A Rational Method for Determining Safe Foundation Pressures and Embankment Stability

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The purpose of this paper is to describe the application by the Wayne County Road Commission of Housel's[1] method for determining safe foundation pressures and stability of sloping embankments in cohesive soils. The county had been using his method in designing foundations for highway and railroad grade separation structures since the late twenties with satisfactory results.

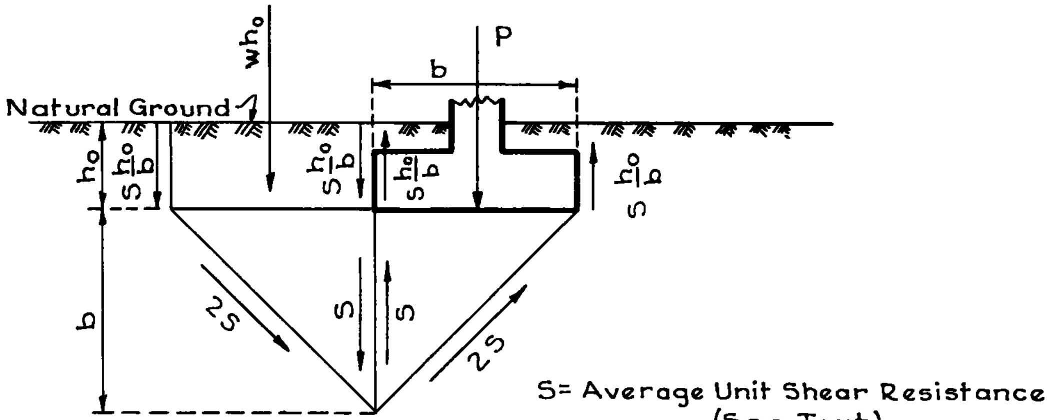
However, at the conclusion of World War II, the county had undertaken an extensive program of expressway construction necessitating a fivefold increase in the engineering staff. The young engineers available at that time had little or no experience in the art of soil mechanics, and a bottleneck was created whenever they had to determine the allowable foundation bearing pressures for the structures they were designing. To obtain uniform and safe design computations, the writer had undertaken to standardize and simplify the design procedure. Thus, once the laboratory had furnished the shear values of the various soil strata in the test borings, the designer would arrive at the correct answer without having to rely on his own judgment. This does not mean, of course, that inexperienced designers were relied on to make major decisions on the type of design to be used. This did, however, provide the squad leader or chief designer a basis on which to exercise his judgment.

The writer will readily concede that a number of simplifications have been made to avoid the complexity of more theoretical treatments and that some of the assumptions may be difficult to prove. However, in the 12 years that this procedure has been used, it has amply demonstrated its reliability. It has afforded a simple and rational method of evaluating the benefits of struts, subbases, ties and permanent steel sheet pile cofferdams around footings in the design of highway and railroad structures; it has eliminated the need for costly piles under foundations, where piles have previously been used; and it has enabled safe embankment slopes to be determined, by a few simple calculations. On a few occasions, soil samples have been taken, adjacent to old structures designed without the benefit of Housel's theory and it was found that where his formulas indicated safe bearing pressures lower than the actual, the structure has actually undergone some progressive settlement.

Before proceeding with the description of the procedure and with some typical examples of actual designs, a brief description of Housel's basic equations will be given.

ALL THE factors of resistance determining the ultimate bearing capacity of a strip footing in cohesive soil are shown in Figure 1. These are composed of the developed pressure in the compression block immediately below the bearing area ($4S$); the lateral distribution below the bearing area ($2S$); the resistance to upheaval ($2S\frac{h_o}{b}$); the perimeter shear resistance ($2S\frac{h_o}{b}$), and the static head (wh_o). For square or round footings the lateral distribution and the perimeter shear resistance is doubled ($4S$ and $4S\frac{h_o}{b}$).

[1] Professor of Civil Engineering, University of Michigan, Research Consultant, Michigan State Highway Department.



For strip footings,

$$\frac{P}{A} = wh_o + 6S + 4S\frac{h_o}{b} \tag{1}$$

For square or round footings,

$$\frac{P}{A} = wh_o + 8S + 6S\frac{h_o}{b} \tag{1a}$$

Figure 1. Basic equations for bearing capacity.

The unit shear resistance S is obtained by means of a transverse or ring shear test as developed by the University of Michigan Soil Mechanics Laboratory. It is a measure of the shear stress greater than which the soil will suffer progressive deformation and is approximately equal to one-quarter the shear resistance value obtained by the unconfined compression test.

Basic Equations for Stability of a Sloping Embankment

Figure 2 shows the pressure intensities at a depth h_o, acting on the principal planes of an elementary cube of cohesive soil of unit dimensions. The shear on the maximum shear plane is upward if P_v is greater than P_h, and downward if P_v is less than P_h. From mechanics of materials,

$$P_h = P_v - 2S = wh_o - 2S, \quad \text{when } P_v > P_h \tag{2}$$

and

$$P_h = P_v + 2S = wh_o + 2S, \quad \text{when } P_v < P_h. \tag{2a}$$

Eq. 2 represents the intensity of the active earth pressure at any point on a vertical plane through the top of slope. Eq. 2a represents the intensity of passive earth pressure offered at any point on a vertical plane through the toe of slope. The total active pressure above plane B (at a depth d below the bottom of slope plane A) is resisted by the total passive pressure between planes A and B, plus the resistance to sliding mobilized on plane B for a length of L + 2d. Referring to Figure 3, L is the horizontal width of the slope, while 2d represents an additional length on plane B assumed to be effective in resistance to sliding and is due to the lateral distribution of the vertical load. This lateral distribu-

