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**DETERMINATION OF EARTH  
PRESSURE DISTRIBUTIONS  
FOR  
LARGE-SCALE RETENTION  
STRUCTURES**

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## **DETERMINATION OF EARTH PRESSURE DISTRIBUTIONS FOR LARGE-SCALE RETENTION STRUCTURES**

### **1.0 Introduction**

Various earth pressure theories assume that soils are homogeneous, isotropic and horizontally inclined. These assumptions lead to hydrostatic or triangular pressure distributions when calculating the lateral earth pressures being exerted against a vertical plane.

Field measurements on deep retained excavations have shown that the average earth pressure load is approximately uniform with depth with small reductions at the top and bottom of the excavation. This type of distribution was first suggested by Terzaghi (1943) on the basis of empirical data collected on the Berlin Subway and Chicago Subway projects between 1936-42. Since that time, it has been shown that this uniform distribution only occurs when the following conditions are met:

1. The upper portions of the vertical side walls of the excavation are supported in stages as the excavation is deepened;
2. The walls of the excavation are pervious enough so that water pressure does not build up behind them; and
3. The lateral movements of the walls are kept below 1% to 2% of the depth of the excavation.

With the passage of time, the approximately uniform pressure distribution evidenced during construction has been observed to transition toward the more traditional triangular distribution. In addition, it has been found that the tie-back force in anchored bulkhead walls generally increases with time.

The actual load imposed on a semi-vertical retaining wall is dependent on eight aspects of its construction:

1. The degree of saturation of the wall backfill in the zone of active or at-rest earth pressure.
2. The degree of relative compaction of the wall backfill within the active or at-rest envelopes.
3. The structural stiffness of the retaining wall system and its ability to deflect outward in response to "active" earth loads.
4. The magnitude and inclination of thrust generated by soil loads due to either inclined bedding, filling, or finished grades behind the wall.
5. The inclination between the wall itself and the backfill materials.

6. The applicable skin friction at the wall/backfill interface.
7. The vertical and horizontal components of loads associated with structural tie-backs.
8. Live or dead surcharge loads above and/or behind the wall.

Each of these factors should be considered and appropriately evaluated separately. The final earth pressure calculations can then be made which will incorporate the design life of the wall and the environmental extremes to which the wall will be subjected (such as likely fluctuations in the groundwater table or freezing ground, etc.).

The following section outlines the design aids and procedures available for evaluating the above list of factors when calculating earth pressure values.

## **2.0 Calculating Earth Pressure Values**

The engineer should begin by neatly sketching a scaled elevation view of the design section selected for consideration. Upon this section, the various limits and types of subsurface materials may be labeled. Below this (or on the next page) a neat tabulation of the various soil types and their available soils properties should be listed as "Given Values". These values would typically include: soil bulk density and strength values derived from appropriate subsurface materials sampled in the field and then tested in the soils laboratory. At this juncture, the engineer should evaluate these values and the soils classifications made to see if they are commensurate with what he feels is representative of the site. Anomalous or unrepresentative values should be stricken with a single pencil line and no longer considered in the analysis. A parenthetical note as to the reason for deleting this data should be added on the paper margin if that data is to be retained on the boring logs.

## **2.1 Calculations Involving Landslides**

In many situations, such as landslides, a discrete weak zone or discontinuity may have been responsible for the failure or another future movement. If such weak zones were encountered or suspected in the field investigation, care should be exercised in evaluating all of the field data. In some situations, averaging of empirical data might be a viable approach to classifying some material. In other situations, values at the more conservative end of the data range might be selected as being more-representative of that particular weakness zone or zones which are critical to a stability analysis. To error on the side of conservatism is always considered to be the prudent alternative to choose. A back analysis of the slippage is another method which can be used to check the strength parameters chosen for the slip surface.

## **3.0 Material Properties**

In some cases, there is either a lack of reliable data or much of the data yielded in the investigation is not felt to be representative of, say, the weakest material.

