Lecture 31

Reinforced Soil Retaining Walls-Design and Construction

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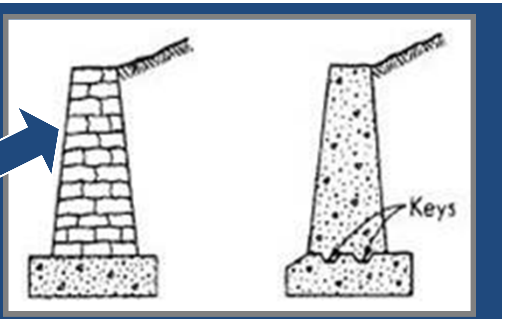
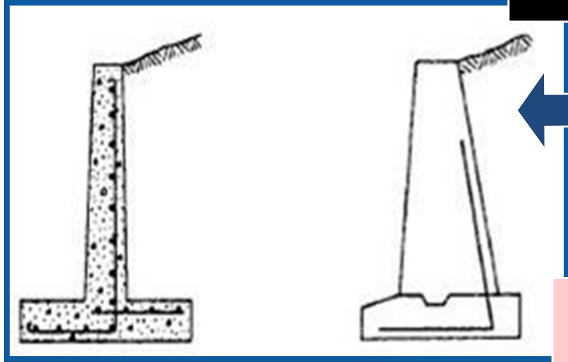
Evolution of RS-RW

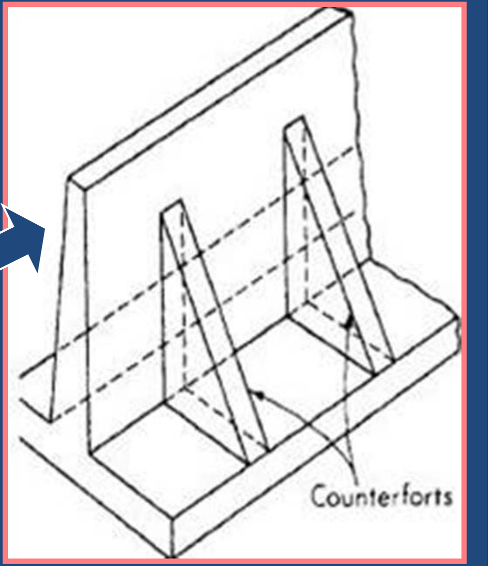


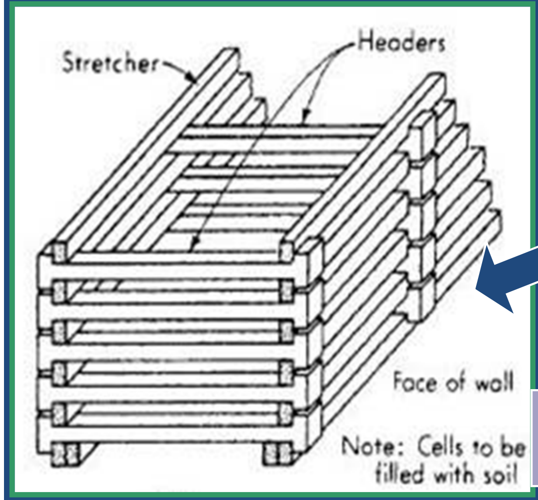
Classical gravity retaining walls



Reinforced concrete types

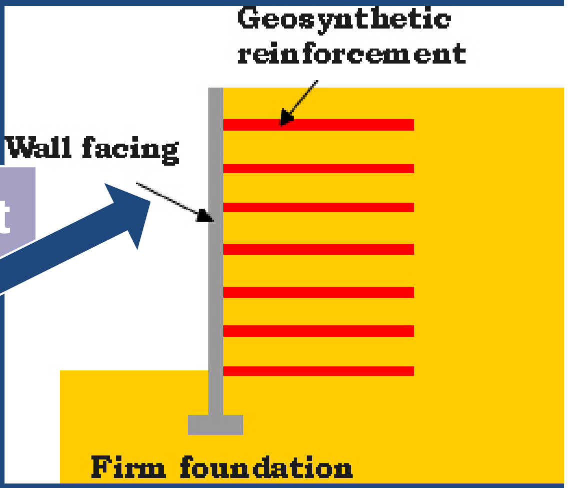
Buttressed and counterfort walls



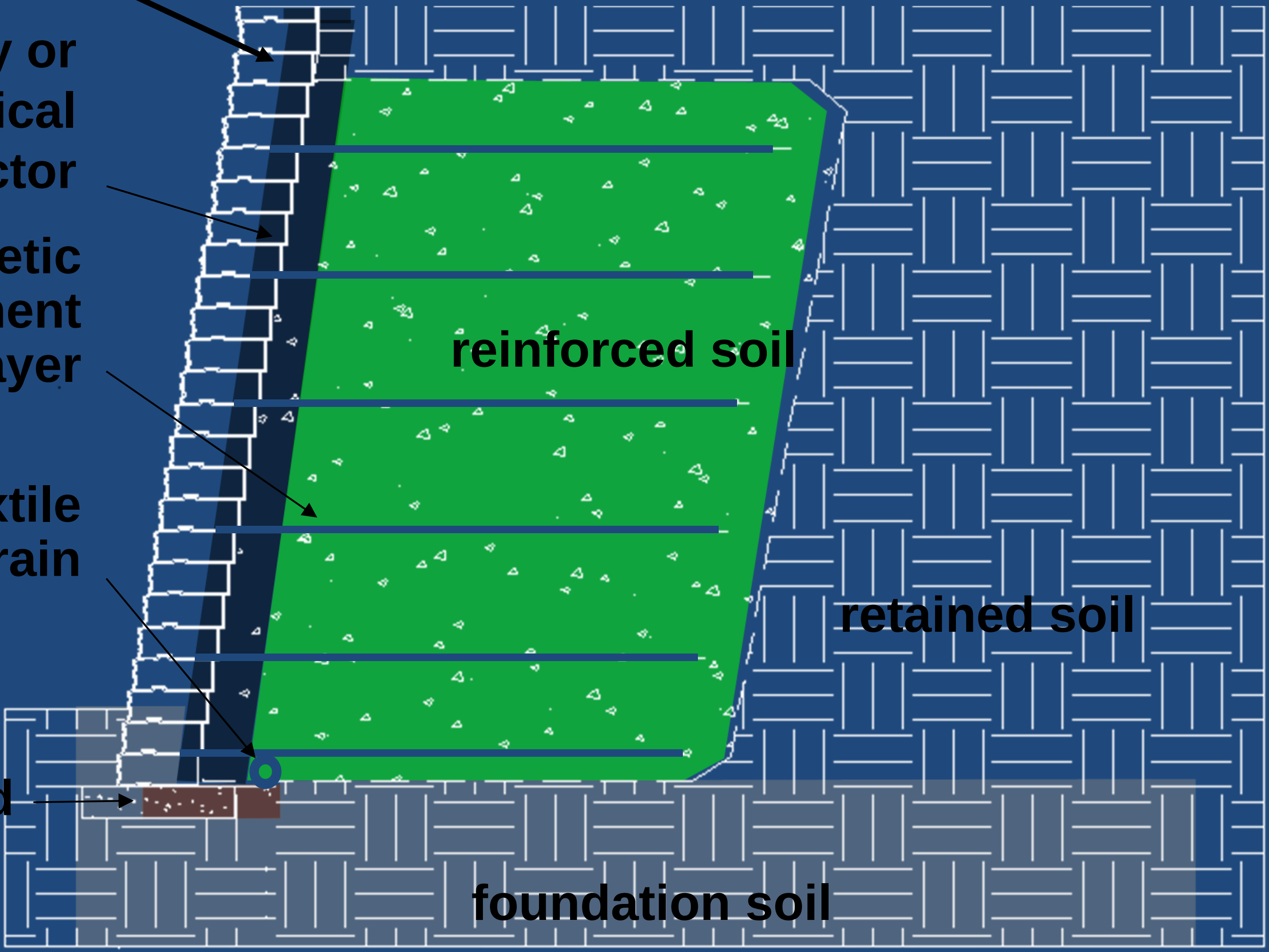


Prefabricated and compartmentalized gravity walls (cribs and Bins, gabions)

MSE with metal reinforcement



MSE with Geosynthetic reinforcement



segmental facing units

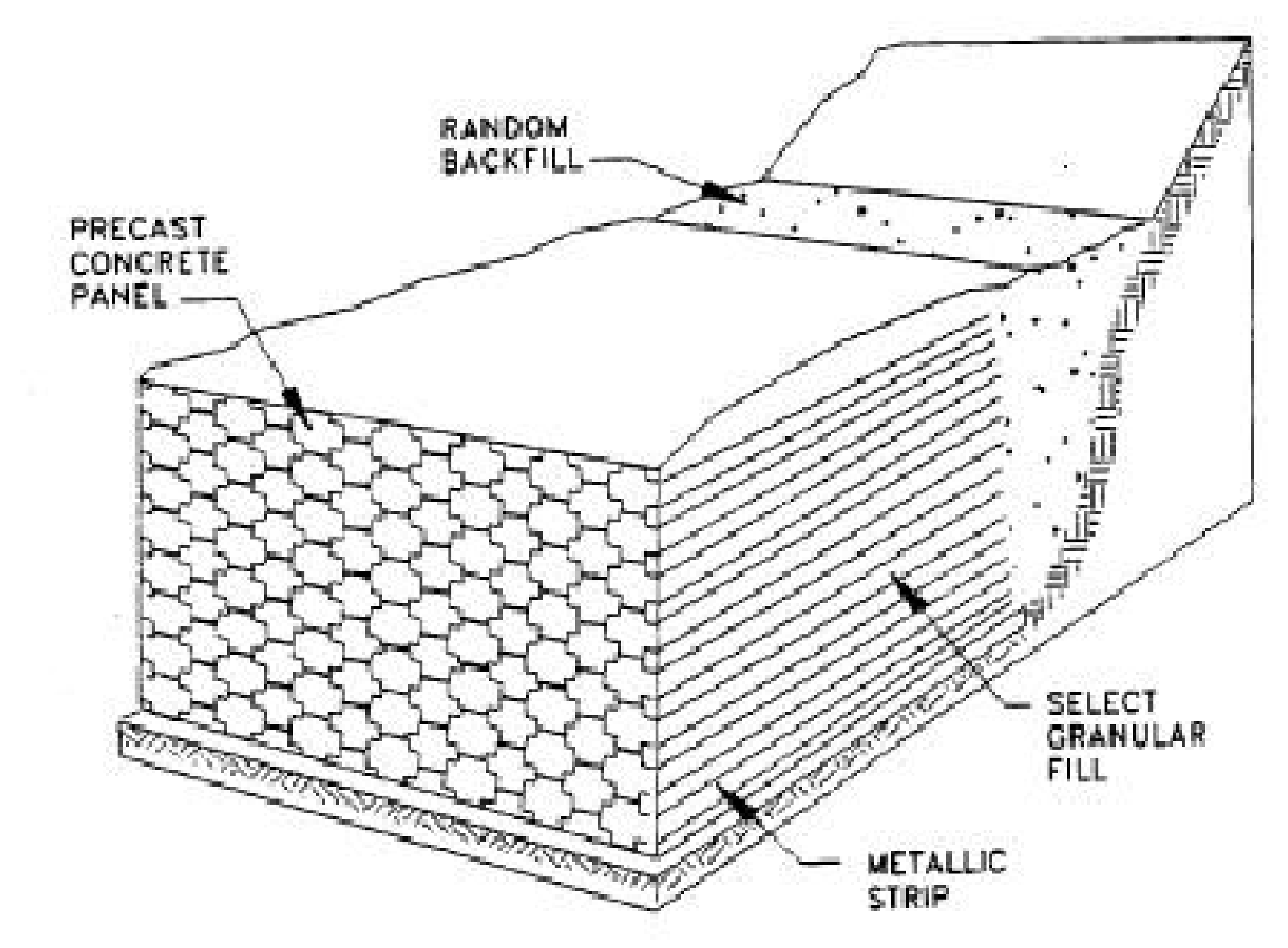
shear key or mechanical connector

geosynthetic reinforcement layer

geotextile wrapped drain

granular levelling pad

Component parts of Reinforced Earth wall (Vidal's Reinforced Earth system)



- Steel strips
- Geotextile materials
  - Conventional geotextiles
    - nonwovens, woven, knitted and stretch bonded textiles
  - special geotextiles
    - geosynthetics in two forms geo-grids and geo-composites

The principal requirements of reinforcement

- strength and stability (low tendency to creep),
- durability, ease of handling,
- high coefficient of friction and/or adherence with the soil,
- low cost and
- ready availability.

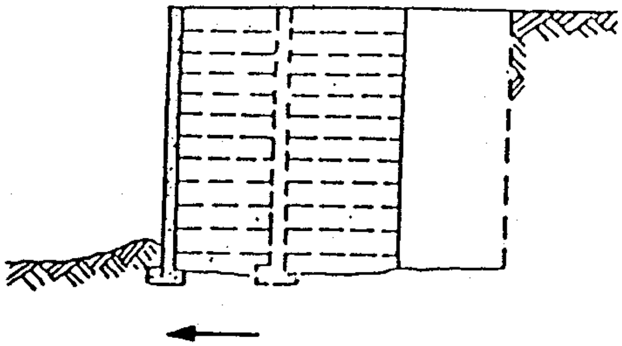
- geosynthetic acts as reinforcement and the most important properties are
  - tensile strength,
  - tensile modulus and
  - interface shear strength

