DESIGN OF A CELLULAR COFFERDAM

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SYNOPSIS

This paper presents the theory for design of a cellular cofferdam consisting of a number of steel sheet pile cells, circular in cross section, and filled with earth The theory is illustrated by a practical design for a specific location showing computations for determining the diameter of the cylindrical cells, the necessity for and the amount of foreign fill required within the cell, the depths to which the sheet piles forming the cells must be driven and the required section of the piles In this case the piles must be 79 5 ft long and driven through 45 ft of tule mud, 10 ft of blue clay and penetrate 3 ft of a 35 ft stratum of yellow clay which rests upon hardpan The excavation inside the cofferdam is carried 54 5 ft below the top of the sheet piles

The cofferdam is a gravity retaining wall and is designed accordingly The overturning moment of the fill outside the cofferdam, the resting moment of the cylindrical cells, pressures at toe and heel of cells, resistance to sliding and horizontal shear are computed and from these the design is made

PROBLEM

An 800-ft long graving dock is to be built at a site whose average soil formation is as given in Figure 1 It is assumed that due to the proximity of existing buildings and the very flat angle of repose of the tule mud, the site of the dock cannot be pre-dredged lower than elevation 90

Dredging to elevation 90 is desirable not only from the standpoint of reducing the pressures on the cofferdam structure in which the dock is to be built, but also to enable floating equipment to be used in constructing the cofferdam

The dock is to be founded on piles A cofferdam free of internal bracing is desired in order to facilitate proper driving of the bearing piles, and the placing of the concrete in the dry

The use of a cellular steel sheet pile cofferdam surrounding the site of the dock is indicated The bottoms of the sheet piles forming the cofferdam cells are to be in the yellow clay if possible, but, for reasons of economy, not down to the hardpan, unless found necessary

The original site material in the cells of the cofferdam may be replaced with fill material having an angle of internal friction of 30°.

The characteristics of the soils at the site as determined from laboratory experiments are given in Table 1

TABLE 1

Material	Weight in water, lb per cu ft	Angle of internal friction, deg	Cohesion lb per sq ft
Tule mud	36	25	150
Blue Clay	45	15	420
Yellow clay	60	30	500
Fill (100 lb per cu ft in air)	60	30	400

The maximum allowable interlock stress in steel sheet piles is 6,000 lb per lin inch

It is required to find the diameter of the cylindrical cells, the depths to which the sheet piles forming the cells will have to be driven (depth required both at front and at back), as well as the required section of the sheet piles

The design of the cofferdam becomes unique and somewhat complicated due to the various soil strata of widely differing characteristics through which the cells pass and also to the fact that it is desirable to stop the cells, for reasons of economy before reaching the hard-pan The discussion of the problem may appear lengthy, but for clarity the solution is presented step by step

