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A STUDY TO DETERMINE THE ECONOMICAL TANK SIZE FOR RADIOACTIVE WASTE DISPOSAL

BY

H. W. STIVERS

PROCESS ENGINEERING SUB-SECTION
DESIGN SECTION - ENGINEERING DEPARTMENT

FEBRUARY 11, 1955

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Acknowledgement is also made to the Estimating Unit, Project Auxiliaries Sub-Section, for their assistance in establishing the lump-sum bid costs for construction material costs and labor costs used in this report.

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A STUDY TO DETERMINE THE ECONOMICAL TANK SIZE
FOR RADIOACTIVE WASTE DISPOSAL

INTRODUCTION

Gradual economic progress has been made in the design and construction of tanks for the storage of radioactive plant effluent wastes from the separation processes. The attainment of tanks of truly optimum proportions was approached but not fully achieved. Site conditions, urgency of the work and lack of time for extensive investigation, precedent, existing and available construction equipment, and limitations on the volume of wastes to be stored, among other reasons, were limiting factors in achieving tanks of optimum proportions.

Including two presently nearing completion, there are 11 tank farms at HAP0 which are used for the storage of radioactive plant effluent wastes. These tank farms comprise a total of 129 tanks varying in capacity from 533,000 to 1,000,000 gallons and represent a total nominal capacity of 89.4 million gallons.

All of these underground waste storage tanks are composite consisting of an open top welded carbon steel tank enveloped by a cylindrical concrete tank with a dome of elliptical shape. All of the tanks have inside diameters of 75 feet. All have dished bottoms with the exception of six which have flat bottoms. The steel liner bottoms in 108 tanks are connected to the shell plates by long radius "knuckle" or transitional sections. The steel bottoms of 21 tanks are connected to the steel shell by right-angle tee connections. See Fig. 13, 14, 15 and 16 for composite sections of these tanks.

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(4) floor 2 percent, (5) steel liner 20 percent prior to 1946 plus 12 tanks. If the optimum tank size is not utilized, an additional capacity of 533,000 gallons, items exists and the unit cost (dollars per gallon) will be increased.

Significant changes in tank design. For any given tank capacity there is a 42 percent greater volume with the capacity increases, the diameter increases. Forty-eight of the larger the height increases but at a lesser rate. Having a nominal capacity of 758,000 diameter tank, the optimum height is 34.8 feet. For a nominal capacity of 1,000,000 tank, the optimum height is 39.8 feet. Each increase of about 32 percent in the capacity. The optimum height is that height at which the unit cost is minimized. Other a minimum for a given tank diameter. From which further reduced costs. Each of optimum unit cost is \$0.136 per gallon for tanks of 1,000,000 gallons. and 130 feet. For tanks with diameters less than 130 feet, the unit cost increases. Tanks greater than 130-foot diameter.

To determine an optimum tank size by the use of construction materials with pre-determined unit costs. Any alterations in the design criteria will alter the optimum tank dimensions and their relationship to the unit costs of construction materials will alter the optimum tank dimensions but do not appreciably affect the optimum tank dimensions. Arrangement and cost of the connection and vapor manifolds must be considered to determine the optimum layout. These factors are not a part of this study.

Tank Farms 241-S, 241-SX and 241-A were analyzed in this study. The storage cost of the analyzed tanks was \$0.136 per gallon for 241-SX and 241-A, and \$0.136 per gallon for 241-S. The assumed unit costs for concrete and the foundation 6 percent, and the cost of the connection and vapor manifold costs were omitted. The factors that were analyzed are: (1) concrete, (2) steel, (3) wood formwork, (4) 3/8-inch earthwork excavation and backfill, (5) connection and vapor manifold costs were omitted. The unit costs were approximately as follows:

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approximately 10 percent less than those of Tank Farms 241-SX and 241-A which caused the lower unit cost (\$0.121 per gallon) for the analyzed tank. This difference in unit cost has little or no effect on the optimum height-diameter ratio for the purposes of this report.

Tanks used for farms 241-SX and 241-A are fairly close to the optimum tank size. Although a small savings in storage unit cost will be realized by using optimum dimensions, a greater savings could be realized by increasing the tank capacity 20 to 25 percent without changing the tank diameter or the storage unit cost.

RECOMMENDATIONS

It is recommended that the curves in the Appendix of this report be used to select optimum tanks to facilitate the requirements of new tank farms. Consideration of the site location, piping and vapor manifolds, and over-all layout of the tank farm will affect the capacity of the optimum tank. Alterations in material costs must be analyzed to adjust the tank cost.

DISCUSSION

Design Criteria

The following design criteria used for the stress analysis of tanks was contributed by M. J. Rutherford, Design Engineering, who has participated in the design of all the tank farms under General Electric Company management.

Loads

Shielding: Equivalent to seven feet earth coverage at 115 psf
plus 1'-3" concrete slab at 150 psf

Equipment: Two D-8 tractors at 35,000 lb each

Vacuum: Equivalent to 12 in. of water

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Allowable Stresses

Use conventional reinforced concrete

Concrete $f'_c = 3000$ lb

Dome

Elliptical without openings in its surface

Rise to span ratio 0.15

Floors and Walls

Liner: 3/8-in. plate, carbon steel

Floor: 6-in. thick with #5 at 12 in. EW

Liquid sp gr: 1.50

When designing the concrete wall, consider neither the 3/8-inch steel plate liner nor the earth pressure. Consider only the storage liquid as supported by the concrete wall.

In addition to the above criteria, the allowable compressive stress in concrete was assumed to be $0.10 f'_c$ (300 psi). The existing compressive stress in most concrete domes is seldom more than 150 to 200 psi. However, in the design of this particular dome with total load nearly 1000 psf, it is necessary to stipulate an allowable compressive stress in the concrete. In the design of the tank walls, an allowable compressive stress in concrete of 300 psi⁷ is used for concrete requirements. The allowable compressive stress in the extreme fiber for 3000-pound concrete is 1350 psi¹, and for bearing on the full area is 750 psi, and for shear in beams with properly designed web reinforcement is 360 psi. All of these allowable stresses are considerably higher than the assumed allowable stress of 300 psi. It is believed by the writer when using this allowable stress, the indeterminate stresses

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near edge members, and stresses due to volume changes and settlement, etc., will not be critical under normal conditions. The maximum concrete stresses in the dome of previous tank designs are very close to 300 psi.¹⁰

The reinforcement steel for the dome was assumed to have an allowable stress of 14,000 psi which is standard for intermediate grade steel used in reinforced concrete tank design.⁷

Tables

Table 1, below, is a tabulation of material unit costs (in-place) used to compute the material costs for the 21 tanks analyzed in this report.

Table 1. Material Unit Costs (In-Place)

Material	Location	Units	Costs
Concrete	Walls	Cubic Yard	\$ 18.00
	Dome	Cubic Yard	\$ 18.00
	Foundation	Cubic Yard	\$ 15.00
	Floor	Cubic Yard	\$ 15.00
Wood Forms	Walls	Square Foot	\$ 0.70
	Dome	Square Foot	\$ 2.00*
	Foundation	Square Foot	\$ 1.00
Re-Steel	Entire Tank	Ton	\$ 200.00
Steel Liner	Walls & Floor	Ton	\$ 310.00
Earthwork	Excavation	Cubic Yard	\$ 0.30
	Backfill	Cubic Yard	\$ 0.36

*The high cost of the dome formwork is due to labor in handling the forms through a 42-inch access opening at the crown of the dome.

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Table 2, page 13, is a cost comparison of Tank Farms 241-S, 241-SX, 241-A, and a tank as analyzed in this report with the same diameter and nominal capacity.

The 241-S tank farm was constructed during 1950, but no alteration in costs was made to compensate for the increase in material and labor costs. Therefore, to construct a similar tank today would be \$0.163 per gallon. A high "Miscellaneous Tank Cost" is due to the painting and sandblasting of the steel plate tank liner which has been discontinued on later tanks. The "Construction Cost of the Tank Farm" includes all risers and dome plugs, miscellaneous pipe for tanks, instrument mounting assemblies and electrical installations, retention basins, waste cribs, process sewers, temporary construction, steam and water lines, and the "Bare Tank Cost".

