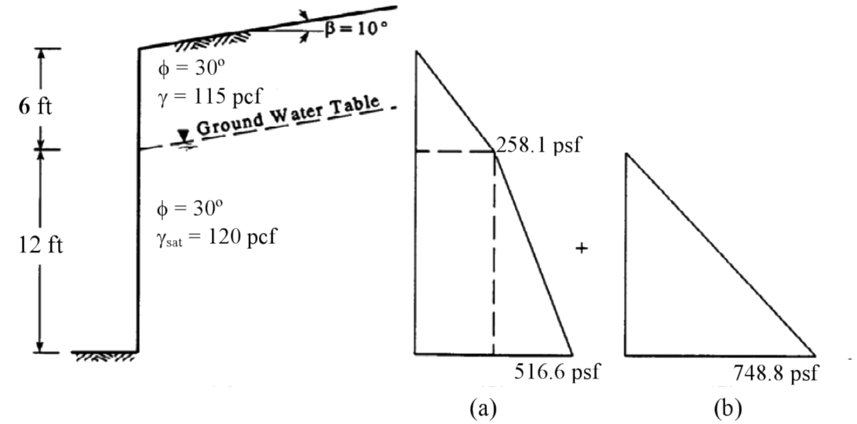
Groundwater Example

Example 10-1: For the wall configuration shown below, construct the lateral pressure diagram. Assume the face of the wall to be smooth ($\delta = 0$, $c_w = 0$).



Hydrostatic Pressure, $u = K_w u_w$

z, ft	z_w, ft	$u_w = z_w \gamma_w$, psf	Lateral water pressures, u psf
0	0	0	= 0
6	0	0	= 0
18	12	12 ft (62.4 pcf) = 748.8 psf	1.0(748.8 psf) = 748.8 psf



(a) Lateral effective earth pressure diagram and (b) Lateral water pressure diagram.

Solution:

Use the Coulomb method (Figure 10-5) for $\phi = 30^\circ$, $\beta = 10^\circ$, $\theta = 0$, and $\delta = 0$:

$$K_a = 0.374$$

The pressures at various depths can then be calculated as shown in a tabular format as follows. Based on the values in the table, the lateral pressure diagrams due to earth and water can be constructed as shown below. The total lateral pressure diagram is the sum of the two lateral pressure diagrams shown in the figure accompanying this example.

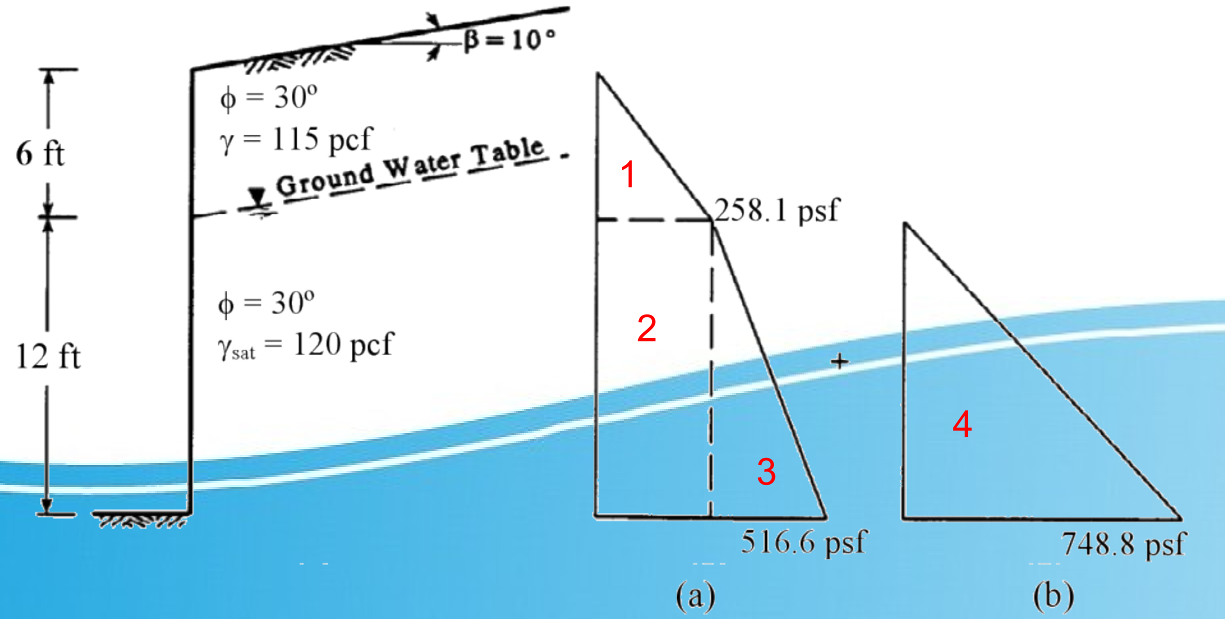
Effective Lateral Earth Pressures, p'_a

z, ft	p_o, psf		$p_a = K_a p_o$, psf
0	0		0
6	(115 pcf) (6 ft)	= 690.0 psf	0.374(690.0 psf) = 258.1 psf
18	690 psf +(120 pcf-62.4 pcf)(12 ft)	= 1381.2 psf	0.374(1381.2 psf) = 516.6 psf

Groundwater Example: Computation of Resultants

- Region 1: Triangular, P_1 = (258.1)(6)/2 = 774.3 lb/ft, resultant located (6)(⅔) = 4’ from top of wall
- Region 2: Rectangular P_2 = (258.1)(12) = 3097.2 lb/ft, resultant located (12)(½) = 6’ from bottom of wall = 10’ from top of wall
- Region 3: Triangular P_3 = (516.6-258.1)(12)/2 = 1551 lb/ft, resultant located (12)(⅓) = 4’ from bottom of wall = 14’ from top of wall
- Region 4: Triangular P_4 (unbalanced hydrostatic pressure) = (748.8)(12)/2 = 4492.8 lb/ft of wall, resultant located in the same place as P_3
- Unbalanced hydrostatic force 45% of total force against wall in this case

http://vulcanhammer.net/2021/12/11/what-is-a-resultant-in-geotechnical-engineering/



External vs. Internal Stability

There are many different types of walls as shown in Figure 10-3. All walls have to be evaluated for stability with respect to different modes of deformation. There are four basic modes of instability from a geotechnical viewpoint. These are (a) sliding, (b) limiting eccentricity or overturning, (c) bearing capacity, and (d) global stability. The four modes of instability are shown in Figure 10-15. Since these modes of instability assume that the wall is intact, the evaluation of these modes is commonly referred to as the "external stability" analysis. All four modes may or may not be applicable to all wall types. Furthermore, depending on the wall type and its load support mechanism (refer to Section 10.1), there may be additional instability modes, such as pullout, tension breakage, bending and shear. The evaluation of these additional modes of instability are commonly referred to as "internal stability" analyses.

Concrete Gravity Walls

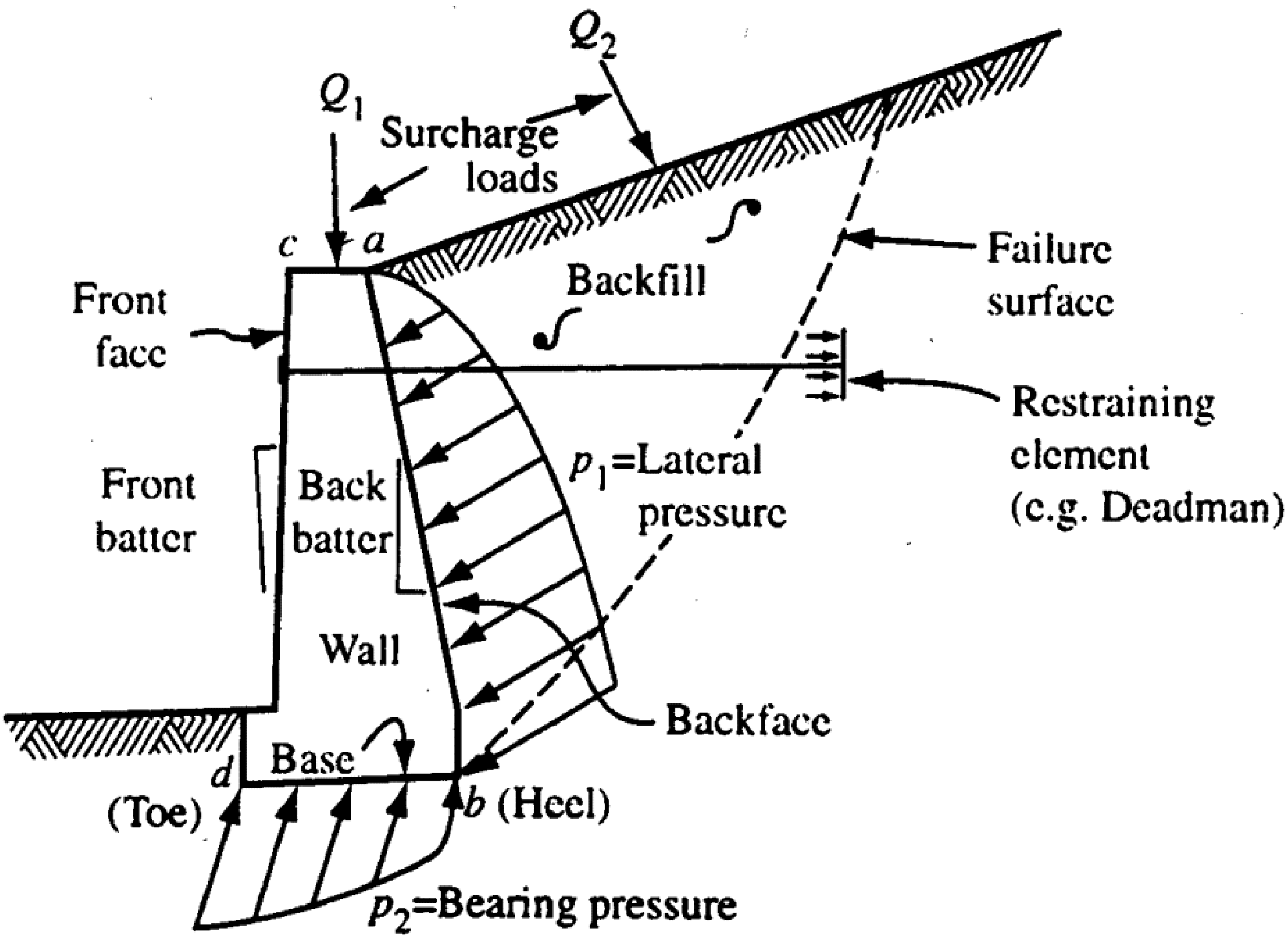
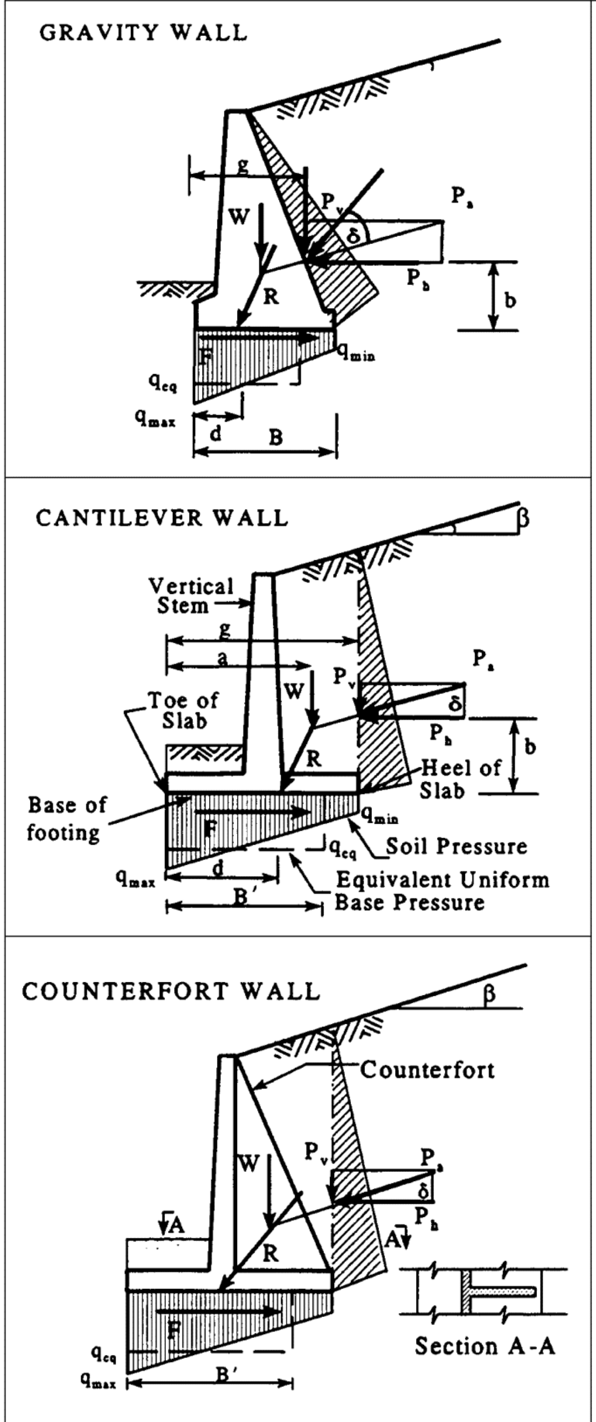


Figure 10-1. Schematic of a retaining wall and common terminology.

Design Parameters for Rigid Walls

- Contact Pressure on Foundation: Use Shallow Foundation Bearing Capacity Methods
- Global Stability: use slope stability methods



DEFINITIONS

B = width of the base of the footing

$\tan \delta_b$ = friction factor between soil and base (see Table 10-1 for guidance)

W = weight at the base of wall. Includes weight of wall for gravity walls. Includes weight of the soil above footing for cantilever and counterfort walls

c = cohesion of the foundation soil

c_a = adhesion between concrete and soil

δ = angle of wall friction

P_p = passive resistance

LOCATION OF RESULTANT, R

Based on moments about toe (assuming P_p=0)

$$d = \frac{Wa + P_v g - P_h b}{W + P_v}$$

CRITERIA FOR ECCENTRICITY, e

$$e = d - \frac{B}{2}\ ;\ e \le B/6 \text{ for soils; } e \le B/4 \text{ for rocks}$$

FACTOR OF SAFETY AGAINST SLIDING

$$FS_s = \frac{(W + P_v)\tan \delta_b + c_a B}{P_h} \ge 1.5\,(\text{min})$$

APPLIED STRESS AT BASE (q_{max}, q_{min}, q_{eq})

$$q_{max} = \frac{(W + P_v)}{B}\left(1 + \frac{6e}{B}\right)$$

$$q_{min} = \frac{(W + P_v)}{B}\left(1 - \frac{6e}{B}\right)$$

Equivalent uniform (Meyerhof) applied stress, q_{eq}, is given as follows:

$$q_{eq} = \frac{(W + P_v)}{B'} \text{ where } B' = B - 2e$$

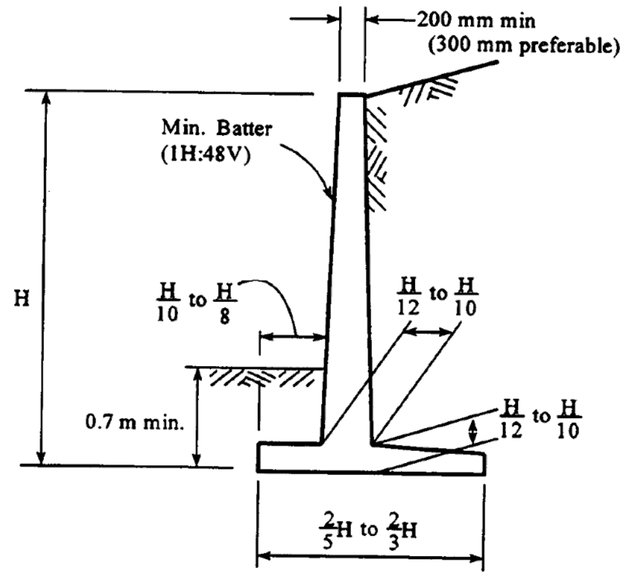
Use uniform stress, q_{eq}, for soils and settlement analysis; use trapezoidal distribution with q_{max} and q_{min} for rocks and structural analysis

DEEP-SEATED (GLOBAL) STABILITY

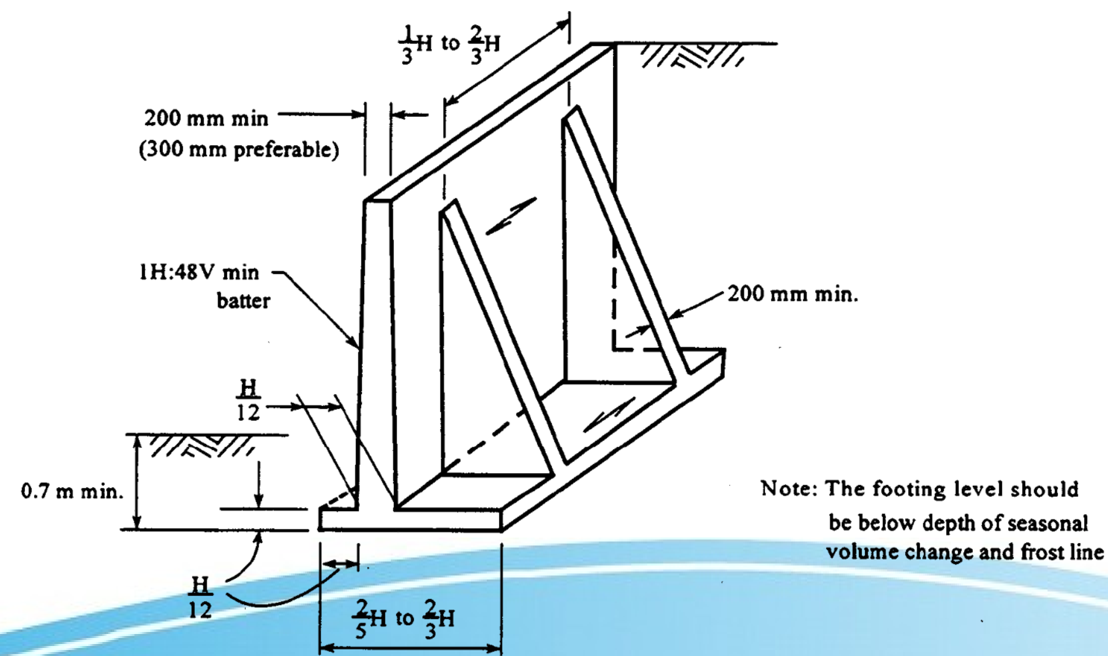
Evaluate global stability using guidance in Chapter 6 (Slope Stability)

Figure 10-17. Design criteria for cast-in-place (CIP) Concrete retaining walls (after NAVFAC, 1986b).