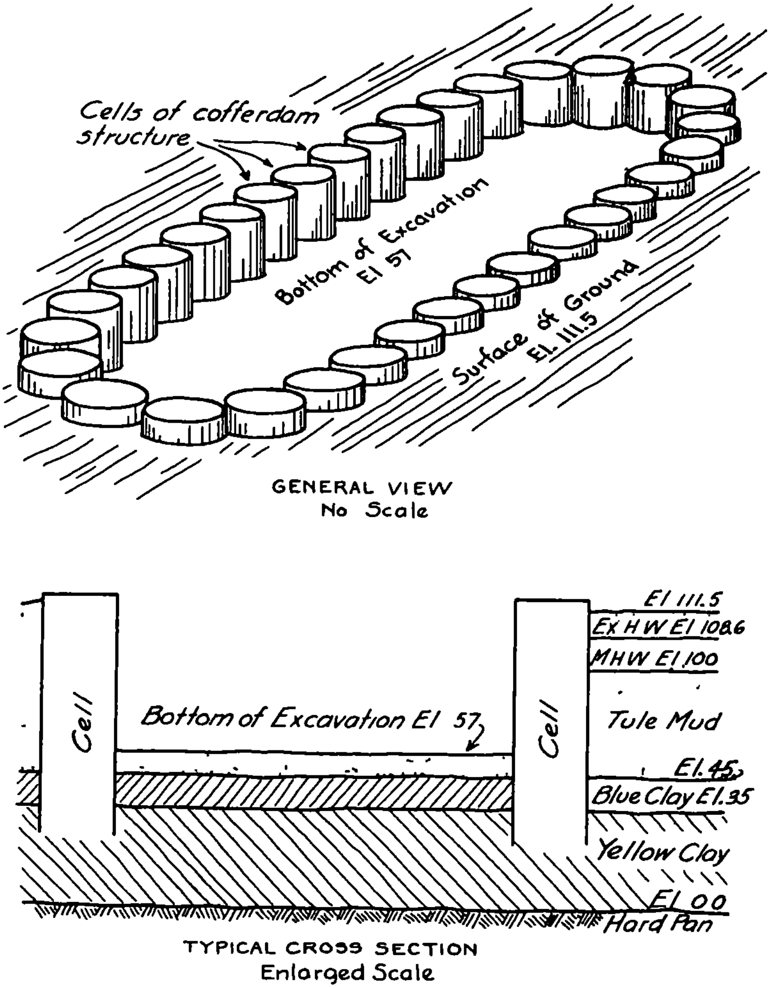

Figure 1

The cellular cylindrical cofferdam is in fact a gravity wall retaining the earth fill behind it and has to be designed as such The moment of the retained fill around the toe M_0 is the overturning moment, the moment of the weight of the fill inside the cell around the toe of the cell M_1 is the resisting moment The resisting moment must be larger than the overturning moment The ratio of the resisting moment to the overturning moment is the factor of safety against overturning The overturning moment produces pressures on the base which are assumed to vary uniformly from a minimum at the heel to a maximum at the toe Ordinarily the pressure at the toe is limited to a certain maximum value, while the pressure at the heel is assumed to be 0 The last condition, however, is not a condition *sine qua non* It is true that the soil cannot take any tension but this condition can also be fulfilled with the neutral axis passing through any plane within the circular area

Let R = radius of cylindrical cell
M = overturning moment of the fill around base in kip ft per ft of wall
P = intensity of vertical pressure at bottom of cell per sq ft in kips

Then $M_0 = 2RM$, The overturning moment on one cylindrical cell
$P_0 = P\pi R^2$, Total vertical load inside one cylindrical cell
$M_r = P\pi R^3$, Resisting moment of one cell

$$S_b = \pm\frac{M_0}{I/c} = \pm\frac{8M}{\pi R^2}$$

= unit pressure due to bending

In order that the stress at the heel of the cylinder shall be zero, we must have the intensity of vertical pressure equal to the unit pressure due to bending, thus

$$R = \sqrt{\frac{8M}{\pi P}} = 1\ 6\sqrt{e} \qquad (1)$$

where

$$e = \frac{M}{P}$$

Factor of safety against overturning

$$f_0 = \frac{M_r}{M_0} = \frac{P\pi R^3}{2RM} = \frac{\pi R^2}{2e} \qquad (2)$$

The radius, R, required for a given factor of safety, f_0, is then obtained from

$$R = 0\ 8\sqrt{ef_0} \qquad (2a)$$

When R is as given by equation (1) we get,

$$f_0 = 4 \qquad (3)$$

and maximum compression at the toe in kips per sq ft is

$$P + S_b = 2P \qquad (4)$$

The minimum stress at the heel = 0

In order that the cylinder should not slide on the base we must have the resistance to the sliding either equal to or greater than the horizontal force H_o of the fill behind the cylinder The resistance of cylinder to sliding on the base is made up of the frictional resistance plus the cohesive resistance

Let $\mu = \tan\phi$ = tan of the angle of internal friction of the material at base
and C = coefficient of cohesion in kips per sq ft,

$$H_0 = 2RH$$

where H = horizontal force of retained fill in kips per lin ft of wall

Then $$2RH = \pi R^2(P\mu + C)$$

and $$R = \frac{2H}{\pi(P\mu + C)} \qquad (5)$$

Factor of safety against sliding

$$f_s = \frac{\pi R^2(P\mu + C)}{2RH} = \frac{\pi R(P\mu + C)}{2H} \qquad (6)$$

The maximum intensity of horizontal shear for a circular beam occurs at the neutral axis and equals

$$v = \frac{Vm}{It} = \frac{4V}{3\pi R^2} = \frac{8H}{3\pi R} \text{ kips per sq ft}$$

where V = total shear for one cell = 2HR

The resistance of material at base in kips per sq ft = $P\mu + C$

Then $$\frac{8H}{3\pi R} = P\mu + C$$

and $$R = \frac{0\ 85H}{P\mu + C} \qquad (7)$$

Factor of safety against horizontal shear

$$f_v = \frac{3\pi(P\mu + C)R}{8H} \qquad (8)$$

The radius, R, required for a given factor of safety, f_v, is then obtained from

$$R = \frac{0\ 85Hf_v}{P\mu + C} \qquad (8a)$$

Comparing equations (5) and (7) we conclude that the controlling factor in determining the radius of the cylindrical cell is the maximum shear intensity rather than the sliding resistance

Using formula (1) and (8a) we can find the radius of the cylindrical cells to satisfy the given conditions However before proceeding with the actual design of the cofferdam, we will derive the expression for horizontal active and passive pressure exerted by a soil possessing an angle of internal friction ϕ and a co-

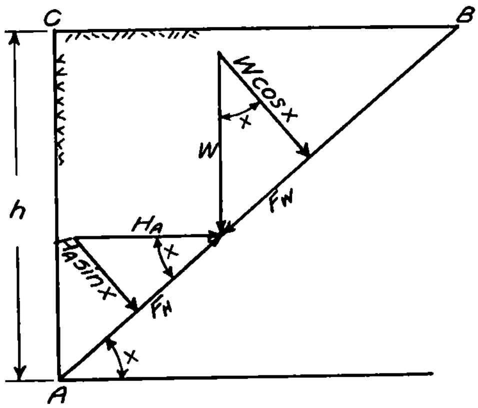

Figure 2

hesive strength of C per sq unit The rupture is assumed to occur along a plane as is done by Coulomb In Figure 1, let AB be the plane of rupture and let x = angle which this plane makes with the horizontal Let also $r = \tan x$ and $\mu = \tan \phi$ The forces acting on this plane are given in Figure 2