General

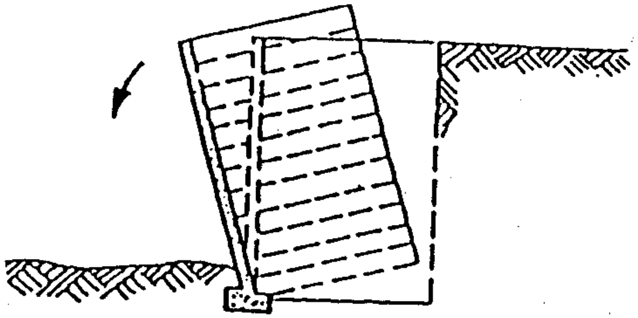
- Limit equilibrium approach
- Two primary forms of stability must be investigated:
  - External stability
  - Internal stability
- Critical state soil properties (ϕ'_{cv} and c'_{cv})
- Design strength of the grids

$$\mathbf{MILTS = P_c / (f_m \times f_e \times f_d \times f_j)}$$

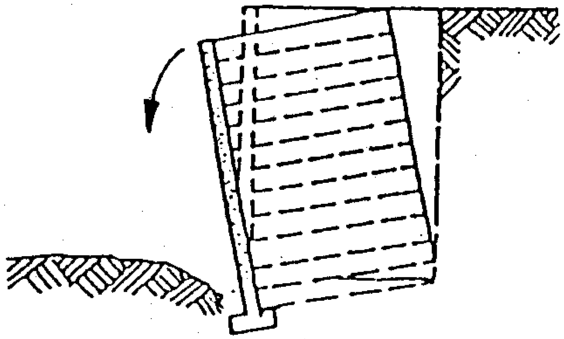
External stability



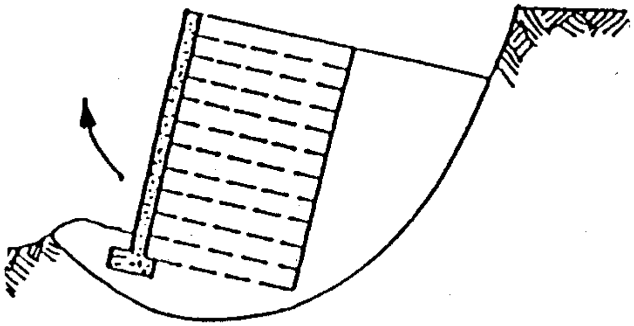
a) sliding



b) overturning

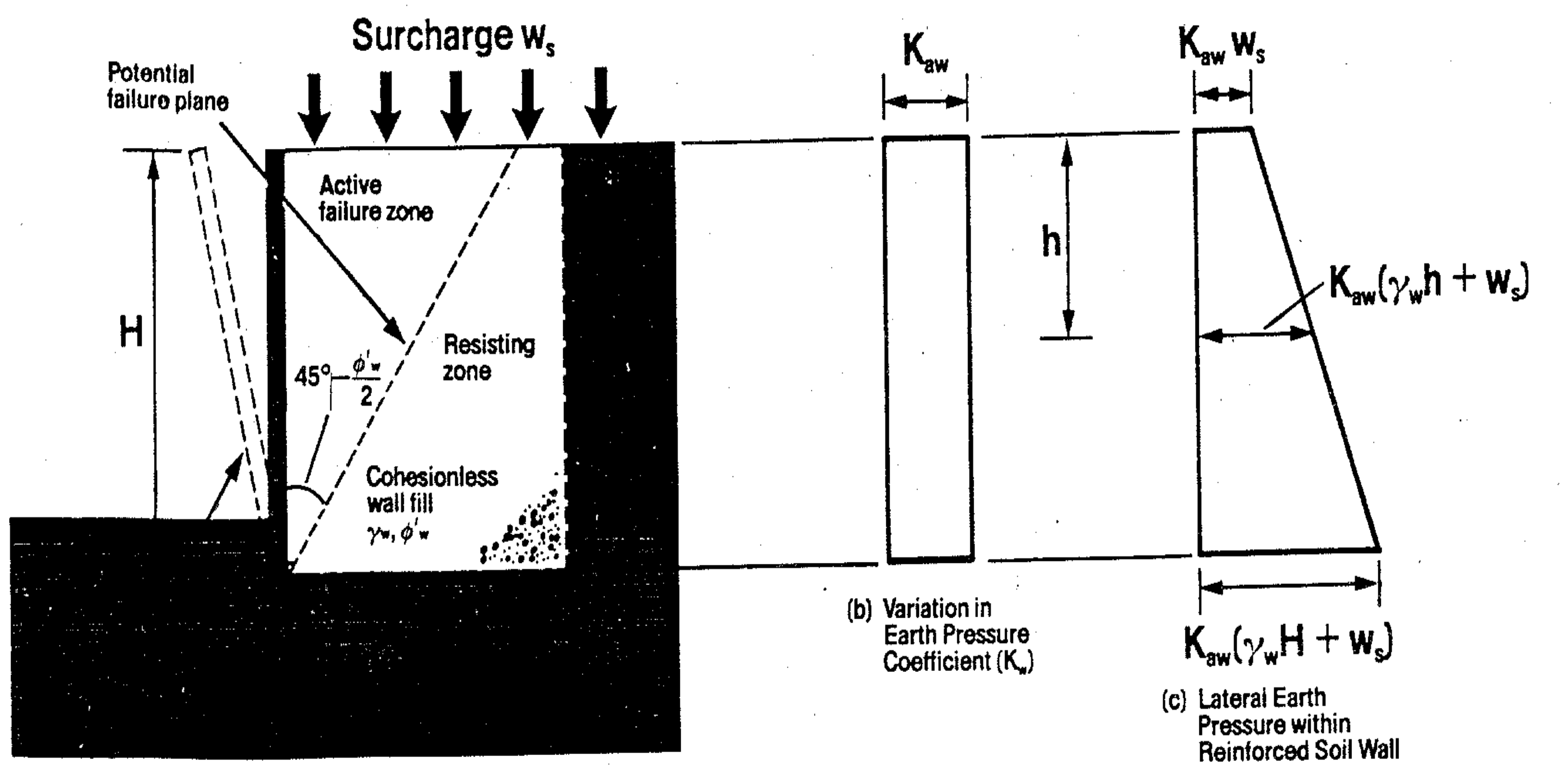


c) bearing capacity



d) deep stability

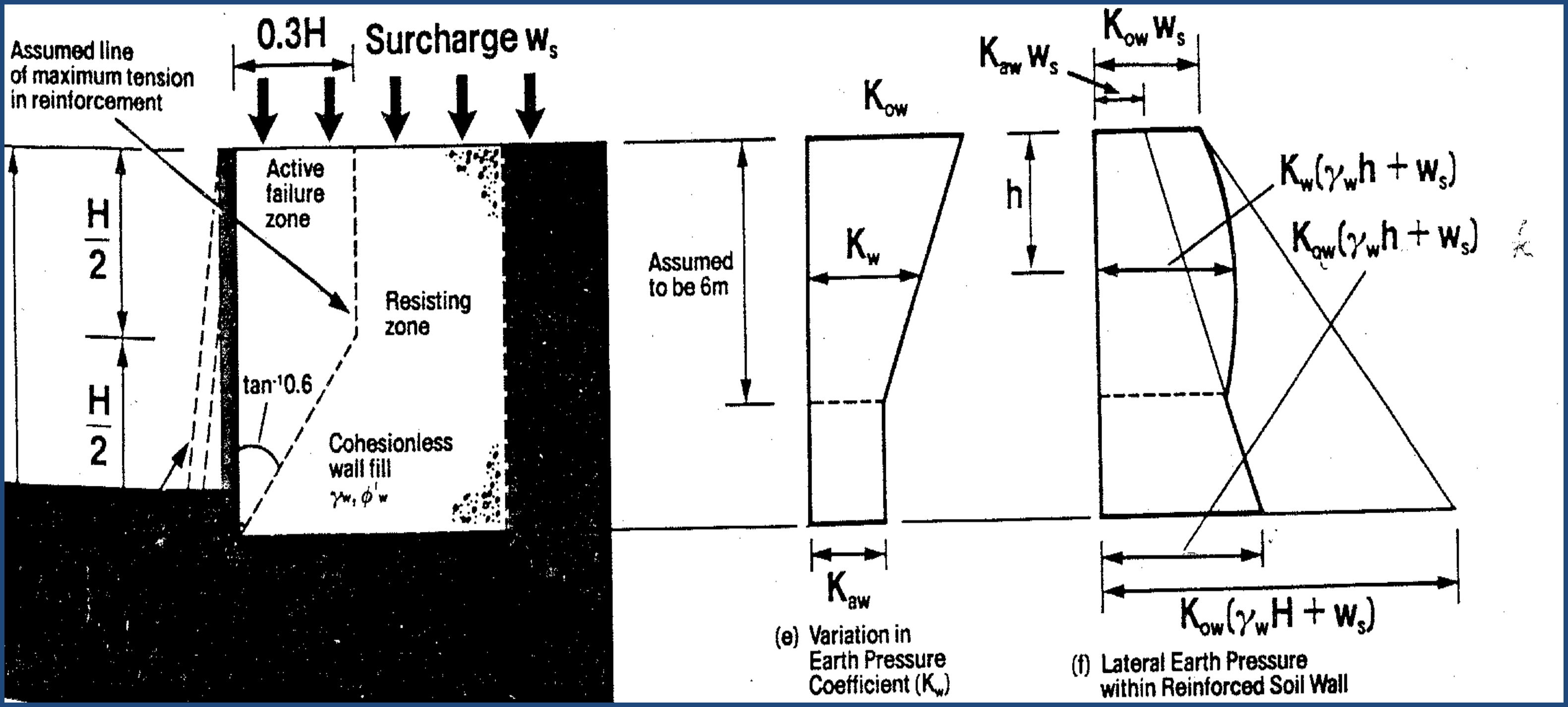
Tie back wedge method



(b) Variation in Earth Pressure Coefficient (K_w)

(c) Lateral Earth Pressure within Reinforced Soil Wall

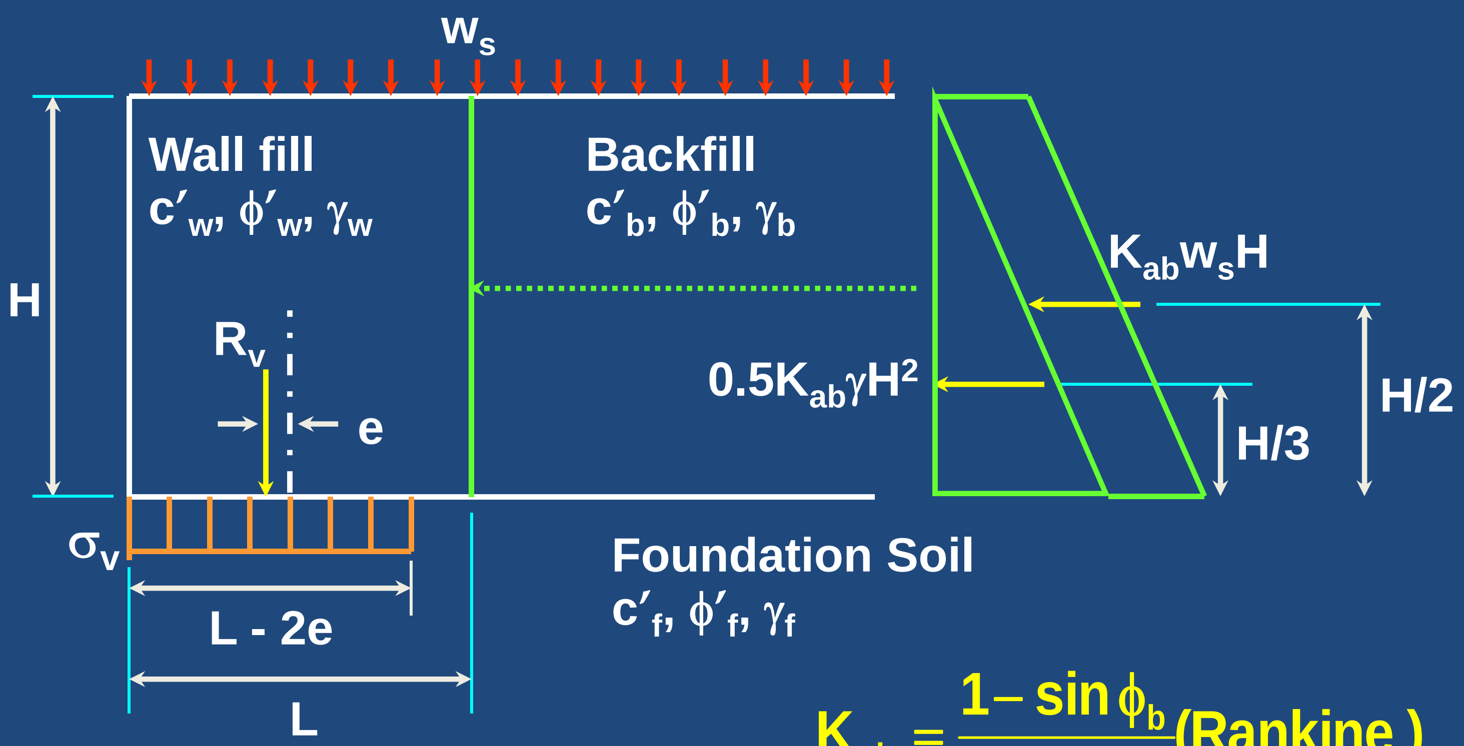
Coherent gravity method



(e) Variation in Earth Pressure Coefficient (K_w)

(f) Lateral Earth Pressure within Reinforced Soil Wall

External Forces



$$K_{ab} = \frac{1 - \sin\phi_b}{1 + \sin\phi_b} (\text{Rankine})$$

External Sliding

- Factor of Safety for sliding is given by:

$$\text{Fos} = \frac{\text{Resisting force}}{\text{Sliding force}} = \frac{2\mu\left(\gamma_w H + w_s\right)}{K_{ab}\left(\gamma_b H + 2w_s\right)\left(\frac{H}{L}\right)}$$

where μ is the coefficient of friction on the base of the reinforced soil block (= $\alpha\tan\phi'_w$ or $\alpha\tan\phi'_f$)

Target factor of safety is usually 2.0

Overturning Failure

- Factor of safety against overturning is given by:

$$\mathrm{Fos} = \frac{\text{Restoring moment}}{\text{Overturning moment}} = \frac{3\left(\gamma_w H + w_s\right)}{K_{ab}\left(\gamma_b H + 3w_s\right)\left(\frac{H}{L}\right)^2}$$

- Target factor of safety is usually 2.0
- Seldom a critical failure criterion

Bearing Capacity

- Assume a Meyerhof pressure distribution at the base of the structure
- Usually, an allowable bearing pressure of half the ultimate pressure is satisfactory providing settlements can be tolerated (i.e. factor of safety = 2.0)
- The ground bearing pressure is given by

$$\sigma_v = \frac{\left(\gamma_w H + w_s\right)}{1 - \frac{K_{ab}\left(\gamma_b H + 3w_s\right)}{3\left(\gamma_w H + w_s\right)}\left(\frac{H}{L}\right)^2}$$

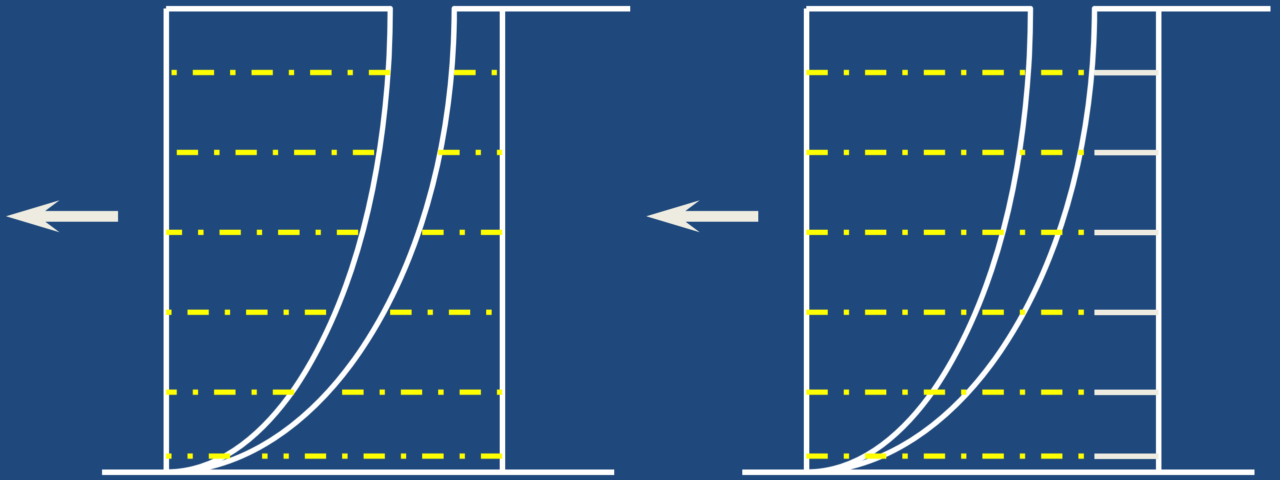
- Allowable bearing pressure given in codes.

