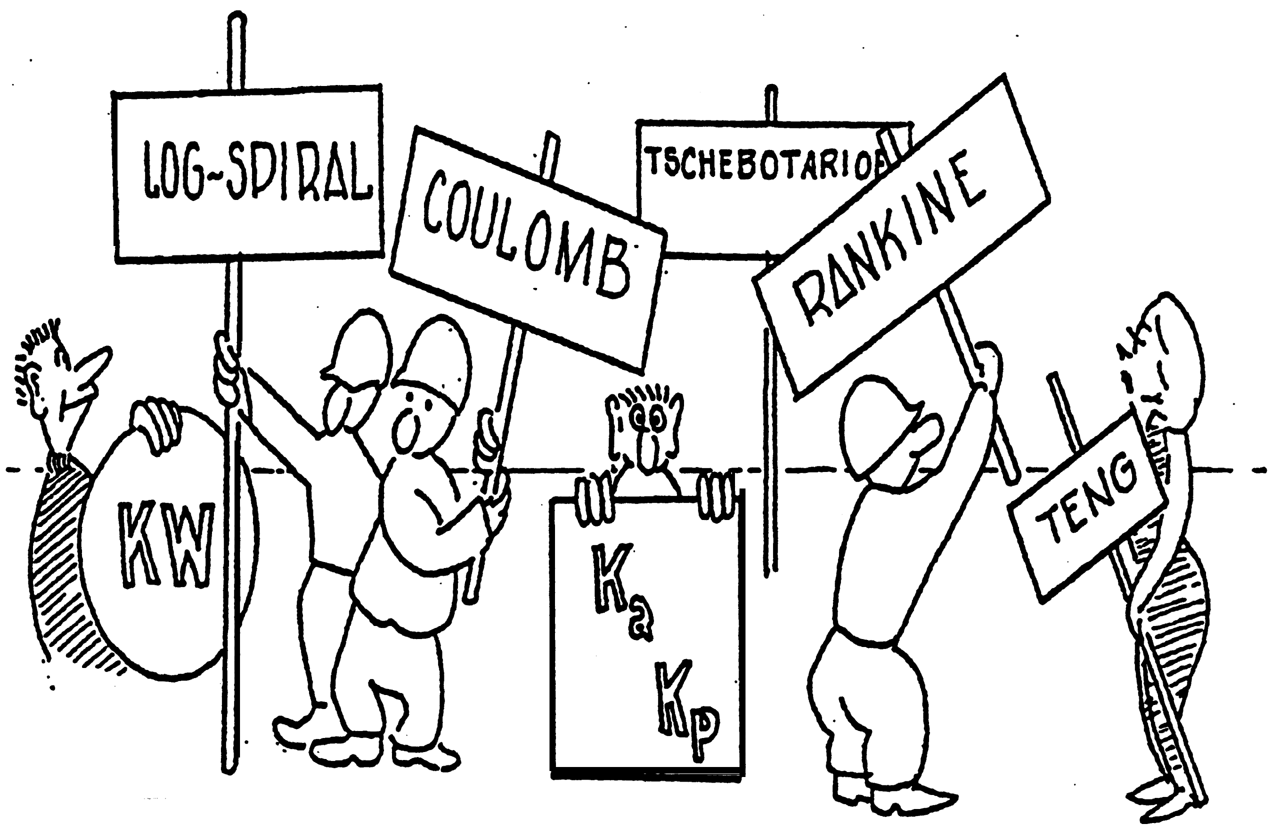
EARTH PRESSURE THEORY AND APPLICATION

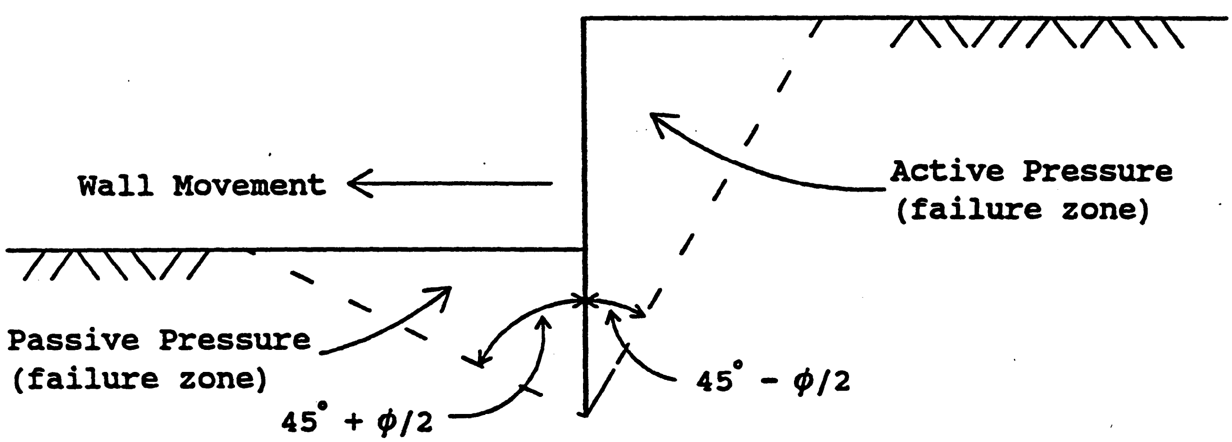


EARTH PRESSURE THEORY AND APPLICATION

Earth pressure is the lateral force exerted by the soil on a shoring system. It is dependent on the soil structure and the interaction or movement with the retaining system. Due to many variables, shoring problems can be highly indeterminate. Therefore, it is essential that good engineering judgment be used.

ACTIVE AND PASSIVE EARTH PRESSURES

Active and passive earth pressures are the two stages of stress in soils which are of particular interest in the design or analysis of shoring systems. Active pressure is the condition in which the earth exerts a force on a retaining system and the members tend to move toward the excavation. Passive pressure is a condition in which the retaining system exerts a force on the soil. Since soils have a greater passive resistance, the earth pressures are not the same for active and passive conditions. When a state of oil failure has been reached, active and passive failure zones, approximated by straight planes, will develop as shown in the following figure (level surfaces depicted).



The well known earth pressure theories of Rankine and Coulomb provide expressions for the active and passive pressure for a soil mass at a state of failure.

COEFFICIENT OF EARTH PRESSURE

