CE 468 GEOTECHNICAL DESIGN
RENIFORCED EARTH WALL DESIGN

A Reinforced earth wall is to be designed for the conditions shown in Fig. 1.

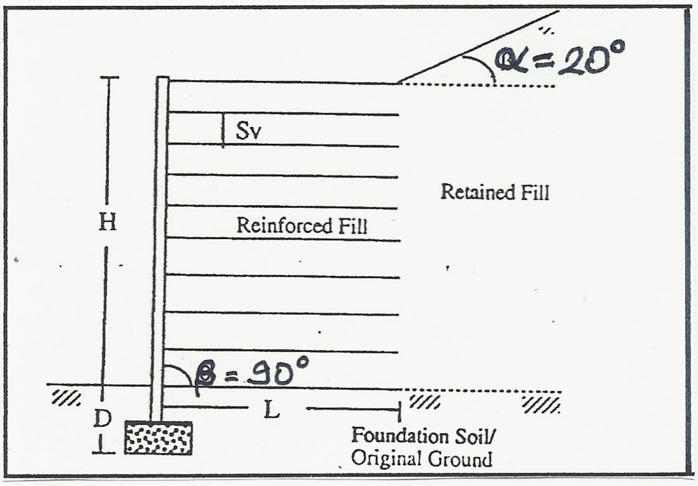


Figure 1. Reinforced soil system

H = 10m $\alpha = 20°$ $\beta = 90°$

Soil Properties:

Reinforced Fill:	$\gamma = 19.5\ kN/m^3$;	c = 0;	$\Phi' = 33°$
Retained Fill :	$\gamma = 18.0\ kN/m^3$;	c = 0;	$\Phi' = 30°$
Foundation Soil:	$\gamma = 20.0\ kN/m^3$;	c = 0;	$\Phi' = 39°$

a. Determine the required length of reinforcement to assure external stability. Minimum safety factors should be :

Sliding:	FS = 1.5
Overturning	FS = 2.0
Bearing Capacity	FS = 2.0

   The soil/wall friction angle is $\delta = \phi = 30°$, use Coulomb active earth pressure theory. Negative soil pressure is not allowed under the wall.

b. Check the overturning and the sliding stability for the earthquake condition using Mononobe-Okabe method. Note that the vertical earthquake aceeleration coefficient $K_v = 0$ and horizontal earthquake acceleration coefficient $K_h = 0.3$. Assume $\alpha = 0$ (horizontal backfill in EQ analysis).

c. A Tensar SR2 geogrid reinforcement system (extensible) is being considered for this wall. Relevant properties of Tensar SR2 reinforcement are :

Characteristics Strength : 30 kN/m
Minimum safety factor for tie break FS_{tb} = 1.7

Determine suitable spacing (S_v) for the Tensar reinforcement as a function of depth.

d. Determine factors of safety against pull-out (FS_{po}) as a function of depth. Assume that the pull-out resistance of Tensar for unit width is given by:

$$P = 2\, L_e\, \gamma\, z_i\, \alpha_b\, (\tan\varphi),$$

Where $\alpha_b = 0.9$; and $\varphi = 33^o$ (friction angle of reinforced fill)

