STRUCTURAL DESIGN OF
A CEMENT FACTORY IN ALEPPO.

BY

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" This thesis is submitted to the Civil Engineering Faculty in partial fulfillment of the requirements for the degree of Bachelor of Science in Civil Engineering."

The candidate feels indebted to Prof. J.R.Osborn, chairman of the Engineering Faculty, for his supervision and valuable suggestions.

T A B L E   O F   C O N T E N T S.

TABLE OF PLANS.

I N T R O D U C T I O N .

Aleppo, a city with 350000 inhabitants in the northern part of Syria, had since a long time realized the need of a cement factory.

Both, the unusual increase of the population and the sudden expansion of some of the city's light industries (mainly textile) during the war years accelerated building construction to a great extent.

Although Aleppo is very rich in limestone quarries, concrete is coming more andmore into ùse, as it is more apt to satisfy modern building requirements.

To meet this need all over Syria, a factory was built in Damascus few years ago. But, unfortunately, the entreprise did not succeed in realizing the expected returns.

Both, inexperienced management and extravagant desires for benefits, helped in ruining this important entreprise.

The economic separation of Lebanon and Syria on March I4, I950 gave a new impetus to the old problem in Aleppo. The sudden rise in the prices of building materials led a group of contractors to establish a society, which enterdd immediately into contact with a german firm to prepare the project.

During the summer vacation , some of the plans were ready and the preliminary works started.

Although the design (both architectural and structural) of the entire factory would be very interesting, this thesis is confined to the structural design of some of the components.

Planning the entire factory requires not only a thorough technical knowledge of the manufacture of cement, but also some experience in machine design, which is beyond the capacities of the candidate.

In selecting the components of the factory for design, it was kept in mind to choose those which are more or less common to all factories.

EXPLANATION OF THE DESIGN PROCEDURES.

I. THE INCLINED PORTICO OR THE MILLBENT.

Due to the scarcety of steel and its consequent high cost, all large spans have to be executed by concrete in Aleppo. The arch form would of course be the best remedy. EXpensive form-work , however, prevents the extensive use of the arch. The millbent, with some improvements in the design, may solve the problem.

The millbent is considered to be a rigid frame consisting of members joined in such a way that at the joints, the construction is able to resist all the bending moments and shears.

Due to this requirement, the structure becomes statically indeterminate. The problem can then be solved by many simlifying propositions and theories such as Castigliano's Theorem of Displacement and the Elastic theory.

The design, in this thesis is based on the elastic theory. Tests and the performance of numerous structures have definitely proved the reliability of the elastic theory as applied to reinforced concrete.

Objections are raised that reinforced concrete does not act like a homogenous material, hence the deflections of the structure cannot be computed with exactness.

For all practical purposes, these objections are not valid. In applying the elastic theory, only the relation of deflections is used; so, the modulus of elasticity is finally eliminated.

Taylor, Thompson and Smulski recommend the following requirements to get successful results with a rigid frame.

I. The frame must be properly designed. At all points the most unfavorable bending moments and shears must be taken care of. Where reversal of bending moments is possible, the most unfavorable negative and positive bending moments must be provided for.

2. Proper foundation must be provided so that no unequal settlement takes place. Where appreciable settlement cannot be avoided, either a rigid frame should not be used or provision should be made to resist stresses produced by unequal settlement. The foundation must be able to resist the horizontal thrust.

3. The frame must be connected to the foundation in the manner contemplatedin the design. Obviously a frame designed as fixed at the support and built without any provision for fixity will not have the expected factor of safety.

4. Each frame should be constructed where possible in one continuous operation. When this is not possible, construction joints should be placed at points of minimum shear. To take care of any possible shear, recessing s in concrete should be provided so that old and new concrete should dovetail in the direction of the shear. Proper care should be made in joining old and new concrete. Any laitance should be removed. The surface should be roughened and neat cement paste spread on the top.

The millbent, as stated elsewhere, will be hinged at the base, to eliminate bending moments in the foundation. The only statically indeterminate value remains the the horizontal thrust at the hinges. This thrust may be easily computed from requirement, that when loaded, the hinges must remain on the same level and at the same distance apart.

To determine the formulas, one of the hinges is substituted with a roller, which is not capable of resisting any horizontal thrust. Under the loads, the ends spread. All reactions can be determined then and the bending moments computed.

To restore the original situation, the forces are computed that will bring the ends to their previous conditions.

All the formulas for design are taken from "CONCRETE" by Taylor, Thompson and Smulski.

THE WATER RESERVOIR.

In designing the various parts of the reservoir, advantage is taken of the circular form. The most peculiar in the lot is the dome.

Keeping a certain proportion between the rise and the span, the thrust is assumed to act in a direction normal to the section of concrete, which stands compressive stresses. The steel is made to stand the shearing forces perpendicular to the plane of the springing. It is, however, possible to follow the standard procedure of finding the moments at critical sections and providing the necessary steel and concrete. For this, wedge-shape sections are cut from the dome, with a unit length at the springing.

Where only tensile forces are acting, the concrete functions as a cover only: the walls of the reservoir are designed on this principle.

THE GRAIN ELEVATOR.

A comlete analysis of the design procedure is given with the computations.

NOTATION FOR THE DESIGN OF THE MILLBENT.

h	height of vertical member.
h	height of roof.
s	length of inclined member.
l	span of frame.
ϕ	angle of inclination of inclined member with horizontal.
I	moment of inertia of inclined member.
I	moment of inertia of vertical member.
H	horizontal thrust.
P	concentrated load.

All the designs are done with the foot-pound system, except that of the water reservoir.

DESIGN OF THE POWER HOUSE.

The power house will have a reinforced concrete framed structure, including inclined porticos for the roof and longitudinal purlins. This form of the roof has been selected, because

I. The span being quite large, an ordinary beam would have a great depth, hence would reduce the headroom and obstruct light.

2. Steel is more expensive than reinforced concrete in this locality for spans smaller than I00 feet.

3. An arched roof would require more expensive form-work.

The inclined porticos will be hinged at their base, because

I. Otherwise it would be necessary to construct the foun dations to resist fixed end moments, which means a great deal more expense.

2. The joint between the portico and the foundation can never be considered to be completely fixed.

3. The vertical members of the portico will be reduced at their base, adding available space.

Although, providing actual hinges is the most effective method, the MESNAGER hinges will be used

I. for the simplicity of their construction.

2. for their relatively small expense.