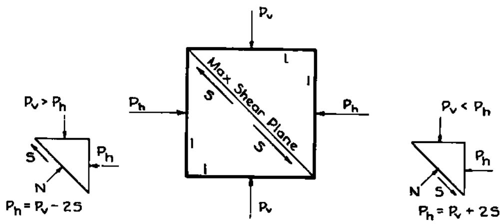
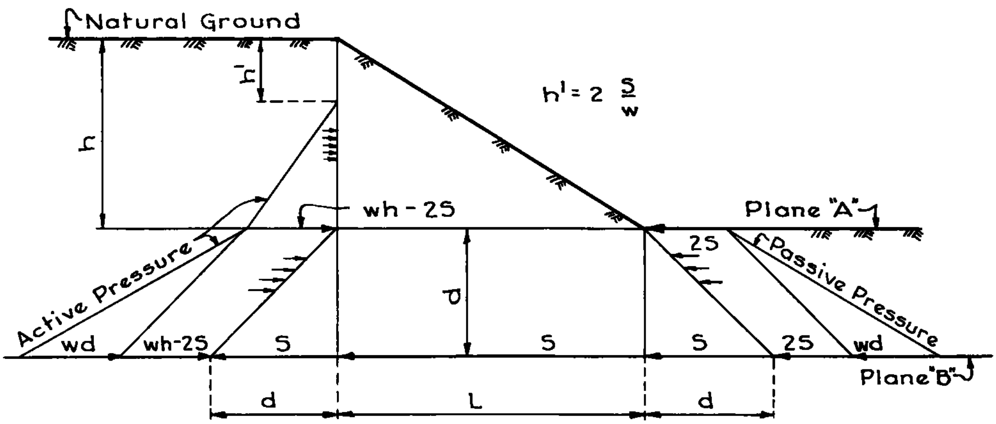


Figure 2.



Active Pressure = $\frac{w}{2}\left(h-2\frac{S}{w}\right)^2 + (wh-2S)\,d + \frac{wd^2}{2}$

Passive Pressure = $2Sd + \frac{wd^2}{2}$

Shear Resistance = $(L+2d)S$

The minimum length required for a stable embankment is determined by equating the active pressure to the passive pressure plus the shear resistance and then solving for "L"

$$\frac{w}{2}\left(h-2\frac{S}{w}\right)^2 + (wh-2S)d + w\frac{d^2}{2} = 2Sd + w\frac{d^2}{2} + (L+2d)S$$

$$L = \frac{\frac{w}{2}\left(h-2\frac{S}{w}\right)^2 + (wh-6S)d}{S} \tag{3}$$

Figure 3. Stability of a sloping embankment (based on Housel's method).

tion is assumed to take place below plane A on a 1 : 1 slope, and the horizontal pressures shown in Figure 3 are, therefore, applied on 45 deg planes instead of the vertical planes mentioned before.

The equations derived from Figure 3 are based on Housel's original method of analyzing the stability of sloping embankments. The resistance due to lateral distribution below plane A has also been derived by the so-called element method following the procedure used in derivation of the bearing capacity of spread footings (2). The mass stability analysis of embankments has also been modified by Housel to include the additional vertical shear resistance mobilized along the vertical plane of a potential failure surface (3, 4). However, it is sufficient to use Figure 3 in its original form and obtain the basic equations for slope stability therefrom. For a later and more detailed description, see references at the end of the text.

Overload Ratio vs Safety Factor

The shearing resistance in unconfined compression tests is taken as one-half the unconfined compressive strength, and a safety factor of four is used to determine the safe shearing resistance. Since the unit shear resistance obtained from ring shear tests has a value of one-quarter of the above, a safety factor of four is inherent in the term S in the Housel equations.

The relationship between the desired safety factor of four, based on the allowable shear values, to the actual safety factor resulting when higher than allowable shear values are used is termed the Overload Ratio. Thus,

$$\text{Overload Ratio} = \frac{4}{\text{Factor of Safety}} = R$$

The Overload Ratio, R, is applied as a coefficient of S in all the preceding equations.

Table 1 gives the overload ratios as recommended by Housel and corresponding factors of safety recommended by Terzaghi and Peck (5) with reference to the ultimate shearing resistance from the rapid unconfined compression test.

TABLE 1

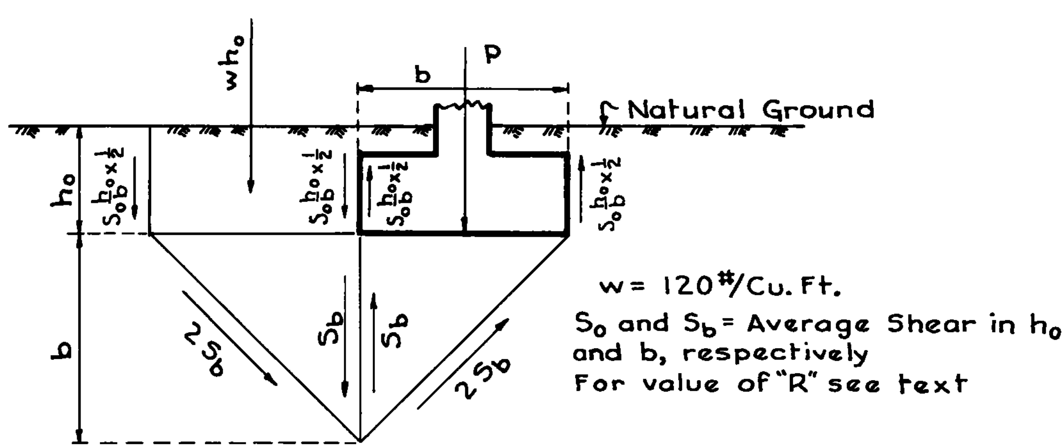
HOUSEL'S RECOMMENDED OVERLOAD RATIOS

Description	Overload Ratio (R)	Factor of Safety (F S)
Permanent structures	1.00	4.00
	1.33	3.00
Temporary structures	2.00	2.00
	2.50	1.60
Failure condition	4.00	1.00

