# THESIS

## ANALYSIS OF DESIGN
## M. A. C. WATER TOWER.

J. U. LAYER. A. J. RITCHIE.

1916

ANALYSIS OF DESIGN

OF

M. A. C. WATER TOWER.

A THESIS SUBMITTED TO

THE FACULTY OF

MICHIGAN AGRICULTURAL COLLEGE

by

J. [illegible]. Layer

A. J. Ritchie

Candidates for the Degree of

BACHELOR OF SCIENCE

June, 1916.

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# INTRODUCTION.

During the winter and early spring of 1916 a steel water tower was built on the M.A.C. campus, just in the rear of the Farm Mechanics Bldg. The tower is 177½ ft high, from the top of the piers to the finial; and the cylindrical tank has a capacity of 30,000 gals. The steel work was designed, furnished and put in place by the Chicago Bridge and Iron Co. The College built the concrete foundations, and put the finishing coats of paint on the steel.

The water tower was built as a remedy to meet difficulties arising from the high pressure due to an increase in pumping capacity. Several years ago difficulty was encountered because not enough water could be obtained from the wells to meet the peak loads. This necessitated the driving of a 12 in. well, in the rear of the Forestry Bldg., and installing an electrically driven 400 gal. per minute pump, which is sufficiently large to meet the anticipated future growth of the College. The pumping capacity now consists of the 400 gal. pump mentioned above, and a 100 gal. steam driven pump, attached to three of the older wells. As the ordinary consumption of the College is about 150 gals. per minute; and it is planned to run the steam pump constantly, this leaves only 50 gals. per minute to be pumped by the 400 gal. pump. With the 400 gal. pump working directly on the mains the pressure would become excessive, and would be apt to burst the pipes . So the water tower was constructed to relieve this pressure; and the 400 gal. pump need only to be run for 3hrs. a day, to meet the requirements of the system.

The site upon whibh the tower was built was selected because the tower will not in any way conflict with the future building operations.

## Specified Unit Stresses

Specifications of C.W. Birch-Nord.

Ketchums S.H.B. Pg.379.

Tension in tank plates 12000#/sq. in. net area

Tension in other parts of structure 16000#/sq.in. net area

Compression 16000#/sq.in. (reduced)

| | | |
|---|---|---|
| Shear on shop rivets and pins | | 12000#/sq.in. |
| " " field " (tank rivets) and bolts | | 9000#/sq.in. |
| " in plates | (Gross Area) | 10000#/sq.in. |
| Bearing pressure on shop rivets and pins | | 24000#/sq.in. |
| " " " field " (tank rivets) | | 18000#/sq.in. |
| Fiber strain in pins | | 24000#/sq.in. |

For compression members the permissable unit stress of 16000# shall be reduced by the formula

$$P = 16000 - \frac{70\,L}{R}$$

## Wgt. of Piers

Volume of one pier

V= 10x10x1 *( 16 * 4 x6.5 x6.5 * 81) x 4.25/6= 288 cu.ft.

Wgt. of concrete per cu. ft. = 140#

140 x 288 * 40,300# wgt. of one pier.

## Centre of gravity of pier

Pyramid extended to apex.

Altitude of pyramid = 7.65 ft.

" " frustrum = 4.25

Volume of total pyramid = 81 x 7.65/3 = 206.5 cu. ft.

Distance of c. of g. above base of pier = 2.91 ft.

Volume of small pyramid = 16 x 3.4/3 = 18.15 cu. ft.

Distance of c.of g. above base = 6.1ft.

Volume of Parallelopiped = 10 x 10 x 1 = 100 cu. ft.

Distance of c. of g. above base = .5 ft.

Distance of c.of g. of entire pier above its base

Bar x = 206.5 x 2.91 * 100 x .5 - 18.15 x 6.10/288.4

Bar x = 1.88 ft.

**NOTE THIS STAR AS SHOWN * USED AS A PLUS SIGN THROUGHOUT THIS THESIS.**

## Wgt. of tower and tower bracing in detail to balcony

| Panel | Member | Make up | Wgt./ft. | Length | No. of mem. | wgt./mem. | Total wgt. |
|---|---|---|---|---|---|---|---|
| Panel 1 | Posts | 1Chan.10"a15# | 15# | 14'_o½" | 4 | 211. | 844 |
| | | 1Chan.8" a11.15# | 11.15 | 11'-o7/16" | 4 | 124. | 496. |
| | Tower rods | 5/8" s9 | 1.325 | 9'-5½" | 8 | 12.5 | 100 |
| | Pipe Rods | 5/8 a | 1.043 | 9'-3½" | 4 | 9.7 | 19 ½on pipe |
| | Struts | 2Chan.5" a 6.5# | 13# | 13'-3 3/8" | 4 | 173. | 692. |
| Panel 2 | Posts | 1Chan.10" a 15# | 15# | 20'- 8 5/8" | 4 | 545. | 2180 |
| | | 1Chan.8" a 11.25# | 11.25 | | | | |
| | Tower rods | ¾ " s9 | 1.91 | 24'-11½" | 8 | 47.7 | 382 |
| | Pipe rods | 5/8 " a | 1.043 | 11'-7¾" | 4 | 12.2 | 24 |
| | Struts | 2Chan.5" a 6.5# | 13 | 16'- 7 3/8" | 4 | 216 | 864 |
| Panel 3 | Posts | 1Chan.10" a 15# | 26.25 | 20'- 8 5/8 " | 4 | 545 | 2180 |
| | | 1Chan.8" a 11.25# | | | | | |
| | Tower rods | 7/8 " s9 | 2.603 | 22'- 2¼ " | 8 | 57.7 | 462 |
| | Pipe rods | 5/8" a | 1.043 | 14'- 0" | 4 | 14.6 | 29 |
| | Struts | 2Chan. 5" a 6.5# | 13 | 19'-117/16" | 4 | 259 | 1037 |
| Panel 4 | Posts | 1Chan.10" a 15# | 26.25 | 20'- 85/8" | 4 | 545 | 2180 |
| | | 1Chan.8" a 11.25# | | | | | |
| | Strut rods | ¾ " a | 1.502 | 21'- 2¼ " | 4 | 31.8 | 127 |
| | Tower rds | 7/8" s9' | 2.603 | 23'- 7" | 8 | 61.5 | 492 |
| | Pipe rods | 5/8" a | 1.043 | 16'- 4½" | 4 | 17.1 | 34 |
| | Struts | 2Chan.5"-6½# | 13 | 23'-3½ " | 4 | 303. | 1212 |
| Panel5 | Posts | 1Chan.10" a 15# | 26.25 | 20'-8 5/8" | 4 | 545 | 2180 |
| | | 1 Chan.8" a 11.25# | | | | | |
| | Strut rods | ¾" a | 1.502 | 21'-2¼" | 4 | 31.8 | 127. |
| | Tower Rods | 7/8" s9 | 2.603 | 23'=7" | 8 | 61.5 | 492. |
| | Pipe rods | 5/8" a | 1.043 | 18' - 8¾ " | 4 | 19.5 | 39 |
| | Struts | 2Chan.5" a 6.5# | 13 | 26'- 7½" | 4 | 346 | 1385 |
| Panel 6 | Posts | 1Chan.10" a 15# | 28.75 | 20'- 8 5/8" | 4 | 596 | 2384 |
| | | 1Chan. 8" a 13.75# | | | | | |
| | Strut Rods | ¾" a | 1.502 | 21'- 2¼" | 4 | 31.8 | 127 |
| | Tower rods | 7/8" s9 | 2.603 | 25'- 3¾ " | 8 | 66 | 527 |
| | Pipe rods | 5/8" a | 1.043 | 21'-1" | 4 | 22 | 44 |
| | Struts | 2 Chan.5" a 6.5# | 13 | 29'- 11½" | 4 | 390 | 1560 |

| Panel | Member | Make up | Wgt./ft. | Length | No. of mem. | Wgt./mem. | Total wgt. |
|---|---|---|---|---|---|---|---|
| Panel 7 | Posts | 1Chan.10"-20# 1Chan.8"-13.75# | 33.75 | 20'-85/8" | 4 | 700 | 2800 |
| | Strut rods | ¾" round | 1.502 | 21'-2¼" | 4 | 31.8 | 127. |
| | Towerrods | 1" s9 | 3.4 | 25'-3¾" | 8 | 86. | 688. |
| | Pipe rods | 5/8" round | 1.043 | 23'-5¼" | 4 | 24.5 | 49. |
| | Struts | 2Chan.5" a 6.5# | 13 | 33'-3½ | 4 | 433 | 1732 |
| Panel8 | Posts | 1Chan. 10# a 20# | 20 | 20'-117/8 | 4 | 420 | 1680 |
| | | 1Chan. 8" a 13.75# | 13.75 | 20'-119/16 | | 288 | 1152 |
| | Tower rods | 1"s9 | 3.4 | 27'-0" | 8 | 91.8 | 735. |
| Stem | Ladder bars | 2"x3/8" | 2.55 | 156' | 2 | 398 | 796 |
| Ladder | rungs | ¾ round | 1.502 | 1-2¼ | 156 | 1.78 | 278 |
| Indicate guide | Ind.guide | 1Chan 6" a 8# | 8 | 25'-6" | 1 | 204 | 204 |
| bent bars for ladder | | 4½ x¼ | 3.825 | 10½" | 32 | 3.35 | 107. |
| Struts to parts | (Splice plate bent | 10¼ x5/16 | 10.89 | 1'-2¼" | 56 | 12.93 | 665 |
| | " " " | 16¾ x 5/16 | 17.78 | 15¾ | 28 | 23.0 | 664 |
| | (splice plate | 4½ x 5/16 | 4.52 | 10¾" | 28 | 3.86 | 108 |
| | lacing bars | 1½ x¼ | 1.488 | 12" | 1232 | 1.488 | 1835 |
| | Pins in struts. | 2" round | 10.68 | 12" | 42 | 10.68 | 448 |
| Rivets in | (Struts | | 17.2/100 | | 1232 | | 212 |
| | (Splice plates | 5/8" | 17.2/100 | | 2660 | | 457 |
| | (Posts | | 17.2/100 | | 1248 | | 214 |
| | (Tankconnection | | 9/100 | | 300 | | 27 |
| | (Ladder connection | | 17.2/100 | | 112 | | 19 |
| | (Plates | 8"x3/8" | 10.2 | 5'-5¾" | 4 | 56 | 224 |
| At post &strut conn. | Chans.conn to tank | 6"x3½ x 3/8 | 11.7 | 5'-0½" | 8 | 59 | 472 |
| splice channels | | 10"Chan a 15# | 15 | 0'-8¼" | 28 | 10.32 | 283 |
| ANCHOR for strut.rods | | L 3x3x3/8 | 7.2 | 0'-9½" | 16 | 5.7 | 91 |
| | | | | | | | 16052 |
| | | | | | | Total | 38,271# |