The coefficient of earth pressure (K) is the term used to express the ratio of the lateral earth pressure to the vertical earth pressure or unit weight of the soil. For a true fluid the ratio would be l. The vertical pressure is determined by using a fluid weight equal to the unit weight of the soil: $P_v = KP_v$ The basic formulas for horizontal earth pressures are as follows:

$P_H = KP_V = K\gamma H$ = Lateral earth pressure

If a soil has a cohesive value the formula becomes:

$$P_H = K\gamma H \pm 2C[K]^{1/2}$$

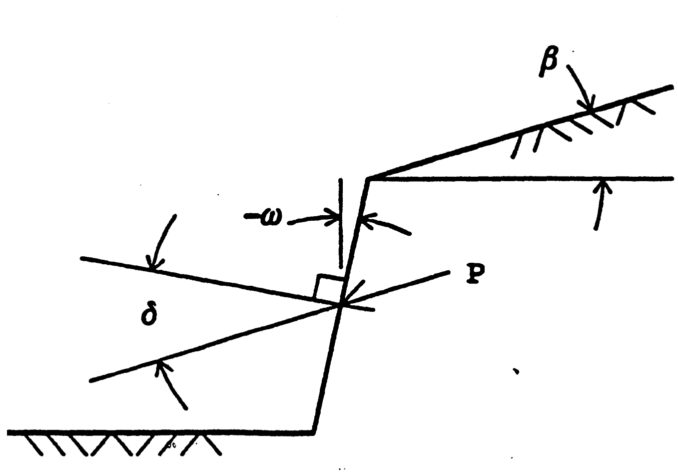
There are three ranges of earth pressure coefficients to be considered:

- Ka = Coefficient of Active earth pressure (0.17 to 1.0)
- K_p = Coefficient of Passive earth pressure (1.0 to 10.0)
- K_0 = Coefficient of earth pressure for soils at rest or in place (0.4 to 0.6 for drained soils).

The next step is to determine the value of the earth pressure coefficient (K). This is accomplished by utilizing the known soil properties and the accepted theories, formulas, graphs and procedures that are available.

Refer to the Table of Simplified Typical Soil Values TABLE 11, which lists active coefficient (K_a) and equivalent fluid (Kw) directly.

Earth pressure coefficients may also be calculated by acceptable soil mechanics formulas. Two of the more well known authors are Rankine and Coulomb.