PROCEDURE FOR DETERMINING ALLOWABLE FOUNDATION PRESSURES

In determining the allowable foundation pressures on strip footings, the values of the perimeter shear resistance and the resistance to upheavel have been arbitrarily reduced by 50 percent, since the excavated area is frequently larger than the footing area, and the backfill material cannot be relied upon to offer appreciable shear resistance (see Fig. 4). Two values have also been established for the Overload Ratio, R, to be used in the equations: a value of R = 1 for pressures resulting from dead load plus overturning, and a value of R = 1.5 for pressures resulting from a combination of dead load plus overturning plus live load. This results in a safety factor of 4 and 2.67, respectively.

If, as sometimes happens, the shear values in the soil strata at 5 or 10 ft below the bottom of footing permit higher pressures, steel sheet piling is driven to that depth and the bottom of the piling is considered as the bottom of the footing plane. The sheet



For strip footings,

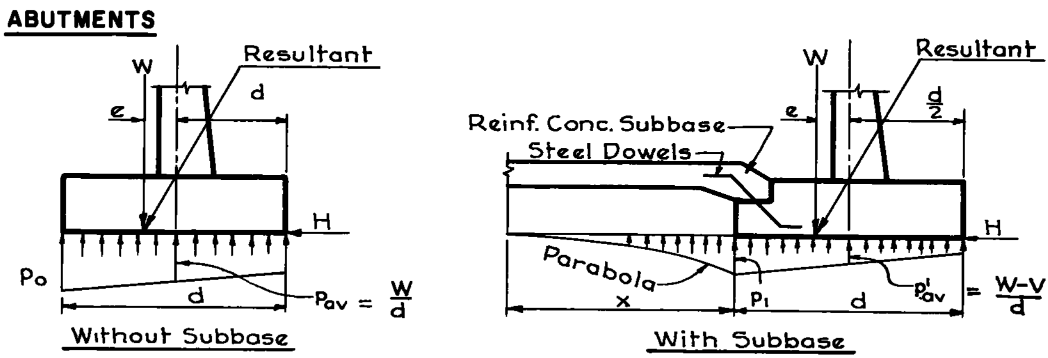
$$\frac{P}{A} = 120\,h_o + 6S_bR + 2S_oR\frac{h_o}{b} \qquad (4)$$

Figure 4.

piling is anchored to the footing and sheet pile diaphragms placed at intervals equal to the width of the footing. Figure 4 shows the Housel method as modified by the county.

EFFECT OF A CONCRETE SUBBASE ON FOUNDATION PRESSURES

A reinforced concrete strut placed between two abutment footings serves to resist the active soil pressures on the abutments. If, instead of a strut, a reinforced concrete subbase is used, the subbase acts as an auxiliary footing as well as a strut. Figures 5 and 6 show the assumed pressure distribution on abutment and pier footings with and without subbases. The assumed pressure distribution on the subbase is admittedly an approximation but is considered sufficiently accurate for design purposes in determining



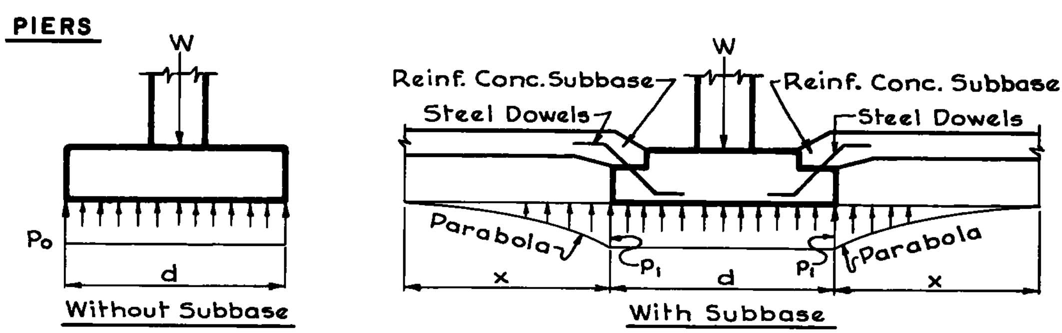
p_o = Computed max. toe pressure without subbase

p_1 = Computed max. toe pressure with subbase

$V = \frac{1}{3} P_1 x$ = Total pressure taken by subbase = Load on dowels

$x = \begin{cases} \frac{d}{2} & \text{for 12" to 18" thick subbase} \\ d & \text{for 24" thick subbase.} \end{cases}$

Figure 5. Effect of concrete subbase on foundation pressures.



$p_o = \frac{W}{d}$

$V = \frac{1}{3} p_1 x$

$p_1 = \frac{W - 2V}{d}$

$x = \begin{cases} \frac{d}{2} & \text{for 12" to 18" thick subbase} \\ d & \text{for 24" thick subbase.} \end{cases}$

Figure 6.

the reduced footing pressures resulting from subbase action.

The conventional straight line pressure distribution is assigned to the footing, while a parabolic pressure distribution is assumed acting on a portion of the subbase. This portion was arbitrarily assumed to be one-half the width of the footings for a subbase 12 to 18 in. thick and the width of the footing for a subbase 24 in. thick.