Total wgt. of tower piers to balcony not including the shoes.

| Part | Member | Make up | wgt./ft. | Length | No. of members | weight of mem. | total wgt. # |
|---|---|---|---|---|---|---|---|
| | Fill plate | 2½ x 3/8 | 3.19 | 4¼" | 8 | 1.13 | 9. |
| | Bent " | 9" x 3/8" | 11.48 | 9¼" | 8 | 8.85 | 70 |
| Four | L S. | 5 x 3x 3/8 | 9.8 | 1'-8" | 8 | 16.33 | 131 |
| shoes | Bent L | 3x5x3/8 | 9.8 | 0'-8¼" | 4 | 6.74 | 27 |
| | L S | 3x5x3/8 | 9.8 | 1'-05/8" | 8 | 10.0 | 80 |
| | Channel | 10" | 15.0 | 0'10¼ | 4 | 12.85 | 51 |
| | Bott. Plate | 16"x 3/4" | 40.8 | 1'- 8" | 4 | 68.1 | 273 |
| | Rivets | 5/8" | 17.2/100 | | 180 | | 31 |

Total 672 #

F---Wgt. of hemispherical bottom including expansion.

| Name of Part | Make up | wgt./ft. | wgt./cuft. | wgt/mem | no.of mem. | area sq. ft. | volume cu.ft. | total wgt. | length of mem. |
|---|---|---|---|---|---|---|---|---|---|
| Hemisph bottom. | 16' diam. 3/16"thick hole for 8" pipe including lap joints | | 489.6# | | | 419' | 6.55 | 3210# | |
| Comm p/t | 8¼"X 5/16" x11" | 9.3# | | 8.53 | 4 | | | 34# | 0'-11" |
| Stuffing Box C.L. | 33.8" x¼ x4" | | 450# | | | | .0587 | 26# | 33.8" |
| Brass gland | 29.3" x⅛" X4" | | 525# | | | | .034 | 18# | |
| Brass ring | ⅛" X¼"X29.3" | | 525# | | | | .006 | 3# | |
| Rivets | ½" diam | 9#/100 | | | 204 | | | 18# | |
| " | 5/8" " | 17.2/100 | | | 20 | | | 3# | |
| | | | | | | Total | | 3312# | |

Wgt. of 50% of Pipe and 2 ply Casing

Timber wghs 40# per cu. ft. or .0231 # per cu. in.

| | |
|---|---|
| 127.5x12x0.75x70.8x0.0231= | 1880# |
| 127.5x12x0.75 x 44 x0.0231= | 1170 |
| 48x2x2x0.75x57.2x0.0231 = | 350 |
| 96 x 2 x3 34.5 x0.0231 = | 460 |
| 127.5 ft.of 8 in W. I. pipe @ 28.177#per ft | 3600 |
| Total= | 7460# |

50% of Pipe and casing = 3730#

WGT. OF TANK ROOF, FINIAL, LADDER AND INDICATOR -CONTINUED

| Name of part | Make up | Wgt.#/ft. | Wgt.#/cu. ft. | Length | No.of mem.s | Area sq. ft. | Vol. cu. ft. | Wgt#/mem. | Total wgt. |
|---|---|---|---|---|---|---|---|---|---|
| Ladder bars | 2 1/2 x 3/8 | 3.19 | | 125' | 2 | | | | 82# |
| Ladder rungs | 3/4 rd. | 1.502 | | 1'-2 1/4 | 13 | | | 1.78 | 24 |
| Ind.rope and float | | | | | | | | | 75 |
| | | | | | | | | TOTAL | 2741# |

WGT. OF TANK CYLINDER

| Name of part | Make up | Wgt#/ft. | Length | No of mem.s | Wgt#./mem. | Total wgt.# |
|---|---|---|---|---|---|---|
| Pl.#3 | 61 1/2" x 3/16 | 39.2 | 12'-8 11/16" | 4 | 498 | 1992 |
| Pl.#4 | " " | 39.05 | 12'-8 9/16 | 4 | 497 | 1988 |
| pl.#5 | " " | 39.2 | 12'-8 7/8 | 4 | 500 | 2000 |
| Angles | 3" x 2" x 1/4 | 4.1 | 13'-0 1/2 | 4 | 53.5 | 214 |
| Rivets | 3/8 | 43#/100 | | 1450 | | 63 |
| Overflow | 3" pipe | 18 | 1'-8" | 16 | 30 | 480 |
| | | | | | TOTAL | @6900# |

## WGT. OF TANK ROOF, FINIAL, LADDER AND INDICATOR

| Name of part | Make up in. | Wgt#/ft. | Wgt.#/cu. ft. | Length | No. of mem.s | Area sq. ft. | Vol. cu. ft. | Wgt. of mem. | Total wgt. |
|---|---|---|---|---|---|---|---|---|---|
| Conical roof | 1/8 thick | | 489.6 | | | 423.96 | 4.4 | | 2165. |
| Door ang. | 1 1/2 x 1 1/2 x 3/16 | 1.80 | | 2' - 11" | 2 | | | 5.25 | 10. |
| Door ang. | " " " | 1.80 | | 2' - 5" | 1 | | | 4.35 | 4. |
| Roof ang. | 2x1 3/8 x 1/4 | 2.50 | | 12' - 7 1/8" | 5 | | | 31.5 | 157. |
| Splice " | 1 3/4 x 1 1/8 x 3/16 | 1.60 | | 8" | 5 | | | 1. | 5. |
| Rivets | 5/16 | 4#/100 | | | 750 | | | | 30. |
| Roof bolts | 5/8 x 1 3/4 | 17#/100 | | | 100 | | | | 17. |
| Finial | | | | | | | | | 50. |
| Plates | 7 1/2 x 1/8 x 10 | 3.18 | | 10" | 4 | | | 2.65 | 11. |
| Sheaves | | | | | 2 | | | 5. | 10. |
| Ladder bars | 2 1/2 x 3/8 | 3.18 | | 25.' | | | | 80. | 80. |
| " rungs | 3/4 rd. | 1.502 | | 1' - 2 1/4 | 12. | | | 1.78 | 21. |

## WEIGHT OF BALCONY

| Name of part | Make up in inch. | Wgt./ft. # | Length | No. of member | Wgt. #/ mem. | Total wgt. # |
|---|---|---|---|---|---|---|
| Angles | 3x5x3/8 | 9.8 | 1' 11" | 8 | 18.7 | 150 |
| Plates | 10 1/2x 1/4 | 8.925 | 1 - 5" | 4 | 12.7 | 51 |
| Angles | 2x2 1/2x 1/4 | 3.62 | 1' - 5" | 8 | 5.1 | 41 |
| Angles | 5 x 3 x 3/8 | 9.8 | 0' - 10 1/2" | 4 | 8.5 | 34 |
| Cir. gird. | 24 x 1/4 | 20.4 | 56' - 6" | 1 | 1153 | 1153 |
| Chan. | 4 | 6.25 | 62.8" | 1 | 393 | 393 |
| Posts of railing | 1 1/2x 1 1/2 x 1/4 | 3.3 | 3' - 0" | 16 | 9.9 | 158 |
| Hand rail | " " " | 3.3 | 62.8" | 1 | 207 | 207 |
| Diag. bars | 1 1/2 x 1/4 | 1.275 | 5' | 32 | 64 | 205 |
| | | | | | Total | 2392# |

## Entire wgt. of metal in tower

| | | |
|---|---|---|
| Tankroof, finial & Indicator | 2741# | 4.7% |
| Tank Cylinder | 6900 | 11.9% |
| Hemisphemical bottom including | | |
| expansion connection | 3312 | 5.7% |
| 50% of wgt.of piping and casing | 3730 | 6.4% |
| Balcony | 2392 | 4.1% |
| Four shoes | 672 | 1.2% |
| Tower puers to Balcony | 38271 | 66.0% |
| | 58, 018# | 100% |

## Wgt. of water when full

Volume of cylinder plus hemisphere =

$3.14R^2 H + 2x3.14 R^3 /3 = 3.14 x 64x 14.83 + 2x3.14x512 = 4055$ CU.ft.

Volume of columnof water supported by pipe = 23x3.14 x0.11=8Cu.ft.

Total volume of water = 4055-8=4047 cu.ft.

wgt. volume one cu. ft, water = 62.5#

Total wgt. of water =4047 x 62.5 = 252,500#

Wgt. of metal in % of wgt. of water = 23%

WIND PRESSURE

Wind blowing diagonally on tower .

Wind calculated as 20# per sq.ft.for flat projected areas, and 2/3 of that amount for the curved projected areas.

| | |
|---|---|
| Roof= 0.5x20x7.5 = 75 sq. ft.= | 1500 |
| Tank =13.8 x 16= 221 sq. ft. | 4420 |
| Hemispherical bottom=0.5x3.14x64x2/3=67.3sq.ft= | 2010 |

BALCONY

| | |
|---|---|
| Angle posts 8x0.125 x3 = | 3.0sq.ft |
| Handrail 31.4x 0.125x2/3 | 2.7sq.ft. |
| Diag.bars 8x5x0.125= | 5.0 " " |
| Circ.Channel31.4x0.33x2/3 = | 7.0 " " |
| Brackets 0.416 x 2x8 = | 4.7 " " |
| | Total 22.4 " " = 672 |

PANEL 1

| | |
|---|---|
| Posts 0.882 x12x4 = | 42.4 sq. ft. |
| Tower rods .052 x 9.46 x8= | 4.0 " " |
| Pipe rods 2x.052 x 9.25 = | 1.0 " " |
| Chan Struts 0.416 x 8 x13.20 x 0.707 | 31.3 " " |
| Inlet Pipe 2x 4 x2/3 = | 5.3 " " |
| | 84.0 " " = 1680 |

## PANEL # 2

| | | |
|---|---|---|
| Posts 4x0.882x20.72 | = | 73.0 Sq.ft. |
| Tower Rods 8x0.625x24.9 | = | 12.5 " " |
| Pipe Rodso.052x2x11.62 | 6 | 1.2 " " |
| Chan.Struts 8x0.416x16.62x.707 | = | 39.2 " " |
| Ind,Guide 26x 00.5 | = | 13.0 " " |
| Inlet Pipe 20.58.x2x2/3 | = | 27.5 " " |
| | | 166.4 " " = 4992# |

## PANEL #3

| | | |
|---|---|---|
| Posts 0.882 x 4x 0.72 | = | 73.0 sq. ft. |
| Tower rods 8x073 x22.19 | = | 13.0 " " |
| Pipe rods2x14 xX .052 | = | 1.5 |
| Struts 8x0.416 x19.95 | = | 47.0 |
| Inlet Pipe 2x20.58 x 2/3 | = | 27.5 " " |
| | | 162.0 " " = 4860# |