Tank Farm 241-SX was constructed during 1953. The "Miscellaneous Tank Cost" consists of cement grout and membrane waterproofing for the tanks. The "Construction Cost of the Tank Farm" consists of diversion box, piping systems, condenser water supply and drainage, miscellaneous appurtenances and equipment, cathodic protection, final grading, fencing, patrol road, removal of temporary construction facilities, and the "Bare Tank Cost".

Tank Farm 241-A was constructed during 1954. The principal facilities that increased the "Construction Cost of Tank Farm" were the process building, waste line encasement, diversion box, ventilation building, sampler and pump pits, ditch relocation, cribs, steam condensate lines, waste cooling water and drain water piping systems, electrical installations, and the "Bare Tank Cost".

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Table 2. Cost Comparison of Tank Farms

Item (1)	241-S (2)		241-SX (3)		241-A (4)		Analyzed Tank (5)	
Nominal Capacity (Gal)	758,000		1,000,000		1,000,000		1,000,000	
Inside Diameter (Ft)	75		75		75		75	
Number of Tanks	12		15		6		1	
	<div>Cost</div> <div>Total \$/Gal</div>		<div>Cost</div> <div>Total \$/Gal</div>		<div>Cost</div> <div>Total \$/Gal</div>		<div>Cost</div> <div>Total \$/Gal</div>	
Concrete & Formwork	\$ 498,970	\$0.055	\$ 735,588	\$0.049	\$ 281,000	\$0.047	\$ 36,504	\$0.037*
Reinforcing Steel	302,500	0.033	462,500	0.031	174,000	0.029	32,380	0.032
Steel Plate Tank Liner	350,550	0.039	534,600	0.036	182,000	0.030	28,800	0.029*
Earthwork	<u>230,000</u>	<u>0.025</u>	<u>308,800</u>	<u>0.020</u>	<u>180,000</u>	<u>0.030</u>	<u>23,200</u>	<u>0.023</u>
Bare Tank Cost	\$1,382,020	\$0.152	\$2,041,488	\$0.136	\$ 817,000	\$0.136	\$120,884	\$0.121*
Misc. Tank Cost	\$ 210,000	\$0.023	\$ 39,060	\$0.0026	\$ 27,500	\$0.0046		
Constr. Cost of Tank Farm	\$2,225,326	\$0.244	\$2,575,120	\$0.173	\$1,750,000	\$0.292		
Bare Tank Cost Percent of Construction Cost	71%		79%		47%			

*The unit cost variations are contributed to the following: (1) 241-S Concrete - Dish-shaped concrete floor slab, concrete volume includes all grouting, storage capacity was 25 percent less which decreased the concrete volume only in the walls. Steel Plate Tank Liner - Large radius "knuckle"-connection and dish-shaped bottom, and the short wall height kept the unit price of storage space higher. (2) 241-SX Concrete - Dish-shaped concrete floor slab. (3) 241-A Concrete - Higher concrete volume estimated for domes. (4) Analyzed Tank - Estimated concrete volume was approximately eight percent less. No grouting was included. Combined unit cost of concrete and formwork was approximately five percent less. Steel Plate Tank Liner - Neglects cost of angle stiffeners. The unit price of the analyzed bare tank is 11 to 12 percent low; however, uniform alterations of material unit costs will not change the proportional dimensions of the optimum tanks.

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Table 3, page 15, is a summary of the various tank dimensions, capacities, and material quantities for tanks that were analyzed to determine an optimum tank size. Small tanks with a diameter less than 50 feet were not investigated due to the increase in unit cost and the increase in piping facilities and condenser systems required for a complete tank farm which would increase the unit cost of the over-all project. Large tanks in excess of 130-foot diameter are not feasible with plain-reinforced concrete due to the material mass for a continuous pour, depth of excavation, and construction difficulties encountered with no saving realized in unit cost of the storage space. A "pre-stress" design may be feasible for tanks greater than 130 feet in diameter if a comparable unit cost per gallon (say \$0.130 per gallon) could be realized and if there is no effect due to the "concentration of mass" of the disposal wastes. Large tanks are desirable to minimize the number of required condensers, connecting piping systems and diversion boxes, which would decrease the over-all unit cost of the tank farm.

It is interesting to note from Table 3, page 15, that a 130-foot diameter dome requires about 21 times as much concrete, 26 times as much reinforcing steel, and only 6.7 times as much formwork as a 50-foot diameter dome, and still the unit cost of storage space for a 130-foot diameter tank is less than that for a 50-foot diameter tank when using the optimum tank dimensions. Even with these extreme quantities of material the ratio of steel to concrete is practically a constant for various tank sizes at the same critical sections. Table 4, page 16, gives the steel percentage at various critical sections.

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Table 3. Tank Dimensions, Capacities, Unit Costs and

Diam (Ft)	Liquid Depth (Ft)	Well Height (Ft)	Capacity MG (Ft)	Unit Cost (\$/gal)	Dome			Walls			Co
					Conc. Cu Yd	Forms Sq Ft	Re-Steel Ton	Conc. Cu Yd	Forms Sq Ft	Re-Steel Ton	
50	10	12	0.147	0.196	94	2,310	13.8	63.8	1,950	7.8	30
50	20	22	0.294	0.139	94	2,310	13.8	153	3,600	16.7	33
50	30	32	0.445	0.131	94	2,310	13.8	352	5,280	41.8	42
50	40	42	0.588	0.141	94	2,310	13.8	645	7,210	82.5	51
75	20	22	0.660	0.134	334	5,480	56.2	281	5,370	30.2	1
75	30	32	1.000	0.121	334	5,480	56.2	537	7,850	92.0	1
75	40	42	1.320	0.122	334	5,480	56.2	1230	10,700	146	2
85.5	20	22	0.858	0.139	565	6,710	93.0	340	6,090	38.8	2
85.5	24.5	26.5	1.050	0.129	565	6,710	93.0	532	7,410	60.0	2
85.5	30.3	32.3	1.300	0.123	565	6,710	93.0	830	9,150	99.5	3
85.5	35	37	1.50	0.121	565	6,710	93.0	1160	10,600	130	3
85.5	43	45	1.85	0.123	565	6,710	93.0	1850	13,100	204	1
85.5	54	56	2.32	0.131	565	6,710	93.0	3060	16,700	331	5
100	22	24	1.29	0.138	831	9,230	156	536	7,810	62.4	
100	30	32	1.82	0.124	831	9,230	156	1170	10,900	141	
100	37.5	39.5	2.20	0.120	831	9,230	156	1750	13,250	195	
100	47	49	2.78	0.123	831	9,230	156	2900	16,800	325	
130	30	32	3.00	0.132	2002	15,600	359	1630	13,900	183	1
130	40	42	3.98	0.122	2002	15,600	359	2950	18,200	334	1
130	52	54	5.17	0.123	2002	15,600	359	5290	24,000	592	2
130	60	62	5.96	0.131	2002	15,600	359	7460	28,100	832	3

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ons, Capacities, Unit Costs and Material Quantities Summary