Referring to Figure 5:

$$P_0 = \frac{W}{d} + \frac{6We}{d^2}$$

$$p_1 = \frac{(W-V)}{d} + \left(We - \frac{Vd}{2}\right) \times \frac{6}{d^2} = p_0 - 4\,\frac{V}{d} = p_0 - \frac{4}{3}\,p_1X$$

$$p_1 = \frac{3}{5}\,p_0 \text{ for } X = \frac{d}{2} \tag{5}$$

$$p_1 = \frac{3}{7}\,p_0 \text{ for } X = d \tag{5a}$$

Referring to Figure 6:

$$W = p_0d = p_1d + 2V = p_1d + 2p_1 \times \frac{X}{3}$$

$$p_1 = \frac{3}{4}\,p_0 \text{ for } x = \frac{d}{2} \tag{6}$$

$$p_1 = \frac{3}{5}\,p_0 \text{ for } x = d \tag{6a}$$

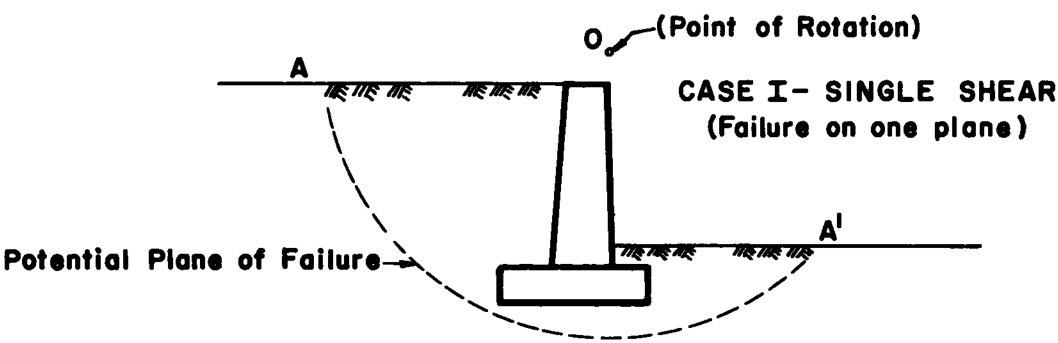


Figure 7. Showing bank supported by wall without struts.

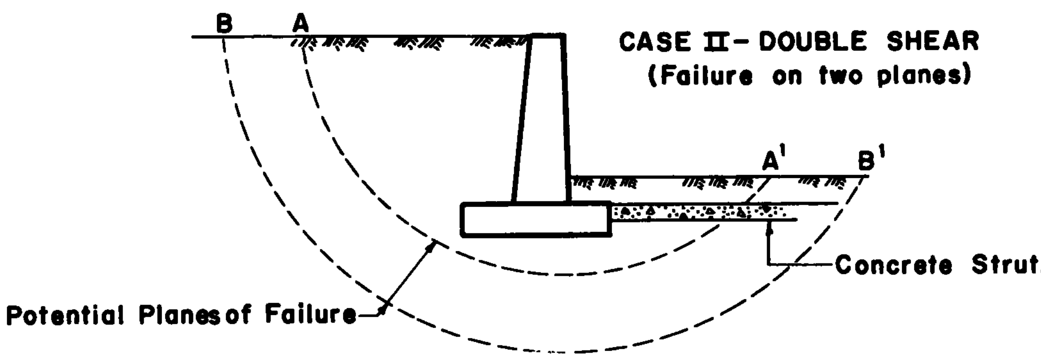


Figure 8. Showing bank supported by wall with struts.

STABILITY OF AN EMBANKMENT SUPPORTED BY A WALL

Figure 7 shows a wall retaining a high bank of earth. If the soil conditions are such as to make the bank unstable, there will be a tendency of the unstable mass to rotate on the potential failure surface AA' about some point 0. If, however, the rotation of the wall is prevented by means of a concrete strut between footings on the opposite banks, failure of the bank on a single surface is no longer possible. As shown in Figure 8, failure of the bank can now occur only by a movement of a portion of the retained mass along two sliding surfaces AA' and BB'—one at or near the back of the wall and the other at some finite distance farther back. As a result, shear resistance against sliding is now mobilized on the two surfaces, thus materially increasing the factor of safety.

The county has recently constructed an earth-filled exit ramp near the terminus of the John C. Lodge Expressway in the City of Detroit. The terminus is located near the Detroit River, and the soil conditions at this location are the worst so far encountered along the entire Expressway. The fill is retained by a wall on each side, with reinforced concrete ties placed at intervals between the walls. The walls are supported on cast-in-place concrete piles extending to hardpan (the only area on the Expressway where piles were used). The main function of the ties is similar to that described for struts; namely, to prevent movement or rotation of the walls. However, since the ties are anchored to the walls above the footings, they also reduce the moments on the stems from horizontal earth pressures. See Figure 9 for a typical cross-section of the ramp.

A reinforced concrete subbase is an extension of the strut to make a continuous slab between footings. While acting as a strut between footings on the opposite banks, it will, if properly reinforced, provide additional passive resistance against upheavel. An approximate method of evaluating this resistance will be discussed later.

PROCEDURE FOR COMPUTING THE STABILITY OF AN EMBANKMENT

As previously stated, the county's procedure differs somewhat from Housel's method (3 and 6). It has also been the practice to carry the investigation to not more than 50 ft from the top of the natural ground or to not more than 30 ft from the bottom of the slope. If the computed overload ratio, R, does not exceed 1.5 on any plane in the above depths, the embankment is considered safe.