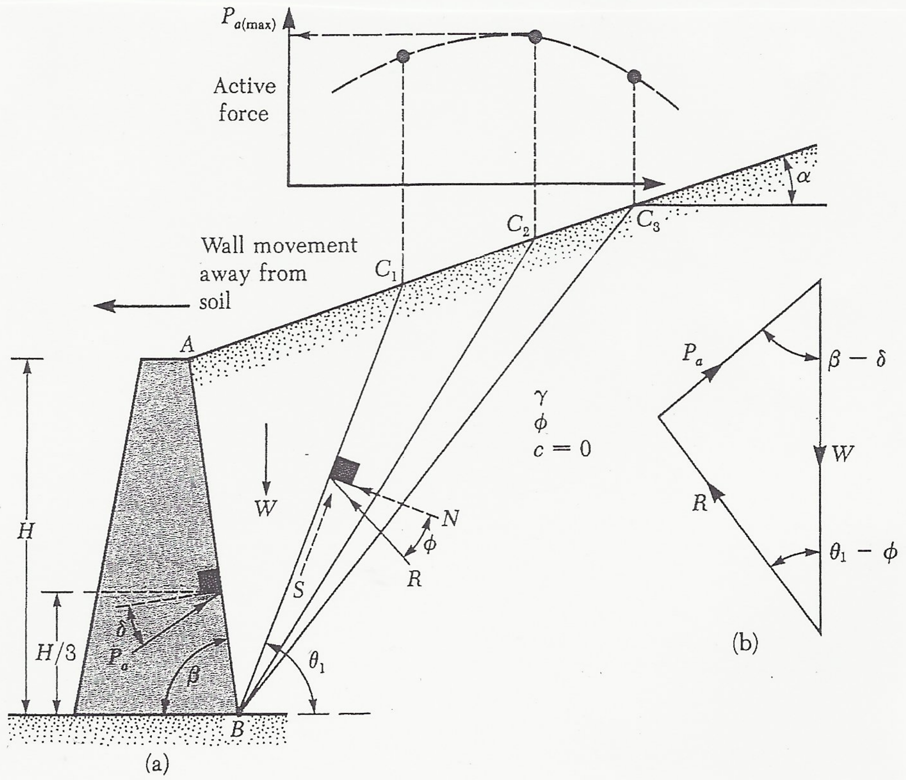
e. An alternate solution is the use of reinforced concrete facing units tied to teel strips (inextensible). The strip will be spaced at vertical and horizontal spacings of S_v and S_h respectively. Select the spacing of the tension strips to satisfy internal equilibrium requirements. Strip material properties are :

Yield stress f_y = 390 MPa,
Strip/soil friction angle $\delta = 25^o$.
FS_{tb} = 3.0 and FS_{po} = 3.0 is required

Hint: try width b = 100mm, thickness t = 5mm for the steel strips.

COULOMB'S ACTIVE EARTH PRESSURE

The Rankine active earth pressure calculations discussed in the preceding sections were based on the assumption that the wall is frictionless. In 1776, Coulomb proposed a theory to calculate the lateral earth pressure on a retaining wall with granular soil backfill. This theory takes wall friction into consideration.



▼ FIGURE 6.12 Coulomb's active pressure

For equilibrium purposes, a force triangle can be drawn, as shown in Figure 6.12b. Note that θ_1 is the angle that BC_1 makes with the horizontal. Because the magnitude of W as well as the directions of all three forces are known, the value of P_a can now be determined. Similarly, the active forces of other trial wedges, such as ABC_2, ABC_3, . . . can be determined. The maximum value of P_a thus determined is Coulomb's active force (see top part of Figure 6.12), which may be expressed as

$$\boxed{P_a = \tfrac{1}{2}K_a\gamma H^2} \tag{6.25}$$

where

$$\boxed{\begin{aligned} K_a &= \text{Coulomb's active earth pressure coefficient} \\ &= \frac{\sin^2(\beta+\phi)}{\sin^2\beta\,\sin(\beta-\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\alpha)}{\sin(\beta-\delta)\sin(\alpha+\beta)}}\right]^2} \end{aligned}} \tag{6.26}$$