## PANEL #4

| | | |
|---|---|---|
| Posts 4 x0.882 x20.72 | = | 73.2 sq. ft. |
| Strut rods 4x.062 x21.19 | = | 5.3 " " |
| Tower rods 8x23.8 x.062 | = | 13.8 |
| Pipe rods2x.052x16.37 | = | 1.7 |
| Struts 8x0.416 x 23.29x.707 | = | 54.7 |
| Inlet Pipe 2x20.58 x 2/3 | = | 27.5 " " |
| | | 176.2 " " = 5280# |

PANEL # 5

| | | |
|---|---|---|
| Posts 4x0.882 x20.72 | = | 73.2 sq ft. |
| Strut rods4x.062x21x19 | = | 5.3 |
| Tower rods8x.073x23.58 | = | 13.8 |
| Pipe rods 2x.052x18.73 | = | 1.9 |
| Struts 8x0.416 x.707x26.62 | = | 63.0 |
| Inlet Pipe 2x20.58 x2/3 | = | 27.5 " " |
| | | 184.7 " " = 5540# |

PANEL # 6

| | | |
|---|---|---|
| Posts4 x 0.882 x 20.72 | = | 73.2 sq. ft. |
| Strut rods 4 x .062x21.19 | = | 5.3 |
| Tower rods 8x.073 x 25.2 | = | 14.7 |
| Pipe rods 2x.062 x 21.0[illegible] | = | 2.2 |
| Struts 8x.416x29.[illegible]5 x .707 | = | 71.0 |
| Inlet Pipe [illegible].0x20.58 x 2/3 | = | 27.5 " " |
| | | 193.9 " " = 5820# |

PANEL # 7

| | | |
|---|---|---|
| Posts 4x.882x20.72 | = | 73.2 sq. ft. |
| Strut rods 4x.062 x21.19 | = | 5.3 |
| Tower rods 8x08x25.29 | = | 16.9 |
| Pipe rods 2x.052 x23.44 | = | 2.5 |
| Struts 8x.416 x33.29 | = | 78.5 |
| Inlet Pipe 2x20.58 x2/3 | = | 27.5 " " |
| | | 203.9 " " = 6120# |

PANEL # 8

| | | |
|---|---|---|
| Posts 4x.882 x20.9 | = | 74.2 sq. ft. |
| Tower rods 8x.08 x27 | = | 1[illegible].0 |
| Inlet Pipe 2x20.81 x 2/3 | = | 27.7 " " |
| | | 119.9 " " = 3600# |

WIND BLOWING SQUARELY ON TOWER

Roof, tank, hemisphere bottom and balcony same as for wind diagonal.

PANEL # 1

| | | |
|---|---|---|
| Posts 4x .882 x12 | = | 42.4 sq. ft. |
| Tower rods 8x.052 x9.44 | = | 4.0 |
| Pipe rods 4x .052 x9.29 x.707 | = | 2.8 |
| Struts 4x.416 x13.28 | = | 22.1 |
| Inlet pipe 2x4x2/3 | = | 5.3 " " |
| | | 76.6 " " = 2300# |

PANEL #2

| | | |
|---|---|---|
| Posts 4x.882 x20.72 | = | 73.0 sq.ft. |
| Tower rods 8x.062 x 24.95 | = | 12.5 |
| Pipe rods 4x.052 x 11.64x707 | = | 1.7 |
| Struts 4x.416 x16.64 | = | 27.7 |
| Ind. Guide.5x26 x.707 | = | 9.2 |
| Inlet Pipe 2 x20.58x2/3 | = | 27.5 " " |
| | | 151.6 " " = 4550# |

6 PANEL # 3

| | | |
|---|---|---|
| Posts 4x.882 x20.72 | = | 73.0 sq. ft. |
| Tower rods 8x.073 x22.19 | = | 13.0 |
| Pipe rods 4x.053 x14x.707 | = | 2.1 |
| Struts 4x.416 x19.95 | = | 33.3 |
| Inlet Pipe 2x 20.58 x2/3 | = | 27.5 " " |
| | | 148.9 " " = 4470# |

## PANEL #4

| | | |
|---|---|---|
| Posts 4x.882 x20.72 | = | 73.2 sq. ft. |
| Strut rods 4x.062 x21.19 | = | 5.3 |
| Tower " 8x.073 x23.58 | = | 13.8 |
| Pipe rods 4x.052 x16.37x.707 | = | 2.4 |
| Struts 4x.416 x23.29 | = | 38.7 |
| Inlet Pipe 2x20.72 x2/3 | = | 27.5 " " |
| | | 160.9 " " = 4827 # |

## PANEL #5

| | | |
|---|---|---|
| Posts 4x.882 x20.72 | = | 73.2 sq. ft |
| strut rods 4x062 x21.19 | = | 5.3 |
| Tower rods 8x.073 x23.58 | = | 13.8 |
| Pipe rods4x.052 x16.73 x.707 | = | 2.8 |
| Struts 4x.416 x 26.62 | = | 44.3 |
| Inlet pipe 2x20.58 x 2/3 | = | 27.5 " " |
| | | 166.9 " " = 5010# |

## PANEL #6

| | | |
|---|---|---|
| Posts 4x.882x20.72 | = | 73.2 sq. ft. |
| Strut rods 4x.062 x21.19 | = | 5.3 " |
| Tower rods 8x.073 x25.29 | = | 14.7 |
| Pipe rods 4x.052 x21.0[illegible]x707 | = | 3.1 |
| Struts 4x416 x29.95 | = | 50.0 |
| Inlet pipe 2x20.58 x2/3 | = | 27.5 " " |
| | | 173.8 " " = 5220# |

## PANEL #7

| | | |
|---|---|---|
| Ports 4x.882 x20.7[illegible] | = | 73.2 sq. ft. |
| Strut rds. 4x.062 x21.39 | = | 5.3 |
| Tower rds. 8x.08x25.29 | = | 16.9 |
| Pipe rds 4x.05[illegible] x23.44x.707 | = | 3.5 |
| Struts 4x.416 x33.29 | = | 5[illegible].4 |
| Inlet Pipe 2x20.72 x2/3 | = | 27.5 " " |
| | | 181.8 " " = 5450# |

## PANEL #1

Same as for wind diagonal.

Max Stress in Plate of Spherical bottom.

$T = 2.6 \times h \times r / t$ Ref. Ketchum's S.H.B.pg.367

T = radial stress per sq. in.

h = head of water in ft.

r = radius of tank in ft.

t = thickness of plate in inches

$T = 2.6 \times 23 \times 8 / 0.1875 = 2550$ # per sq. ft.

Test of rivets in radial joint.

a = pitch of rivets = 1.52 in.

d = diam. of rivet hole = 17/32 in.

D = diam " " " = ½"

S* = tensile stress in plate at joint per square inch

$S_s$ = unit shear on rivet

stress per lineal inch of plate = 2550# x 0.1875 = 478#

P = load on joint per pitch = 1.52 x 478 = 727#

S* = P H x(a-d) = 727/(1.52-0.53) x0.1875 = 3920

lbs per sq. in. of net plate.

Bearing on plate from rivets of radial joint

$S_c$ = unit compressive stress

$S_c$ = P/* D = 727/0.1875x 0.5 = 7750 lbs per sq. in.

Shearing stress on rivets of radial joint .

$S_s$ = unit shearing stress

$S_s = 4 P/3.14 D^2 = 4 \times 727/3.14 \times 0.25 = 3700$ lbs per sq. in.

## Rivet and plate stresses in circumferential joint of of spherical bottom

T = 2.6 h r/t = 2.6 x 22.41 x8/0.1875 = 2490 lbs.per sq. in. of plate.

Stress per lin. inch of plate = 2490 x 0.1875 = 467 lbs.

Pitch = 1.5 inches.

P = load per pitch = 1.5 x467 = 700 lbs.

D = 0.5 inch

d = 17/32 inch

$S_t$ = P/t(a-d) = 700/0.1875 (1.5 -0.53) = 3840lbs./sq.in.

$S_s$ = 4 P/ 3.14 x $D^2$ = 4x 700/3.14 x 0.25 = 3560 lbs" "

$S_c$ = P/t D = 700/0.1875 x 0.5 = 7450 lbs.per sq. in.

## Riveted connection between cylindrical tank and hemispherical bottom.

| | |
|---|---|
| Wgt. of water | 252,500# |
| wgt. of hemispherical bottom | 3 312 |
| " " 50% of piping | 3 730 |
| Total wgt. | 259,542# |

Load per inch of circumference = 259, 542 /603 = 430 lbs.

Load per pitch = 1.572" x 430 = 676#

d = 17/32 inch, D = 0.5 inch.

$S_t$ = P/t (a-d) = 676/0.1875 (1.572-0.53) = 3440#/sq.in.

$S_s$ = 4 P/3.14 $D^2$ = 4x 676/3.14 x 0.25 = 3450#/sq. in.

$S_c$ = P/t D = 676/0.1875 x0.5 = 7210#/sq. in.

VERTICAL JOINT CYLINDRICAL TANK

T = 2.6 h r/ t = 2.6 x 15 x8 /0.1875 = 1664 #/sq. in.

Stress per lin. inch = 1664 x 0.1875 = 312 #

P = 312 x 1.487 = 463 #

This is a smaller load on the same size rivet and a smaller pitch than for the previously calculated joints as the stresses will be smaller.

VERTICAL JOINT IN THE CENTRAL SECTION OF THE CYLINDER TANK

d = 13/32 inch

D = 3/8 "

T = 2.6 h r /t = 2.6 x 10x8/0.1875 = 1110 #/sq. in.

1110 x0.1875 = 208# /lin. inch of joint .

a = 1.25 inches

P = 1.25 x 208 = 260#

$S_t$ = P/t 9a-d) = 260/0,1875 (1.25-0.41) = 1650#/sq. in.

$S_s$ = 4P/3.14 $D^2$ = 4x260/3.14 x0.141 = 2350#/sq. in.

$S_c$ = P/t D = 260/0.1875 x 0.375 = 3700# /sq. in.

HORIZONTAL JOINT TANK CYLINDER

D = 3/8 inch, d = 13/32, a = 1.351 inches

| | |
|---|---|
| Wgt. of roof | = 2741# |
| Wgt. of 2/3 tank cylinder | = 4600 |
| Total | 7341# |

Circumference of tank is 604 inches

7341/604 = 12.2# per lin. inch.

P = 1.351 x 12.2 = 16.5#

As the load is exceedingly small the stress will be correspondingly small.

## TYPES AND EFFICIENCIES OF JOINTS IN TANK AND BOTTOM.

Radial joint in hemispherical bottom.

It is a single riveted lap joint.