Force tending to slide down

$$F_S = F_W - F_H$$

But $F_W = W \sin x$ and $F_H = H_A \cos x$

$\therefore F_S = W \sin x - H_A \cos x$

where W = the weight of ABC

$$= \frac{wh^2}{2} \cot x = \frac{wh^2}{2r}$$

and H_A = horizontal force required to stop W from sliding down

w = density of material

The forces resisting sliding F_R = force of friction F_F + resistance due to cohesion F_C

Now $F_F = N\mu$ and $F_C = CA$

Where N = normal pressure on $AB = W \cos x + H_A \sin x$

and $A = \text{Area of } AB = \dfrac{h}{\sin x}$

$$\therefore F_R = F_F + F_C = \mu(W \cos x + H_A \sin x) + \frac{Ch}{\sin x}$$

At the point of sliding we have $F_S = F_R$

or
$$W \sin x - H_A \cos x = (W \cos x + H_A \sin x)\mu + \frac{Ch}{\sin x}$$

$$\therefore H_A(\cos x + \mu \sin x) = W(\sin x - \mu \cos x) - \frac{Ch}{\sin x}$$

or
$$H_A = \frac{W(\sin x - \mu \cos x)}{\cos x + \mu \sin x} - \frac{Ch}{\sin x(\cos x + \mu \sin x)}$$

Dividing numerator and denominator by cos x we get

$$H_A = \frac{W(\tan x - \mu)}{1 + \mu \tan x} - \frac{Ch}{\sin x \cos x(1 + \mu \tan x)}$$

or
$$H_A = \frac{W(r - \mu)}{1 + \mu r} - \frac{Ch}{\sin x \cos x(1 + \mu r)}$$

Now $\dfrac{\sin x}{\cos x} = r \quad \therefore \sin x = r \cos x$

But $\sin^2 x + \cos^2 x = 1$

$$\therefore \cos^2 x(1 + r^2) = 1 \quad \therefore \cos^2 x = \frac{1}{1 + r^2}$$

and $\sin x \cos x = r \cos^2 x$

$$\therefore \sin x \cos x = \frac{r}{1 + r^2}$$

$$\therefore H_A = \frac{W(r - \mu)}{1 + \mu r} - \frac{Ch(1 + r^2)}{r(1 + \mu r)} \quad (9a)$$

Substituting for $W = \frac{wh^2}{2r}$ we get

$$H_A = \frac{wh^2(r-\mu) - 2Ch(1+r^2)}{2r(1+\mu r)} \qquad (a)$$

In order to find for what value of r, H will be a maximum we make $\frac{dH_A}{dr} = 0$

or $\frac{dH_A}{dr}$

$$= \frac{wh\mu(1+2\mu r - r^2) + 2C(1+2\mu r - r^2)}{4r^2(1+\mu r)^2} = 0$$

$$\therefore (1+2\mu r - r^2)(wh\mu + 2C) = 0$$

or $\quad 1 + 2\mu r - r^2 = 0$

and $\quad r = \sqrt{1+\mu^2} + \mu = \tan(45 + \phi/2)$

Substituting this value in equation (a) and simplifying we get

$$H_A = \frac{wh^2}{2}\tan^2(45-\phi/2) - 2Ch\tan(45-\phi/2) \qquad (9)$$

The weight of sliding wedge is

$$W = \frac{wh^2}{2}\tan(45° - \phi/2)$$

$$\therefore \frac{H_A}{W} = \left(1 - \frac{2Ch}{W}\right)\tan(45-\phi/2)$$

and $H_A = (W - 2Ch)\tan(45° - \phi/2)$ (10)

The intensity of the horizontal active pressure I_A at any depth h from the surface is

$$I_A = \frac{dH_A}{dh} = wh\tan^2(45-\phi/2) - 2C\tan(45-\phi/2) \qquad (11)$$

The vertical intensity at the same point $I_V = wh$ The ratio of the intensity of the horizontal active pressure to the vertical intensity is

$$K_A = \frac{I_A}{I_V} = \tan^2(45-\phi/2) - \frac{2C\tan(45-\phi/2)}{I_V} \qquad (12)$$

Proceeding in a similar manner for the passive pressure we find that when the wedge of earth is on the verge of moving up

$$H_P = \frac{wh^2}{2}\tan^2(45+\phi/2) + 2Ch\tan(45+\phi/2) \qquad (13)$$

or $H_P = (W + 2Ch)\tan(45+\phi/2)$ (14)

The intensity of the horizontal passive pressure I_P at any depth h from the surface is

$$I_P = \frac{dH_P}{dh} = wh\tan^2(45° + \phi/2) + 2C\tan(45+\phi/2) \qquad (15)$$

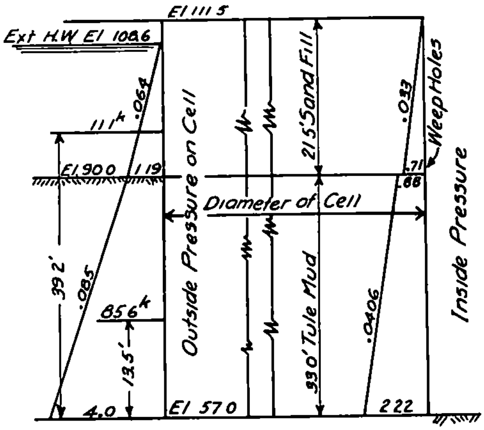

Figure 3

The ratio of the intensity of the horizontal passive pressure to the vertical intensity is

$$K_P = \frac{I_P}{I_V} = \tan^2(45+\phi/2) + \frac{2C\tan(45+\phi/2)}{I_V} \qquad (16)$$