Steel cn	Walls			Foundation			Floor		Earthwork		Steel Liner Tons
	Conc. Cu Yd	Forms Sq Ft	Re-Steel Ton	Conc. Cu Yd	Forms Sq Ft	Re-Steel Ton	Conc. Cu Yd	Re-Steel Ton	Excav. Cu Yd	Backfill Cu Yd	
3.8	63.8	1,950	7.8	30.3	174	2.23	33.4	1.74	7,900	6,412	29.4
3.3	153	3,600	16.7	33.2	177	2.30	33.0	1.72	14,100	11,867	41.4
3.8	352	5,280	41.3	42.0	196	2.96	32.3	1.70	21,300	17,060	54.0
3.8	645	7,210	82.5	51.8	209	3.16	32.2	1.67	30,000	25,781	65.5
16.2	281	5,370	30.2	136	445	8.83	72.2	3.76	27,400	21,840	72.6
16.2	537	7,850	92.0	189	534	10.0	70.5	3.7	36,300	28,800	93.0
16.2	1230	10,700	146	211	526	14.0	69.5	3.62	53,000	43,075	109.5
93.0	340	6,090	38.8	272	668	14.8	90.1	4.69	36,700	28,847	89.1
93.0	532	7,410	60.0	290	689	16.3	89.0	4.63	43,300	34,275	98.4
93.0	830	9,150	99.5	322	725	20.9	88.6	4.61	51,900	41,315	110.3
93.0	1160	10,600	130	349	745	22.2	88.0	4.58	59,500	47,590	119.9
93.0	1850	13,100	204	420	819	26.5	86.2	4.48	74,600	62,245	136.4
93.0	3060	16,700	331	586	960	35.2	79.5	4.14	101,000	83,020	159.0
156	536	7,810	62.4	510	1010	31.6	119	6.23	55,100	42,860	118
156	1170	10,900	141	585	1060	34.0	113	6.20	71,500	55,310	139
156	1750	13,250	195	624	1070	38.2	117	6.04	86,200	68,240	155
156	2900	16,800	325	855	1270	51.8	112	5.80	111,000	88,880	178
359	1630	13,900	183	1540	1970	113	188	9.80	114,000	85,400	202
359	2950	18,200	334	1970	2260	118	184	9.58	146,000	111,130	232
359	5290	24,000	592	2620	2640	151	174	9.05	198,000	154,440	271
359	7460	28,100	832	3320	3010	193	164	8.54	235,000	184,720	295

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Table 4. Required Steel Percentage at the Critical Sections

Diameter (Ft)	Wall Height (Ft)	Dome		Wall	Foundation	Floor
		Crown % Steel	Haunch % Steel	Maximum % Steel	Maximum % Steel	Maximum % Steel
50	12	1.03	3.54	2.00	1.20	0.85
50	22	1.03	3.54	1.78	1.05	0.85
50	32	1.03	3.54	1.94	1.15	0.85
50	42	1.03	3.54	2.09	0.99	0.85
75	22	0.98	4.22	1.75	1.06	0.85
75	32	0.98	4.22	2.80*	0.86	0.85
75	42	0.98	4.22	1.94	1.08	0.85
85.5	22	0.81	4.46	1.86	0.89	0.85
85.5	26.5	0.81	4.46	1.84	0.92	0.85
85.5	32.3	0.81	4.46	1.96	1.06	0.85
85.5	37	0.81	4.46	1.83	1.04	0.85
85.5	45	0.81	4.46	1.80	1.03	0.85
85.5	56	0.81	4.46	1.76	0.98	0.85
100	24	1.03	5.05	1.90	1.01	0.85
100	32	1.03	5.05	1.96	0.95	0.85
100	39.5	1.03	5.05	1.82	1.00	0.85
100	49	1.03	5.05	1.83	0.99	0.85
130	32	0.75	5.05	1.83	1.20	0.85
130	42	0.75	5.05	1.85	0.97	0.85
130	54	0.75	5.05	1.83	0.94	0.85
130	62	0.75	5.05	1.82	0.95	0.85

*For tanks used in construction of 241-SX.

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At the crown section of the dome, the steel ratio varies from 0.75 percent to 1.03 percent. The crown is in axial compression similar to a short column in which a minimum of one percent steel is required. The steel ratio at the haunch varies from 3.54 to 5.05 percent which is the most variation of any critical section. This can be understood due to the indeterminate stresses that occur near the end members of this type of structure. Also, the haunch has been added to provide concrete coverage for the required steel. For the tank walls used in the construction of 241-SX, the steel ratio is 2.8 percent and the steel ratio of the walls as computed in this report is approximately 1.9 percent. This increase of 50 percent steel will increase the concrete stress due to shrinkage of the concrete. However, if the tanks are not used for a period greater than six months after construction, it is possible that some of this stress is relieved by plastic flow of the steel and concrete. However, it was the intent of this report to follow standards that have been established by the Portland Cement Association and the American Concrete Institute, and therefore no assumptions of this nature have been made.

Explanation of Curves

From Table 3, page 15, there have been five curves developed which will enable the selection of the most economical tank size when the assumptions and analyses in this report can be used for a tank farm design. If these tanks were placed above ground, or if steel liners were not required, or if a pre-stress design was used, or if differently shaped tanks with different covers were considered, these graphs would not conform. Also, the unit costs of materials have been derived from conservative lump-sum bid.

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costs during 1954 and any fluctuation in these would alter the curves. From the material quantities in Table 3, page 15, and any desired unit costs, new curves may be easily calculated at a later date to coincide with new material costs.

Fig. 7. Cost per Gallon - Tank Capacity Curve

With the tank cost (dollars per gallon) as ordinates and tank capacity (million gallons) as the abscissae, curves of constant diameter and varying height have been plotted from Table 3. Each curve is concave-shape with the 50-foot diameter tank having the most pronounced shape. This illustrates that the smaller the diameter, the more critical is the wall height. The 130-foot diameter tank could have a large variation in capacity (between four and five million gallons) with little effect on the unit cost, while the 50-foot diameter tank must have a 410,000-gallon capacity for a minimum unit cost. Tanks with diameters between 75 and 130 feet have the same minimum unit cost per gallon, while tanks of less than 75 feet diameter have a greater minimum unit cost. The minimum cost of a 50-foot diameter tank is \$0.130 per gallon, while 75-foot diameter tanks and larger have a minimum unit cost of \$0.12 per gallon.

Tanks constructed during the management of DuPont were 533,000-gallon, 75-foot diameter tanks with the same characteristics as the tanks in this report. From Fig. 7, the unit cost of this tank would be \$0.143 per gallon. A 50-foot diameter tank with the same capacity could have been constructed for \$0.135 per gallon. A savings of \$4260 per tank could have been realized and since approximately 60 tanks of this type have been constructed, \$256,000 would have been saved by using a 50-foot diameter tank.

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Under General Electric management, the 75-foot diameter tank was increased in capacity to 758,000 gallons by increasing the liquid storage depth. From Fig. 7, the unit cost of this tank was \$0.128 per gallon. This savings of \$0.015 per gallon was accomplished by this change. Later, for Tank Farm 241-SX, the volume of the 75-foot diameter tank was increased to 1,000,000 gallons by increasing the liquid depth to 30 feet. From Fig. 7, the unit cost of this tank would be \$0.121 per gallon. An additional savings of \$0.007 per gallon was accomplished by this change.

The 1,000,000-gallon tanks, as now constructed, are very near the optimum unit cost for the tank alone. However, Fig. 7, with a 75-foot tank, illustrates a 20 to 25 percent increase in tank size may be realized with no increase in the unit cost of the tank. The required number of tanks for an entire farm could be reduced 25 percent eliminating at least five percent of the over-all farm cost for piping and condensers, etc. For instance, a 24,000,000-gallon tank farm would require 24 1,000,000-gallon tanks and only 20 1.2 million-gallon tanks. If the average cost of a condenser and vapor manifolding alone is \$50,000 per tank, then a savings of \$200,000 is realized by the 20 percent volume increase.

Fig. 8. Optimum $\frac{h}{d}$ Ratio - Diameter Curve

Fig. 7 illustrates that for each diameter there is a liquid depth which will give a minimum unit cost per gallon. This liquid depth is the optimum liquid depth for that tank diameter. Each diameter has only one optimum liquid depth.

In Fig. 8, with the ordinates as the ratio of the optimum wall height to the tank diameter $\left(\frac{h}{d}\right)$, and the tank diameter as the abscissae, a curve

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is plotted to illustrate the relationship between the optimum $\frac{h}{d}$ ratio and the tank diameter. As the diameter increases the optimum $\frac{h}{d}$ ratio decreases, similar to the general shape of the hyperbola. This curve may be used to determine the optimum $\frac{h}{d}$ ratio for any tank diameter between 35 and 150 feet.

Fig. 9. Optimum Wall Height - Diameter Curve

The "Optimum Wall Height - Diameter Curve" is developed from Fig. 8. With the ordinates as the wall height and the abscissae as the tank diameter, a straight-line relationship develops between the optimum wall height and the tank diameter. The wall height is two feet greater than the liquid depth of the waste storage.

