ABUTMENTS FOR SMALL HIGHWAY BRIDGES

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SYNOPSIS

The first part of this report contains a general outline of all factors relating to the design of abutments for small highway bridges The main portion of the report includes a discussion of one of these factors—Forces and Resistances. The elements considered under this heading are (1) Active Earth Pressure, (2) Passive Earth Resistance, (3) Foundation Pressures, (4) Bearing Value of Piles, (5) Uplift Resistance of Piles, (6) Lateral and Pull-Out Resistance of Soils, (7) Lateral Resistance of Piles, (8) Ice Pressure and Uplift, and (9) Scour of Water and Silt

DEFINITION AND LIMITATION OF SUBJECT MATTER

This report lists the latest consistent data available on the design of abutments for the support of short-span bridges carrying highway loads Where the abutment is self-supporting and does not require the presence of the bridge slab for support to resist the fill, it is classed as a gravity type, including not only gravity walls, but also solid crib boxes, braced or cantilever sheeting, and anchored bulkheads As a separate class of non-self-supporting walls there are various types which cannot be backfilled until the bridge slab or its framework is completed, including slab walls and legs of rigid frame structures The outline of the subject matter to be covered in the complete study follows, but only part A is discussed in this report.

A. *Forces and Resistances*
   (a) Active Earth Pressure
   (b) Passive Earth Resistance
   (c) Foundation Pressures
   (d) Bearing Value of Piles
   (e) Uplift Resistance of Piles
   (f) Lateral and Pull-out Resistance of Soils
   (g) Lateral Resistance of Piles
   (h) Ice Pressure and Uplift
   (i) Scour of water and silt
B. *Materials and Stresses*
   (a) Mass Masonry
   (b) Concrete (plain and reinforced)
   (c) Sheeting (steel, concrete, timber)
   (d) Cribbing (steel, concrete, timber)
C. *Types of Abutments—Methods of Design*
   (a) Gravity Walls
   (b) Cribbing
   (c) Counterforted Walls
   (d) Cantilever Walls
   (e) Sheeting
   (f) Open Frames
   (g) Hollow Boxes
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   (i) Anchored Bulkheads
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D. *Types of Foundations*
   (a) Spread Footing
   (b) Pile Footing
   (c) Column Footing
   (d) Processed Fills
E. *Bearing and Drainage Details*
   (a) Abutment Seat Details
   (b) Backfill Materials
   (c) Compaction of Backfill Materials
   (d) Stabilization of Backfill for Road Base
   (e) Drainage Methods and Details
F. Correlation of Soil Profile Studies and Investigations with above subject matter
G. Correlation of Report with Previous Studies of Standard Abutment Designs
H. Comparative Costs of Different Types for 25, 50, 75, and 100 ft. spans.

A. FORCES AND RESISTANCES

(a) *Active Earth Pressure* can be safely and economically determined by the following rules.

1. Horizontal component for any material for a normal type wall is closely given by the general wedge theory for the case of a vertical wall and substantially horizontal fill. Such wall is assumed to be backfilled by usual construction methods and is not so rigid that a small rotational movement (of the magnitude .001H) cannot occur. The necessary rotation to mobilize the internal friction of the backfill is equivalent to an outward movement of ¼ in. at the top of a 20-ft. wall. The horizontal component of lateral pressure, E_a, is

closely given for vertical walls and horizontal fills by the formula:

$$E_a = \tfrac{1}{2}\, wH^2 \tan^2 \left(45^\circ - \frac{\phi}{2}\right) = \tfrac{1}{2}\, wH^2 K_a \quad \ldots\ldots (1)$$

where w is the average unit weight of the filling in lb. per cu. ft.; H is the height of the fill in ft. and ϕ is the angle whose natural tangent is the coefficient of internal friction of the filling. The value of $\tan^2 (45^\circ - \frac{\phi}{2})$ is often designated by the symbol K_a.

TABLE 1
NORMAL VALUES OF W AND ϕ

Material	w (lb per cu ft)	ϕ (Internal Friction)
Soft Plastic Clay	105–120	0–10±
Wet Fine Silty-Sand	110–120	15–30
Dry Sand	90–110	25–40
Gravel	120–135	30–40
Loose Loam	75– 90	30–45
Compact Loam	90–100	30–45
Compact Clay	90–110	25–45
Cinders	40	25–45
Compact Sand-Clay	115–125	40–50
Water	62½	0

TABLE 2
Values for $K_a = \tan^2\left(45^\circ - \frac{\phi}{2}\right) = \frac{1 - \sin\phi}{1 + \sin\phi}$
and for $K_p = \tan^2\left(45^\circ + \frac{\phi}{2}\right) = \frac{1 + \sin\phi}{1 - \sin\phi}$

ϕ	$\tan\phi$	K_a	K_p
0	0	1 00	1 00
10	.176	70	1 42
20	.364	49	2 04
25	.466	41	2 47
30	.577	33	3 00
35	.700	27	3 69
40	839	.22	4 40
45	1 000	17	5 83
50	1.192	13	7 55
60	1 732	07	13 90
70	2 748	.03	32 40
80	5 671	.01	132.20
90	Inf.	0	Inf

Table 1 gives average values for w and ϕ

Table 2 gives values for $\tan^2 (45^\circ - \frac{\phi}{2})$ for various values of ϕ.