They consist of inclined bars imbedded in the foundation and in the frame in such a way that adjacent bars cross each other at the center of the hinge. Such bars resist shear but are not able to resist bending moments. With such construction, to allow free rotation of the frame, a clear space will be provided at the bottom by rounding up the top of the foundation and the bottom of the frame. The space will then be filled with asphalt.

SPECIFICATIONS.

Span	60 feet.
Height of columns	30 "
Height of roof	8 "
Spacing of frames	20 "
Live load (dynamic forces)	40 lbs./sq. foot.
DEad load of roofing	20 " " "
Wind load	30 " " "

STRESSES.

f_c = 800 p.s.i. f_{c_1} = 900 p.s.i.

n = I5 f_s=I6000 p.s.i.

f_c = 500 p.s.i. (in direct compression)

DIMENSIONS OF FRAME.

Inclined member.

Effective width=60/2 x I/5 x I2 = 72 inches.

Stem width = I4 inches.

h = 42 inches T = 4 inches.

t/h .4/42 = 0.095 b'/b = I4/72 = 0.I945

Using diagram I2, page I34,

Moment of inertia = 0.0275 x 72 x 42^3 = I46000 in.

Vertical member.

I4" x 50" M.I. = I/I2 x I4 x 50^3 = I46000 in.^{4}

Rigidity ratio

l = 60' h_1 = 8' s = $\sqrt{900+64}$ = 3I

$$\frac{I/S}{I/H} = \frac{I46000\ 30}{I46000\ 3I} = 0.97$$

DEad load

INclined member I4 x 42/I44 x I50 x 3I/30 = 634

Slab and roofing I0 x I.03 x (42+20) = 640

Stem of purlin I0 x I5/I44 x I50 = I56

796 #

Concentrated load at end of purlins

800 x (20-I) = I5200

Live load

At the beam ends $= 40 \times I0 \times 20 = 8000$ lbs.

Total concentrated load at panel points $= I5200$

8000

23200 lbs.

Wind load

$p = 30 \times 20 = 600$ lbs. per linear foot of frame.

DEAD LOAD AND LIVE LOAD MOMENTS.

Uniform load

Vertical reaction $V_A = 640 \times 60/2 = I9200$ lbs.

Horizontal reaction $H = I/32 \dfrac{8+5h_1/h}{I_1/I \; H/s \; h_1/h(3+h_1/h)+3} \; 1/h \cdot wl.$

$= I/32 \dfrac{8+5 \times 8/30}{I+30/3I \times 8/30(3+8/30)+3} \times 60/30 \times 640 \times 60$

$= 5900$ lbs.

Corner bending moments

$= 5900 \; 30 = - I77500$ ft-lbs.

Bending moments in inclined member

at $x = I/6 \; l$

$M = \left[I/2 \; x/l \; (I-x/l) - C_7 \; (I+2h_1/h \cdot x/l)\right] \cdot wl^2$

$= \left[I/2 \times I/6 \times (I-I/6) - 0.076(I+2 \times 8/30 \times I/6) \right] \; 640 \times 3600$

$= -30500$ ft-lbs.

at $x = I/3 \; l$

$M = \left[I/2 \cdot I/3 \times 2/3 - 0.076(I+0.I8)\right] \times 640 \times 3600 = 49400$ ft-lbs.

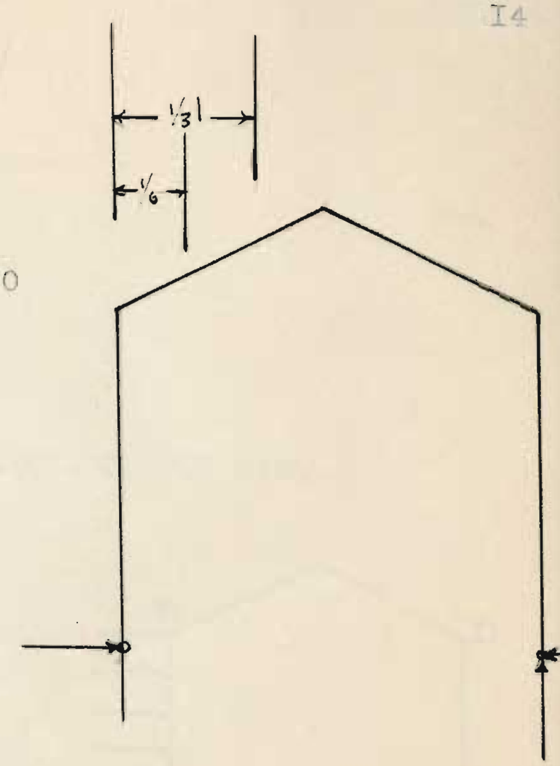
Moment at ridge

$M = \left[I/8 - 0.076(I \; 0.266)\right] \; 640 \times 3600 = +67500$ ft-lbs.

Concentrated load moments.

$H = l/h \; (5.833+3.667 \; h/h) \; C_8 P$

$C_8 = \dfrac{I}{4\left[I_1/I \cdot h/s \; h_1/h(3+h/h)+3\right.}$



$$= \frac{I}{4[I+30/3I+8/30(3+8/30)+3}$$

$= 0.065$

$H = 60/30(5.833+3.667\ 8/30) \times 0.065 \times 23200$

$= 20600$ lbs.

Corner bending moment

$0.885 \times 23200 \times 30 = -6I5000$ ft-lbs.

Moment at $x = I/6\ l$

$M = 5/I2\ Pl-(I+I/3\ h_1/h)\ Hh$

$= 5/I2 \times 23200 \times 60-(I+I_1/3 \times 8/30)20600 \times 30$

$= -90500$ ft-lbs.

Moment just above bracket

$I0000 \times 2 - I6800 = 3200$ ft-lbs.

Corner bending moment

$I0000 \times 2-672\ 38 = -I60$ ft-lbs.

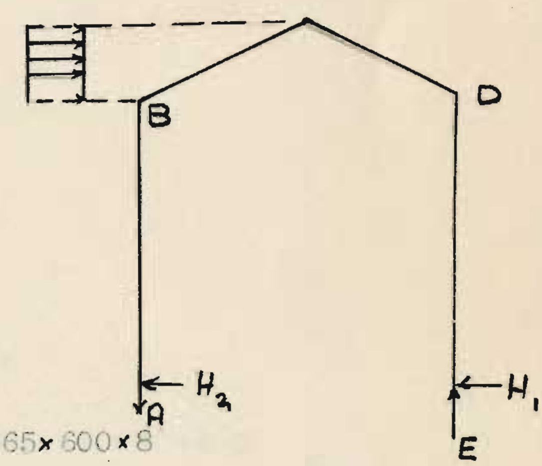
Bending moment at ridge

$20000 - 672 \times 38 = - 5600$ ft-lbs.