Fig. 10. Optimum Tank Capacity - Diameter Curve

The "Optimum Tank Capacity - Diameter Curve" is developed from Fig. 9. With the ordinates as the optimum tank capacity and the abscissae as the tank diameter a curve is plotted to illustrate the relationship between the optimum tank capacity and the diameter. From this curve it is possible to determine an optimum tank size of any capacity.

Fig. 11. Optimum Tank Capacity - Cost Curve

The "Optimum Tank Capacity - Cost Curve" is developed from Fig. 7 when multiplying the optimum unit cost by the optimum capacity for the plotted tank diameter. With the ordinates as the optimum tank cost, and the abscissae as the tank diameter, a straight-line relationship develops between the optimum tank cost and the tank diameter. From this curve it is possible to determine the optimum tank cost for any tank diameter.

Fig. 12. Optimum Tank Data

The "Optimum Tank Data" monograph was developed from Fig. 7, 8, 9, 10

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and 11 to be used in selecting an optimum tank size for any tank farm. A horizontal line must always be used to give results determined in this report.

Example 1.

Assume a 40-foot tank diameter. By projecting a horizontal line through this point, the wall height is 26.7 feet, the liquid depth is 24.7 feet, the tank capacity is 230,000 gallons and the tank cost would be \$29,000.

Example 2.

Assume a 75-foot tank diameter. By projecting another horizontal line through this point, the wall height is 36.4 feet, the liquid depth is 34.4, the tank capacity is 1.15 MG and the tank cost would be \$138,000.

Example 3.

Assume a tank cost of \$280,000. By projecting another horizontal line through this point, the wall height is 41.95 feet, the liquid depth is 39.95 feet, the tank diameter is 100 feet and the tank capacity would be 2.34 MG.

Sample Solution for Tank Farm

With the "Optimum Tank Data" monograph, Fig. 12, page 54, it is possible to determine any optimum tank size for any tank farm capacity. These results are to be used in conjunction with a study on piping arrangements, condensers and vapor manifolds for the particular farm. If the cost of these is high it will be desirable to use a larger tank than if these costs were low.

Assume a 20-MG tank farm. Table 5, page 22, is tabulated from the monograph. To enter the monograph, the tank capacity is determined by dividing the farm capacity by the number of tanks that will give the best layout.

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Table 5. Tank Data for a 20 MG Tank Farm

Number of Tanks	Tank Dia (Ft)	Depth (Ft)		Tank Capacity (MG)	Cost (\$) One Tank	Cost (\$) All Tanks
		Wall	Liquid			
6	115	44.8	42.8	3.34	\$401,000	\$2,406,000
8	102.5	42.4	40.4	2.50	\$300,000	\$2,400,000
12	87	39.3	37.3	1.67	\$200,000	\$2,400,000
14	82	38.1	36.1	1.43	\$172,000	\$2,408,000
16	77.5	37.0	35.0	1.25	\$150,000	\$2,400,000
18	73.8	36.2	34.2	1.11	\$133,000	\$2,400,000
20	71.0	35.4	33.4	1.00	\$120,000	\$2,400,000

From Table 5 it is seen that regardless of the tank capacity, the total tank cost for the farm is a constant when using optimum tanks. Therefore, the economical tank farm now becomes a study of piping arrangements, types of condensers and vapor manifolds.

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APPENDIX

Calculations for Dome

ACI ST 55

Assume

Use the inside surface distances to simplify calculations.

No openings in the dome surface

Inside Tank Diameter 100 ft

Major Axis of Dome 50 ft

Minor Axis of Dome 0.15 (100 ft) = 15 ft

$$f'_c = 3000 \text{ lb}$$

$$f_c = 300 \text{ psi}$$

$$f_s = 14,000 \text{ psi}$$

Loads

1. Dead Load

$$\text{a. Try concrete thickness} = 23 \text{ in. } w_{DL} = \frac{23(150)}{12} = \underline{288 \text{ psf}}$$

The total effective shielding is 8.6 ft.

2. Live Load

$$\text{a. Earth Coverage} = 8.6 - \frac{150}{115} \frac{(23)}{12} = 7.1'; \text{ say } 7'-0"$$

$$w_a = 7(115) = 805 \text{ psf}$$

b. Two D-8 tractors are equivalent to 61.5 psf over total roof area.

$$\text{c. 12 in. Vac} = 62.4 \text{ psf}$$

$$\text{d. Constant } w_{LL} = a + b + c = 805 + 61.5 + 62.4 = \underline{928.9 \text{ psf}}$$

e. Variable w_{LL} = The volume of earth fill between the surface of the dome and an imaginary line projected horizontally from the crown of the dome.

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Formulas for Elliptical Domes⁶

When 'w' is the weight per square foot of dome surface, the meridional thrust at a given point on the curve is

$$(18) \quad T = \frac{wa^2}{b} \left(\frac{CQ}{1 - g^2} \right)$$

The hoop force at a given point on the curve is

$$(19) \quad H = \frac{wa^2}{b} \left(g - \frac{C}{(1 - g^2)Q} \right)$$

At the end of the minor axis, i.e., at the axis of rotation, the meridional thrust is equal to the hoop force

$$(20) \quad T = H = \frac{wa^2}{2b}$$

When 'w' is the weight per square foot uniformly distributed over the horizontal projection of the dome, the meridional thrust at a given point on the curve is

$$(22) \quad T = \frac{wa^2}{2b} Q$$

The hoop force at a given point on the curve is

$$(23) \quad H = \frac{wa^2}{2b} \left(\frac{2g^2 - 1}{Q} \right)$$

At the end of the minor axis, i.e., at the axis of rotation, the meridional thrust is equal to the hoop force

$$(24) \quad T = H = \frac{wa^2}{2b}$$

Plotting the Curve

The general equation of the ellipse is

$$\frac{x^2}{a^2} + \frac{y^2}{b^2} = 1$$

in which 'a' is the radius of the major axis and 'b' is the radius of the

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minor axis with 'x' and 'y' as the respective variables. To determine the curve coordinates assume increments Δy equal to 0.1b, 0.2b, 0.3b, etc.

Solving the general ellipse equation for 'x', we have

$$x = \sqrt{a^2 \left(1 - \frac{y^2}{b^2}\right)}$$

$$\text{Let } g = \frac{y}{b}$$

$$\text{Then } x = \sqrt{a^2 (1 - g^2)}$$

$$\text{For } g = 0.1, y = 0.1 (15) = 1.5$$

$$\begin{aligned} x &= \sqrt{(50)^2 (1 - 0.1^2)} \\ &= 49.7 \text{ ft} \end{aligned}$$

Results of the calculations for the curve coordinates are tabulated in Table 6.

Table 6. Curve Coordinates for 100 Ft Dome, 15 Ft Rise

g	y	x
0.1	1.5	49.7
0.2	3.0	49.0
0.3	4.5	47.8
0.4	6.0	45.8
0.5	7.5	43.2
0.6	9.0	40.0
0.7	10.5	35.7
0.8	12.0	30.0
0.9	13.5	21.8
1.0	15.0	0

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Fig. 1. Ellipse Plotted from Table 6.

Stresses at End of Minor Axis

$$\begin{aligned}
 T &= H = \frac{wa^2}{2b} \\
 &= \frac{(288 + 930)(50)^2}{2(15)(1000)} \\
 &= \underline{101 \text{ kips/ft}}
 \end{aligned}$$

The stress in the concrete without steel reinforcement is

$$\begin{aligned}
 s &= \frac{T}{tb} \\
 &= \frac{101}{(23)(12)} \\
 &= \underline{368 \text{ psi}} > 300 \text{ psi allowable.}
 \end{aligned}$$

However, there shall be steel furnished equal to $0.0025 A_g$ each way, each face, and there will be a combined axial stress in the steel and concrete.

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The Joint Code (1940)¹³ specifies, for spiral and longitudinal reinforcement, the safe axial load

$$P = 0.225 f'_c A_g + f_s A_s$$

and for columns with lateral ties and longitudinal reinforcement

$$P = 0.180 f'_c A_g + 0.8 A_s f_s$$

with f_s equal to 16,000 pounds per square inch.