Typical Dimensions for Cast-in-Place Concrete Retaining Walls



(a)

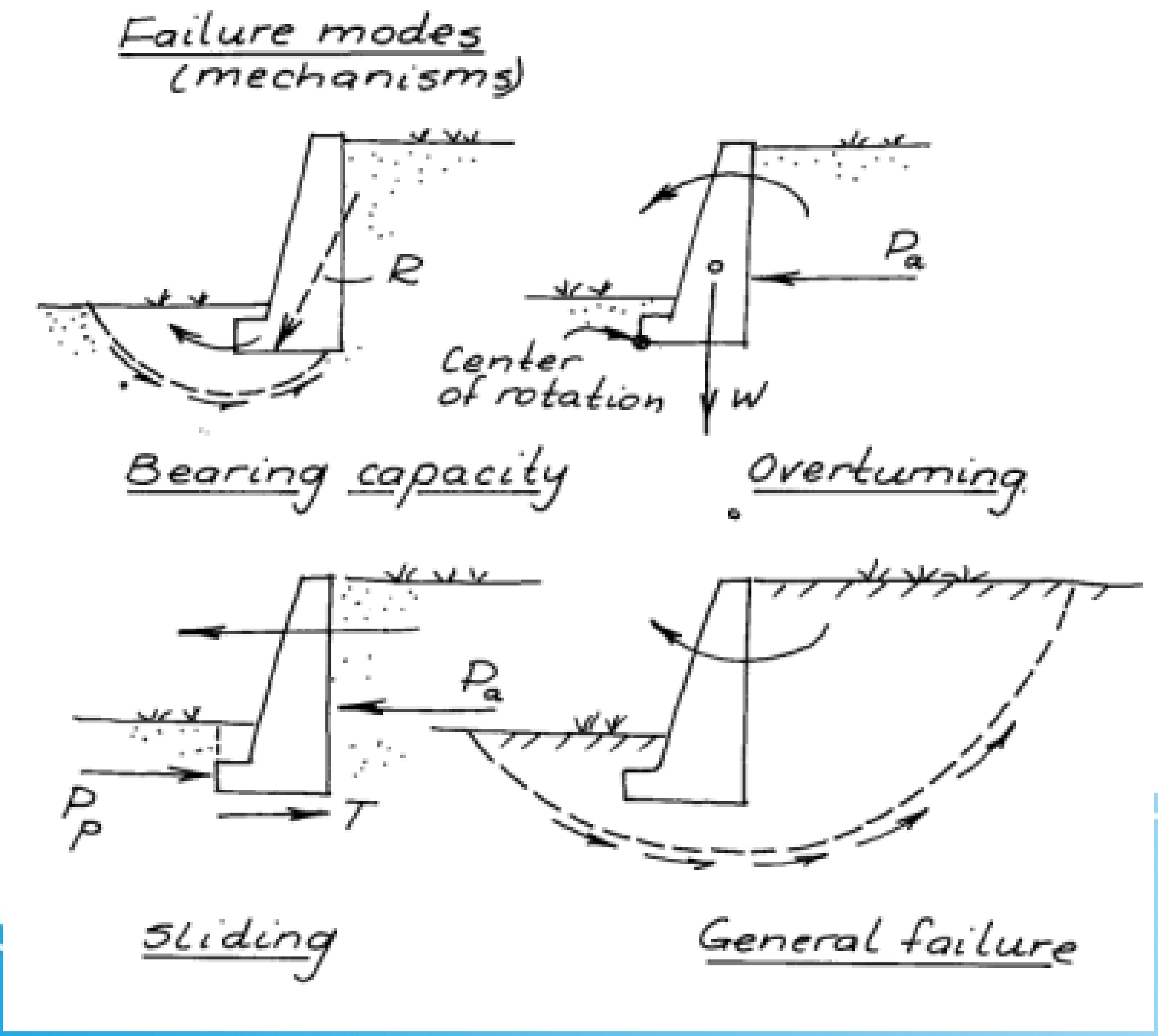


(b)

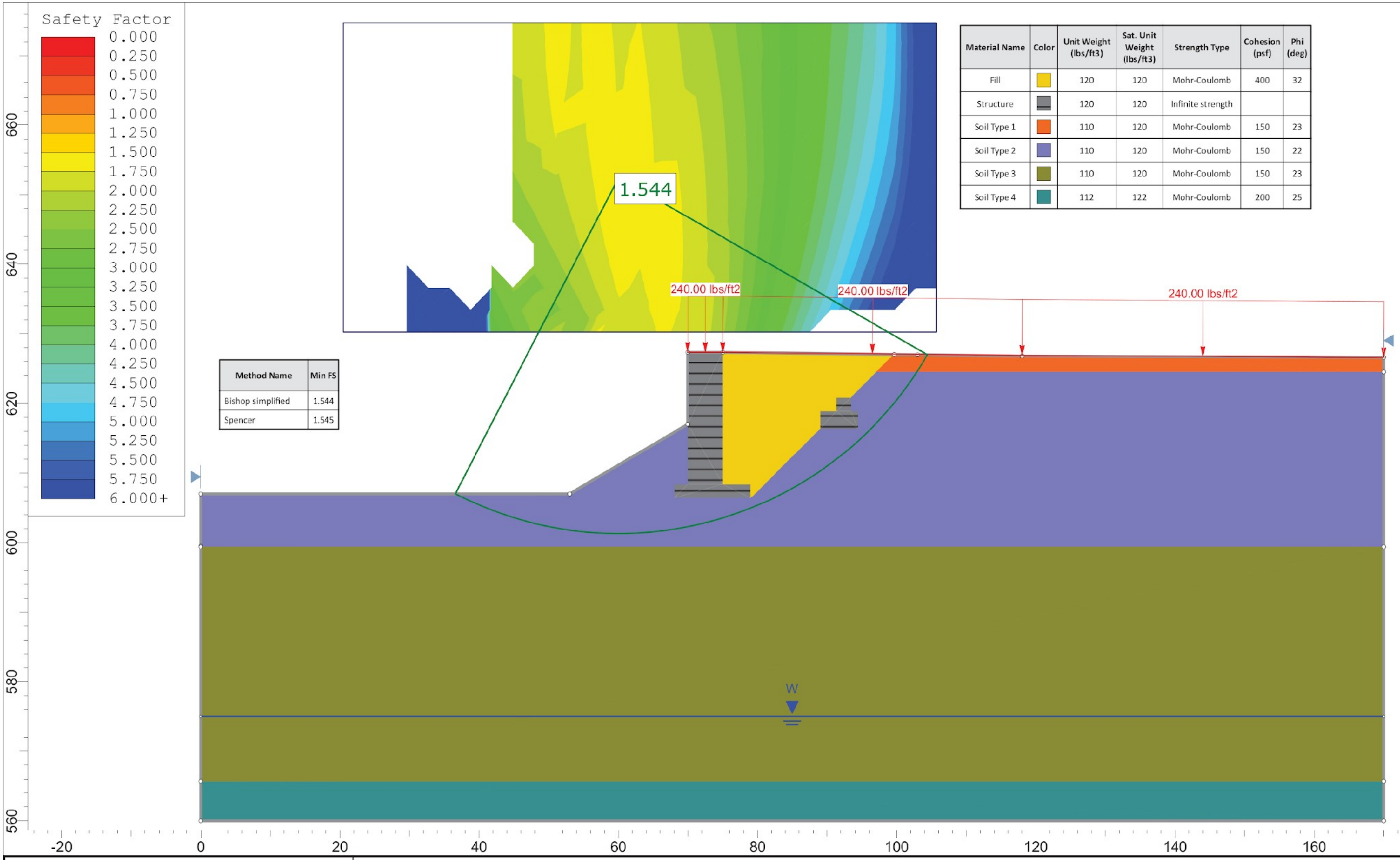
Figure 10-16. Typical dimensions (a) Cantilever wall, (b) Counterfort wall (Teng, 1962).

[1 m = 3.28 ft; 25.4 mm= 1in]

Failure Modes for Gravity Retaining Walls



Global Stability of Retaining Walls



Basic Cantilever Wall Geotechnical Design Considerations (from ERDC/ITL TR-02-3 Appendix A)

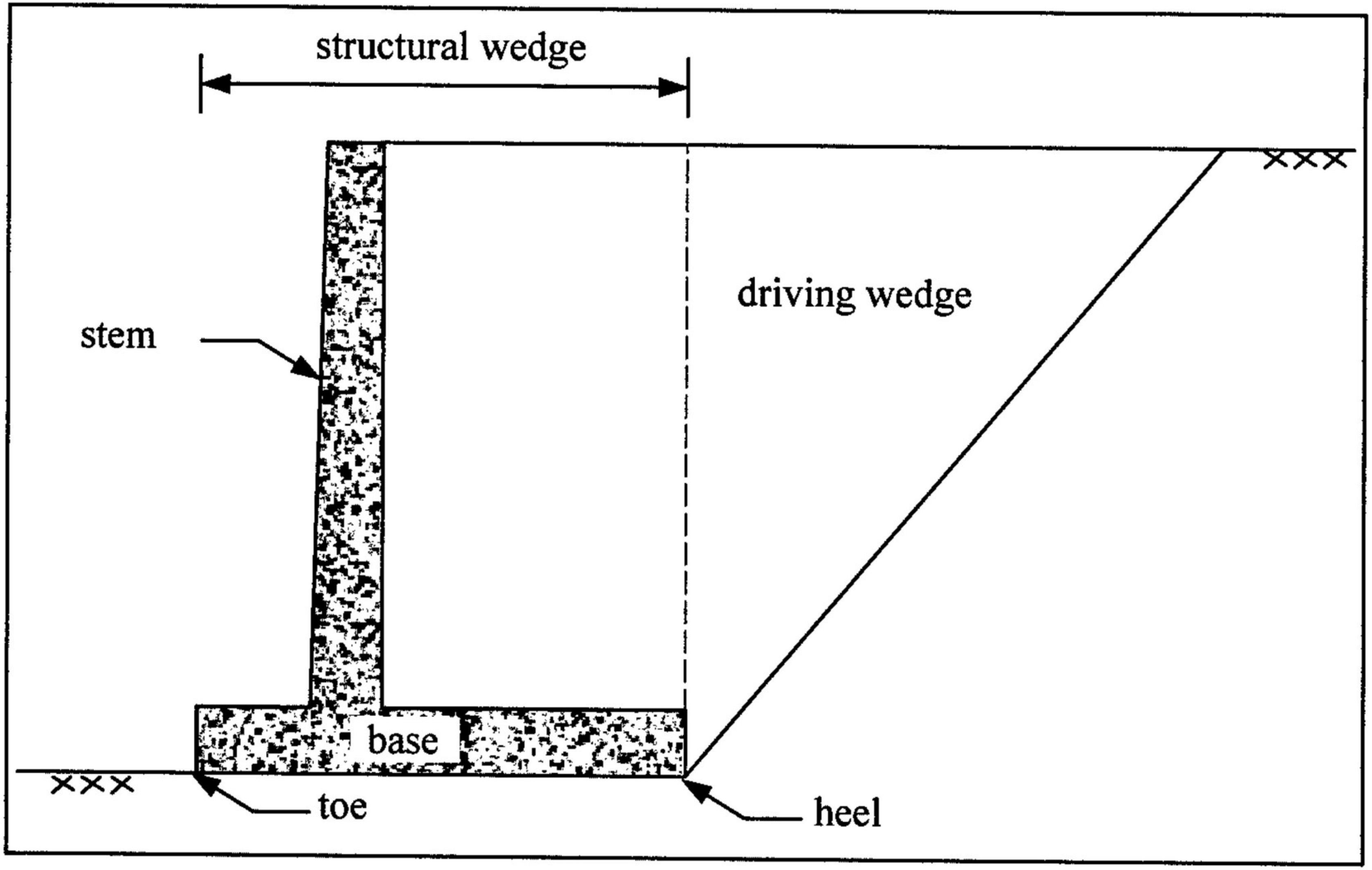


Figure 1-1. Typical Corps cantilever wall, including structural and driving wedges

Cantilever Wall Design Example

Backfill: medium-dense cohesionless compacted fill, $\gamma_m = 125$ pcf, $\phi = 35°$
Foundation: thin layer of compacted fill overlying natural deposit of dense cohesionless soil, $\gamma_m = 125$ pcf, $\phi = 40°$
Reinforced concrete: $\gamma = 150$ pcf, $f'_c = 4$ ksi, $f_y = 48$ ksi
Hydraulic factor: 1.3

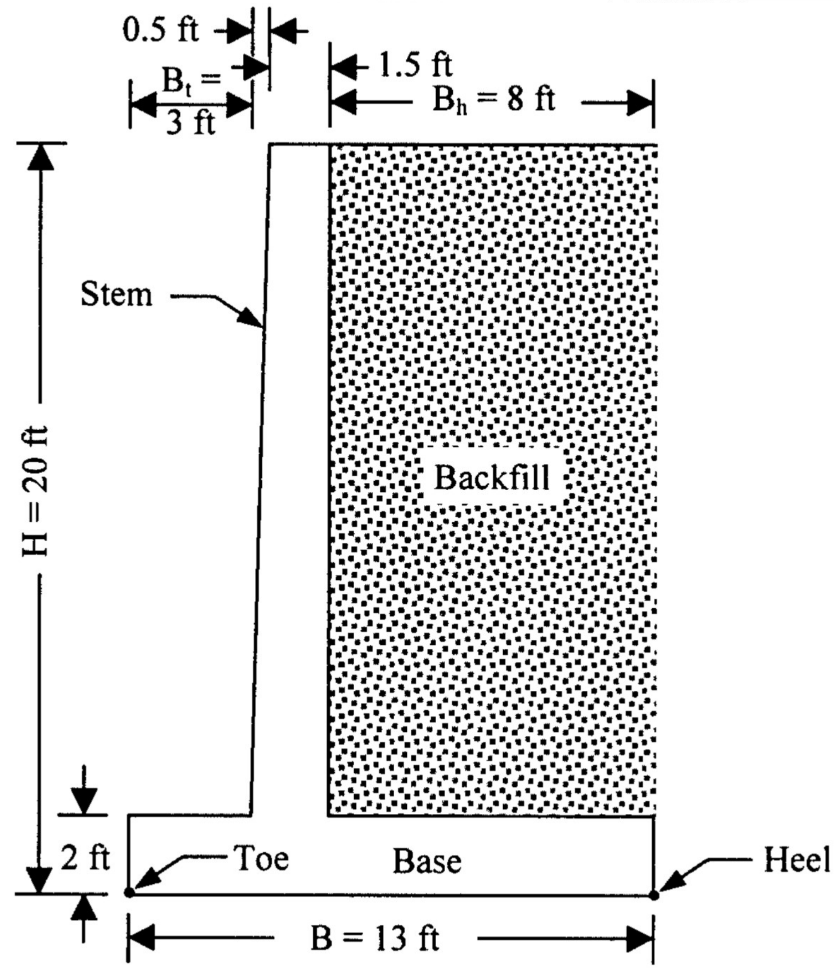


Figure A-1. Structural wedge of proposed wall, and backfill and foundation properties

Cantilever Wall Design Example

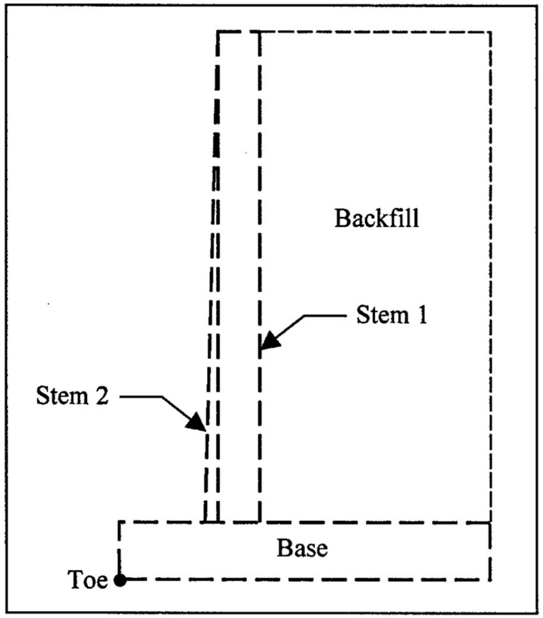
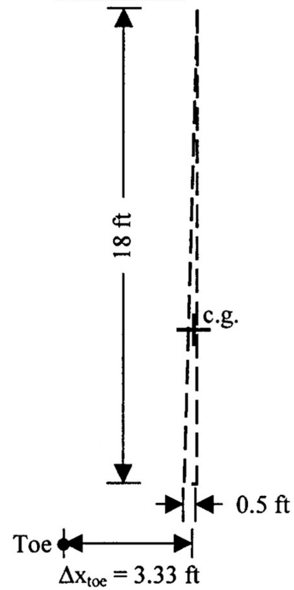


Figure A-2. Division of wall and backfill into sub-sections for determining internal forces and centroids of structural wedge

Stem 2:



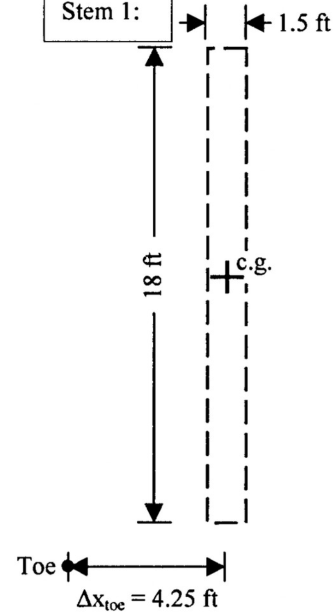
$W_{c\ stem\ 2} = 150\text{pcf} \times 0.5' \times 18' \times 0.5$

$= 675\text{ lbs}$

$\Delta x_{toe} \cdot W_{c\ stem\ 2} = 675\text{ lbs} \times 3.33'$

$= 2250\text{ ft-lbs}$

Stem 1:



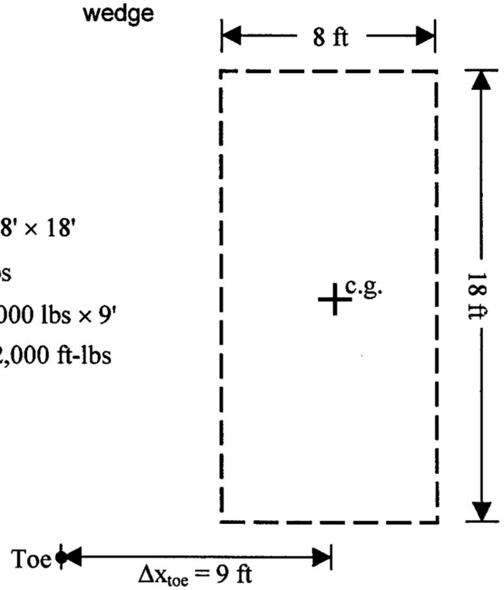
$W_{c\ stem\ 1} = 150\text{pcf} \times 1.5' \times 18'$

$= 4050\text{ lbs}$

$\Delta x_{toe} \cdot W_{c\ stem\ 1} = 4050\text{ lbs} \times 4.25'$

$= 17{,}212.5\text{ ft-lbs}$

Backfill:



$W_s = 125\text{pcf} \times 8' \times 18'$

$= 18{,}000\text{ lbs}$

$\Delta x_{toe} \cdot W_s = 18{,}000\text{ lbs} \times 9'$

$= 162{,}000\text{ ft-lbs}$

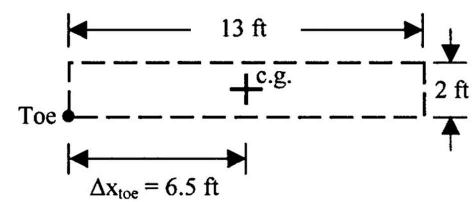
Base:

$W_{c\ base} = 150\text{pcf} \times 2' \times 13'$

$= 3900\text{ lbs}$

$\Delta x_{toe} \cdot W_{c\ base} = 3900\text{ lbs} \times 6.5'$

$= 25{,}350\text{ ft-lbs}$



Note: backfill cross-section will not be rectangular with sloping backfill, see SFH for how to deal with this

Cantilever Example

Weight, moment and centroid of wall around toe:

$$\sum_{i=1}^{3} W_{c,i} = 3900 \text{ lbs} + 4050 \text{ lbs} + 675 \text{ lbs}$$

$$= 8625 \text{ lbs}$$

Centroid of entire wall:

$$\sum_{i=1}^{3} W_{c,i} \cdot \Delta x_{toe,i} = 25{,}350 \text{ ft-lbs} + 17{,}212.5 \text{ ft-lbs} + 2250 \text{ ft-lbs}$$

$$= 44{,}812.5 \text{ ft-lbs}$$

$$\frac{\sum_{i=1}^{3} W_{c,i} \cdot \Delta x_{toe,i}}{\sum_{i=1}^{3} W_{c,i}} = 5.2 \text{ ft from the toe}$$

The shear mobilisation factor (SMF) is the inverse of the factor of safety, thus SMF = 1/FS. Using the SMF as shown below, we reduce the tangent of the internal friction angle of the soil, and by doing so increase the lateral earth pressure coefficient and thus adds a factor of safety to the design. The calculations below show the computation of the internal friction angle we will use (ϕ'_{mob}) for the wedge pressure on the wall.

$$\text{SMF} = 2/3 = 1/1.5$$

$$\tan(\phi'_{mob}) = \text{SMF} \cdot \tan(\phi')$$

$$= \frac{1}{1.5} \cdot \tan(35°)$$

$$\Rightarrow \phi'_{mob} = 25°$$

Important: for problems with c' > 0, we also modify c'_{mob} = SMF x c'

Cantilever Example

http://vulcanhammer.net/2021/09/28/rankine-and-coulomb-earth-pressure-coefficients/

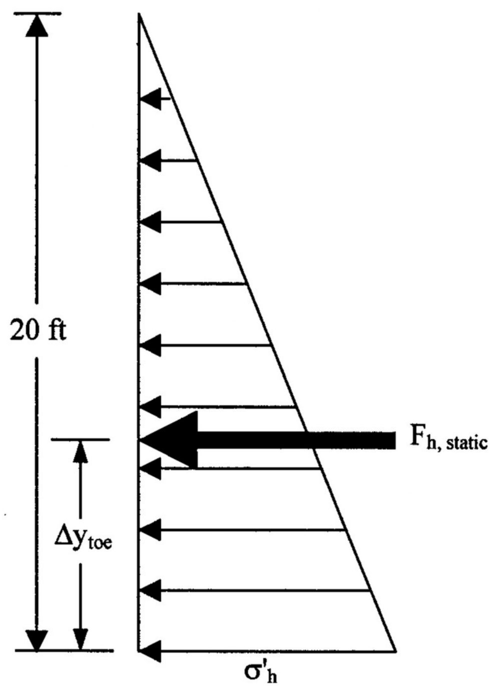
Note use of Rankine theory. If backfill is sloping, formula is different (see earlier in the slides)

$$k_h = \tan^2\left(45° - \frac{\phi'_{mob}}{2}\right) = \tan^2\left(45° - \frac{25°}{2}\right) = 0.41$$

$$\sigma'_h = \gamma_{moist} \cdot H \cdot k_h = 125\ \text{pcf} \cdot 20' \cdot 0.41 = 1013.8\ \text{psf}$$

$$F_{h,\,static} = \frac{1}{2} \cdot \sigma'_h \cdot H = \frac{1}{2} \cdot 1013.8\ \text{psf} \cdot 20' = 10{,}137.5\ \text{lbs}$$

$$\Delta y_{toe} = \frac{H}{3} = \frac{20'}{3} = 6.667\ \text{ft}$$



$$F_{h,\,static} \times \Delta y_{toe} = 10{,}137.5\ \text{lbs} \cdot 6.667' = 67{,}583\ \text{ft-lbs}$$

$$+\uparrow \sum F_v = 0 = N' - W_c - W_s$$

$$\Rightarrow N' = W_c + W_s = 8625\ \text{lbs} + 18{,}000\ \text{lbs} = 26{,}625\ \text{lbs}$$

$$\xrightarrow{+} \sum F_h = 0 = T - F_{h,\,static}$$

$$\Rightarrow T = F_{h,\,static} = 10{,}137.5\ \text{lbs}$$

$$\circlearrowleft + \sum M_{toe} = 0 = x_{N'} \cdot N' + F_{h,\,static} \cdot \Delta y_{toe} - W_c \cdot \Delta x_{toe} - W_s \cdot \Delta x_{toe}$$

$$\Rightarrow x_{N'} \cdot N' = -F_{h,\,static} \cdot \Delta y_{toe} + W_c \cdot \Delta x_{toe} + W_s \cdot \Delta x_{toe}$$

$$\Rightarrow x_{N'} = \frac{-F_{h,\,static} \cdot \Delta y_{toe} + W_c \cdot \Delta x_{toe} + W_s \cdot \Delta x_{toe}}{N'}$$

$$= \frac{-10{,}137.5\ \text{lbs} \cdot 6.667' + 8625\ \text{lbs} \cdot 5.2' + 18{,}000\ \text{lbs} \cdot 9'}{26{,}625\ \text{lbs}}$$

$= 5.23$ ft from the toe

Note that the units for forces are in pounds when they should be in pounds/ft of wall

Cantilever Example

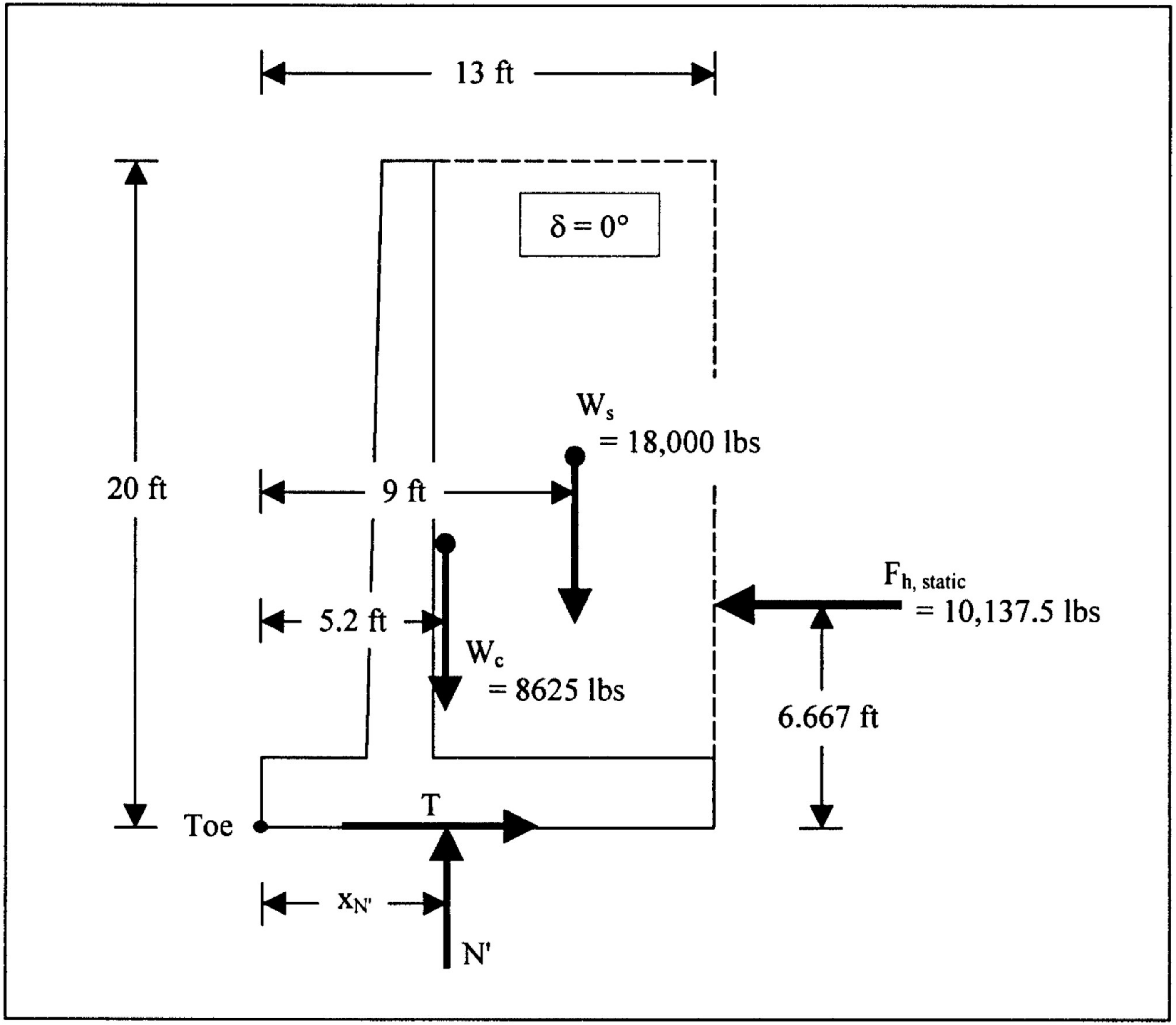


Figure A-3. Free body diagram of the structural wedge, with known forces (W_s, W_c, and $F_{h,\ static}$) and unknown forces (N' and T) identified

Cantilever Example

EM 1110-2-2502 requires the following minimum factors of safety agains sliding ($FS_{sliding}$) for cantilever retaining walls:

a. 1.5: usual loadings.

b. 1.33: short-duration (unusual) loads, such as those that might occur during high winds, construction activities, or the Operational Basis Earthquake (OBE).

c. 1.1: extreme loads, such as those that might occur during the Maximu Design Earthquake (MDE).

The $FS_{sliding}$ for usual loadings is computed as follows:

$$T_{ult} = N' \cdot \tan(\delta_{base})$$

$$\delta_{base} = \phi_{base}$$

$$= 26{,}625 \text{ lbs} \cdot \tan(35°)$$

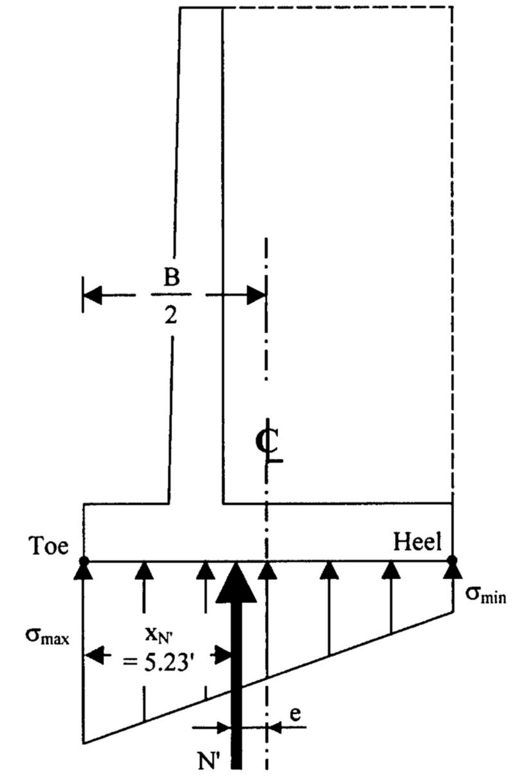
$$= 18{,}643 \text{ lbs}$$

$$FS_{sliding} = \frac{T_{ult}}{T} = \frac{18{,}643 \text{ lbs}}{10{,}137.5 \text{ lbs}} = 1.84 \geq 1.5 \;\; \textit{okay}$$

$$e = \frac{B}{2} - x_{N'} = \frac{13'}{2} - 5.23' = 1.27'$$

$$\sigma_{max} = \frac{N'}{B} \cdot \left(1 + \frac{6\,e}{B}\right) = \frac{26{,}625 \text{ lbs}}{13'} \cdot \left(1 + \frac{6 \cdot 1.27'}{13'}\right) = 3249 \text{ psf}$$

$$\sigma_{min} = \frac{N'}{B} \cdot \left(1 - \frac{6\,e}{B}\right) = \frac{26{,}625 \text{ lbs}}{13'} \cdot \left(1 - \frac{6 \cdot 1.27'}{13'}\right) = 848 \text{ psf}$$



$\sigma_{min} \geq 0$ ∴ 100% base area in compression: *okay*

Note: Per Corps design criteria, a linear effective base pressure is assumed.

Resultant of foundation load must remain in the kern. If, after overturning analysis, it does not, design must be altered (usually extending the heel to the right) until it is in the kern.

http://vulcanhammer.net/2021/12/13/more-about-resultants-in-geotechnical-engineering/

Cantilever Example

$$q_{ult} = s_c b_c l_c c N_c + s_q b_q l_q C_{wq} q_o N_q + \frac{1}{2} s_\gamma b_\gamma l_\gamma C_{w\gamma} \gamma B_f N_\gamma$$

- Shape factors ("s" factors) are unity since the foundation is continuous (true for any retaining wall)
- Base factors ("b" factors) are unity since the base is horizontal (true for this wall, may vary with others)
- Water table factors ("C_w" factors) are unity since water table is not mentioned (may be a factor for other walls)
- Base width (B_f) must be computed using equivalent footing width (see previous slide set, true for all walls)
- Assume q_0 = 0 since soil above base on left side is unreliable (true for all walls)

- "N" factors based on ϕ at the base, not the reduced value used for lateral earth pressure. In this case ϕ = 40° (see problem statement)

Table 8-2

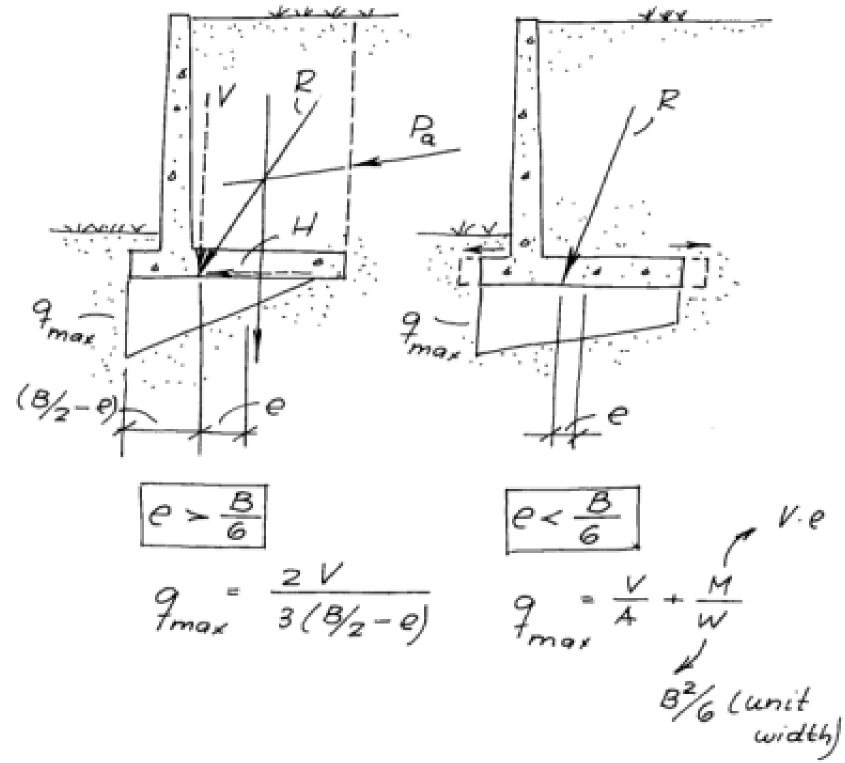
Bearing Capacity Factors (AASHTO, 2004 with 2006 Interims)

ϕ	N_c	N_q	N_γ	ϕ	N_c	N_q	N_γ
0	5.14	1.0	0.0	23	18.1	8.7	8.2
1	5.4	1.1	0.1	24	19.3	9.6	9.4
2	5.6	1.2	0.2	25	20.7	10.7	10.9
3	5.9	1.3	0.2	26	22.3	11.9	12.5
4	6.2	1.4	0.3	27	23.9	13.2	14.5
5	6.5	1.6	0.5	28	25.8	14.7	16.7
6	6.8	1.7	0.6	29	27.9	16.4	19.3
7	7.2	1.9	0.7	30	30.1	18.4	22.4
8	7.5	2.1	0.9	31	32.7	20.6	26.0
9	7.9	2.3	1.0	32	35.5	23.2	30.2
10	8.4	2.5	1.2	33	38.6	26.1	35.2
11	8.8	2.7	1.4	34	42.2	29.4	41.1
12	9.3	3.0	1.7	35	46.1	33.3	48.0
13	9.8	3.3	2.0	36	50.6	37.8	56.3
14	10.4	3.6	2.3	37	55.6	42.9	66.2
15	11.0	3.9	2.7	38	61.4	48.9	78.0
16	11.6	4.3	3.1	39	67.9	56.0	92.3
17	12.3	4.8	3.5	40	75.3	64.2	109.4
18	13.1	5.3	4.1	41	83.9	73.9	130.2
19	13.9	5.8	4.7	42	93.7	85.4	155.6
20	14.8	6.4	5.4	43	105.1	99.0	186.5
21	15.8	7.1	6.2	44	118.4	115.3	224.6
22	16.9	7.8	7.1	45	133.9	134.9	271.8

Load Inclination Factors

$$l_c = l_q = \left(1 - \frac{\delta}{90°}\right)^2$$

$$l_\gamma = \left(1 - \frac{\delta}{\phi}\right)^2$$



- where
  - ***δ*** = angle formed by the resultant of the loads between the normal and tangential loads on the base
  - ***δ*** = arctan (T/N')
  - ***δ*** = arctan (10,137.5 lbs/26,625 lbs) = 20.8°
  - ***ϕ*** = 40° (see problem statement)

- l_γ = (1-20.8/40)^2 = 0.23
- l_c = l_q = (1-20.8/90)^2 = 0.591 (but not used in this case)
- Additional check: since we assume that the ***ϕ*** at the base is the same as the friction angle between the soil and the base, T_{max} = N' tan(***ϕ***) = (26625) tan(40) = 22341 lbs > 10137.5 lbs