Slip Failure

- All potential slip surfaces should be investigated
- Target factor of safety of 1.5 usually adopted for rotational slip type failures

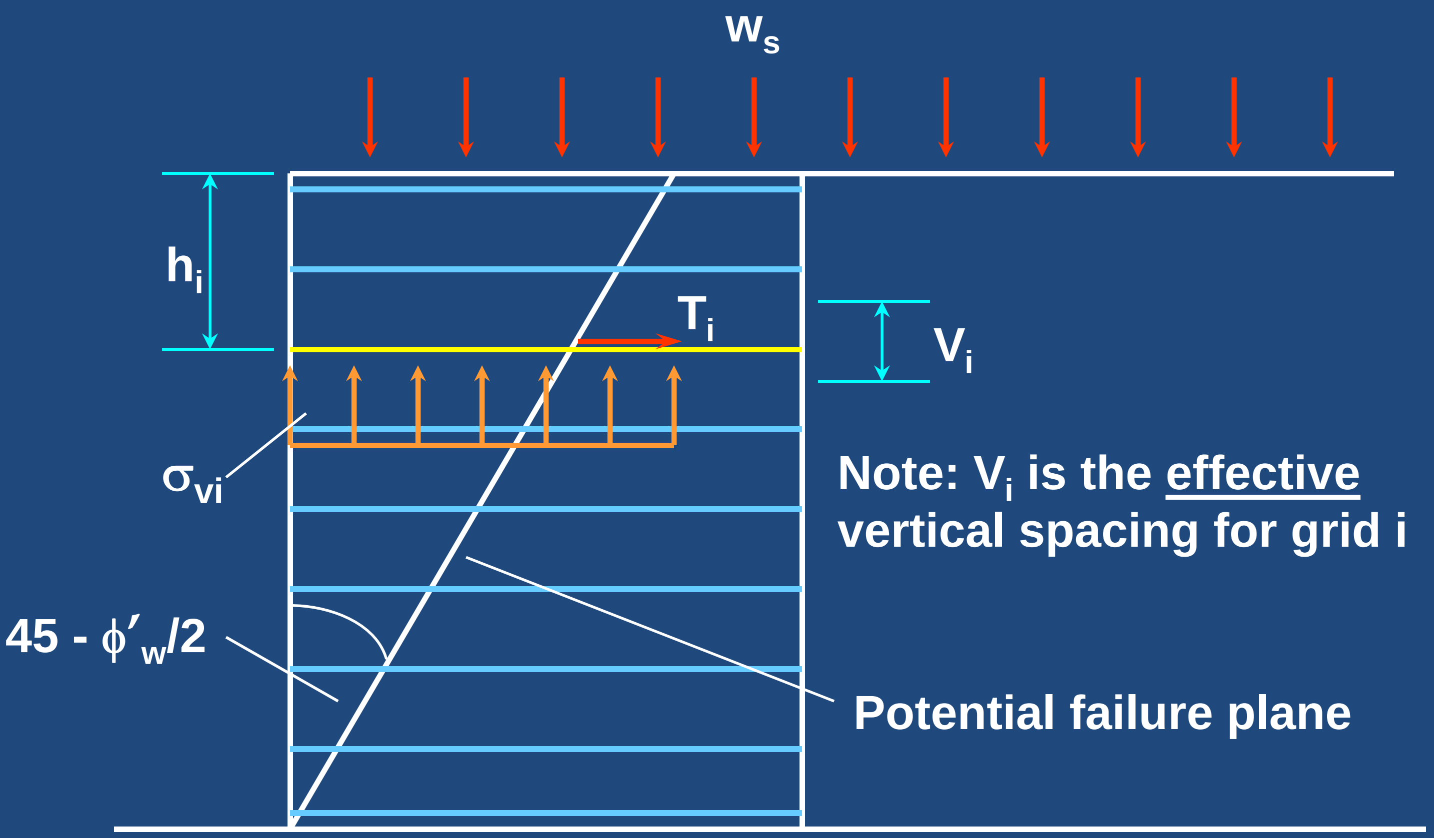
Internal Stability

- Two main failure mechanisms need to be investigated:
  - tension failure
  - pull-out failure



Tension Failure **Pull-out Failure**

Tension Failure (1)



Note: V_i is the **effective** vertical spacing for grid i

Potential failure plane

T_i has four components:

- Weight of fill
- Active pressure from behind RSB
- Surcharge on top of RSB
- c′ within RSB (restoring force)

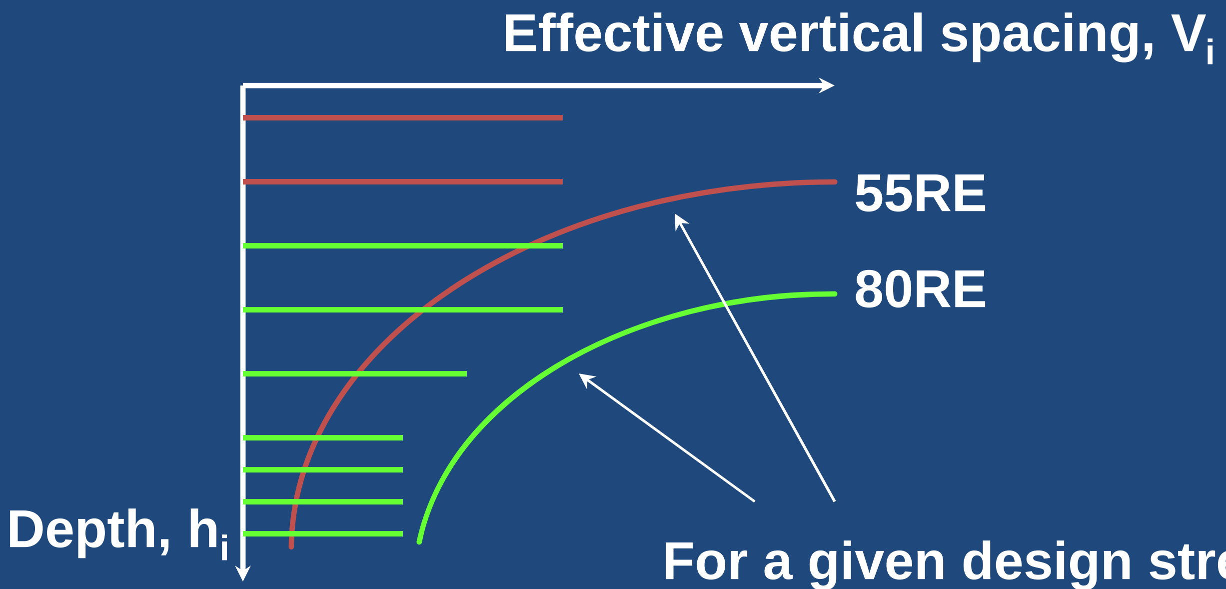
Tension Failure (2)

- Grids carry tension as a result of the self weight of the fill and the surcharge acting on top of the reinforced soil block

$$T_i = \left[K_{aw} \left[\frac{\left(\gamma_w h_i + w_s\right)}{1 - \frac{K_{ab}\left(\gamma_b h_i + 3w_s\right)}{3\left(\gamma_w h_i + w_s\right)} \left(\frac{h_i}{L}\right)^2} \right] - 2c'_w \sqrt{K_{aw}} \right] V_i$$

Tension Failure (3)

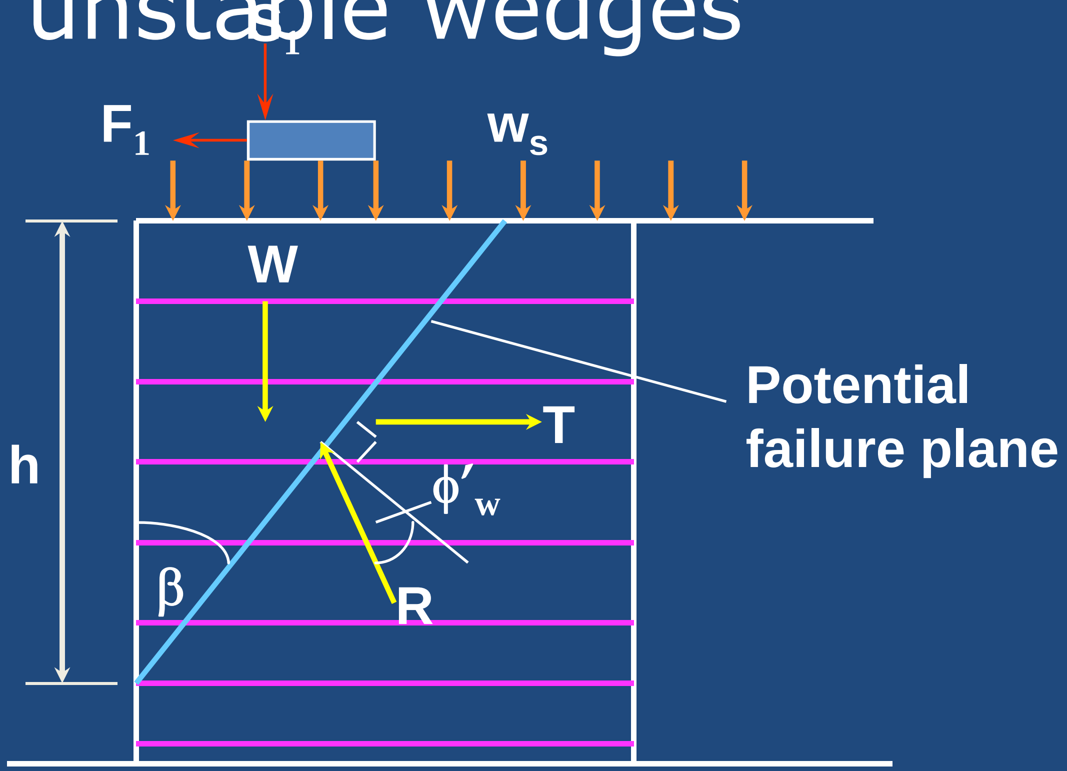
- A spacing curve approach is used



For a given design strength, the maximum vertical grid spacing $V_{i(max)}$ can be calculated for a range of depths

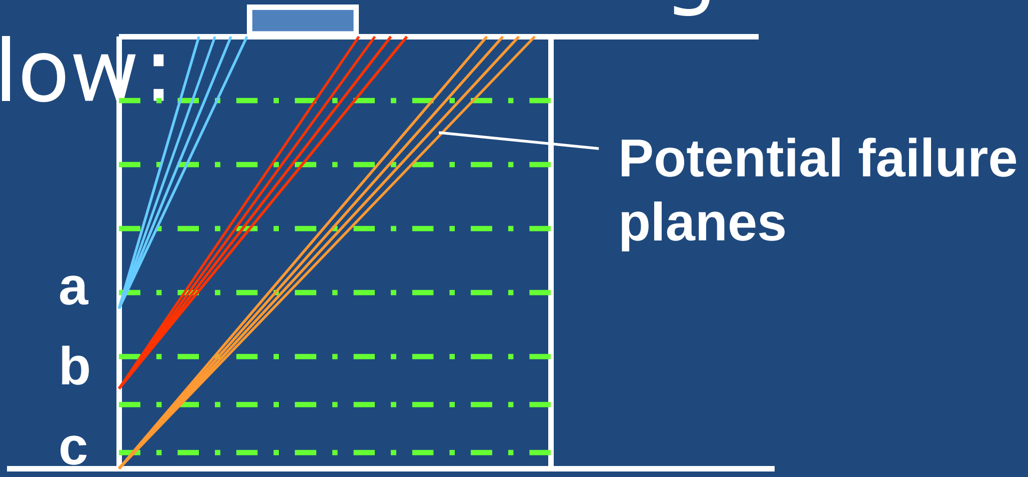
Wedge/Pull-out Failure (1)

- Consider the possibility of failure planes passing through the wall and forming unstable wedges



Wedge/Pull-out Failure (2)

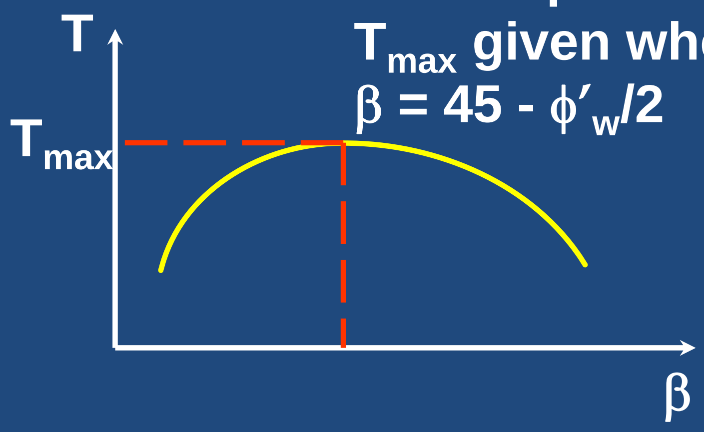
- Assumptions:
  - each wedge behaves as a rigid body
  - friction between the facing and the fill is ignored
- Investigate series of wedges as shown below:

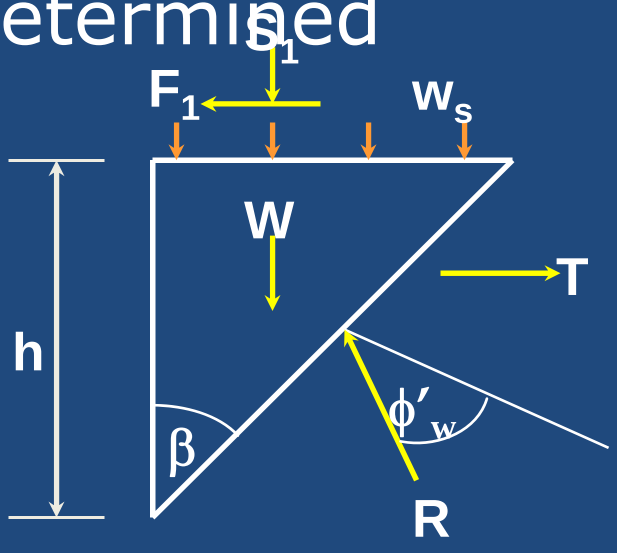


Wedge/Pull-out Failure (3)

- Mobilising force
  - At any level, by changing β, a value for T_{max} can be determined