Since the allowable stress in the concrete has already been established as $0.10 f'_c$, the formula used to determine the combined axial load was

$$P = 0.10 f'_c A_g + 0.8 A_s f_s$$

The minimum steel requirement each way, each face is

$$\begin{aligned} A_s &= 0.0025 A_g \\ &= 0.0025 (12)(23) \\ &= \underline{0.69 \text{ sq in./ft}} \end{aligned}$$

Try $A_s = \#6$ at 7-1/2 in. E.W., E.F. ($A_s = \underline{1.42 \text{ sq in./ft E.W.}}$)

The allowable axial load is

$$\begin{aligned} P &= 0.10 f'_c A_g + 0.80 A_s f_s \\ &= 0.10 (3000)(12)(23) + 0.80 (1.42)(16,000) \\ &= \underline{101 \text{ kips/ft}} = 101 \text{ kips/ft actual} \end{aligned}$$

Therefore, a 100-foot diameter dome with a rise of 15 feet must have a minimum thickness of 23 inches with steel reinforcement of #6 at 7-1/2 inches each way, each face at the crown, which gives a total steel ratio of 1.03 percent.

Stresses at End of Major Axis

1. The evaluation of the variable earth load on the dome at the end of the

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major axis for the computation of the meridional thrust and hoop force is rather difficult to determine since the formulas are based on total load distributed uniformly and symmetrically on the dome's surface such as a snow load. For the meridional thrust the uniform load, 'w' is assumed to be a column of earth one foot square with a height equal to the minor axis of the dome. For the hoop force the variable load is assumed to be distributed uniformly over the horizontal projected area of the dome. As previously determined the uniform live load is 928.9 pounds per square foot. The variable live load for the meridional thrust is

$$\begin{aligned}w_{LL} &= (115)(15) \\ &= \underline{1725 \text{ psf}}\end{aligned}$$

The unit weight for the variable live load to be used in determining the hoop force is

$$\begin{aligned}w_{LL} &= \left(\frac{\text{vol of cyl} - \text{vol of ellipse}}{\text{horiz proj area}} \right) \text{unit weight of earth} \\ &= \left(\frac{hD^2/4 - 2/3(abc)}{D^2/4} \right) \gamma \\ &= \left(\frac{15(100)^2/4 - 2/3(50)(50)(15)}{(100)^2/4} \right) \frac{115}{1000} \\ &= \underline{0.575 \text{ kips/sq ft}}\end{aligned}$$

2. From the "Table of Coefficients for Elliptical Domes"⁶ when

$$g = y/b$$

$$b/a = 0.30$$

the values of C and Q are

$$C = 0.588$$

$$Q = 0.300$$

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The uniform dead load for 23 inches of concrete is 288 psf and the total uniform live load for the meridional thrust will be 928.9 psf plus 1725 psf (say 2.655 kips per square foot). Solving for the meridional stress

$$\begin{aligned} T &= \frac{wa^2}{b} \left(\frac{CG}{1 - g^2} \right) + \frac{wa^2}{2b} (Q) \\ &= \frac{0.288(50)^2}{15} \left(\frac{0.588(0.300)}{1 - 0} \right) + \frac{2.655(50)^2(0.300)}{2(15)} \\ &= 74.9 \text{ kips/ft} \end{aligned}$$

If the meridional thrust produces axial stress only, the allowable load when using one percent reinforcement is

$$\begin{aligned} P &= 0.10 f'_c A_g + 0.80 f_s A_s \\ &= 0.10 (300)(12)(23) + 0.80(16,000)(0.01)(12)(23) \\ &= 108 \text{ kips/ft} > 74.9 \text{ kips/ft actual} \end{aligned}$$

This indicates that if only axial stress is encountered at the junction of the wall and dome, the section could be decreased to satisfy the actual stresses for a more economical section. However, due to the indeterminate stresses that occur at this section it is not advisable to have a smaller section at the end of the major axis than at the end of the minor axis. To illustrate an eccentric load condition at the end of the major axis, assume the meridional thrust lies two inches from the center of the section which is considered to be the neutral axis. From Diagram 17, page 553¹³, when

$$\begin{aligned} e/a &= \frac{2}{23} \\ &= 0.087 \\ \frac{d'}{a} &= \frac{2.5}{23} \end{aligned}$$

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$$\frac{d'}{a} = 0.11$$
$$np_g = 0.10$$

the value of

$$K = 1.35$$
$$f_c = \frac{NK}{ba}$$
$$= \frac{74.9(1.35)}{12(23)}$$
$$= 365 \text{ psf} > 300 \text{ psf allowable}$$

A section with a depth of approximately 28 inches would be required to satisfy an eccentricity of two inches. Due to the indeterminate eccentricity and the approximation of the formulas at the end of the major axis and the allowable stresses used, a 12-inch by 23-inch section with one percent reinforcement at the end of the major axis is considered satisfactory for the meridional thrust. One percent reinforcement is satisfied by using

$$A_s = 0.01 A_g$$
$$= 0.01(12)(23)(1/2)$$
$$= 1.38 \text{ sq in./ft}$$

Use #8 at 6-1/2 in. E.F. ($A_s = 1.45 \text{ sq in./ft}$)

The gross steel ratio for the meridional thrust is

$$p_g = \frac{A_s}{A_g}$$
$$= \frac{2(1.45)}{12(23)}$$
$$= 0.0105$$

Solving for the hoop force

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$$\begin{aligned} H &= \frac{wa^2}{b} \left(g - \frac{c}{(1-g^2)Q} \right) + \frac{wa^2}{2b} \left(\frac{2g^2-1}{Q} \right) \\ &= \frac{0.288(50)^2}{15} \left(0 - \frac{0.588}{(1-0)0.300} \right) + \frac{1.5(50)^2}{2(15)} \left(\frac{0-1}{0.300} \right) \\ &= -511 \text{ kips/ft } (- \text{ indicates tension}) \end{aligned}$$

The hoop force is in tension, and the steel must be furnished for the total force

$$\begin{aligned} A_s &= \frac{H}{f_s} \\ &= \frac{511}{14} \\ &= 36.5 \text{ sq in./ft} \end{aligned}$$

Assume #11 bars, then the required number of bars is

$$\begin{aligned} N &= \frac{A_s}{\text{bar area}} \\ &= \frac{36.5}{1.56} \\ &= 23.4 \quad \text{Say } \underline{24 \text{ bars}} \end{aligned}$$

These bars are to be furnished in the first 12 inches of the dome. It is necessary to form a haunch at the end of the major axis to provide concrete for the steel as shown in Fig. 2. The steel ratio for the hoop force will be

$$\begin{aligned} p_g &= \frac{A_s}{A_g} \\ &= \frac{24(1.56)}{6.4(12)(12)} \\ &= \underline{0.04} \end{aligned}$$

The total steel ratio at the haunch for the meridional force and the hoop force is

$$p_g = 0.04 + 0.0105 = \underline{0.0505}$$

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Fig. 2. Typical Haunch, 100-Ft Diameter, 15-Ft Rise

Calculations for Walls

ACI ST 57

Assume

$$\text{Liquid Depth} = H = 31 \text{ ft}$$

$$\text{Sp Gr} = 1.50$$

$$\begin{aligned} \text{Unit Load} &= w = 1.50 (62.4) \\ &= 93.5 \text{ psf} \end{aligned}$$

$$\begin{aligned} \text{Wall Height} &= \text{Liquid Depth} + 2 \text{ ft} \\ &= 33 \text{ ft} \end{aligned}$$

$$\text{Tank Radius} = 50 \text{ ft}$$

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Neglect earth pressure and steel liner.

Fig. 3. Wall with Hinged Base and Free Top

For the purpose of this report the wall was assumed free to rotate at the base with a free top. Actually there is restraint and moment at the top of the wall due to the dome connection and the base is partially restricted from rotation by the foundation. The requirement of this report is to determine approximately the material quantities of various circular tank sizes with elliptical domes to obtain a comparative cost of various tanks. To eliminate excessive calculations for steel discontinuity and reduction of concrete area at points of lesser stress, the steel and concrete requirements at only critical sections were used for material estimates. This will tend to

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increase the actual unit cost of a tank but the increase will be proportional to the tank size and therefore will not materially alter the results.