DESIGN OF COFFERDAM

Assume cofferdam to stop at elevation 57 and fill inside cofferdam from elevation 90 to surface with a drain at elevation 90 The pressures inside and outside of cells are plotted on Figure 3 according to Table 2 which gives the strength characteristics of the various soils encountered The pressure at rest for the tule mud was determined by laboratory test and found to be the equivalent of a

TABLE 2

Material	φ deg	tan φ	C—k per sq ft	W in Water K per cu ft	Active pressure						Passive pressure					
					$\tan(45 - \varphi/2)$	$\tan^2(45 - \varphi/2)$	Without cohesion		With cohesion		$\tan(45 + \varphi/2)$	$\tan^2(45 + \varphi/2)$	Without cohesion		With cohesion	
							I_A	K_A	I_A	K_A			I_P	K_P	I_P	K_P
Tule Mud	25	0 466	0 15	0 036	0 637	0 406	2 015h	0 406	0 015h − 0 192	$0\ 406 - \frac{0\ 192}{I_v}$	1 570	2 464	0 089h	2 464	0 089h + 0 472	$2\ 464 + \frac{0\ 472}{I_v}$
Blue Clay	15	0 268	0 42	0 045	0 767	0 589	0 027h	0 589	0 027h − 0 644	$0\ 589 - \frac{0\ 644}{I_v}$	1 303	1 698	0 076h	1 698	0 076h + 1 094	$1\ 698 + \frac{1\ 094}{I_v}$
Yellow Clay	30	0 577	0 50	0 060	0 577	0 333	0 020h	0 333	0 020h − 0 577	$0\ 333 - \frac{0\ 577}{I_v}$	1 732	3 000	0 180h	3 000	0 180h + 1 732	$3\ 000 + \frac{1\ 732}{I_v}$
Fill (100 lb per cu ft in Air)	30	0 577	0 40	0 060	0 577	0 333	0 020h	0 333	0 020h − 0 462	$0\ 333 - \frac{0\ 462}{I_v}$	1 732	3 000	0 180h	3 000	0 180h + 1 386	$3\ 000 + \frac{1\ 386}{I_v}$

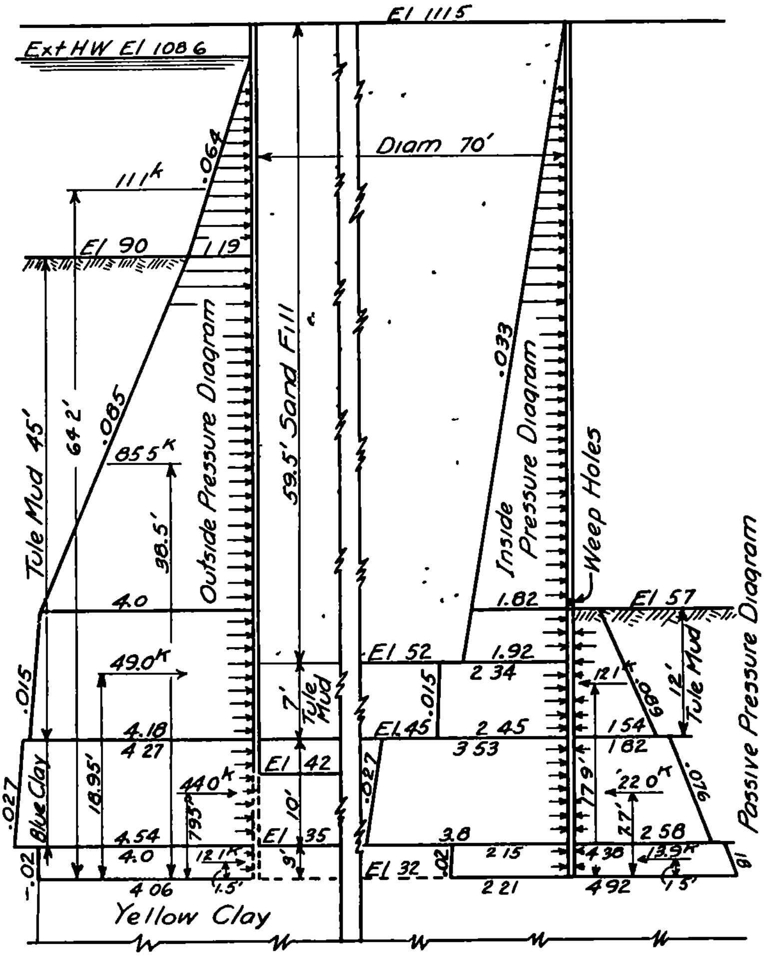

Figure 4

liquid weighing 85 pounds per cubic foot This was used in figuring the pressure behind the cell The pressure inside of the cell was taken as the active pressure In both cases the cohesion was neglected as it is on the side of safety