ϕ = Angle of internal friction

β = Angle of the backfill slope

δ = Angle of wall friction

ω = Angle of the wall with respect to vertical (negative value shown)

P = Active lateral earth pressure

THE RANKINE THEORY

The Rankine theory assumes that there is no wall friction ($\delta = 0$) the ground and failure surfaces are straight planes, and that the resultant force acts parallel to the backfill slope. The coefficients according to Rankine's theory are given by the following expressions:

$$K_a = \cos\beta\left[\frac{\cos\beta - [\cos^2\beta - \cos^2\phi]^{1/2}}{\cos\beta + [\cos^2\beta - \cos^2\phi]^{1/2}}\right]$$

$$K_p = \cos\beta\left[\frac{\cos\beta + [\cos^2\beta - \cos^2\phi]^{1/2}}{\cos\beta - [\cos^2\beta - \cos^2\phi]^{1/2}}\right]$$

If the embankment is level ($\beta = 0$) the equations are simplified as follows:

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \tan^2(45^\circ - \phi/2)$$

$$K_p = \frac{1 + \sin\phi}{1 - \sin\phi} = \tan^2(45^\circ + \phi/2)$$

The Rankine formula for passive pressure can only be used correctly when the embankment slope angle β equals zero or is negative. If a large wall friction value can develop, the Rankine Theory is not correct and will give less conservative results. Rankine's theory is not intended to be used for determining earth pressures directly against a wall (friction angled does not appear in equations above). The theory is intended to be used for determining earth pressures on a vertical plane within a mass of soil.

THE COULOMB THEORY

The Coulomb theory provides a method of analysis that gives the resultant horizontal force on a retaining system for any slope of wall, wall friction, and slope of backfill provided $\beta \le \phi$. This theory is based on the assumption that soil shear resistance develops along the wall and failure plane. The following coefficient is for a resultant pressure acting at angle δ.

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

The passive K_p value for sloping embankment is not listed since this value can be drastically overestimated.

The following coefficients are for a horizontal resultant pressure and a vertical wall:

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

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

Wall friction angle (δ) varies from 0° to 22°, but is always less than the internal angle of friction of the soil (ϕ). It is accepted practice to assume a value of $\delta = 1/3(\phi)$ to $2/3(\phi)$. See TABLE 14 for some typical friction values.

If the shoring system is vertical and the backfill slope and wall friction angles are zero (ω, β and $\delta = 0$), Coulomb's equation will be the same as Rankine's for a level ground condition. Coulomb's pressure distribution has been shown to be essentially correct for the lateral movements of sheeting of braced cuts which closely correspond to the conditions of rotation of a wall around its top.

Since wall friction requires a curved surface of sliding to satisfy equilibrium, the Coulomb formula will give only approximate results as it assumes planar failure surfaces. The accuracy for Coulomb will diminish with increased depth. For passive pressures the Coulomb formula can also give inaccurate results when there is a large back slope or wall friction angle. These conditions should be investigated and an increased factor of safety considered.

LOG-SPIRAL THEORY

A Log-spiral theory was developed because of the unrealistic values of earth pressures that are obtained by theories which assume a straight line failure plane. The difference between the Log-Spiral curved failure surface and the straight line failure plane can be large and on the unsafe side for Coulomb passive pressures (especially when wall friction exceeds $\phi/3$). The following figure shows a comparison of the Coulomb and Log-Spiral failure surfaces:

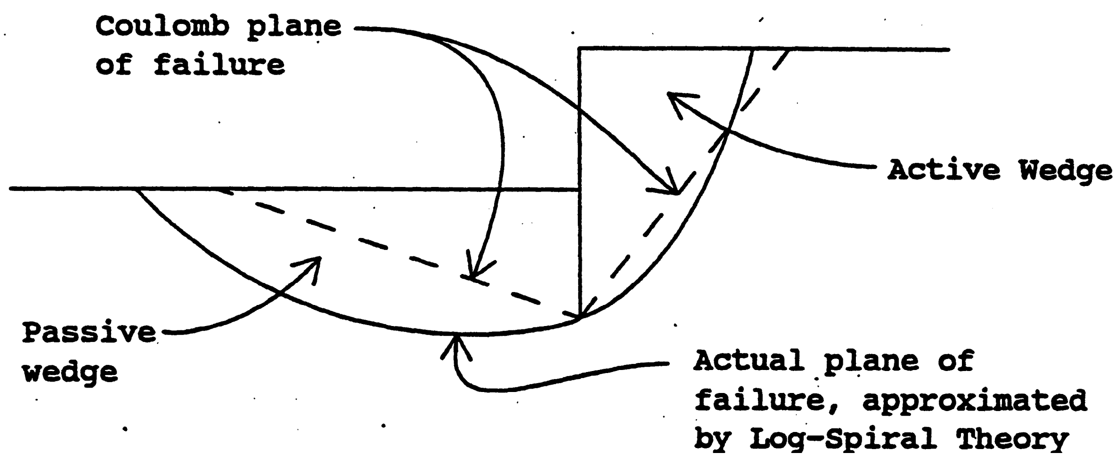


FIGURE 7

The coefficient of earth pressure values for the log-spiral failure surf ace can be obtained from FIGURE 8. K_a may be read directly from the curves using the lower portion of FIGURE 8 whereas K_p must be multipliedby a reduction factor (R) located at the top of the figure.

