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DEPARTMENT OF CIVIL AND
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14.533 ADVANCED FOUNDATION ENGINEERING

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2013
CLASS NOTES

EARTH PRESSURES

- At Rest Lateral Pressure
- Rankine Active and Passive Earth Pressure States
- Relations Between Earth Pressures and Wall Movements
- DIA – Dual Interfacial Apparatus
- Interfacial Friction and Adhesion
- Active and Passive Earth Pressure Coefficients
- Computation of a General Active Case
- Horizontal Pressure from Surface Loads
- Effect of Ground Water and Filter on Wall Pressures
- Earth Pressure due to Compaction
- Earth Pressure on Rigid Retaining Walls Near Rock Faces

EARTH PRESSURES

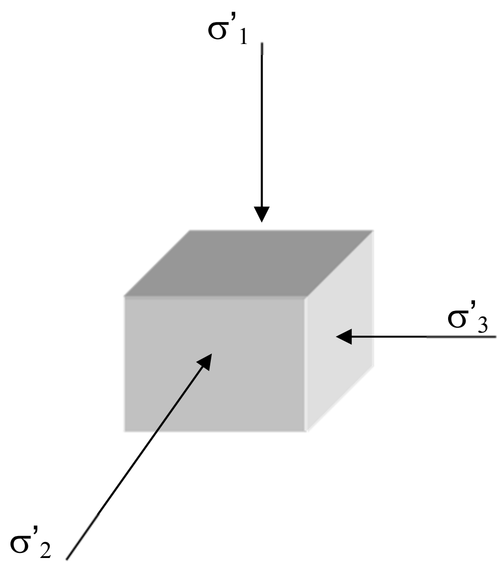
- Review Ch. 6 in H.Y. Fang, “Foundation Engineering Handbook” or Ch. 11 in J.E. Bowles, “Foundation Analysis and Design” (5th ed.) or Ch 7 in B.M. Das “Foundation Engineering” (7th ed.)
- Design of earth-retaining structures requires knowledge of the earth, water, and external loads that will be exerted on the structures.

AT REST LATERAL PRESSURE

(a) Theoretical elasticity under conditions of lateral zero disp. (referring to effective stresses).

(1) $\varepsilon_2 = \frac{1}{E}\left[\Delta\sigma_2 - \nu\left(\Delta\sigma_3 + \Delta\sigma_1\right)\right]$

(2) $\varepsilon_3 = \frac{1}{E}\left[\Delta\sigma_3 - \nu\left(\Delta\sigma_1 + \Delta\sigma_2\right)\right]$



for zero lateral yield $\varepsilon_3 = \varepsilon_2 = 0$

for orthotropic or oedometer conditions

$$\Delta\sigma_2 = \Delta\sigma_3$$

(1) + (2) $2\Delta\sigma_3 = 2\nu\left(\Delta\sigma_2 + \Delta\sigma_3\right)$

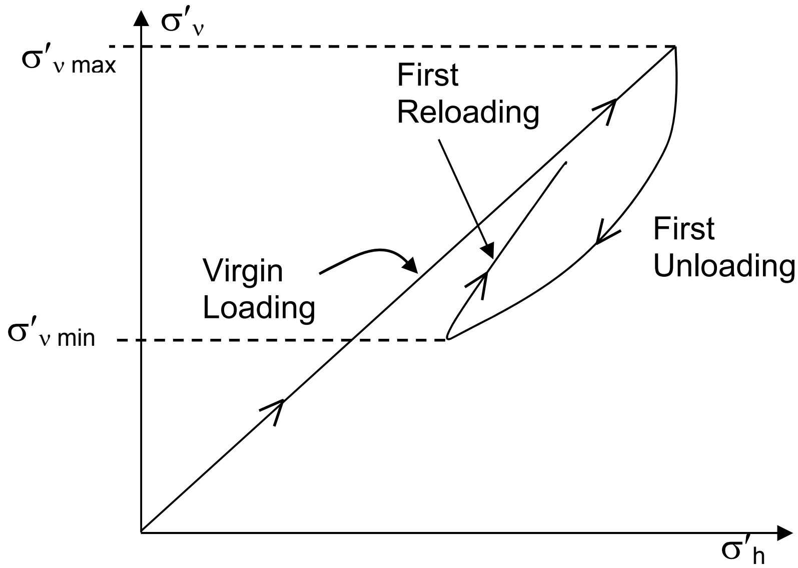
$$\Delta\sigma_3\left(1-\nu\right) = \nu\Delta\sigma_1$$

$\Delta\sigma_3 = \frac{\nu}{1-\nu}\Delta\sigma_1$ K = $\sigma_h/\sigma_v = \Delta\sigma_3/\Delta\sigma_1$

$\boxed{K_o = \nu / 1-\nu}$

say	ν = 0.15	K_O = 0.18
	ν= 0.30	K_O = 0.43
	ν= 0.50	K_O = 1.00

(b) Empirical Correlations After Mayne & Kulhawy (1982)



$$OCR_{max} = \frac{\sigma'_{\nu\,max}}{\sigma'_{\nu\,min}}$$

$$OCR = \frac{\sigma'_{\nu\,max}}{\sigma'_{\nu}}$$

Reloading (empirical)

$$K_o = (1 - \sin\phi)\left[\frac{OCR}{OCR_{max}^{(1-\sin\phi)}} + \frac{3}{4}\left(1 - \frac{OCR}{OCR_{max}}\right)\right]$$

Unloading

O.C. $OCR = OCR_{max} \rightarrow Ko = (1\text{-}\sin\phi')\,OCR^{\sin\phi'}$

Virgin loading

N.C. $OCR = OCR_{max} = 1 \rightarrow K_o = (1\text{-}\sin\phi')$ (Jacky, 1944)

References:

Mayne, P., and Kulhawy, F. (1982). "K_o-OCR Relationships in Soil", *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 108, GT6, pp. 851-872.

Kulhawy, F., and Mayne, P. (1990). *Manual on Estimating of Soil Properties for Foundation Design,* Electric Power Research Institute Report EPRI EL-6800, Palo Alto, CA.

$$K_0 = K_{onc}\left[\frac{OCR}{OCR_{max}^{1-\alpha}} + m_r\left(1 - \frac{OCR}{OCR_{max}}\right)\right]$$

Kulhawy & Mayne (1990)

Substituting (see below) $K_{onc} = 1 - \sin\phi$, $\alpha = 1 - K_{onc} \rightarrow 1 - \alpha = 1 - \sin\phi'$, $m_r = 0.75$

brings to the equation previously presented

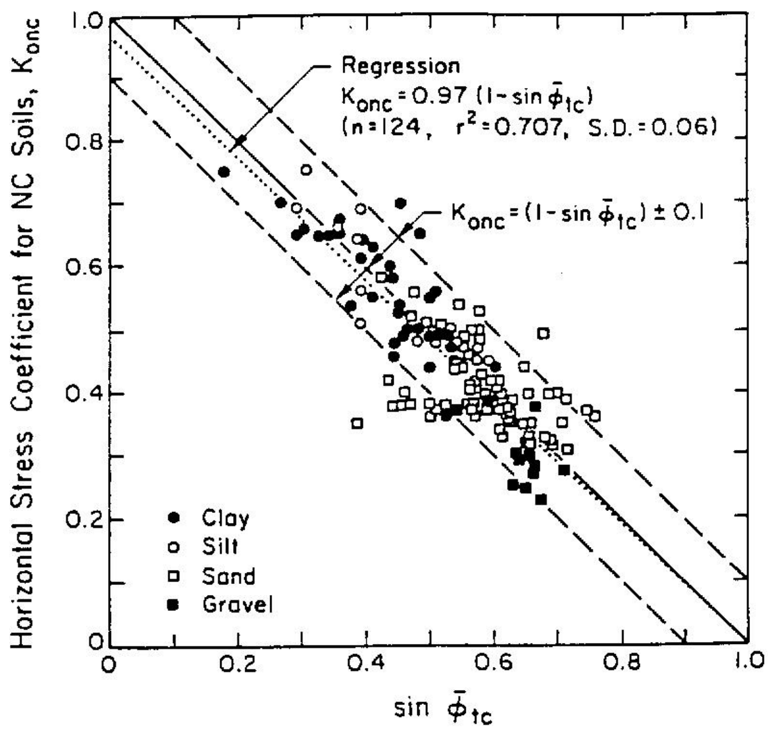


Figure 3-3. Horizontal Stress Coefficient for NC Soils from Laboratory Tests

Source: Mayne and Kulhawy (1), p. 862.

$K_{onc} = (1 - \sin\phi'_{tc}) \pm 0.1$(range)

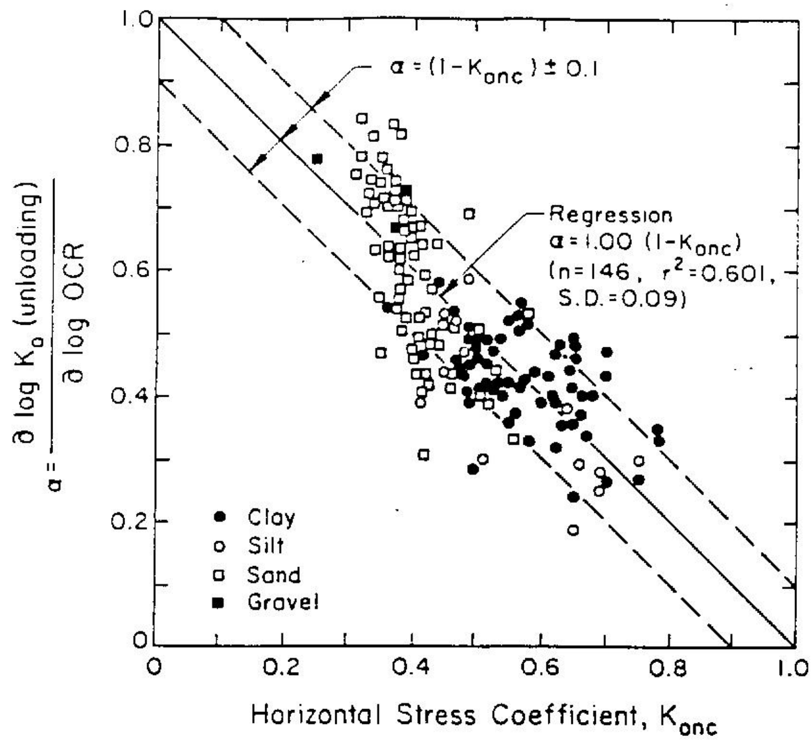


Figure 3-4. Unload Coefficient for OC Soils

Source: Mayne and Kulhawy (1), p. 864.

$\alpha = (1 - K_{onc}) \pm 0.1$(range)

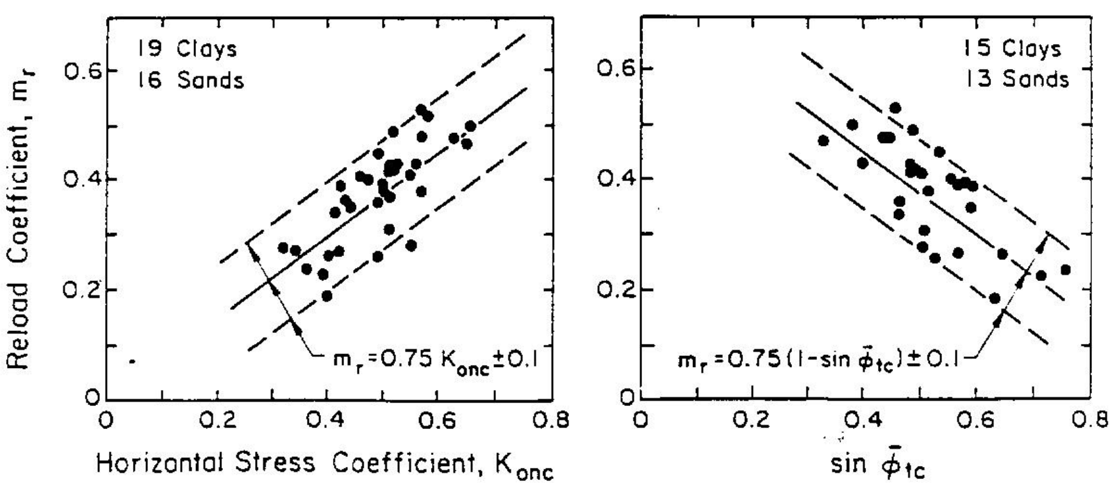
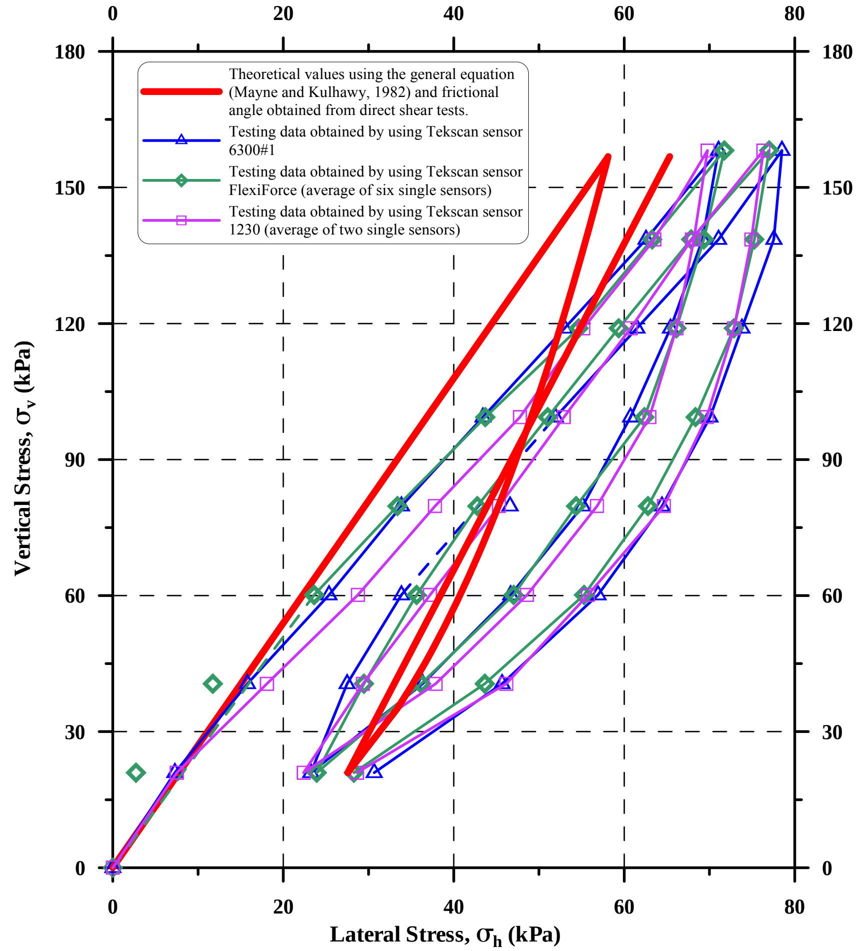


Figure 3-5. Reload Coefficient for OC Soils

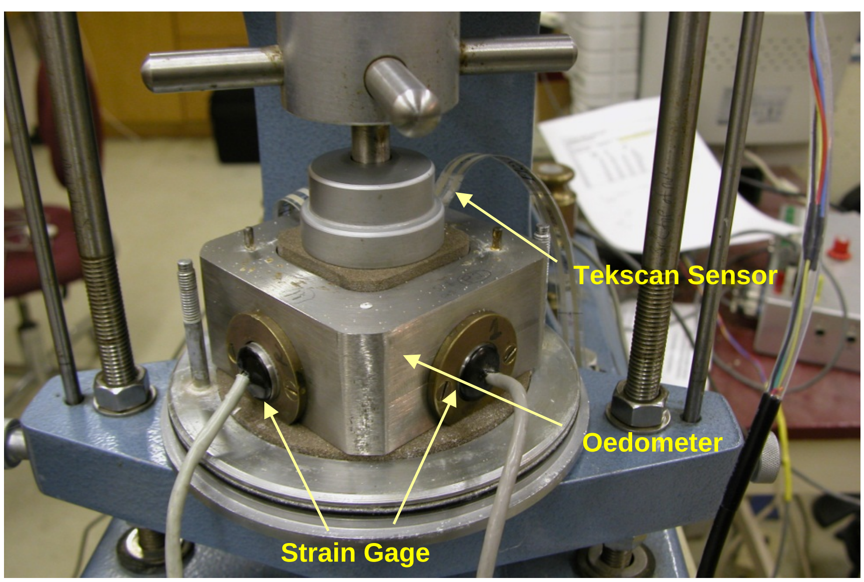
Source: Mayne (4), p. 269.