and H = height of the wall

ACTIVE EARTH PRESSURE FOR EARTHQUAKE CONDITIONS

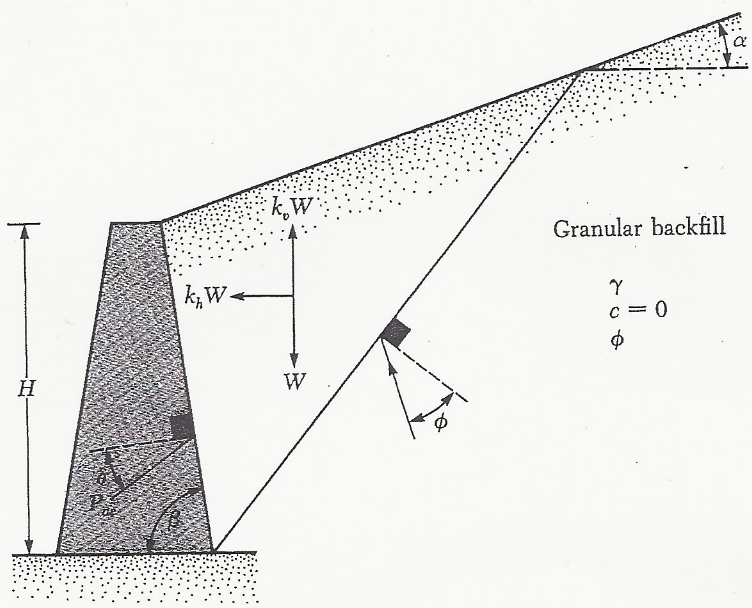
Coulomb's active earth pressure theory (see Section 6.5) can be extended to take into account the forces caused by an earthquake. Figure 6.14 shows a condition of active pressure with a granular backfill ($c = 0$). Note that the forces acting on the soil failure wedge in Figure 6.14 are essentially the same as those shown in Figure 6.12a, with the addition of $k_h W$ and $k_v W$ in the horizontal and vertical directions, respectively; k_h and k_v may be defined as

$$k_h = \frac{\text{horizontal earthquake acceleration component}}{\text{acceleration due to gravity, } g} \tag{6.29}$$

$$k_v = \frac{\text{vertical earthquake acceleration component}}{\text{acceleration due to gravity, } g} \tag{6.30}$$

As in Section 6.5, the relation for the active force per unit length of the wall (P_{ae}) can be determined as

$$\boxed{P_{ae} = \tfrac{1}{2}\gamma H^2(1 - k_v)K_{ae}} \tag{6.31}$$



▼ **FIGURE 6.14** Derivation of Eq. (6.31)

where

$$\boxed{\begin{aligned} K_{ae} &= \text{active earth pressure coefficient} \\ &= \frac{\sin^2(\phi + \beta - \theta')}{\cos\theta' \sin^2\beta \sin(\beta - \theta' - \delta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \theta' - \alpha)}{\sin(\beta - \delta - \theta')\sin(\alpha + \beta)}}\right]^2} \end{aligned}} \tag{6.32}$$

$$\theta' = \tan^{-1}\left[\frac{k_h}{1 - k_v}\right] \tag{6.33}$$

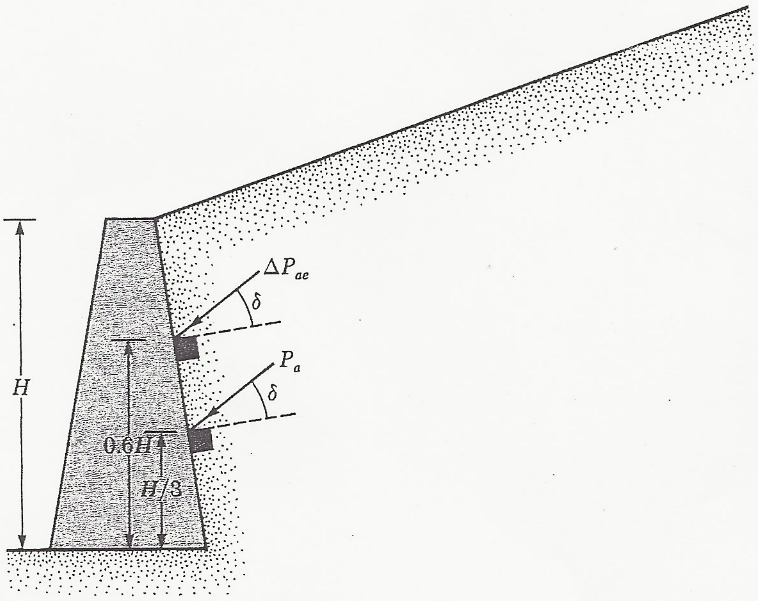
Equation (6.31) is usually referred to as the *Mononobe–Okabe* solution. Unlike the case shown in Figure 6.12a, the resultant earth pressure in this situation, as calculated by Eq. (6.31) *does not act* at a distance of $H/3$ from the bottom of the wall. The following procedure may be used to obtain the location of the resultant force P_{ae}:

1. Calculate P_{ae} by using Eq. (6.31)
2. Calculate P_a by using Eq. (6.25)
3. Calculate

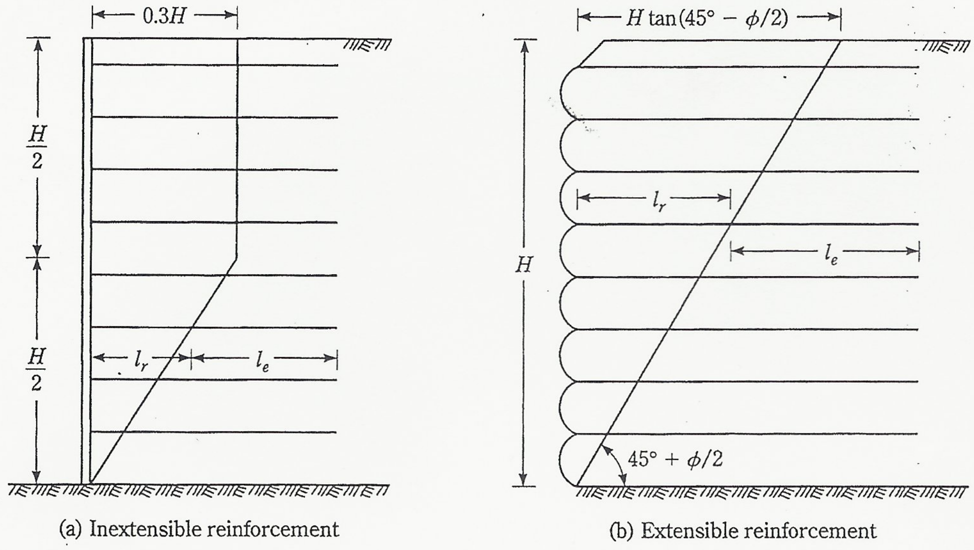
$$\Delta P_{ae} = P_{ae} - P_a \tag{6.34}$$

4. Assume that P_a acts at a distance of $H/3$ from the bottom of the wall (Figure 6.16).
5. Assume that ΔP_{ae} acts at a distance of $0.6H$ from the bottom of the wall (Figure 6.16).
6. Calculate the location of the resultant as

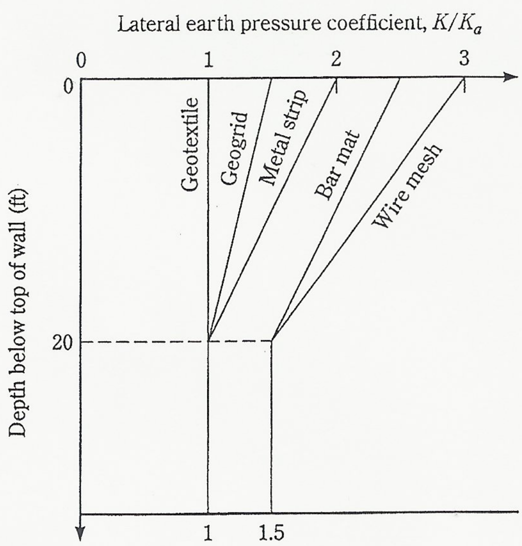
$$\bar{z} = \frac{(0.6H)(\Delta P_{ae}) + \left(\frac{H}{3}\right)(P_a)}{P_{ae}} \tag{6.35}$$



▼ FIGURE 6.16 Determining the line of action of P_{ae}

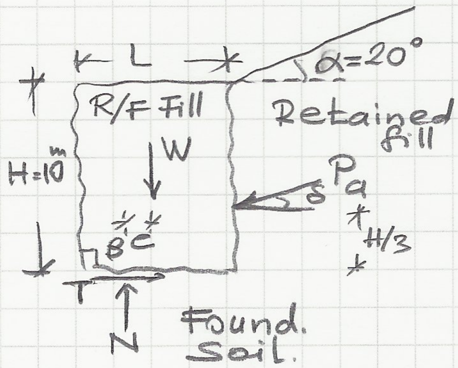


▼ **FIGURE 7.40** Location of potential failure surface (after Transportation Re-



▼ **FIGURE 7.39** Recommended design values for lateral earth pressure coefficient, K (after Transportation Research Board, 1995)

ASSGN: 3 REINFORCED EARTH WALL DESIGN GUIDELINES:



Criteria for external stability:

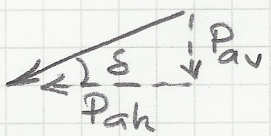
Sliding FS = 1.5
Overturning FS = 2.0
Bearing Cap. FS = 2.0.