Bending moment in bracket

$-I0000 \times 2 = - 20000$ ft-lbs.

WIND LOADS



Wind on inclined member.

$R_A = -I/60(30+4)600 \times 8 = -2720$ lbs.

$R_E = 2720$ lbs.

Leeward horizontal thrust

$H = [2 \times I_1/I \times h/s+5/4\ h_1/h(4+h_1/h)+6]\ \gamma ph$

$= [2+I \times 30/3I+5/4 \times 8/30(4+8/30)+6]\ 0.065 \times 600 \times 8$

$= 2920$ lbs.

Windward horizontal thrust

$H = 600 \times 8-2920 = I980$ lbs.

Bending moments at corners

$M_B = I980 \times 30 = 59400$ ft-lbs.

$M_D = -2920 \times 30 = 87600$ ft-lbs.

Moment at ridge

MOment at ridge

$2720\times 30-(I+8/30)\times 2720\times 30 = 24400$ ft-lbs.

Wind on vertical member

$R_A = -I/2\times 30/60\times 600\times 30 = -4500$ lbs.

$R_E = +4500$ lbs.

$H_E = I/4\left[5\times I_1/I\times h/s+6h_1/h+I2\right] Cph$

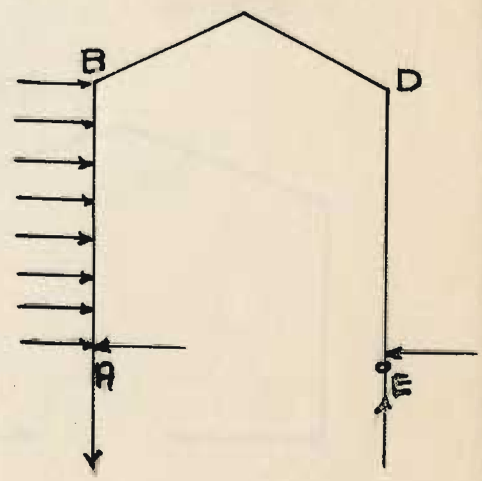
$I/4\left[5\times I\times 30/3I+6\times 8/30+I2\right]\ 0.065\times 600\times 30 = 5400$ lbs.

$H_A = 600\times 30-5400 = I2600$ lbs.

Bending moment in corner

$M_B = (I2600-9000)\times 30 = I08000$ ft-lbs.

$M = -5400\ 30 = -I62000$ ft-lbs.



Moment at $x = I/3\ l$

$M = 2/3\ Pl-(I+2/3\ h_1/h)\ Hh$

$= 2/3\ 23200\times 60-(I+2/3\times 8/30)\ 20600\times 30$

$= 204000$ ft-lbs.

MOment at ridge

$M = 3/4\ Pl-(I+h_1/h)\ Hh.$

$= 3/4\times 23200\times 60-(I+8/30)\ 20600\times 30$

$= 264000$ ft-lbs.

CRANE LOAD COMPUTATIONS.

A I0-ton crane load will be used.

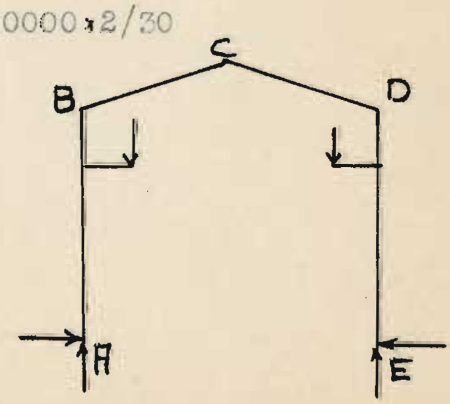
$H = 6\left\{I_1/I\ h/s\left[I-(h/h)^2\right]+h/h+2\right\} CPl_1/h$

$= 6\left\{I\times 30/3I\left[I-(5/30)^2\right]+8/30+2\right\}\times 0.065\times I0000\times 2/30$

$= 672$ lbs.

Moment just below bracket

$-672\ 25 = -I6800$ ft-lbs.



Bending moment at ridge

$4500\times 30-5400\times 38=-70000$ ft-lbs.

Maximum positive moment in column

$1/2\ p(H_1/p)^2=1/2\times 600\ (12600/600)^2=+132000$ ft-lbs.

Point of maximum moment

$12600/600=21$ feet.

RISE OF TEMPERATURE.

$H=12CE\alpha tI_1/2s$

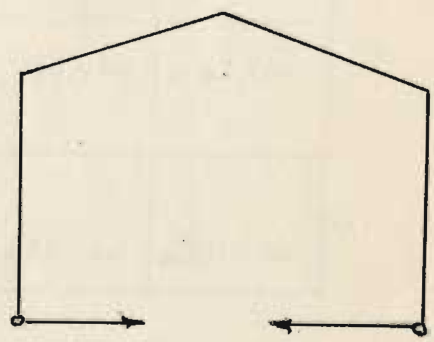
$=12\times 0.065\times 2500000\times 0.000006\times 40\times 146000\times 60/62$

$=3180$ lbs.

$M_B=M_D=-3180\times 30=-95400$ ft-lbs.

Moment at ridge

$3180\times 30=-121000$ ft-lbs.



Fall of temperature.

$M_B=+95400$ ft-lbs.

Moment at ridge $=+121000$ ft-lbs.

SUMMARIES OF BENDING MOMENTS

BENDING MOMENTS FOR DEAD AND LIVE LOAD

	POINT A	B	C	D
LIVE LOAD	-615 000	-90 500	+204000	+264000
Dead Load	-177500	-30300	+49400	+67500
Total	-792500	-120800	+253400	+331500

SUMMARY OF REACTIONS

	VERTICAL	HORIZONT.
LIVE LOAD	19200	5900
Dead LOAD	128720	20600
Crane Load	10000	670
WIND ON INCLIN.	2720	2920
WIND ON VERT.	4500	12600
Temperature		3180
Total	165140	49870

MAXIMUM POSITIVE MOMENTS

	A	B	C	D	above bracket	below bracket
D. L. + L. L.			253400	331500		
Crane					3200	
WIND ON INC.	59400		24400			
WIND ON VER.	108000					
Temperature	95400			121000		
TOTAL	262800		277800	452500	3200	

MAXIMUM NEGATIVE MOMENTS

	A	B	C	D	above bracket	below bracket
D. L. + L. L.	792500	120800				
Crane Load	160			5600		16800
WIND ON INCL.	87600					
WIND ON VERT.	162000			7000		
Temperature	95400			121000		
Totals	1137660	120800		133600		16800

As it is seen in the bending moment tables, large negative moments are developed at the knee. This is peculiar to all rigid frames. Hence particular attention should be paid in constructing those joints on the field.