Rankine is conservative relative to other methods. Except for the passive condition when δ is greater than approximately $\phi/3$, Coulomb is conservative relative to Log-Spiral. These methods developed as refinements to one another; each in its turn accounting for more variables and thereby requiring increasing levels of analytic complexity.

EARTH PRESSURE COEFFICIENT WHEN AT-REST

The at-rest earth pressure coefficient (Ko) is applicable for, determining the active pressure in clays for strutted systems. Because of the cohesive property of clay there will be no lateral pressure exerted in the at-rest condition up to some height at the time the excavation is made. However, with time, creep and swelling of the clay will occur and a lateral pressure will develop. This coefficient takes the characteristics of clay into account and will always give a positive lateral pressure. This method is called the Neutral Earth Pressure Method and is covered in the text by Gregory Tschebotarioff.

$$K_o = \frac{v}{1 - v}$$

v = The Poisson's Ratio. It is determined by a Laboratory test (Maximum value = 0.5)

An alternate solution for K_o is to use Jaky's equation:

$$K_o = 1 - \sin \phi'$$

Where ϕ' is the effective angle of internal friction and not the total stress value. For most short term shoring situations the internal friction angle ϕ may be substituted for ϕ'.

In general, for a level ground situation, values of K_o will be greater than K_a. If movement of a retaining system is severely restricted (approaching a fixed condition) the active failure wedge cannot fully develop and consideration should be given to using K_o in lieu of K_a. For very deep excavations the horizontal movement that can occur is usually less than that needed to develop active failure condition, therfore K_o values should be used. It is noted that for deadman anchorages, K_o could be used to calculate the passive resistance.

WALL FRICTION (δ)

Wall friction angle (δ) varies from 0° to 22°, but is always less than the internal angle of friction of the soil (ϕ). It is accepted practice to assume a value of $\delta = 1/3(\phi)$ to $2/3(\phi)$. For systems subject to dynamic loading (adjacent railroads, pile driving operations, etc.) use $\delta = 0$. It is important to note that as wall friction increases, lateral pressures decrease, but the vertical load on the shoring system increase. Vertical load components must be considered in shoring design. TABLE 14 lists friction of select soil types acting against various structural materials.

ULTIMATE FRICTION FACTORS AND ADHESION FOR DISSIMILAR MATERIALS

INTERFACE MATERIALS	FRICTION ANGLE, δ DEGREES
Steel sheet piles against the following soils:	
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22
Clean sand, silty sand-gravel mixture, single size hard rock fill	17
Silty sand, gravel or sand mixed with silt or clay	14
Fine sandy silt, nonplastic silt	11
Formed concrete or concrete sheet piling aginst the following soils:	
Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22 to 26
Clean sand, silty sand-gravel mixture, single size hard rock fill	17 to 22
Silty sand, gravel or sand mixed with silt or clay	17
Fine sandy silt, nonplastic silt	14
Mass concrete on the following materials:	
Clean sound rock	35
Clean gravel, gravel-sand mixtures, coarse sand	29 to 31
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29
Clean fine sand, silty or clayey fine to medium sand	19 to 24
Fine sandy silt, nonplastic silt	17 to 19
Very stiff and hard residual or preconsolidated clay	22 to 26
Medium stiff and stiff clay and silty clay	17 to 19
(Masonry on foundation materials has same friction factors.)	
Various structural materials:	
Masonry on masonry, ingneous and metamorphic rocks:	
Dressed soft rock on dressed soft rock	35
Dressed hard rock on dressed soft rock	33
Dressed hard rock on dressed hard rock	29
Masonry on wood (cross grain)	26
Steel on steel at sheet pile interlocks	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