2. The general wedge theory formulas (taking into account the friction along the surface of the wall) may be used for the evaluation of horizontal component for all other conditions of wall batters and sloping fills. However, comparison with experimental results indicates that the results are somewhat too small for negative surcharges and somewhat too large for positive surcharges, but the differences are no greater than 10 per cent. It is quite accurate enough, taking into account the uncertainties of conditions as to actual slope, to use a table of ratios, referred to the simplified case of vertical wall and horizontal fill (see Table 3).

3. The vertical component of lateral pressure, in all cases, is such that the resultant pressure is inclined from the normal to the back of the wall by the angle of wall friction. However, under no condition can this angle exceed the angle of internal friction of the fill.

4. The pressure of materials which, because of lack of drainage and because of their nature may become fluid at any time, whether such fluid material is widespread or only a narrow layer against the wall, is the same as liquid pressure of a liquid having the same density as that material.

TABLE 3
LATERAL PRESSURE RATIOS FOR GENERAL CONDITIONS

For Horizontal Fill	and	*Sloped Walls*
107		+1 6
103		+1 12
100		Vertical
95		−1 12
90		−1 6

For Vertical Walls			and	*Sloping Fill*
$\phi =$ 40°	30°	20°		
123	110	109		+10°
100	100	100		0
74	83	88		−10°

All figures are percentages of values given by formula (1) for the case of vertical wall and horizontal fill and for a specific value of ϕ In computing the above values, the angle of friction between the wall and the fill is assumed at 30° or equal to the angle of friction ϕ, if $\phi < 30^\circ$.

5 The pressure of submerged soils is given by the same formula (1), with the weight of material reduced by buoyancy (for the solid fraction of the soil only) and the coefficient of internal friction evaluated for the submerged condition, and in addition thereto there is acting the full hydrostatic pressure of water. For granular materials, submergence affects the coefficients of internal and wall friction very little, submergence changes silt materials to liquids

6 The pressure of fills during saturation and prior to complete submergence and the pressure during drainage periods is affected by the rapidity of water movement. Drainage produces a slight temporary decrease in pressure from normal. Submergence produces an expansion of the fill with consequent in-

crease in pressure. Such variations will not occur if adequate provision is made for drainage, if such provision is not made, the soil may become submerged and the pressure should be computed as in paragraph 5 above.

7. The pressure of granular fills is affected by earthquake and other vibrations by an increase of approximately 10 per cent in value, which increase remains for a time and slowly disappears. Silt materials and some kinds of clay (thixotropic clays) under the action of earthquakes or vibrations by heavy trucking may become liquid.

8 The pressure of fills varies directly with temperature and normally decreases with age. Both factors can be safely disregarded in the design of abutments.

9. Surface loading of the fill increases the lateral pressure on the wall The assumption of a trapezoidal pressure distribution, by adding the pressure computed from formula(1) for a depth of earth equivalent in weight to the surface loading or surcharge, although not in complete accord with recent experimental work, is considered safe for abutment walls because of the unlikely occurrence of the full surcharge over the entire area of influence. In the case of liquid fill even though it has been drained but remained fluid, the moisture in the pores transmits the full weight of the surcharge to the wall. Hence in such cases that part of the horizontal pressure on the wall which is due to the surcharge, equals the full weight of the latter at all depths.

10. The resultant horizontal pressure acts somewhere between 0.33 and 0 45 of the height of the wall. Liquid and negative sloped fill pressures act at 0 33 of the height. Theoretical point of application of surcharged fills, based on the method in paragraph 9 above are listed in Table 4 Except for liquid and negative sloped fill pressures, the point of application should be assumed no lower than 0 375 of the height, and higher if the surcharge ratio so requires This assumption is on the safe side of the true condition sufficiently to compensate for any effect of vibration, age and local fill compaction The location of resultant pressure application affects the design of walls much more than corresponding accuracy in the evaluation of the amount of pressure.

11. The pressure in pits and bins is not given by the wedge theory unless a correction for side wall friction is made, in which case actual field observations of pressures are closely checked. Such correction shows that after certain depth to width ratios are exceeded, there is no addition in pressure, as long as the earth is prevented from movement For usual soils, there is no horizontal pressure at the base of a pit six times the width or more in depth.