The ring tension is $T = CwHR$

Assume a wall thickness of 2'-9"; then from Table II, page 29⁷, the maximum value of C is + 0.549 at 0.6H when

$$\frac{H^2}{Dt} = \frac{(31)^2}{100(2.75)} = 3.5$$

$$T = 0.549(93.5)(31)(50) \\ = 79,500 \text{ lb/ft (tension)}$$

The steel requirements for ring tension are

$$A_s = \frac{T}{f_s} = \frac{79,500}{14,000} = 5.67 \text{ sq in./ft}$$

Try three rows of steel

$$A_s = \frac{5.67}{3} = 1.89 \text{ sq in./ft/row}$$

Select three rows of #9 at 6 in.

$$A_s = 3(2.00)$$

$$A_s = 6.00 \text{ sq in./ft furnished}$$

The steel ratio for ring tension becomes

$$p_g = \frac{A_s}{A_g} \\ = \frac{6.00}{12(33)} \\ = 0.0152$$

To determine the stress in the concrete use the following formula and assume the shrinkage coefficient 'C' of concrete as 0.0003.

$$f_c = \frac{CE A_s + T_{\max}}{A_c + nA_s}$$

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$$f_c = \frac{(0.0003)(30 \times 10^6)(6.00) + 79,500}{12(33) + 10(6.00)}$$

$$f_c = \underline{294 \text{ psi}}$$

Since 300 psi is considered allowable, the 33-inch wall thickness is sufficient.

The maximum moment in a vertical one-foot strip of wall is determined by the formula

$$M = GwH^3 \text{ ft-lb per ft}$$

When $\frac{H^2}{Dt} = 3.5$, the maximum moment occurs at $0.8H$.

$$\begin{aligned} M &= 0.0135 (93.5)(31)^3 \\ &= \underline{37.60 \text{ kip-ft/ft}} \end{aligned}$$

The steel requirements are determined by the formula

$$\begin{aligned} A_s &= \frac{M}{ad} \\ &= \frac{37.60}{1.44(30)} \\ &= \underline{0.87 \text{ sq in./ft}} \end{aligned}$$

Use #6 at 6 in. each face, then $A_s = 2(0.88) = \underline{1.76 \text{ sq in./ft.}}$

The steel ratio for maximum wall moment is

$$\begin{aligned} p_g &= \frac{A_s}{A_g} \\ &= \frac{1.76}{12(33)} \\ &= \underline{0.0044} \end{aligned}$$

The total steel ratio for the maximum ring tension and wall moment is

$$\begin{aligned} p &= 0.0152 + 0.0044 \\ &= \underline{0.0196} \end{aligned}$$

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Shear at the base of the cylindrical wall is determined from the formula

$$V = CwH^2$$

For a triangular load and a hinged base, and $\frac{H^2}{Dt} = 3.5$

$$V = 0.148 (93.5)(31)^2$$

$$= \underline{13.3 \text{ kips}}$$

The unit shearing stress is

$$v = \frac{V}{bjd}$$

$$= \frac{13,300}{12(7/8)(30)}$$

$$= \underline{42 \text{ psi}}$$

Since 75 pounds per square inch is considered allowable shearing stress¹, the 33-inch wall thickness is sufficient.

Calculations for Foundation

Analyze the foundation as a continuous wall footing with an allowable soil bearing of 6000 pounds per square foot.

Fig. 4. Typical Wall Footing

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Loads

$$\text{Meridional thrust from Dome} = 74.9^k$$

$$\text{Wall: } \left(\frac{33}{12}\right)(150)(33)\left(\frac{1}{1000}\right) = 13.6^k$$

$$\text{Ftg: Try 1} = 16' \text{ and } t = 2'-9'' = \underline{6.6^k}$$

$$\text{Total Soil Load} = 95.1^k$$

$$\text{Actual Soil Bearing} = \frac{95.1}{16} = \underline{5.93 \text{ kips}}$$

Therefore, the assumed footing length of 16 feet is sufficient.

The critical section (under concrete walls) for moment will occur at the face of the wall and the maximum bending moment is given by the equation

$$M = \frac{w(1 - a)^2}{8}$$

where 'w' is the net soil bearing in pounds per square foot

$$w = \frac{74.9 + 13.6}{16}$$

$$= \underline{5530 \text{ psf}}$$

$$M = \frac{5530(16 - 2.75)^2}{8}$$

$$= \underline{122 \text{ kip-ft}}$$

From Table 6, page 529¹³, when $f'_c = 3000$ pounds, $n = 10$, $f_s = 20,000$ psi,

the values of f'_c , k , j , p , K are

$$f'_c = 1000 \text{ psi}$$

$$K = 0.333$$

$$j = 0.889$$

$$p = 0.0083$$

$$K = 148$$

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Solving for the effective depth of the footing for moment requirements

$$d = \sqrt{\frac{M}{Kb}}$$

$$d = \sqrt{\frac{122(12)(1000)}{148(12)}}$$

$$= 28.6 \text{ in. Say } 29 \text{ in.}$$

The allowable shearing stress in the footing shall be the same as the allowable shearing stress in beams without web reinforcement¹³ which is $0.03 f'_c$, but not to exceed 75 psi for footings¹. For concrete walls, the critical section for shear occurs at 'd' distance from the face of the wall¹³ ('d' being the distance from the center of the reinforcing bars to the top of the footing at the edge of the wall). Using Fig. 4, page 36, and the above statement a formula may be derived to determine the footing depth for shear. The formula for maximum shear is

$$V = p(g - d)$$

where 'p' is the unit soil pressure and 'g' is the distance from the edge of the footing to the edge of the wall. Also the maximum allowable shear is

$$V = vbjd$$

Solving simultaneously, the effective depth is

$$p(g - d) = vbjd$$

$$d = \frac{pg}{vbj + p/12}$$

$$d = \frac{5.93(8 - 1.375)}{75(12)(7/8) + 5.93/12}$$

$$= \frac{39,300}{788 + 494}$$

$$= 30.6 \text{ in. Say } 31 \text{ in.}$$

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The 31-inch effective depth required for shear is greater than the 29-inch effective depth required for moment. Therefore, shear governs and an effective depth of 31 inches is used with a minimum coverage of three inches. The footing thickness ($t = 34$ inches) is one inch greater than the assumed thickness for which no revision is necessary. The steel requirements are satisfied by the formula

$$\begin{aligned} A_s &= \frac{M}{f_s j d} \\ &= \frac{122(12)}{20(0.889)(31)} \\ &= \underline{2.65 \text{ sq in./ft}} \end{aligned}$$

Use #9 at 4-1/2 in. ($A_s = 2.67 \text{ sq in./ft}$)

For the reinforcement in the top of the footing use #6 at 12 inches ($A_s = 0.44$ square inch per foot). To satisfy shrinkage and temperature stresses normal to the principal reinforcement use ¹

$$\begin{aligned} A_s &= 0.002 b d \\ &= 0.002 (12)(31) \\ &= \underline{0.744 \text{ sq in./ft}} \end{aligned}$$

Use #5 at 10 in. E.F. ($A_s = 0.74 \text{ sq in./ft}$)

The total steel ratio for all the steel in the footing is

$$\begin{aligned} p_g &= \frac{A_s}{A_g} \\ &= \frac{2.67 + 0.44 + 0.74}{12(34)} \\ &= \underline{0.00945} \end{aligned}$$

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Calculations for Floor

Assume a six-inch floor slab with one mat of #5 at 12 inches each way. The steel ratio is

$$\begin{aligned} p_g &= \frac{A_s}{A_g} \\ &= \frac{2(0.31)}{6(12)} \\ &= \underline{0.0086} \end{aligned}$$

Material Quantities and Costs

Dome

The surface area of an ellipse when revolved 180 degrees about its major axis and 360 degrees about its minor axis is

$$A = 2\pi a^2 c$$

The volume of concrete in an elliptical dome is equal to the surface area times the thickness of the dome. Therefore,

$$\begin{aligned} V &= 2\pi a^2 c t \\ \text{Elliptical Volume} &= V_1 = 2\pi a^2 c t \\ &= \frac{2\pi (50.96)^2 (0.588) (1.92)}{25} \\ &= \underline{736 \text{ cu yd}} \end{aligned}$$

$$\begin{aligned} \text{Haunch Volume} &= V_2 = \text{Triangular Area} \times \text{dome circumference} \\ &= \frac{2.2(6.8)\pi(51.0)(2)}{2(25)} \\ &= \underline{95.0 \text{ cu yd}} \end{aligned}$$

$$\begin{aligned} \text{Dome Volume} &= V = V_1 + V_2 \\ &= 736 + 95 \\ &= \underline{831 \text{ cu yd}} \end{aligned}$$

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Fig. 5. Approximate Dome Measurements for Volume Calculations

From Table 2, page 13, the unit cost of concrete in place for the dome is \$18.00 per cubic yard. The total cost of concrete for the dome is \$15,000.