In such cases, it is often possible to make correlations between either Standard Penetration Resistance (SPT) Values or Atterberg Limits values with the angle of internal friction,  $\phi$ , of certain types of materials. Many such relationships have been presented over the years in the engineering literature. A brief listing of these relationships follows:

<u>Correlation</u>	<u>Type of Soil</u>
SPT vs $\phi$	for sands
$e$ vs $n$ vs $\lambda_d$	for sands
Soil type vs unit weight	all types
$\lambda_d$ vs $w/c$	all types
$\phi$ vs type of soil	all types
PI vs $\phi$	for clays (drained)
$\phi$ vs Rel. Density	for sands
$\phi$ vs $\lambda_d$ , $D_r$	sands, silts, gravel
$\phi$ vs $N$	granular mtl's.
$S_u$ vs SPT	for clays
PI vs OCR	for clay
$S_u$ vs	for normally consolidated clay
$E$ , $q_u$	for types of rock
$V_p$ (P-wave velocity)	all types

When a property is chosen from one of these correlation charts or tables, the engineer should make a xerox copy of the chart or table, note its source, and properly highlight or underline the particular values given and the values chosen. This information should then be included with the engineer's other calculations in the job file.

#### **4.0 Degree of Saturation**

The build-up of pore water behind a retaining wall can roughly double the active force on such a structure from that imposed by a dry backfill. It is likely that the vast majority of continuous retaining wall failures occur due to the build-up of excess hydrostatic (water) pressures due to either a lack of adequate drainage or impermeability of the backfill materials.

Bentonite slurry walls and steel sheet pile walls are fairly impervious and can, therefore, experience the build-up of significant hydrostatic pressure, regardless of the groundwater level on the front (open) side of such a wall.

Discontinuous retaining structures, such as drilled piers, H-piles, or crib walls, can be more free-draining by their open nature. However, a suitable amount of *free-draining material* must be emplaced behind such structures to allow the dissipation of hydrostatic pressures. The proper conveyance of groundwater via such a system is commonly specified in Guide Specifications for Engineered Fill, prepared by geotechnical engineers or government agencies. The engineer should check for details as to:

1. The effective width and inter-connectivity of pervious back-fill material

2. Check filtration criteria so that hydraulic piping and clogging of the pervious material will be lessened during the life of the structure.
3. Evaluate the subdrain system for a way of both checking and insuring that groundwater flow continues and is uninhibited.

In most states, Item 1 can be evaluated by specifying the placement of permeable material according to the respective DOT sections pertaining to Permeable Backfill Material or a suitable alternative (generally listed under Classes of Aggregate Subbase). An effective width of free-draining material behind the wall is largely a matter of judgment. Most DOTs specify 1 cubic foot of gravel wrapped in a bur-lap bag behind weepholes in continuous walls. The U.S. Bureau of Reclamation recommends backfilling a wall with free-draining material, 3 feet width at the bottom, tapering to 1 foot just below the top of a wall. The topmost 6 inches to 1 foot of wall backfill is normally composed of structural backfill of a more impervious nature to prevent the subdrain wall from collecting surface water.

Until the 1990s most practitioners typically specify a 1-foot-wide zone of free-draining material behind a wall with a 6-inch clay cap on top. Walls in excess of 15 feet height may require a wider drain blanket behind the wall like the U.S.B.R. recommends. Most practitioners are beginning to utilize prefabricated drainage membranes like Miradrain, Enkadrain, Tensar DC-1200, Tenax TNT, MNT, etc.

The possibility of hydraulic piping of fine materials into the subdrain can be accounted for by employing a graded filter or geotextile material between the excavation/subdrain interface. Graded filter specifications are available from the U.S. Corps of Engineers (whose filter criteria was revised in 1976) or in the U.S. Bureau of Reclamation's Earth Manual (pp. 305-309). Oftentimes, a geotextile fabric may be used instead of a graded filter fill. There are over 200 textile types presently available. Through some experiences with rock flow and silt, it has recently been suggested that it is no longer desirable to wrap subdrain pipes in geotextile cloth because the textile can plug with fines. It is considered desirable to place the cloth at the excavation face so that it lies between the native materials and the free-draining fill. It is still considered prudent to wrap perforated hydrauger pipes with geotextile cloths as these drains are not designed for the same life-expectancy as a retaining wall.