$m_r = 0.75$

Vertical Stress vs. Horizontal Stress in K_o Test on Ottawa Sand
Test Results from unpublished data, Y.G. Lu and S.G. Paikowsky.



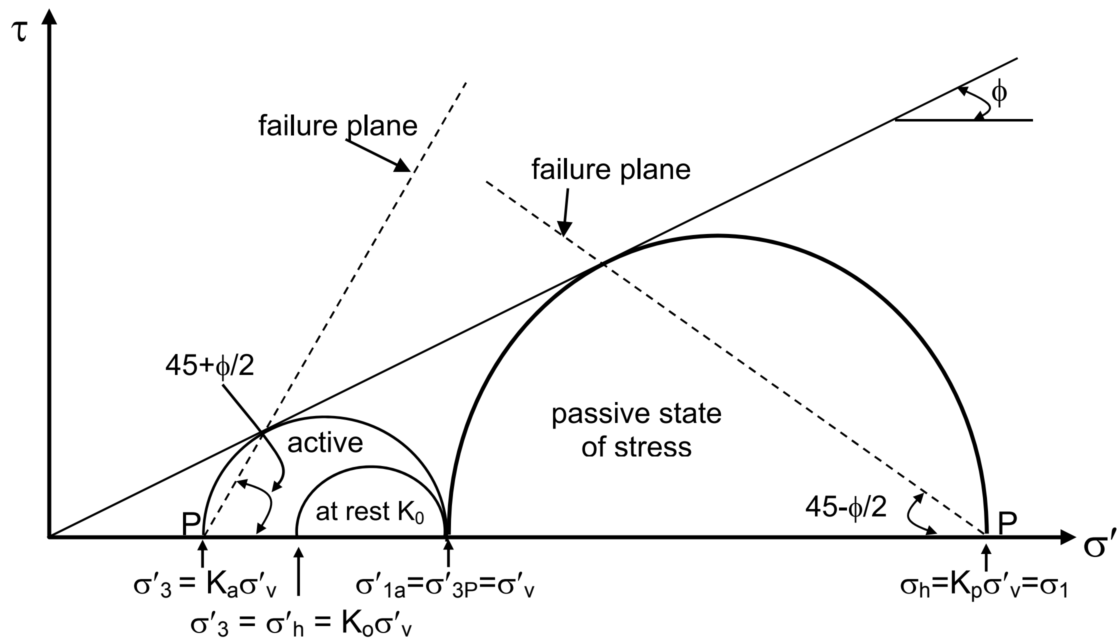
Material: Dry Ottawa Sand
γ = 16.8 kN/m^3
ϕ = 39°



Amherst Test Equipment for K_o Measurement with UML Measuring Device

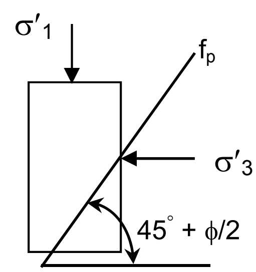
RANKINE ACTIVE & PASSIVE EARTH PRESSURE STATES

- Basic Assumptions
  - (i) The soil is in a state of plastic equilibrium according to Mohr Coulomb → Rigid body translation
  - (ii) There is no friction or adhesion along the wall, principle stresses orientation remain the same as in the soil.
- Frictional Material (Cohesionless)



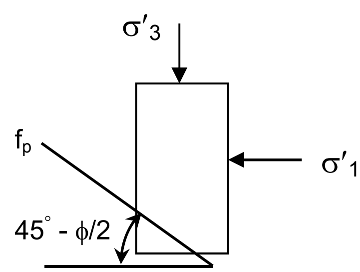
Active:

$$K_a = \tan^2\left(45 - \phi/2\right) = \frac{1-\sin\phi}{1+\sin\phi} \qquad \sigma'_h = \sigma'_v \bullet K_a$$



Passive:

$$K_p = \tan^2\left(45 + \phi/2\right) = \frac{1+\sin\phi}{1-\sin\phi} \qquad \sigma'_h = \sigma'_v \bullet K_p$$

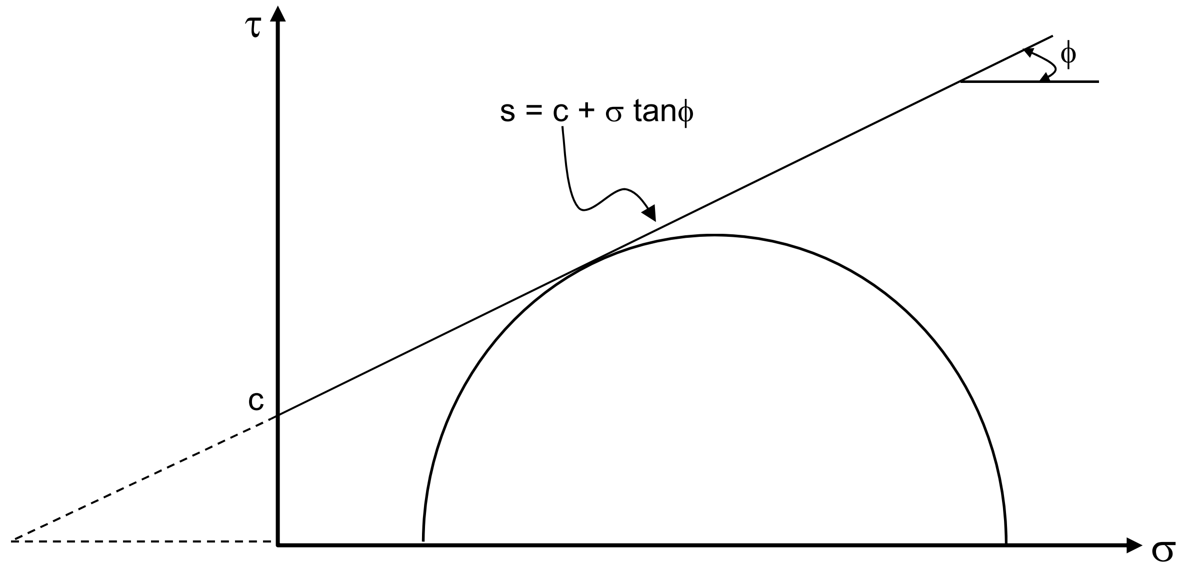


Example:

$\phi = 30^o \rightarrow K_o \approx 0.5$, $K_a = 1/3$, $K_p = 3$

- Material with Cohesion (Active)

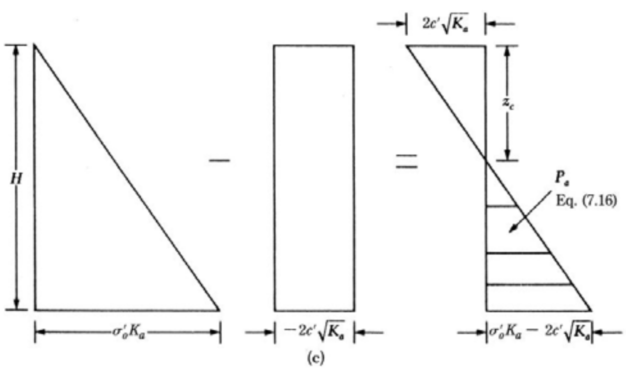
$$\sigma h_a = \sigma_v^1 K_a - 2C\sqrt{K_a} \qquad \sigma h_p = \sigma_v K_p + 2C\sqrt{K_p}$$



$$\sigma_{ha} = \sigma_A = \sigma_v' \bullet \underbrace{\tan^2(45-\phi/2)}_{K_a} - 2C \underbrace{\tan(45-\phi/2)}_{\sqrt{K_a}} = \sigma_v' \bullet K_a - 2c\sqrt{K_a}$$

$K_a = \tan^2(45-\phi/2)$

$Z_c = 2c/\gamma\sqrt{Ka}$



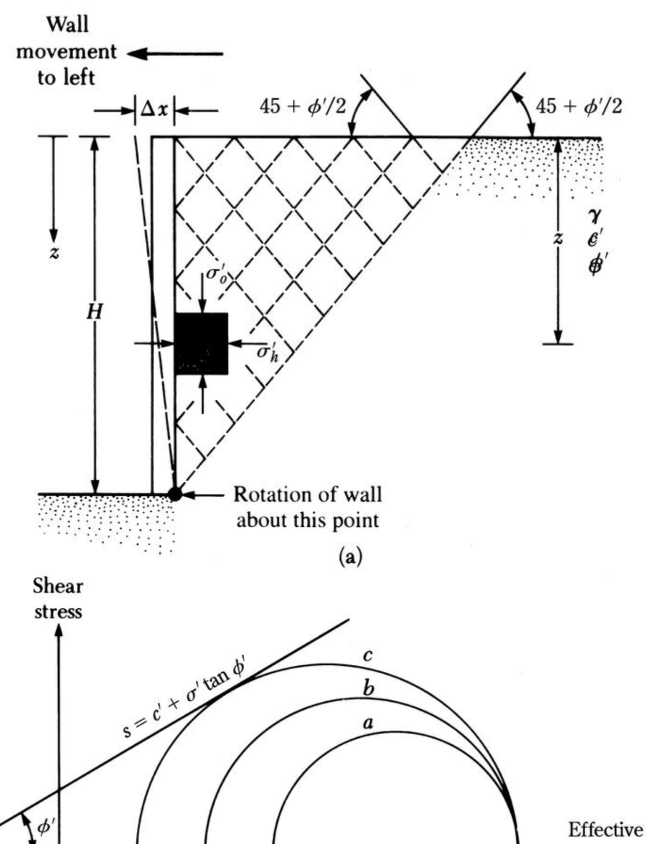
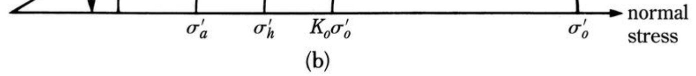


Figure 7.6 Rankine active pressu
(pp. 298-299)



- Material with Cohesion (Passive)

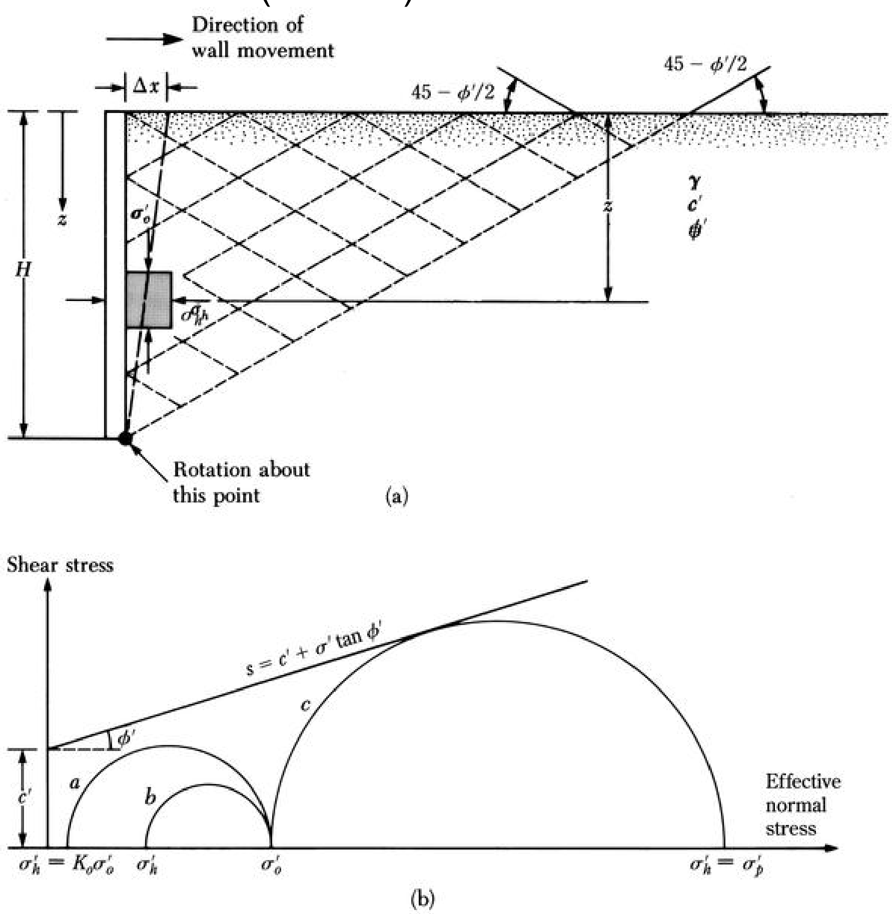
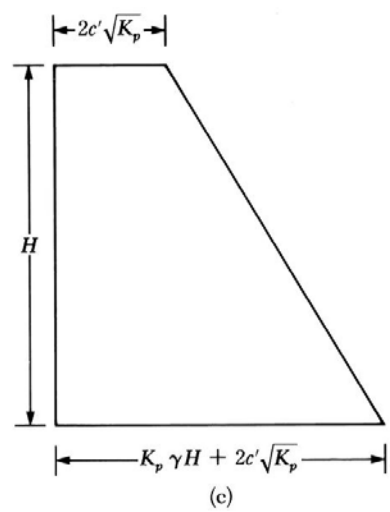


Figure 7.13 Rankine passive pressure (p. 316)

$\sigma_p = \sigma_v' \bullet \tan^2(45^\circ + \phi/2) + 2C \tan(45^\circ + \phi/2) = \sigma_v' \bullet K_p + 2c\sqrt{Kp}$

Figure 7.13 (continued) (p.317)



SLOPING BACKFILL – COHESIONLESS SOIL

Analytical Solution (Rankine) Active And Passive

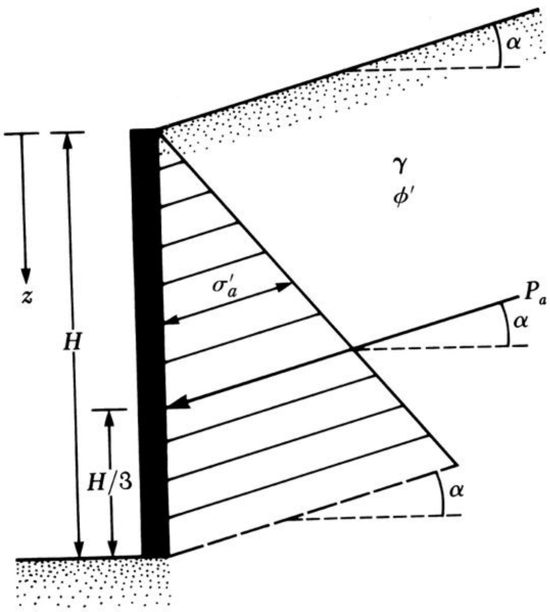


Figure 7.8 Notations for active pressure

$$K_a(upper)\ \&\ K_p(lower) = \frac{\cos\alpha \mp \sqrt{\cos^2\alpha - \cos^2\phi}}{\cos\alpha \pm \sqrt{\cos^2\alpha - \cos^2\phi}}$$

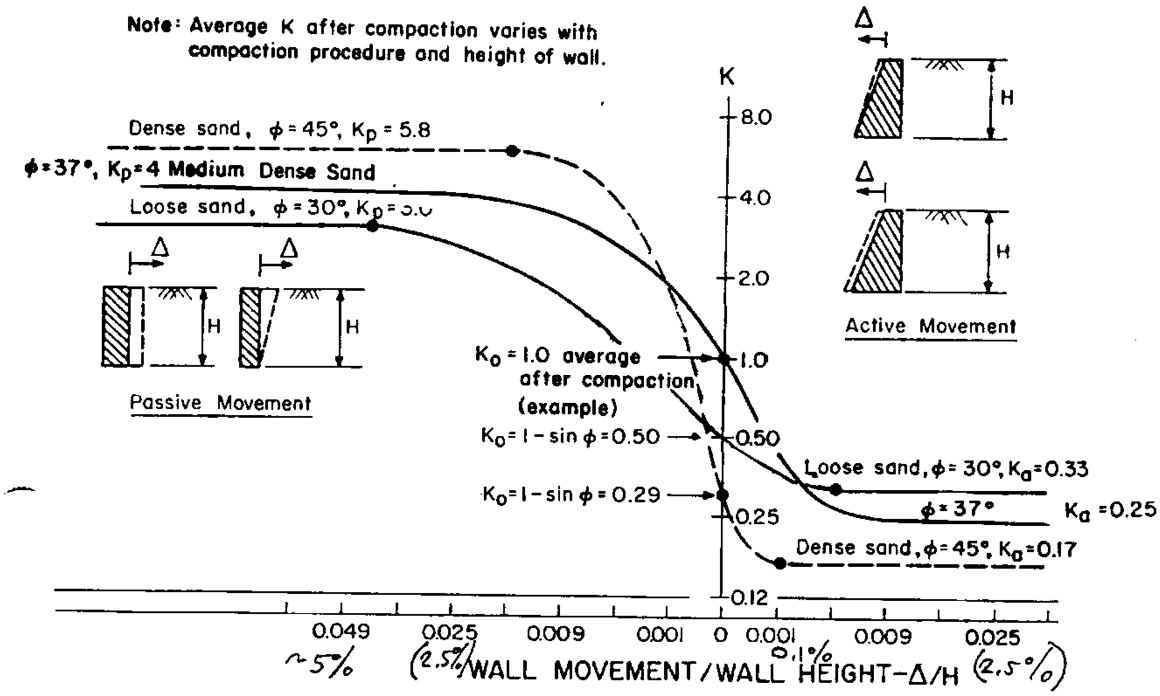
$\rightarrow \alpha = 0 \rightarrow K_a = \tan^2(45 - \frac{\phi}{2})$ $\qquad \rightarrow \alpha = 0 \rightarrow K_P = \tan^2(45 + \frac{\phi}{2})$

For c - ϕ soil

$K_P, K_a =$

$$\frac{1}{\cos^2\phi'}\left\{\begin{array}{l} 2\cos^2\alpha + 2\left(\frac{c'}{\gamma z}\right)\cos\phi'\sin\phi' \\ \pm\sqrt{\left[4\cos^2\alpha\left(\cos^2\alpha - \cos^2\phi'\right) + 4\left(\frac{c'}{\gamma z}\right)^2\cos^2\phi' + 8\left(\frac{c'}{\gamma z}\right)\cos^2\alpha\sin\phi'\cos\phi'\right]} \end{array}\right\} - 1$$

RELATIONS BETWEEN EARTH PRESSURES AND WALL MOVEMENTS



Relationship between wall movement and earth pressure for ideal cases of walls that are "wished" into place.