12. Many attempts have been made to derive a consistent formula for the pressure of cohesive soils, including a reduction factor for the cohesive resistance. The most recent empirical evaluation in Chicago, indicates a formula of the form·

TABLE 4

THEORETICAL POINT OF APPLICATION OF PRESSURE FROM SURCHARGE FILLS

H = height of fill
S = equivalent height of surcharge weight
X = height of resultant above base, as a fraction of H

S/H	X	Total Pressure Ratio	
		Theoretical	Spangler
0	33	100	100
0 1	36	120	105
0.2	38	140	120
0 3	40	160	150
0 4	.41	180	210
0.5	42	200	290
0 6	42	220	
0 7	42	240	
0 8	43	260	
0 9	43	280	
1 0	48	300	

The Spangler ratios are based on data shown in Figure 10, p. 63, *Proceedings*, Highway Research Board, Vol 18, Part II, 1938, assuming normal unsurcharged pressure equal to 16 h² Since the exterpolation is from a single surcharge load (100 lb per sq. ft), no accuracy is to be expected for these values

$$K_a = 0.75 - \frac{q}{wH} \cdots\cdots \quad (2)$$

(See chart by R. S. Knapp and R. B. Peck in *Eng. News Record*, Nov. 20, 1941.)

where K_a is the hydrostatic pressure ratio to be used in formula (1)

$$E_a = K\frac{wH^2}{2}$$

w is unit weight of soil, in lb per cu. ft.

H is depth of excavation of fill, in ft.

q is average unconfined compressive strength of soil samples in lb. per sq. ft

(b) *Passive Earth Pressure*

1 The horizontal component of passive (or maximum) resistance to pressure before failure (termed also "passive pressure"), is often computed by the formula

$$E_p = \tfrac{1}{2} wH^2 \tan^2 (45° + \tfrac{\phi}{2}) = \tfrac{1}{2} wH^2K_p \quad . (3)$$

designations w, H and ϕ being the same as in formula (1), and $K_p = \tan^2 (45° + \frac{\phi}{2})$.

In common highway work formula (3) furnishes satisfactory results Experimental work, especially with sheet piling in sand shows, however, that actual values of passive resistance are larger than those given by formula (3). For the design of sheet piling embedded in sandy materials a maximum value of passive resistance equal to $2E_p$ may be recommended as permissible

2. The vertical component of the passive resistance is such that the resultant pressure is inclined from the normal to the back of the wall by the angle of wall friction

3. Little is known about the location of the resultant passive pressure, which is probably affected by the rigidity of the wall The usual assumption is that the distribution of the lateral resistance is linear

(c) *Foundation Pressure*

Permissible base pressures depend upon the nature of the soil, size and shape of the foundation, depth of subgrade and to some extent on the relative values of the surcharged live loads Because of the rigidity of abutments, linear distribution of base pressure may be assumed If soil bearing tests are made for the determination of permissible pressure for any given settlement, the depth of the test area must be comparable to the final design depth The increase cover from the backfilling, however, occurs on the side where base pressures are low and should not be considered Since all foundations must settle when loaded, abutments must be designed to permit such settlement (usually also tipping from unequal base pressures at toe and heel). For deeply bedded foundations, where there is no likelihood of deep scour or erosion, the lateral resistance of the soil on the toe side may be considered as a balancing force, tending to resist tipping and tending to reduce the inequality of base pressures In all cases, however, the requirements of statics must govern; the total resistances in any direction are equal to (not less nor more than) the total forces acting It is recommended that the foundation be treated separately in the analysis, and the design of an abutment be considered in two parts the retaining wall and the foundation.

(d) *Bearing Value of Piles*

The bearing value of piles should be determined from pile load tests If the cost of such tests is prohibitive, the bearing value in question is to be determined from local experience and may be estimated by using simple pile driving formulas such as the well known Engineering News formula

Besides the latter formula the so-called Hiley formula is sometimes used

$$P = \frac{R}{F} = \frac{eWh}{s+k} \times \frac{W + n^2M}{W + M} \qquad (4)$$

where

R is dynamic pile resistance

P is allowable load

F is factor of safety, a value of 3 being sufficient

W is weight of striking part of the hammer

M is weight of the pile

h is height of fall of striking part of the hammer

s is penetration of pile per blow

k is half the rebound of pile cap
 (1 00 for drop hammer)
 (0 1 for steam hammers)

n is coefficient of restitution
 (0 5 for steel hammer on steel or concrete)
 (0 4 for cast iron hammer on concrete)
 (0 25 for cast iron hammer on sound wood)

e is efficiency of hammer (0 75 for drop hammer and 0 90 for single acting steam).

For double acting hammer substitute for eWh the rated available energy at instant of impact.