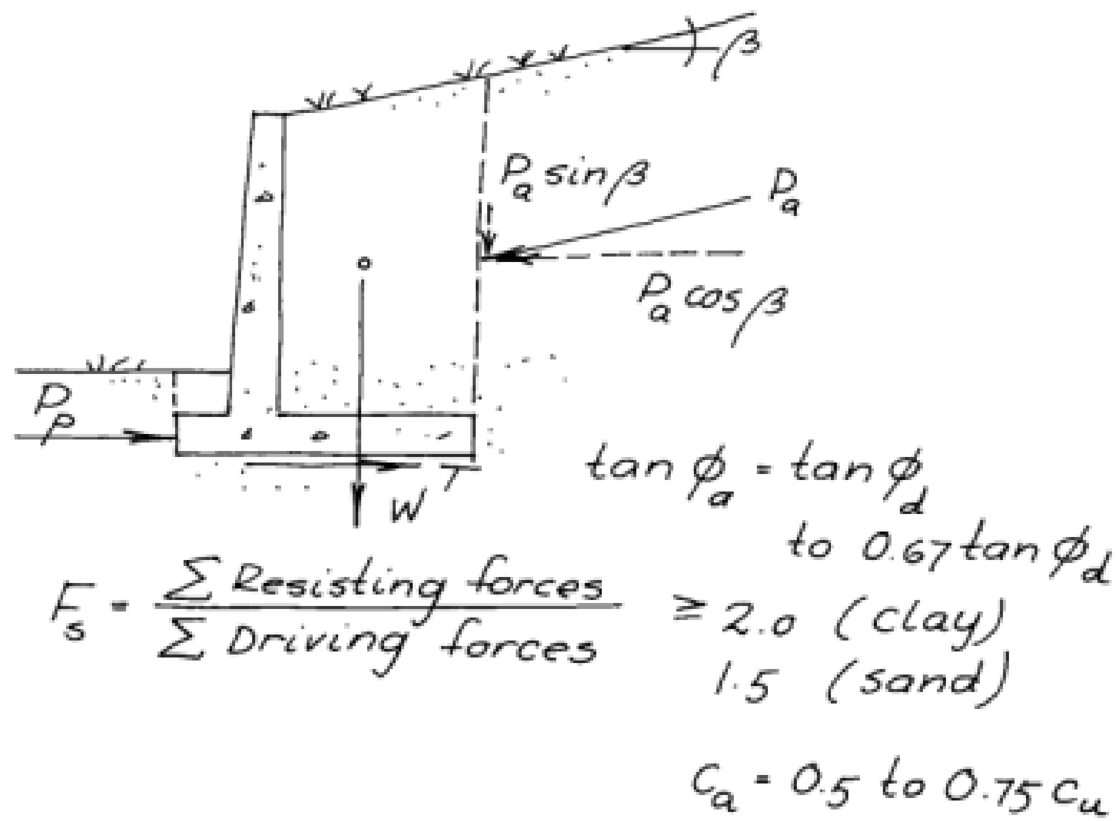
Cantilever Example

$$q_{ult} = s_c b_c l_c c N_c + s_q b_q l_q C_{wq} q_o N_q + \frac{1}{2} s_\gamma b_\gamma l_\gamma C_{w\gamma} \gamma B_f N_\gamma$$

- Compute equivalent footing width
  - B' = B - 2e = 13 - (2)(1.27) = 10.46' (which is what we will use for B_f)
- As noted earlier, all of the other factors (shape, etc.) are unity
- First two terms are zero (see earlier slide)
- We compute as follows:
  - q_{ult} = (0.5)(1)(1)(0.23)(1)(10.46)(125)(109.4) = 16,450 psf
  - $Q_{ult} = q_{ult} B_f$ = (16,450)(10.46) = 172,067 lbs. = 172 kips
- Compute the factor of safety for bearing capacity failure
  - N = 26,625 lbs
  - FS = Q_{ult}/N = 172067/26625 = 6.46 (acceptable)
- Summary
  - FS applied to driving force of soil wedge
  - Overturning moment analysis shows no "lift-off" at base, resultant within the kern
  - Maximum tangent force at base is greater than driving force of soil wedge
  - FS of bearing capacity is acceptable
- If c > 0, then this will have to be included in the analysis

Effect of Sloping Backfill

- With sloping backfill, we have another vertical force at the heel of the wall, which produces a clockwise moment around the toe
  - The weight also produces a clockwise moment
  - The horizontal component of the soil produces a counterclockwise moment
  - The clockwise moments are resisting, the counterclockwise moments are overturning
  - They are handled as shown to determine the FS against overturning



Lateral Squeeze

10.5.4.2 Deep Foundations

CIP walls founded on a deep foundation may be subject to potentially damaging ground and structural displacements at sites underlain by cohesive soils. Such damage may occur if the weight of the backfill material exceeds the bearing capacity of the cohesive subsoils causing plastic displacement of the ground beneath the retaining structure and heave of the ground surface in front of the wall. When the cohesive soil layer is located at or below the base of the wall, the factor of safety against this type of bearing capacity failure can be approximated by the following equation (Peck, *et al.*, 1974):

$$FS = \frac{5c}{(\gamma H + q)} \qquad 10\text{-}15$$

where H is the height of the fill, γ is the unit weight of fill, c is the shear strength of the cohesive soil and q is the uniform surcharge load.

The computed factor of safety should not be less than 2.0 for the embankment loading. Below this value progressive lateral movements of the retaining structure are likely to occur (Peck, *et al.*, 1974). As the factor of safety decreases, the rate of movement will increase until failure occurs at a factor of safety of unity. For CIP walls founded on vertical piles or drilled shafts, this progressive ground movement would be reflected by an outward displacement of the wall. CIP walls founded on battered piles typically experience an outward displacement of the wall base and a backward tilt of the wall face (Figure 10-19).

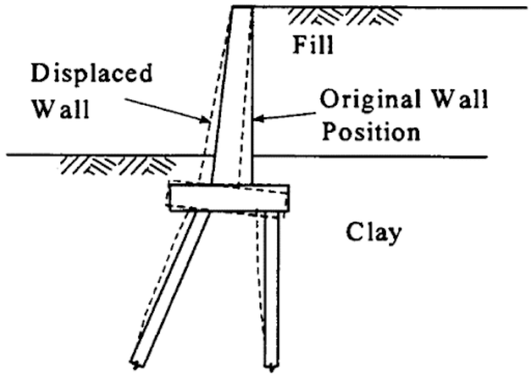
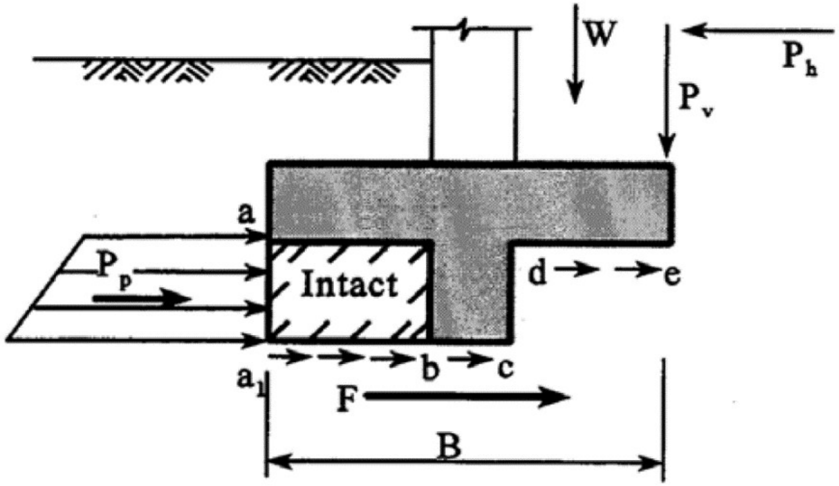


Figure 10-19. Typical Movement of pile-supported cast-in-place (CIP) wall with soft foundation.

Keyed Foundation for Sliding Resistance

If the wall is supported by rock, granular soils or stiff clay, a key may be installed below the foundation to provide additional resistance to sliding. The method for calculating the contribution of the key to sliding resistance is shown in Figure 10-20.



Cohesive Soils:

$$F = (W + P_v)\mathrm{Tan}\,\delta_b + c_a(B - \overline{a_1 b}) + c(\overline{a_1 b}) + P_p$$

Granular Soils:

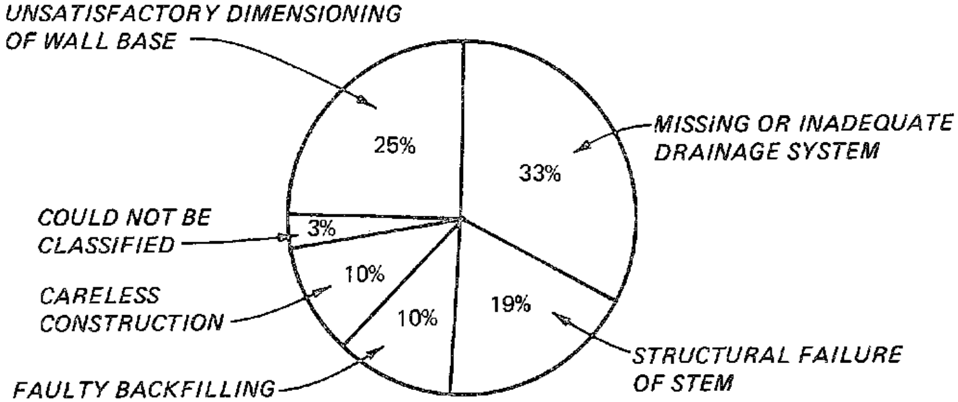
$$F = (W + P_v)\mathrm{Tan}\,\delta_b + P_p$$

Factor of Safety $\quad FS = F/P_h$

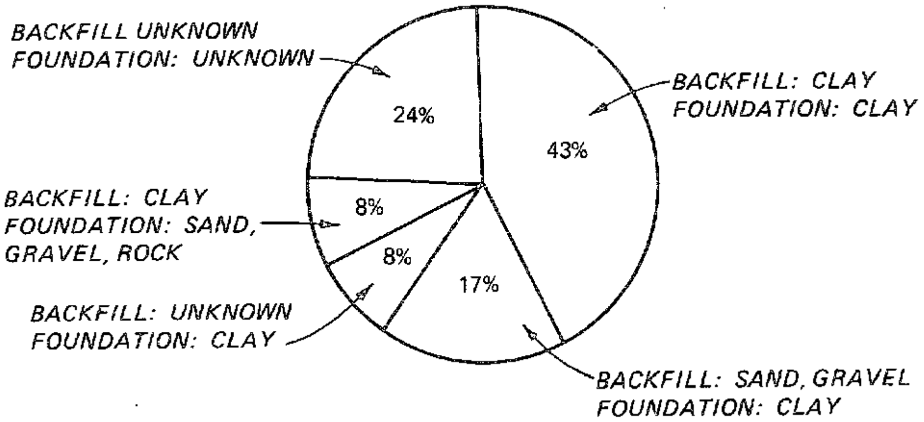
Note: See Figure 10-17 for list of symbol definitions.

Figure 10-20. Resistance against sliding from keyed foundation.

Failure Causes for Retaining Walls

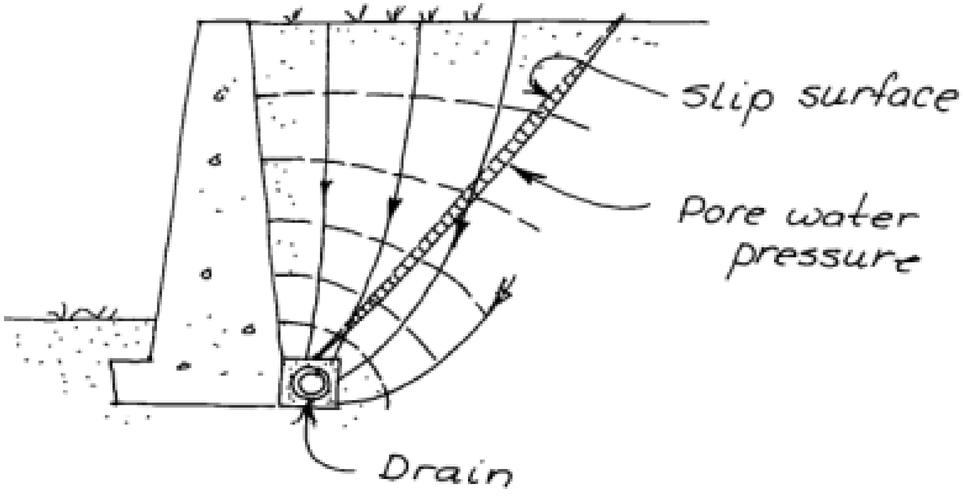


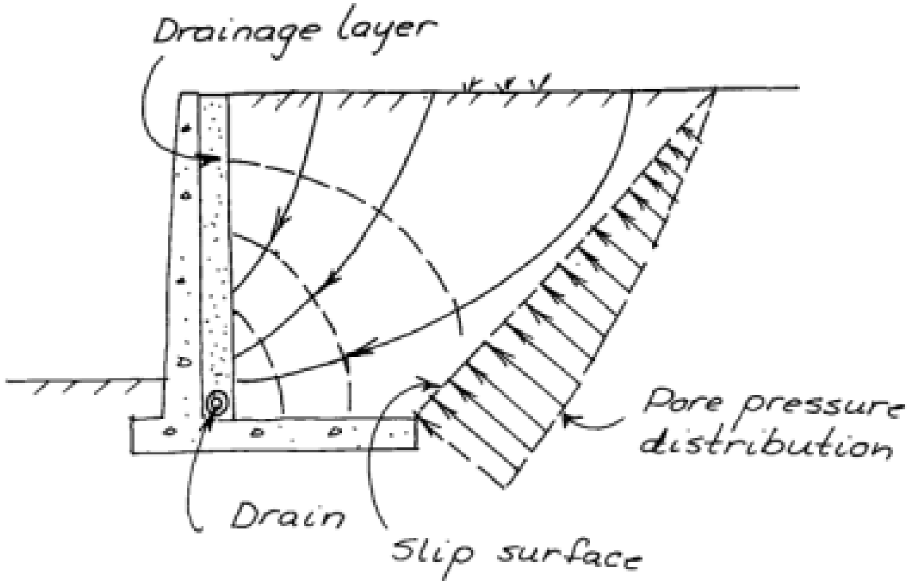
a. Causes of failure of rigid concrete retaining walls (Techeng and Iseux 1972)

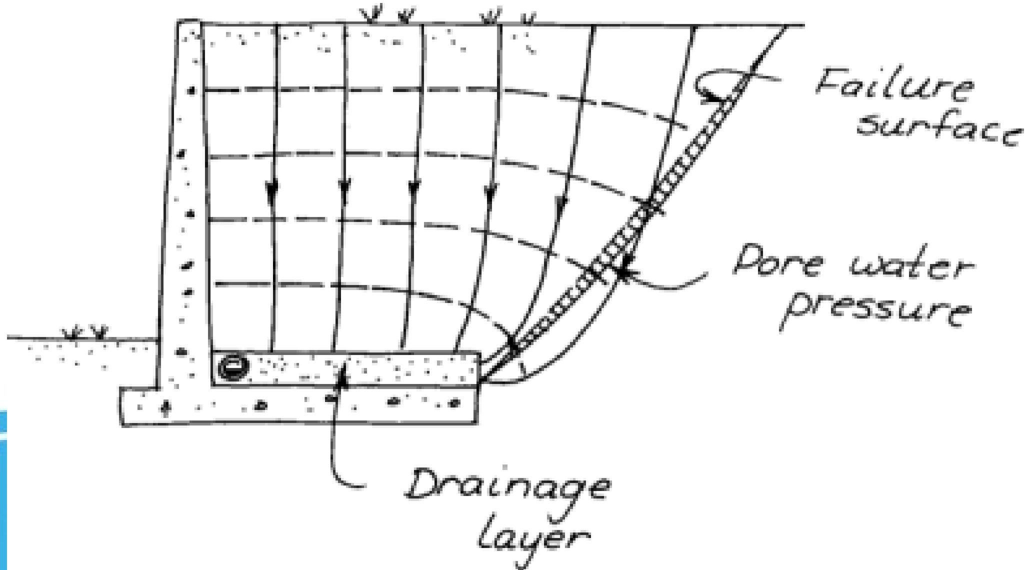


b. Foundation and backfill material of unsatisfactory retaining walls (Ireland 1964)

Importance of Proper Drainage







Retaining Wall Drainage

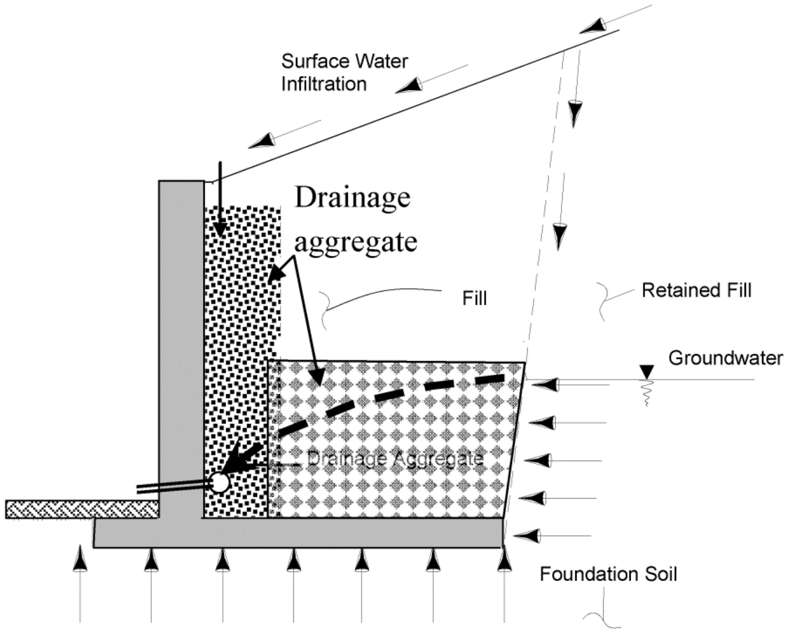


Figure 10-22. Potential sources of subsurface water.

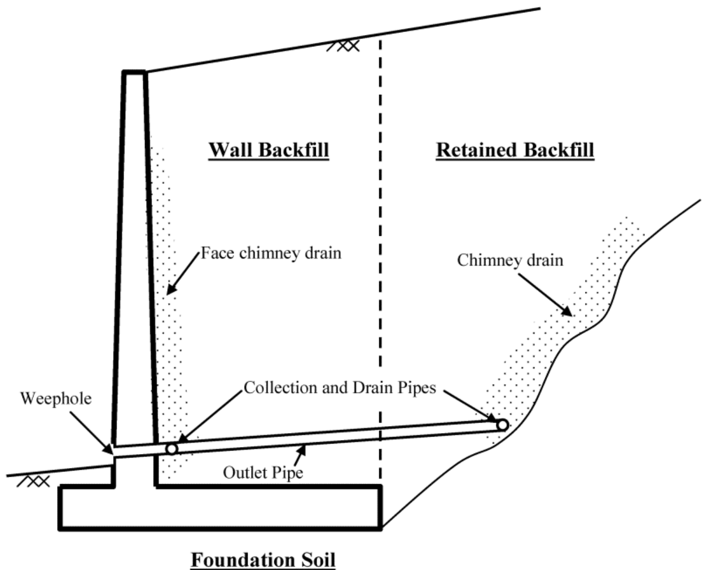


Figure 10-24. Drains behind backfill in cantilever wall in a cut situation.