Subdrain pipes should generally be designed as continuous systems with as few tight bends as possible. Typically, the pipes are contained in the free-draining material approximately 6 inches off the bottom of the soil/subdrain interface. Normally, a perforated SDR 23.5 or equivalent ABS/PVC or perforated pipe is laid in the subdrain excavation with the perforations down. SDR 23.5 possesses a wall thickness sufficient to allow for clean-out with a roto-rooter. It is the only PVC or ABS schedule pipe now acceptable for sewer lines by most sanitation districts in the western United States. Vitri-fied clay pipe (VCP) possesses a greater

resistance to corrosion and chemico/environmental effects. However, due to its brittleness, its employment is usually limited to situations where adverse differential settlement is considered unlikely.

Subdrain pipes should be connected to solid clean-out sections which extend to the ground surface and are capped. The clean-out should not connect to the perforated subdrain pipes on an angle in excess of 45 degrees. Experience has also shown that clean-out sections should not extend more than 18" above grade as they may attract attention and people tend to drop rocks down them to see what they'll hear. Screw caps inside of precast concrete utility boxes is recommended in most urban environments.

Guide specifications for clean-out maintenance and energy dissipation at the point of outflow should be included in any construction plans. If as-built drawings of a wall are required, the general clean-out and drain locations should be noted.

### **5.0 Relative Compaction of Wall Backfill**

Guide specifications for structural backfill are normally included in all geotechnical engineering reports. These guidelines typically specify the structural fill gradation requirements and the degree of relative compaction with respect to depth of the required fill. These are usually given as:

- 90% relative compaction required for fills up to 10 feet deep
- 95% relative compaction required for fills over 10 feet deep

Problems with backfill construction operations or techniques have led to many retaining wall failures. The effects of the surcharge load imposed by construction equipment must be adequately considered in the wall design. Vibration caused by equipment or by the loose dumping of material behind the wall can lead to excessive pore pressures in a semi-granular fill. These locked-in pressures will cause the wall to feel the At-Rest earth pressure in lieu of the desired Active or Apparent earth pressures. This build-up of the earth pressure can also cause excessive tilting and/or strain-failure of the wall stem.

### **6.0 Structural Stiffness**

Rankine Earth Pressure Theory states that a soil mass must strain some finite amount in order to mobilize either passive or active shear resistance. The amount of this lateral strain is commonly in the range of 0.002H (sand) to 0.02H (clay) for the active range and 0.005H (sand) to 0.04H (clay) for the passive range. This is where H is equivalent to the total height of the wall (including the buried portion).

Walls which are retained in some manner or are built with too stiff a basal stem *may not deflect sufficiently* to mobilize the shear strength of the soil about them. Structures of this character must necessarily be designed to withstand the full hydrostatic pressure of the soil/water mixture they hold in retention.

Examples of such structures are rigid counterfort walls, tanks, bridge abutments, basement walls, walls of structures with integral transverse sections (such as floors and ceilings) and walls with "wing sections" return walls, and/or corners.

Tied-back walls may be considered to be either rigid or non-rigid, depending on the section modulus ( $EI$ ) of the wall section and the walls' ability to deflect between points of like support.

The rigidity or geometry of a chosen structure might require that the geotechnical engineer calculate the At-Rest Earth Pressures so the structural engineer can realistically design the section. The procedures for this calculation are presented later in Section 8.

Flexible wall systems such as sheet piles, which are normally tied back, possess the desirable attribute of arching the earth pressure loads betwixt the dead man anchor and passive pressure areas. This arching (in the vertical plane) of the lateral earth pressures reduces the load transferred to that portion of the wall where the maximum bending moment occurs. As a result, a significant reduction in the design bending moment and, hence the design section ( $EI$ ), can be realized.

Whereas flexibility can be shown to be advantageous in a single tied-back wall system, such deflections can play havoc with a cantilevered wall system. As the cantilever wall deflects laterally beyond 1% or 2% of the total wall height,  $H$ , the soil is strained beyond its pseudo-elastic limit and shear strength is now lost (i.e., soil is now in its residual strength range). This reduction in shear strength will cause an increase in the lateral earth pressure in excess of the At-Rest earth pressures. This type of deformation can be particularly detrimental in situations where a sloping backfill lies behind the wall. In some situations, a wall which is tilting excessively ( $>2\%H$ ) will be supporting an entire active-failure mass extending a considerable distance upslope. This mass could easily load such a wall with 200% to 1000% of a traditionally-derived active pressure force.