The total value of a pile cluster is always less than the sum of individual pile values A reduction must be made when piles are spaced less than five diameters (of the butt) on centers A simple rule is to reduce the value of each pile by one sixteenth for each other pile less than five diameters away

For large operations, test piles should be driven and static load tests used for determination of correct bearing values If the load test is done by jacking the pile from a beam fixed to auxiliary piles, the latter should be at least 10 ft distant from the pile tested

The pile should be loaded at least with the design load; and the settlement under load as recorded after the removal of the test load should not be over 0.01 ft per ton of test load

(e) *Uplift Resistance of Piles; Friction Piles*

When piles are used for uplift resistance at the heel, the value of each pile, unless previously determined by proper full scale pulling test, can be taken as equal to 600 lb per sq. ft. of embedded surface unless the cohesive resistance of the material is smaller. This value may be also considered as average possible skin resistance of a friction pile. The correct way to determine the skin resistance of a friction pile is by making either an extraction or a loading to failure test The pulling out strength of tapered piles is practically equal to that of cylindrical piles having the maximum diameter of the tapered pile. The upward movement of the pile top at such loads is very little more than the elastic elongation and can be assumed to be not over 0.1 in.

(f) *Lateral and Pull-Out Resistance of Soils*

The lateral resistance of soils to pressure is covered in section (b) above.

The frictional resistance of soils along the base of the footing to translation, acts independently of the passive resistance along the toe It equals the product of the total vertical load imposed on the base and the coefficient of friction between the soil and the concrete (or other base material) if less than that of the earth No allowance should be taken of the cohesive resistance of the soil Since the subgrade is usually rough and the concrete poured directly against it comes into intimate contact with numerous projections, it is quite safe to assume that the coefficient of friction at the base is that of the earth itself.

The pull-out resistance of embedded anchors varies with the type of soil, and the size of the anchor but seems to be the same for all shapes of anchors having the same area of section normal to the direction of pull. The relative values of pull-out resistance for soils is.

1.0 for compact sand and stiff clay
0 5 for fairly soft clay and loose sand

For equal vertical depth of embedment, the ratio of resistance increases with the slope of the pull from the vertical, approximately as follows:

1.0 for vertical pull
1.5 for pull at 1 1 slope
2 0 for pull at 1.2 slope
2 1 for pull at 1.3 slope
2 3 for pull at 1.4 slope

The resistance to vertical pulls in rammed compact loam is.

800 lb per sq. ft anchor face at 1 ft. depth
1000 lb. per sq ft anchor face at 1 5 ft depth
1900 lb per sq. ft anchor face at 2 ft. depth
3000 lb per sq ft anchor face at 3 ft depth
5400 lb. per sq ft anchor face at 4 ft. depth
8000 lb per sq ft anchor face at 5 ft depth

These values should be used only for anchors where the depth of embedment is larger than the maximum dimension of the anchor

(g) *Lateral Resistance of Piles*

Customary values used in design of river works by the U. S Engineers for lateral maximum movements of $\frac{1}{4}$ in. are as follows

8000 lb. per wood pile
10000 lb. per concrete pile
12000 lb per steel H pile

when driven for the usual bearing values of the respective piles and when acted upon by repetitive lateral loads For static lateral loads, the values can be increased by 10 per cent For lateral maximum movements of $\frac{1}{8}$ in., the values should be reduced by 30 per cent.

(h) *Ice Pressure and Uplift*

Where abutments may be exposed to the pressure of heavy ice formed on water in direct contact with the face of the wall, the maximum pressure to be considered is 50,000 lb per lin ft, which is the crushing strength of block ice 1 ft thick In the design of dams with sloping faces, a value from 20,000 to 30,000 lb per lin ft. is usually used

If the base of the abutment may be below flood level, the uplift pressure equal to full hydrostatic pressure acting on the entire area of the foundation, must be taken into consideration in the stability analysis.

(i) *Scour of Water and Silt*

Where the underlying soil is subject to scour of water and silt, special study must also be made of possible weakening of the sub-soil by infiltration of water in addition to the physical removal of part of the bearing. Some lessons learned from the May 1942 floods in Pennsylvania were stated by J L Herber of

Penn State Hy. Dept. (*Civil Engineering*, Nov. 1942, p. 623) as follows·

"The flood, though disastrous, taught several important lessons Waterway areas at bridge sites in the hilly parts of northeastern Pennsylvania should be designed to carry maximal flood volume. To avoid damage to the approaching roadway, stream channels should be straight through the abutments. Channel widening and cleaning must be studied and kept up to date in the vicinity of the highways. Everything should be done to encourage the public to keep the stream channels

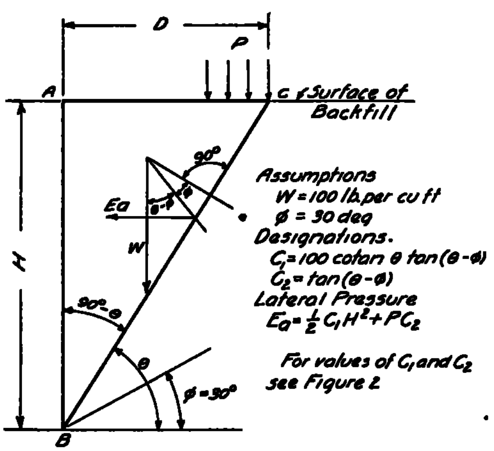
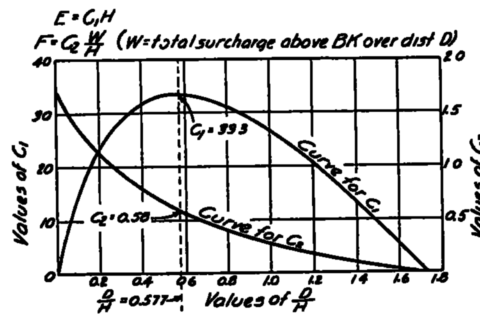


Figure 1. Lateral Pressure on a Wall (New York City Board of Transportation, 1932, by M. A. Drucker).

free from all encroachments and obstructions which might increase the danger during flood stages.