TABLE 14 - WALL FRICTION AND ADHESION

POISSON'S RATIO

Soil Type	Typical Value for Poisson's Ratio *	K_o
Clay, saturated	0.40 - 0.50	0.67 - 1.00
Clay, unsaturated	0.10 - 0.30	0.11 - 0.42
Sandy Clay	0.20 - 0.30	0.25 - 0.42
Silt	0.30 - 0.35	0.42 - 0.54
Sand		
Dense	0.20 - 0.40	0.25 - 0.67
Coarse		
(void ratio 0.4 - 0.7)	0.15	0.18
Fine-grained		
(void ratio 0.4 - 0.7)	0.25	0.33
Rock	0.10 - 0.40	0.11 - 0.67

TABLE 15

MOVEMENT OF WALL NECESSARY TO PRODUCE ACTIVE PRESSURES

Soil Type	Wall Yield
Cohesionless, dense	0.001 H
Cohesionless, loose	0.001 - 0.002 H
Clay, firm	0.010 - 0.020 H
Clay, soft	0.020 - 0.050 H

* New Zealand Department of Public Works Retaining Wall Manual

TABLE 16

For sands, Terzaghi & Peck have indicated one could expect that a movement of 0.5% times the height of the support system would be needed to obtain a complete active condition. For a 20' deep excavation the movement needed at the top of the excavation would amount to (0.005) (20) = 0.1 foot of movement to develop a fully active condition.

Example:

Given: $\phi = 26°$, $\beta = -10°$, $\delta = \pm 7.5°$
$\beta/\phi = -0.4$, $\delta/\phi = -0.3$ (Passive)

To determine K_a use the lower portion of FIGURE 8.

1) Locate the curve that coresponds to a $\beta/\phi = -0.4$.

2) Find the vertical line for $\phi = 26°$.

3) Follow that line up to where it intersects with the curve for $\beta/\phi = -0.4$.

4) Proceed to the left of the graph and read K_a directly.

For this example K_a is approximately 0.34. The resultant force ($P = \gamma H K_a$) acts downward at an angle δ to the horizontal.

To determine K_p use the upper portion of FIGURE 8.

1) Locate the curve that corresponds to a $\beta/\phi = -0.4$.

2) Find the vertical line for $\phi = 26°$.

3) Follow that line up to where it intersects with the curve for $\beta/\phi = -0.4$.

4) Proceed to the left of the graph and read a value of approximately 3.4.

5) Next apply the reduction factor from the table at the top. NOTE: the R is not a safety factor.
For $\delta/\phi = -0.3$ and $\phi = 26°$, R = 0.694 by interpolation.