In Figure 10 the values of S', S_1, S_2 and S_4 are the average shear values in the respective strata, while S_3 and S_5 are the smallest shear values in the immediately

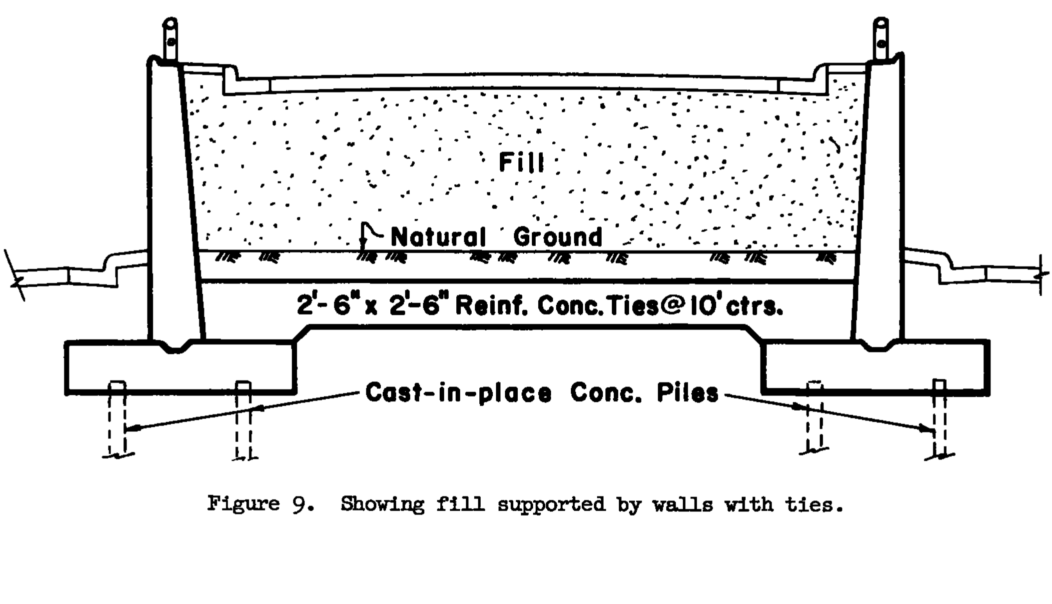


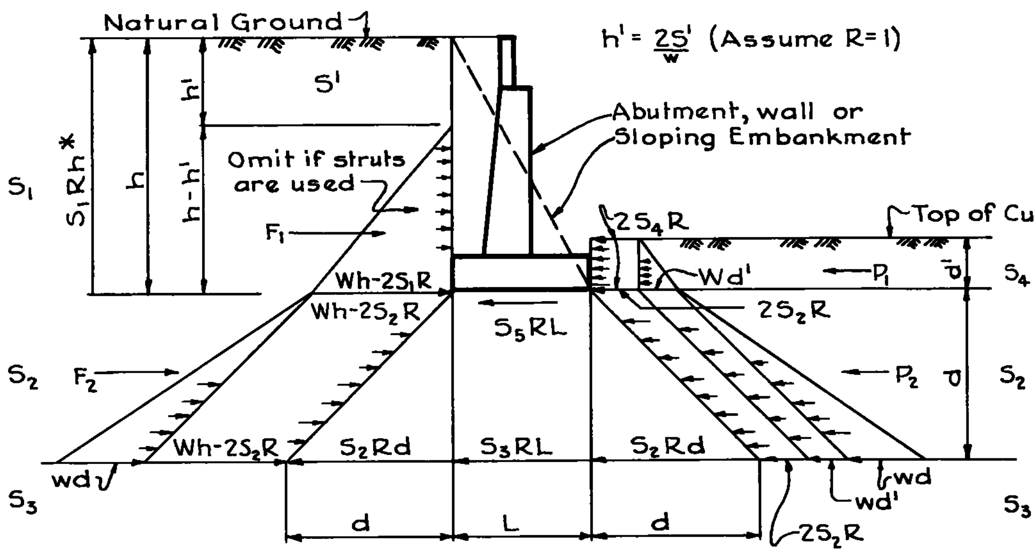
Figure 9. Showing fill supported by walls with ties.

adjacent planes. To simplify the computations, a value of R equal to one is used for determining h'.

Figure 10 is shown as applying either to an abutment, a retaining wall or to a sloping embankment. In the case of a sloping embankment, all terms containing d' are omitted from the equation, since d' is equal to zero. The shear resistance, S_5RL, at the bottom of the footing plane is utilized only if struts are used. The triangle of active pressure above the footing plane is neglected if struts are used. The vertical shear resistance, S_1Rh, is fully utilized in the case of abutments and walls with struts. However, for sloping embankments and for abutments or walls without struts, it may be utilized only when investigating the overload ratios on planes when d exceeds $\frac{h}{2}$.

PREVENTING UPHEAVAL BY MEANS OF A CONCRETE SUBBASE

As was mentioned before, the county considers an overload ratio of 1.5 to be satisfactory. This is equivalent to a factor of safety of 2.67. It is assumed that no displacement takes place as long as this overload ratio is not exceeded, but once exceeded, a progressive displacement of the soil will take place unless prevented by some positive



* To be included only if struts are used. If no struts are used, it may be included only when "d" is greater than "$\frac{h}{2}$"

$F_1 = (Wh - 2S_1R)\frac{(h-h')}{2}$ $\qquad$ $P_1 = 2S_4Rd' + \frac{w(d')^2}{2}$

$F_2 = (Wh - 2S_2R)\,d + \frac{wd^2}{2}$ $\qquad$ $P_2 = 2S_2Rd + wd'd + \frac{wd^2}{2}$

Bottom of Footing Plane (d=o) — No Struts used:-

$$F_1 = P_1 + S_5RL \tag{7}$$

Planes below bottom of Footing — No Struts used:-

$$F_1 + F_2 = P_1 + P_2 + S_3RL + 2S_2Rd + S_1Rh\,* \tag{8}$$

Planes below bottom of Footing — Struts used:-

$$F_2 = P_1 + P_2 + S_5RL + S_3RL + 2S_2Rd + S_1Rh \tag{9}$$

Figure 10. Procedure for computing the stability of an embankment.

means. A reinforced concrete subbase, anchored securely at each end to the abutment, pier or wall footings, may be designed to contain the upward pressure imposed on it by an incipient displacement and, thus, prevent the displacement.