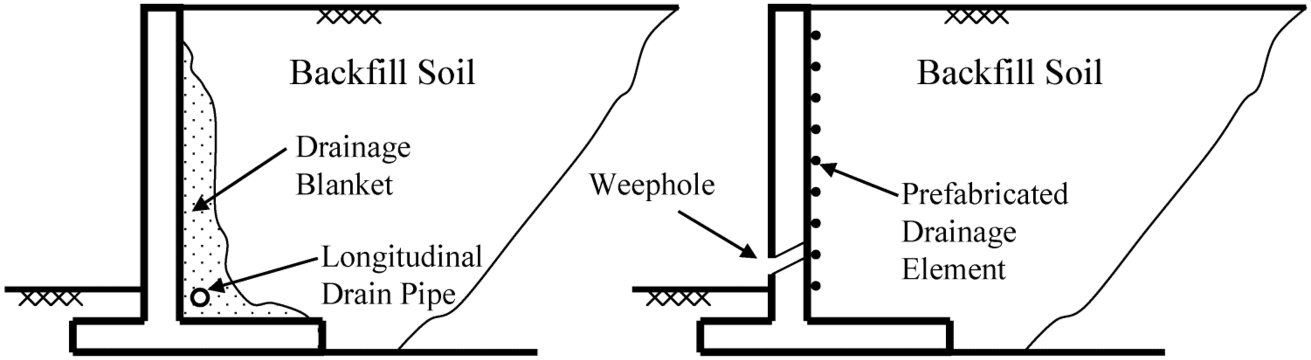


Figure 10-23. Typical retaining wall drainage alternatives.

Mechanically Stabilized Earth (MSE) Walls




Overview of MSE Walls

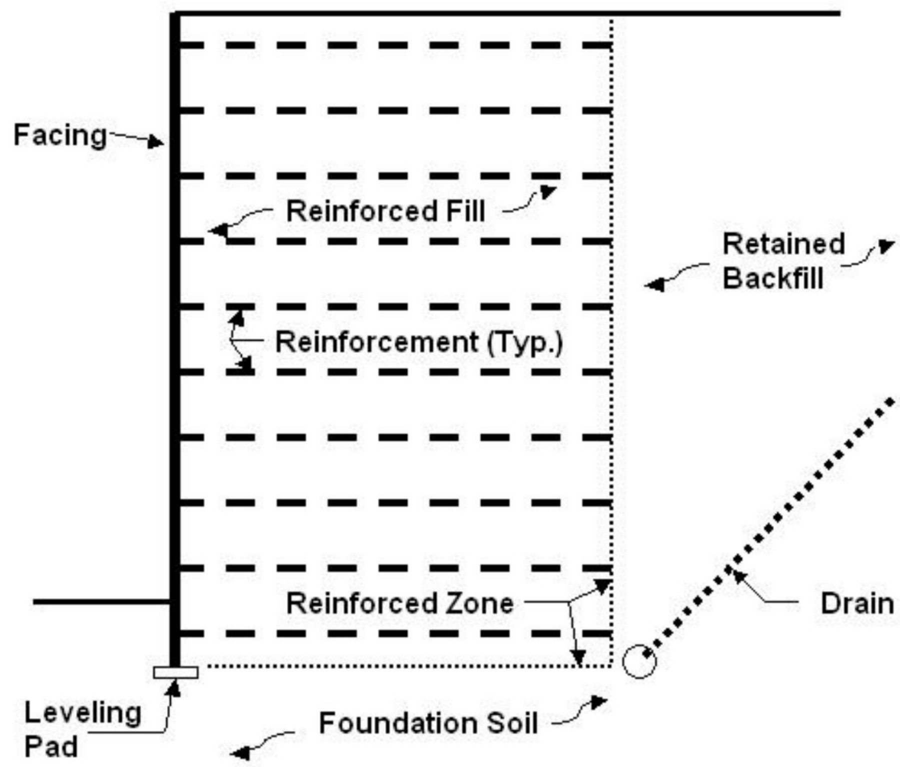
- Definitions
  - Mechanically Stabilized Earth Wall (MSEW) is a generic term that includes reinforced soil (a term used when multiple layers of inclusions act as reinforcement in soils placed as fill).
  - Geosynthetics is a generic term that encompasses flexible polymeric materials used in geotechnical engineering such as geotextiles, geomembranes, geonets, and grids (also known as geogrids).

- Definitions

Facing is a component of the reinforced soil system used to prevent the soil from raveling out between the rows of reinforcement. Common facings include precast concrete panels, dry cast modular blocks, metal sheets and plates, gabions, welded wire mesh, shotcrete, wood lagging and panels, and wrapped sheets of geosynthetics. The facing also plays a minor structural role in the stability of the structure. For RSS structures it usually consists of some type of erosion control material.

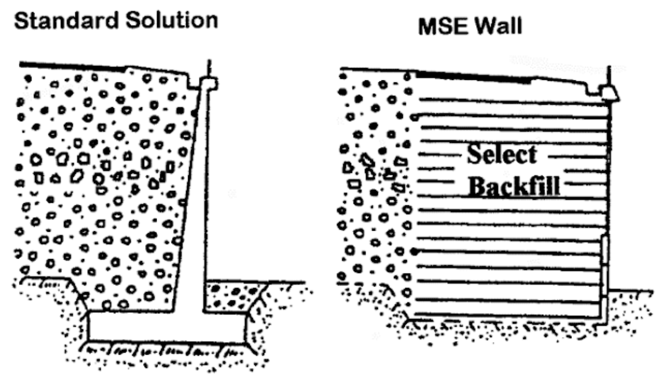
Overview of MSE Walls

- Retained backfill is the fill material located between the mechanically stabilized soil mass and the natural soil.
- Reinforced backfill is the fill material in which the reinforcements are placed.
- Foundation Soil is the soil under the MSE wall and its reinforced backfill
- All three of these can be different (generally,) or the same (infrequently)

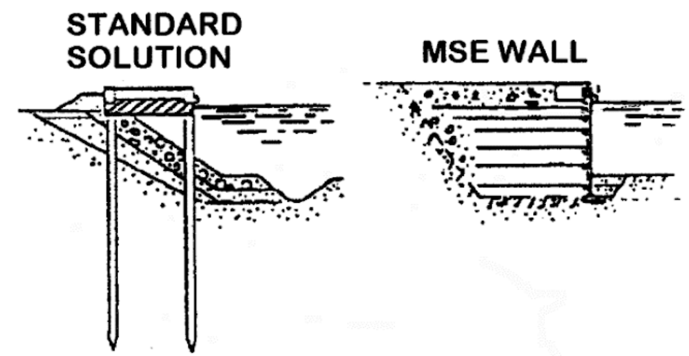


Applications of MSE Walls

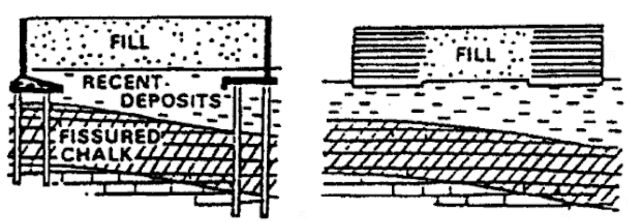
Retaining Wall



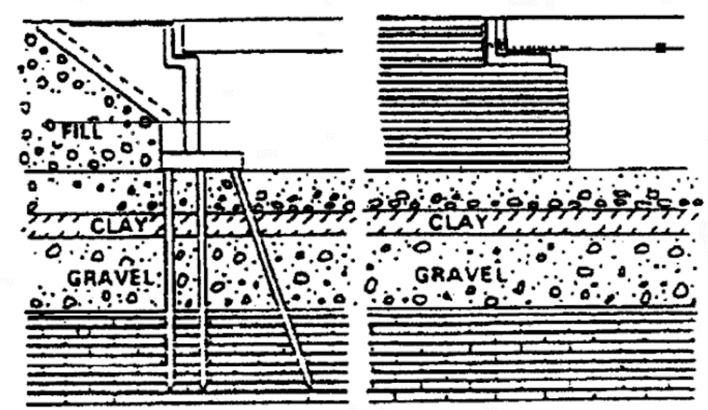
MARINE WALL



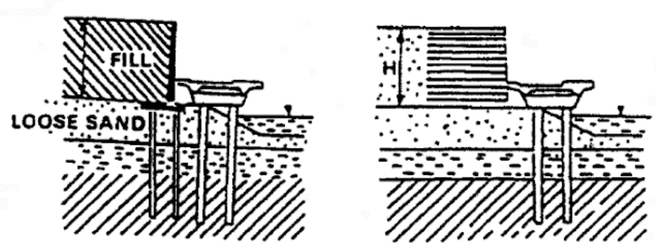
Bridge Approach Fill Over Compressible Foundation



BRIDGE ABUTMENT



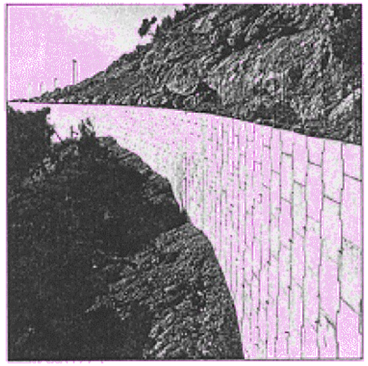
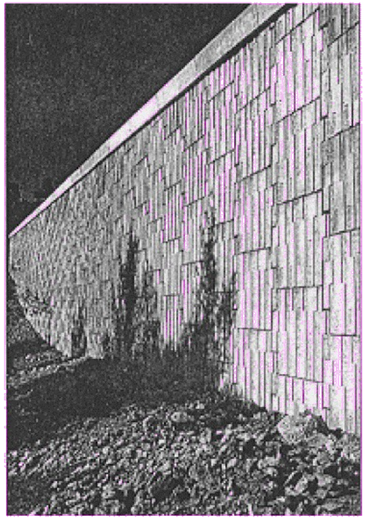
Interchange with Access Ramps

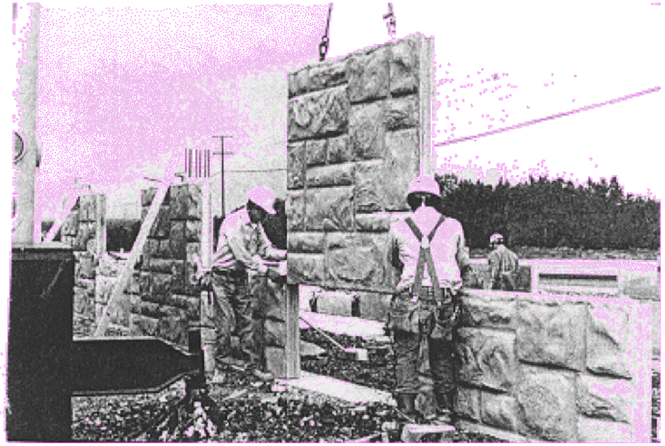
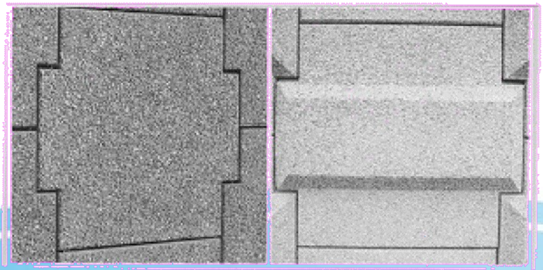


Advantages and Disadvantages of MSE Walls

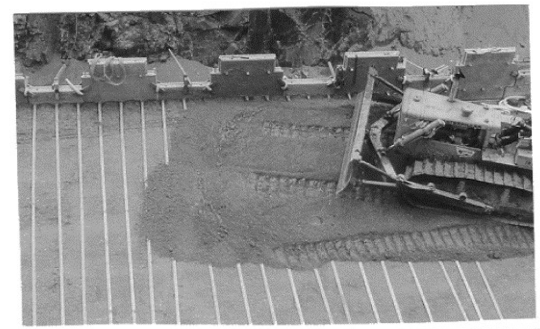
- Advantages
  - Use simple and rapid construction procedures and do not require large construction equipment.
  - Do not require experienced craftsmen with special skills for construction.
  - Require less site preparation than other alternatives.
  - Need less space in front of the structure for construction operations.
  - Reduce right-of-way acquisition.
  - Do not need rigid, unyielding foundation support because MSE structures are tolerant to deformations.
  - Are cost effective.
  - Are technically feasible to heights in excess of 25 m (80 ft).
- Disadvantages
  - Require a relatively large space behind the wall or outward face to obtain enough wall width for internal and external stability.
  - MSEW require select granular fill. (At sites where there is a lack of granular soils, the cost of importing suitable fill material may render the system uneconomical).
  - Suitable design criteria are required to address corrosion of steel reinforcing elements, deterioration of certain types of exposed facing elements such as geosynthetics by ultra violet rays, and potential degradation of polymer reinforcement in the ground.
  - Since design and construction practice of all reinforced systems are still evolving, specifications and contracting practices have not been fully standardized, especially for RSS.
  - The design of soil-reinforced systems often requires a shared design responsibility between material suppliers and owners and greater input from agencies geotechnical specialists in a domain often dominated by structural engineers.

MSE Wall Facings

MSE Wall Construction



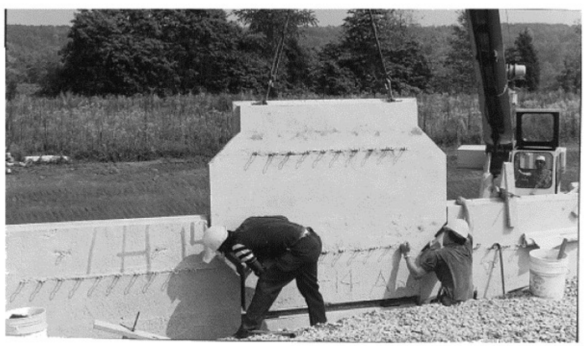
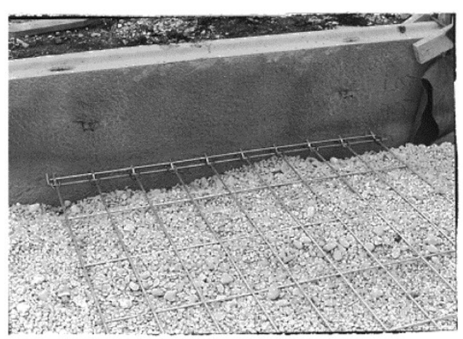


Figure 13. Compaction of backfill.



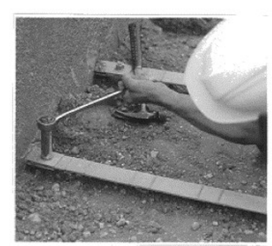


Figure 11. Erection of precast panels.

Figure 12. Fill spreading and reinforcement connection.

Types of Reinforcement

- Steel Strips
  - The currently commercially available strips are ribbed top and bottom, 50 mm (2 inches) wide and 4 mm (5/32-inch) thick. Smooth strips 60 to 120 mm (2-d to 4¾-inch) wide, 3 to 4 mm (c to 5/32-inch) thick have been used.
- Steel Grids
  - Welded wire grid using 2 to 6 W7.5 to W24 longitudinal wire spaced at either 150 or 200 mm (6 or 8 inches). The transverse wire may vary from W11 to W20 and are spaced based on design requirements from 230 to 600 mm (9 to 24 inches). Welded steel wire mesh spaced at 50 by 50 mm (2 by 2-inch) of thinner wire has been used in conjunction with a welded wire facing. Some MBW systems use steel grids with 2 longitudinal wires.
- Geogrids
  - High Density Polyethylene (HDPE) geogrid. These are of uniaxial manufacture and are available in up to 6 styles differing in strength.
  - PVC coated polyester (PET) geogrid. Available from a number of manufacturers. They are characterized by bundled high tenacity PET fibers in the longitudinal load carrying direction. For longevity the PET is supplied as a high molecular weight fiber and is further characterized by a low carboxyl end group number.
- Geotextiles
  - High strength geotextiles can be used principally in connection with reinforced soil slope (RSS) construction. Both polyester (PET) and polypropylene (PP) geotextiles have been used.