To relieve part of the bending moment, the vertical members will be provided with cantilevered sheds, which will serve as parking place for the transporting trucks on one side of the power house, and as a repair platform on the other.

The span of the cantilever will be 25 feet. It will consist of ribs and slabs spanning the ribs. To help the ribs in compression and at the same time, to offer an even surface to the sight, the slab will be made flush with the bottom of the ribs.

DESIGN OF THE CANTILEVER.

D.L. of slab	$= 4/12 \times 8/2 \times 150$	$= 200$	lbs./lin.foot	
D.L. of rib		$= 450$	"	"
Snow load	$8 \times 30/2$	$= 120$	"	"
		770	"	"
	say	800	"	"

$M = 1/2 \times 800 \times 625 = 250000$ ft-lbs.

$d = \sqrt{250000 \times 12/139 \times 6 \times 12} = 30$ inches.

Overall depth use $= 35$ inches.

Width of rib $= 800 \times 25/120 \times 0.87 \times 30 = 6.5$ inches.

To furnish enough place for the steel, the width will be made 15 inches.

$A_s = 250000 \times 12/18000 \times 0.87 \times 30 = 6.4$ sq.inches

Use 9 ϕ 1 inch round bars.

DESIGN OF RIGID FRAME

Maximum negative moment $= 792500 - 250000 = 542500$ ft-lbs.

Maximum reaction $= 185140$ lbs.

Eccentricity $542500 \times 12/185140 = 35$ inches.

Specified stresses

$f_c = 650$ p.s.i. $f_s = 18000$ p.s.i. $n = 15$

Dimensions

$b = 14''$ $h = 50''$ $d' = 2''$

$d = 50 - 2 = 48$ $2a = 46$

Tension steel.

$A_s = (e/a + d_s/a - 1) N/2F + C_1bd$

Compression steel

$A'_s = C_2 (e/a + d_s/a + 1) N/2f - C_3bd$

$a/d = 23/48 = .48$ $e/a = 35/23 = 1.52$

$N/2f_s = 185140/2 \times 18000 = 5.15$

Assume $d_s/a = 0.1$ Then $e/a + d_s/a - 1 = 1.52 + .1 - 1 = 0.62$

$e/a + d_s/a + 1 = 1.52 + .1 - 1 = 2.62$

$k = 0.351$ $C_1 = 0.0003$ $C_2 = 2.1$ $C_3 = 0.012$

$A_s = 0.62 \times 5.15 + 0.0003 \times 14 \times 50 = 2.77$

Use 4 ϕ 1 inch round bars

$A'_s = 2.1 \times 2.62 \times 5.15 - 0.012 \times 50 \times 14 = 20$

Use 8 ϕ 1" round bars continuous from the inclined member

4 ϕ 1 sole bars (chapeaux)

The 4 bars for tension will be carried down to the base of the column. The four sole bars used at the knee will be carried do down to the crane console. Four of the 8 bars will be carried down to the ~~crane console~~ base, while the remaining four will be stopped at the middle of the column.

DESIGN OF INCLINED MEMBER.

Maximum positive moment = 452500 ft-lbs.

The thrust due to the inclination of the member will be neglected.

$K = 173$ $k = 0.400$ $j = 0.867$

stem width = 14" depth = 42"

$M_1 = 173 \times 14 \times 40^2 = 3888000$ in-lbs.

$M_2 = 5400000 - 3880000$ "

A_{s_1} 3880000/18000 × 0.867 × 40 = 6.2

A_{s_2} = 1520000/18000 (40-2.5) = 2.25

Total tensile steel = 8.45 □"

Compression steel 2.25 × 1-0.400/0.4-2.5/40 = 3.97 □"

THE WATER RESERVOIR

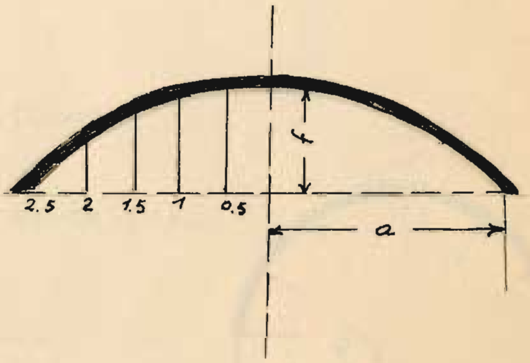
DESIGN OF THE COVER.

The cover, in the form of a bowl, will be designed according to a method proposed by the French engineer Dunod.

Let ACB be the profile of the bowl, the semi-chord being a and the flèche f.

The co-ordinates are:

X	H
0.5	0.72
I	0.63
I.5	0.48
2	0.27
2.25	0.I4
2.5	0



Radius of curvature of the bowl: 6.25+.56 / I.5 = 4.55

Assumed loads:

Dead load	300 Kgs. per sq. meter.
Snow load	50 " " " "
Wind load	50 " " " "
	400 " " " "

Developed surface of the bowl: 2×π× 4.55×0.75=2I.4 m

Total load=2I.4×400=8560 Kgs.

Load/lin. meter on circumference=8560/5π=545 Kgs.

Horizontal force/lin. meter on circumference

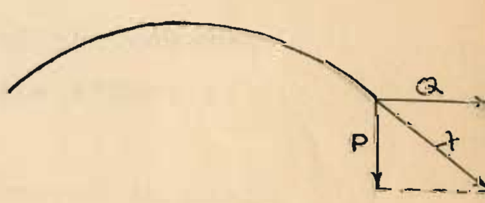
545 3.80/2.5 830 Kgs.

M_1 = − 830×0.63 = −523

0.86×I+0.4/2 ×I.5×400 = 362

+545×I.5 = +820

−65 Kg.- meters.



M_2 − 830×0.27 = 224

0.26×I+0.8/2×0.5×400 = 47

−27I

+545×0.5 +27I

000

Point 2 is the inflection point.

Moment at the top

$- 830 \times 0.75$	$= 623$
$2.5 \times 2/3 \times 2 \times 2.5 \times 400$	$= 833$
	$- 1456$
$+ 545 \times 2.5$	$+ 1360$
	$- 98$ Kgs.-meters.