Eff. of plate in tension = (a-d)/a = (1.52-0.53)/1.52

= 65%

Eff. of rivet in bearing = $2D\,S_c$ /a $S_t$ =

2 x 0.5 x 18000/1.52 x 12000 = 98.7%

Eff. of rivet in shear = $3.14\,D^2\,S_s$ / 2xa * $S_t$ =

3.14 x 0.25 x9000 /2 x1.52 x0.1875 x12000 = 104%

This joint is 65% efficient.

## CIRCUMFERENTIAL JOINT IN HEMISPHERICAL BOTTOM.

It is a single riveted lap joint..

Eff. of plate in tension = (a-d) /a = (1.5 -0.53)/1.5

= 64.6%

Eff. in shear and bearing same as for radial joint.

The efficiency of this joint is 64.6%

## Circumferential joint between cylinder tank and hemispherical bottom.

It is a single riveted lap joint.

Eff. of plate in tension (a-d)/a = (1.572 -0.53)/1.572

= 66.3%

Effs. in shear and bearing same as for radial joint.

The efficiency of the joint is 66.3%

VERTICAL JOINT IN CYLINDER TANK

It is a single riveted lap joint

Eff. of plate in tension = (a-d)/a = (1.487- 0.53)/1.487 = 64.3%

The effs. in shearing and bearing same as for radial joint.

The eff. of the joint is 64.3%.

Calculation for the efficiency of the tank plate in tension, at the riveted connection of the column to the tank plate.

Total area of plate at connection = 14 x 21 = 294 sq. in.

Area of 64 -½ inch rivets = 0.22 x 64 = 14 sq. in.

Efficiency of plate = 280/294 = 95 %

## AXIAL COLUMN STRESS DUE TO NO BRACING DIAGONALLY ON THE TOWER

S = M /2 r  Ketchum's S H B pg. 370

Stress in column $C_8$

Taking moments about the bottom of panel #8

M = 172.6 x 1500 + 163.2 x 5050 + 152.8 x2020

+ 150.3 x 2520 + 134 x4992 + 113.4 x4860

+ 92.9 x 5280 + 72.3 x5540 + 51.7 x5820

+ 31.1 x 6120 + 10.5 x3360 = 4,411, 400 ft.lbs.

r = 25.92 ft.

sec a = 1.01 where a = angle plane of bent makes with the vertical

S = sec axM/2r

S = 1.01 x 4, 411, 400/2 x25.92 = -85. 800#

THE STRESS IN COLUMN $C_5$

M = 110.6 x1500 + 101.2 x5050 + 90.8 x2020

+ 81.3 x2520 + 72 x4992 + 51.5 X 4860

+ 30.9 x5280 + 10.3 x5540 = 1,914,700 ft.lbs.

r = 18.83 ft.

S = sec.a xM /2 r = 1.01 x 1, 914, 700 /2 x18.83= -51,300#

THE STRESS IN COLUMN $C_6$

$$M = 131.2 \times 1500 + 121.8 \times 5050 + 111.4 \times 2020$$

$$+ 108.9 \times 5[illegible]0 + 92.6 \times 499[illegible] + 72 \times 4860$$

$$+ 51.5 \times 5[illegible]0 + 30.9 \times 5540 + 10.4 \times 5820$$

$$= 2,626,000 \text{ ft. lbs.}$$

$$r = 21.18 \text{ ft.}$$

$$M = \sec a\ P/2\ r = 1.01 \times 2,626,000/2 \times 21.18$$

$$M = -62,600 \ \#$$

MAX. RESULTANT AXIAL COLUMN STRESSES

| Member | Dead empty | Tank filling | Wind Diagonal | Max. comp.total. |
|---|---|---|---|---|
| $C$ | 5180 | 6[illegible]000 | | |
| $C_2$ | 6420 | 70000 | | |
| $C_3$ | 7440 | 71400 | | |
| $C_4$ | 8[illegible]00 | 72750 | | |
| $C_5$ | 10000 | 74000 | 51,300 | 135,300 |
| $C_6$ | 11310 | 75250 | 62,600 | 137,850 |
| $C_7$ | 12620 | 76700 | | |
| $C_8$ | 13870 | 77750 | 85 800 | 163 550 |

| Member | Dead empty | Wind diagonal | Max.uplift total at footing |
|---|---|---|---|
| $C_8$ | 13,870 | 85 800 | 71,930 |

## -TABULATION OF STRESSES IN BRACING

| Member | Water & dead load | Wind load square | Initial tension | Max . Comp. | Max tension. |
|---|---|---|---|---|---|
| $B_1$ | | * 8600 | * 3000 | | * 11,600 |
| $S_1$ | -141 | - 5260 | | - 5400 | |
| $D_1$ | | * 8100 | * 3000 | | * 11,100 |
| $S_2$ | -141 | - 6470 | | - 6610 | |
| $D_2$ | | * 14350 | * 3000 | | * 17350 |
| $S_3$ | -141 | - 2420 | | - 2560 | |
| $D_3$ | | * 15700 | * 3000 | | * 18700 |
| $S_4$ | -141 | - 9000 | | - 9140 | |
| $D_4$ | | * 16200 | * 3000 | | * 19200 |
| $S_5$ | -141 | - 2530 | | - 2670 | |
| $D_5$ | | * 17600 | * 3000 | | * 20,600 |
| $S_6$ | -141 | - 11920 | | - 12060 | |
| $D_6$ | | * 18400 | * 3000 | | * 21,400 |
| $S_7$ | -141 | - 2320 | | -/ 2460 | |
| $D_7$ | | * 19850 | * 3000 | | * 22,850 |
| $T_1$ $T_2$ $T_3$ $T_4$ | | | * 3000 | | * 3000 |

## MAX. UNIT STRESSES IN BODY OF COLUMNS.

### UNIT STRESS IN COLUMN $C_5$

Make up is one 10 in.chan. @ 15# & one 8 in. chan. @ 11¼ #

Area 10" Chan. @ 15# = 4.46 sq. in/

" 8" " " 11¼# = 3.35 " "

Totalarea7.81 " "

Total compressive stress in column = 125, 300 #

Unit stress = 125, 300/7.81 = 16030 lbs. per sq. in.

Allowable stress = 16000-70 L/R

Considering the (y) axis coincident with the outer edge of the flange of the 10" chan., and the (x) axis coincident with the outer edge of the extreme flange of the 8" chan.

Bar (y) = 4.46 x 8.639 + 3.35 x 4/7.81 = 6.65 in.

Bar (x) = 4.46 x5 + 3.35 x4.201 /7.81 = 4.66in.

Moment of inertia about (x) axis thru c.of g. of column.

$I = 2.30 + 1.989^2 \times 4.46 + 32.3 + 2.65^2 \times 3.35$

$I = 75.7$ inches$^4$

Moment of inertia about (y) axis thru c.of g. of column.

$I = 66.9 + 4.46 \times 0.34^2 + 1.33 + 3.35 \times 0.449^2$

$I = 69.42$ Inches$^4$

R = square root of I/A

R = square root 69.42/7.81 = 2.98 Inches.

Allowable stress = 16000-70 x 20.72 /2.98

" " = 10160 lbs.per sq. in.

L/R = 83.5

## UNIT STRESS ON COLUMN C6

Make up is one 10" Channel @ 15# and one 8" channel at 13 3/4#

Area 10" Chan. = 4.46 sq. in/

" 8" " = 4.04 " "

Total area 8.50 " "

Total compressive stress in column = 137,850#

Unit stress = 137,850 #/8.50 = 16,200 lbs. per sq. in.

Bar (y) = 4.46 x 8.639 + 4.04x 4/8;50 = 6.44 inches

Bar (x) = 4.46 x 5 + 4.04 x 4.182/8.50 = 4.62 inches.

Moment of inertia about (y) axis thru c.of g. of column.

I = 66.9 + 4.46 x 0.38² + 1.55 + 4.04 x 0.438² = 69.88 in.⁴

Moment of inertia about (x) axis thru c.of g. of column.

I = 2.30 + 4.46 x 1.99² + 36.0 + 4.04 x 2.44² = 80.23 in.⁴

R = square root of 69.88/8.50 = 2.87 inches

L/R = 20.72 x 12/2.87 = 86.7

Allowable unit stress = 16000 - 70 x 86.7 = 9930 lbs/sq. in.

## UNIT STRESS IN COLUMN C8

Make up is one 10" Chan. @ 20 # and one 8" Chan 13 3/4#

Area 10" Chan. = 5.88 sq. in.

" 8" " = 4.04 " "

Total area 9.92 " "

Total compressive stress in column = 163, 550 #

Unit stres = 163, 550 /9.92 = 16. 500 lbs per sq. in.

Bar (y) = 5.88 x 8.609 + 4.04 x 4/9.92 = 6.73 inches

Bar (x) = 5.88 x 5 + 4.04 x 4.182/ 9.92 = 4.67 inches.

Momentof inertia about (x) axis thru c. of g. of column

I = 2.85 + 5.88 x $1.88^2$ + 36.0 + 4.04 x $2.73^2$ = 89.75 $in^4$.

Moment of inertia about (y) axis thru c. of g. of column

I = 78.7 + 5.88 x $0.33^2$ + 1.55 + 4.04 x $0.488^2$ = 81.85 in 4.

R = square root of 81.85/9.92 = 2.87 inches

L/R = 20.98 x 12/ 2.87 = 87.5

Allowable unit stress = 16000-70 x 87.5

" " " = 9880 lbs per sq in.

Determination of compressive stress in tank plate considered as a simple beam between column connections; to note if this stress exceeds the stress due to the outward pressure of water.

Length of beam $= 1/4 \times 3.14 \times 16 = 12.56$ ft.

Depth " " $= 15$ ft.

Thickness " $= 3/16$ inch.

Wgt. of water in tank = 252, 500 lbs.

" " Hemispherical Bot. 3 312 "

" " 50% of pipe = 3 730 "

Total 259,542 "

Wgt. falling on one span = 259,542/4 = 64,885 lbs.

Uniform load per ft. = 64,885/12.56 = 5165 lbs.

Considered as a sample beam fixed at both ends.

Max. moment occurs at ends $= w\, l^2/12$

$M = 5165 \times 12.56^2/12 = 68{,}000$ ft. lbs.

$M = SI/c$

$I/c = b\, d^2/6 = .1875 \times 32{,}400/6 = 1012$

$M = 68{,}000 \times 12 = S \times 1012$

$S = 806.$ lbs. per sq. in.

Stress outward due to water

$T = 2.6\, h\, r/t = 2.6 \times 15 \times 8/.1875 = 1{,}670$ lbs /sq. in.

These results indicate that this is about the largest size tank that can be safely built with only four points of support .

Determination of the shear on the heads of the rivets at the connections of columns to tank due to bending moment produced by wind.