"Concrete decks or steel-pan structures may be submerged in strong currents and still survive if the foundations remain secure and the superstructures are not struck by heavy debris Outstanding examples of this were the concrete bridge at Bear Creek, Luzerne County, and the steel truss over Dyberry Creek at Upper Honesdale. At certain locations it may be well to design for unusual flood stages by depressing the roadway approaches so that the stream can flow over them, provided the berms and slopes of the roadway are properly protected with a paving which will withstand erosion."

Where likely danger of undermining exists, it might be considered economic practice to design the structure to be self-supporting when one abutment is partially or even completely undermined. This can be done by physical connections between bridge and abutments with the wing walls acting as stay anchors or else providing special anchorage.



Angle θ deg	Angle 90-θ deg	Angle θ-ϕ deg	D/H tan 90-θ	C_2 tan θ-ϕ	C_1 100 x tan(90-θ) x tan(θ-ϕ)
90	0	60	0	1 7321	0
85	5	55	0 0875	1 4281	12 5
80	10	50	0.1763	1 1917	21 0
70	20	40	0.3640	0 8391	30 6
60	30	30	0.5773	0.5773	33.3
50	40	20	0 8391	0 3640	30.6
40	50	10	1 1917	0 1763	21 0
35	55	5	1 4281	0.0875	12.5
30	60	0	1 7321	0	0

Values for D/H, C_1 and C_2

Figure 2

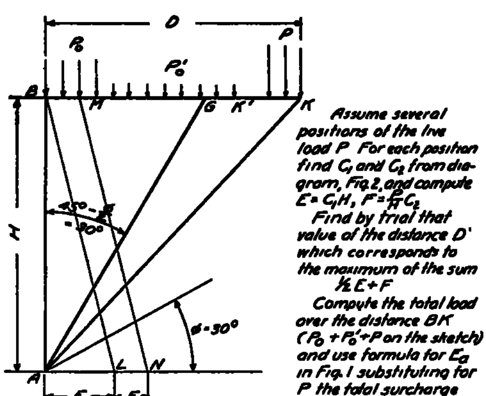


Figure 3. (After Drucker, 1932)

SPECIAL EXAMPLES

Some examples of methods employed for design under special conditions are:

(1) Lateral pressure diagram used in

design at Panama Canal (Cornish, ASCE Trans. Vol. 81, 1917) is given in Figure 4.

(2) Methods for computing lateral pressure on the walls of subway structures to include concentrations from buildings, etc., in New York are shown on Figures 1, 2, 3 and tables.

(3) The usual equivalent liquid pressures used in subway design in New York are 33 lb. for dry soil and 95.5 lb. for soil below water level.

(4) In Philadelphia, the usual values are 25 lb. above water level and 62½ lb. below water level. It must be noted that the New York designs are based on the assumption of complete waterproofed structures, while the Philadelphia structures have the fills drained and the assumption is made that 60 per cent of hydrostatic pressure acts.

(5) In general the U. S. Engineer Department uses 33 lb. above water and 83 lb. below the water level.

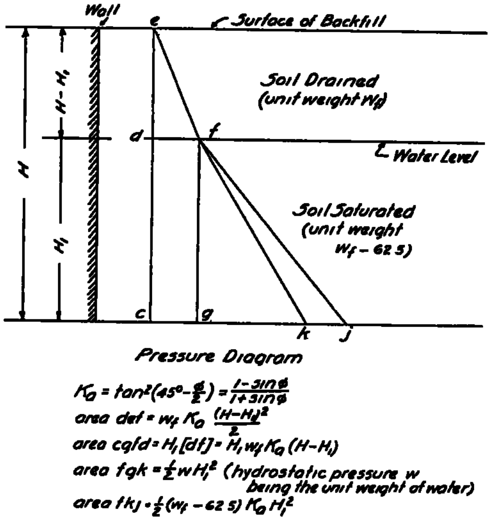


Figure 4. Method used at Panama Canal (After Comish, 1916)

DISCUSSION ON ABUTMENTS

Mr. V. T. Boughton, *Associate Editor, Engineering News Record:* I would like to see something about vibrations due to heavy trucking With any irregularity of pavement surface such as an open expansion joint in the pavement on the fill behind the abutment, vibration may be quite an item. It seems to me that the chance of such vibration developing is much greater than the chance of having to provide for vibration due to earthquakes.