The area of formwork is equal to the inside surface area of the dome.

$$\begin{aligned}\text{Formwork Area} &= 2\pi a^2 c \\ &= 2\pi(50)^2(0.588) \\ &= \underline{9230 \text{ sq ft}}\end{aligned}$$

From Table 2, page 13,

$$\begin{aligned}\text{Cost} &= \$2.00(9230) \\ &= \underline{\$18,460}\end{aligned}$$

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The required steel reinforcing at the crown is 1.03 percent and at the haunch is 5.05 percent. For an estimate of the steel required, assume that 5.05 percent steel is furnished between limits of 'a' and 0.80 'a' and the 1.03 percent is required between 0 and 0.80 'a' ('a' being the radius of the major axis plus one half the dome thickness). For the crown the required steel is 1.03 percent of the concrete volume.

$$\begin{aligned}\text{Concrete Volume} &= V = 2\pi a^2 ct \\ &= 2\pi(50.96)^2(0.328)(1.92)\left(\frac{1}{25}\right) \\ &= \underline{412 \text{ cu yd}}\end{aligned}$$

$$\begin{aligned}\text{Reinforcement Steel Weight} &= (\text{steel \%})(\text{steel unit wt})(\text{conc vol}) \\ &= \left(\frac{1.03}{100}\right)\left(\frac{490}{2000}\right)(412)(25) \\ &= \underline{25.6 \text{ tons}}\end{aligned}$$

For the haunch the required weight of reinforcing steel is

$$\begin{aligned}&= \left(\frac{5.05}{100}\right)\left(\frac{490}{2000}\right)(831 - 412)(25) \\ &= \underline{130 \text{ tons}}\end{aligned}$$

Weight of reinforcing steel in the dome is 156 tons and total cost at \$200.00 per ton (in place) is \$31,200.

Walls

The required concrete for the walls was determined by assuming a uniform wall thickness required at the critical section. In detail design a reduction of this quantity can be obtained by varying the wall thickness with relation to the wall stresses. A two-foot freeboard was assumed for these calculations.

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$$\begin{aligned}\text{Concrete Volume} &= \pi D h t \\ &= \pi (100 + \frac{33}{12}) (33) (\frac{33}{12}) (\frac{1}{25}) \\ &= \underline{1170 \text{ cu yd}} \\ \text{Cost} &= 1170 (\$18.00) \\ &= \underline{\$21,100}\end{aligned}$$

Wood formwork is required on the exterior wall and the steel liner forms the interior wall.

$$\begin{aligned}\text{Formwork Area} &= \pi D h \\ &= \pi [100 + \frac{2(33)}{12}] (33) \\ &= \underline{10,900 \text{ sq ft}} \\ \text{Cost} &= \$0.70 (10,900) \\ &= \underline{\$7650}\end{aligned}$$

Discontinuity of reinforcing steel is neglected and the maximum required steel ratio at the critical section is used for the entire wall.

$$\begin{aligned}\text{Reinforcement Steel Weight} &= \frac{490(0.0196)(1170)(25)}{2000} \\ &= \underline{141 \text{ tons}} \\ \text{Cost} &= 141 (\$200.00) \\ &= \underline{\$28,200}\end{aligned}$$

Foundation

The volume of concrete for the foundation is the cross-sectional area times the diameter of the center of gravity for the footing.

$$\begin{aligned}\text{Concrete Volume} &= (16 \times \frac{34}{12}) (\pi) (100 + \frac{33}{12}) (\frac{1}{25}) \\ &= \underline{585 \text{ cu yd}}\end{aligned}$$

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$$\begin{aligned}\text{Cost} &= 585(\$15.00) \\ &= \underline{\$8780}\end{aligned}$$

It is assumed the foundation and floor are monolithic. Therefore, only the exterior face of the foundation will require formwork, and the floor will not require formwork.

$$\begin{aligned}\text{Formwork Area} &= \pi Dh \\ &= \pi(100 + \frac{33}{12} + 16)(\frac{34}{12}) \\ &= \underline{1060 \text{ sq ft}}\end{aligned}$$

$$\begin{aligned}\text{Cost} &= 1060(\$1.00) \\ &= \underline{\$1060}\end{aligned}$$

$$\begin{aligned}\text{Reinforcement Steel Weight} &= \frac{585(25)(490)(0.00945)}{2000} \\ &= \underline{34 \text{ tons}} \\ &= 34(\$200.00) \\ &= \underline{\$6800}\end{aligned}$$

Floor

The six-inch floor slab is assumed to have a diameter equal to that of the inside face of the footing.

$$\begin{aligned}\text{Concrete Volume} &= \frac{\pi D^2 t}{4} \\ &= \frac{\pi \sqrt{118.75 - 2(16)}^2 (0.5)}{4(25)} \\ &= \underline{118 \text{ cu yd}}\end{aligned}$$

$$\begin{aligned}\text{Cost} &= 118(\$15.00) \\ &= \underline{\$1770}\end{aligned}$$

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Reinforcement Steel Weight = $\frac{(0.0086)(490)(25)(118)}{2000}$
= 6.2 tons
Cost = 6.2(\$200.00)
= \$1240

Steel Liner

A 3/8-inch steel plate liner is used to line the floor and the wall of the tank for corrosion resistance. The weight of the liner is equal to its unit weight times the surface area of the floor and the wall times the plate thickness.

$$\begin{aligned}\text{Steel Liner Weight} &= \gamma \left(\frac{\pi D^2 t}{4} + \pi D t h \right) \\ &= \gamma \pi D t \left(\frac{D}{4} + h \right) \\ &= \frac{490 \pi 100(3)}{2000(b)(12)} \left(\frac{100}{4} + 33 \right) \\ &= \underline{139 \text{ tons}} \\ \text{Cost} &= 139 (\$310.00) \\ &= \underline{\$43,200}\end{aligned}$$

Earthwork

For an approximate calculation of earthwork quantities, a corner tank excavation of a typical tank farm is assumed with earth sloped at 1-1/4 to 1 on the two adjacent sides. The other two adjacent sides are vertical because they are adjacent to other tanks. Fig. 6, page 46, illustrates the excavation dimensions for a 100-foot diameter tank with a 33-foot wall height.