For simple cases, T_{max} given when $\beta = 45 - \phi'_w/2$

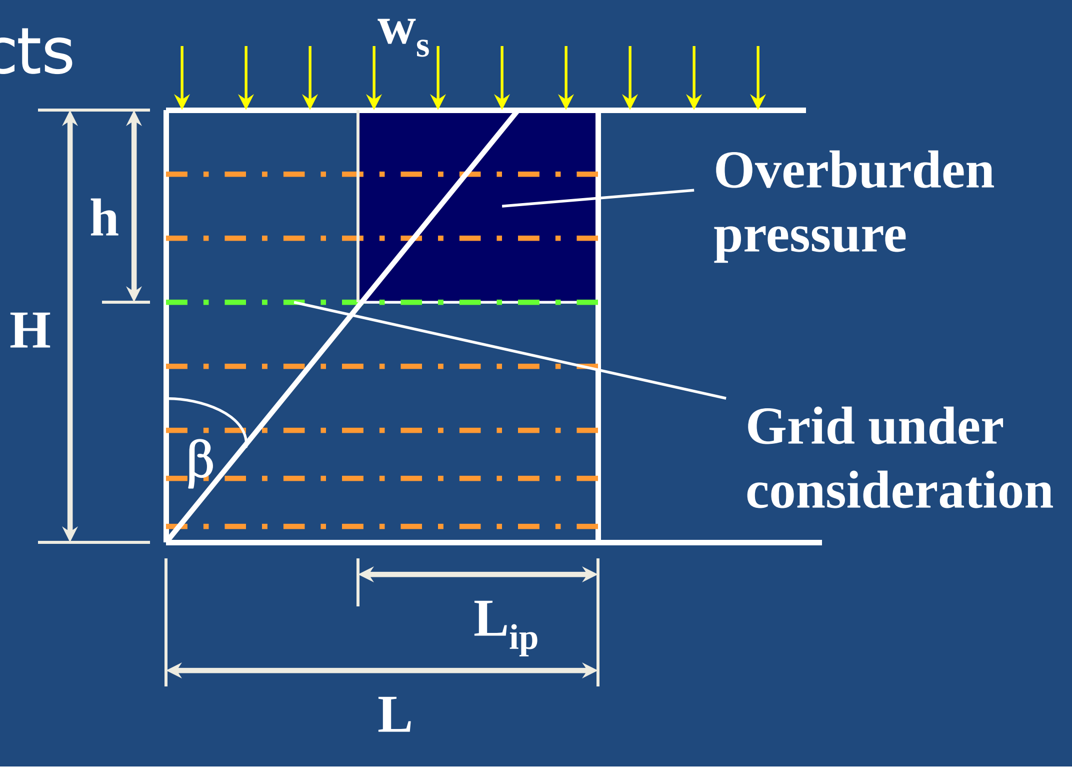




$$T = \frac{h \tan\beta \left(\gamma_w h + 2w_s\right)}{2\tan\left(\phi'_w + \beta\right)}$$

Wedge/Pull-out Failure (4)

- Resisting force
  - This is normally the design strength of the grid
  - Account must be taken of the anchorage effects



Wedge/Pull-out Failure (5)

- Resisting force (continued)
  - Anchorage force, T_{ai} available in a grid is given by:

$$T_{ai} = \frac{2L_{ip}\alpha_p \tan\phi'_w \left(\gamma_w h_i + w_s\right)}{\text{factor of safety}}$$

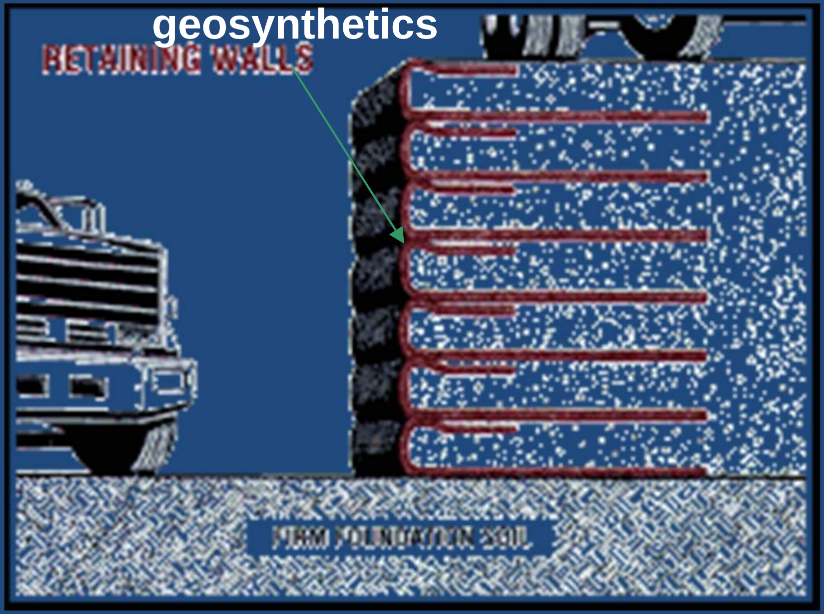
For each layer of reinforcement cut by the wedge, the lower of the design strength, T_{des} or T_{ai} is used to determine the contribution from the reinforcement

- Compare the mobilising force with the resisting force
  i.e. $\Sigma\,(T_{ai}\ \text{or}\ T_{des}) \geq T$

Geosynthetic Reinforced Soil Walls

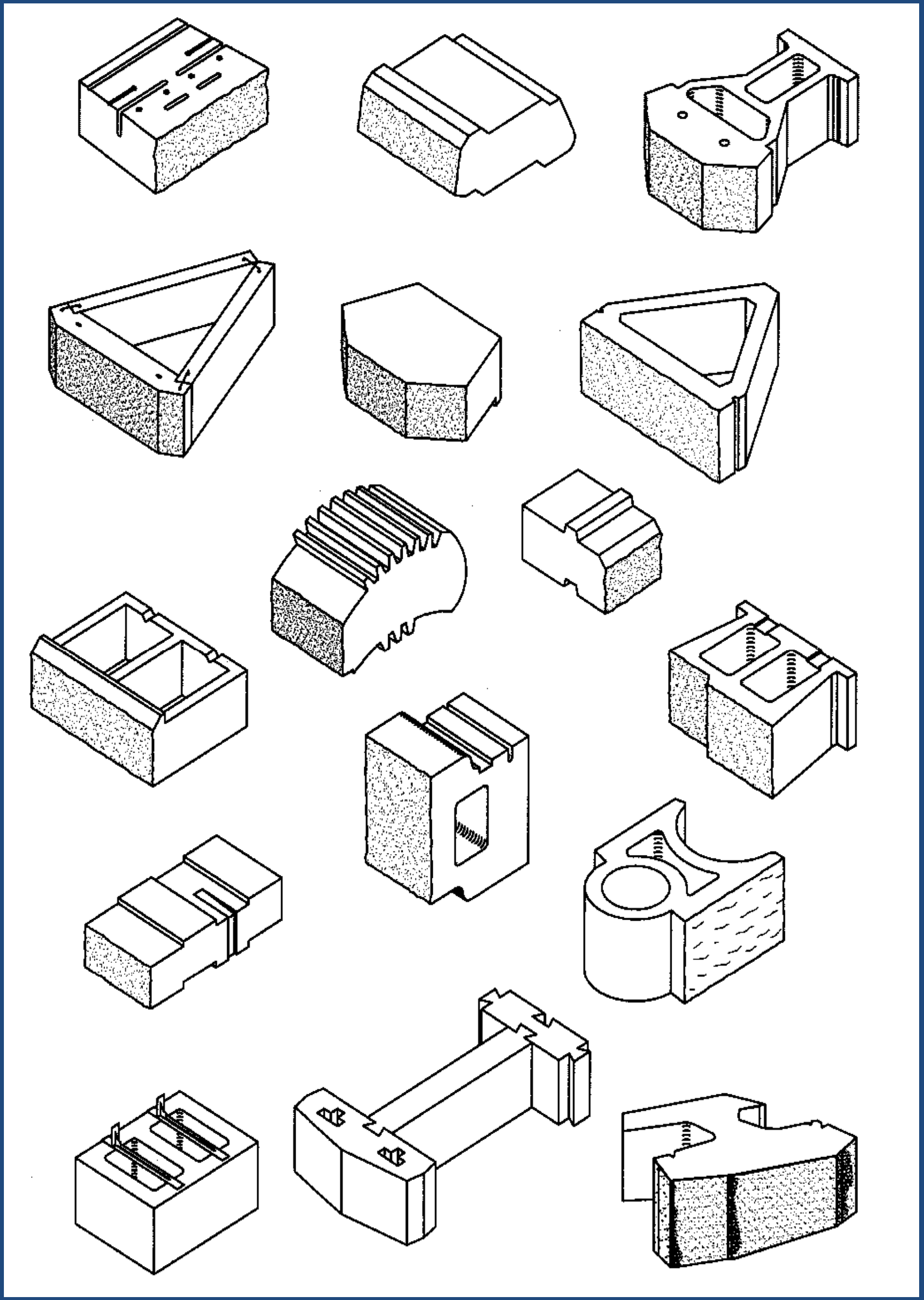
GRS walls are increasingly becoming popular.

Concrete facing



Wrapped geotextile facing

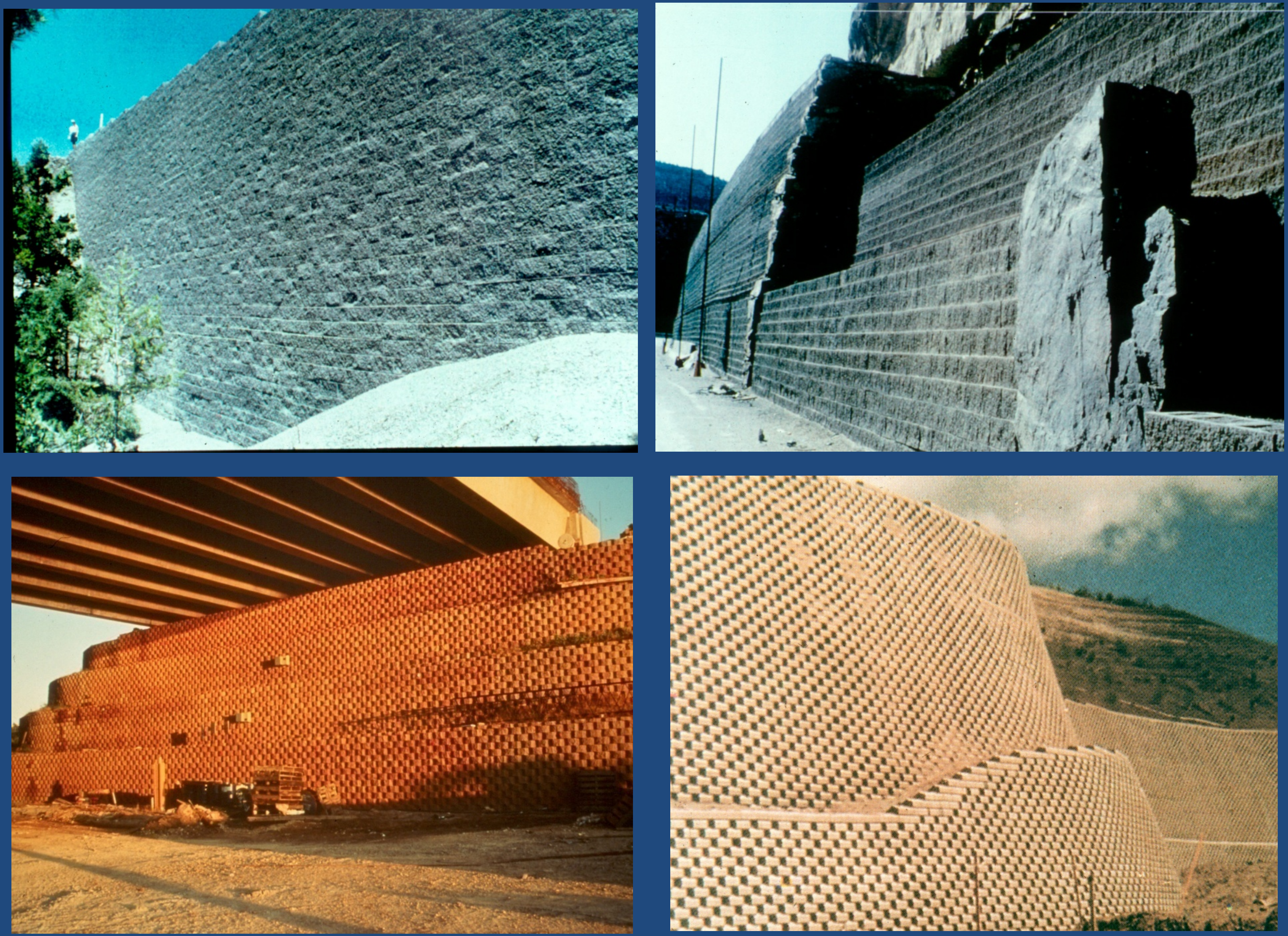
- GRS-RW Features
- Advantages
- Stability Considerations:
  - External stability
  - Internal stability
- Design methods (koerner (2001)
  - Modified Rankine approach-most conservative
  - FHWA method- intermediate
  - NCMA approach- least conservative



Example masonry concrete segmental retaining wall units

Not to scale

Different Styles of Facing



Blockwork wall

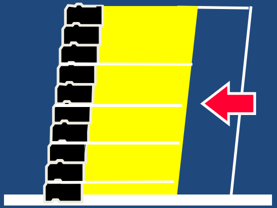
Wall in Residential Development

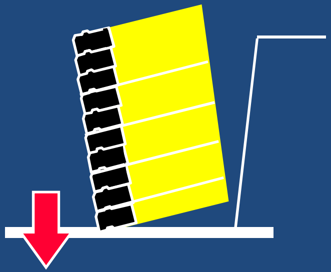
Blockwork Wall Adjacent to Highway

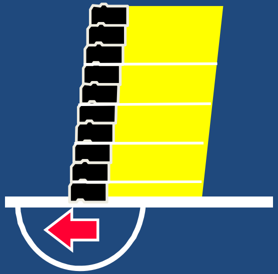
Construction of Walls

Modes of Failure

External


a) base sliding

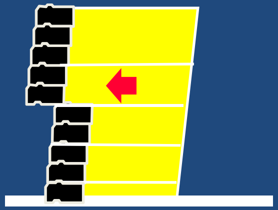

b) overturning


c) bearing capacity
(excessive settlement)