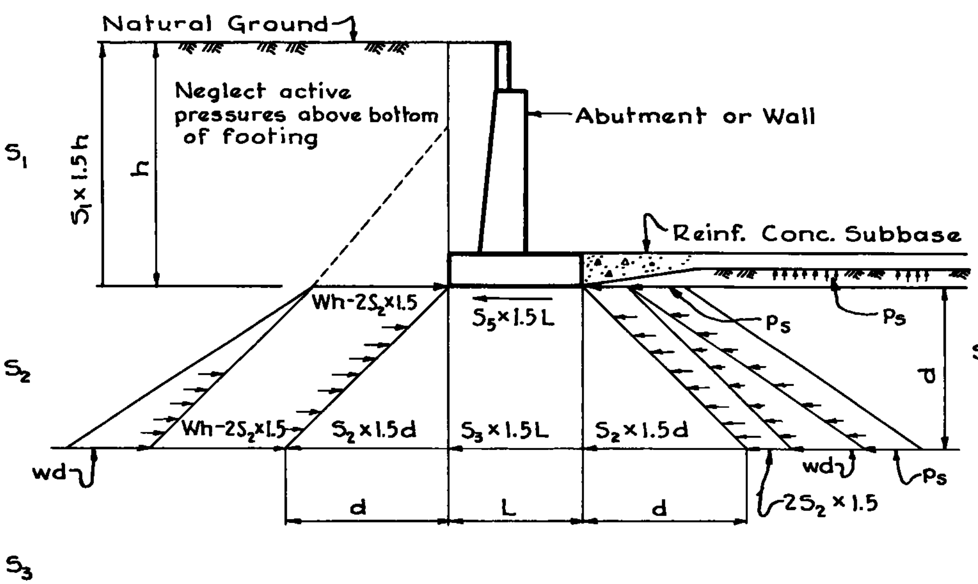
Inasmuch as an exact determination of the magnitude of the pressures exerted on the subbase is rather involved, an approximate method for evaluating these pressures with the use of Housel's equations has been developed. This method, although lacking in elegance, appears to be rational enough to yield results on which the subbase may be safely designed.

Briefly, the method is based on the assumption that no pressure is exerted on the subbase as long as the overload ratio does not exceed 1.5. When the overload ratio exceeds this figure, the upward pressure on the subbase is equal to the amount of overburden which would have to be placed on the bottom of the cut in order to reduce the overload ratio to 1.5. The weight of this overburden, p_s, when converted to passive pressure may be determined by considering it as a uniform passive pressure applied horizontally on depth d as shown in Figure 11. A value of 1.5 is then assigned to all terms containing the factor r and p_s is computed from Eq. 11. Note that the active pressure above the bottom of footing is neglected in the computations, since the subbase also acts as a strut.

SUMMARY

The procedure for determining safe foundation pressures, the stability of sloping embankments, the value of reinforced concrete struts and the effect of a reinforced concrete subbase in reducing foundation pressures and preventing upheaval has been

Assume that no pressure is exerted on the subbase until the overload ratio reaches 1.5.
Convert pressure on subbase, p_s, into a horizontal passive pressure.



$$(wh-2S_2\times 1.5)d+\frac{wd^2}{2} = 2S_2\times 1.5d+\frac{wd^2}{2}+p_s d+S_5\times 1.5L+S_3\times 1.5L+2S_2\times 1.5d+S_1\times 1.5h \quad (10)$$

$$P_s = \frac{whd-(9S_2 d+1.5S_5 L+1.5S_3 L+1.5\,S_1 h)}{d} \quad (11)$$

Figure 11. Effect of concrete subbase in preventing upheaval.

described. The methods used for determining this procedure are rational and practical. It has time and again proven its value in comparing test boring data obtained in one location with test boring data obtained in other locations, and thus enabling the exercize of judgment with a certain degree of confidence.

Because of the simplicity of the procedure, little experience is necessary for performing the calculations. Where the computed overload ratios fall below the allowable, the designer may be allowed to proceed on his own; where the overload ratios exceed the allowable, the Chief Designer may then exercise his judgment in selecting the method best suited to insure a safe design.

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3. "Design Memorandum on Embankment Stability—Lateral Distribution or Weight Transfer above the Loading Plane." Soil Mechanics Laboratory, University of Michigan, (April 1952).
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5. Tschebotarioff, G.P., "Soil Mechanics, Foundations and Earth Structures." McGraw-Hill (1951).
6. Kerkhoff, G.O. and Housel, W.S., "Uplift Soil Pressure on Bridge Foundations as Revealed by Shear Tests." Proceedings, American Society for Testing Materials, Volume 47 (1947).