## **7.0 Inclined Backfills, Toefills, and Walls**

The most common mistake in the determination of lateral earth pressures is that incurred in hillside situations where active loads are normally increased and passive resistance decreased. Quite simply, this is because there is volumetric increase or decrease in the amount of soil material involved in loading the wall or in resisting its deflection.

Several loading situations exist which can alter the calculation of the Active Pressure coefficient,  $K_a$ . These include:

1. Inclined backfill or hillside slope behind the wall;
2. Another retaining wall close enough upslope to be within the active pressure wedge of a lower wall and thereby receiving a thrust from the upper wall;



3. The presence of downward-sloping geologic strata or fills behind the wall within its active pressure wedge;
4. Situations where active landslide masses are being retained.

Those loading conditions which can cause a decrease in the passive resistance to a wall include:

5. A downward-sloping toe slope in front of the wall;
6. The presence of active landslipping or the potential for sliding of that portion of a slope downslope of the wall;
7. An adjacent, lower retaining wall within the passive pressure wedge of the upper wall;
8. The use of individual piers subject to arching versus a continuous wall system.

Once an active pressure coefficient ( $K_a$ ) value is chosen for the appropriate loading condition, this value may be factored to account for At-Rest or apparent earth pressure conditions.

A number of graphical approximation methods have been traditionally employed to evaluate sloping backfill loading conditions (Coulomb, Engesser, or vector-sum methods to name a few).

A design chart for evaluating the active and passive pressure coefficients has been prepared. This chart is Figure 6 on page 7.2-67 of NAVFAC DM-7.2 (1982), reproduced herein as Figure 7.10. The engineer should be familiar with the use of this chart. It has been developed from Rankine Theory for continuous walls, and, therefore, is felt to generally be conservative.

The proper utilization of this chart requires the input of  $\beta$ ,  $\delta$ , and  $\Phi$  where:

$\beta$  = Inclination from horizontal of either the backfill slope or the toeslope;

$\delta$  = Friction angle between the structure and the soil;

$\Phi$  = Angle of internal friction of the soil.

$\delta$  is usually considered to be approximately equal to  $.33 \Phi$  for smooth walls such as reinforced concrete walls poured to timber form work, sonotubes, or sheet-pile walls.  $\delta = 0.67 \Phi$  for structures cast-in-place directly against the soil (such as drilled piers). The placement of a foot or more of gravel behind the wall creates the need for averaging the  $\Phi$  values of the gravel and the backfill with respect to the wall.