It may be outside of the scope of the committee, but it seems to me that this section might also include some parenthetical statement about vibration as affecting the abutments of railway bridges. While this is a Highway Research Board study, nevertheless it seems obvious that the railway engineers may use this document to help them in the design of their structures

I question the advisability of including Section "i" in this discussion If a discussion on this subject is to be included, I recommend limiting it to the first sentence and the last two of section "i." The quotation about size and shape of waterways does not appear to me to be pertinent to this subject. It seems to me that questions of possible scour and channel area must be considered in the preliminary design of the bridge.

Mr. A. W. Bushell, *Deputy Commissioner, Connecticut State Highway Department* (in cooperation with Mr. L. G. Sumner) states. It is agreed that results from formulas are not satisfactory, when applied to sloping fills. However, the statement if I understand it correctly, that a 10 per cent variation in pressure as computed for level backfills is sufficient to take care of all usual conditions of slope would seem somewhat dangerous.

The need for proper drainage because of the greatly increased pressure due to saturation of the fills serves to remind us of the importance of this feature

Statements in the report regarding the effect of surcharge loads and points of application of resultant pressure are in general conformity with our present practice.

The formula given for the pressure of cohesive soils is noted with interest, but until closer cooperation between designers and soils laboratories exists, it seems doubtful if methods involving such factors as soil density and its average unconfined compressive strength will receive very wide acceptance

Passive earth pressure and foundation pressure offer nothing of a controversal nature but the pile loading formula (4) seems somewhat complicated. It is my experience and, I think, that of many others, that all such values may be very unreliable An article in the Nov. 18, 1943 issue of *Engineering News-Record* states.

"Piles were driven to a resistance that by any of the commonly used bearing capacity formulas, based upon the 'set' produced, would have safely supported 70 tons, yet many of the piles settled several inches under loads of 35 or 40 tons"

Uplift resistance of piles seems to me of doubtful use in design since its value is uncertain and to develop it requires careful anchorage of pile to footing Except in unusual cases, the need to rely on such values can be eliminated by a modification in footing dimensions and pile layout.

Data on lateral resistance of piles are useful and are something upon which but little information is available. The values given are for movements of ½ in. only with a statement that for ¼ in movement the values should be reduced by 30 per cent Are these for average conditions of pile length and materials penetrated? It would seem that this section might be elaborated somewhat

The statement that "where likely danger of undermining exists, it might be considered economic practice to design the structure to be self-supporting when one abutment is partially or even completely undermined," leaves a large question in my mind, first, as to just how this is to be done with assurance of success, and second, as to the justification for a design which admits "likely danger of undermining."

MR C. N CONNER, *Chairman, Department of Design, Highway Research Board:* I believe it would be unwise to rely on the passive lateral resistance of the soil on the toe side of the abutment if there were any likelihood of severe scour or deep erosion in that area. A word of warning to that effect would be desirable

Paragraph "e"—Uplift Resistance of Piles It is understood from the text that piles may be counted on to take tension as well as compression. If this is the case it seems to me a definite statement to that effect would be helpful Also somewhere in the report there should be an example illustrating the character and distribution of pressure on piles under an abutment for different positions of the resultant of all forces acting at the elevation pile cut-off.

Paragraph "g"—Lateral Resistance of Piles. Since all soils do not offer the same lateral resistance it seems to me there should be a statement to that effect or values should be inserted if available.

Special Examples—The analysis of pressures resulting from saturated soils is particularly important and interesting. It so happens that at one time I worked with Mr. Cornish on the design of masonry for locks and dams along the Ohio River At that time the general method of determining external pressures as developed by Mr Cornish was used in the design of retaining walls on the land side of locks built by the U. S Engineering Department. Insofar as I have information, all of these walls have given satisfactory service.

PROF R. G. HENNES, *University of Washington:* I believe enough references should be cited to enable the reader to gain a better background than can be provided in the body of the report This is especially true where quantitative recommendations are made.

The use of Table 3 in connection with Article Aa2 would be simplified by a sketch showing positive and negative slopes and batters. For greater clarity Table 3 could be split into two tables, one for batter and one for slope

Article Aa3 provides little aid to the designer in selecting a value of wall friction Terzaghi's M I.T wall tests suggest that this value is especially erratic Where the physical facts are obscure, as here, the designer is most in need of the competent professional judgment that the Outline is intended to provide Perhaps a recommendation to use some small but definite percentage of ϕ would be the best answer for general design purposes, say, 20 per cent of ϕ for a wall with vertical back and no projecting heel. As a tentative proposal for wall with batter, and/or projecting heel, the slope of the resultant earth pressure might be taken as 20 per cent ϕ plus the acute angle included between the vertical and a line joining the back of the wall crest with the extreme corner of the base. A more rational criterion would be better, but in any case some specific recommendation should be made.

Article Aa8 sounds somewhat dogmatic, although I am not prepared to offer a substitute recommendation.

Is Article Aa11 pertinent to abutment design?