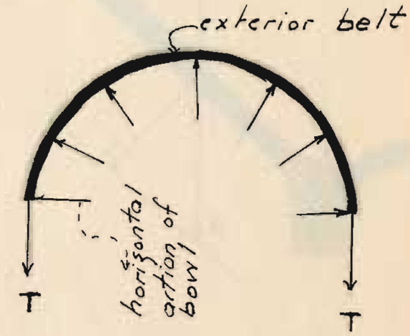
$M = 1/2 fkjbd^2$

$6500 = 1/2 \times 50 \times 0.375 \times 0.87 \times 0.40\ D^2$

$4.45 = D$ Take over-all $D = 10$ cms.

$A_s = 6500/1200 \times 0.87 \times 6 = 104$ mm.

Use 2 ϕ 10 m/m at the springing.



DESIGN OF THE EXTERIOR BELT.

$T = Q \times a = 830 \times 2.5 = 2080$ Kgs.

$A_s = 2080/1200 = 174$ m/m Use 4 ϕ 10mm.

This reinforcement takes care of the horizontal thrust only. No investigation need be made for the vertical component of the bowl's thrust, because the circular beam is supported all along its length.

The depth of the beam will be made 3o cms. and the width 4o cms., to house the exterior veneer wall and leave an indenting of 8 cms.

DESIGN OF THE WALL

This design assumes that the only function of the concrete is to house the steel that will counteract the effort of extension due to hydrostatic pressure.

The total height of the reservoir proper is divided into three parts of 83 cms. The design will be carried on in such a way so as to have a uniform spacing of the bars in the three parts. More bars of smaller diameter will be used to insure water tightness.

Steel section for the first part fromthe bottom

$A_s = 2.50/3 \cdot 500 \times 5/10 \times 2.50 = 520$mm.

Use 10 ϕ bars of 8m/m

Spacing 250/3 x 10 = 8 cms.

Steel section for the second part

$A_s = \frac{2.50}{3} \times 500 \times 5/10 \times 2.50 \ (1-1/3) = 347$mm.

USE 10 bars of 8m/m.

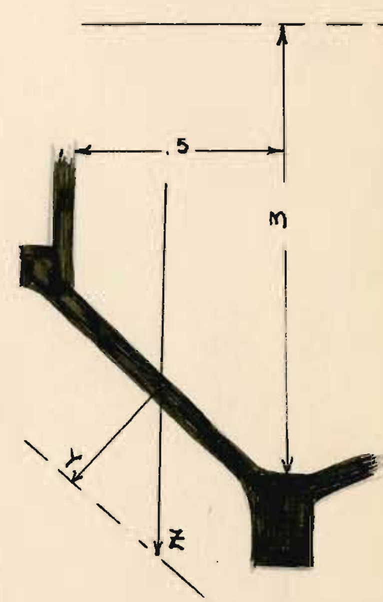
Spacing 8cms.

Steel section for the third part.

$A_s = 2.50/3 \times 500 \times 5/10 \times 2.5 \times (1-2/3) = 174$m/m.

Use 6 ϕ 8m/m

Spacing 250/3 x 6 = 12 cms.



DESIGN OF CANTILEVER.

Load acting on the cantilever

$2.5 + 3/2 \times 0.50 \times 2\pi \times 2.25 \times 1000 = 19400$ Kgs.

$Z = 19400/1.41 = 13800$ Kgs.

Load per linear meter $13800/2\pi 2.25 = 975$ Kgs.

Dead load $0.70 \times 1 \times 300 = 200$

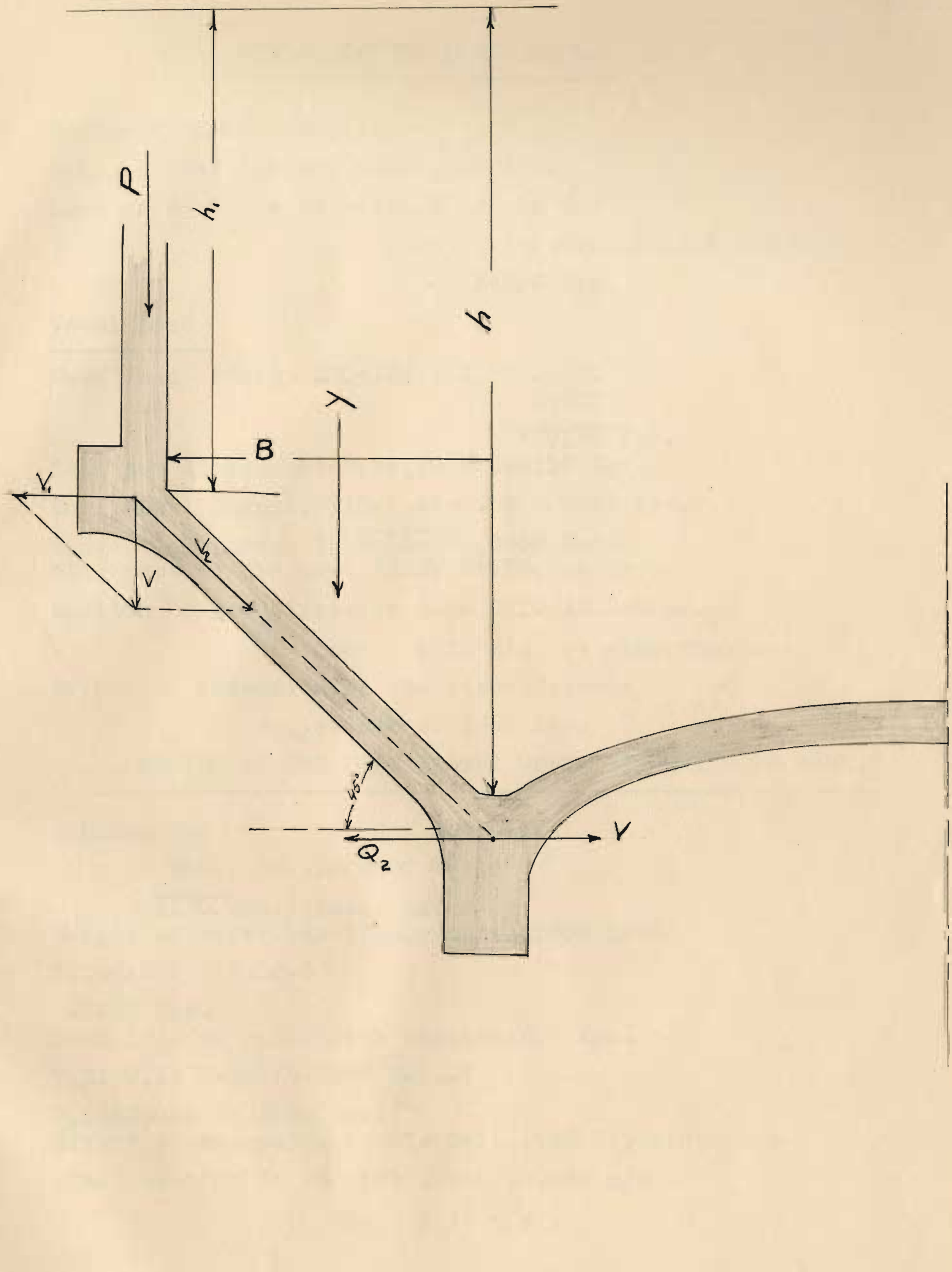
1175 Kgs.