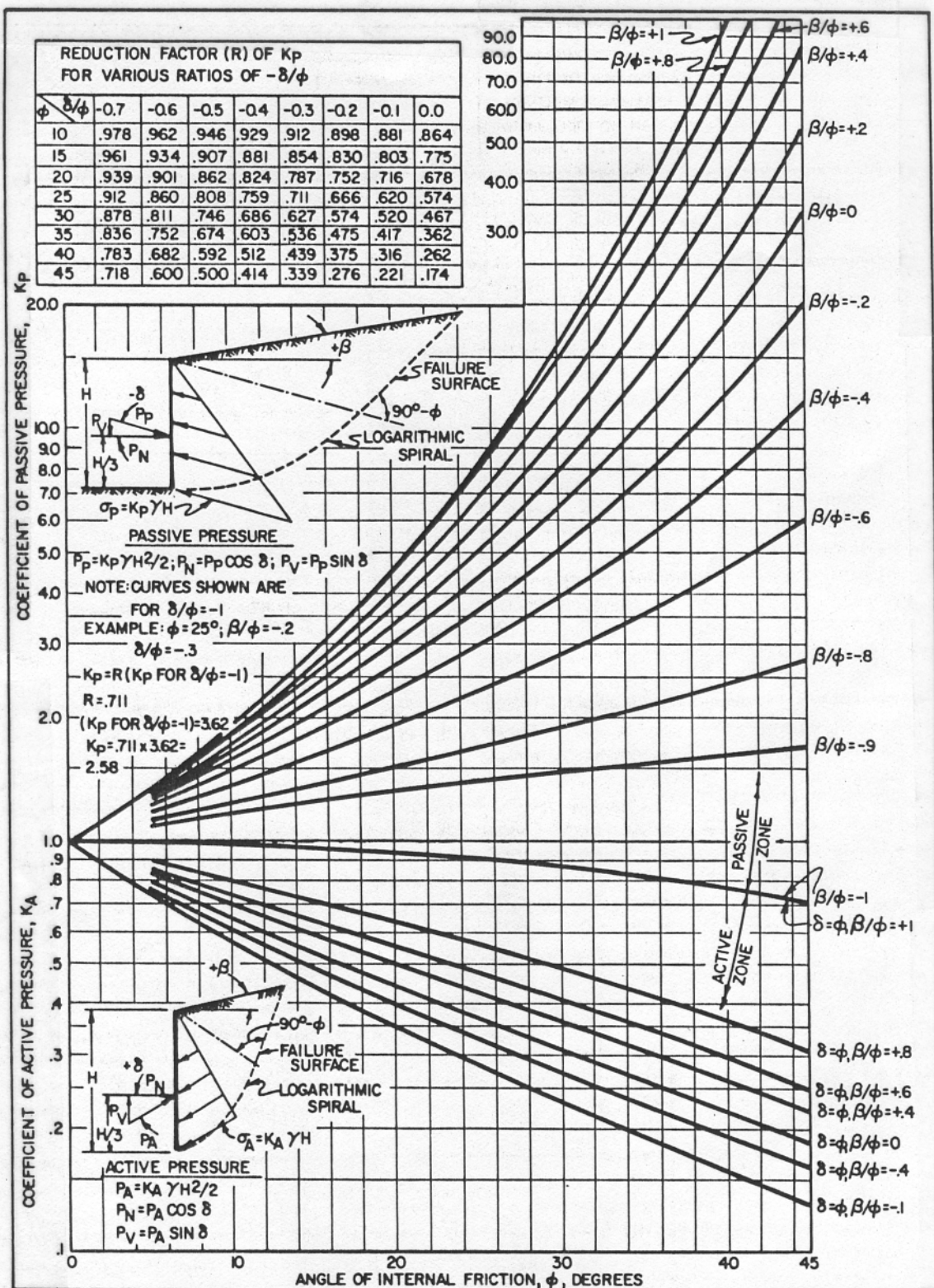


Figure 7.10 - Design nomograph for determination of active and passive coefficients with wall friction,  $\delta$ , and sloping ground. Taken from NAVFAC DM-7.2 (1982).

## **8.0 Earth Pressures on Vertical Retaining Wall Elements**

Cylindrical drilled piers which have been instrumented show that active pressure loads arch between the pier spacings and load the pier accordingly (see Terzaghi, 1943, pp. 66-76; Wang and Yen, 1974; and Bosscher and Gray, 1986). As a consequence, active loads on these piers should be calculated for a tributary area equal to the center-to-center spacing of the piers. Cantilever piers can be designed for the full hydrostatic active pressure distribution taken to the bottom of the pier in normal, long-term loading situations. In landslide situations, the active loading zone can be taken to the depth of sliding and no deeper. This is because the active deflection experienced by such walls is usually confined to the failing soil mass. Care should be taken in these situations to insure that the slide mass is not reactivated during construction, or the wall must be designed to carry the entire thrust of the slide mass. Practice has shown that the optimum C-to-C pier spacing for a retaining structure is between 2.0 and 3.0 times the pier diameter,  $D$ . A spacing of around  $2.5D$  is considered optimum for reasons of passive resistance to be discussed shortly.

Cylindrical piers can be used to best advantage on the passive side where their independent action is spread out over the slope in a fan shape. This fan shape can be approximated by drawing limits flaring out from the pier sides at  $45 - \phi/2$  degrees from the line of the imposed load. Comparison of schematic diagrams of the passive wedges generated by each pier in both plan and section will show that:

1. There is a significant reduction of the soil mass resisting lateral motion of the pier in the section view.  
  
This reduction can be as much as 50% or more on significantly steep slopes (45 degrees).
2. The passive wedges engendered by the piers overlap one another, thereby negating a portion of the passive resistance in a plan view.