Steel Reinforcement

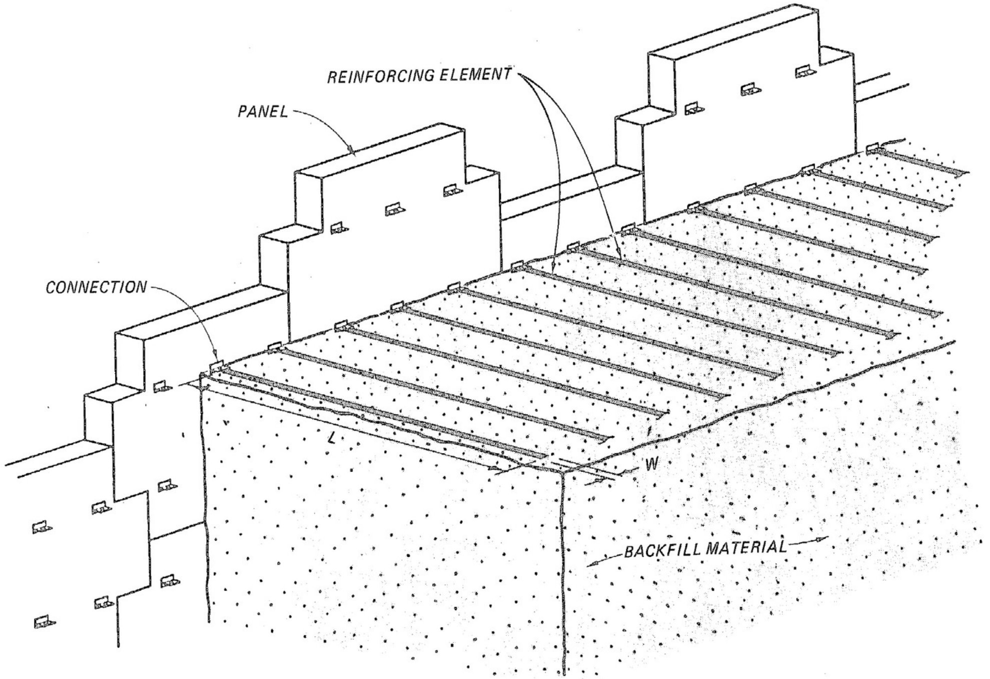
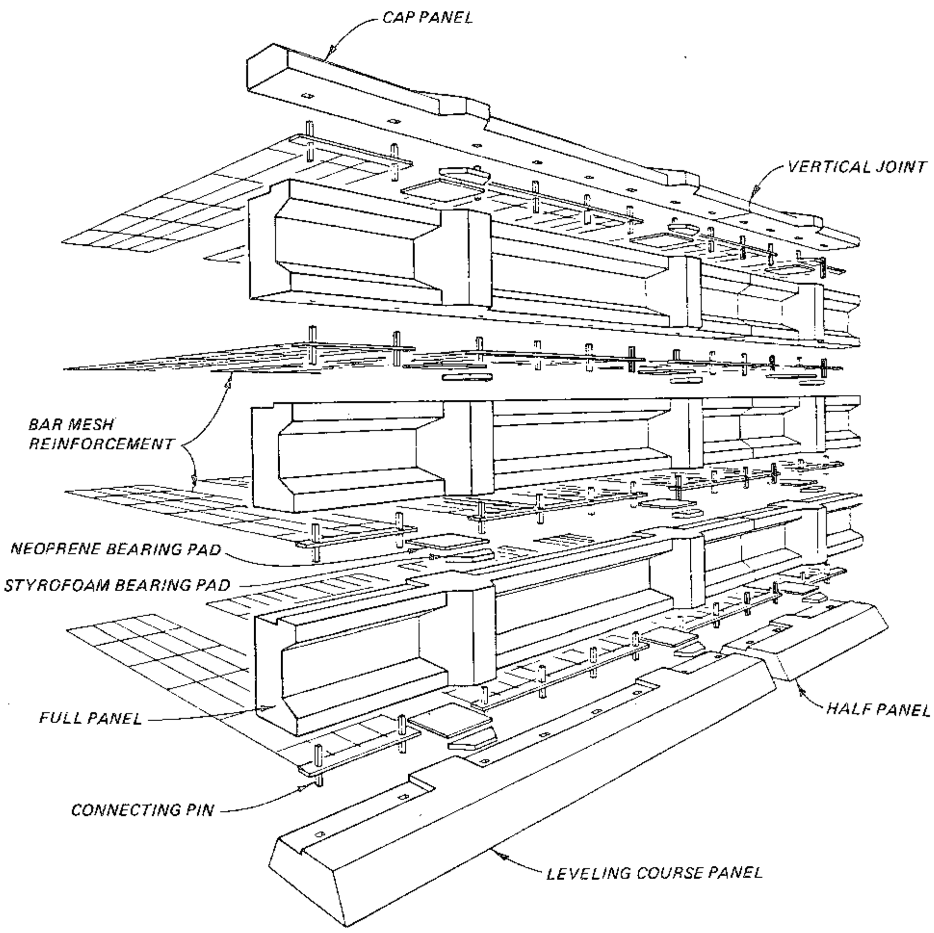
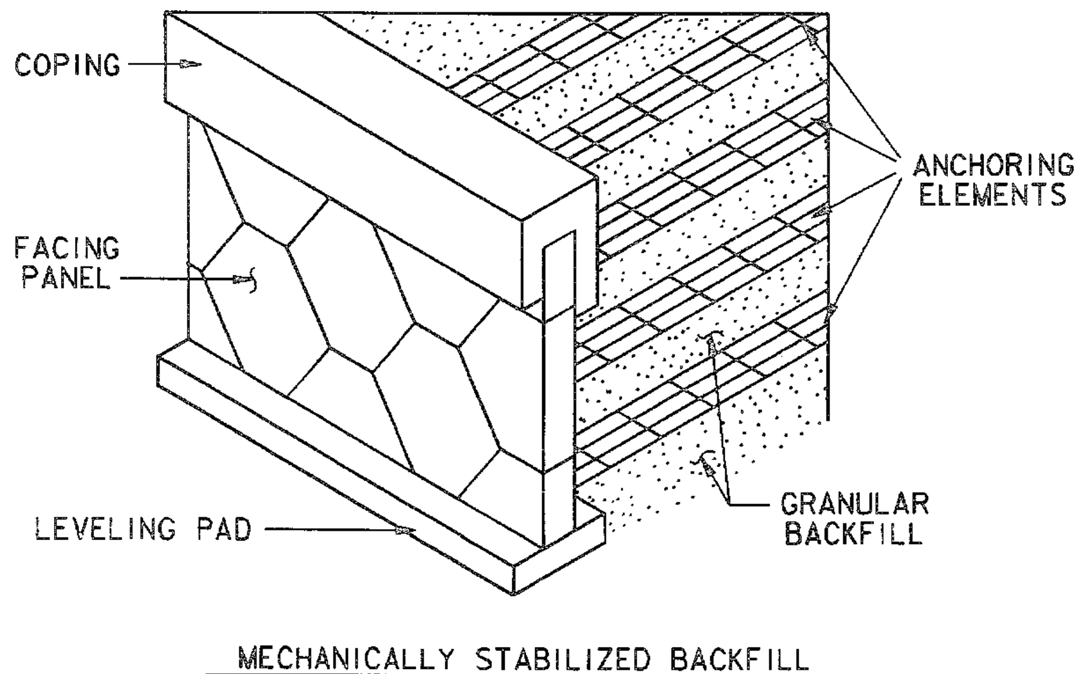
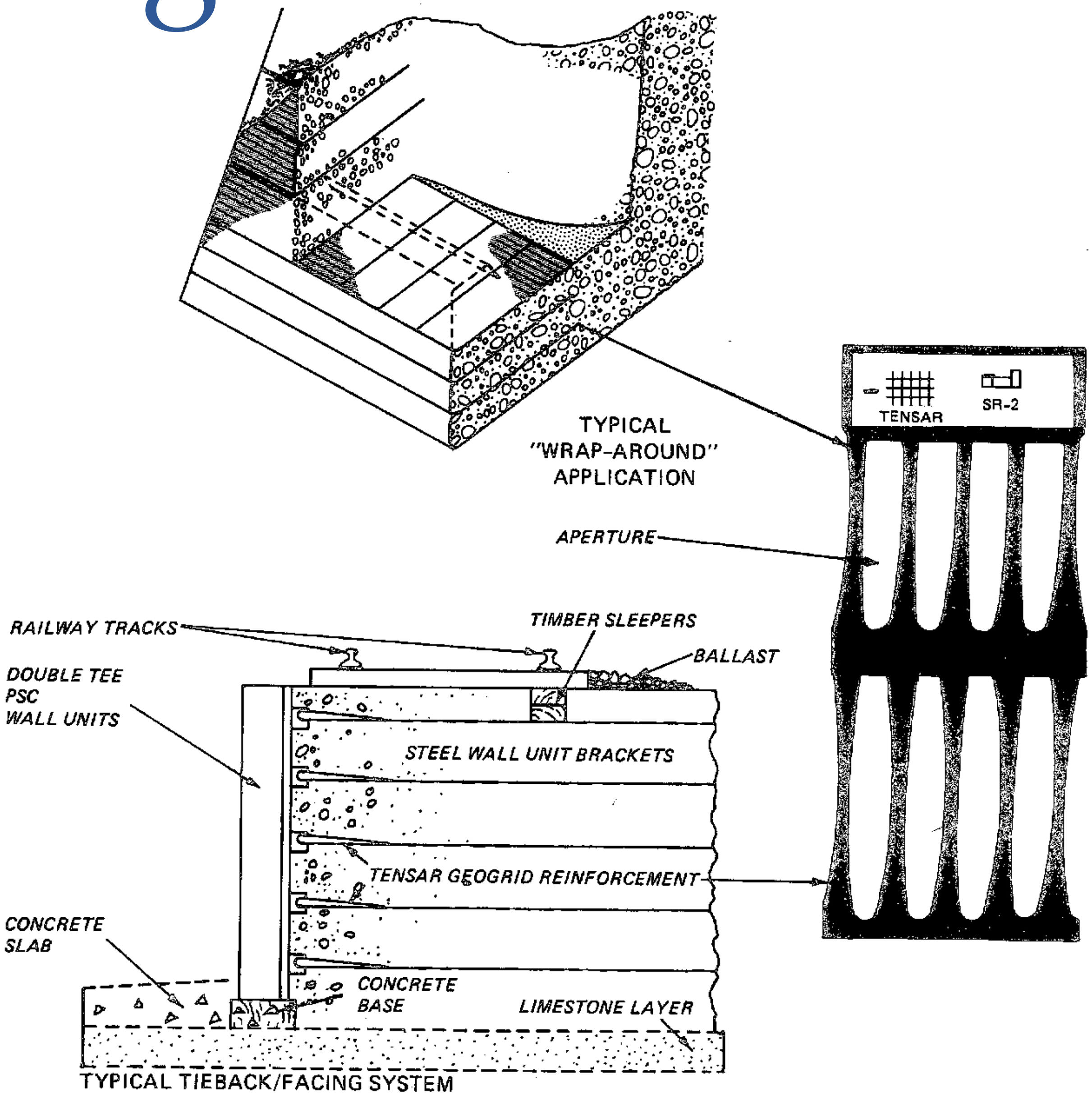


Figure 10-1. Schematic diagram of reinforced earth retaining wall





Geogrid Reinforcement



MSE Backfill

- MSE walls require high quality backfill for durability, good drainage, constructability, and good soil reinforcement interaction which can be obtained from well graded, granular materials. Many MSE systems depend on friction between the reinforcing elements and the soil. In such cases, a material with high friction characteristics is specified and required. Some systems rely on passive pressure on reinforcing elements, and, in those cases, the quality of backfill is still critical. These performance requirements generally eliminate soils with high clay contents.
- From a reinforcement capacity point of view, lower quality backfills could be used for MSEW structures; however, a high quality granular backfill has the advantages of being free draining, providing better durability for metallic reinforcement, and requiring less reinforcement. There are also significant handling, placement and compaction advantages in using granular soils. These include an increased rate of wall erection and improved maintenance of wall alignment tolerances.

MSE Wall Design for This Course

- MSE Walls can have a wide variety of configurations and applications. To simplify the initial learning process, the MSE walls considered in this course will be subject to the following limitations:
  - Vertical walls (no sloping walls or reinforced slopes)
  - Level backfills (no sloping backfills)
  - Uniform surcharge, if any surcharge (no application of Boussinesq or other theory to simulate the application of point, line or strip loads)
  - Uniform length of reinforcement strips
    - As will be shown, reinforcement strip length can vary from the standpoint of geotechnical pullout resistance, but other problems arise (computation of sliding and overturning resistance becomes difficult)
  - Spacing of reinforcement strips is uniform. Also, distance from top reinforcement strip to the surface (and bottom to the base) is half the spacing
- Notes for Geo-Wall competition: all of these restrictions can be waived for actual design. Proper implementation of these will require a spreadsheet. Method with the above restrictions can be used for initial design. Lifting the restrictions will generally require a spreadsheet or software solution, which you can use in the project.

Design of MSE Walls

- External Stability

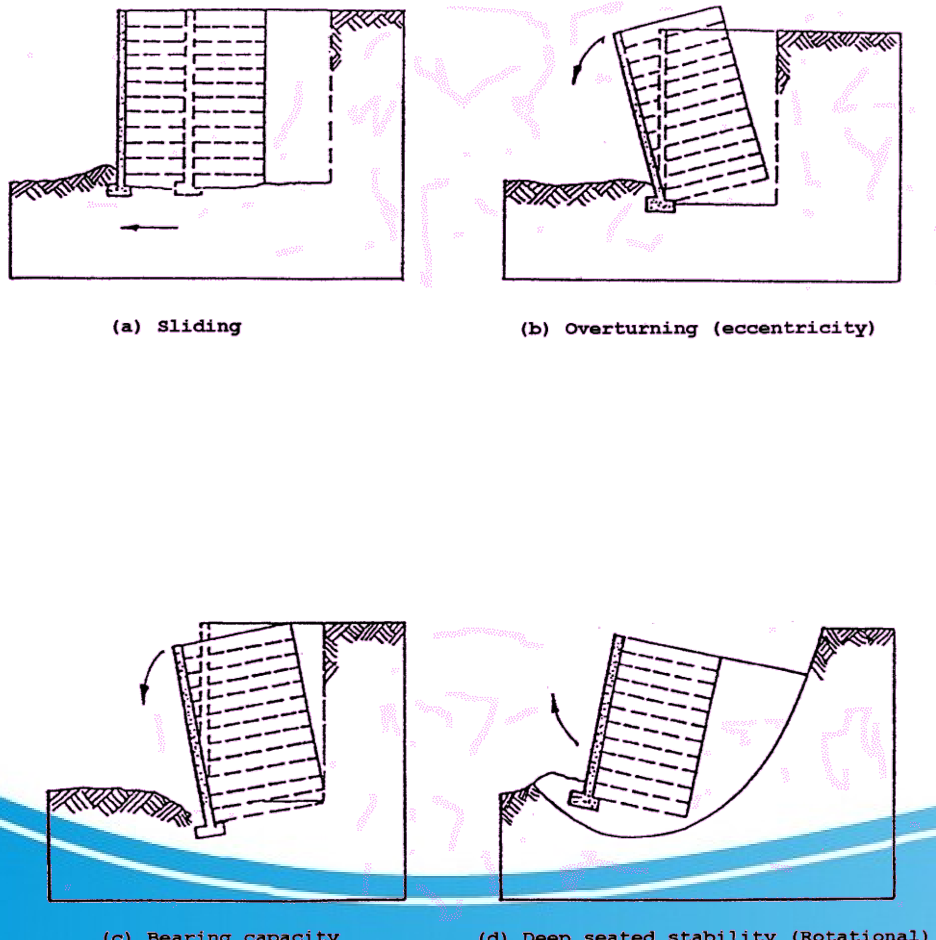
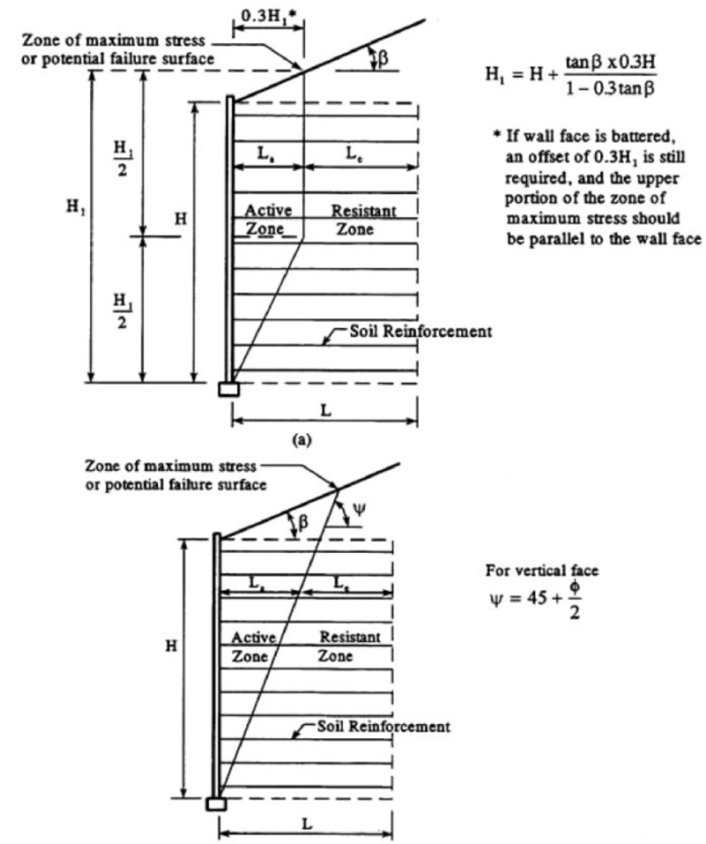


Figure 21. Potential external failure mechanisms for a MSE wall.

- Internal Stability



For walls with a face batter angle (θ) 10° or more from the vertical,

$$\tan(\psi-\theta)=\frac{-\tan(\phi-\beta)+\sqrt{\tan(\phi-\beta)[\tan(\phi-\beta)+\cot(\phi+\theta-90)][1+\tan(\delta+90-\theta)\cot(\phi+\theta-90)]}}{1+\tan(\delta+90-\theta)[\tan(\phi-\beta)+\cot(\phi+\theta-90)]}$$

with $\delta=\beta$

θ = wall batter angle (b)

For wall with a broken backslope, use $\delta=\beta_{ca}$

Figure 4-9. Location of potential failure surface for internal stability design of MSE Walls (a) inextensible reinforcements and (b) extensible reinforcements.

Steps in MSE Wall Design

- External Stability
  - Sliding of the reinforced block
  - Overturning of the reinforced block
  - Bearing capacity of the foot (not emphasised in this course)
  - Global Failure (not discussed in this course)
- Internal Stability
  - Pullout of the reinforcing strips from the reinforced soil
  - Breakage of the reinforcing strips from the wall

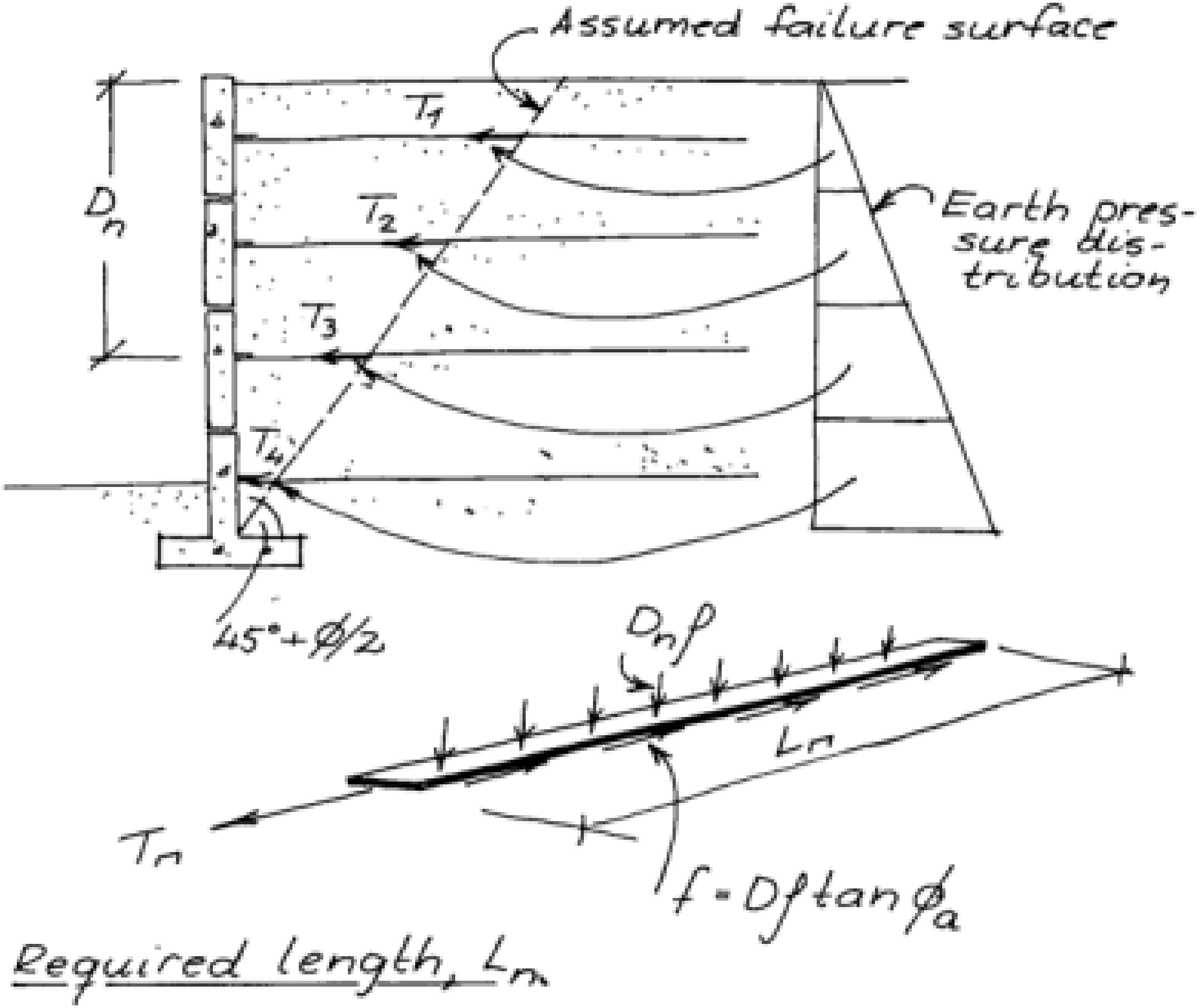
- Pullout Failure
  - The failure surface divides the reinforcement strips/grids into two regions
  - The region to the left of the failure surface
    - Length L_r depends upon the location of the failure surface at a given elevation
    - This region cannot contribute to the pull resistance of the reinforcements
  - The region to the right of the failure surface
    - Length L_e depends upon the length of the strips/grids needed to resist the pull of the wall
  - The total length of the reinforcement at any level in the system is the sum of these two
  - Both can vary with each level but in some cases L_e can be simplified