M = 9.4 x 2450 x 12 = 276, 500 in. lbs.

Taken from graphical solution.

276,500 x .707 = 195, 300 in. lbs is the moment perpendicular to the connection.

d = lever arm of resisting force

d = 7 in.

V = avge, shear on one rivet head

There are 32, ½ inch rivets resisting the force

M = 32 V d

195,300 = 32 V x 7

V = 872 lbs.

Max. shear on one rivet head = 2 x 872 = 1744 lbs.

Shearing area of one ½" rivet = 3.14 x .5 x .281 = .4425 sq.in.

Allowable shear one one rivet = .4425 x 9000 = 3980 lbs.

## DETERMINATION OF SHEAR ON CROSS SECTION OF RIVETS

## AT THE COLUMN CONNECTION TO TANK

| | | | |
|---|---|---|---|
| Wgt. water | | | 252,500 lbs. |
| " | of | cylinder | 6,900 " |
| " | " | Hemispherical bottom | 3 312 " |
| " | " | Piping | 3 730 " |
| " | " | roof, etc. | 2 741 " |
| | | Total | 269, 183" |

Wgt. on one connection = 269, 183/4 = 67, 296 lbs.

No. of ½" rivets taking shear = 64

Shear on one rivet = 67,296/64 = 1052 lbs.

Area of ½" rivet = .196 sq. in.

Allowable value on one rivet = .196 x 9000 = 1765 #

Test for bearing value of ½ in rivets on 3/16 inch plate.

Bearing area of one rivet = .1875 x .5 = .0938 sq. in/

Total bearing area = 64 x .0938 = 6.0 sq. in.

Unit bearing value = 67, 296/6.0 = 11,200 lbs/sq. in.

Allowable value = 18000 lbs per sq. in.

STRESSES IN COLUMN C DUE TO ECCENTRICITY

Diagonal wind , and dead load plus water.

STRESS DUE TO ECCENTRICITY

Eccentricity = the distance between the extended c.of g. of the column proper, and the c. of g. of the column section at the connection to the tank.

Ecc.= e = 2.72 inches

Moment due to eccentricity = total load on one column x e = 68261 x 2.72 = 185, 600 in. lbs.

Make up of section at connection

One 10" Chan, @ 15#

One 8" x 3/8" plate on outside of channel.

One 6" x 3½" x 3/8 ang. on each side of Chan,

(x) axis coincident with the back of the channel

Bar (y) = 4.46 x .639 + .437 x 3 + 6.84 x 2.04 / 14.3 = 1.267 in.

Moment of inertia about (x) axis thru the c. of g. of section.

FOR ANGLES

$I = 2(12.86 + 3.42 \times .773^2) = 29.82$

FOR PLATE

$I = b\ d3/12 = 8 \times .053/12 + 3 \times .83^2 = 2.11$

FOR CHANNEL

$I = 2.30 + 4.46 \times .628^2 = 4.06$

Total $I = 4.06 + 2.11 + 29.82 = 35.99$

$M = S\ I\ /C$

$185,600 = S \times 35.99/4.73$

S = 24,400 lbs per sq. in.

STRESS DUE TO DIAGONAL WIND

Moment of wind loads about the bottom of Panel #1

M = 1500 x 28.3 + 5050 x 18.9 + 2020 x 8.5 + 2520 x 6

M = 170,270 ft. lbs.

r = 9.39 ft.

S = sec ax M/2 r = 1.01 x 170,270 /18.78 = 9,170lbs.

Unit stress = 9,170/14.3 = 642 lbs per sq. in.

STRESS DUE TO DEAD LOAD PLUS WATER

Total stress = 69,000lbs.

Unit " = 69,000/14.3 = 4820 lbs per sq, in.

MAX. RESULTANT COMPRESSIVE STRESS IN COLUMN C.

| | | | | | |
|---|---|---|---|---|---|
| Eccentricity = | 24, 400 | lbs | per | sq. | in. |
| Diagonal Wind = | 642 | " | " | " | " |
| Dead plus water= | 4, 820 | " | " | " | " |
| Total | 29, 862 | " | " | " | " |

Allowable stress = 16000-70L/R

" " = 16000-70 x 1 x12/1.585

" " = 15,400 lbs per sq. in.

STRESSES IN SHOE DETAILS

Total compression on bearing plate = 506 lbs per sq. in.

Test for shear on cross-section of rivets connecting side angles to columns.

Area of plate covered by one angle = 3" x 20" = 60 sq. in.

Shearing force = 60 x 506 = 30,400#

This force taken by two 3/4" rivets and two 5/8" rivets

Two 3/4" rivets @ 9000 are worth 2(.442x9000) = 7950#

Two 5/8" " " " " " 29.307 x9000) 5530#

Total amount rivets can take =13,480 #

This is considerably less than 30,400#

Test fro shear on cross-section of four 3/4" rivets at back of column.

Total force taken by these four rivets = 30,4000"30x506 = 45,600#

Value of four 3/4 " rivets in single shear =4(.442 x9000) = 15,900#

One 3/4" rivet on 1/4" web of chan. @ 18000 is worth 3380#

Total value = 4 x 3380 = 13,500# hence bearing governs; but is much less than amount taken by rivets.

Test to see if angles on the side of the columns are thick enough to take the shear.

The angles are 3/8" thick and 20" long

20 x .375 = 7.5 sq. in.

Allowable shearing value = 10,000# persq. in.

One angle is worth 10,000 x7.5 = 75,000#

It has to take only 30,400#

Test on the ten 5/8" rivets on front of column in single shear.

Force coming to the 10 rivets = 30,400 * 87.5 x566= 74,800#

One 5/8" rivet in single shear @ 9000 is worth 2,756 #

Ten 5/8" rivets " " " " " " " 27,650#

which is considerably less than 74,800#

Max. uplift of 71,930# is taken by thirteen 5/8" rivets in single shear, which are worth only 35,950 #

Test of shear on rivets holding the diagonal rod $D_7$

It is held by four 5/8" rivets

Stress in rod is 19,850#

Total stress on section of rivets =19,850x.707=14,030#

Allowable stress =4x2765 = 11,000#

Total shear on heads of rivets = 19,850x.707=14,030#

Shear on cross section governs as this is weaker than the head in shear.

## CALCULATION OF UNIT STRESSES IN STRUTS

Made up is two 5" cahnnels @ 6.5 #

Same section used thruout the tower

Total max. stress = 12,060 # compression area = 3.90 sq. in.

Unit stress = 12, 060 /3.9 = 3,100 lbs sq. in.

Area = 3.90 sq. in.

Allowable stress = 16000 -70 L/R

(x) axis considered as coincident with back of inside chan.

Bar (y) = 1.95 x 4.89 • 1.95 x 7.9[illegible]/3.9 = 4.24 inches.

Mom. of inertia about (x) thru c.of g.

I = .48 • 1.95 x $3.75^2$ • .88 • 1.95 x $3.75^2$ = 55.-96

Mom. of inertia about (y) axis thru c. of g.

I = 7.4 • 7.4 = 14.8

R = sq. root 14. 8/3.9 = 1.95"

L/R = 14.98 x 12 / 1.95 = 92.2

Allowed stress = 16000 -70 x92.2 = 9,540# per sq.in.

## CALCULATION OF UNIT STRESSES IN BRACING

| Member | Make up | Area | Total stress | Unit stress |
|---|---|---|---|---|
| R1 | 5/8 sq. | | | |
| $D_1$ | 3/4 sq. | .5625 | #11,100 | #19730 |
| $D_2$ | 7/8 sq. | .765 | #17,350 | #22700 |
| $D_3$ | 7/8 sq. | .765 | #18700 | #24450 |
| $D_4$ | 7/8 sq. | .765 | #19200 | #25100 |
| $D_5$ | 7/8 sq. | .765 | #20600 | #26900 |
| $D_6$ | 1 sq. | 1. | #21400 | #21400 |
| $D_7$ | 1 sq. | 1. | #22850 | #22850 |
| $T_1$ T2 T3 $T_4$ | 3/4 rd. | .442 | #3000 | #6780 |

## CALCULATION OF UNIT STRESSES IN BRACING

### LACING OF HORIZONTAL STRUT

P = stress in lacing bar = 280 A r csc.a/c

A = area of strut

r = least rad. of gyr.

C = dist, nuetral axis to most remote fibre

a = angle made by bar with axis of strut csc.a =1.167

P = 280 x 3.9 x 3.78 x1.17 /5.01 = 963 #

Lacing bar is 1 3/4" x 1/4 " = .4375 sq. in.

Unit stress = 963/.4375 = 2200# sq. in.

Test for shear on one rivet holding lacing bar.

cos.a = .517

S = shear on s ection of rivet

S = 2 x P x.517 = 2x 963 x .517 = 997#

one 5/8 " rivet in single shear is worth 2765#

### UNIT STRESSES IN STRUT CONNECTIONS TO COLUMNS

Same design used in each of the connections .

$S_6$ gets the largest compressive stress of 12,060 #

5/8 in rivets on 5/16 in plate, shear governs.

Eight 5/8 in rivets take theestress in single shear.

Value of one 5/8" rivet in S.S. = 2765 #

Connection is worth 8 x 2765# = 22,100#

These plates are bent and are riveted to the flanges of the column channel.

Twelve 5/8 " rivets take the stress in C. S.

Stress on rivets = 12060 x .707 = 8540#

Value of the rivets = 12 x 27 65 = 33, 200#

STRESS ON PINS

Greatest stress produced in pin at the intersection of $S_7$ $D_6$ $T_4$ $D_7$

Stress in $D_6$ = 18400 # tens for one only

" " $D_7$ = 19850 # " " " "

Stress " $T_4$ = 3000 " " " "

Horiz. comp. of $D_6$ = 18400 x16,65/25.31 = + 12, 100 #

Vertical " " " = 18400x21.19/25.31 = + 15,400#

Horiz. " " $D_7$ = 19850 x 16.65/27 = -12250#

Vert. " " " = 19850 x 20.5/27 = -15060

Vert. comp.of $T_4$ = 3000#

Horiz. " " " = 0

Length of pin is 9 inches, & $T_4$ is at centre.

Horiz.componet of the left reaction.

R = 12, 250 x 6. 38 - 12, 100 x 5.38/9 = + 1456#

Horiz.Moment at centre line $T_4$

M = 1456 x 4.5 - 12,250 x 1.875 + 12, 100 x .875 = -5870 in lbs.

Horiz. Moment at centre line $D_6$

M = 1456 x 3.625 - 12,250 x 1 = - 6970 in lbs.

Horiz Moment at centre line $D_7$

M = 1456 x 2.625 = + 3830 in lbs.

Vertical component of left reaction.