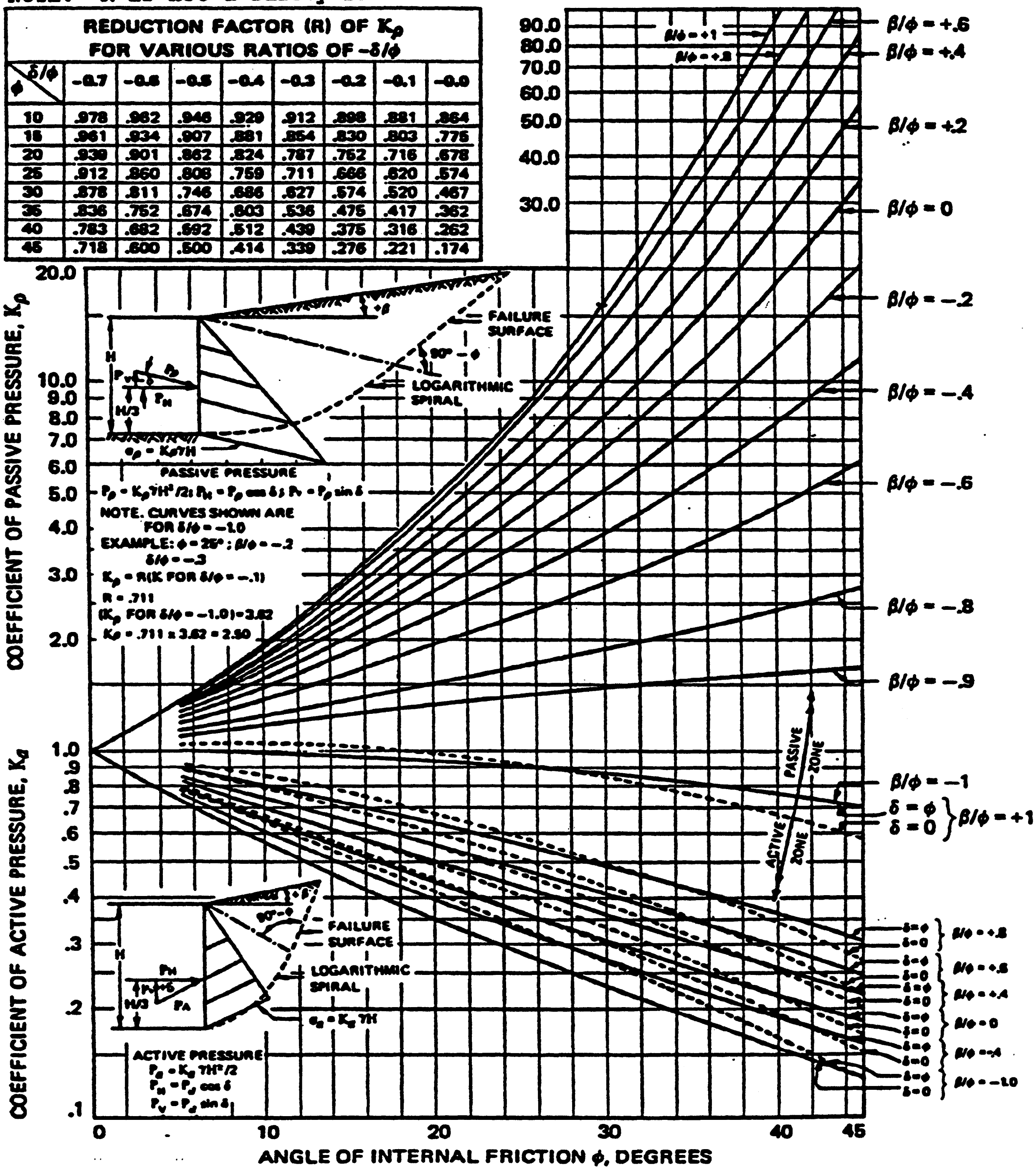
For this example then, K_p is approximately (3.4)(0.694) = 2.36. The resultant force ($P = \gamma H K_p$) acts upward at an angle δ to the horizontal.

LOG - SPIRAL FAILURE SURFACE

NOTE: R is not a safety factor.

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



Active and passive coefficients with wall friction (sloping backfill)

FROM USS STEEL SHEET PILING DESIGN MANUAL

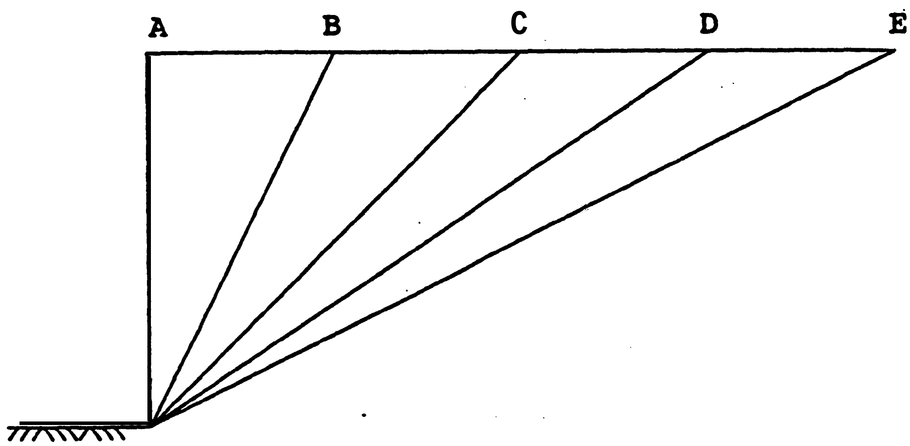
FIGURE 8

APPROXIMATE ANGLES OF REPOSE FOR SOILS

Soils will stand at some natural slope, the angle of repose, unless acted upon by some external force, or unless it is subjected to an internal change of composition, such as a change in water content. FIGURE 9 depicts some repose angles for various materials.

A slope of 1:l corresponds to Type B soil per CAL/OSHA.
A slope of 1.5:l corresponds to Type C soil per CAL/OSHA.

A running soil by Cal/OSHA defintion would have a repose angle of less than 2:l.



A. Solid Rock, Shale or Cemented Sand and Gravels ...... (90°)

B. Compacted Angular Gravels .... (1/2:1)

C. Recommended Slope for Average Soils .... (1:1)

D. Compacted Angular Sand .... (1.5:1)

E. Well Rounded Loose Sand .... (2:1)

FIGURE 9