Reference: Clough, G.W. & Duncan, J.M., (1991). "Earth Pressure", Chapter 6, in *Found* Eng. *Handbok*, 2nd edition, ed. Hsai-Yang Fang, Van Norstrand, Reinhold.

Type of Backfill	Values of Δ/H	
	Active	Passive
Dense Sand	0.001	0.01
Medium Dense Sand	0.002	0.02
Loose Sand	0.004	0.04
Compacted Silt	0.002	0.02
Compacted Lean Clay	0.01	0.055
Compacted Fat Clay	0.01	0.05

Usual Range for Earth Pressure Coefficients (Bowles)

Soil	**K_A**	**K_0**	**K_P**
Cohessionless	0.22 – 0.33	0.4 – 0.6	3 – 14
Cohessive	0.50 – 1.00	0.4 – 0.8	1 – 2

Guide for Lateral Displacements for Developing Active Stresses (Bowles)

Soil and Condition	δ_{hA}
Cohessionless – Dense	0.001 to 0.002H
Cohessionless – Loose	0.002 to 0.004H
Cohesive – Firm	0.01 to 0.02H
Cohesive - Soft	0.02 to 0.05H

DIA – DUAL INTERFACIAL APPARATUS

Reference: Paikowsky,S.G., Player,C.M., and Connors,P.J. (1995). "A Dual Interface Apparatus for Testing Unrestricted Friction of Soil Along Solid Surfaces", *Geotechnical Testing Journal,* June 1995, ASTM, Philadelphia, PA.

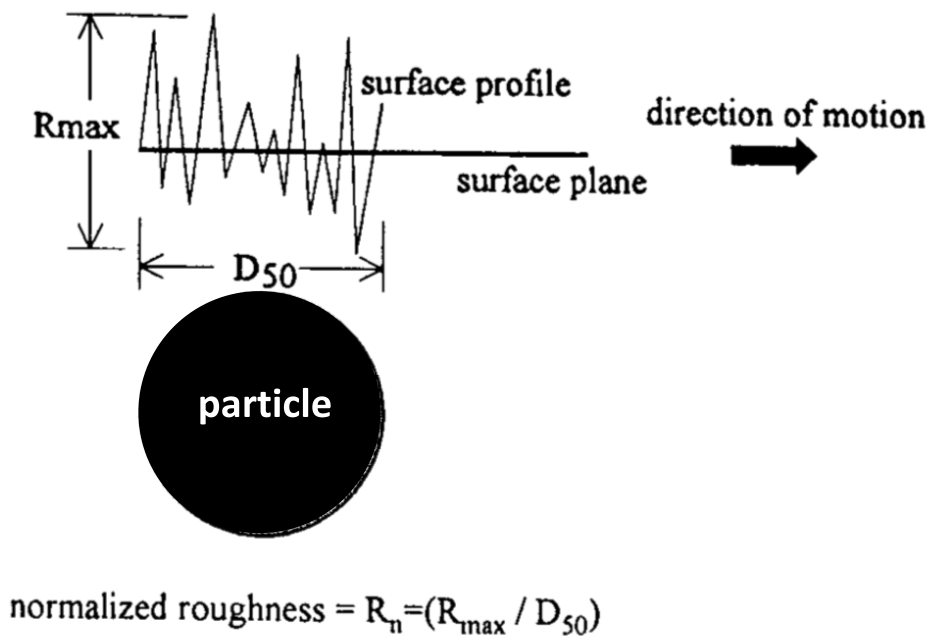


Figure 1 – Solid surface topography and its representation through normalized roughness.

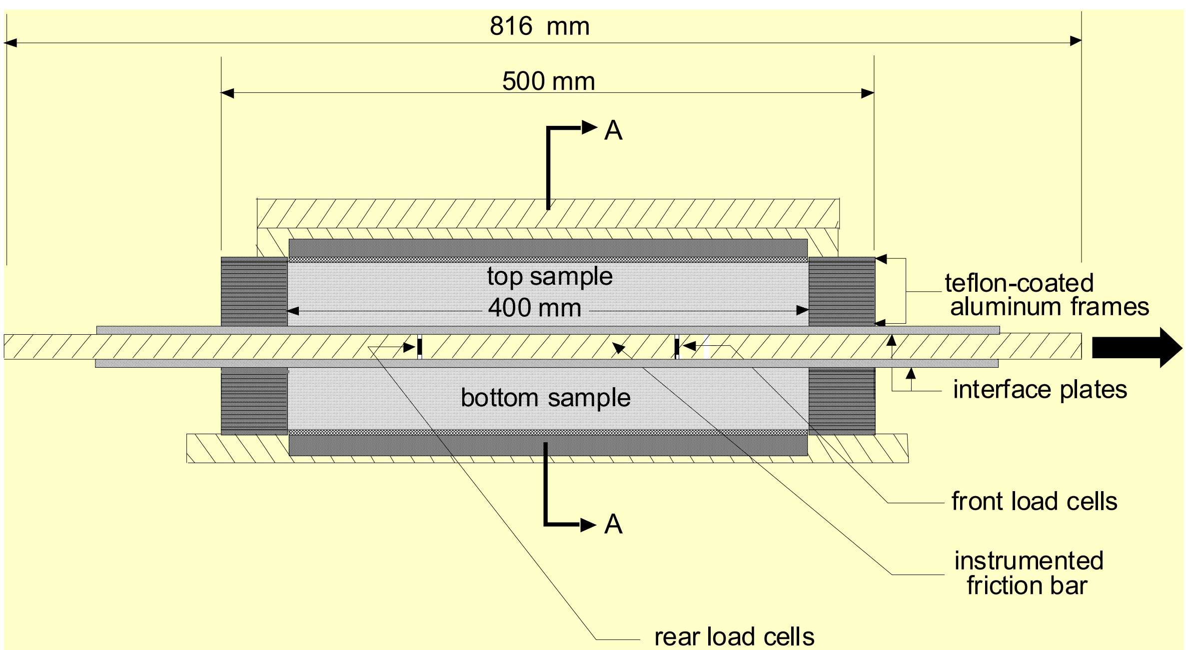


Figure 6– Longitudinal section (B-B) of shear box and external reaction frame.

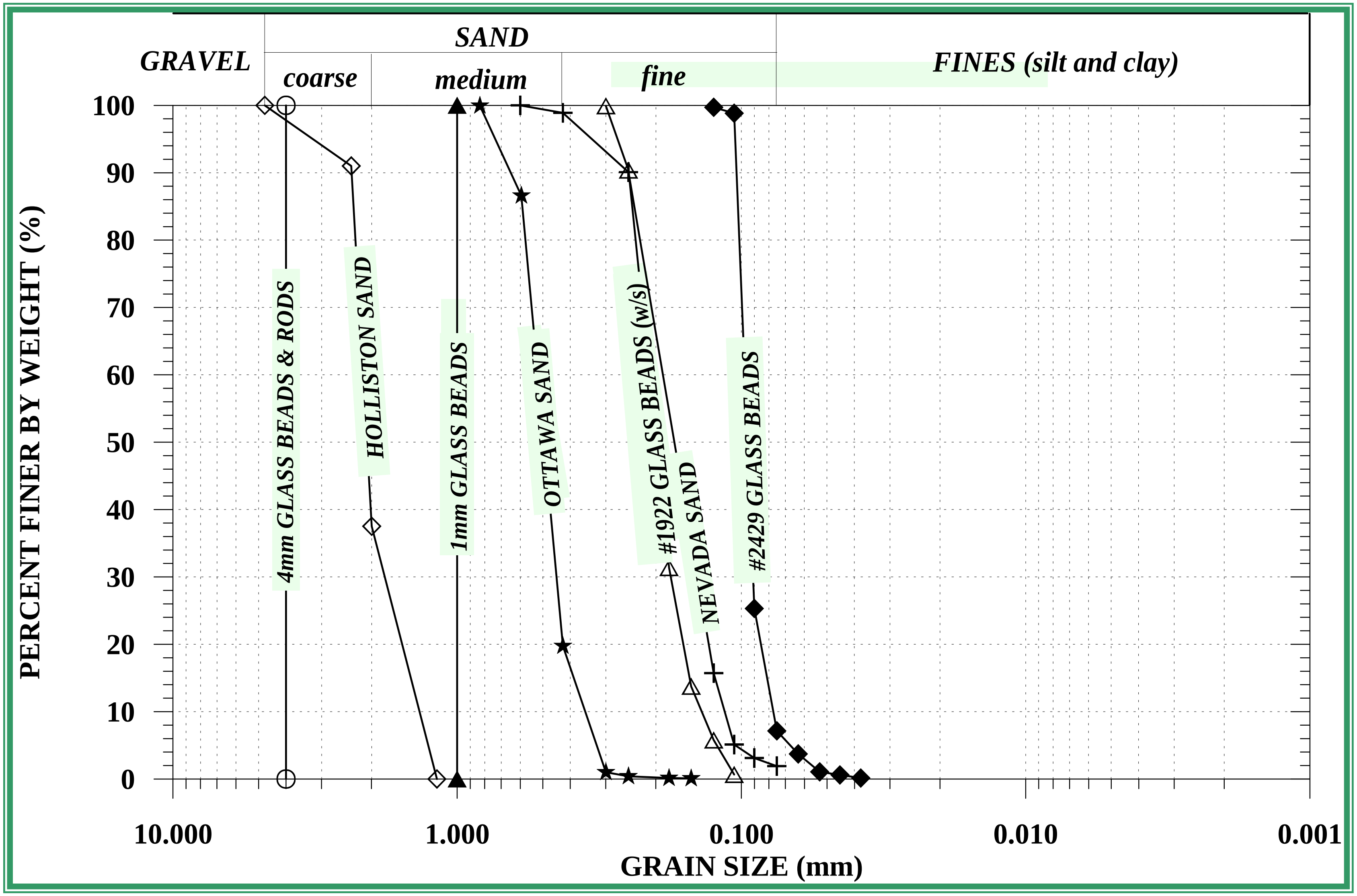


Figure 14 – Grain size distribution of tested granular materials.

Reference: Paikowsky,S.G., Player,C.M., and Connors,P.J. (1995). "A Dual Interface Apparatus for Testing Unrestricted Friction of Soil Along Solid Surfaces", *Geotechnical Testing Journal,* June 1995, ASTM, Philadelphia, PA.

Reference: Paikowsky,S.G., Player,C.M., and Connors,P.J. (1995). “A Dual Interface Apparatus for Testing Unrestricted Friction of Soil Along Solid Surfaces”, *Geotechnical Testing Journal,* June 1995, ASTM, Philadelphia, PA.

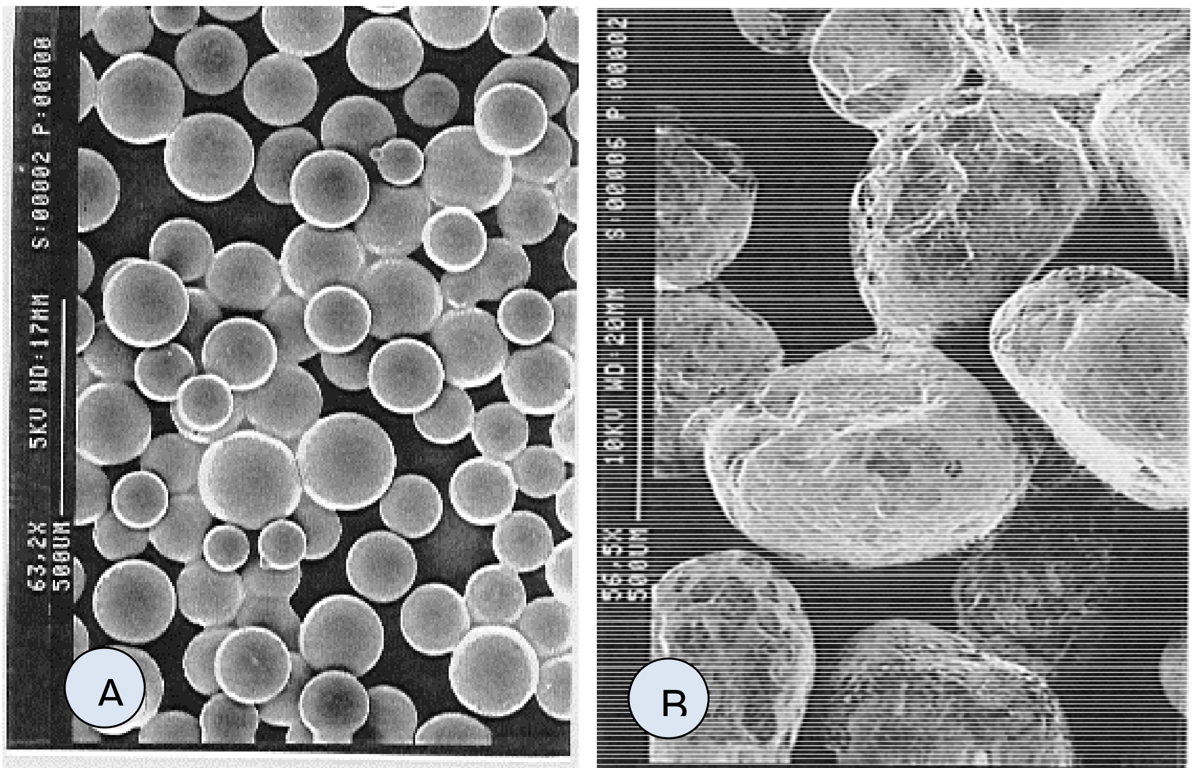


Figure 15 – SEM (scanning electron microscope) images of: (a) washed and sorted No. 1922 glass beads at a magnification of X63.2, and (b) Ottawa sand at a magnification of X56.5.

Reference: Paikowsky,S.G., Player,C.M., and Connors,P.J. (1995). "A Dual Interface Apparatus for Testing Unrestricted Friction of Soil Along Solid Surfaces", *Geotechnical Testing Journal,* June 1995, ASTM, Philadelphia, PA.