Appendix A

SHOWING TYPICAL LABORATORY TEST RESULTS OF SOIL SAMPLES

TEST RESULTS

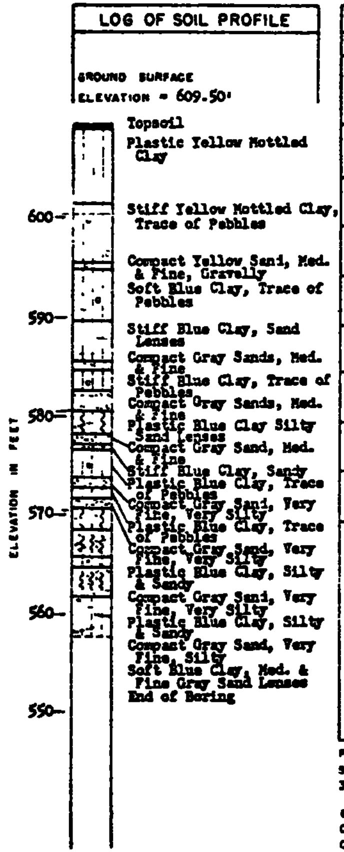
	Laboratory Numbers 558-			314		317		321	
	A.A.S.H.O. Soils Classification	Sieve Size	Sieve Opening mm.	Cumulative Per cent Passing	Per cent Retained	Cumulative Per cent Passing	Per cent Retained	Cumulative Per cent Passing	Per cent Retained
Sieve Analysis	Gravel	3/4 inch	19.10						
		1/2 inch	12.70						
		3/8 inch	9.52						
		No. 4	4.76						
		No. 10	2.00	100		100		100	
	Coarse Sand	No. 18	1.00	98		99		99	
		No. 20	0.84	97		98		99	
		No. 35	0.50	95		96		97	
		No. 40	0.42	94	6	95	5	97	3
	Fine Sand	No. 60	0.25	90		89		94	
		No. 140	0.105	80		65		83	
		No. 200	0.074	76	18	56	39	78	19
Hydrometer	Silt		0.050	70		49		67	
			0.005	29	47	21	35	24	54
	Clay		0.001		29		21		24
	Colloids								
Soil Constants	Liquid limit			34		17		19	
	Plasticity index			17		6		7	
	Specific gravity			2.73		2.67		2.68	
	Shrinkage limit, per cent by weight			14.1		10.6		11.4	
	Shrinkage ratio			1.92		2.04		2.04	
	Organic content, per cent by weight								
	Loss on ignition, per cent by weight								
	Shear stress, psf			1,008		No Test		274	
	One-half compressive strength, psf			2,688		1,152		1,008	
	Natural moisture, per cent by weight			18.9		12.2		17.6	
	Dry density, lb. per cu. ft.			110.4		122.9		114.2	

REMARKS:

558-314 Sample No. 1 Sample Depth 5' Elevation 604.0'
558-317 Sample No. 4 Sample Depth 20' Elevation 589.0'
558-321 Sample No. 8 Sample Depth 40' Elevation 569.0'

Appendix B

SHOWING TYPICAL SOIL ANALYSIS DATA SHEET



SOIL SAMPLE

FIELD DATA					LABORATORY DATA		
SA. NO	ELEV.	PENETRATION: NO OF BLOWS	PENETRATION: DRIVE IN INCHES	CONSISTENCY	CONSISTENCY	MOISTURE PER CENT DRY WT.	DRY WT LB. PER CU. FT.
1	604.0	7	12	Plastic	Plastic	18.9	110.4
2	599.0	13	12	Stiff	Stiff	18.1	113.6
3	594.0	Sampler Pushed		Soft	Soft	24.9	101.4
4	589.0	"	"	Sand & Stiff Clay	Compact Sand & Firm Clay	12.2	122.9
5	584.0	13	12	Stiff	Stiff	13.6	126.0
6	579.0	Sampler Pushed		Plastic	Soft	11.5	127.9
7	574.0	"	"	Plastic	Plastic	19.0	109.8
8	569.0	"	"	Plastic	Plastic	17.6	114.2

PENETRATION NOTE NUMBER OF BLOWS REQUIRED TO DRIVE CORE SAMPLER DISTANCE GIVEN, USING 140 POUND WEIGHT FALLING 30 INCHES

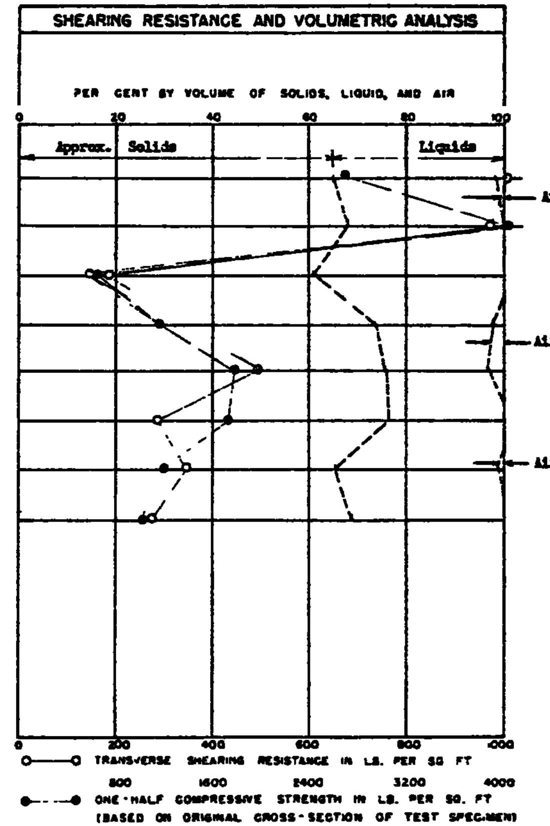
GENERAL NOTE FIELD CONSISTENCY DETERMINED BY INSPECTION OF SAMPLES AND SUBSTANTIATED BY RESISTANCE TO PIPE CASING AND JET ROD. BELOW DEPTH OF SAMPLING, CONSISTENCY DETERMINED BY SOIL RESISTANCE TO JET ROD.

BORING DATA

NUMBER 43

LOCATION Centerline of Southfield r.o.w., 370' north of centerline of Ford Rd. r.o.w.