The inclusion of Section Ab, Passive Earth Resistance, appears warranted mainly by its relation to the general topic through its bearing on sheet piling design If such is the case, the section should be amplified to cover the computation of sheet piling penetration, if only to the extent developed in the publications of steel manufacturers.

Section Ac leaves the designer with the following questions unanswered:

1. How should allowable base pressures be determined?
2. What, if any, laboratory tests should be made?
3. What, if any, field tests should be made?
4. How are test results to be applied to design?

Uplift would seem to be more appropriately considered under Section Ac than under Section Ah.

PROF. W. P. KIMBALL, *Dartmouth College*, formulated quite a few valuable remarks and most of them have already been taken into account in editing the report, especially his opinion about both the chairman's introduction and Hiley's pile driving formula. However, in view of the special general interest of Professor Kimball's discussion, the largest part of it is quoted literally hereafter.

"It is not entirely clear to me just what we are trying to accomplish by this work I think the aim should be more definitely stated and emphasized in the introduction Is this theory? Is it general practice? Is it good practice? Is it handbook material or textbook material? Is it a compilation of what should be done, what is being done, or both? It seems to me the paper breaks away from the usual soil mechanics discussion by setting forth actual formulas, rules and figures which the designer can use. I am very much in favor of doing just this whenever our knowledge justifies it. Too much of our soil mechanics has been devoted to showing how complicated the problems are without giving the designer anything he could really put into practice If this committee believes that the time has come to stop beclouding some of these issues and that we are prepared to give the designer something he can really apply, I think the introduction should state this belief clearly.

"Following up these thoughts somewhat, I can't help wondering what is the source of some of the statements and values in the report. It may be that where authorities are available they should be indicated by footnotes, or by reference to a bibliography at the end of the paper. This might increase the value of the paper considerably For example, the part with which I am most familiar is the discussion of pile foundations I feel that equation (4) does not represent either common practice or good practice, and the statements following equation (4) don't improve the presentation any This, of course, shakes my confidence somewhat in the remainder of the paper. If authorities were stated, it would help the reader to judge the worth of individual parts.

"As indicated by the paragraph above, the section on 'Bearing Value of Piles' is not acceptable to me.

"I do not think the long quotation in the section on 'Scour of Water and Silt' is appropriate or consistent with the rest of the paper. Here, however, as in paragraph 12, references are given This seems like a good idea and one which should be enlarged on in other sections."

MR L. A PALMER, *Senior Engineer, Bureau of Yards and Docks, U. S Navy Department*, proposes the following approximate method of determining the active pressure E_a in the case when besides the weight of the wedge W, there is a surcharge P located at a certain distance, d, from the retaining wall (or sheet piling) AB. (Fig. 5)

The weight of the wedge E_a causes the driving downward force along CB equal to $W \cdot \sin\theta$ At the same time the restraining force, acting upward along the direction CB, equals $[E_a \cdot \cos\theta + W \cdot \cos\theta + E_a \sin\theta] \tan\phi$, where $\tan\phi$ is the coefficient of internal friction of the earth material along CB. From the condition of equilibrium:

$$E_a = \frac{(W_e + P)(\tan\theta - \tan\phi)}{1 + \tan\theta\cdot\tan\phi}$$

$$= \frac{(W_e + P)(n - m)}{1 + nm}$$

$$= \frac{W_e(n - m)}{1 + nm} + \frac{P(n - m)}{1 + nm} = E_a' + E_a'' \quad (5)$$

The symbols n and m in Formula (5) designate $\tan\theta$ and $\tan\phi$ respectively. It is obvious

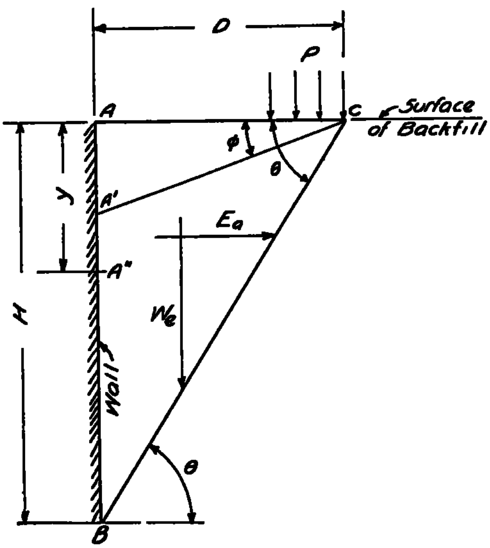


Figure 5

that the first member in Formula (5), namely $E_a' = \frac{W_e(n-m)}{1+nm}$ expresses pressure due to the wedge W_e, whereas the second (E_a'') corresponds to the pressure due to the surcharge.