The forces and their locations are indicated in Figure 3 from which

$$H = 96\,7 \text{ kips} \quad M = 1591 \text{ kip-ft}$$

$$P = 21\,5 \times 100 + 33 \times 36 = 3\,34 \text{ kips per sq ft}$$

Using a factor of safety against horizontal shear of 1 5 we have from formula 8a and Table 2

$$R = \frac{1\,5 \times 85H}{P\mu + C} = \frac{1\,275 \times 96\,7}{3\,34(\,466) + 15} = 72\,1 \text{ ft}$$

The tension on the interlock, T, of the steel sheet piles at base,

$$T = \frac{IR}{12} = \frac{2\,22 \times 72\,1}{12} = 13\,3 \text{ kips per lin in}$$

This value is more than twice the value allowed, hence we must replace the tule mud inside the cell by the same fill used at the top and make a drain at elevation (57) to reduce the inside pressure The inside and outside pressures on the cell will be as shown on Figure 4

For this case

$$P = 54\,5 \times 100 = 5\,45 \text{ kips per sq ft}$$

$$e = \frac{1591}{5\,45} = 292$$

Using a factor of safety against horizontal shear of 1 5 as before we have

$$R=\frac{1\ 275\times 96\ 7}{5\ 45(\ 577)+\ 4}=35\text{ ft}$$

For R=35 ft we have Factor of safety against overturning (formula 2) $f_0=6\ 6$ Maximum compression under toe

$$=P+\frac{8M}{\pi R^2}=8\ 76\text{ kips per sq ft}$$

Maximum tension on interlock

$$T=\frac{1\ 8\times 35}{12}=5\ 3\text{ kips per lin in}$$

At the allowable 6 kips per linear inch we have allowable $I=2\ 06^K/\text{sq ft}$

The actual I due to fill is 1 8, the difference therefore is $2\ 06-1\ 8=\ 26^K/\text{sq ft}$ which allows for a head of water inside the cell of $\frac{26}{064}=4\ 0'$ in case the drain does not act temporarily

However, if we stop the sheet piles at elevation 57, the pressure of $8\ 76^K/\text{sq ft}$ at the toe will cause the bottom of the excavation to blow up In order to prevent this blowing we have to drive piles in the front to such a depth as to make I_A, the active intensity of the horizontal pressure from inside the cell at that depth, at least equal to the passive intensity in front of the cell at the same point However, before determining the depth to which the front piles will have to be driven, we will determine how far below elevation 57 the fill inside the cell will have to be carried We can not stop it at 57 for in that case the intensity of the active pressure immediately below 57 in the tule mud is

$$I=\ 406\times 5\ 45=2\ 22^K/\text{sq ft}$$

$\therefore$ T=6 5 kips per linear inch, which is more than the allowable

Assume that we will carry the fill to a distance x below elevation 57 Then $I_V=5\ 45+\ 06x$, 060 being the weight of the fill in water and $I_A=0\ 406(5\ 45+\ 06x)=2\ 22+0\ 024x$ (neglecting cohesion) The passive intensity at x below elevation 57 for tule mud

$$I_P=\ 089x\text{ (neglecting cohesion)}$$

and the resultant intensity

$$I_r=I_A-I_P=2\ 22-\ 065x$$

$$T=\frac{I_rR}{12}=\frac{(2\ 22-\ 065x)35}{12}$$

Substituting for T=6 0 kips per inch, we get

$$x=2\ 5\text{ ft}$$

We will carry the fill down 5 ft below elevation 57

Now let us assume that we will drive the piles in the front of the cell a distance y in the yellow clay.

Neglecting at present the increase in the toe pressure due to the increased overturning arm, the vertical intensity, I_V, at the toe inside the cell is

$$I_V=8\ 76+5\times\ 06+7(\ 036)+10\times\ 045+\ 06y\text{ (see Fig 4)}$$

or $$I_v=9\ 76+\ 06y$$

Now $$I_A=I_VK_A$$

From Table 2

$$K_A\text{ for yellow clay is }\ 33-\frac{\ 577}{I_V}$$

$$I_A=(9\ 76+\ 06y)\ 33-\ 577=2\ 64-\ 02y$$

The vertical intensity, I_V, at the same point on the excavated side is

$$I_V=\ 12\times\ 036+10\times\ 045+\ 06y=\ 882+\ 06y$$

$\therefore I_P=K_PI_V$

From the table we get

$$K_P = 3 + \frac{1\ 73}{I_V}$$

$$I_P = (\ 882 + \ 06y)3 + 1\ 732$$
$$= 4\ 378 + \ 18y$$

In order to be safe against blowing of bottom we must have

$$I_A = I_P$$

or $$2\ 64 - \ 02y = 4\ 378 + \ 18y$$

or $$y = -8\ 7$$

which apparently means that the pile need not be driven in the yellow clay,

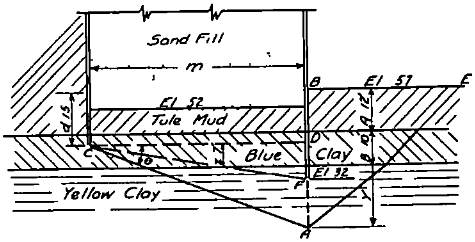

Figure 5

however, if we stop the pile at the yellow clay we get

$$I_A = 9\ 76 \times \ 589 - \ 644$$
$$= 5\ 10 \text{ kips per sq ft}$$
$$I_P = \ 882 \times 1\ 698 + 1\ 094$$
$$= 2\ 59 \text{ kips per sq ft}$$