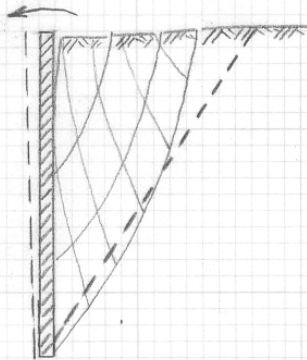
Because of the overlap of their respective passive wedges, passive pressures can only be increased to twice the pier diameter for center-to-center spacings between 2.0 and 3.0.

## **9.0 Example Calculation of Active and Apparent Earth Pressures**

Rankine Active Earth Pressure Theory is a conservative linear approximation of the actual log-spiral shaped active and passive soil mobilization zones (see Terzaghi, 1943, pp. 44, 67, and 84). Rankine Pressure Theory is presented in Figure 9.10. The one instance where Rankine Pressure Theory may not be adequate is that of temporary braced excavations. In such cases, the straight line (triangular) Rankine soil pressure distribution may severely underestimate loads near the top of an excavation wall and severely overestimate the load near the bottom of the braced excavation (see Terzaghi and Peck, 1967; pp. 193-299).

# RANKINE EARTH PRESSURE THEORY

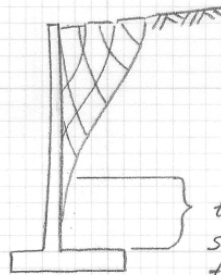
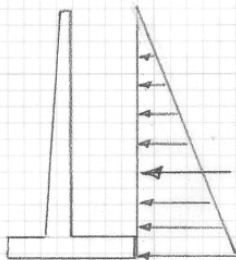
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## Active Earth Pressures

- generated when soil mass expands and strains downward due to free tilt of retaining wall
- Rankine Pressure envelope inclined at  $45 - \phi/2$  degrees from vertical, taken from lowest element of structure

$$P_A = \frac{1}{2} K_a \gamma H^2$$



- In actuality, only the top  $2/3$  of a conventional unrestrained wall stem is able to deflect outward.

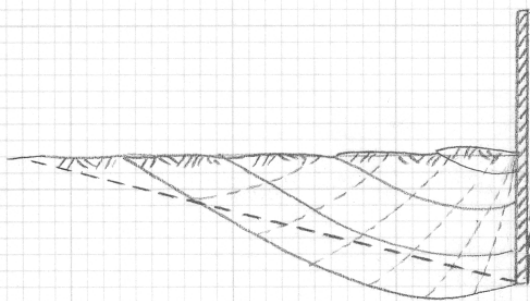
the lower third of the wall is sufficiently stiff to deflect the req'd  $.002H$  to  $.02H$  this area feels At-Rest pressures.

## At-Rest Earth Pressures

- If the soil is naturally deposited and not subject to dilation or compression, it is said to be between the active and passive states, or "At-Rest". The Coefficient of At-Rest Earth Pressure is given as  $K_0$ .  $K_0$  ranges from 0.40 for dense sand up to 0.60 for loose sand. Well compacted fill may increase  $K_0$  up to 0.80. Normally consolidated soil is taken to be  $K_0 = 0.50$ . Overconsolidated clay could range up to 0.90.

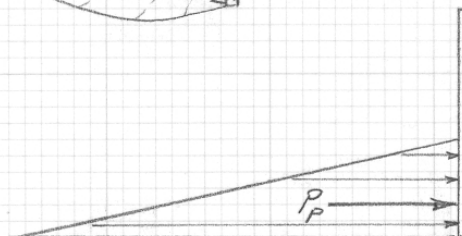
$$P_0 = \frac{1}{2} K_0 \gamma H^2 \quad \text{No } K_0 \text{ method is given for slopes.}$$

## Passive Earth Pressures



- generated when soil mass compresses outward due to wall pushing on it.

- Rankine pressure envelope inclined at  $45 + \phi/2$  degrees from lowest element of the structure.



$$P_P = \frac{1}{2} K_p \gamma H^2$$

Figure 9.10

In the 1940's, Terzaghi and Peck began to alter their earth pressure diagrams to more closely follow the actual lateral loads measured on deep-braced excavations in Chicago and Boston. This alteration is presented in Figure 9.11 and is known as Apparent Earth Pressures (Terzaghi and Peck, 1967, pp. 394-413). These relationships apply to temporary, braced excavations.