Internal


d) pullout

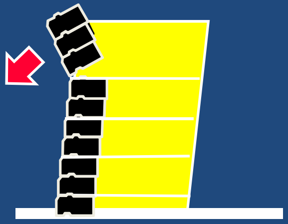

e) tensile over-stress


f) internal sliding

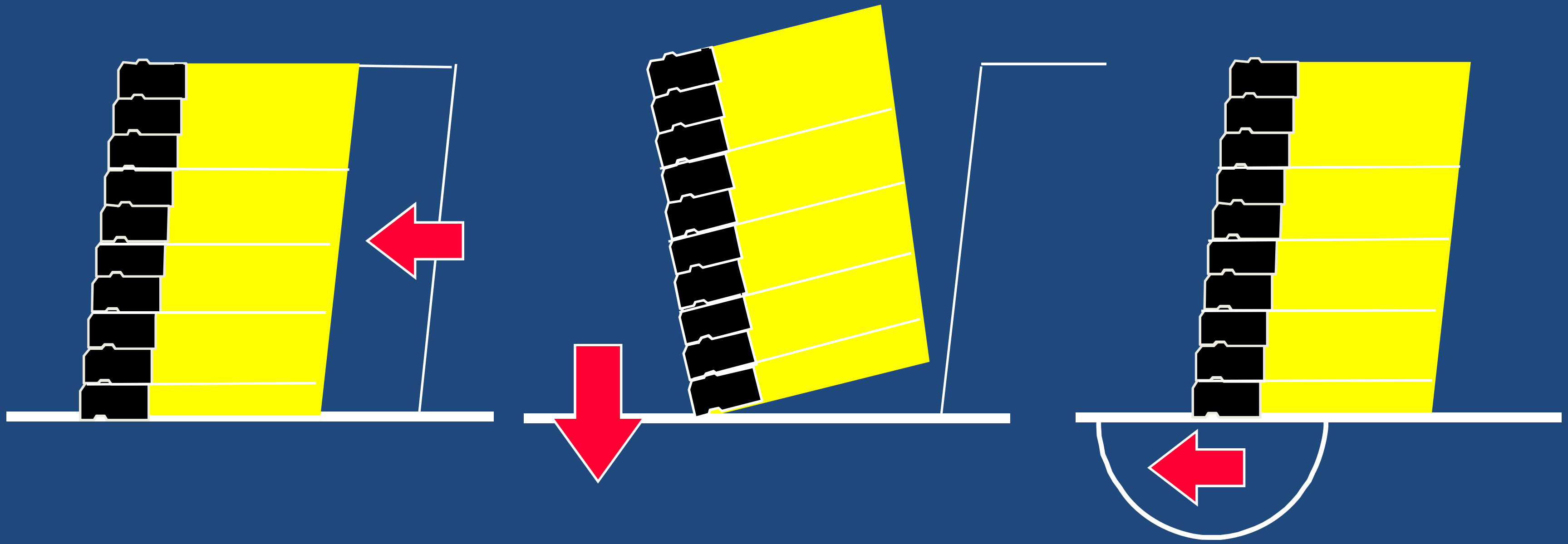
Facing


g) connection
failure


h) column shear failure


i) toppling

External Modes of Failure

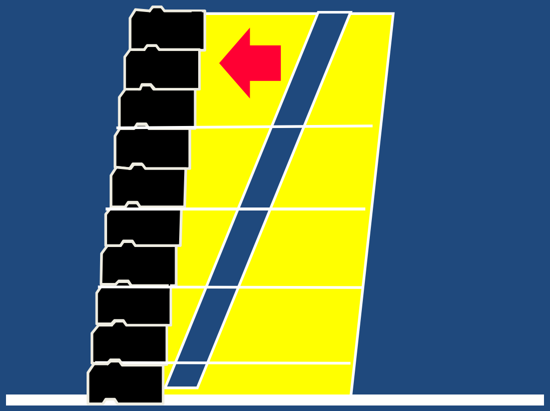


a) base sliding

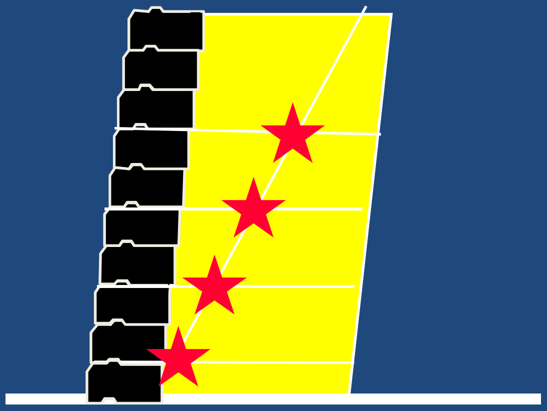
b) overturning

c) bearing capacity
(excessive settlement)

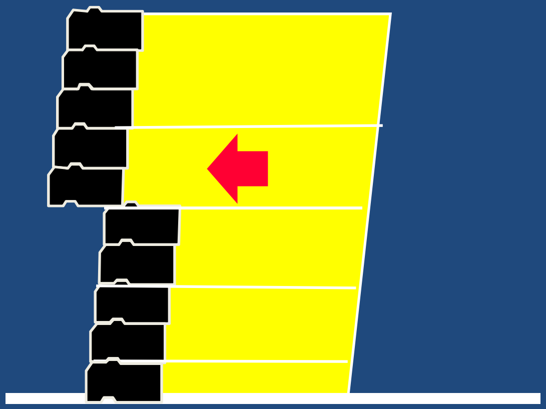
Internal Modes of Failure



d) pullout

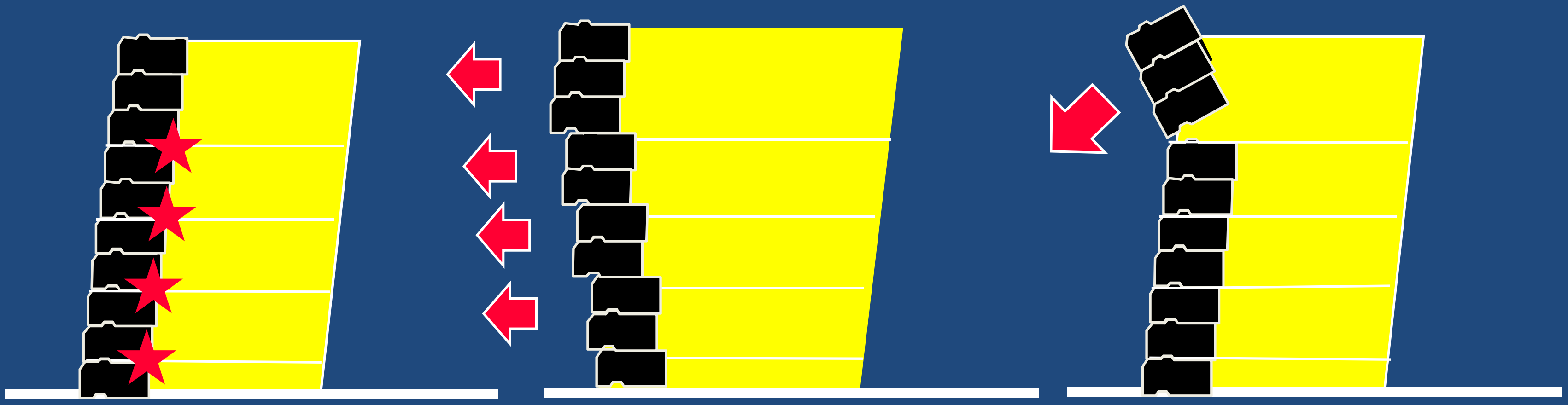


e) tensile over-stress



f) internal sliding

Facing Modes of Failure

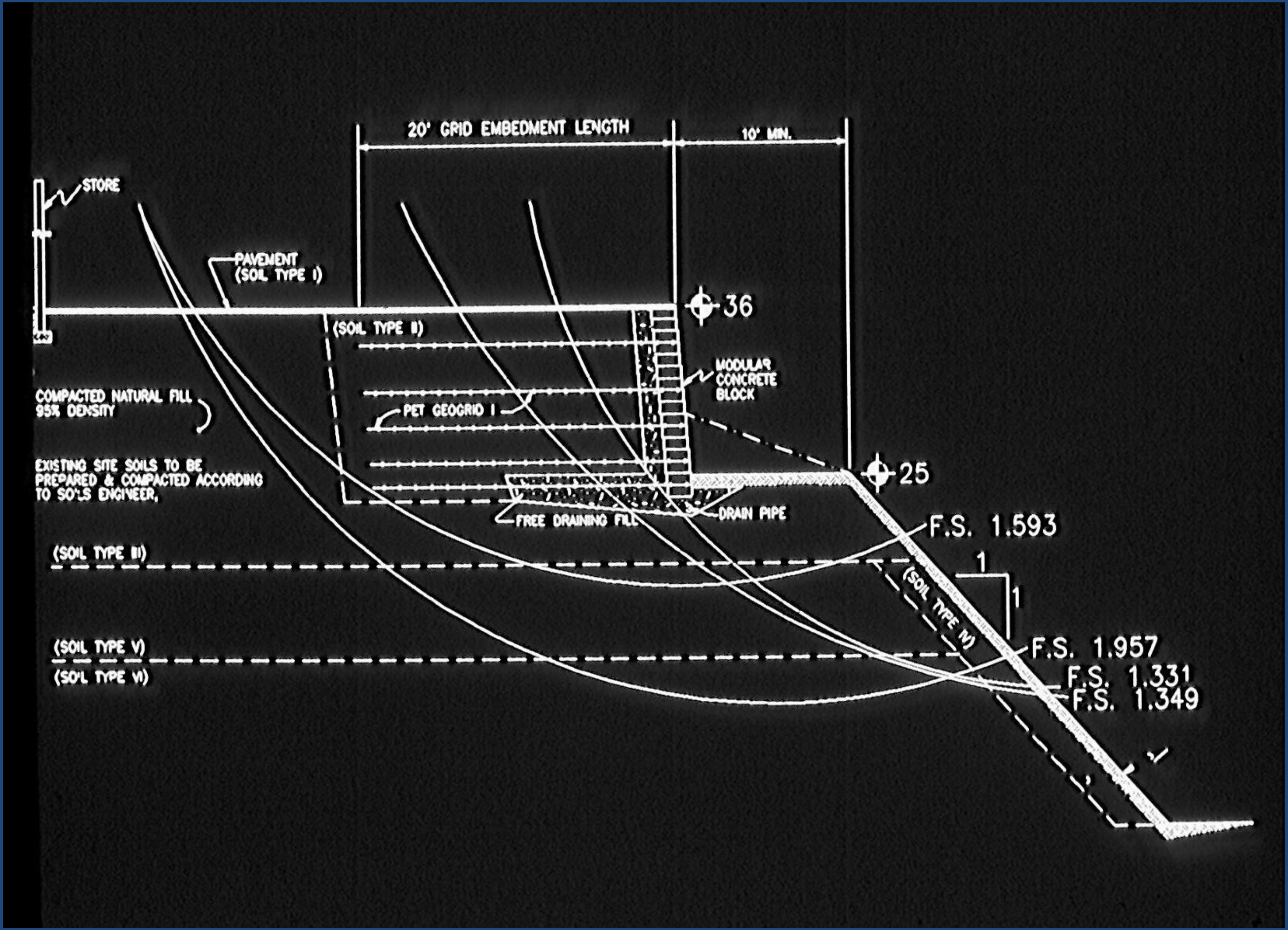


g) connection failure

h) column shear failure

i) toppling

Global Stability



Typical Factors of Safety Against (Collapse) Failure Mechanisms

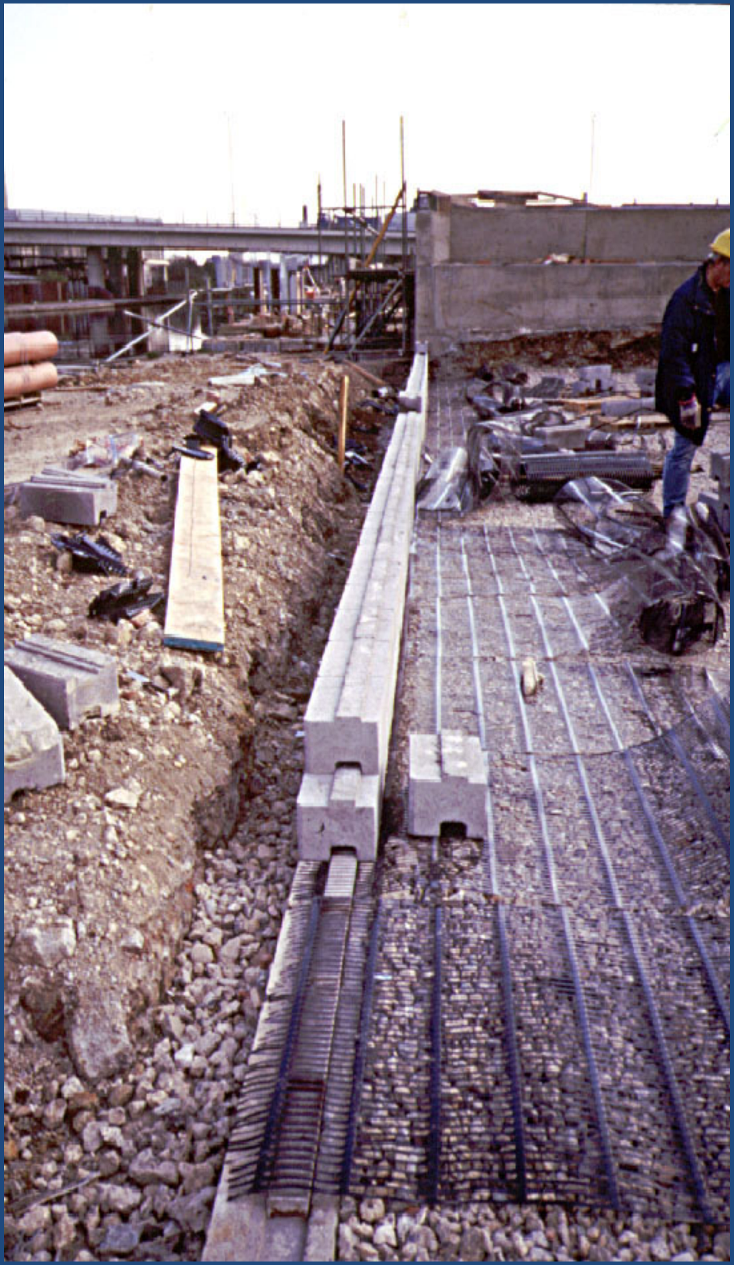
a)	Base sliding	1.5
b)	Overturning	2.0
c)	Bearing capacity	2.0
d)	Tensile over-stress	1.0
e)	Pullout	1.5
f)	Internal sliding	1.5
g)	Connection failure	1.5
h)	Column shear failure	1.5
i)	Toppling	2.0
	Global stability	1.3 - 1.5

Construction Details

Wall Construction

Locking Bar

General view on Wall During Construction



Placing Facing Blocks

Wall Ties Fixing False Facing

Locking Geogrid Between Blocks

Safety Barriers at Top of Wall

Completed Wall with Fence

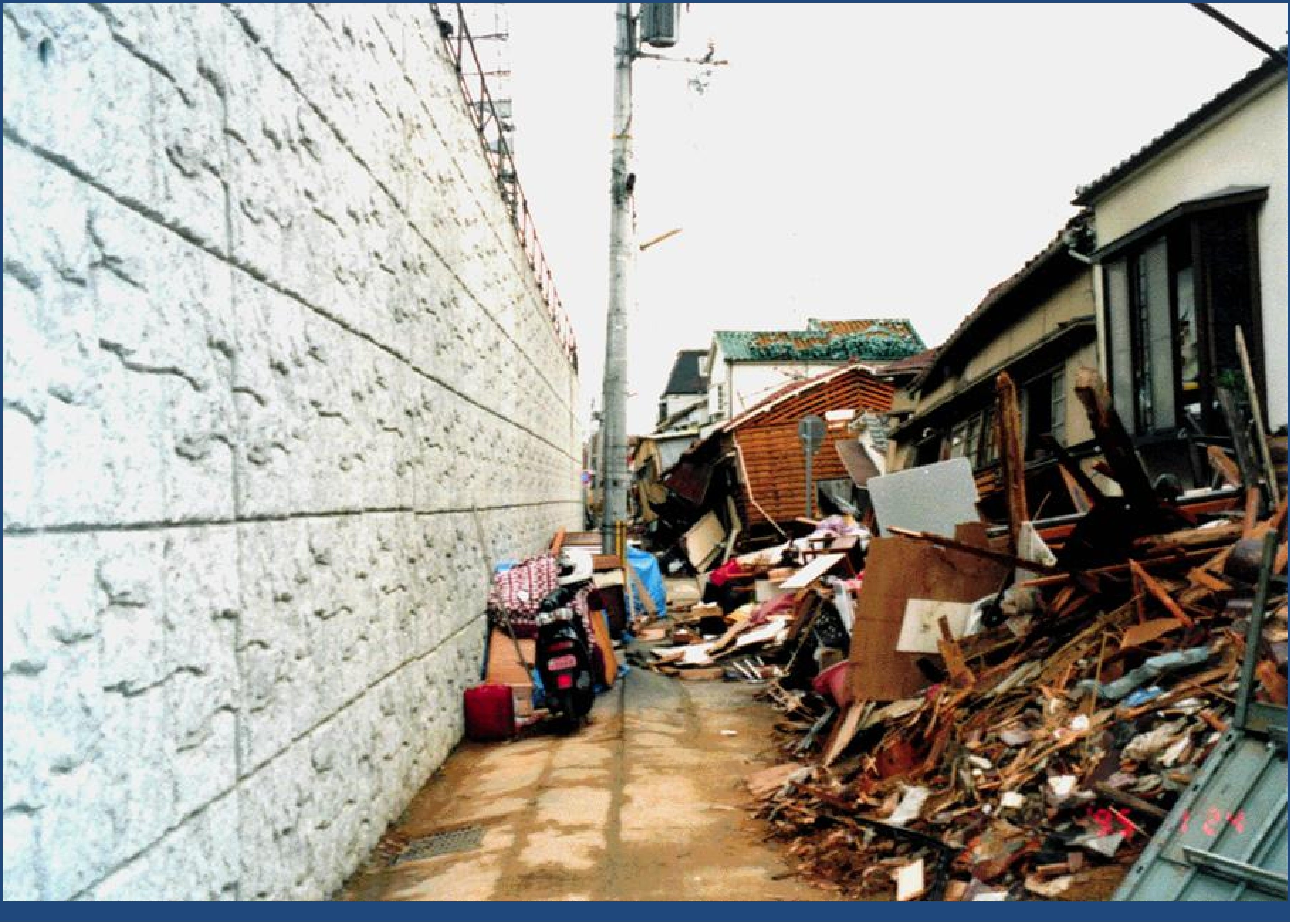
Examples Of Finished Structures

Examples Of Finished Structures

Examples Of Finished Structures

Goegrid-reinforced soil RW along JR Kobe Line (1992)