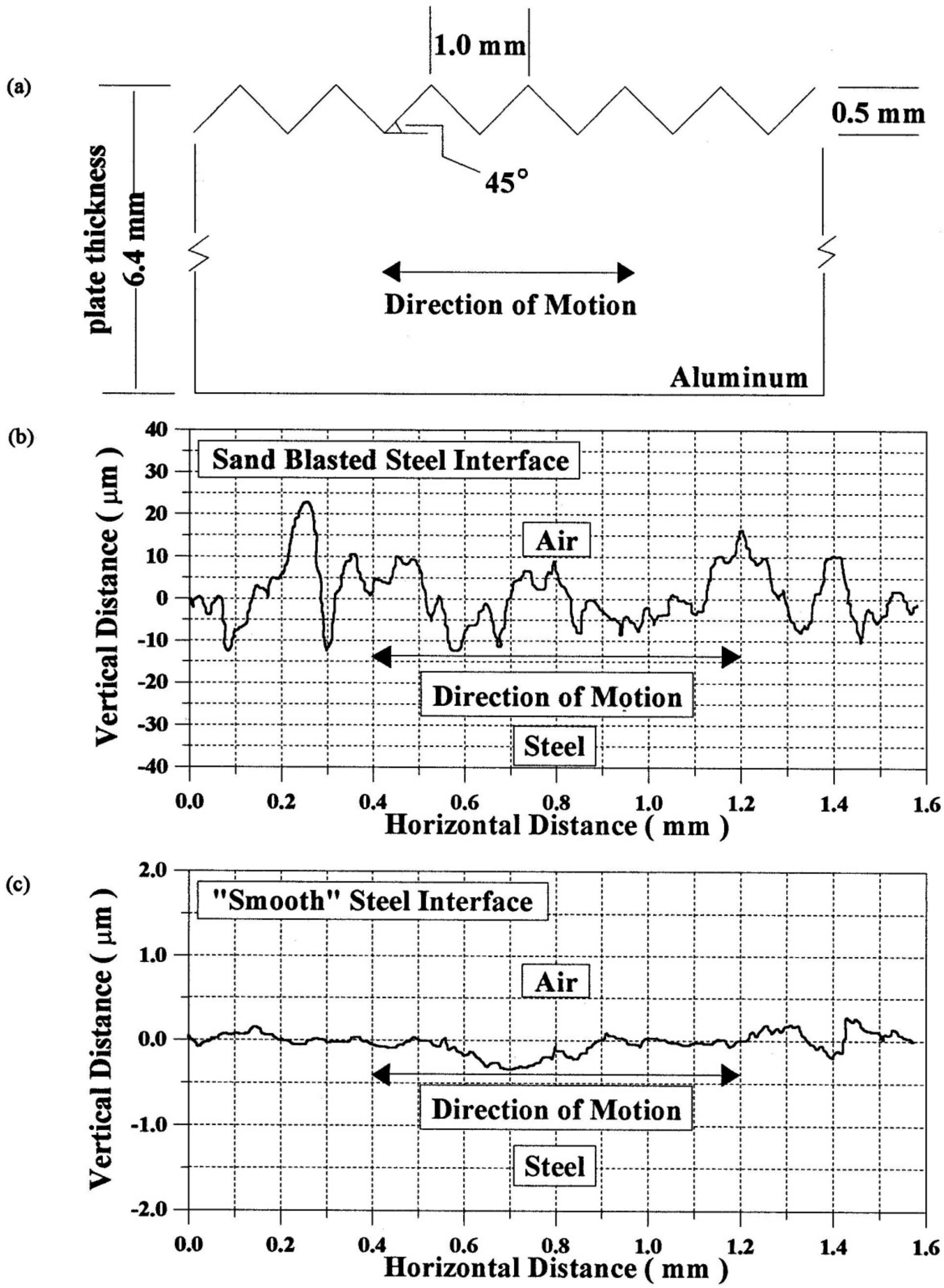


Figure 16– Solid surface topography: (a) "rough", (b) sand blasted, and (c) "smooth".

Reference: Paikowsky,S.G., Player,C.M., and Connors,P.J. (1995). "A Dual Interface Apparatus for Testing Unrestricted Friction of Soil Along Solid Surfaces", *Geotechnical Testing Journal,* June 1995, ASTM, Philadelphia, PA.

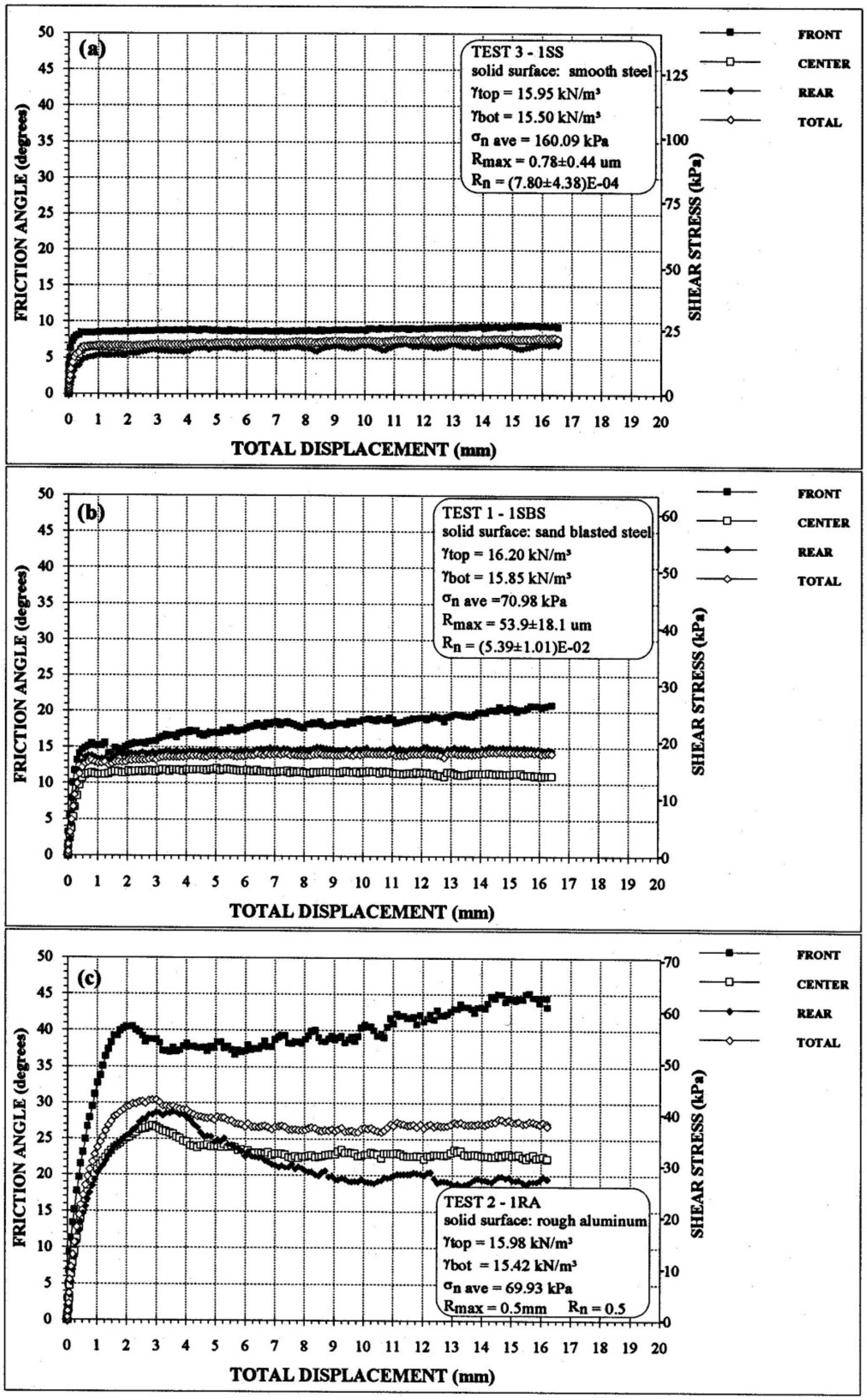


Figure 17– Distribution of friction angles and stresses along an interface of 1-mm glass beads and various solid surfaces: (a) smooth, (b) intermediate, and (c) rough.

Reference: Paikowsky,S.G., Player,C.M., and Connors,P.J. (1995). "A Dual Interface Apparatus for Testing Unrestricted Friction of Soil Along Solid Surfaces", *Geotechnical Testing Journal,* June 1995, ASTM, Philadelphia, PA.

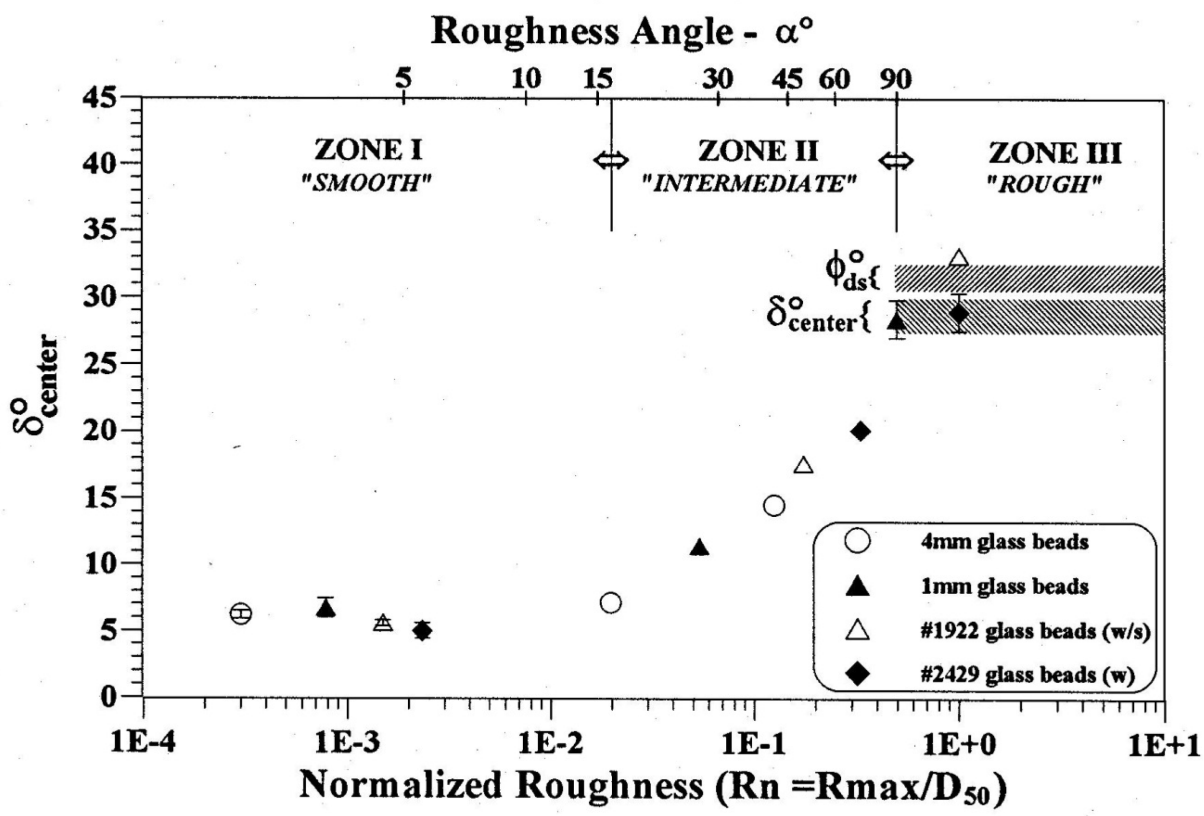


Figure 25– Interfacial characterization according to zones identifies through the relations existing between average interfacial friction angles (measured along the central section) of glass beads and normalized roughness.

Reference: Paikowsky,S.G., Player,C.M., and Connors,P.J. (1995). "A Dual Interface Apparatus for Testing Unrestricted Friction of Soil Along Solid Surfaces", *Geotechnical Testing Journal,* June 1995, ASTM, Philadelphia, PA.

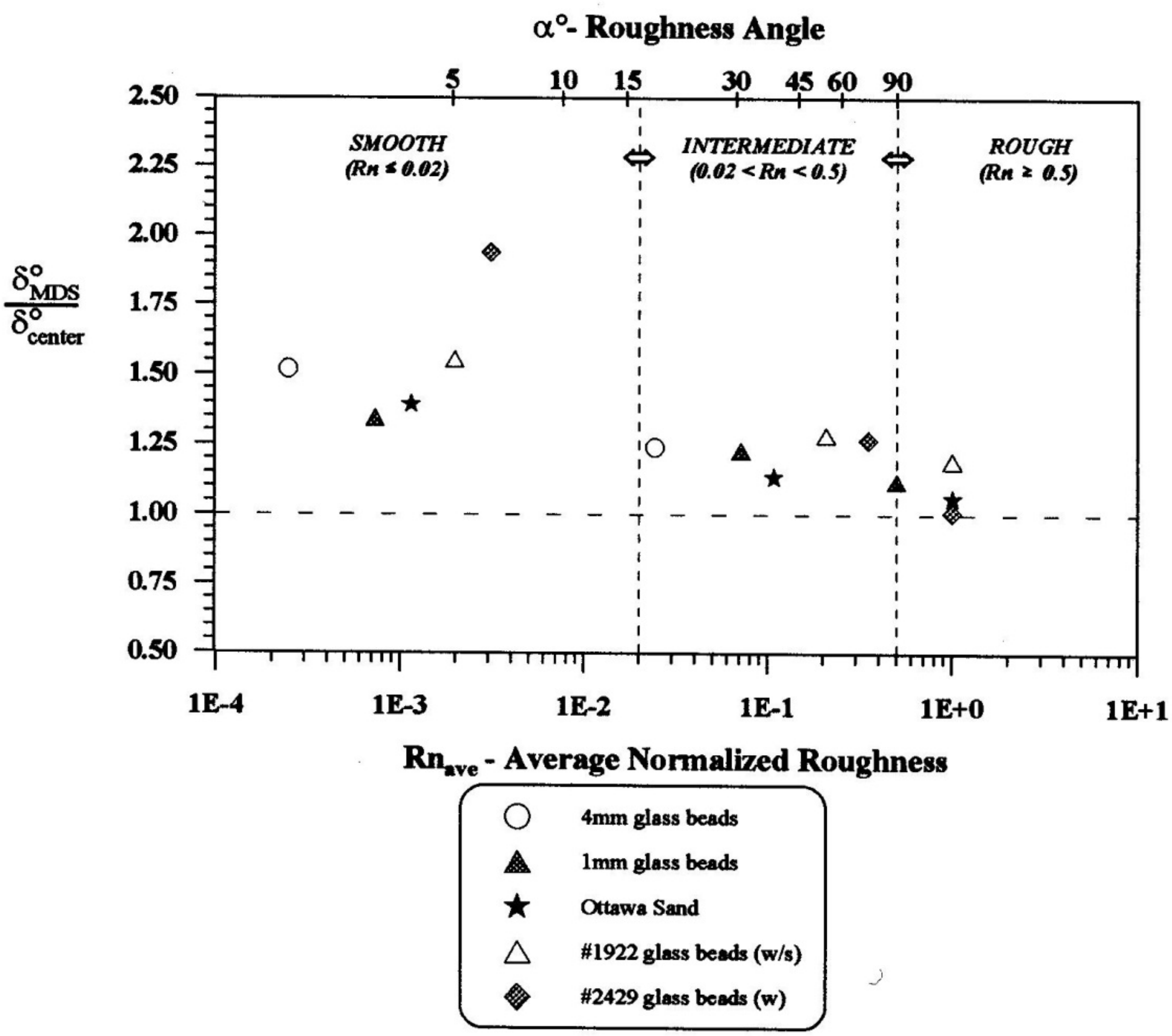


Figure 26– The ratio of modified direct shear box to central section interfacial friction angles versus average normalized roughness.