As to the distribution along the height of the wall H (Fig. 5) of the total pressure E_a as furnished by Formula (5), Mr. Palmer proposes linear distribution for the pressure due solely to the weight of the wedge W_e. This part of the total horizontal pressure equals $\frac{W_e(n-m)}{1+nm}$. The maximum unit pressure due to the weight of the wedge W_e will be at the base of the wall, its value being equal to:

$$\frac{W_e(n - m)}{1 + nm} \div \frac{wH}{2} = \frac{2W_e}{wH}\cdot\frac{n - m}{1 + nm} \dots \quad (6)$$

where H is the height of the wall (Fig. 5) and w—the unit weight of earth material.

There is no pressure on the wall due to the surcharge P between points A and A' (Fig. 5), where point A' has been obtained by drawing the straight line CA' making an angle ϕ with the horizon. For any point A'' between A' and B, located at a distance y measured vertically downward from A, the pressure due to the surcharge P will be:

$$E_a'' = \frac{P\left(\frac{y}{D} - m\right)}{1 + m\cdot\frac{y}{D}} \dots\dots\dots\dots (7)$$

The unit pressure due to the surcharge at a point located at a depth y below point A, is obtained by differentiating E_a'' with respect to y.

$$\frac{dE_a''}{dy} = \frac{P}{D}\cdot\frac{1 + m^2}{\left(1 + m\cdot\frac{y}{d}\right)^2} \dots\dots\dots (8)$$

This formula is applicable to points from A' to B only. If the unit pressure $\frac{dE_a''}{dy}$ is plotted against y, a curve strikingly similar to the Boussinesq distribution for a concentrated load is obtained The magnitude of the horizontal pressure due to the surcharge P is, however, larger than that computed after Boussinesq for the case when the wall is replaced by earth. In the preceding derivation the angle of friction between earth and wall is taken equal to zero which is on the side of safety.

In general the shearing plane BC passes to the edge C of the surcharge P as in Fig. 5 if $P \geq W_e$. If $P < 4W_e$, a few trial values of the angle θ are taken by drawing a straight line through point B. The corresponding full values of E_a are then determined, and its maximum quickly approximated.

The foregoing procedure is described for cohesionless soils only. It has considerable application in waterfront structures where sand often predominates.

Mr. Palmer also states "Again, the wealth of material pertinent to the subject of retaining walls, bulkheads, etc., that Dr. Terzaghi has provided the profession should be utilized. Particularly is this true in the consideration of passive earth resistance It is accurate enough to use Coulomb's development for active pressure, but for passive pressure the combination log spiral-plane shearing surface should be assumed."

PROF. G. P. TSCHEBOTARIOFF, *Princeton University:* My general impression is favorable, although similarly to Professor Kimball, I feel that this "Summary" appears to have gone far beyond the scope of stress distribution studies. In fact, the majority of phases of applied soil mechanics have been touched upon. The question arises whether it is advisable in a necessarily abbreviated form. In the affirmative, a very careful study of all details would have to be undertaken.

RESEARCH ON SOIL STABILIZATION

BY MO CHIH LI

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SYNOPSIS

On most of the highways in China which were designed and built for the light traffic before the war and on those which have been hastily built during the war to keep pace with the needs of military operations, the road surfaces have proven to be inadequate to carry the continuously increasing, heavily-loaded truck traffic. On some stretches of the highways which are of considerable military importance the maximum daily traffic is over 1,000 vehicles, of which 85 to 90 per cent are trucks.

It is apparent that, by improvement of the existing road surfaces of the 10,000 miles of trunk military highways, millions of dollars could be saved annually by reduction of gasoline consumption, tire wear, replacement of spare parts, depreciation, and other factors pertaining to the cost of vehicle operation and road maintenance Owing to the necessity of improving the highways, the National Tsing Hua University has been co-operating since December 1939, with the Bureau of Highways of the Ministry of Communications in carrying out an extensive highway research project in which soil stabilization is one of the most important problems.

In dealing with the specific problems in accordance with the prevailing local conditions there are some differences which must be borne in mind, but the underlying principles of soil stabilization are the same One of the main differences is to obtain immediately a serviceable surface course, while the prevailing practice in the United States is to use a stabilized base course which, sooner or later, will be surfaced. Economic conditions in China will prevent improvement to a higher type of surface in the near future. In addition to the use of soil stabilization in its strictest sense, the soil binder of clay-bound macadam surfaces, which are the typical type of road surface in China, must also be stabilized Although the clay-bound macadam is an all-weather surface, it tends to become dusty in dry weather and muddy in rainy seasons

Another difference is that use must be made of cheap, local stabilizing agents, such as burnt-clay, quick lime, cinders, tung-oil, etc., instead of cement, asphalt emulsion or oil, tar, calcium chloride, sodium chloride, etc , which are commonly used in the United States Research work on soil stabilization has been carried on under adverse conditions and must be strictly limited to local materials. The application of the science of soil stabilization will mark a new era in road-building in China.

NECESSITY OF SOIL STABILIZATION

Roads in China fall, generally, into five types: earth, sand-clay, gravel, untreated macadam and bituminous surface treatment, with the earth and untreated macadam predominating. When both the soil and climatic