R = 15060 x 6.38 - 15,400 x 5.38 -3000x4.5/9 = -44.5#

Vert. moment at centre line $T_4$

M = -44.5x4.5 - 15060 x 1.875 + 15400 x .875 = -14990#

Vert, moment at centre line $D_6$

M = 44.5 x 3.625 -15060 x 1 = 15221 in lbs.

Vert moment at centre line $D_7$

M = -44.5 x 2.625 = 117 in. lbs.

Max. resultant moment occurs at D6

M = square root of $6970^2 + 15\,221^2$ = 16 760 in lbs.

Diam. of pin = 2 in.

I/c = .784

M = SI/c = 16760 = S x .784

S = 21,400 # sq. in.

Allowable stress = 24,000# sq. in.

ANALYSIS OF FLOOR PLATE OF BALCONY

30# sq. ft. as uniform load.

Plate is 24" wide and 1/4 " thick

$M = w\ l^2/8$

Plate wghs. 20.4 lbs per lin ft.

$M = 80.4 \times 14.13^2 /8 = 2010$ ft lbs.

$I/c = 0.25$

$M = S\ I/c = 2010 \times 12 = S \times .25$

S = 9,660# sq. in.

Allowable stress = 16,000# sq. in.

ANALYSIS OF BRACKET CONNECTION TO COLUMNS

Force tending to shear rivets

1/4 wgt of balcony = 578#

Uniform loadon floor

plate = 848

total 1426

Lever arm of force = 10.5 in.

Bending moment = 10.5 x 1426 = 15000 in lbs.

Distance between gage lines of rivets = 2 1/4 in.

Resisted by four 5/8" rivets in single shear.

S = stress on one rivet

15000 = 2.25 x 4x S

S = 1667#

Value of 5/8" rivet in S. S. = 2765#

" " " " " bearing on 3/8" plate = 4220#

STRESS IN ANCHOR BOLTS.

Max. uplift total = 71930#

Vert. comp. in a anchor bolt = 71930 x 156,3/1.01 x 157.3 = 70,700#

Diam of anchor bolt = 2" at root of thread

Unit stress in anchor bolt = 70,700/3.14 = " 22,500#

## FACTOR OF SAFETY AGAINST UPLIFT .

Moments about axis of shoes.

Over turning moment due to wind.

| | | |
|---|---|---|
| Roof | (1500# x 172.61 ) = | 259,000 |
| Tank & Bal | (5092 # x163.21 ) = | 831,000 |
| Hem Bot. | (2020 # x 152.81) = | 309,000 |
| $P_1$ | (2300x 150.31 ) = | 346,000 |
| $P_2$ | (4550x 134.02 ) = | 610,000 |
| $P_3$ | (4470 x 113.44 ) = | 507,000 |
| $P_4$ | (4827 x 92.86 ) = | 448,000 |
| $P_5$ | (5010 x 72.28 ) = | 362,000 |
| $P_6$ | (5220 x 51.7 ) = | 270,000 |
| $P_7$ | (5450 x 31.12 ) = | 169,500 |
| $P_8$ | (3600 x 10.54 ) = | 38,000 |
| Total overturning moment | | 4,149,500 ft.lbs. |

Resisting moment - Tank empty.

Moments about axis of shoes.

| | |
|---|---|
| Entire wgt. of steel in tower. | 58,018# |
| subtract 50% of wgt. of pipe | 3,730 |
| Net Total = | 54,288# |

Lever arm of wgt. of steel = 1/2 square dustance of tower be tween anchor bolts = 18.5 ft.

54,288# x 18.5 = 1,005,000 ft. lbs.

wgt. of 2 piers = 2 x42,850# = 85,700#

lever arm = 37 ft.

85,700# x 37' = 3,170,000#

Calculation of wgt. of earth around pier which offers resistance to overturning.

Volume of masonry pier = 288.cu. ft.

Volume of solid equal to area of base of pier times height of pier, equals 10' x 10' x 5.25' = 525.cu. ft.

525 - 288 = 237 cu. ft. of earth.

wgt. of earth on 2 piers. 237 x 2 = 474 cu. ft.

wgt. of earth = 100# per cu. ft.

474 x 100 = 47,400

lever arm equals 37 ft.

47,400 x 37 = 1,755,000 ft. lbs.

Total resisting moment.

| | |
|---|---|
| Steel | 1,005,000 |
| Masonry piers | 3,170,000 |
| Earth | 1,755,000 |
| Total | 5,930,000 ft. lbs. |

Factor of safety against overturning = 5,930,000 ÷ 4,149,500 = 1.43

## SETTLEMENT DATA ON FOUNDATIONS TAKEN UNDER VARIOUS LOADING CONDITIONS

(a) With four panels of tower in place taken Feb,12, 1916.

(b) With all the steel in place April 7, 1916.

(c) With water on tank, June 2,1916.

| North West Pier | Elevations (a) | (b) | (c) |
|---|---|---|---|
| W corner | 103.107 | 103.101 | 103.106 |
| N " | 103.151 | 103.144 | 103.151 |
| E " | 103.142 | 103.139 | 103.141 |
| S " | 103.139 | 103.129 | 103.131 |
| **North East Pier** | | | |
| W Corner | 103.165 | 103.156 | 103.162 |
| N " | 103.111 | 103.097 | 103.104 |
| E " | 103.116 | 103.106 | 103.108 |
| S " | 103.138 | 103.118 | 103.128 |
| **South East Pier** | | | |
| W Corner | 103.122 | 103.115 | 103.109 |
| N " | 103.112 | 103.098 | 103.109 |
| E " | 103. 095 | 103.086 | 103.091 |
| S " | 103.071 | 103.066 | 103.067 |

| South West Pier | Elevations | | |
|---|---|---|---|
| | (a) | (b) | (c) |
| W Corner | 103.165 | 103.161 | 103.165 |
| N " | 103.173 | 103.156 | 103.171 |
| E " | 103.152 | 103.146 | 103.148 |
| S " | 103.169 | 103.161 | 103.165 |

Three pipes were driven in the ground near the North West pier

| Pipe | | | |
|---|---|---|---|
| West of pier | (a) | (b) | (c) |
| pipe 4 ft. long | 103.144 | 103.141 | 103.141 |
| East of Pier | | | |
| (1) 6 ft.from pier6ft long | 10 2.969 | 102.961 | 102.966 |
| (2) 9ft. " "7½" long | 103.010 | 103.006 | 103.011 |

The above data shows that the Pier s have not settled materially. The apparent rise in the elevations of the piers in the later survey is in all probability due to an error in the height of instrument.

THE EFFECT OF THE FUTURE BUILDING EXCAVATION ON THE SAFETY OF THE TOWER

It is a know fact that quick-sand is predominant in the sub-soil of the M A C campus. Therefore the most logical thing to have done would have been to set the piers on piles. If future excavation caused the quick sand to run, the failure of the tower could cause material damage to adjoining property.

CRITISIZM WITH RESPECT BIRCH NORDS SPECIFICATIONS.

Numbers refer to spec. articles as quoted in Ketchums SHB

10. For compression numbers, the permissible unit stress of 16,000 lb. shall be reduced by the formula:

$$p = 16,000 - 70 l/r,$$

where p - permissible working stress in compression, in lb. per sq. in. l = length of member, from center to center of connections, in inches; r = least radius of gyration of section, in inches. The ratio, l/r, shall never exceed 120 for main members and 180 for struts and roof construction members.

The allowable value of l ÷ r is not exceeded in any of the members. However the allowable unit compressive stresses are exceeded in all of the parts of the columns. The most extreme cases are an excess of 95% in colum $C_1$ and an excess of 67% in column C8.

11. Stresses due to wind may be neglected if they are less than 25 per cent of the combined dead and live loads.

All wind stresses are in excess of 25% of the combined dead and live load.

12. Unit stresses in bracing and other members taking wind stresses may be increased to 20,000 lb. per sq. in., except as shown in Section 11.

In the tension members of the bracing the allowable stress of 20,000 lbs. per sq. in. has been exceeded in nearly every case. The extreme case being an excess of 35%.

13. Portland cement concrete................ 350 lb. per sq. in.

The allowable unit pressures on the piers are exceeded 45%.

14. The plates forming the sides of cylindrical tanks shall be of different diameters, so that the courses shall lap over each other, inside and outside, alternately

This item has been very aptly complied with.

15. The joints for the horizontal seams, and for the radial seams in spherical bottoms, shall preferably be lap joints.

The lap joint has been used throughout this design.

16. For vertical seams double-riveted lapjoints shall be used for 1/4, 5/16, and 3/8in. plates. Triple lap joints shall be used for 7/16 and 1/2 in. plates; double-riveted butt joints shall be used for 9/16, 5/8, 11/16 and 3/4 in. plates; and triple-rivited butt joints for 13/16, 7/8, 15/16 and I in. plates.

Single riveted lap joints with 3/16 inch plates have been used, which is allowable with plates less then 1/4 inch.

17. Rivets 5/8 in. in diameter shall be used for 1/4 in. plates; rivets 3/4 in in diameter shall be used for 5/16" plates; rivets 7/8" in diameter shall be used for 3/8 to 7/8 in. plates, inclusive. Rivets 1" in diameter shall be used for 15/16 in. and 1 in. plates.

1/2" rivets have been used which is allowable with 3/16 inch plates. The efficiency falls slightly below the specified amount. The lowest value being about 65%.

18. In no case shall the spacing between rivets along the caulked edges of plates be more than ten times the thickness of the plates. All rivets shall be entered from the inside of the tank, and shall be driven from the outside, that is, new heads on rivets shall always be formed from the opposite side of the plate on which the caulking is done.

This requirement has been complied with.

20. The minimum thickness of the plates for the cylindrical part shall be 1/4in. The thickness of the plates in spherical bottoms shall never be less than that of the lower course in the cylindrical part of the tank.

The thickness of the plates in the cylindrical part of the tank are 3/16 of an inch thick which is thinner than the required amount

25. The radial sections of spherical bottoms shall be made in multiples of the number of columns supporting the tank, and shall be reinforced at the lower parts, where holes are made for piping.

There are 12 radial sections of the spherical bottom and this is a multiple of the 4 columns. The spherical bottom is reinforced with a circular head plate where the piping is connected.

26. When the center of the spherical bottom is above the point of connection with the cylindrical part of the tank, there shall be provided a girder at said point of connection to take the horizontal thrust. The horizontal girder may be made in

connection with a balcony. This also applies where the tank is supported by inclined columns.

The centre of the spherical bottom is at theline of connection of the spherical bottom to the cylindrical part, so there is no horizontal thrust.

27. The balcony around the tank shall be 3 ft wide, and shall have a floor-plate 1/4 in. thick, which shall be punched for drainage. The balcony shall be provided with a sutiable railing, 3 ft 6in. high.