Therefore I_A is greater than I_P and unless we drive the pile in the yellow clay the bottom will blow Let us drive the front of the piles a minimum of three feet in the yellow clay or to elevation 32 The depth to which the piles in the back of the circular cell will have to be driven is determined by the resistance of the wedge of earth ABE to move up under the active pressure of the earth in back of line AB as shown in Figure 5 Assume the piles in the back to be driven a distance d below the excavation line (elevation 57 0) in the front Let the rupture plane AC make an angle θ with the horizontal so that $\tan\theta = x$ Then by formula 9a we get for the active pressure

$$H_A = \frac{W(x-\mu)}{1+x\mu} - \frac{Ch(1+x^2)}{x(1+x\mu)}$$

μ = tan of the angle of internal friction of the particular strata and W = the weight of the sliding wedge plus the surcharge load P coming on the wedge The surcharge P on line CD is not uniform, but for the purpose of simplicity, it will be taken to be the weight of material above line CD in kips per sq ft Therefore denoting the thickness of any strata by A, its μ by a, its C by C_A and the density by w_a, we have the horizontal base of the sliding triangle

$$b_a = A \cot\theta \text{ or } b_a = \frac{A}{x}$$

and the total sliding weight W_A is

$$W_A = \frac{PA}{x} + \frac{w_a A^2}{2x} = \frac{A}{2x}(2P + Aw_a)$$

and the active pressure for this strata A is

$$H_{AA} = \frac{A(2P+Aw_a)(x-a)}{2x(1+ax)} - \frac{C_A A(1+x^2)}{x(1+ax)}$$

For any other strata of depth B underlying the strata A we have by similarity

$$H_{AB} = \frac{B[2(P+Aw_a)+Bw_b](x-b)}{2x(1+bx)} - \frac{C_B B(1+x^2)}{x(1+bx)}$$

In like manner the value of the active pressure for any other strata can be found

Let the bottom of the last full strata be a distance z below the assumed distance d Denote CD by m Then we get from the figure $Y = mx - z$

Then,

$$H_{AY}=\frac{(mx-z)\,[2(P+Aw_a+Bw_b+Cw_c+\quad)+(mx-z)w_y]\,(x-y)}{2x(1+xy)}-\frac{C_Y(mx-z)\,(1+x^2)}{x(1+xy)}$$

The total active pressure of ACD is

$$H_A=H_{AA}+H_{AB}+H_{AC}+\quad+H_{AY}$$

or

$$H_A=\frac{1}{2x}\left\{\frac{A(2P+Aw_a)\,(x-a)}{1+ax}+\frac{B[2(P+Aw_a)+Bw_b]\,(x-b)}{1+bx}+\cdots+\frac{(mx-z)\,[2(P+Aw_a+Bw_b+\quad)+(mx-z)w_y]\,(x-y)}{1+xy}\right\}-\frac{(1+x^2)}{x}\left\{\frac{AC_A}{1+ax}+\frac{BC_B}{1+bx}+\quad+\frac{(mx-z)C_Y}{1+xy}\right\} \tag{17}$$

Besides this active pressure there is also the active pressure (*not the pressure at rest*) of the retained material behind the cell above line CD, both of which are resisted by the passive pressure of the wedge ABE in front of the cell

Let ϕ_A, ϕ_B, ϕ_C, ϕ_Y denote the angles of internal friction for stratas A, B, C, Y respectively Let also

$$\alpha=\tan\left(45^\circ+\frac{\phi_A}{2}\right)$$

$$\beta=\tan\left(45^\circ+\frac{\phi_B}{2}\right)$$

$$\sigma=\tan\left(45^\circ+\frac{\phi_C}{2}\right)$$

$$\gamma=\tan\left(45^\circ+\frac{\phi_Y}{2}\right)$$

Then by formula (14) we get the passive pressure for strata A

$$H_{PA}=(W_A+2AC_A)\alpha=\frac{w_aA^2\alpha^2}{2}+2AC_A\alpha$$

For strata B underlying strata A, considering the weight of strata A as a surcharge on strata B, we get

$$H_{PB}=\frac{B\beta^2}{2}\,(2Aw_a+Bw_b)+2BC_B\beta$$

For strata Y whose thickness is $mx-z$ we get

$$H_{PY}=\frac{C\gamma^2}{2}[2(Aw_a+Bw_b+Cw_c+\ldots)+(mx-z)w_y]+2(mx-z)C_y\gamma$$

The total passive resistance is

$$H_P=H_{PA}+H_{PB}+H_{PC}+\quad H_{PY} \tag{18}$$

The difference between the passive and active pressure is

$$H_r=H_P-H_A$$

H_P is as given above by equation (18) and H_A is as given by equation (17) above, both of which are expressed in terms of the only unknown x That value of x is to be taken which will render H_r a minimum

In our case we will figure the width m of rectangular cofferdam equivalent to the given circular cell as being

$$m=R+K$$

where

$$2KR=\frac{\pi R^2}{2}$$

$$m=R+\frac{\pi R}{4}=1\ 785\times 35=62\ 5\text{ ft}$$

For active pressure A=0, B=7 0 ft, while for passive pressure A=12 0 ft; B=10 0 ft