Goegrid-reinforced soil RW along JR Kobe Line (1995)



Damaged masonry RW, reconstructed to a GRS RW with a full-height rigid facing

Some examples of poor quality

Example calculation

An 8 m high wall is to be built using sand fill and polymer-grid reinforcement. The sand has $\phi' = 30^0$, $\gamma = 18$ kN/m^3 and is to be used for the wall and the backfill. A surcharge loading of 15 kPa is to be allowed for, and the maximum safe bearing pressure for the foundation soil is 300 kPa. Two grids of different design strength are available: grid A at 20 kN/m and grid B at 40 kN/m (both have a bond coefficient f_b of 0.9). The fill will be compacted in layers 250 mm thick.

External stability (sliding)

$K_a = (1 - \sin 30^0) / (1 + \sin 30^0) = 0.333$
$\mu = f_b \tan \phi = 0.9 \times \tan(30) \approx 0.5$.

For a factor of safety against sliding of 2.0, the minimum length of layers is:

$$L_{\min} \geq \frac{F_S K_{ab} H \left(\gamma_w H + 2w_S\right)}{2\mu\left(\gamma_w H + w_S\right)}$$

$$L \geq \frac{2x0.333 \times 8 \times \left(18 \times 8 + 2 \times 15\right)}{2 \times 0.5 \times \left(18 \times 8 + 15\right)} \geq 5.83\text{m.}$$

Therefore adopt a length of 6m.

External stability (Overturning)

Overturning moments about the toe =

$$(k_{ab}\gamma_b \frac{H^3}{6} + k_{ab} \frac{w_s H^2}{2})$$

Restoring moments about the toe =

$$(\gamma_\omega \frac{HL^2}{2}) + (\frac{w_s L^2}{2})$$

Factor of safety against overturning =

$$\frac{3(\gamma_w H + w_s)}{k_{ab}(\gamma_b H + 3w_s)(H/L)^2}$$

$$FS = \frac{3(18x8 + 15)}{0.333(18x8 + 45)(8/6)^2} = 4.26 > 2$$

Bearing pressure

Using trapezoidal distribution,

$\sigma_{v\ max}$ = (18 × 8 + 15) + 0.333 × (18 × 8 + 45) $(8/6)^2$ = 159 + 112 = 271 kPa. (< 300 kPa)

Check that contact stresses at the base of reinforced zone are compressive everywhere (i.e. no tension):

$\sigma_{v\ min}$ = 159 – 112 = 47 kPa. (> 0)

$$T = \sigma'_h S_V = K\sigma'_v S_V$$

$$\sigma_v = (\gamma z + w_S) + K_a (\gamma z + 3w_S)(z / L)^2$$

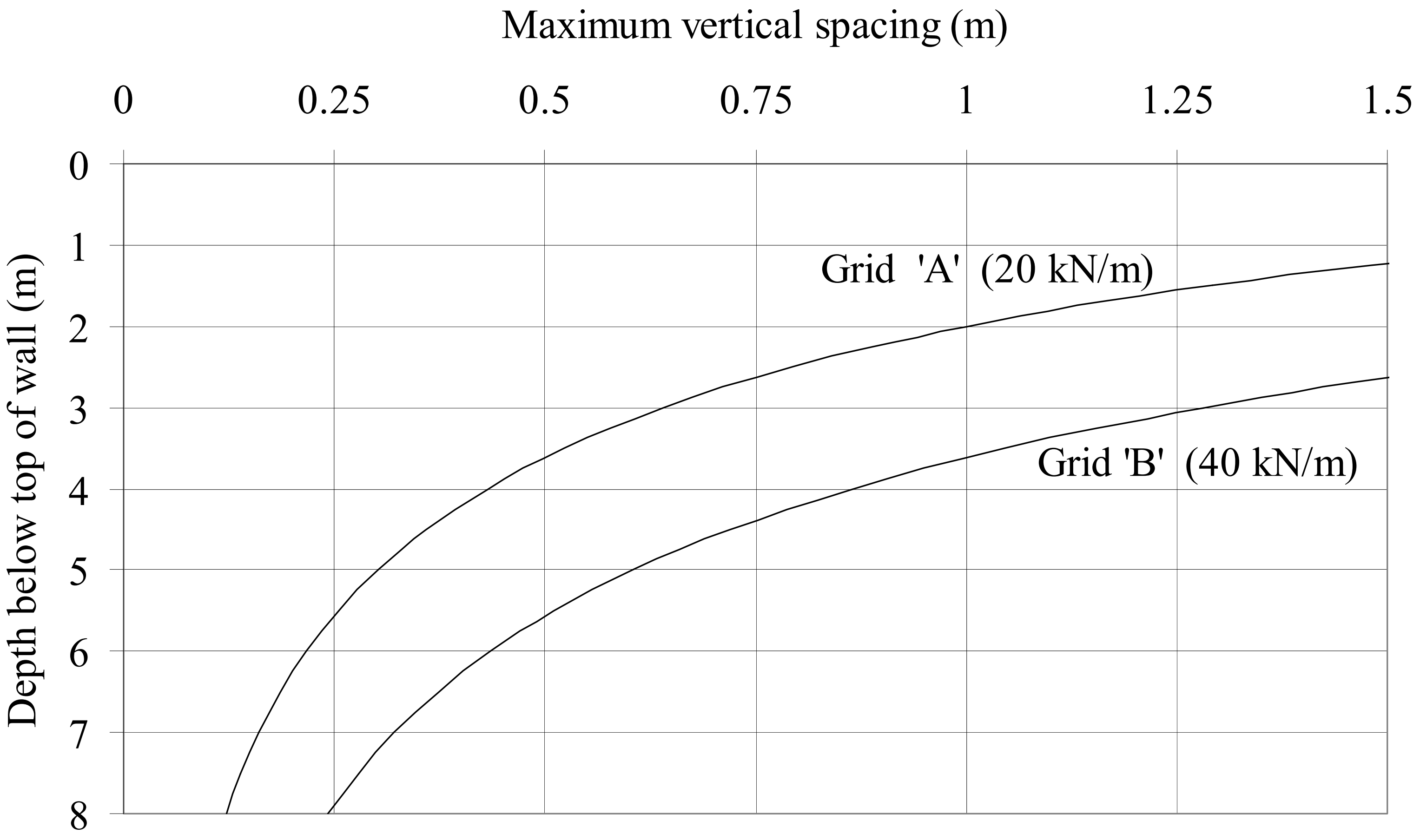
$$T_i = 0.333\ [(18z + 15) + 0.333\ (18z + 45)\ (z/6)^2]\ S_V$$

$$(S_V)_{max} = \frac{P_d}{0.333\left[(18z+15)+0.333(18z+45)(z/6)^2\right]}$$

Two different grids that are available the use of above equation results in the values presented in the Table.

Maximum spacing of geogrids, $(S_v)_{max}$		
z (m)	Grid A (P_d=20 kN/m)	Grid B (P_d=40 kN/m)
0.5	2.46	4.93
1.0	1.73	3.46
1.5	1.29	2.58
2.0	1.00	2.00
2.5	0.79	1.59
3.0	0.64	1.28
3.5	0.52	1.05
4.0	0.43	0.86
4.5	0.36	0.72
5.0	0.30	0.60
5.5	0.26	0.51
6.0	0.22	0.44
6.5	0.19	0.37
7.0	0.16	0.32
7.5	0.14	0.28
8.0	0.12	0.24

Spacing versus depth plot for grids A and B



Wedge stability check

Select trial wedges at depths, 1 to 8 m below the top of the wall and calculate the total required force T. Carry out check with and without surcharge w_s. For critical wedge angle $\beta = (45^0 - \phi'_w/2 = 30^0$ for a wedge of height h, the total tension force T is given by

$$T = \frac{h \tan 30^0 (18h + 2 \times 15)}{2 \tan (30^0 + 30^0)} = 3h^2 + 5h$$

For a reinforcing layer at depth z below the top of the wall, the pullout resistance is given by

$P_P = 2\,[L - (h - z) \tan \beta] \times (\gamma z + w_s) \times 0.9 \times \tan 30^0 / 2.$

The factor 2 in the numerator denotes the upper and lower surfaces on either side of the geogrid and factor 2 in the denominator refers to the factor of safety.

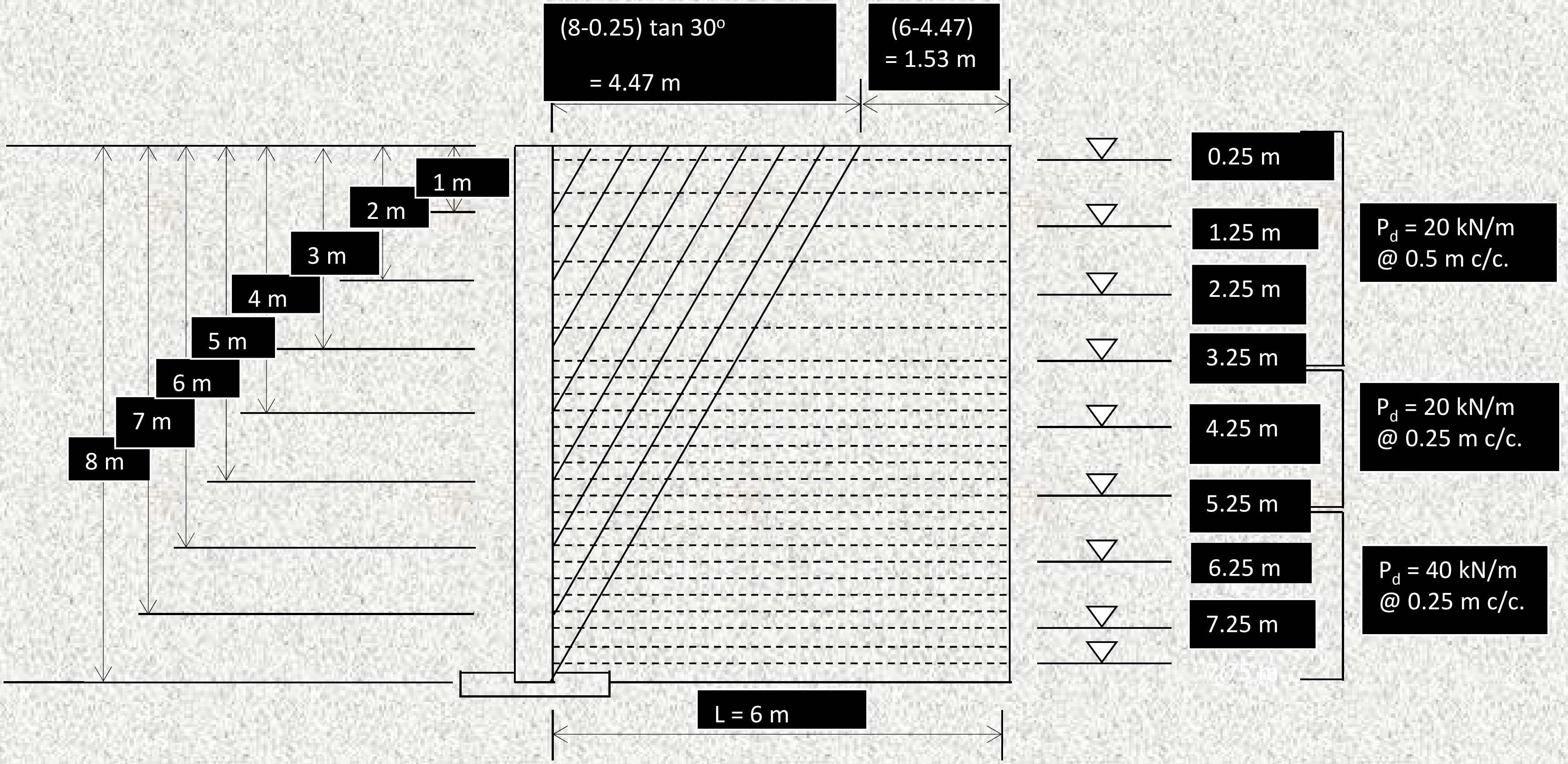
$$P_P = 2\,[6 - (h - z)\tan 30^0] \times (18z + 15) \times 0.9 \times \tan 30^0 / 2.$$

For each reinforcement intersected, the available force is taken as the lesser of the pullout resistance P_P and the design tensile strength P_d. For all wedges and both load cases, available force is greater than required force, T. A suitable reinforcement layout is arrived at based on the above considering the thickness of compaction lifts.

Calculation of mobilizing and resisting forces for wedge stability

Wedge Depth (m)	Force to be resisted T (kN/m)		Grids Involved	Design Tensile force, P_d (kN/m)	Pullout resistance P_p (kN/m)		Available force (kN/m) (minimum of P_d & P_p)	
	$w_s = 0$	$w_s = 15$ kPa			$w_s = 0$	$w_s = 15$ kPa	$w_s = 0$	$w_s = 15$ kPa
1	8	3	2A	40	42	16	40	16
2	22	12	4A	80	141	80	80	80
3	42	27	6A	120	318	213	120	120
4	68	48	9A	180	732	548	180	180
5	100	75	13A	260	1495	1189	260	260
6	138	108	15A+2B	380	2538	2092	380	380
7	182	147	15A+6B	540	3905	3301	540	540
8	232	192	15A+10B	700	5639	4859	700	700

Reinforcement Layout



(8-0.25) tan 30^o

= 4.47 m

(6-4.47)
= 1.53 m

P_d = 20 kN/m
@ 0.5 m c/c.

P_d = 20 kN/m
@ 0.25 m c/c.