This item has not been strictly adhered to . The balcony is 24 inched wide, made of 1/4 inch plates, and the railing is 2 ft 9-3/4" high.

28. The upper parts of spherical bottom plates shall always be connected on the inside of the cylindrical section of the tk tank.

The plates of the spherical bottom are connected to the inside of the plates of the cylindrical part.

29. In order to avoid eccentric loading on the tower columns, and local stresses in spherical bottoms, the connections between the columns and the sides of the tank shall be made in such a manner that the center of gravity of the column section intersects the center of connections between the spherical bottom and the sides of the tank. Enough rivets shall be provided above this intersection to transmit the total column h load.

This item has been somewhat overlooked in this design; and eccentric stresses of considerable value are produced. The riveted connection at this point is strong enough to transmit the load.

30. If the tank is supported on columns riveted directly to the sides, additional material shall be provided in the tank plates riveted directly to the columns to take the shear. The shear may be taken by providing thicker tank plates, or by reinforcement plates at the column connections, while bending moments shall be taken by upper and lower flange angles. Connections to columns shall be made in such a manner that the efficiency of the tank plates shall not be less that that of the vertical seams.

There is no additional, material riveted to the tank plates, however they are safe in bearing value. There is a plate riveted to the outside of the channel of the column to strengthen the web in bearing value. The efficiency of the tank plates at this connection is about 95% while the efficiency of a vertical joint is about 65%.

31. For high towers, the columns shall have a batter of 1 to 12. The height of the tower shall be the distance from the top of the masonry to the connection of the spherical bottom, or the flat bottom, with the cylindrical part of the tank.

The columns have a batter of 1-3/8 inches in 12 inches.

32. Near the top of the tank there shall be provided one Z-bar to act as a support for the painter's trolley, and for stiffening the tank. Its section moulus shall not be less than $D^2/250$. where D is the diameter of the tank in feet. If the upper part of the tank is thoroughly held by the roof constructions this may be reduced.

There is no Z bar to act as a support for the painter's trolley and for stiffening the tank there is a 3" x 2" x 1/4" angle at the top of the tank. Its section modulus is 0.26 in 3 for bending in a vertical direction which is considerably less than $\frac{D^2}{250} = 1.02$

33. On large tanks, circular stiffening angles shall be provided in order to prevent the plates from buckling during windstorms. The distance between the angles shall be determined by the formula : $d = 900\, t^{\frac{3}{2}} / D$ where d = approximate distance between angles in feet; t = thickness of tank plates in inches; D = diameter of tank, in feet.

$d = 900 \frac{t^{\frac{3}{2}}}{D} =$ 24.3 feet. No circular stiffening angles are required in this design, and none are placed on the tank.

34. The top of the tank will generally be covered with a conical roof of tin plates ; and the pitch shall be one to six. For tanks up to 22ft in diam. , the roof plates will be assumed to be self supporting. If the diameter of the tank exceedes 22 ft, angle rafters shall be used to support the roof plates which are generally 1/8" thick

Plates of the following thickness will be assumed to be self supporting for various diameters; 3/32 inch plate, up to a diameter of 18 feet. 1/8" plate up to a diameter of 20 feet 3/16 inch plate, up to a diameter of 22 feet.
Rivets in the roof plates shall be from 1/4 to 5/16 of an inch in diameter and shall be driven cold. These rivets need not be headed with a button set.

The pitch of the roof is greater than 1 to 6 inch The roof plates are m 1/8 inch thick, as the diameter of the tank is 16 ft, the plates are self supporting. The rivets are 5/16 inch round.

35. The trap door 2 feet square, shall be provided in the roof plate. Near the top of the higher tanks, there shall be a platform with a railing for the safety of the men operating the trap door.

There is a trap door 22 inches by 28" There is no platform and railing. There is an ornamental finial.

37/ There shall be a ladder 1 foot 3 in. wide, extending from a point about 8 feet above the foundation to the topof the tank, and also one on the inside of the tank. Each ladder shall be made of two 2-1/2 by 3/8 in. bars with 3/4" round ruggs one foot apart . On large high tanks 30 feet or more in diameter, a walk shall be provided from the column nearest the ladder to the expansion joint on the riser or inlet pipe.

The ladder extends from the piers to the top of the tank.

It is one foot 2-1/4 inches wide; the bars are 2 inches by 3/8 inch; and the rungs are 3/4 inch round and are spaced one foot apart.

38. In designing a tank a height of 6 in. shall be added to the required height of the tank if an overflow pipe is not specified by the owner.

The riser or inlet pipe is 8 ft.inches in diameter. Ther is an outlet pipe

40. All pipes entering the tank shall have cast iron expansion joints with rubber packing and facilities for tightening such joints the expansion joint, generally, shall be fastened to the bottom of the tank with bolts having lead washers. The tank plates shall be reinforced where the pipe enter the tank.

This design has a cast iron expansion joint wiht brass packing; there is a bolt for tightening the joint. The expansion joint is fastened to the tank with rivets

41. All pipes entering the tank shall be thoroughly braced latterly with adjustable diagonal bracing at the panel points of the tower .

There are latteral brace rods, diagonally placed at the panel points for strengthening the inlet pipe.

42. The diagonal bracing in the tower shall preferably be adjustable, and shall be calculated for an enitial stress 3000 lb. in adition to wind stresses etc.

The max uplift is 71,930 lbs. The anchor bolts are fastened directly to the columns by the means of bent plates bearing on angles.

43. The size and number of the anchor bolts in the tower shall be determined by the maximum uplift when the tank is empty. The anchor bolts in the tower, where the maximum uplift is greater than 10,000 lb., shall be fastened directly to the columns with bent plates or similar details. In all other cases it wouldbe sufficient to connect the anchor bolts directly to the base plates.

The tension and anchor bolts will not exceed 15,000 lbs per sq. in. of net area. The minimum section shall be limited to a diameter of 1-1/4 in. The details shall be made so that the anchor bolts will develop thier full strength and at the lower end, they shall be furnished with an anchor plate, not less than 1/2 in. thick to assure good anchorage to the foundation without depending on the adhesion between the conrete and steel .

The anchor bolts 2inches in diam. The unit tension in the anchor bolt is 22,500 lbs. per sq. in. which exceeds the allowable value of 15,000 lbs. per dq. in.

44/ The concrete foundation shall be assumed to have a weight of 140 pounds per cu. ft. and shall be sufficient in quantity to take the uplift, with a factor of safety of 1-1/2.

The factor of safety against overturning is 1.396 which is slightly below the specified value .

45. Three-ply frostproof casing shall be provided if necessary , around the pipes leading to and from the tank. This casing shall be composed of two layers of 7/8 x 2-1/2 inch dressed lumber , and each layer shall be covered with tar paper or tarred felt, and one outside layer of 7/8 x 2-1/2 in. dressed and matched flooring. The lumber shall be in lengths of about 12 ft. There shall be a one inch air space between the layers of lumber ahd wooden rings or seperators shall be nailed to them every three feet. ( In very cold climate it is good practise to fill the space between the pipes and the first layer of lumber with hay or similar material) The frost casing may be square or cylindrical; it shall be braced to the tower with adjustable diagonal bracing, as described for pipes in section 41.

In this design a two-ply frostproof casing was used it consists of three-quarter inch dressed and matched lumber. A strip of building paper is placed on the outside of the inner sheath. There are wooden seperators and they are spaced about two feet 8 in. apart.

## ACKNOWLEDGEMENTS

In the solution of this work we are indebted to Prof. C. A. Melick for advice and help; to Prof, J.A. Polson and Mr. L. F. Newell for the information regarding the necessity of building a wtaer tower at N A C; and also to Ketchum's Structural Hand-book, the American Civil Engr's Handbook, and Hazelhurst's Design of Steel Water Towers, for formula, information, and general knowledge of steel structures.

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## SUMMARY

The analysis of the various members, shows that the tower is overstressed in almost every detail. The most extreme cases being an excess of stress of 95% in the columns of the uppermost panel,due to eccectricity of centers and an excess of 67% in the columns of the lowest panel. The wind bracing is also overstressed to the extent of 35%.

It would have been better to have designed the diagonal bracing in the lower panels to take compression as well as tension; as shown by the graphical analysis of the wind stresses, there will be a compressive stress of 9,9000# in $D_7$ when the horizontal components of the reactions are equal. This will reduce the initial tension to zero and produce excessive bending in the columns at the first panel points above the piers.

On the whole the design appears to be a scant affair. As shown by the graphhical analysis sent us by the Chicago Bridge and Iron Co., and the unit stresses used in the design; they did not use standard values as good practise would call for. The formula for the allowable compressive stress in the columns, as used by them,is 17,100 - 57 l/R,with a max unit stress of 12000#.

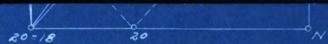
The allowable tensile stress used is 15,000# sq. in. The wind loads appear to have been arrived at in a rather crude manner. The wind on tank being taken at 30# sq. ft. on 60% of projected area, and the wind on tower at 200# vertical ft. of height. This however aggrees with some of the older and more liberal specifications.

The stresses in the columns of the top panel, due to accentricity appear to have been neglected entirely.

The workmanship in riveting on the tower has been done in a careless manner, some of the button heads are scant and irregular, and where counter-sunk rivets are used the head is not large enough to entirely fill the hole.

The site appears to be a rather poor one as regards the nearness to the stack of the power house. The deleterious effect of the acid gases upon the paint is beyond any question of a doubt. A dark green or black paint might better have been selected, as the white paint is all ready becoming discolored.

The extreme thinness of metal used in the entire structure together with the unfortunate exposure to gases and the probability, amounting almost to acertainty that the tank will not be kept properly painted and the large percentage of area rendered inert by a comparatively thin coating of rust do not argue well for a long and uninterrupted life of service to the college.