Substituting these values with the weights cohesion and angle of friction for each stratum in equations (17) and (18) we get the net resistance to sliding on the incline

$$H_r = H_P - H_A = \frac{110x^5 + 615x^4 + 700x^3 - 111x^2 + 517x - 28\ 0}{314x^3 + 1\ 7x^2 + 2x}$$

Since the piles in the back and in the front of the cell are stopped at elevation 42 0 and 32 0 respectively, tan θ or x cannot be less than

$$\frac{42-32}{62\ 5} = 0\ 16$$

Substituting this value of 0 16 for x we get

H_r = 152 5 kips per lin ft of wall

For any value of x larger than 0 16, H_r becomes greater

The horizontal active pressure above line CD (elev 42) is as given in Figure 6 and is = 11 1 + 82 3 + 46 7 + 12 9 = 153 0 kips per lin ft of wall

Hence the factor of safety against sliding $= \frac{152\ 5}{153} < 1$ and it is not safe If we figure the cylinder to slide on line CD (elev 42) the factor of safety against sliding,

$$f_s = \frac{1\ 57(P\mu + C)R}{H}$$

Here H is equal to 153 kips less the passive pressure on BF (see Fig 5) The passive pressure on BF is gotten by substituting for x in equation (18) the value of 16 and is equal to 48 kips per lin ft of wall

$$\therefore f_s = \frac{1\ 57(6\ 14 \times \ 27 + \ 42)35}{153 - 48} = 1\ 09$$

Comparing this factor of safety with the factor of safety foi sliding obtained above, we see that critical sliding is the sliding along line CF, the cell however is not safe against sliding and will continue to be so unless we drive the piles in back of the cell into the yellow clay Assume that we will drive the entire cell to elevation +32

Figure 4 shows the pressure diagrams, as well as the total pressures and their locations with reference to bottom of cell for the pressure behind the cell and the passive pressure in front of the cell The overturning moment around bottom of cell (elev 32) is

M = 4894 kip-ft per ft of wall
H = 153 7 kips per ft of wall

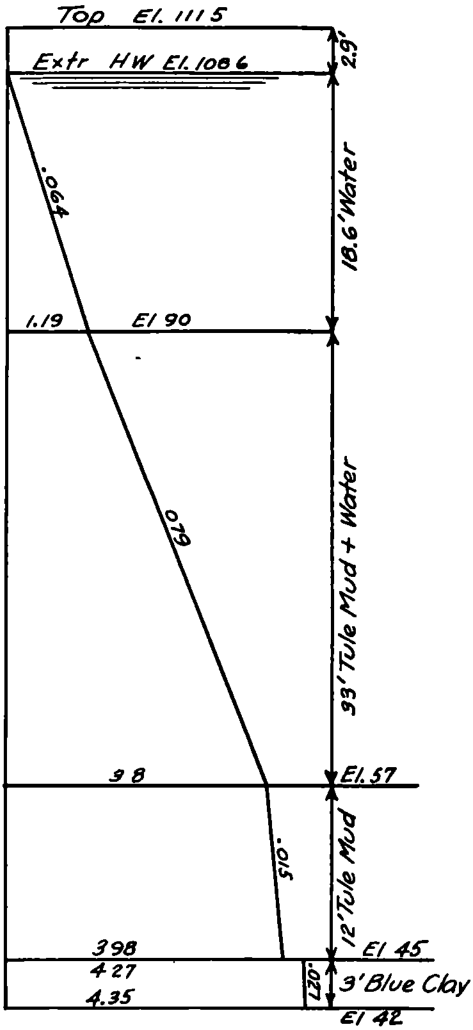

Figure 6

The average pressure on base (elev 32) is

$$P=6\ 65 \text{ kips per sq ft}$$

Now using formulae (2), (6) and (8) we get for factors of safety

$$f_0=2\ 6$$
$$f_s=1\ 6$$
$$f_v=1\ 17$$

The above factors of safety are considered satisfactory considering the temporary nature of the structure

It remains, however, to investigate the sliding of the cell on an inclined plane as well as the blowing of the bottom within the cofferdam

Using formula (17) and making $P=6\ 65$, $A=B=C=z=0$ we get

$$H_A=\frac{171\ 875x^2+695\ 31x-544\ 63}{1\ 16x+2}$$

In formula (18) all symbols used are the same as before except that z becomes now -3 and thus the net resistance to sliding is

$$H_r=H_P-H_A$$
$$=\frac{407\ 81x^3+887\ 65x^2-25\ 31x+640\ 63}{1\ 16x+2}$$

In order to get H_r a minimum we make

$\frac{dH_r}{dx}=0$, which gives $x=186$

Substituting this value of x in the equation for H_r we get

$$H_r=302\ 0 \text{ kips per ft of wall}$$

and $f_s'=\frac{H_r}{H}=\frac{302\ 0}{153\ 7}=1\ 96$ which is safe

The factor of safety against overturning, f_0 is less than 4 and the neutral axis therefore does not pass through the heel of the cell In order to find the maximum pressure on the toe we proceed as follows