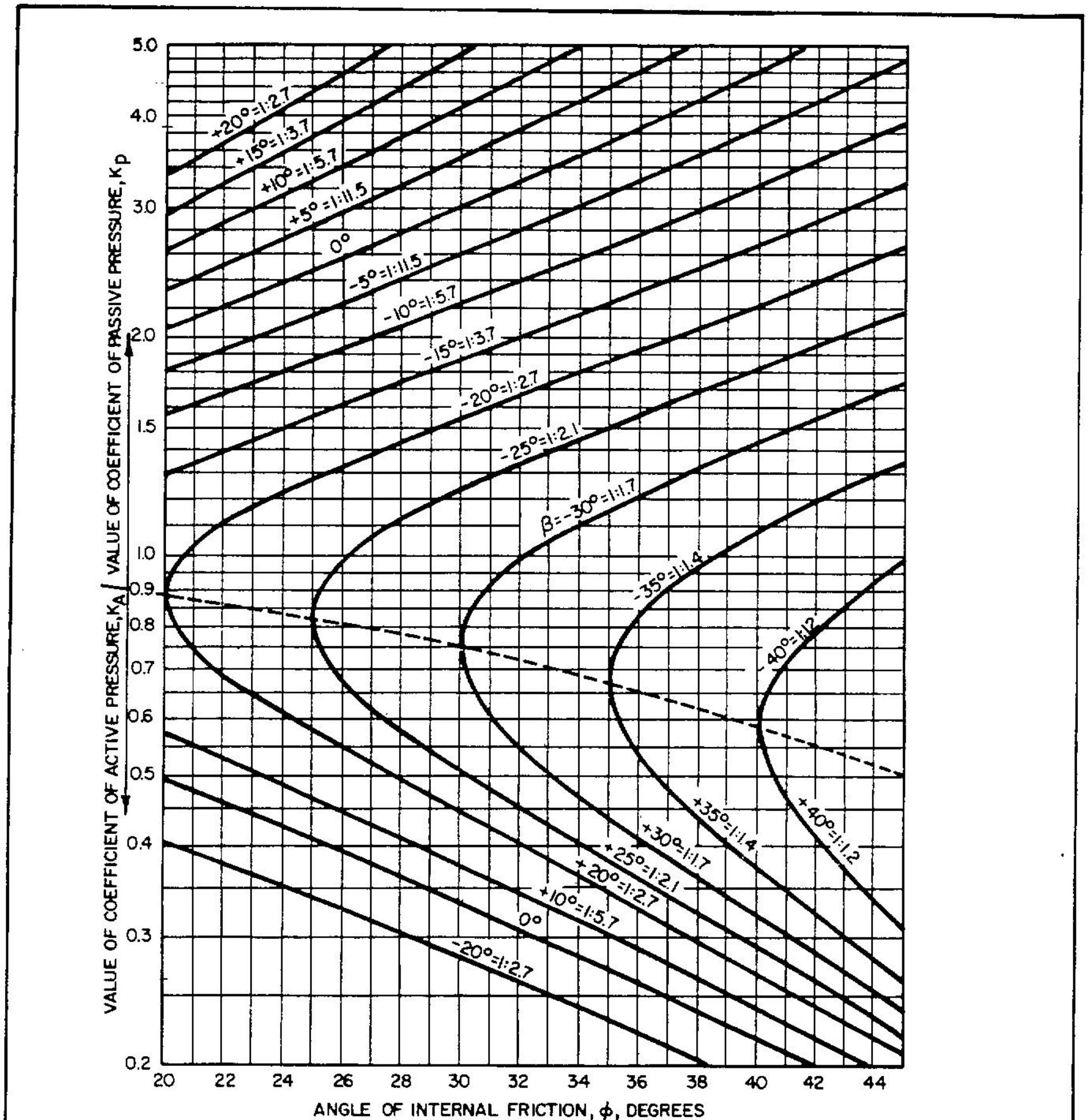
Reference: Naval Facilities Engineering Command. (1986). *Foundations & Earth Structures Design Manual 7.02, Revalidated by Change 1,* September, TRC Environmental Corp., Washington, D.C.

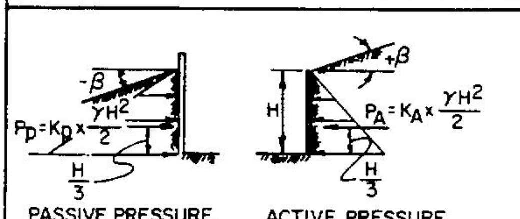
TABLE 1

Ultimate Friction Factors and Adhesion for Dissimilar Materials

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	0.30 to 0.35	17 to 19
(Masonry on foundation materials has same friction factors.)		
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 mixture, 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 (psf)
Very soft cohesive soil (0 - 250 psf)	0 - 250
Soft cohesive soil (250 - 500 psf)	250 - 500
Medium stiff cohesive soil (500 - 1000 psf)	500 - 750
Stiff cohesive soil (1000 - 2000 psf)	750 - 950
Very stiff cohesive soil (2000 - 4000 psf)	950 - 1,300





$$K_A = \left[\frac{\cos\phi}{1+\sqrt{\sin\phi(\sin\phi-\cos\phi\tan\beta)}}\right]^2; \quad K_P = \left[\frac{\cos\phi}{1-\sqrt{\sin\phi(\sin\phi+\cos\phi\tan\beta)}}\right]^2$$

K_A & K_P = COEFFICIENTS FOR COULOMB'S EQUATION FOR ACTIVE AND PASSIVE EARTH PRESSURE (NO SHEAR STRESS ON VERTICAL PLANES).

P_A = ACTIVE RESULTANT

P_P = PASSIVE RESULTANT

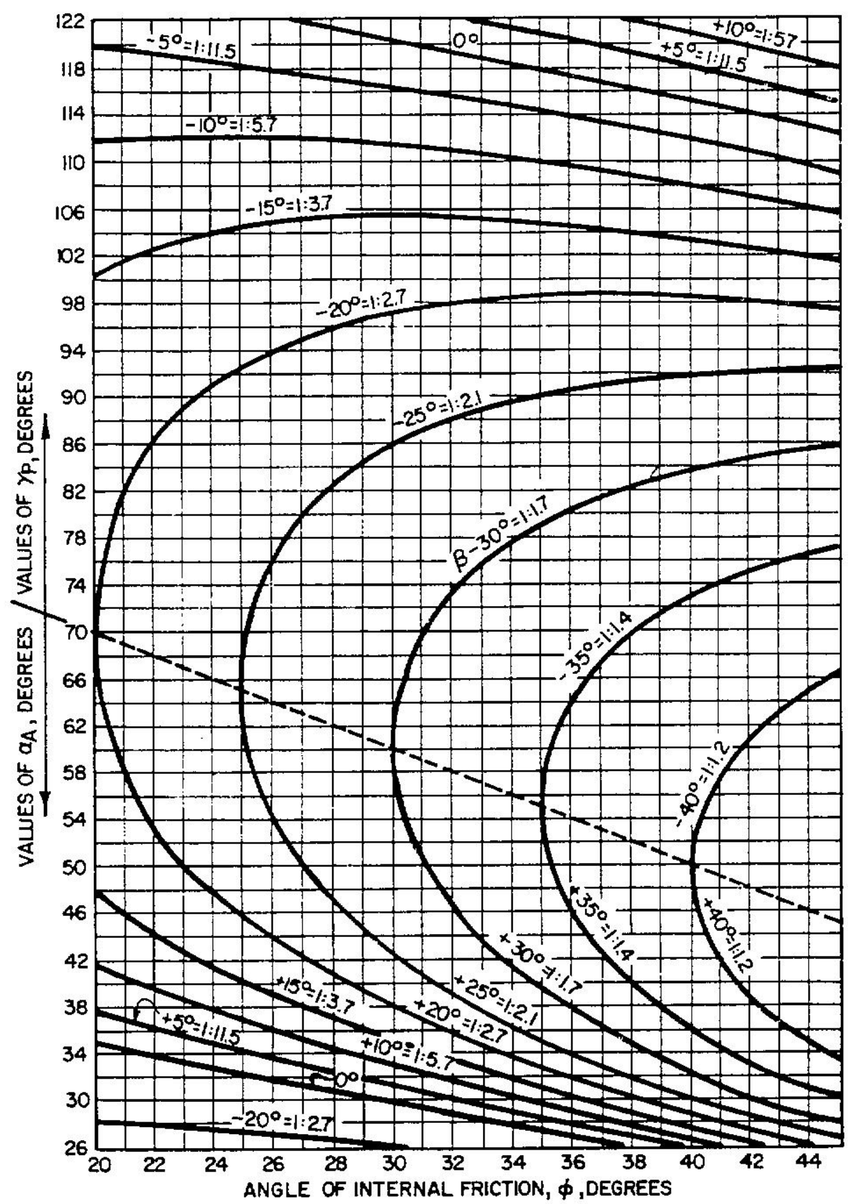
γ = UNIT WEIGHT OF SOIL

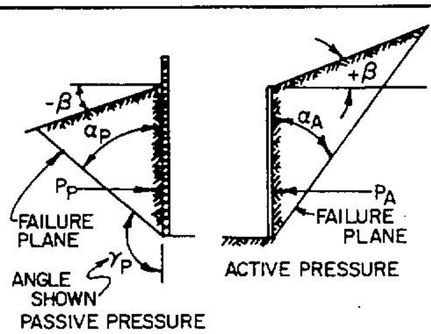
H = HEIGHT OF WALL

ϕ = ANGLE OF INTERNAL FRICTION

β = SLOPE ANGLE

FIGURE 3
Active and Passive Coefficients, Sloping Backfill
(Granular Soils)





$$\cot \alpha_A = \tan\phi + \sqrt{1+\tan^2\phi - \frac{\tan\beta}{\sin\phi\cos\phi}}$$

$$\cot \alpha_P = -\tan\phi + \sqrt{1+\tan^2\phi - \frac{\tan\beta}{\sin\phi\cos\phi}}$$

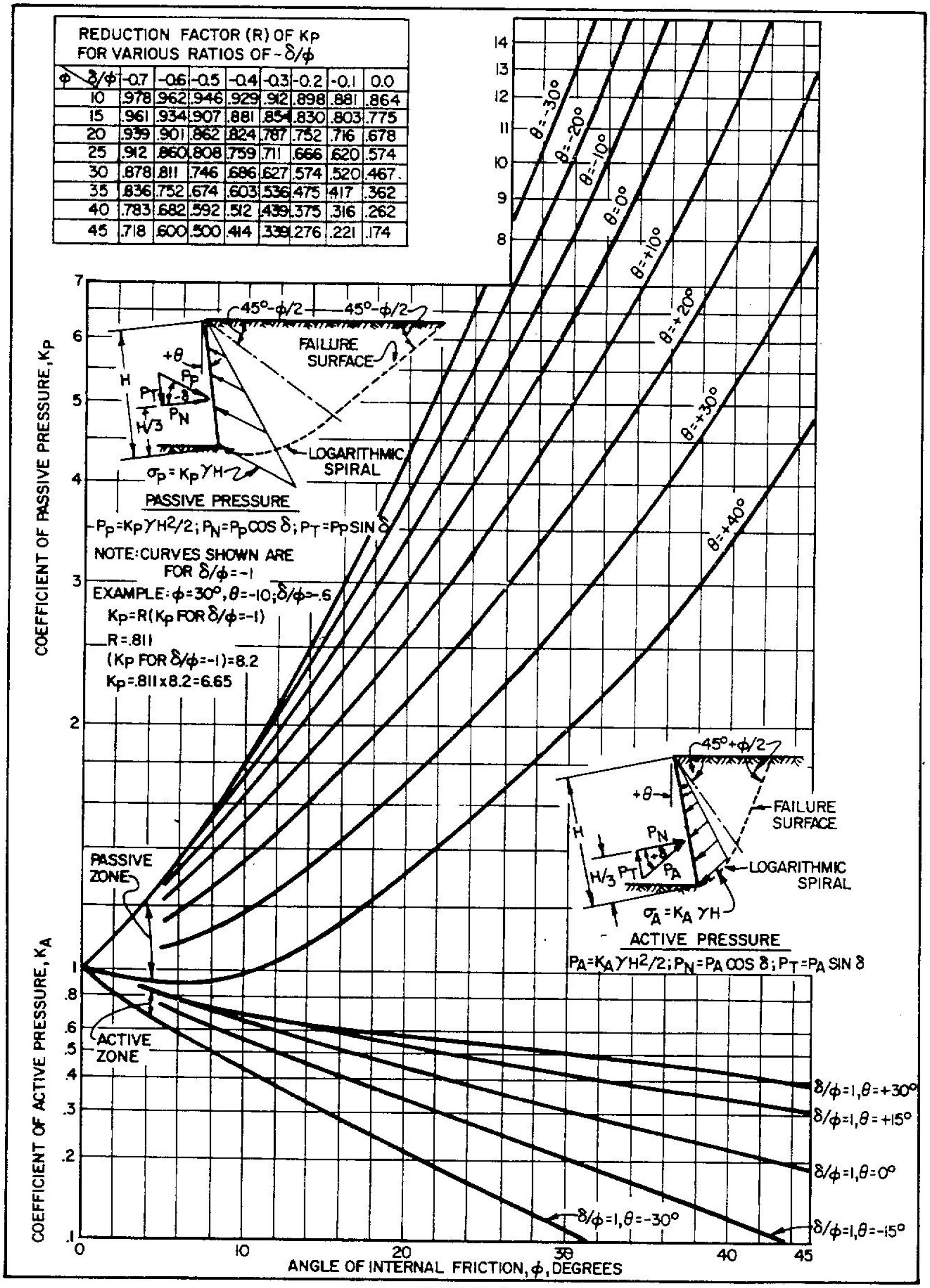
α_A & α_P = ANGLE BETWEEN CRITICAL FAILURE PLANE AND VERTICAL

ϕ = ANGLE OF INTERNAL FRICTION

β = SLOPE ANGLE

THE ANGLES SHOWN CORRESPOND TO THE COEFFICIENTS OF ACTIVE AND PASSIVE PRESSURE GIVEN IN FIGURE 3.

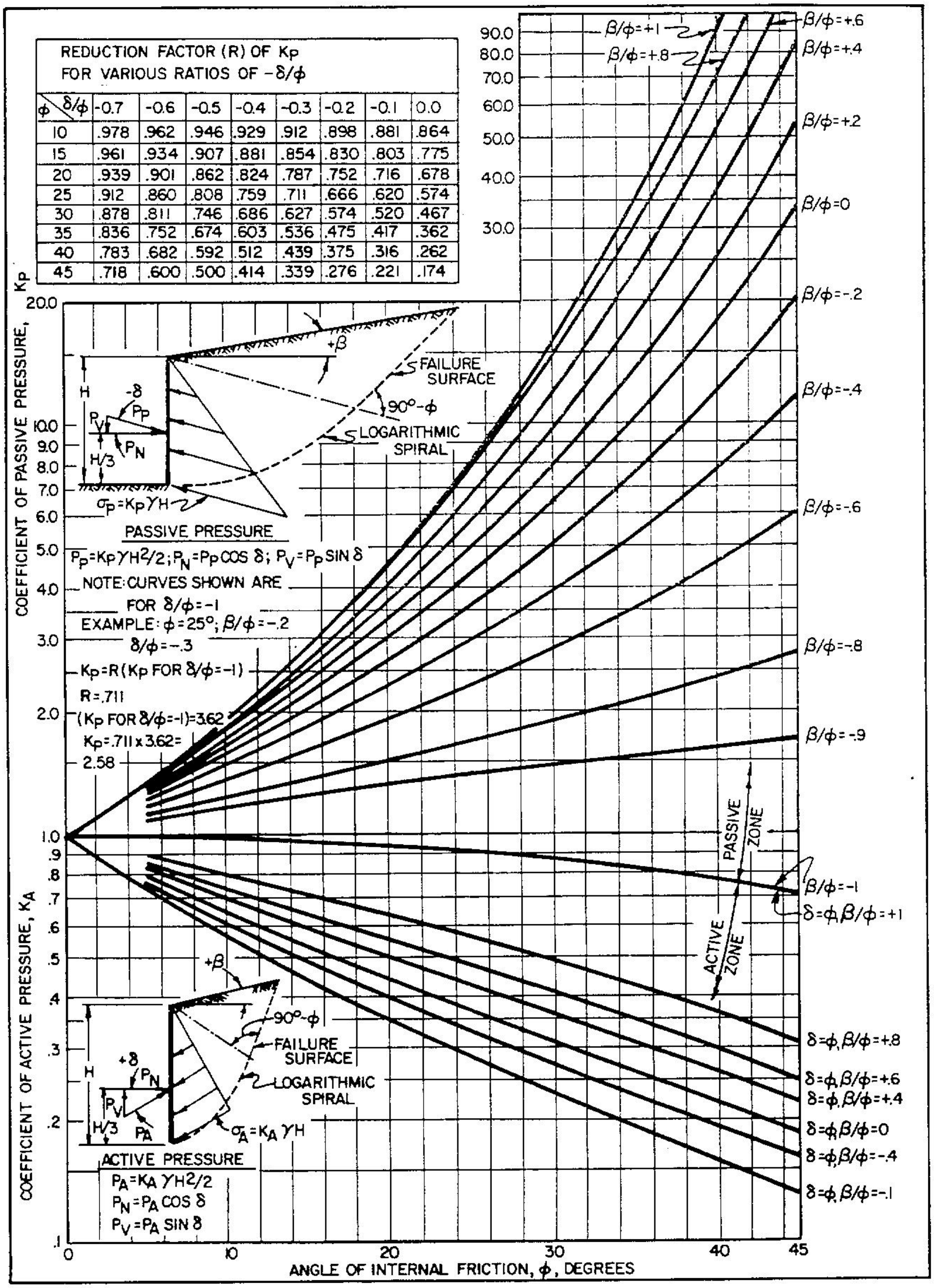
FIGURE 4
Position of Failure Surface for Active and Passive Wedges
(Granular Soils)



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

FIGURE 5
Active and Passive Coefficients with Wall Friction
(Sloping Wall)