The cantilever section of the reservoir is supported on one side by the circular beam of the bottom and on the other by the exterior belt; the circular form permits us then to design the cantilever as a slab.

$M = 1/10 \times 1175 \times 70 = 8200$ Kg-cm.

$D = 0.0367\sqrt{8200} = 3$cms. Use over-all depth 10 cms.

$A_s = 8200/12 \times 8 \times 0.87 = 100$mm. Use 4 ϕ 8 m/m per meter.


P
h₁
h
Y
B
V₁
V₂
V
45°
Q₂
Y

DESIGN OF THE LOWER BOWL.

Radius 4+0.25/0.150=4.25 m.

D.L. on bowl per sq. meter=350 Kgs.

Load on bowl due to water $= \pi (d^{3}h - f r_{1}^{2} f_{1}^{3}/3)$

$= \pi (4 \times 3 - 0.25 \times 4.25 + 1/8\ 1/3)$

= 34500 Kgs.

Total load

Dead load $2\pi r_1 f_1 q_1 = 2 \times \pi \times 4.25 \times 1/2 \times 350 = 4670$

34500

=39170 Kgs.

Load per linear meter=39170/2π2=3110 Kgs.

Horizontal thrust=3110×4.25-0.50/2=5840 Kgs.

Tangential thrust $10\sqrt{3400+970}=6600$ Kgs.

Thickness of the bowl=6600/100×10=6.6 cms.

Section of the directrix bars=3110/12=260 mm.

Use 4 ϕ 10 m/m at thespringing.

Effort of extension on the circumference

$E = Q_2 \times c = 5840 \times 2 = 11680$ Kgs.

DESIGN OF THE BELT AT THE UPPER EXTREMITY OF BOWL.

P=D.L.= 545 Roof load

625=2.50×.10×2500 Walls

1170=Kgs./linear meter.

Weight of water per linear meter=$500B(H_1+H)$

Y=500×1/2 (2.50+3

=1375 Kgs.

Dead load of cantilever section=200 Kgs.

$V_1 = 1170 + 1375 + 200/2 = 1958$ Kgs.=V

$V_2 = 1958/\cos 45° = 2760$ Kgs.

Effort of extension on the belt=1958×2.50=4850 Kgs.

Steel section in the belt=4850/10=485 m/m.

Use 6 ϕ 12 m/m.

DESIGN OF THE CIRCULAR BEAM AT THE BOTTOM.

Horizontal thrust of the cantilever = I958 × 2.25 = 4400 Kgs.
Horizontal thrust of the lower bowl = 5840 Kgs.
Resultant horizontal thrust = 5840-4400 = I440 Kgs.
Steel section in upper part of beam = I440/I0 = I44m/m
Total load on circular beam/linear meter

lower bowl	3II0
Upper "	II70
	I375
	5655 Kgs.

The circumference of the beam is I4 meters; it will be supported by four columns: the length of each beam will be then 3.5 meters.

Total weight of the superstructure of the reservoir

5655 × I4 = 79000 Kgs.

MOMENTS.

Negative moment at the support = -0.034I5 Nr

0.034I5 × 79000 × 2.25 = 6050 Kg-m.

POsitive moment in the middle = +0.02762 Nr

0.02762 × 79000 × 2.25 = 4920 Kg-m.

Torsional moment 0.0053 Nr

0.0053 × 79000 × 2.25 = 945 Kg-m.

$605000 = I/2 \times 50 \times 0.33 \times 0.87 \times 70\ d^2$

53 cms. = d Use overall depth = 60 cms.

A_{s_1} = 605000/I2 × 0.87 × 53 = I090 mm. for negative moment.
A_{s_2} = 492000/I2 × 0.87 × 53 = 890 mm. for positive moment.
A_{s_3} = 94500/I2 × 0.87 × 53 = I7I mm. for torsion.
Total steel section at support I405mm.

Use 4 ϕ I2 m/m at the top
4 ϕ I6 m/m bent up
4 ϕ I8 m/m at thebottom

DESIGN OF COLUMNS.

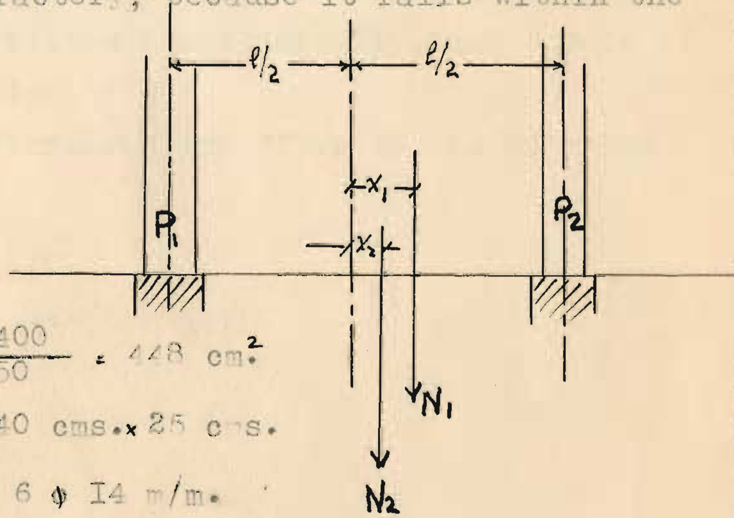
The wind load is taken at I35 kilograms per square meter of curved surface.

Total wind load $2\pi \times 2.5/2 \times 2.5 \times I35 = 2650$ kgs.

Moment due to the wind $= 2650 \times 9 = 23850$ kgs-meters.

Eccentricity of applied loads $= \frac{23850}{30000} = 0.8$ meters.

which is satisfactory, because it falls within the middle third.