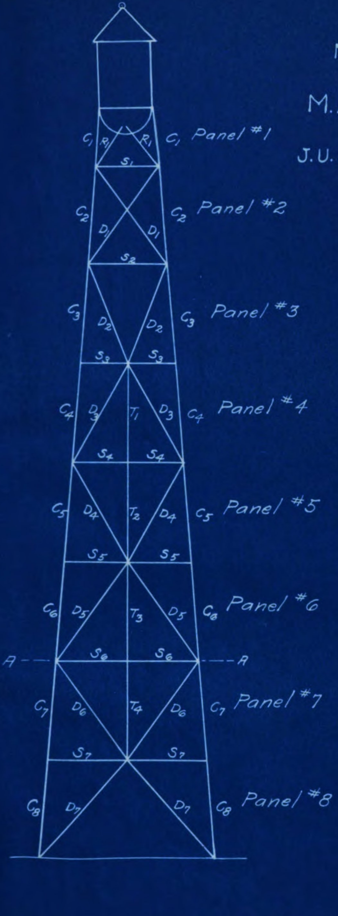

| Member | Wind stress | Wind stress x Sec. θ sec θ = 1.01 |
|---|---|---|
| 6-7 | − 5200 | − 5260 |
| 8-9 | − 6400 | − 6470 |
| 9-11 | − 2400 | − 2420 |
| 12-13 | − 8900 | − 9000 |
| 13-15 | − 2500 | − 2530 |
| 16-17 | − 11800 | − 11920 |
| 17-19 | − 2300 | − 2320 |
| F-7 | + 6000 | + 6060 |
| G-9 | + 12600 | + 12730 |
| H-11 | + 12600 | + 12730 |
| I-13 | + 26250 | + 26500 |
| J-15 | + 26250 | + 26500 |
| K-17 | + 40400 | + 40800 |
| L-19 | + 40400 | + 40800 |
| 6-A | − 6000 | − 6060 |
| 8-A | − 12600 | − 12730 |
| 10-A | − 25870 | − 26100 |
| 14-A | − 40200 | − 40600 |
| 18-A | − 55200 | − 55700 |
| 5-6 | + 8500 | + 8600 |
| 7-8 | + 8000 | + 8100 |
| 9-10 | + 14200 | + 14350 |
| 11-12 | + 15550 | + 15700 |
| 10-12 | − 5000 | − 505 |
| 13-14 | + 16000 | + 16200 |
| 15-16 | + 17400 | + 17600 |
| 14-16 | + 200 | − 202 |
| 17-18 | + 18200 | + 18400 |
| 19-20 | + 19650 | + 19850 |
| 18-20 | 0 | 0 |
| M-N | 55000 | 55600 |
| N-20 | 17200 | 17400 |
| O-20 | 55000 | 55600 |
| O-A | 4500 | 4550 |
| 17-17′ | − 9800 | − 9900 |

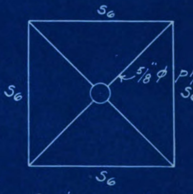
# GRAPHICAL ANALYSIS
# OF
# WIND STRESSES IN TOWER
## THESIS OF
## J. U. LAYER & A. J. RITCHIE
## 1916

Wind taken as blowing square against side of tower
Max. stresses in columns with wind blowing diagonally
θ is the angle between plane of bent & the vertical

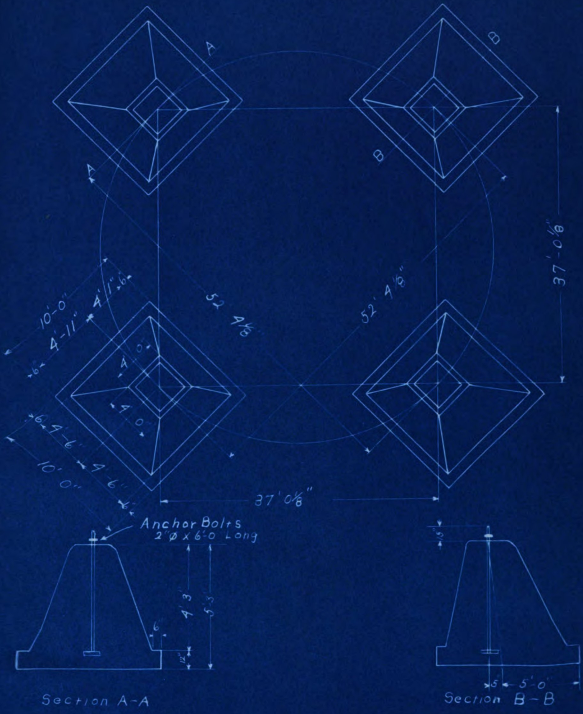
+ = Tension
− = Compression

# MARKING DIAGRA
OF
# M.A.C. WATER TO
THESIS OF
J.U. LAYER & A.J. RI
1916

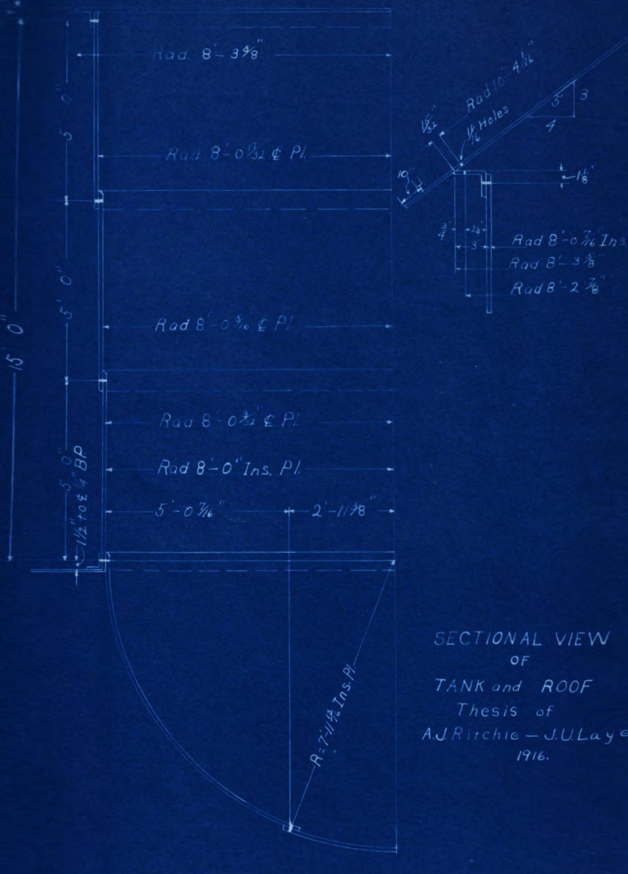


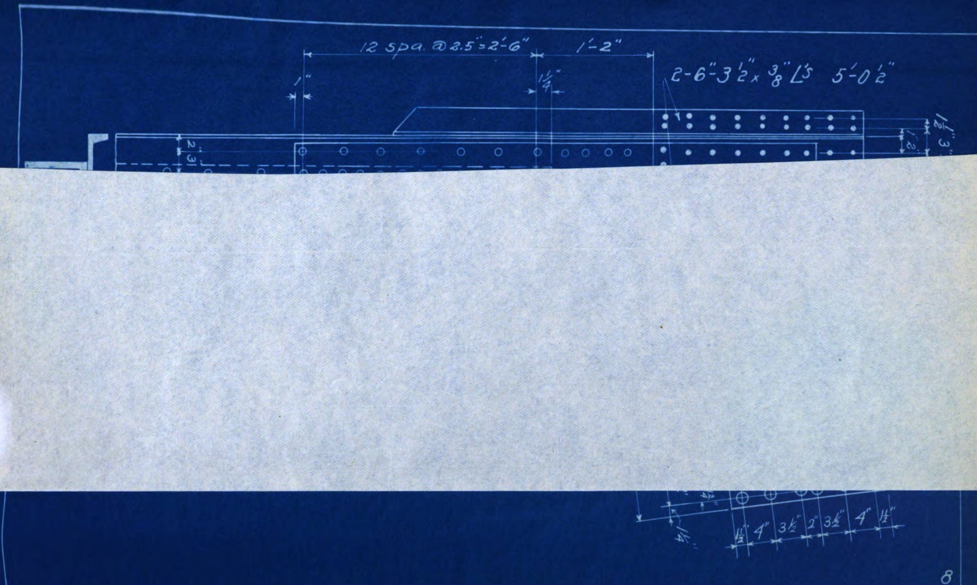

Section on A-A
showing horizont
pipe rods

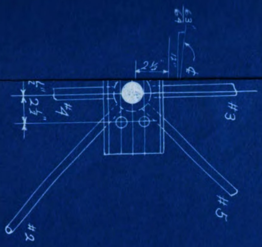


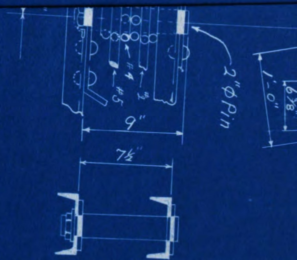

FOUNDATION PLAN
For
M.A.C. WATER TOWER
Thesis of
A.J. Ritchie — J.U. Layer
1916.

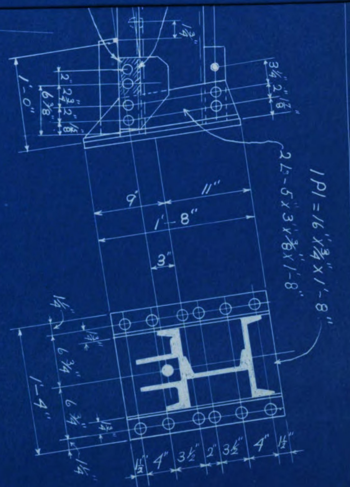



SECTIONAL VIEW
OF
TANK and ROOF
Thesis of
A.J. Ritchie — J.U. Laye
1916.


12 spa. @ 2.5" = 2'-6"
1'-2"
1"
1¼"
2-6"-3½" x ⅜" L's 5'-0½"
1½"
1"
3"
2
3
1½"
4"
3½"
2"
3½"
4"
1½"
1¼"
8




DETAILS OF M.A.C. WATER TOWER

Thesis of

A.J. Ritchie — J. U. Layer.

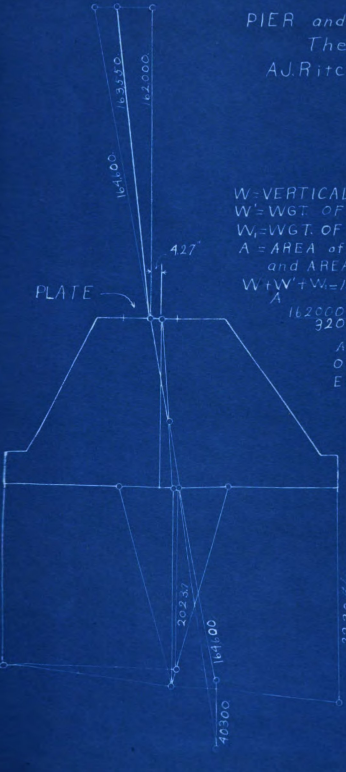
1916.

# GRAPHICAL ANALYSIS OF MAX. UIT PRESSURES ON PIER and FOUNDATION

Thesis of

A.J. Ritchie – J.U. Layer

1916




W = VERTICAL COMP. OF MAX. COL. STRES

W' = WGT. OF PIER

$W_1$ = WGT. OF SHOE

A = AREA of PIER for FOUNDATION and AREA of SHOE for PIER

$\frac{W + W' + W_1}{A} = \frac{162000 + 40300 + 170}{100} = 2023.$ #/

$\frac{162000 + 170}{320} = 506.$ #/□

AS RESULTANT IS IN CENTE OF PLATE ∴ MAX PRESSUR EQUALS AVE. PRESSURE OR 506 #/□