REDUCTION FACTOR (R) OF Kp FOR VARIOUS RATIOS OF −δ/φ

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

FIGURE 6
Active and Passive Coefficients with Wall Friction
(Sloping Backfill)

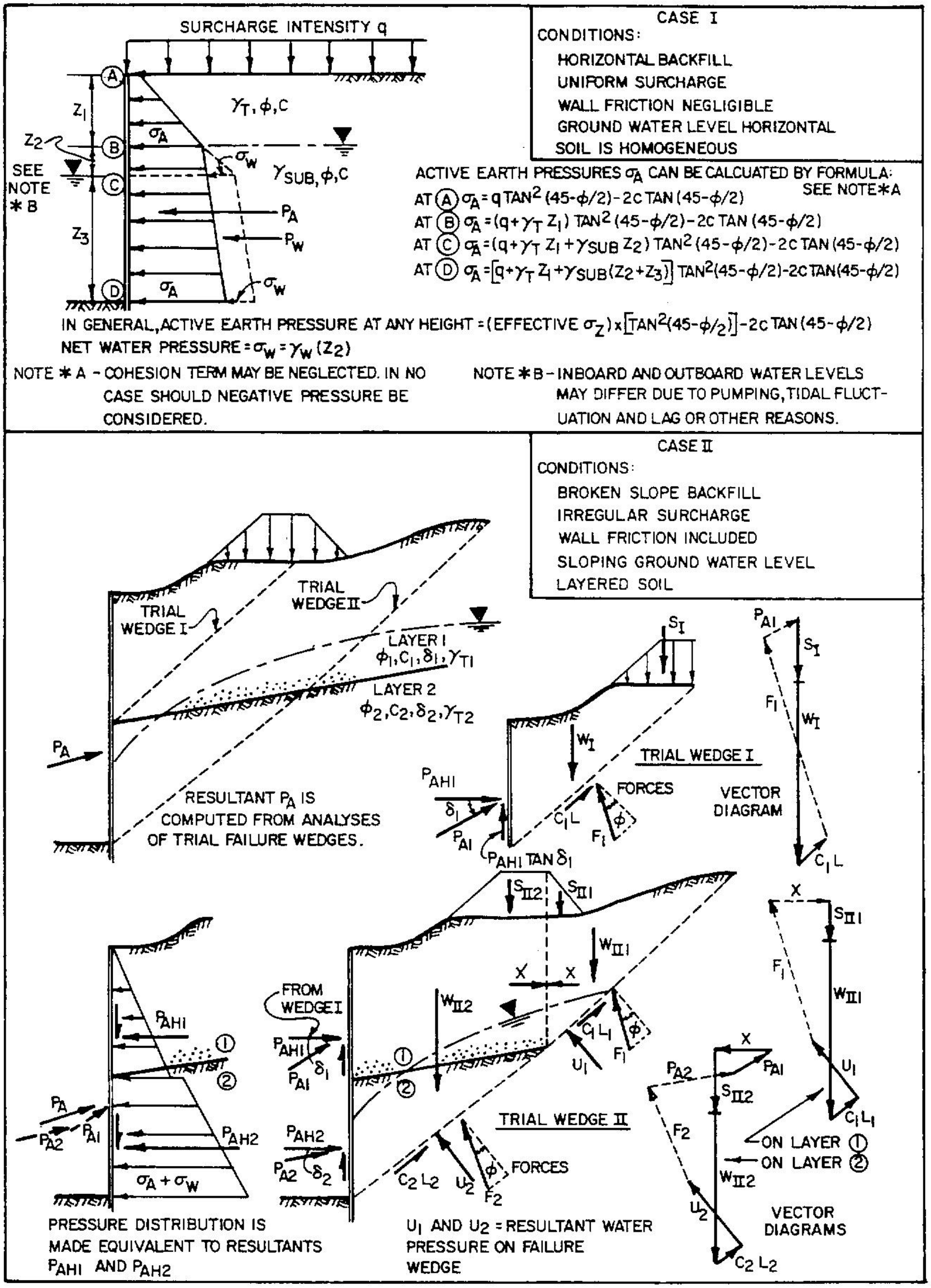
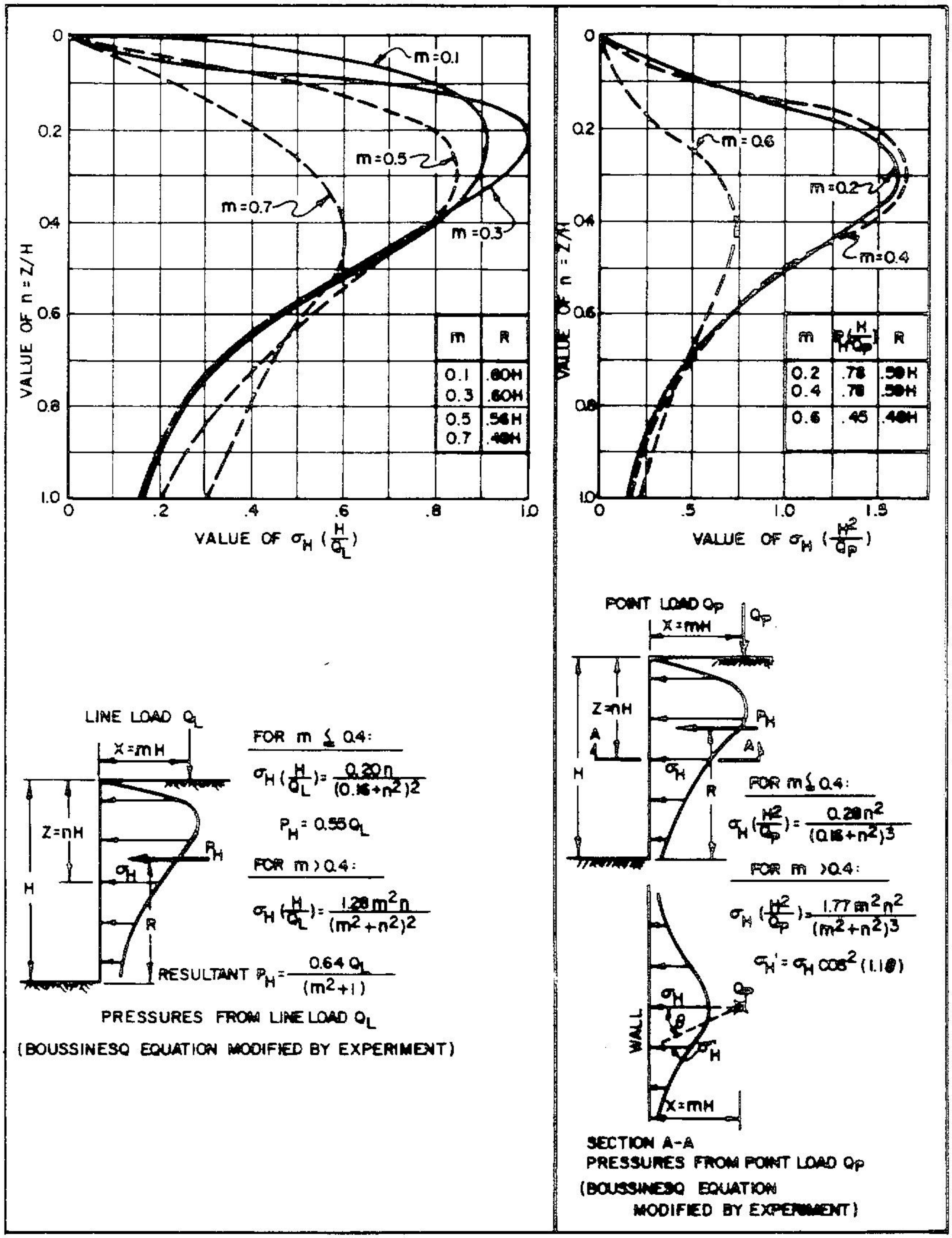


FIGURE 7

Computation of General Active Pressures



m	R
0.1	.60H
0.3	.60H
0.5	.56H
0.7	.48H

m	$P_H\left(\frac{H}{Q_p}\right)$	R
0.2	.78	.59H
0.4	.78	.59H
0.6	.45	.48H

For line load, $m \le 0.4$: $\sigma_H\left(\frac{H}{Q_L}\right) = \frac{0.20\,n}{(0.16+n^2)^2}$, $P_H = 0.55\,Q_L$

For $m > 0.4$: $\sigma_H\left(\frac{H}{Q_L}\right) = \frac{1.28\,m^2 n}{(m^2+n^2)^2}$, resultant $P_H = \frac{0.64\,Q_L}{(m^2+1)}$

For point load, $m \le 0.4$: $\sigma_H\left(\frac{H^2}{Q_p}\right) = \frac{0.28\,n^2}{(0.16+n^2)^3}$

For $m > 0.4$: $\sigma_H\left(\frac{H^2}{Q_p}\right) = \frac{1.77\,m^2 n^2}{(m^2+n^2)^3}$

$\sigma_H' = \sigma_H \cos^2(1.1\theta)$

FIGURE 11
Horizontal Pressures on Rigid Wall from Surface Load

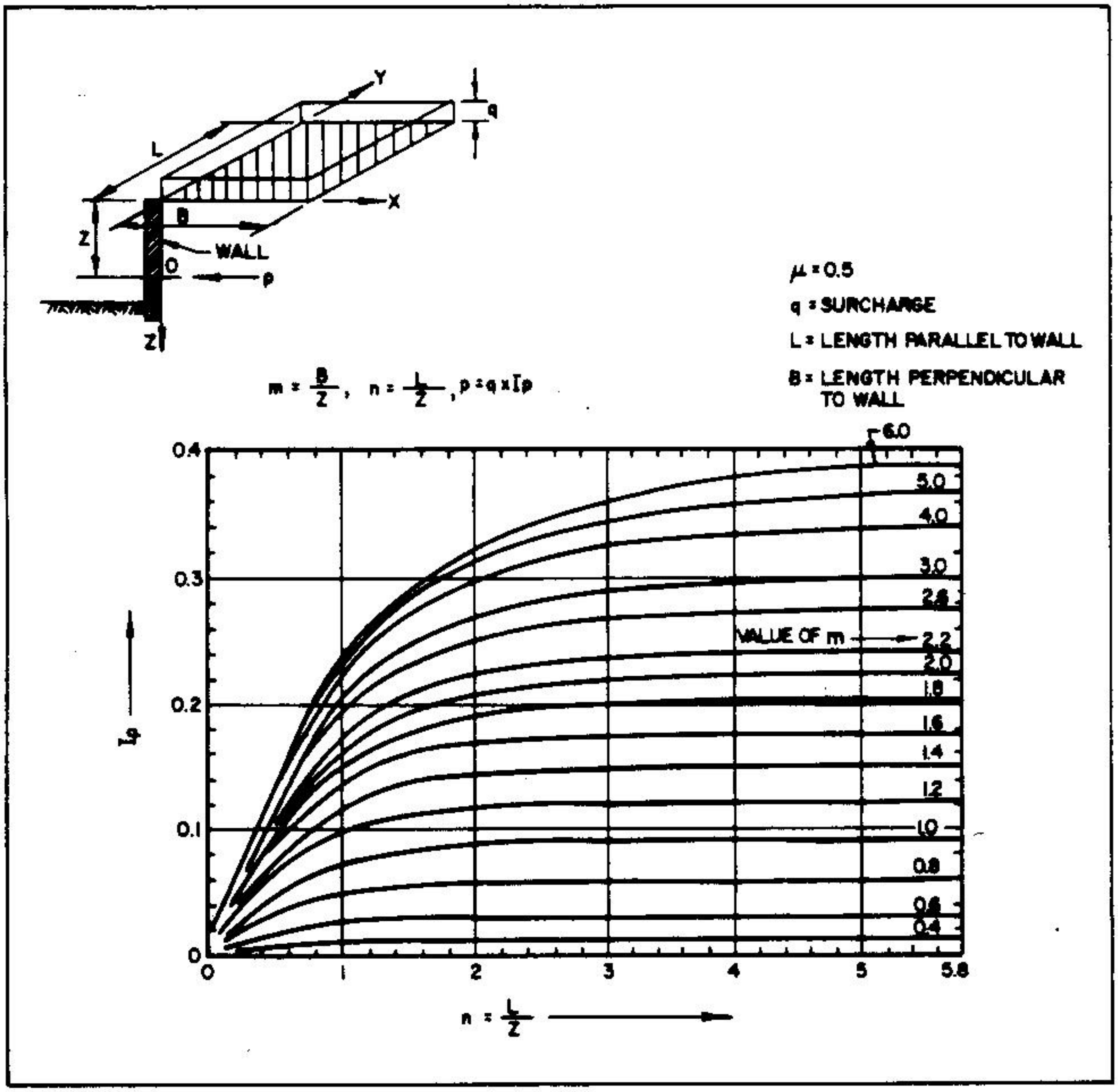
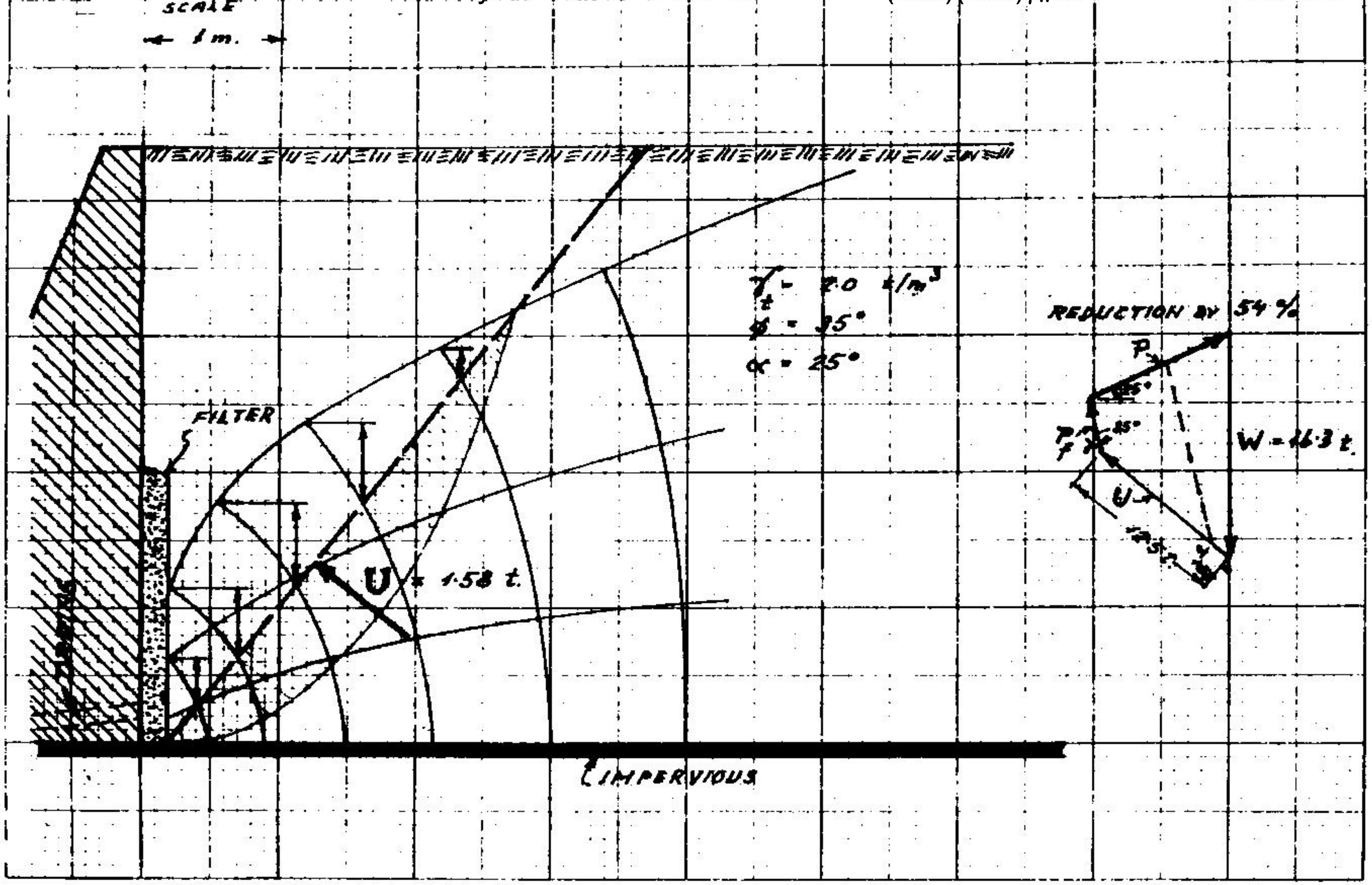