$P_1 + P_2 = 2N/4 = N/2$

$P_1 \times (1/2 + x_2) = P_2\,(1/2 - x_2)$

$P = 80000/2 \times (0.30/5 + 0.50)$

$= 80000/2 \times 0.56$

$= 22400$ kgs.

Section of the columns $= \frac{22400}{50} = 448$ cm^2.

Use a section 40 cms. x 25 cms.

Reinforcement 6 ɸ I4 m/m.

DESIGN OF FOUNDATIONS.

The foundations of the water reservoir will be in the form of continuous inverted T-beams. Such a form is particularly essential in this instance, because due to the height of the structure, the wind loads may produce excessive moments and hence throw the resultant out of the middle third with the consequent lift of part of the foundations. A continuous inverted T-beam foundation holds the whole structure together.

Let P = column load

p = bearing power of soil 3 kgs./cm.

a = width of foundations.

l = distance center to center of columns.

Then $P = pal = 3 \cdot a \cdot 350 = 22400$

$a = \frac{22400}{350\ 3} = 21.4$ cms.

$M = 3/2 \times (50-25)/8 = II7$ kgs-cms.

As it is seen, both the section of concrete and the reinforcement are exceedingly small, as the whole structure weighs only about 80 tons when full of water. Hence the sections will be made large enough for practical purposes.

Width of foundations 50 cms.

Depth of foundations 40 cms. with IO cms of concrete blinding.

The inverted T-beams will have 4 bars of I2 m/m at the top, and 6 bars of I4 m/m at the bottom longitudinally, and 5 bars of I2 m/m per meter transversally.

The details of the reinforcement are given in the attached plans.

DESIGNOF THE GRAIN ELEVATOR.

The problem of calculating the pressure of grain on bin walls is somewhat similar to the problem of the retaining wall, but is not so simple. The theory of Rankine will apply in the case of shallow bins with smooth walls where the plane of rupture cuts the grain surface, but will not apply to deep bins or bins with rough walls.

Stresses in deep bins.

Nomenclature:

ϕ = angle of repose of the filling:

ϕ' = the angle of friction of the filling on the bin walls:

$\mu = \tan\phi$ = coefficient of friction of filling on filling:

$\mu' = \tan\phi'$ = coefficient of friction of filling on the bin walls:

x = angle of rupture:

w = weight of filling in lb. per cu. ft.:

V = vertical pressure of the filling in lb. per sq. ft.:

L = lateral pressure of the filling in lb. per sq. ft.·

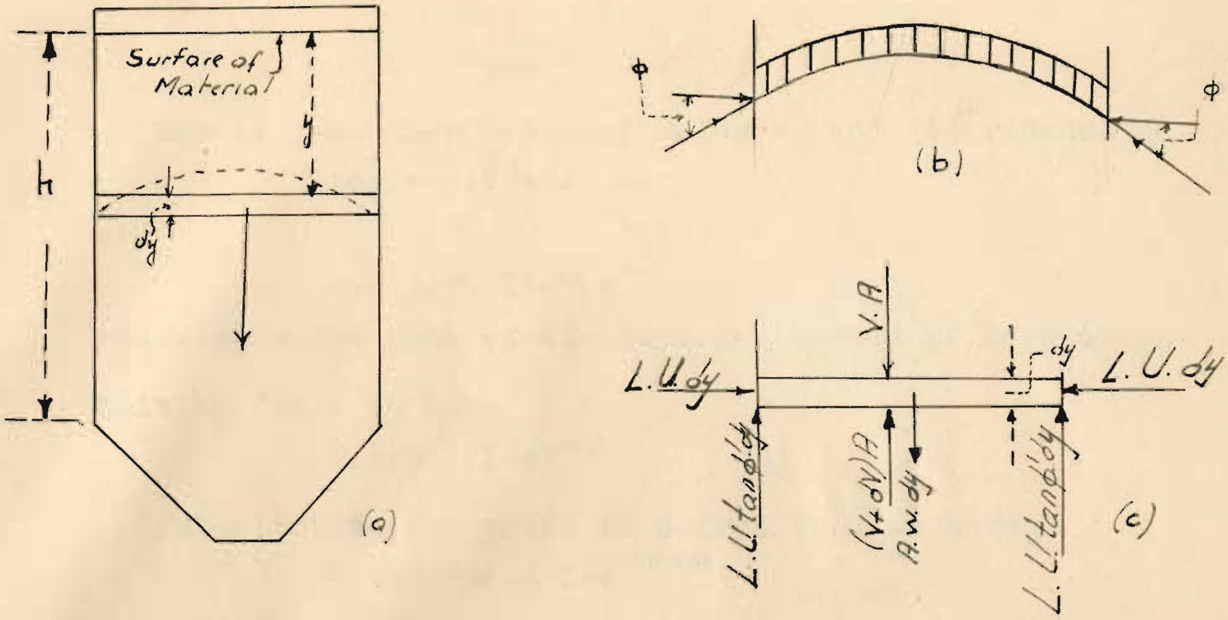
A = area of bin in sq. ft.:

U = circumference of bin in ft.:

R = A/U hydraulic radius of bin.

The following solution belongs to Janssen and is considered thebest by many authorities:; itis reproduced here integrally.

The bin in (a) Fig.I, has a uniform area A, a constant circumference U, and is filled with a granular material weighing w per unit of volume, and having an angle of repose ϕ Let V be the vertical pressure, and L be the lateral pressure at any point, both V and L being assumed as constant for all points on the horizontal plane. (More correctly V and L will be constant on the surface of a dome as in (b).)



The weight of the granular material between the sections of y and $y+dy = A \cdot w \cdot dy$; the total frictional force acting upwards at the circumference will be $L \cdot U \cdot \tan \cdot dy$; the total perpendicular pressure on the upper surface will be $V \cdot A$: and the total pressure on the lower surface will be $(V\ dV)A$.

Now these vertical pressures are in equilibrium, and

$$V \cdot A - (V + dV)A + A \cdot w \cdot dy - L \cdot U \cdot \tan\phi' \cdot dy\ o$$

and

$$dV = (w - L \cdot \tan\phi' U/A)dy \quad \ldots\ldots (1)$$

Now in a granular mass, the lateral pressure at any point is equal to the vertical pressure times k, a constant for the particular granular material, and

$$L = k \cdot V$$

Also let $A/U = R$ (the hydraulic radius), and tan

Substituting the above in (I) we have

$$dV = (w - k \cdot V/R\ \gamma')dy$$

Now let

$$k \cdot \gamma'/R = n \quad \ldots\ldots (2)$$

and

$$dV/w - n \cdot V = dy \quad \ldots\ldots (3)$$

Integrating(3) we have

$$\log (w - n \cdot V) = -n \cdot y + C \quad \ldots\ldots (4)$$

Now if $y=o$, then $V=o$, and $C=\log w$, and (4) reduces to

$$\log(w-n.V/w)= -n.y$$

and

$$w-n.V/w= I/e^{n.y}= e^{-n.y}$$

where e is the base of the Naperian system of logarithms.