Long-term Apparent Earth Pressures on tied back or cantilever retaining walls can be estimated by calculating the triangular Rankine pressure distributions, then using equivalent polygonal areas within the various sub-strata units. This "straightening-out" of the triangular distributions produces a more rectangular loading diagram with increased loads near to top and decreased loads and moments in the lower, embedded portion of the wall. Example calculations of Rankine and Apparent Earth Pressures are included herein as Figure 9.12.

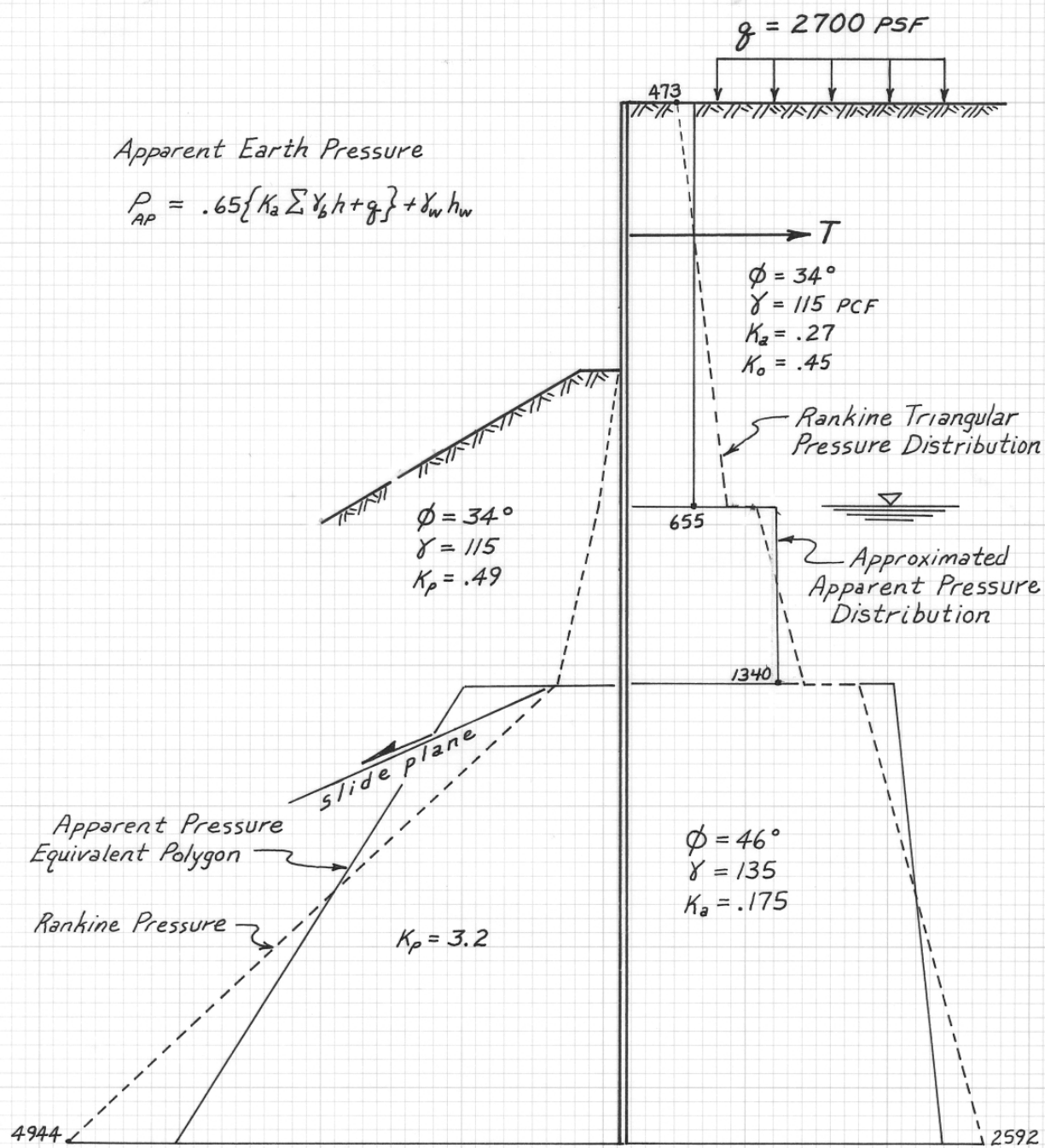


Figure 9.12



# TERZAGHI and PECK'S APPARENT PRESSURE DIAGRAMS

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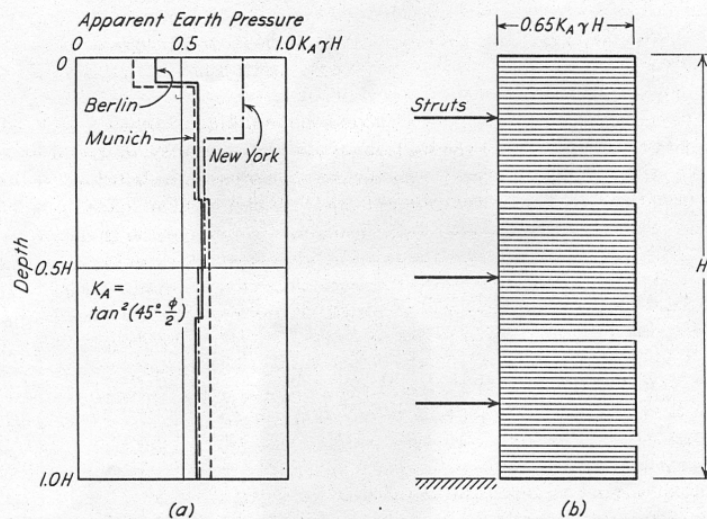


Fig. 48.5. (a) Envelope of apparent earth-pressure diagrams for measured strut loads in cuts in sand. (b) Suggested apparent earth-pressure diagram for design of struts in open cuts in sand.

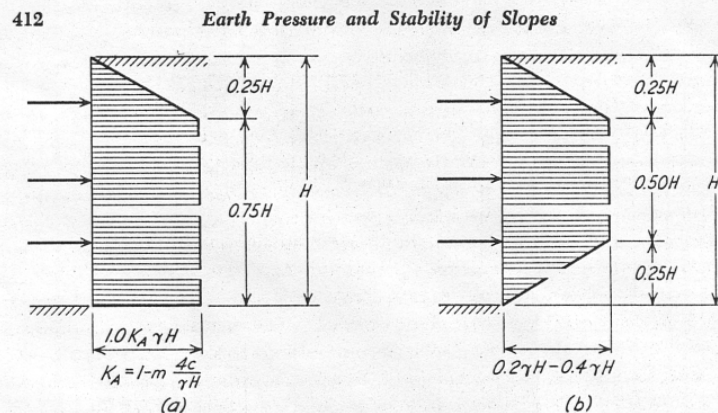