DATE OF BORING February, 1955



Appendix C

TYPICAL EXAMPLE

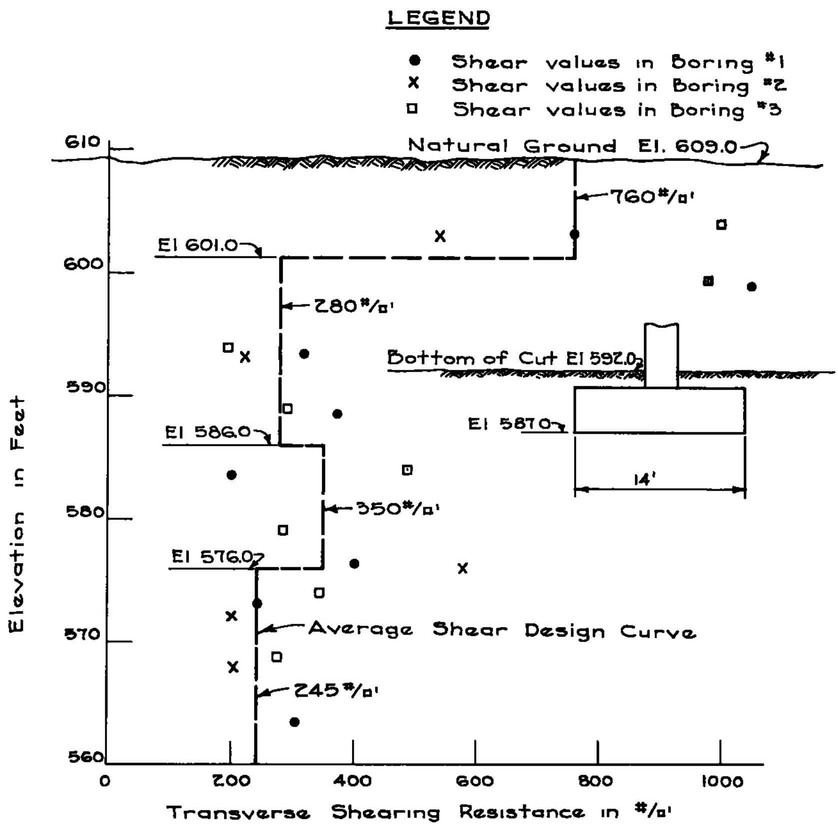
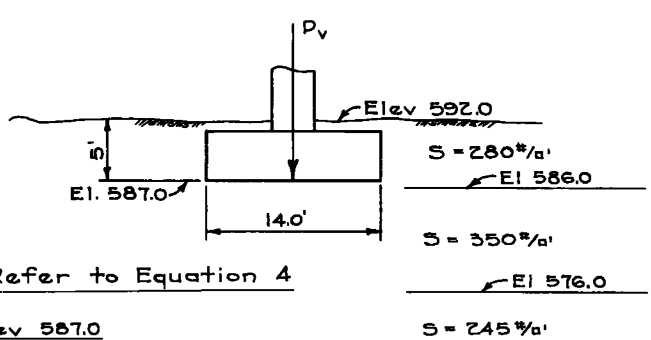


Figure 12. Typical shearing resistance curve.

ALLOWABLE FOUNDATION PRESSURES



Refer to Equation 4

At Elev 587.0

$$\text{Allowable } \frac{P}{A} = 120 \times 5.0 + 6\left(\frac{1 \times 280 + 10 \times 350 + 3 \times 245}{14.0}\right) R + 2 \times 280 \times \frac{5.0}{14.0} \times R$$

$$= 600 + 1935R + 200R = 600 + 2135R$$

$$\text{For } R = 1.0 \quad \frac{P}{A} = 2735 \,\#/\square'$$

$$\text{For } R = 1.5 \quad \frac{P}{A} = 3800 \,\#/\square'$$

STABILITY OF EMBANKMENT AT ABUTMENT

Refer to Figures 10 & 11

To simplify computations, assume $h' = 8.0'$

Then $h - h_1 = 22.0 - 8.0 = 14.0'$, $d' = 5.0'$, $L = 14.0'$

$$F_1 = (120 \times 22.0 - 2 \times 280R) \times \frac{14.0}{2} = 18480 - 3920R$$

$$P_1 = 2 \times 280R \times 5.0' + 120 \times \frac{5.0^2}{2} = 2800R + 1500$$

At Elev. 587.0 (Bottom of Footing Plane)

$F_1 = P_1 + S_5 R \times L$, or

$$18480 - 3920R = 2800R + 1500 + 280R \times 14.0$$

$$R = \frac{16980}{10640} = 1.60 > 1.5 \quad \therefore \text{ use struts}$$

At Elev. 560.0 $\quad d = 27.0' \quad d' = 5'$

$$S_1 = (760 \times 8 + 280 \times 14) \times \frac{1}{22} = 454\ \#/\square'$$

$$S_2 = (280 \times 1.0 + 350 \times 10 + 245 \times 16) \times \frac{1}{27} = 285\ \#/\square'$$

$S_3 = 245\ \#/\square' \qquad S_3 = 280\ \#/\square'$

$$F_2 = (120 \times 22.0 - 2 \times 285R) \times 27.0 + 120 \times \frac{27^2}{2} = 115{,}020 - 15390R$$

$$P_2 = 2 \times 285R \times 27 + 120 \times 5 \times 27 + 120 \times \frac{27^2}{2} = 15390R + 59940$$

Using Equation 9, we get

$$115{,}020 - 15390R = (2800R + 1500) + (15390R + 59940) + (280R \times 14.0) + (245R \times 14.0) + (2 \times 285R \times 27.0) + (454R \times 22.0)$$

$$R = \frac{53{,}580}{66{,}300} = 0.81 < 1.5$$

$\therefore$ struts are sufficient