Internal Stability

- Computation of L_r
  - This will vary with elevation
  - It is zero at the base of the wall
  - It increases as shown as the failure surface slopes upward from the base, depending upon the type of reinforcement
  - This can be laid out in CAD for exact values for each layer

- Computation of L_e
  - L_e is the result of the static equilibrium between the wall pulling on the reinforcement to the left (T_{req}) and the frictional resistance of the soil acting on the reinforcement in the resistant zone (T_{ten})
- We include a factor of safety in this computation

Internal Stability: Forces on the Geogrids/Strips



Internal Stability

Allowable tension pull on geogrid based on frictional interaction of geogrid with soil:

$$T_{ten} = 2C_iC_rL_e\sigma'_v tan\delta'$$

where

- C_i =interaction coefficient
- C_r =coverage ratio

Both unity for our purposes

- L_e =length of geogrid embedment with soil which can resist pullout (with MSE walls, the soil which is past the Rankine failure surface)
- σ'_v =vertical effective stress at the level of the geogrid
- δ' =friction angle between soil and geogrid/strips, degrees or radians

Earth Pressure on strip of wall which is supported by a given level of geogrid

$$T_{req} = s_v\sigma'_h$$

where

- s_v =vertical spacing of geogrid sheets, m or ft
- σ'_h =horizontal (lateral) earth pressure

Internal Stability

Earth Pressure on strip of wall which is supported by a given level of geogrid

$$T_{req} = s_v \sigma'_h$$

where

- s_v =vertical spacing of geogrid sheets, m or ft
- σ'_h =horizontal (lateral) earth pressure

Keep in mind that

$$\sigma'_h = K_a \sigma'_v$$

where

- K_a =active earth pressure coefficient (usually we use Rankine theory)
- σ'_v =vertical effective stress, including any effects of surface loading

Internal Stability

Pullout factor of safety:

$$FS_{pullout} = \frac{T_{ten}}{T_{req}}$$

- $FS_{pullout}$ = factor of safety against pulling the reinforcement from the soil
- T_{ten} =allowable tension pull on geogrid based on frictional interaction of geogrid with soil, kN/m or kips/ft of wall and geogrid
- T_{req} = strength required by the application, kN/m or kips/ft of wall and geogrid

For design to be valid,

$$T_{req} FS_{pullout} \leq T_{ten}$$

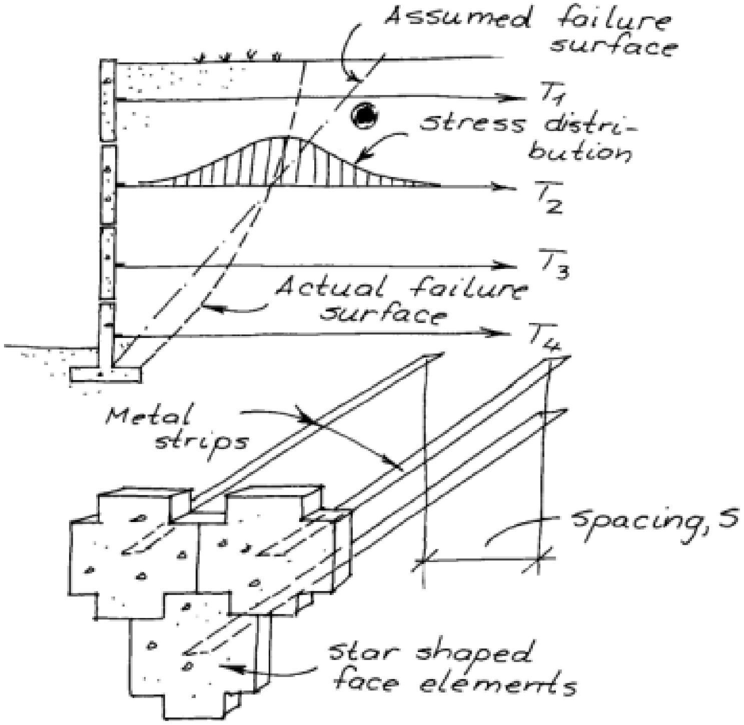
Substituting,

$$s_v \left(\sigma'_h FS_{pullout}\right) \leq L_e \left(2 C_i C_r \sigma'_v tan\delta'\right)$$

Notes:

- This can be used to solve for either the geogrid embedment or the vertical spacing of the geogrid sheets
- With MSE walls, the backfill is chosen and the wall is porous, so no water effects are included

Structural Integrity of the Geogrids



To prevent failure of the geogrids themselves:

$$FS_{mech} = \frac{T_{allow}}{T_{req}}$$

where

- FS_{mech} = factor of safety
- T_{allow} = allowable tensile strength of geogrid, based on laboratory testing, kN/m or kips/ft of wall and geogrid
- T_{req} = strength required by the application, kN/m or kips/ft of wall and geogrid

Structural Integrity of the Geogrids

Allowable tension pull on geogrid based on mechanical properties of material:

$$T_{allow} = \frac{T_{ult}}{RF_{ID} \times RF_{CR} \times RF_{CD} \times RF_{BD}}$$

where

- T_{ult} = ultimate tensile strength of geogrid before reduction due to environmental factors, kN/m or kips/ft of wall and geogrid
- RF_{ID} = reduction factor for installation damage = 1.1 to 1.4 for MSE walls
- RF_{CR} = reduction factor to avoid creep over the lifetime of the structure = 2.0 to 3.0 for MSE walls
- RF_{CD} = reduction factor against chemical degradation = 1.1 to 1.4 for MSE walls
- RF_{BD} = reduction factor against biological degradation = 1.0 to 1.2 for MSE walls Assume this to be equal to unity

For design to be valid,

$$T_{req} FS_{mech} \leq T_{allow}$$

MSE Wall Factors of Safety (ASD Design)

Recommended minimum factors of safety with respect to failure modes are as follows:

! External Stability

Sliding	:	F.S. $\geq$ 1.5 (MSEW); 1.3 (RSS)
Eccentricity e, at Base	:	$\leq$ L/6 in soil L/4 in rock
Bearing Capacity	:	F.S. $\geq$ 2.5
Deep Seated Stability	:	F.S. $\geq$ 1.3
Compound Stability	:	F.S. $\geq$ 1.3
Seismic Stability	:	F.S. $\geq$ 75% of static F.S. (All failure modes)

! Internal Stability

Pullout Resistance	:	F.S. $\geq$ 1.5 (MSEW and RSS)
Internal Stability for RSS	:	F.S $\geq$ 1.3
Allowable Tensile Strength		
for steel strip reinforcement	:	0.55 F_y
for steel grid reinforcement	:	0.48 F_y (connected to concrete panels or blocks)
for geosynthetic reinforcements	:	T_a - See design life, below

MSE Wall Design Criteria

A number of site specific project criteria need to be established at the inception of design:

! **Design limits and wall height**. The length and height required to meet project geometric requirements must be established to determine the type of structure and external loading configurations.

! **Alignment limits**. The horizontal (perpendicular to wall face) limits of bottom and top of wall alignment must be established as alignments vary with batter of wall system. The alignment constraints may limit the type and maximum batter, particularly with MBW units, of wall facing.

! **Length of reinforcement**. A minimum reinforcement length of 0.7H is recommended for MSE walls. Longer lengths are required for structures subject to surcharge loads. Shorter lengths can be used in special situations.

! **External loads**. The external loads may be soil surcharges required by the geometry, adjoining footing loads, line loads as from traffic, and/or traffic impact loads. Traffic line loads and impact loads are applicable where the traffic lane is located horizontally from the face of the wall within a distance less than one half the wall height. The magnitude of the minimum traffic loads outlined in Articles 3.20.3 and 5.8 of current AASHTO, is a uniform load equivalent to 0.6 m (2 ft) of soil over the traffic lanes.

! **Wall embedment**. The minimum embedment depth for walls from adjoining finished grade to the top of the leveling pad should be based on bearing capacity, settlement and stability considerations. Current practice based on local bearing capacity considerations, recommends the following embedment depths:

Slope in Front of Wall	Minimum to Top of Leveling Pad
horizontal (walls)	H/20
horizontal (abutments)	H/10
3H:1V	H/10
2H:1V	H/7
3H:2V	H/5

Larger values may be required, depending on depth of frost penetration, shrinkage and swelling of foundation soils, seismic activity, and scour. Minimum in any case is 0.5 m, except for structures founded on rock at the surface, where no embedment may be used. Alternately, frost-susceptible soils could be overexcavated and replaced with non frost susceptible backfill, hence reducing the overall wall height.

A minimum horizontal bench 1.2 m (4 ft) wide as measured from the face shall be provided in front of walls founded on slopes.

For walls constructed along rivers and streams where the depth of scour has been reliably determined, a minimum embedment of 0.6 m (2 ft) below this depth is recommended.

Embedment is not required for RSS unless dictated by stability requirements.

! **Seismic Activity**. Due to their flexibility, MSE wall and slope structures are quite resistant to dynamic forces developed during a seismic event, as confirmed by the excellent performance in several recent earthquakes.

The peak horizontal ground acceleration for each site can be obtained from Section 3 of AASHTO Division 1-A, Seismic Design. For sites where the Acceleration Coefficient "A" in AASHTO is less or equal to 0.05, static design considerations govern and dynamic performance or design requirements may be omitted.

For sites where the Acceleration Coefficient is greater than 0.29, significant total lateral structure movements may occur, and a seismic design specialist should review the stability and potential deformation for the structure. All sites where the "A" coefficient is greater than 0.05 should be designed/checked for seismic stability. For RSS structures, seismic analyses should be included regardless of acceleration.

! **Tolerance of precast facing panels to settlement**. MSE structures have significant deformation tolerance both longitudinally along a wall and perpendicular to the front face. Therefore, poor foundation conditions seldom preclude their use. However, where significant differential settlement are anticipated (greater than 1/100) sufficient joint width and/or slip joints must be provided to preclude panel cracking. This factor may influence the type and design of the facing panel selected.

Square panels generally adapt to larger longitudinal differential settlements better than long rectangular panels of the same surface area. Guidance on minimum joint width and limiting differential settlements that can be tolerated is presented in table 3, for panels with a surface area typically less than 4.5 m^2 (50 ft^2)

MSE walls constructed with full height panels should be limited to differential settlements of 1/500. Walls with drycast facing (MBW) should be limited to settlements of 1/200. For walls with welded wire facings, the limiting differential settlement should be 1/50.

Where significant differential settlement perpendicular to the wall face is anticipated, the reinforcement connection may be overstressed. Where the back of the reinforced soil zone will settle more than the face, the reinforcement could be placed on a sloping fill surface which is higher at the back end of the reinforcement to compensate for the greater vertical settlement. This may be the case where a steep surcharge slope is constructed. This latter construction technique, however, require that surface drainage be carefully controlled after each day's constructior Alternatively, where significant differential settlements are anticipated, groun improvement techniques may be warranted to limit the settlements, as outlined i geological conditions.

Table 3. Relationship between joint width and limiting differential settlements for MSE precast panels.

Joint Width	Limiting Differential Settlement
20 mm	1/100
13 mm	1/200
6 mm	1/300

Design Example

- Given
  - Project Nature
    - A typical urban highway retaining wall design with precast concrete panels
    - Geogrid reinforcement
    - Vertical spacing of strips = 750 mm (based on selection of wall configuration.) The first row is located 375 mm from the topmost panel
  - Design Height, External Loads
    - Total design height H = 7.8 m, to gutter grade.
    - Required panel height = 7.5 m vertical.

- Given
  - Foundation Soils
    - $\varphi' = 30^\circ$. (clayey sand, dense,) friction angle of foundation same as internal friction angle
    - Allowable bearing capacity - 300 kPa.
    - Differential settlements on the order of 1/300 are estimated.
  - Reinforced (Wall) Backfill
    - $\varphi = 34^\circ$, $\gamma_T = 18.8$ kN/m^3, $\delta = 11^\circ$, $K_a = 0.28$ (Rankine)
  - Retained Backfill
    - $\varphi = 30^\circ$, $\gamma_T = 18.8$ kN/m^3, $K_a = 0.33$ (Rankine)

Design Example

- **Given**
  - Factors of Safety
    - External Stability
      - Sliding = 1.5.
      - Maximum foundation pressure < allowable bearing capacity.
      - Eccentricity < L/6
      - Global stability > 1.3.
    - Internal Stability
      - Pullout = 1.5.
      - Mechanical = 1.5.
- **Find**
  - MSE Wall to suit these parameters

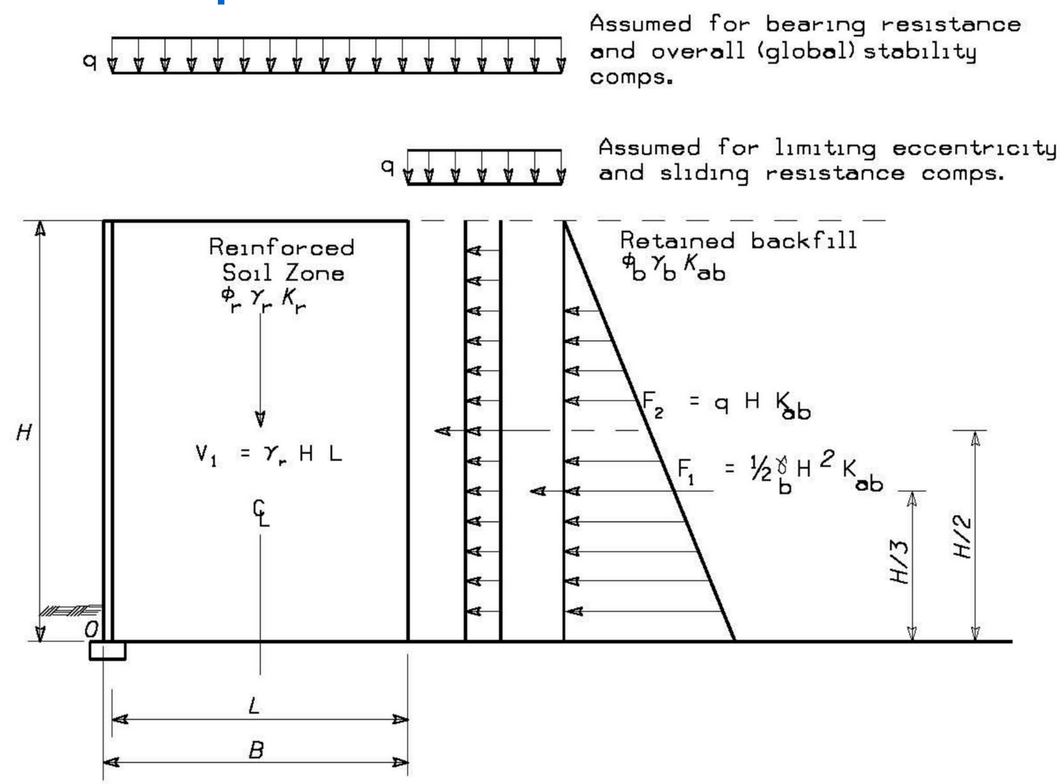


Figure 4-2. External analysis: nominal earth pressures; horizontal backslope with traffic surcharge (after AASHTO, 2007).

Simplified Computation of L_e

- It can be simplified under the following conditions:
  - Uniform spacing of the reinforcement levels
  - Distance from top reinforcement level to surface of the soil is half the spacing
  - Distance from bottom reinforcement level to base of wall is half the spacing
  - Uniform surcharge at soil surface
  - These are true with our simplifying assumptions

Simplified Computation of L_e

Substituting for lateral earth pressure,

$$s_v\left(K_a\sigma'_v FS_{pullout}\right) \leq L_e\left(2C_iC_r\sigma'_v tan\delta'\right)$$

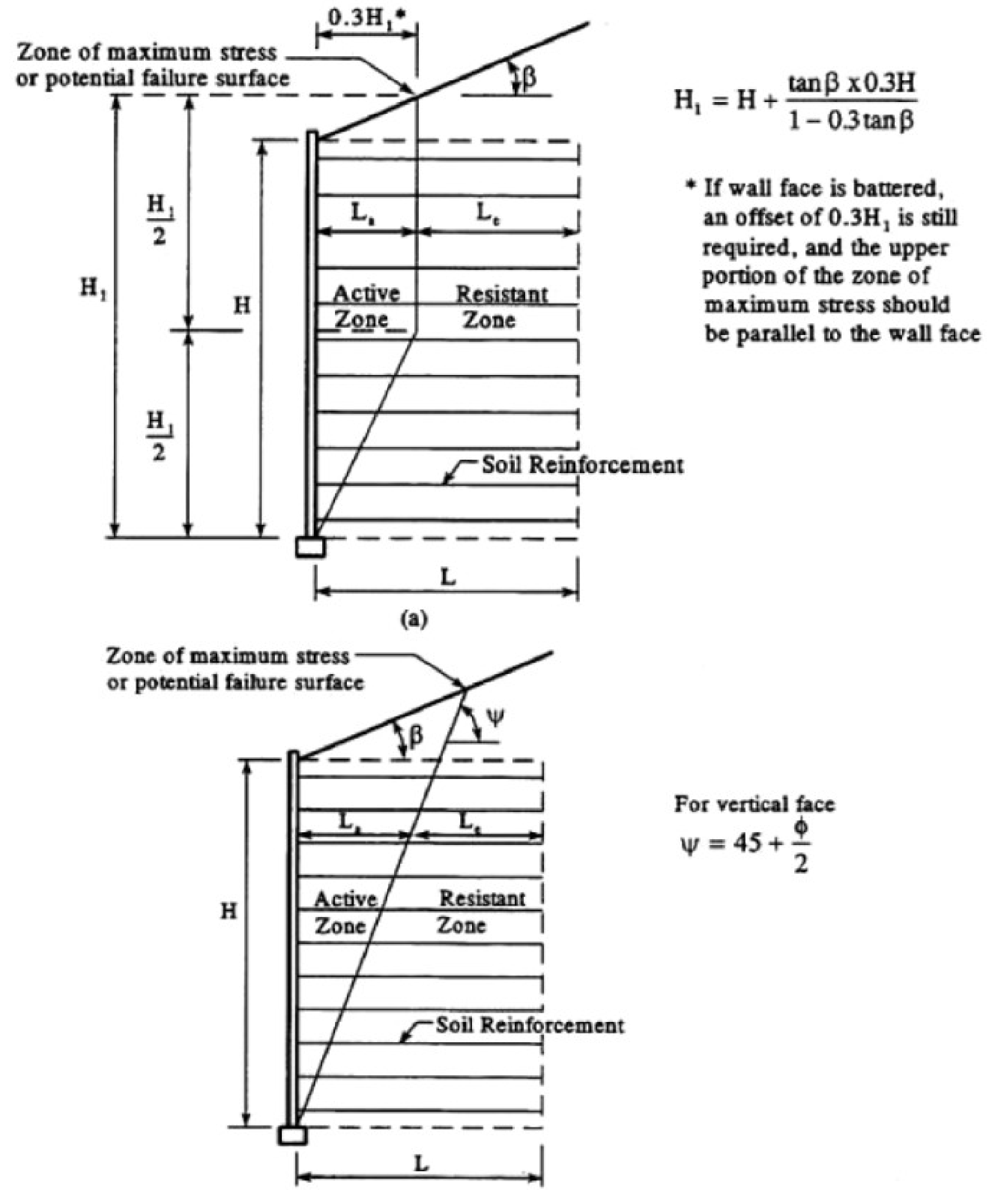
Solving for L_e,

$$L_e \geq \frac{s_v\left(K_a FS_{pullout}\right)}{\left(2C_iC_r tan\delta'\right)}$$