Fig. 48.11. Apparent pressure diagrams for design of struts in cuts excavated in clay soils. (a) Diagram for soft to medium clays; value of  $m$  to be taken as 1.0 except for truly normally loaded clays when  $N = \gamma H/c$  exceeds about 4, under which conditions  $m < 1.0$ . (b) Tentative diagram for stiff-fissured clays; lower pressure to be used only when movement can be kept to a minimum and construction time is short.

Figure 9.11 Apparent Pressure diagrams taken from Terzaghi and Peck (1967).

## REFERENCES

- Bosscher, P. J., and Gray, D.H., 1986, Soil Arching in Sandy Slopes: Journal of Geotechnical Engineering, ASCE, v. 112, No. 6, pp. 626-644.
- Navy, Dept. of, 1982, Foundations and Earth Structures: Design Manual 7.2 (NAVFAC DM-7.2), Naval Facilities Engineering Command (200 Stoval St., Alexandria, VA 22332), May, 1982, 244p.
- Terzaghi, Karl, 1943, Theoretical Soil Mechanics: John Wiley & Sons, New York, 510 p.
- Terzaghi, K., and Peck, R.B., 1967, Soil Mechanics in Engineering Practice, Second Edition: John Wiley & Sons, New York, 729 p.
- Wang, W.L. and Yen, B.C., 1974, Soil Arching in Slopes: Journal of Geotechnical Engineering Division, A.S.C.E., vol 100 (GT 1): pp. 61-78.