P_d = 40 kN/m
@ 0.25 m c/c.

L = 6 m

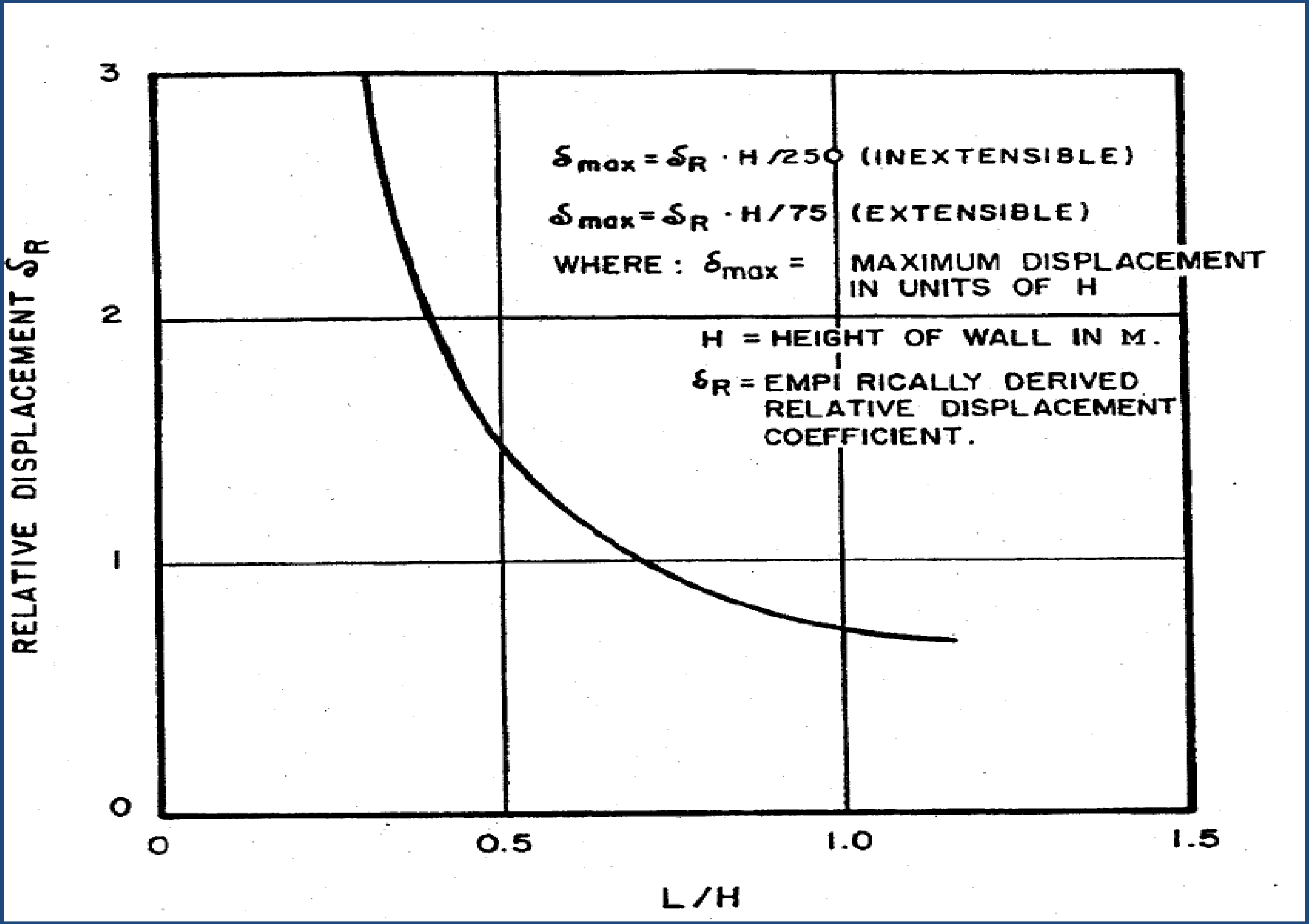
Provisions of FHWA

Recommended minimum factors of safety with respect to External failure modes

Sliding	F.S >= 1.5 (MSEW); 1.3 (RSS)
Eccentricity e, at Base	<= L/6 in soil L/4 in rock
Bearing Capacity	F.S. >= 2.5
Deep Seated Stability	F.S >=1.3
Compound Stability	F.S. >= 1.4
Seismic Stability	F.S. >= 75% of static F.S.

Table1.2: Recommended minimum factors of safety with respect to internal failure modes

Pullout Resistance	**F.S. >= 1.5 (MSEW and RSS)**
Internal Stability for RSS	**F.S >= 1.3**
Allowable Tensile Strength	**0.55 Fy**
(a) For steel strip reinforcement	
(b) For steel grid reinforcementpanels	**0.48 Fy (connected to concrete Panels or blocks)**



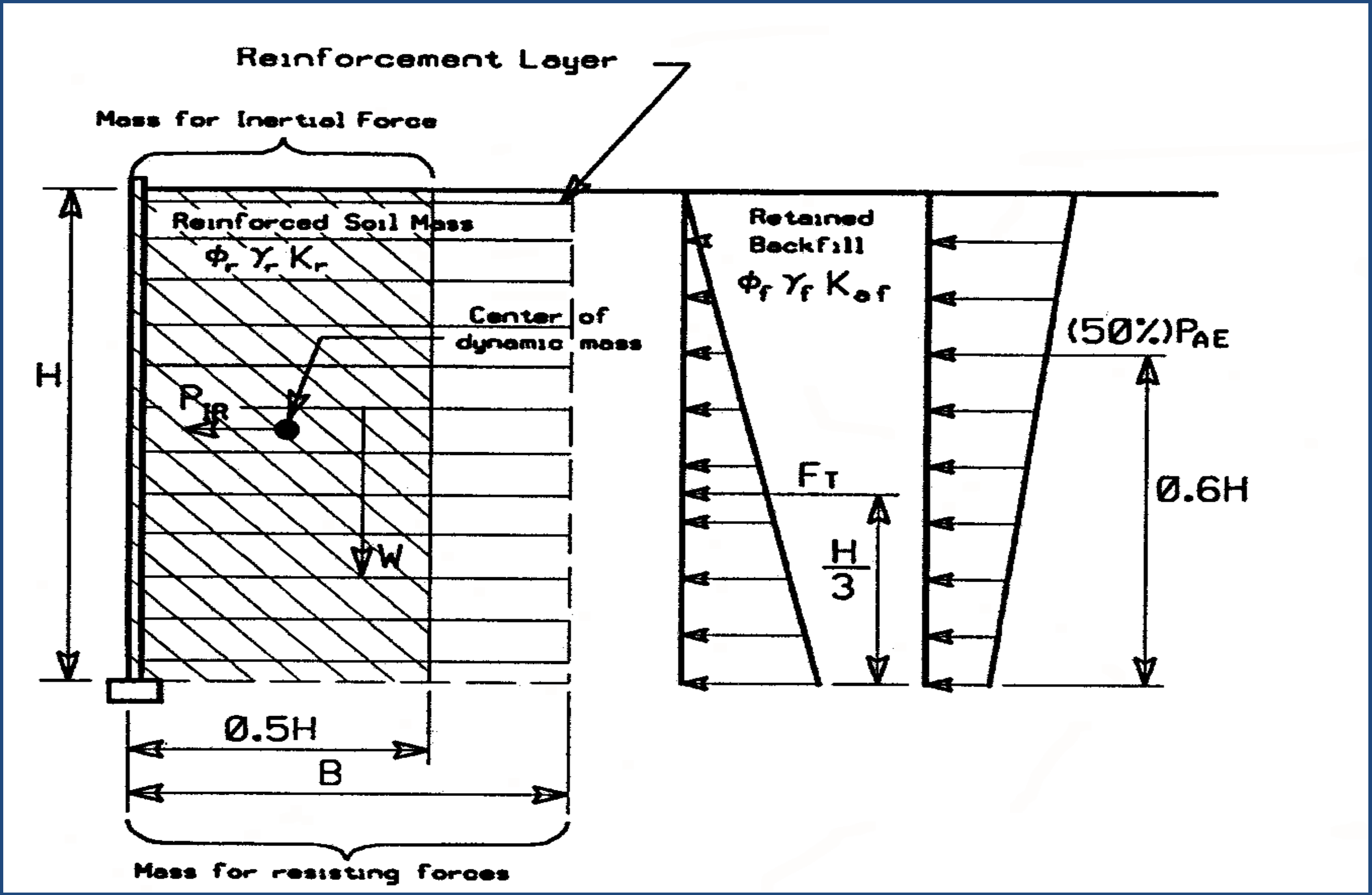
Empirical curve for estimating probable anticipated lateral displacement during construction for MSE walls

Table1.3: Recommended backfill requirements for MSE & RSS construction

U.S Sieve Size	% Passing
For MSE Walls	
102 mm	100
0.425 mm	0-60
0.075 mm	0-15
For RSS Walls	
20mm	100
4.76mm	100-20
0.425mm	0-60
0.075mm	0-50

Table 1.4: Recommended limits of electrochemical properties for backfills when using steel reinforcement

Property	Criteria	Test Method
Resistivity	>3000 ohm-cm	AASHTO
pH	>5<10	AASHTO
Chlorides	<100 PPM	AASHTO
Sulfates	<200 PPM	AASHTO
Organic Content	1% max	AASHTO



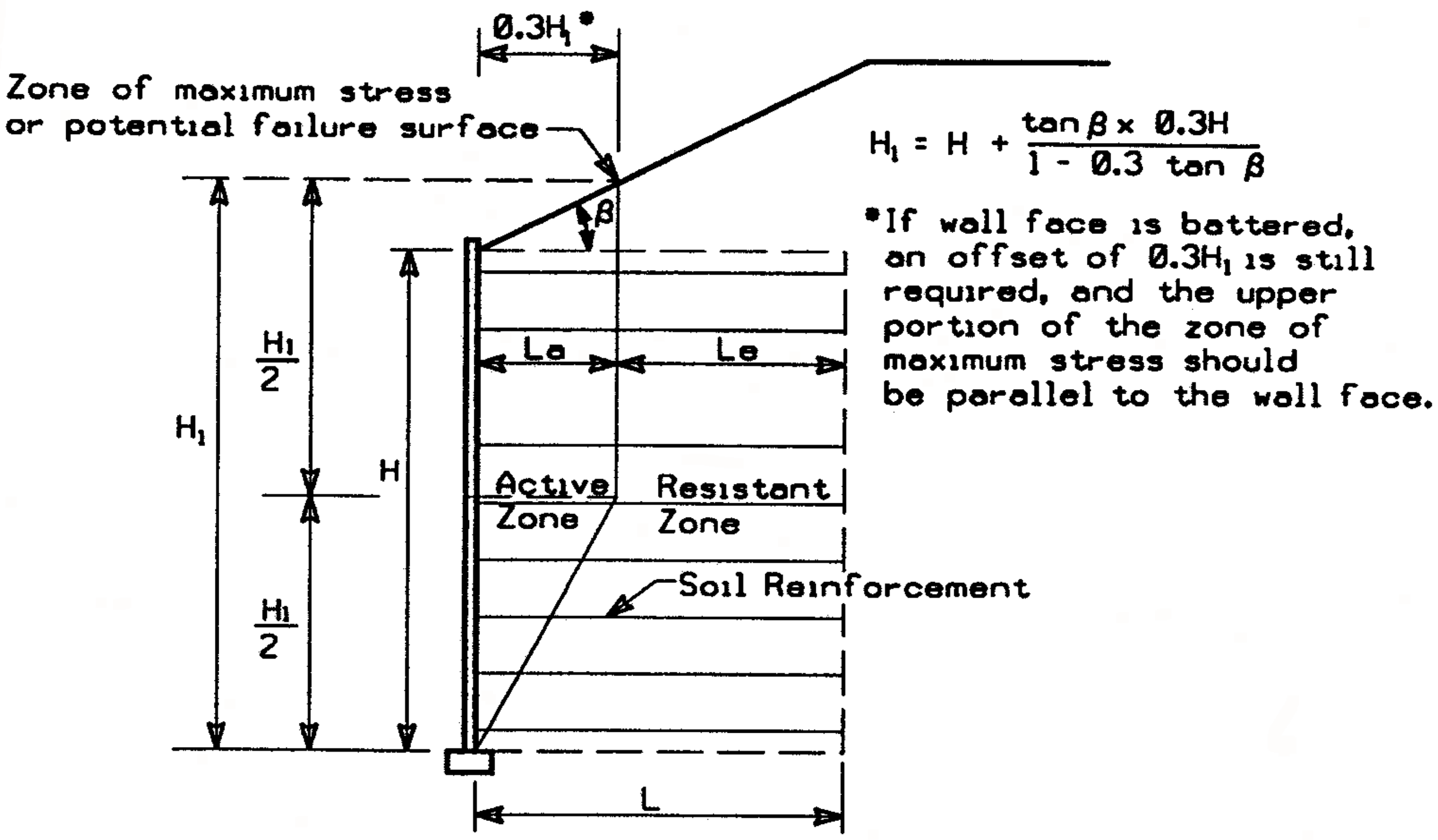
Seismic external stability of a MSE wall under level backfill condition

- Select a horizontal ground acceleration (A) based on design earthquake
- Calculate maximum acceleration (A_m) developed in the wall using $A_m = (1.45 - A)A$
- Calculate the horizontal inertial force (P_{IR}) and the seismic thrust (P_{AE}) using

$$P_{IR} = 0.5\, A_m\, \gamma_r\, H^2$$

$$P_{AE} = 0.375\, A_m\, \gamma_f\, H^2$$

Add to static force acting on the structure, 50% of the seismic thrust P_{AE} and the full inertial; force as both forces do not act simultaneously