FIGURE 12
Lateral Pressure on an Unyielding Wall due to
Uniform Rectangular Surface Load

EFFECT OF FILTER ON LATERAL PRESSURE

No Filter   Full Hydrostatic Pressure $U = \frac{1}{2}(4 \times 4)(5 \times 7)\gamma_w = 12.5t$



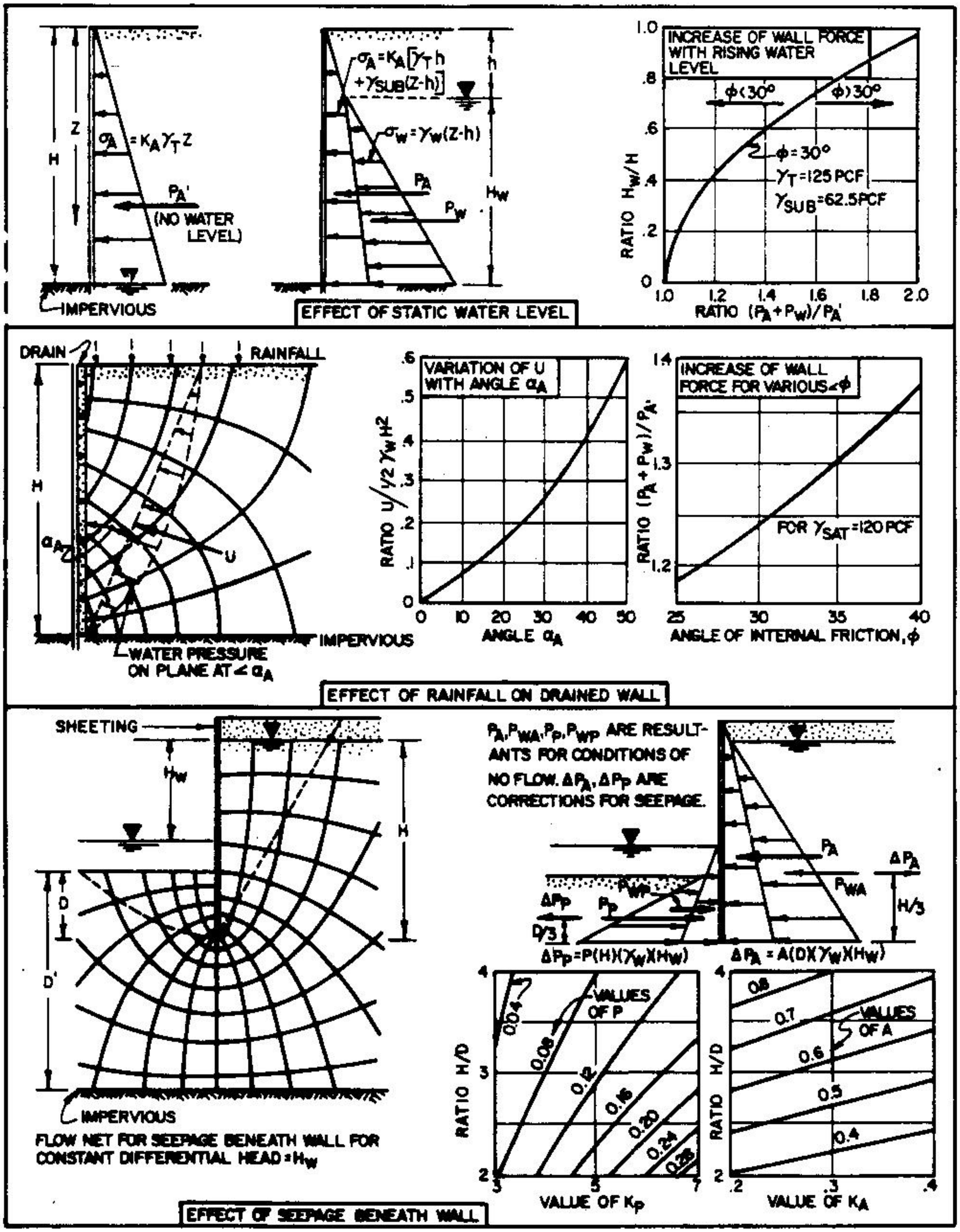


FIGURE 10
Effect of Groundwater Conditions on Wall Pressures

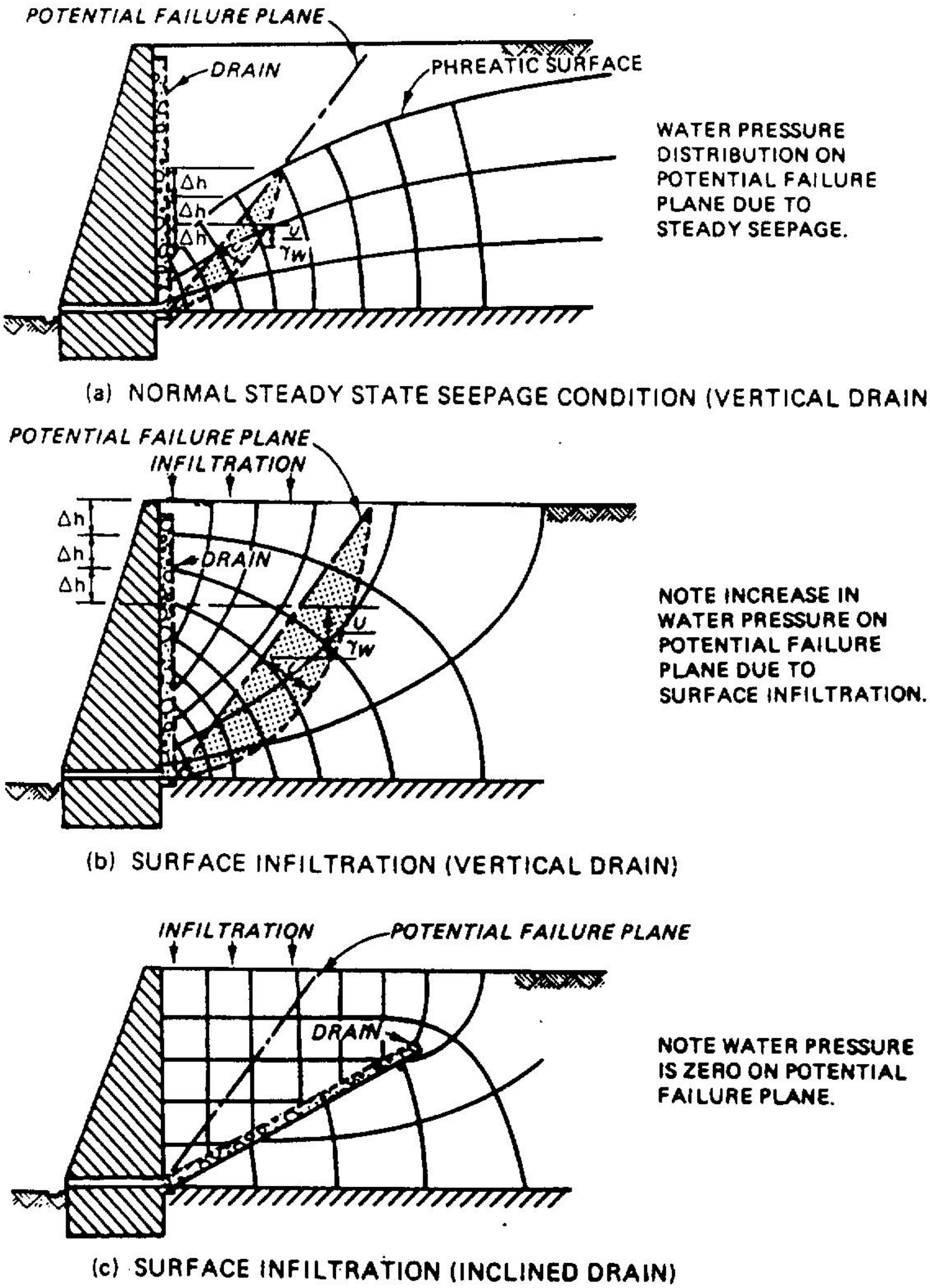


Figure 3.4. Effect of drain location on excess hydrostatic pressures on the failure plane. (From Geotechnical Control Office, 1982)

Reference: Barker, R.M., Duncan, J.M. Rojiani, K.B., Ooi, P.S.K., Tan, C.K., and Kim S.G. (1991). *NCHRP Report 343: Manuals for the Design of Bridge Foundations,* TRB, Washington, DC, December.

EARTH PRESSURE DUE TO COMPACTION

The compaction induces load, unload and reload conditions. The lateral stresses will be therefore higher than those under K_O only. These stresses are usually being referred to as "residual earth pressures".

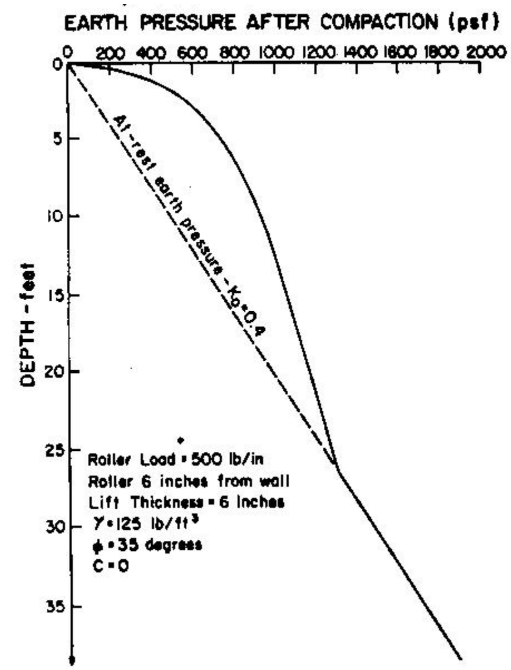


Fig. 6.16 Residual earth pressure after compaction of backfill behind an unyielding wall.

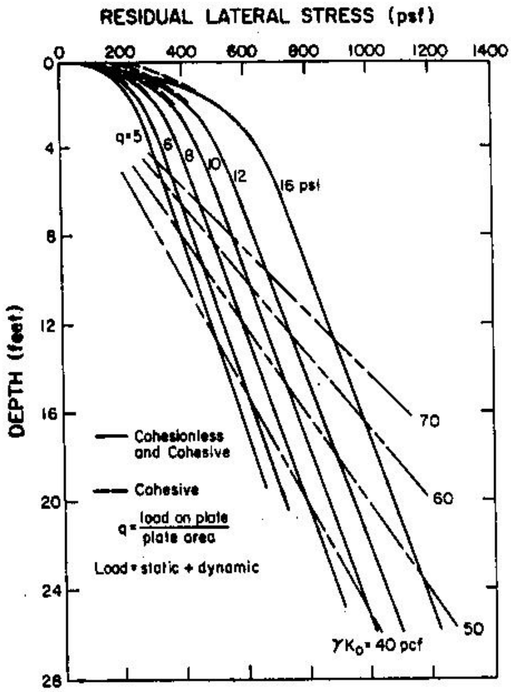


Fig. 6.18 Earth pressures due to compaction by vibratory plates.

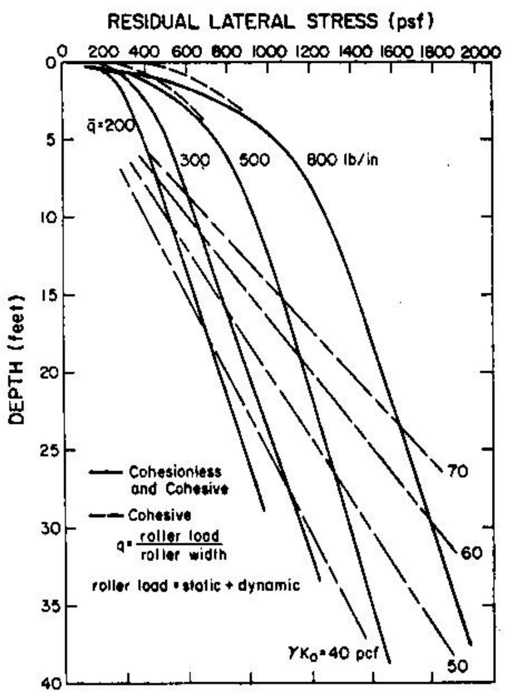


Fig. 6.17 Earth pressures due to compaction by rollers.

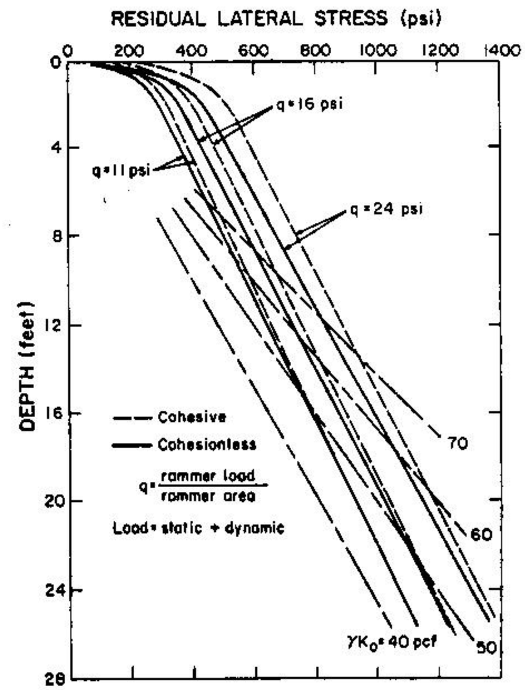


Fig. 6.19 Earth pressures due to compaction by rammer plates.

Reference: Clough, G.W. & Duncan, J.M., (1991). "Earth Pressure", Chapter 6, in *Foundation Engineering Handbook*, 2nd edition, ed. Hsai-Yang Fang, Van Norstrand, Reinhold.

TABLE 6.4 ADJUSTMENT FACTORS FOR EARTH PRESSURES INDUCED BY COMPACTION WITH ROLLERS.

Variables			*Multiplier Factors for z =* 2 ft	4 ft	8 ft	16 ft
Lift thickness and distance from wall (x) (adjustments for these two factors are combined)	6-in lifts	$x = 0$	1.70	2.00	1.90	1.85
		$x = 0.2$ ft	1.50	1.85	1.70	1.65
		$x = 0.5$ ft	1.00	1.00	1.00	1.00
		$x = 1.0$ ft	0.85	0.86	0.87	0.88
	12-in lifts	$x = 0$	1.05	1.10	1.15	1.20
		$x = 0.2$ ft	1.00	1.05	1.10	1.10
		$x = 0.5$ ft	0.90	0.94	0.98	1.00
		$x = 1.0$ ft	0.70	0.70	0.70	0.70
Roller width (w)		$w = 15$ in	0.90	0.85	0.85	0.90
		$w = 42$ in	0.95	0.95	0.95	0.95
		$w = 84$ in	1.00	1.00	1.00	1.00
		$w = 120$ in	1.00	1.00	1.00	1.00
Friction angle (ϕ)		$\phi = 25°$	0.70	0.80	0.90	1.10
		$\phi = 30°$	0.85	0.90	0.95	1.05
		$\phi = 35°$	1.00	1.00	1.00	1.00
		$\phi = 40°$	1.25	1.15	1.10	1.00

TABLE 6.5 ADJUSTMENT FACTORS FOR EARTH PRESSURES INDUCED BY COMPACTION WITH VIBRATING PLATES AND RAMMERS.

Variables			*Multiplier Factors for z =* 2 ft	4 ft	8 ft	16 ft
Lift thickness and distance from wall (x) (adjustments for these two factors are combined)	4-in lifts	$x = 0$	1.00	1.00	1.00	1.00
		$x = 0.5$ ft	0.79	0.81	0.82	0.83
	6-in lifts	$x = 0$	0.83	0.85	0.87	0.90
		$x = 0.5$ ft	0.66	0.69	0.71	0.75
Vibrating plate area		240 in^2	0.85	0.85	0.90	0.95
		480 in^2	1.00	1.00	1.00	1.00
		960 in^2	1.15	1.15	1.15	1.10
Rammer plate area		72 in^2	0.85	0.85	0.90	0.95
		144 in^2	1.00	1.00	1.00	1.00
		288 in^2	1.15	1.15	1.15	1.10
Friction angle (ϕ)		$\phi = 25°$	0.80	0.90	1.05	1.25
		$\phi = 30°$	0.85	0.95	1.00	1.10
		$\phi = 35°$	1.00	1.00	1.00	1.00
		$\phi = 40°$	1.15	1.10	1.00	0.90

Procedure for using the Charts for Earth Pressure after Compaction

1. Knowing the compaction machine, calculate compaction per length or per area (plates or rollers).
2. Get into the right chart at the depth you are looking for → find residual lateral stress.
3. Check Tables 6.4 or 6.5 for the correction factors to correct the stress you found.
4. Make sure that your residual lateral stress ≥ K_0 conditions.