Solving for V we have

$$V= w/n \quad (I-e^{-n.y}) \quad \ldots\ldots (5)$$

Substituting the value of n from (2), we have

$$V= w.R/k.\mu' \quad (I-e^{-k.\mu'.y/R}) \quad \ldots\ldots (6)$$

Also since

$$L = k.V$$

$$L=w.R/\mu' \quad (I-e^{-k.\mu'.h/R}) \quad \ldots\ldots (8)$$

For deep bins with a depth ofmore than two and one-half diameters thelast term of the right hand member of (8) may be omitted, and

$$L=w.R/ \quad \text{(approx.)}$$

Now both μ' and k can only be determined by experiment on the particular grain and kind of bin.

LOAD ON BIN WALLS.

The walls of a deep bin carry the greater part of the weight of the contents of the bin. The total weight carried by the bin walls is equal to the total pressure, P, of the grain on thebin walls, multiplied by the coefficient of friction of the grain on the bin walls.

From formula (8) theunit pressure on a unit at a depth y will be

$$L= w.R/\mu' \quad (I-e^{-k.\mu'.y/R}) \quad \ldots\ldots\ldots (10)$$

and the total lateral pressure for a depth y, per unit of length of the perimeterof the bin, will be

$$P=\int_0^y L.dy= \int_0^y w.R/\mu' \quad (I-e^{-k\mu'.y/R})\,dy$$

$$= w.R/\mu' \left[(y- R/k\mu' + R/k\mu' \times e^{-k\mu'.y/R}\right] \quad \ldots\ldots (11)$$

Now the last term in (II) is very small andmay be neglected for depths of more than two diameters, and

$$P= w.R/\mu' \left[(y-R/k.\mu')\right. \quad \ldots\ldots (12)$$

The total load per lineal foot carried by the side walls of the bin will be

$$P.\mu' = w.R\left[(y - R/k.\mu')\right] \quad \text{------ (13)}$$

For the total weight of grain carried by the side walls multiply (I3) by the length of the circumference of the bin.

DESIGN DATA.

angle ofrepose	= 28°	w=I20 lbs./sq.ft.
height	= 40 ft.	μ= 0.7I
k	= 0.3	diameter=I5 feet.

Maximum lateral pressure

$$L = I20 \times 7.5/0.7I \ \ (I - e^{-0.3 \times 0.71 \times 40/7.5})$$

$$= 860 \text{ lbs./sq.foot.}$$

Load carried by the side walls per lineal foot

$$P = I20 \times 7.5 \ (40 - 7.5/0.3 \ 0.7I \times 0.677)$$

$$= I440 \text{ lbs.}$$

Total load carried by walls

$$I440 \times 2 \times 7.5 = 67500 \text{ lbs.}$$

Total weight of filling material

$$40 \times \pi \times 225/4 \times I20 = 850000 \text{ lbs.}$$

Weight carried by the tremie at the bottom

$$850000 - 67500 = 772500 \text{ lbs.}$$

$T_1 = 860 \times I5/2 = 6450$ lbs.

The windhelps increasing this tractive force.

So, at the rate of 30 lbs./sq.foot and with a coefficient of 0.6, the traction due to wind action will be

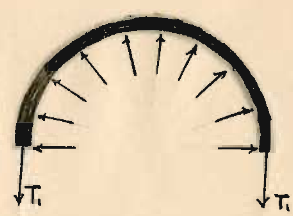
$$30 \times I5 \times 0.6 = 270$$

$$T_2 = 270/2 \ \ I35 \text{ lbs.}$$

$$T = 6450 + I35 = 6585 \text{ lbs.}$$

$$A_s = 6585/I8000 = 0.366 \ \square''$$

Use 2 ϕ I/2 inch round bars.

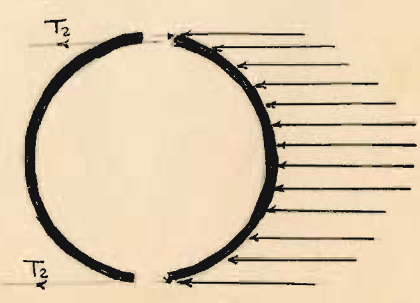


Spacing for the first I0 feet = 5 inches

" " " second" " = 7 "

" " " third " " = 9 "

" " " fourth " " = I0 "



To keep the bars in place, use I/2 inch round bars in the vertical direction, two per foot.

DESIGN OF TREMIES.

Weight carried by tremies = 772500 lbs.

Load per sq. foot = $772500 \; 4/\pi \times 225 = 4350$ lbs.

$q_1 = 4350 \cos 45° = 3080$ lbs.

$q_2 =$ " " $= 3080$ lbs.

The tremie will be considered as a slab supported on one side by the bin wall, and on the other by the circular belt.

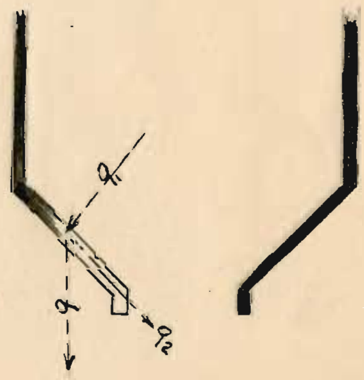
$M = I/I0 \times 3080 \times (7.75)^2 = I8480$ ft.-lbs.

$d = \sqrt{I8480 \times I2 / I73 \times I2} = I0$ inches

Use overall depth = I2 inches.

$A_s = I8480 \times I2 / I8000 \times 0.87 \times I0 = I.4I$ □"

Use 3 o 7/8 inch round bars/foot



Total force parallel to inclination

$3080 \times 7.75 = 24000$ lbs.

Section of steel necessary to resist this tractive force = $24000 / I8000 = I.33$ □"

Use 3 ɸ 3/4 inch round bars.

Place the 7/8 inch bars at the bottob of the tremie and the 3/4 inch bars at the bottom.

B I B L I O G R A P H Y.

I. Ketchum's structural engineers' handbook.

II. "Concrete, plain and reinforced" by Taylor, Thompson and Smulski.

III. Beton Arme by Dunod.

IV. "Cement" by Bertram Blount.