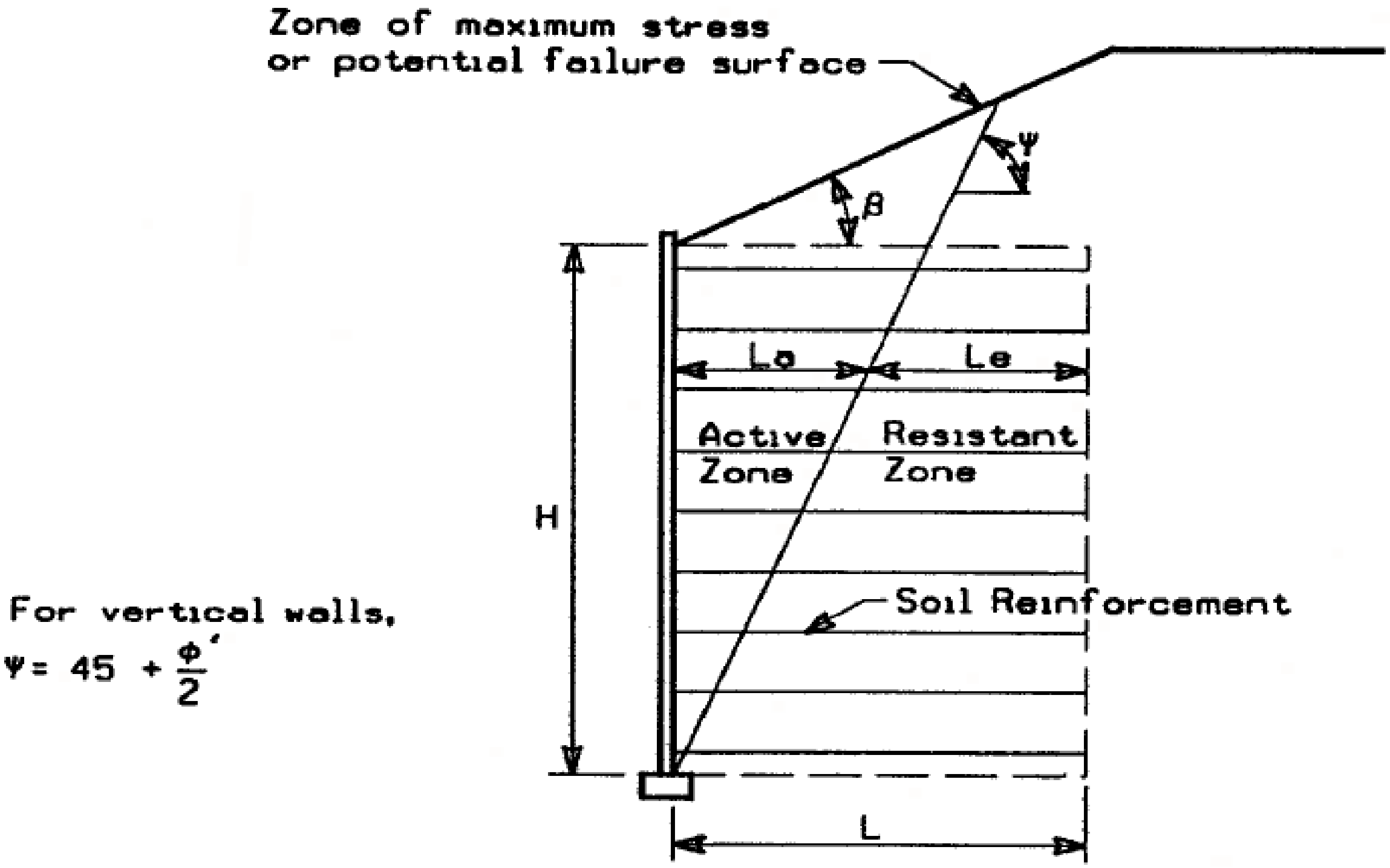


$$H_1 = H + \frac{\tan\beta \times 0.3H}{1 - 0.3\tan\beta}$$

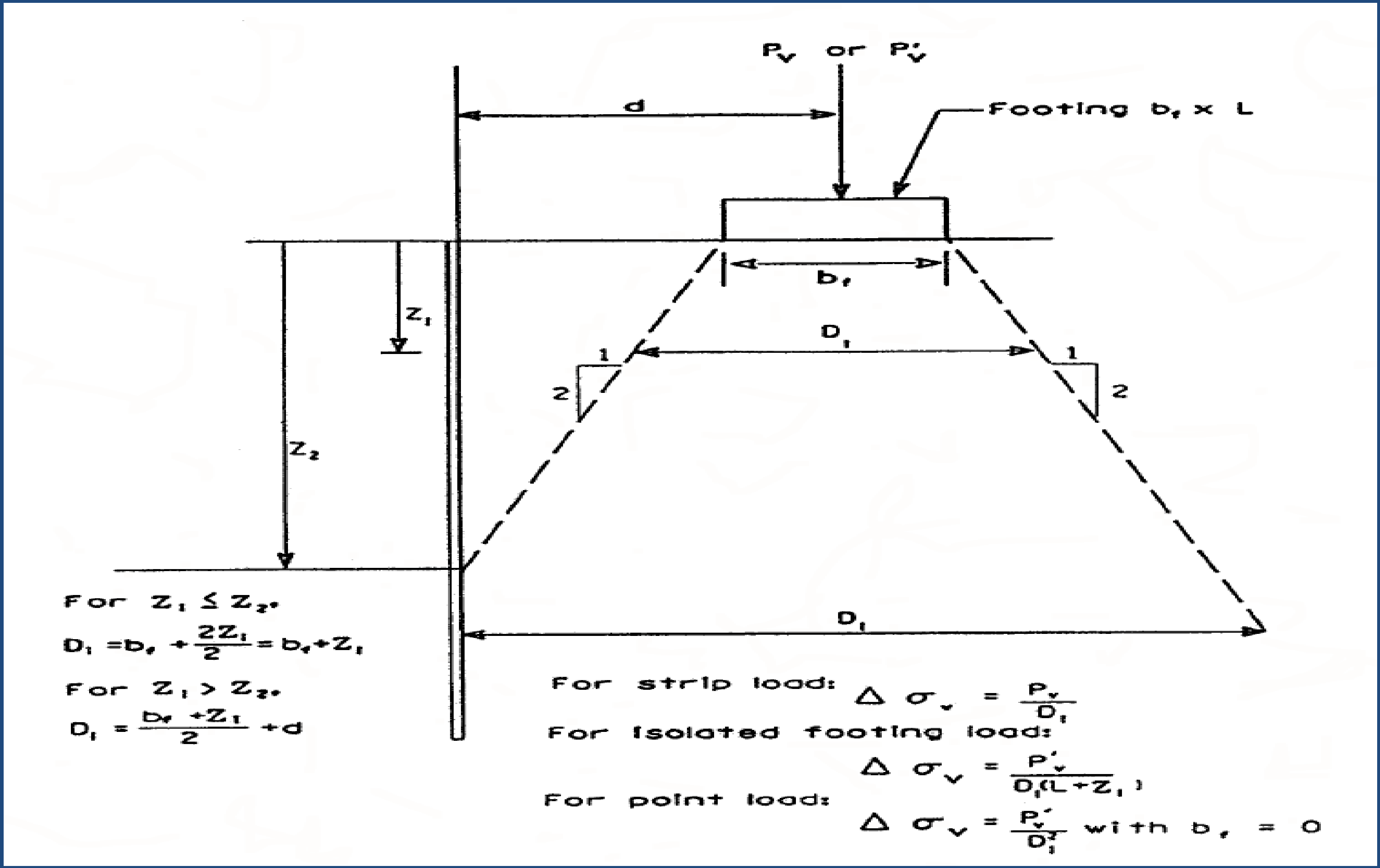
*If wall face is battered, an offset of $0.3H_1$ is still required, and the upper portion of the zone of maximum stress should be parallel to the wall face.

(a) Inextensible Reinforcements

Location of potential failure surface for internal stability design of MSE walls



Location of potential failure surface for internal stability design of MSE walls for extensible reinforcement.



For $Z_1 \le Z_2$,

$D_1 = b_f + \frac{2Z_1}{2} = b_f + Z_1$

For $Z_1 > Z_2$,

$D_1 = \frac{b_f + Z_1}{2} + d$

For strip load: $\Delta \sigma_v = \frac{P_v}{D_1}$

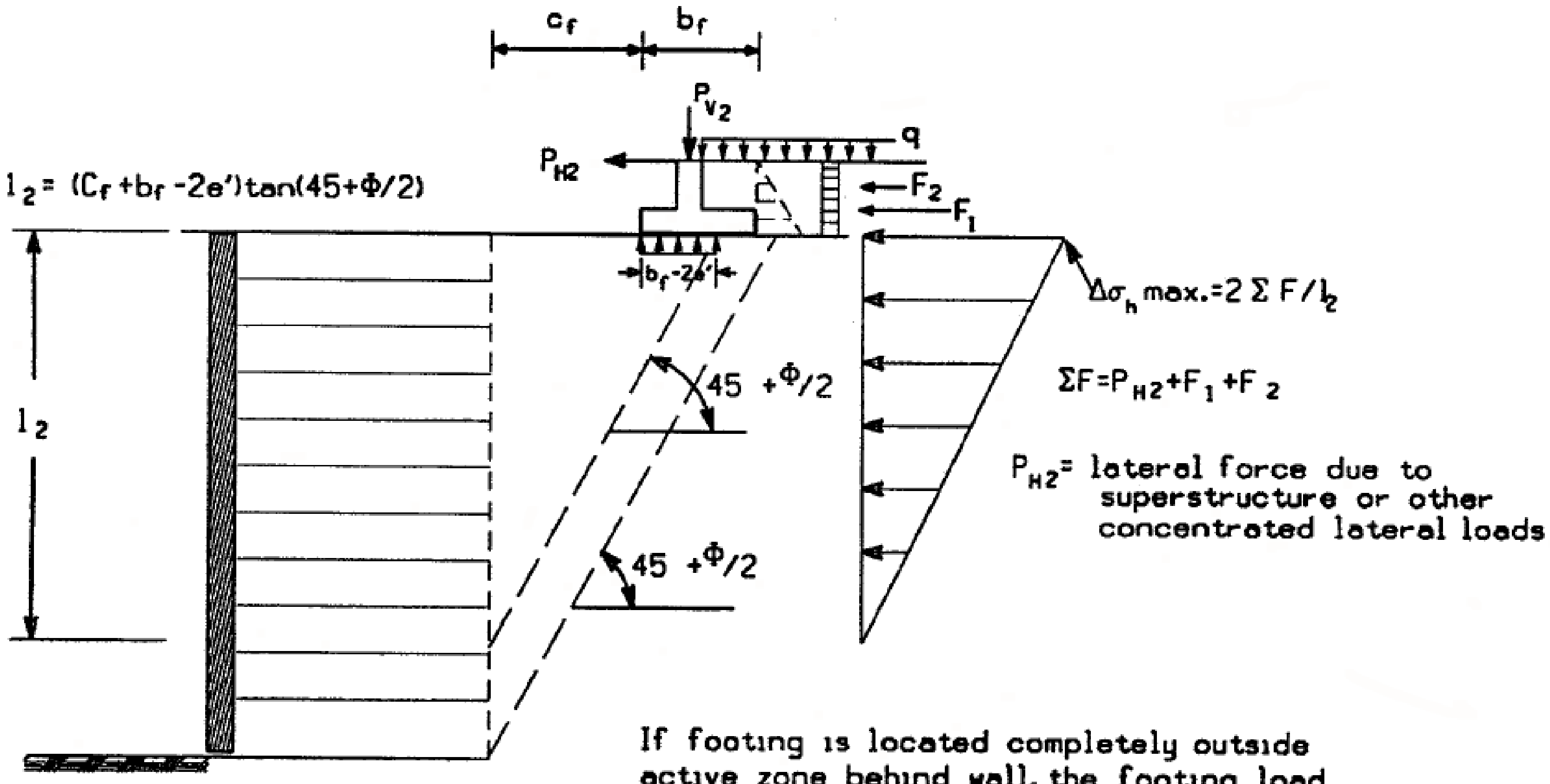
For isolated footing load:

$\Delta \sigma_v = \frac{P'_v}{D_1(L + Z_1)}$

For point load:

$\Delta \sigma_v = \frac{P'_v}{D_1^2}$ with $b_f = 0$

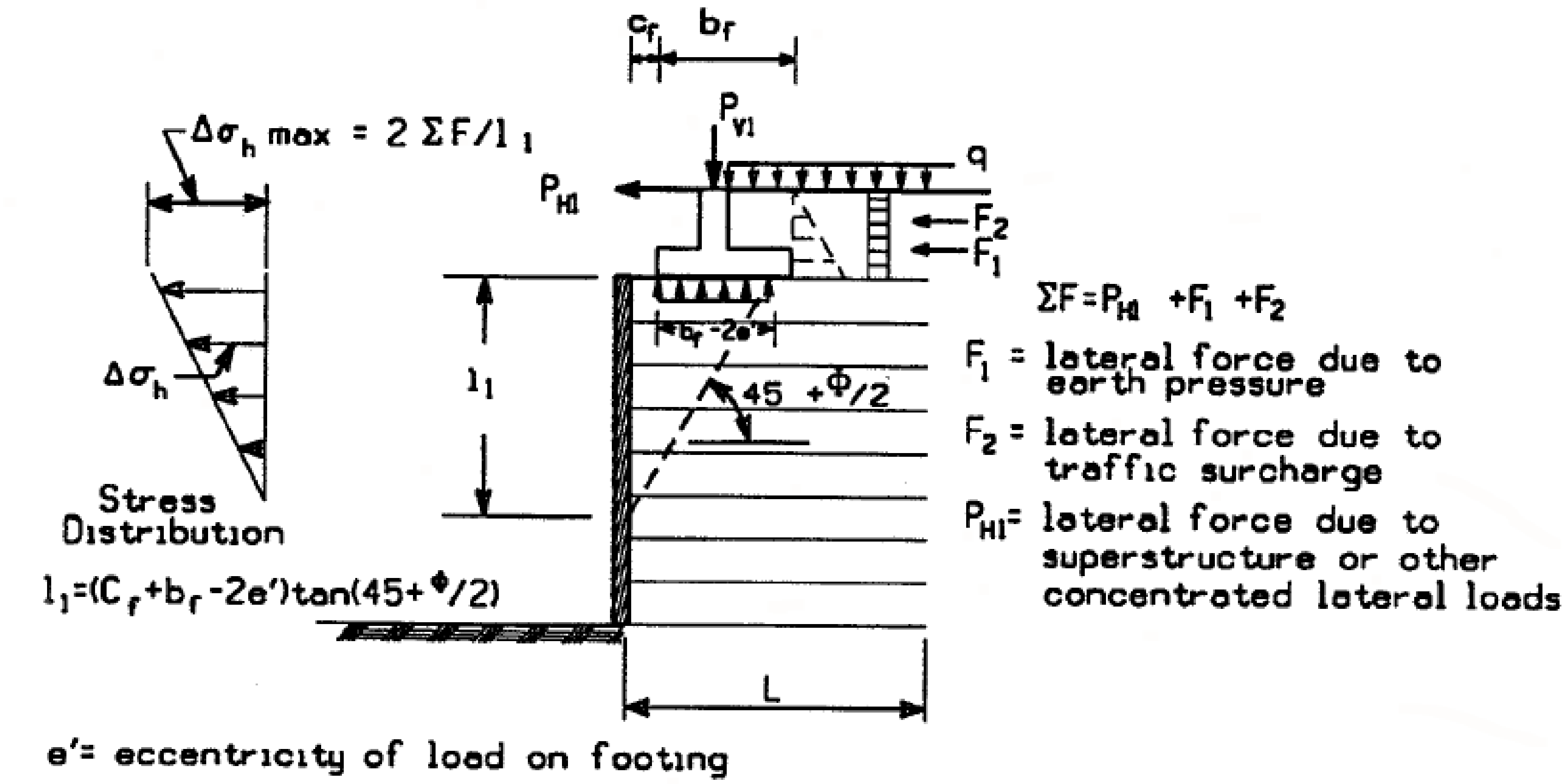
Distribution of stress from concentrated vertical load P_v for internal and external stability calculations.



If footing is located completely outside active zone behind wall, the footing load does not need to be considered in the external stability calculations.

b. Distribution of Stress for External Stability Calculations.

Distribution of stresses from concentrated horizontal loads for external stability.



a. Distribution of Stress for Internal Stability Calculations.

Distribution of stresses from concentrated horizontal loads for internal stability.

Concluding remarks

- Reinforced retaining walls have evolved as viable technique and contributed to infrastructure in terms of speed, ease of construction, economy, aesthetics etc.
- It is a technology that needs to be understood well in terms of its response, construction features etc. Failures of RE walls have also been noted in a few places due to lack of understanding of behavour of RE walls.
- FWHA, NCMA guidelines need to be studied in detail for seismic stability and deformation issues.

THANK YOU