Example:

Estimate the horizontal earth pressure at a depth of 5ft below the surface after compaction in 6in lifts by multiple passes of a Bomag BW 35 walk-behind vibratory roller. The estimated internal friction angle is $\phi = 40°$. The static weight on one drum is **628lb**, and the centrifugal force on one drum is **2,000lb**. The length of the drum is 15.4in. Thus, q = 2,628/15.4 = 171lb/in.

From Figure 6.17, at a depth of 5.0ft, find p_h = 340psf.

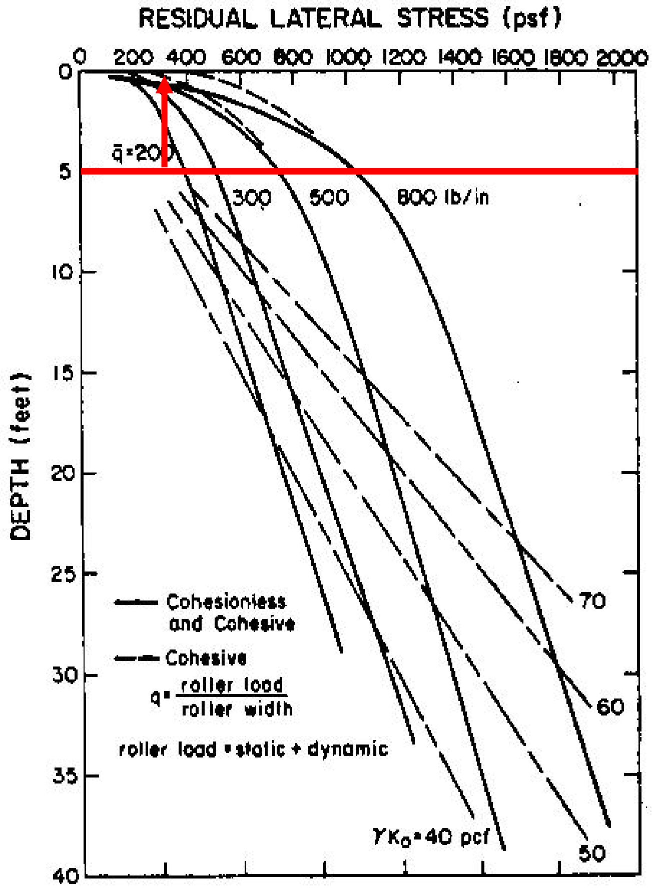


Fig. 6.17 Earth pressures due to compaction by rollers.

Adjustments must be made to this value, however, to account for the facts that: (1) the ϕ for the soil is 40° rather than the standard 35°, (2) the length of the roller is 15.4in rather than the standard 84in, and (3) the roller approaches within 0.2ft of the wall rather than the standard 0.5ft. The adjustment factors for these non-standard values are estimated using the values summarized in Table 6.4. The values of the adjustment factors (called R) are: R_x = 1.8, R_w = 0.85, R_ϕ = 1.14.

TABLE 6.4 ADJUSTMENT FACTORS FOR EARTH PRESSURES INDUCED BY COMPACTION WITH ROLLERS.

			Multiplier Factors for z =			
Variables			*2 ft*	*4 ft*	*8 ft*	*16 ft*
Lift thickness and distance from wall (x) (adjustments for these two factors are combined)	6-in lifts	x = 0	1.70	2.00	1.90	1.85
		x = 0.2 ft	1.50	1.85	1.70	1.65
		x = 0.5 ft	1.00	1.00	1.00	1.00
		x = 1.0 ft	0.85	0.86	0.87	0.88
	12-in lifts	x = 0	1.05	1.10	1.15	1.20
		x = 0.2 ft	1.00	1.05	1.10	1.10
		x = 0.5 ft	0.90	0.94	0.98	1.00
		x = 1.0 ft	0.70	0.70	0.70	0.70
Roller width (w)		w = 15 in	0.90	0.85	0.85	0.90
		w = 42 in	0.95	0.95	0.95	0.95
		w = 84 in	1.00	1.00	1.00	1.00
		w = 120 in	1.00	1.00	1.00	1.00
Friction angle (ϕ)		ϕ = 25°	0.70	0.80	0.90	1.10
		ϕ = 30°	0.85	0.90	0.95	1.05
		ϕ = 35°	1.00	1.00	1.00	1.00
		ϕ = 40°	1.25	1.15	1.10	1.00

			Multiplier Factors for z =			
Variables			*2 ft*	*4 ft*	*8 ft*	*16 ft*
		x = 0	1.70	2.00	1.90	1.85
Lift thickness and distance from wall (x)	6-in	x = 0.2 ft	1.50	1.85	1.70	1.65
(adjustments for these two factors are combined	lifts	x = 0.5 ft	1.00	1.00	1.00	1.00
		x = 1.0 ft	0.85	0.86	0.87	0.88
		x = 0	1.05	1.10	1.15	1.20
	12-in	x = 0.2 ft	1.00	1.05	1.10	1.10
	lifts	x = 0.5 ft	0.90	0.94	0.98	1.00
		x = 1.0 ft	0.70	0.70	0.70	0.70
Roller width (w)		w = 15 in	0.90	0.85	0.85	0.90
		w = 42 in	0.95	0.95	0.95	0.95
		w = 84 in	1.00	1.00	1.00	1.00
		w = 120 in	1.00	1.00	1.00	1.00
Friction angle (ϕ)		ϕ = 25°	0.70	0.80	0.90	1.10
		ϕ = 30°	0.85	0.90	0.95	1.05
		ϕ = 35°	1.00	1.00	1.00	1.00
		ϕ = 40°	1.25	1.15	1.10	1.00

Using this information from Figure 6.17 and Table 6.4, it is estimated that the postcompaction lateral earth pressure is equal to:
p_h = (340psf)(1.8)(0.85)(1.14) = 590psf. This value compares to a value of 570psf calculated by means of detailed computer analyses performed using the methods developed by Duncan and Seed (1986).

By using the same procedure to estimate pressures at other depths, the distribution of earth pressures after compaction can be estimated. At the depth where these become smaller than the estimated at-rest pressures, the lateral pressures are equal to the at-rest values, as shown in Figure 6.16.

Post compaction earth pressures estimated using Figures 6.17, 6.18, and 6.19 and Tables 6.4 and 6.5 apply to conditions where the wall is stiff and nonyielding. These pressures would provide a conservative (high) estimate of pressures on flexible walls or massive walls whose foundation support conditions allow them to shift laterally or tilt away from the backfill during compaction. Such movements would reduce the earth pressures. The reduction would be expected to be less near the surface, where the compaction-induced loads would tend to "follow" the wall as it deflected or yielded.

EARTH PRESSURE ON RIGID RETAINING WALLS NEAR ROCK FACES

Reference: Frydman, S. and Keissar, I. (1987). *Earth Pressure on Retaining Walls Near Rock Faces,* ASCE Journal of Geotechnical Eng., V113, pp.586-599.

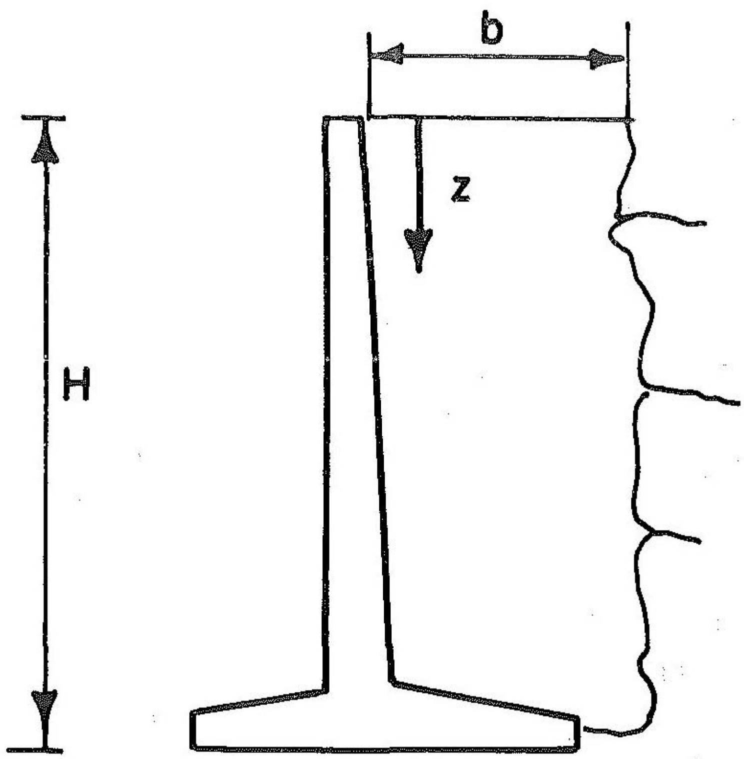


FIG. 1.—Schematic Representation of Retaining Wall near Rock Face

Reference: Frydman, S. and Keissar, I. (1987). *Earth Pressure on Retaining Walls Near Rock Faces,* ASCE Journal of Geotechnical Eng., V113, pp.586-599.

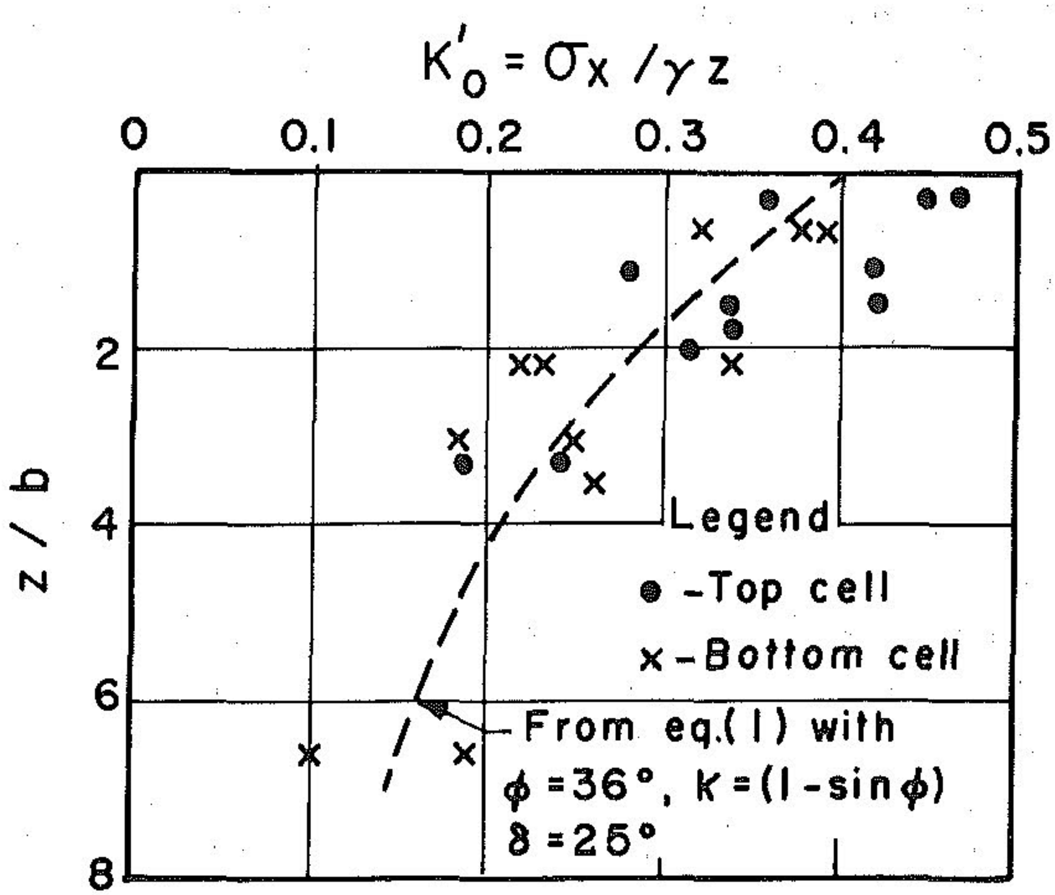


FIG. 7.—Measured and Calculated Values of K_0', versus z/b

$$\sigma_x = \frac{\gamma b}{2\tan\delta}\left[1 - \exp\left(-2K\frac{z}{b}\tan\delta\right)\right] \qquad \text{(Eq. 1)}$$

Reference: Frydman, S. and Keissar, I. (1987). *Earth Pressure on Retaining Walls Near Rock Faces,* ASCE Journal of Geotechnical Eng., V113, pp.586-599.

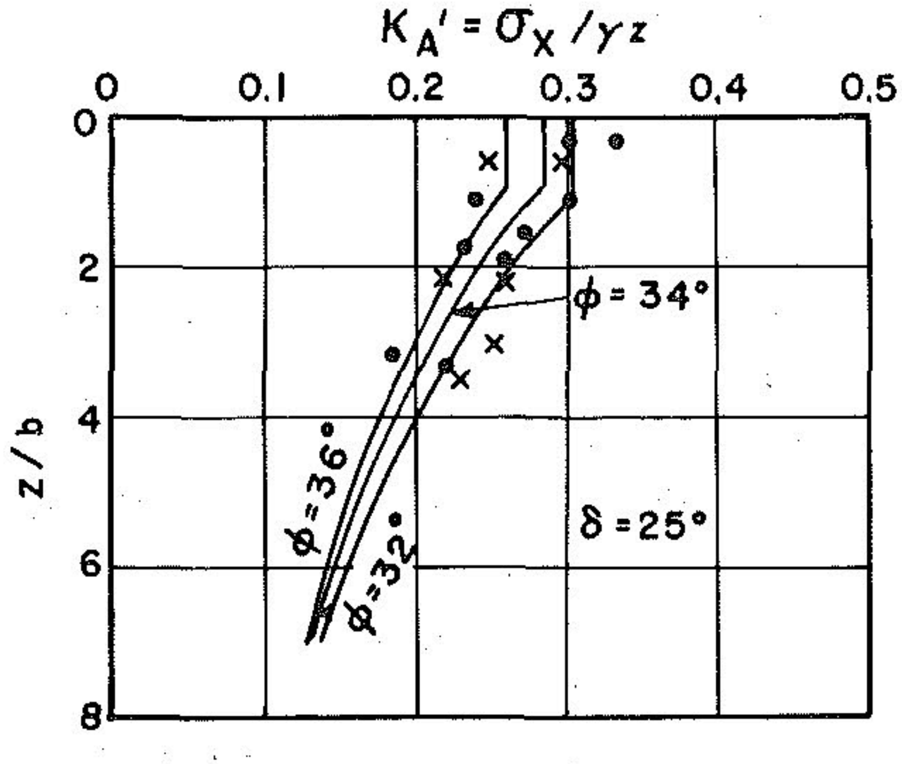


FIG. 15.—Comparison between K_a' Distributions Obtained from Silo Pressure Equation Using K from Eq. 3 and Measured Values

Equation 3:

$$K = \frac{(sin^2\Phi + 1) - \sqrt{(sin^2\Phi + 1)^2 - (1 - sin^2\Phi)(4tan^2\delta - sin^2\Phi + 1)}}{(4tan^2\delta - sin^2\Phi + 1)}$$

Reference: Frydman, S. and Keissar, I. (1987). *Earth Pressure on Retaining Walls Near Rock Faces,* ASCE Journal of Geotechnical Eng., V113, pp.586-599.

Conclusions

The results of a study of the lateral pressure transferred to a rigid retaining wall by granular fill confined between the wall and an adjacent rock face are :

1. It is found that Eq. 1, commonly used for estimating lateral pressure on silo walls, may be used to calculate the pressure for the no-movement (K_0) condition, using a K value of 1 – sin Φ. Significant variations from the estimated pressure value may occur next to the wall, due to small variations in placement conditions (e.g., localized compaction effects, slight variations in density, etc.).
2. A conservative approach could be to use a decreased Φ value in calculating K, so as to obtain an upper envelope to the expected pressure values.
3. The pressures acting on the wall, when it reaches an active condition by rotating about its base, appears to be less sensitive to small variations in placement conditions. Progressive failure, which occurs within the soil mass adjacent to the wall during its rotation, results in a decrease in Φ, and this decreased value must be used in estimating the pressure acting on the wall.
4. Reasonable estimates of wall pressure may be obtained from application of the silo pressure equation, in which a K-value compatible with the values of Φ and δ (see Eq. 3) is used.