Assumptions:
i. No surcharge on the wall
ii. " Hydrostatic pressure
iii. No passive resistance in front side
iv. ignore effective cohesion of the soils.
v. No negative soil pressure

$P_a = \frac{1}{2} K_a \gamma H^2$ (Use Retained soil parameters)

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

$\delta = 30°$
$\alpha = 20°$
$\beta = 90°$
$\phi = 30°$



$P_{av} = P_a \cos\delta$

$P_{ah} = P_a \sin\delta$

$N = W + P_{av}$

External Stability:

i. No tension criteria

assume $e = \frac{L}{6}$ triangular base pressure (No tension)

Take moment wrt center line $M_{center} = 0$

$M_c = Ne + P_{av}\frac{L}{2} - P_{ah}\frac{H}{3} = 0 \quad e = \frac{L}{6} \quad W = \gamma H L$

Solve for L.

ii. Overturning use L found in (i)

$FS = \dfrac{\Sigma M_R}{\Sigma M_D} \geq 2.$

iii. Sliding:

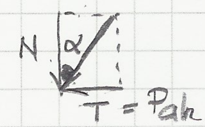
for L from (i) $FS = \dfrac{(W + P_{av})\tan\phi'}{P_{ah}} \geq$ reinforced fill

min L required $FS = 1.5 = \dfrac{(\gamma H L + P_{av})\tan\phi'}{P_{ah}}$

Solve for L, and compare with L from (i)

iv. Bearing Capacity Check

$q_{ult} = \frac{1}{2}\gamma (B')(N_\gamma i_\gamma) \quad i_\gamma = \left(1 - \frac{\alpha}{\phi'}\right)^2$



use $\phi'_{found} = 39°$ $\quad B' = L - 2e$

$\alpha = \text{arch tan}\ \frac{T}{N}$

Use L from (i)

find q_{ult}, find $q_{max} = \frac{N}{L}\left(1 + \frac{6e}{L}\right) = \frac{2N}{L}$

Compare $\frac{q_{ult}}{q_{max}} < 2 \rightarrow$ Not realistic

$q_{av} = \frac{N}{L-2e}$ more realistic (equivalent area method).

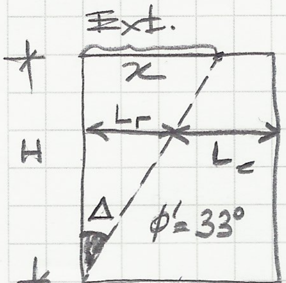
$q_{ult}/q_{av} \leq 2$

CONCLUDE L for EXTERNAL STABILITY

INTERNAL STABILITY:

EXTENSIBLE REINFORCEMENT

Available strength $= \frac{30\,kN/m}{1.7} \cong 17.8\,kN/m \cong$



$\Delta = \left(45° - \frac{\phi'}{2}\right)$

$\tan\Delta = \frac{x}{H} \quad x = H\tan\Delta = H \times \tan\left(45 - \frac{\phi'}{2}\right)$

$L = L_{min} + H\tan\left(45 - \frac{33°}{2}\right) = 0.5 + 5.4\,m \cong 5.9\,m$

Say 6 m

. Tie break Check:

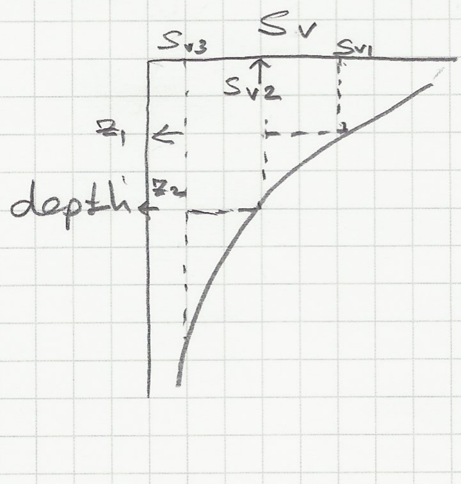
$FS = \frac{T_{available}}{S_v\, S_h\, \sigma_h} = 1.0$ since partial FS are used.