The eccentricity of a cell of radius R is

$$e'=\frac{M_0}{P_0}=\frac{2MR}{P\pi R^2}$$
$$\therefore \frac{e'}{R}=0\ 38$$

With this value of $\frac{e'}{R}$ we enter diagram Figure 7, and proceeding according to instructions given thereon we find $b=1\ 2$ and therefore maximum compression at the toe is

$$C_0=\frac{P_0}{bR^2}=17\ 4 \text{ kips per sq ft}$$

In order to make the cofferdam safe against blowing, let us assume that we will drive the piles in the front (at the excavation side) to a depth y below elevation 32 The vertical intensity of pressure at the toe inside of the cell is

$$I_V=17\ 4+\ 06y \text{ kips per sq ft}$$

and $I_A=I_VK_A=5\ 22+\ 02y$

The vertical intensity at the same point on the excavated side is

$$1\ 062+\ 06y \text{ kips per sq ft}$$

and

$$I_P=4\ 92+\ 18y \text{ kips per sq ft.}$$

In order to be safe against blowing we must have

$$I_A=I_P \text{ or } y=1\ 9 \text{ ft}$$

This would indicate that the front piles should be driven to elevation $32-1.9$ or to elevation 30 1

If we leave the piles at elevation 32 or make $y=0$ in the above equation for I_A and I_P we get $I_A-I_P=0\ 30$ kips per sq ft which can be overlooked in view of the fact that the maximum pressure of 17 4 kips occurs only at one point

Our completed design is therefore as follows

Radius of each cell $=35\ 0$ ft

All sheet piles are to be driven to elev 32 0 and to have a minimum interlock strength of 12000 lb per lin inch

The fill inside of cells is to be sand weighing 100 lb per cu ft and having an angle of internal friction of at least 30° This fill is to extend from elev 52 0 to elev 111 5 Efficient drains are to be provided at elev 57 0

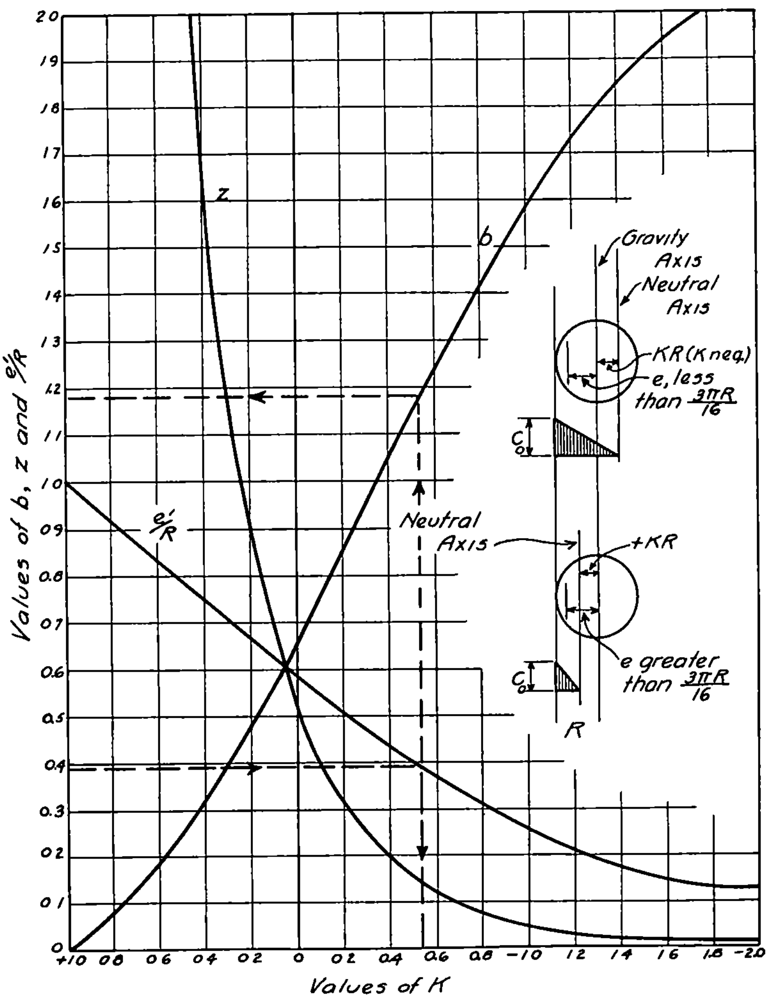

Figure 7

R Radius of circular footing, feet
M_0 Total moment at base of footing, kip-feet
P_0 Total vertical load at base of footing, kips
C_0 Unit pressure on soil (maximum) kips per square foot

1 With M_0P_0 and C_0 known—to find R

$$z = \frac{M_0^2 C_0}{P_0^3}$$

From value of z on z curve project vertically to intersect b-curve, reading value of b on vertical scale, substitute value of b in $R = \sqrt{\frac{P_0}{bC_0}}$

2 With M_0P_0 and R known—to find C_0

$$\frac{e'}{R} = \frac{M_0}{P_0 R}$$

From value of $\frac{e'}{R}$ on $\frac{e'}{R}$ curve project vertically to intersect b curve, reading value of b on vertical scale, substitute value of b in $C_0 = \frac{P_0}{bR^2}$

3. For location of neutral axis continue vertical projection in either case to K scale