- Using the strip at z = 0.75 m and substituting,
  L_e = ((0.75 m)(0.28)(1.5)/((2)(1)(1)tan(11°)) $\geq$ 0.82 m
- $L = L_r + L_e$, which will vary with depth

Computation of L_r

- First, we compute $\boldsymbol{\psi}$ = 45° + $\boldsymbol{\phi}$/2 = 45° + 34°/2 = 62°
- From bottom of wall, $L_r = z_{bottom} * \tan(90° - \boldsymbol{\psi})$
- From top of wall, $L_r = (H - z_{top}) * \tan(90° - \boldsymbol{\psi})$
  - These computations are complicated for inextensible reinforcement (see right)
- Example: L_r for topmost strip
  - z_{top} = 0.375 m
  - H = 20 m
  - L_r = (7.8 - 0.375) * tan (90° - 62°) = 3.95 m
  - The strip at the top drives the length of the (longest) strip, thus $L = L_r + L_e$ = 3.95 + 0.82 = 4.77 or 5 m



For walls with a face batter angle (θ) 10° or more from the vertical,

$$\tan(\psi - \theta) = \frac{-\tan(\phi - \beta) + \sqrt{\tan(\phi - \beta)[\tan(\phi - \beta) + \cot(\phi + \theta - 90)][1 + \tan(\delta + 90 - \theta)\cot(\phi + \theta - 90)]}}{1 + \tan(\delta + 90 - \theta)[\tan(\phi - \beta) + \cot(\phi + \theta - 90)]}$$

with $\delta = \beta$

θ = wall batter angle (b)

For wall with a broken backslope, use $\delta = \beta_{cs}$

Figure 4-9. Location of potential failure surface for internal stability design of MSE Walls (a) inextensible reinforcements and (b) extensible reinforcements.

Structural Integrity of Reinforcement

- To generally pick a reinforcement, first perform the purely geotechnical calculations
- If final inequality is not met, pick a new geogrid with a higher T_{allow} and recalculate as necessary
- May be necessary to reduce spacing as well

To prevent failure of the geogrids themselves:

$$FS_{mech} = \frac{T_{allow}}{T_{req}}$$

where

- FS_{mech} = factor of safety
- T_{allow} = allowable tensile strength of geogrid, based on laboratory testing, kN/m or kips/ft of wall and geogrid
- T_{req} = strength required by the application, kN/m or kips/ft of wall and geogrid

Allowable tension pull on geogrid based on mechanical properties of material:

$$T_{allow} = \frac{T_{ult}}{RF_{ID} \times RF_{CR} \times RF_{CD} \times RF_{BD}}$$

where

- T_{ult} = ultimate tensile strength of geogrid before reduction due to environmental factors, kN/m or kips/ft of wall and geogrid
- RF_{ID} = reduction factor for installation damage = 1.1 to 1.4 for MSE walls
- RF_{CR} = reduction factor to avoid creep over the lifetime of the structure = 2.0 to 3.0 for MSE walls
- RF_{CD} = reduction factor against chemical degradation = 1.1 to 1.4 for MSE walls
- RF_{BD} = reduction factor against biological degradation = 1.0 to 1.2 for MSE walls

For design to be valid,

$$T_{req} FS_{mech} \leq T_{allow}$$

Structural Integrity of Reinforcement

- Assumptions for this problem only
  - Start with geogrid T_{ult} = 600 kN/m
  - Use average values for the RF factors (except biological, which is unity)
- T_{allow} = 600/((1.25)(2.5)(1.25)(1)) = 153.6 kN/m
- T_{req} is the pulling force of the wall on the lowest geogrid, in this case the one at the bottom of the wall, at z = 7.13 m
  - It is the left hand side of the equation that computes L_e
  - T_{req} = (0.75)(0.28)(7.13)(18.9)(1.5) = 42.4 kN/m
  - FS_{mech} = 153.6/42.4 = 3.62

To prevent failure of the geogrids themselves:

$$FS_{mech} = \frac{T_{allow}}{T_{req}}$$

where

- FS_{mech} = factor of safety
- T_{allow} = allowable tensile strength of geogrid, based on laboratory testing, kN/m or kips/ft of wall and geogrid
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Allowable tension pull on geogrid based on mechanical properties of material:

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- RF_{CD} = reduction factor against chemical degradation = 1.1 to 1.4 for MSE walls
- RF_{BD} = reduction factor against biological degradation = 1.0 to 1.2 for MSE walls

For design to be valid,

$$T_{req} FS_{mech} \leq T_{allow}$$

Notes on Sliding and Overturning

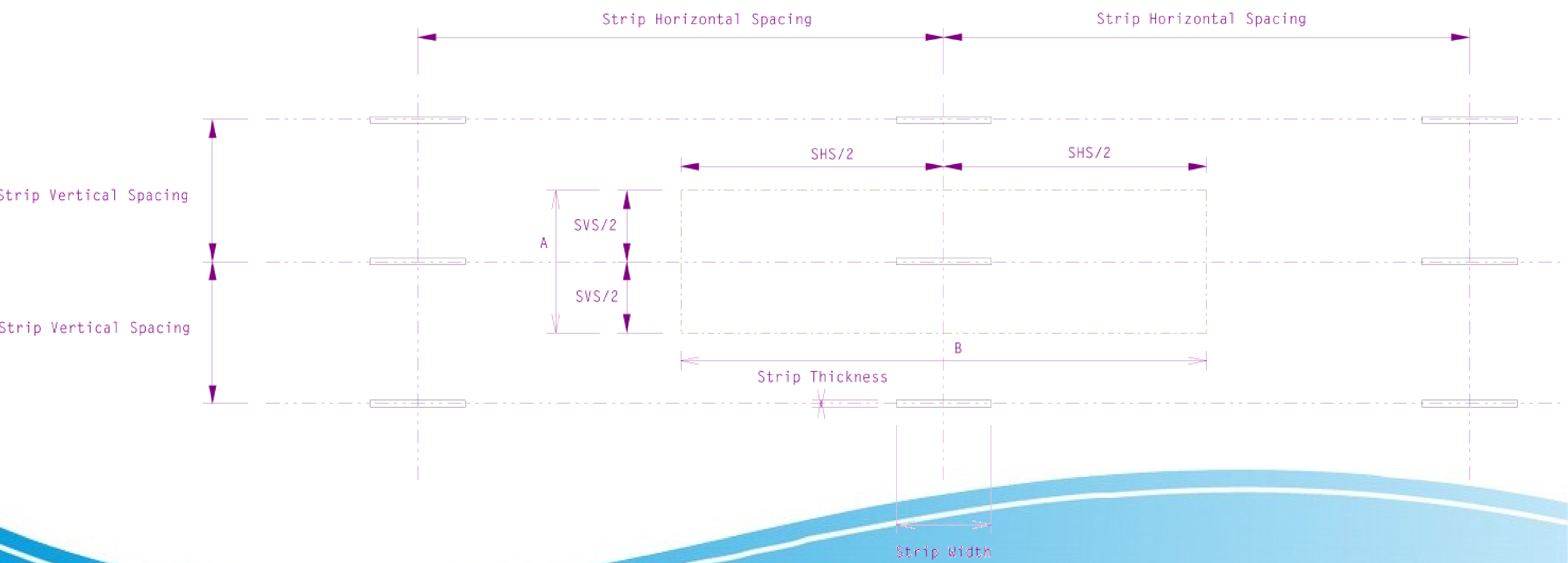
- Sliding Forces
  - Resisting: friction along base, based on weight of reinforced backfill and friction with foundation soil
  - Driving: “triangle” pressure of retained backfill on reinforced backfill
- Overturning Forces
  - Resisting: weight of reinforced backfill, acting at its centroid
  - Driving: same as sliding, acting 1/3 from bottom of reinforced backfill
- Bearing capacity and settlement of footing same as with gravity walls

Sliding and Overturning

- Sliding
  - H = 7.8 m, L = 5 m
  - Weight of reinforced soil = (7.8)(5)(18.8) = 733.2 kN/m
  - Driving Force: Force of retained backfill = (0.5)(0.33)(7.8)^2(18.8) = 190.63 kN/m
  - Resisting Force: Weight of reinforced soil times the friction coefficient (733.2)(tan 30°) = 423.3 kN/m
  - Note that, if there were a surcharge on top of the wall, the total force of the surcharge would be added to the resisting force (and would also affect the vertical and horizontal stresses in the wall)
  - FS = 423.3/190.63 = 2.22
- Overturning
  - To determine this we take the moments around the base of the wall
  - The driving moment is the driving force of the retained backfill times the moment arm from the base upwards, i.e. $M_{driving}$ = (190.63)(7.8/3) = 495.6 kN-m/m
  - The resisting moment is the weight of the reinforced soil times half the length of the strips/reinforced area, i.e., $M_{resisting}$ = (733.2)(5/2) = 1833 kN-m/m
  - FS = 1833/495.6 = 3.7
  - In the event either FS is inadequate, the simplest remedy is to lengthen the strips/reinforced area until they are. In some cases this will drive the length of the strips

Basic Concept of MSE Reinforcement Design (Elevation View) with Steel Strips (Also applicable to GeoWall)

Elevation View of MSE Wall for Static Equlibrium Against Sliding



Driving Force of Lateral Earth Pressure = (Effective Stress)(Ka)(A)(B)

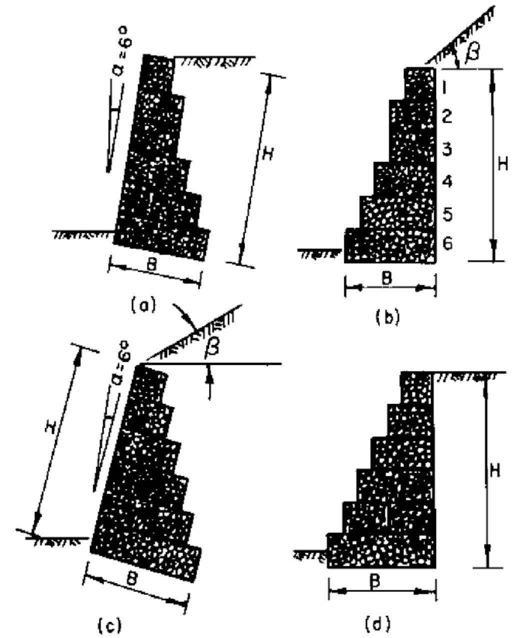
Structural Resisting Force of Strip = (Strip Axial Stress)(Strip Thickness)(Strip Width)

For Static Equilibrium,

Driving Force of Lateral Earth Pressure < Max. Resisting Force of Strip

Gabion Walls

Gabion Retaining Wall



Types – Common Gabion walls shown on accompanying diagrams are:

a) Battered face wall with horizontal backfill.
b) Stepped face wall with sloped backfill.
c) Battered face wall with sloped backfill.
d) Stepped face wall with horizontal backfill.

The choice of either battered or stepped faces rests with designer; stepped face recommended if wall is more than 10 feet high.

Gabion Fill – Hard, durable, clean stone 4 to 8 inches in size or other approved size.

Design: Design criteria for gravity walls apply. Wall section resisting overturning and sliding. To increase wall stability, recommended to tilt the wall at an angle of 6° (i.e. 1:10).

The angle of friction between the base of gabion wall and granular soil may be assumed 0.9 times the angle of internal friction of soil.

For retaining clay slopes, a system of gabion counterforts is recommended.

Compute active soil pressure behind the wall using Coulomb Wedge theory and design mass of the wall to balance the force exerted by that soil wedge. (Higher than active pressures may be used depending on compaction conditions and limitations on deformations.)

Maximum pressure at the base of gabion wall must be less than the anticipated bearing capacity of the soil under the wall.

When water quality is in doubt (pH below 6 or greater than 12) or where high concentration of organic acids may be present, use of PVC (polyvinylchloride) coated gabions is recommended.

FIGURE 34
Gabion Wall

Soil Nailing

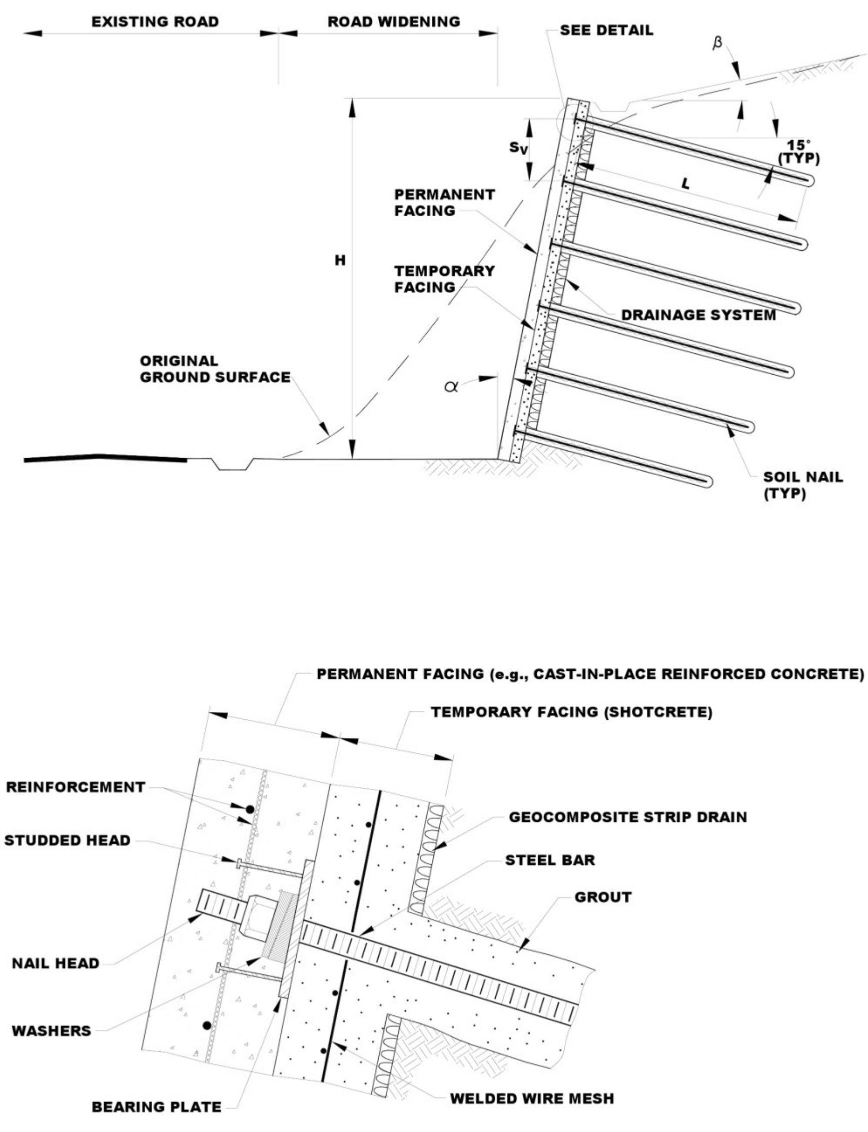
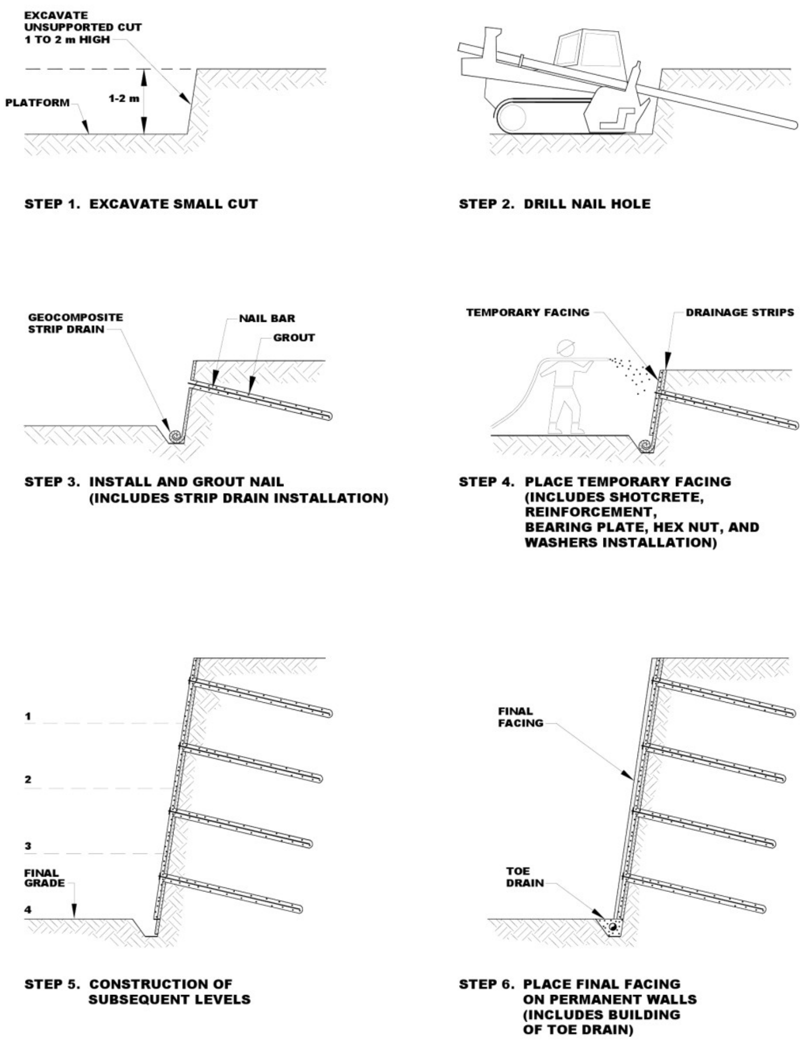


Figure 2.1: Typical Cross-Section of a Soil Nail Wall.



Modified after Porterfield et al. (1994).

Figure 2.2: Typical Soil Nail Wall Construction Sequence.

Questions