$T_{available} \rightarrow 17.8\,kN/m$, $S_h \rightarrow 1.0$

$\sigma_h = \gamma z K_a \quad K_a \rightarrow \phi' = 33°$ of R/F fill.

Depth	σ_h	$T_{av}/\sigma_h = S_v$

$S_v = \frac{T_{av}}{\sigma_h}$



$0 - z_1$	S_{v1}
$z_1 - z_2$	S_{v2}

Pull-out check:

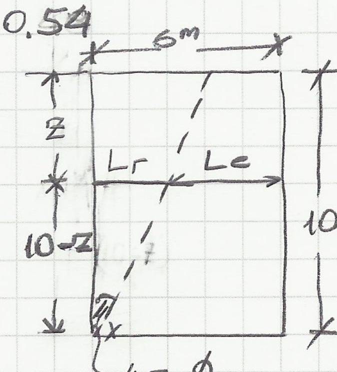
$L_e = L - L_r = L - (H-Z)\tan(45 - \frac{\phi}{2})$

Pull-out resistance $P = 2\alpha_b L_e \gamma z_i \tan\delta$

$\alpha_b = 0.9 \quad \tan(45 - \frac{\phi'}{2}) = 0.54$

$\sigma_h = \gamma z \times K_a$

$L_e = 6 - (10 - z_i) \times 0.54$



$45 - \frac{\phi}{2}$

$L_r = (10-Z)\tan(45+\frac{\phi}{2})$

$L_e = 6 - L_r$

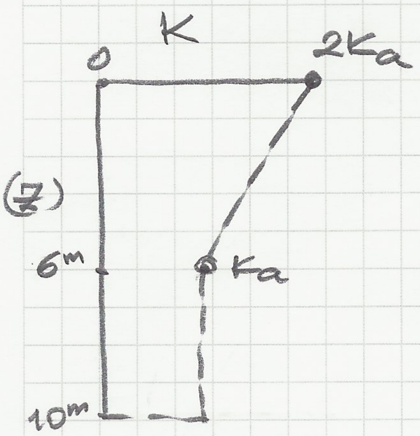
$$FS = \frac{2 \times \alpha_b L_e \; \gamma z_i \tan\delta}{\sigma_h \times S_v \times (S_h = 1.0)}$$

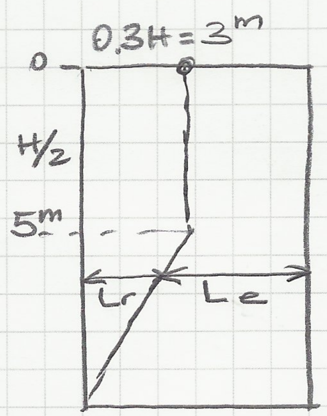
From tie-break

z_i	σ_h	S_v	$\sigma_h S_v$	P	FS

check for FS. for each layer.

IN-EXTENSIBLE REINFORCEMENT

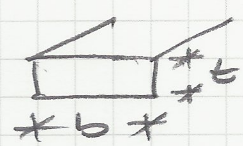




$Z < 5^m \quad L_e = L - 0.3H = L - 3m$

$Z > 5m \quad L_e = L - \frac{3}{5}(H - Z)$

i. Tie break (FIND S_h) $\quad FS = \dfrac{f_y b t}{\sigma_h S_v S_h} > 3$



ii. Pull-out (Find S_h) $\quad FS = \dfrac{2b(\gamma z_i) L_e \tan\delta}{\sigma_h S_v S_h} > 3$

Assume Sv

Z	γz	K	σ_h	S_h (tie break)	S_h pull-out