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Fig. 6. Earthwork Dimensions, 100-Ft Diameter, 33-Ft Wall

$$\begin{aligned}\text{Excavation Depth} = H &= 2.84 + 33.0 + 15.0 + 1.92 + 7.0 \\ &= \underline{59.76 \text{ ft}}\end{aligned}$$

$$\begin{aligned}d &= 1.25(59.76) \\ &= \underline{74.6 \text{ ft}}\end{aligned}$$

$$\begin{aligned}\text{Base Dimension} &= 118.75 + 2(10) \\ &= \underline{138.75 \text{ ft ea way}}\end{aligned}$$

$$\begin{aligned}\text{Top Dimension} &= 138.75 + d \\ &= 138.75 + 74.6 \\ &= \underline{213.35 \text{ ft ea way}}\end{aligned}$$

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Excavation Volume = $\frac{(\text{Sum of the End Areas})}{2(27)} (\text{Height})$

$$= \frac{[(138.75)^2 + (213.4)^2]}{2} \times \frac{59.76}{27}$$
$$= \underline{71,500 \text{ cu yd}}$$

$$\text{Cost} = 71,500(\$0.30) = \underline{\$21,450}$$

$$\begin{aligned} \text{Backfill Volume} &= \text{Exc Vol} - \text{Displaced Tank Vol} \\ &= 71,500 - (\text{Ftg Vol} + \text{Wall Vol} + \text{Dome Vol}) \\ &= 71,500 - \left[\frac{\pi D^2 t}{4} + \frac{\pi D^2 h}{4} + \frac{2abc\pi}{3} \right] \\ &= 71,500 - \left[\frac{\pi(118.75)^2 2.84}{4(27)} + \frac{\pi(105.5)^2 33}{4(27)} + \frac{2\pi 51.92(51.92)(16.92)}{3(27)} \right] \\ &= 71,500 - (1150 + 10,500 + 3540) \\ &= \underline{55,310 \text{ cu yd}} \\ \text{Cost} &= 55,310(\$0.36) \\ &= \underline{\$19,900} \end{aligned}$$

The depth of storage liquid is governed by the invert elevation of the effluent pipe which is assumed to be 31 feet above the bottom of the tank. Liquid storage capacity is

$$V = \frac{\pi D^2 h}{4}$$
$$V = \frac{\pi(100)^2(31)}{4(0.1337)}$$
$$= \underline{1,820,000 \text{ gallons}}$$

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Tank Cost

Dome

Concrete	\$ 15,000
Formwork	18,460
Re-steel	<u>31,200</u>

Dome Cost - - - \$ 64,660

Walls

Concrete	\$ 21,100
Formwork	7,650
Re-steel	<u>28,200</u>

Wall Cost - - - \$ 56,950

Foundation

Concrete	\$ 8,780
Formwork	1,060
Re-steel	<u>6,800</u>

Foundation Cost - \$ 16,640

Floor

Concrete	\$ 1,770
Re-steel	<u>1,240</u>

Floor Cost - - - \$ 3,010

Steel Liner - - - - - \$ 43,200

Earthwork

Excavation	\$ 21,450
Backfill	<u>19,900</u>

Earthwork Cost - \$ 41,350

Total Tank Cost - - - - - \$ 225,810

Unit Tank Cost = $\frac{225,810}{1,820,000}$ = \$0.124/gallon

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UNIT COST - CAPACITY CURVE

Underground Radioactive Waste Storage Facility
Circular Tanks with Elliptical Domes

Process Engineering Sub-Section
Design Section
Engineering Department
Hanford Atomic Products Operation
General Electric Company

February 11, 1955

H. W. Stivers

TANK DATA

(c)

Dia = 85.5'
Depth = 36.8'
h/d = 0.431
Cap. = 1.58 MG
Cost = \$0.120/gal

(d)

Dia = 100'
Liq Depth = 39.8'
h/d = 0.398
Cap. = 2.34 MG
Cost = \$0.120/gal

(e)

Dia = 130'
Liq Depth = 45.6'
h/d = 0.350
Cap. = 4.53 MG
Cost = \$0.120/gal

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Tank Capacity (MG)

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Fig. 8. Optimum $\frac{h}{d}$ Ratio - Diameter Curve

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Fig. 9. Optimum Wall Height - Diameter Curve

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Fig. 10. Optimum Tank Capacity - Diameter Curve

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Fig. 11. Optimum Tank Capacity - Cost Curve

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OPTIMUM TANK DATA

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Fig. 12. Optimum Tank Data

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13: 7 1/2"

615.33 FOR TRAMP 104, 109, 107, 110
612.33 FOR TRAMP 106, 108, 100, 111
616.33 FOR TRAMP 103, 105, 102, 112

FOOTING REINFORCING
TOTAL OF 12 - 1" BARS
CIRCUMFERENTIAL
1/2" @ 4'-0" BARS @ 24 CTS.
3/4" @ 6'-0" BARS @ 8 CTS.
3/4" @ 9'-2" BARS @ 2 CTS.

1. LIQUID IS STORED AT ATMOSPHERIC PRESSURE
2. SPECIFIC GRAVITY OF LIQUID IS 1.85
3. LIQUID WILL HAVE A PHOSPHOR OF 100
4. TANK WILL BE WORKED SLIGHTLY OVER A PERIOD OF AT LEAST 30 DAYS BEFORE BEING PUT INTO SERVICE. WHEN PUT INTO SERVICE, THE RATE OF FILLING WILL BE SUCH THAT THERE WILL BE FILLED DURING A PERIOD OF ONE MONTH.
5. BACKFILL MATERIAL - SAND AND GRAVEL. UNIT WEIGHT: 100 LBS PER CU YD IN PLACE. APPROX. ANGLE OF REPOSE 26°
6. NO HYDRAULIC HEAD WHERE TANK FLOOR.
7. STEEL LINING OF TANK SHALL BE ABSOLUTELY LIQUID TIGHT
8. CONCRETE WALL SURROUNDING TANK IS ASSUMED TO RESIST FULL HYDROSTATIC PRESSURE OF LIQUID IN TANK.
9. BEARING VALUE OF SOIL IS 5000 LBS PER SQ. FT. AT FINISH ELEVATION
10. DOME ROOF SHALL RESIST WEIGHT OF A 3500 LB. CONCENTRATED LOAD APPLIED AT TOP OF ROOF. FILL IN ANY LOCATION DURING BACKFILLING OPERATIONS.
11. SPECIFICATIONS:
STRUCTURAL STEEL TUBES: A.M.S. STD. SPEC. FOR CLEANED STEEL WATER TANKS, STORAGE AND RESERVOIRS, AND AMERICAN WELDING SOCIETY SPECIFICATION'S
12. MATERIALS:
CONCRETE: DESIGN MIX FOR $f'_c = 3000 \pm 50$ PSI MINIMUM SIZE AGGREGATES
REINFORCING STEEL: ASTM SPEC. 205, A57-39 INTERMEDIATE GRADE DEFORMED BARS
STRUCTURAL STEEL: ASTM SPEC. A7-39 STEEL PLATE MUST BE FULL OF WATER WHEN WALL CONCRETE IS CURED.

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CLASSIFICATION
NONE

HANFORD ENGINEER WORKS

COMPOSITE TANK

241-BX

APPROVED			Date
BY	EMPL.	DATE	
			DRAWN BY
			CHECKED
			ELDS. No.
			DWG. No.

FIG. 13

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Fig. 13. Dwg. No. 602

Tank Farms: 241-T, 241-U, 241-B, 241-C, 241-BX

Number Constructed: 60 Tanks

Nominal Capacity: 533,000 Gallons

Inside Diameter: 75'-0"

Steel Tank Depth: 19'-0 $\frac{1}{4}$ "

Concrete Tank Height: 33'-1"

Tank Characteristics: Composite structure consisting of an open-top, cylindrical, all-welded, carbon steel tank with dish-shaped bottom connected to shell plates by a long radius "knuckle" or transitional section, and all enveloped by a cylindrical conventional reinforced concrete tank with an elliptical dome. Membrane waterproofing with protective cement grout or shot-crete applied on the tank base, steel tank shell and concrete dome.

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FIG. 14

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Fig. 14. Dwg. No. H-2-808

Tank Farms: 241-TX, 241-BY, 241-S, 241-TY

Number Constructed: 48 Tanks

Nominal Capacity: 758,000 Gallons

Inside Diameter: 75'-0"

Steel Tank Depth: 24'-11 $\frac{1}{4}$ "

Concrete Tank Height: 39'-1"

Tank Characteristics: Composite structure consisting of an open-top, cylindrical, all-welded, carbon steel tank with dish-shaped bottom connected to shell plates by a long radius "knuckle" or transitional section and entirely enveloped by a cylindrical conventional reinforced concrete tank with an elliptical dome. Membrane waterproofing with protective cement grout or shot-crete applied on tank base, steel tank shell and concrete dome.

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TACK WELD TO REINFT. STL
NOTE: PLATE SECTIONS SHALL BE JOINED BY BUTT WELDING WITH 100% PENETRATION

AS BUILT
N-2-3550

SCALE 3/4\"/>

AS BUILT
N-2-3550

ANGLE TANK LINER
WELD DET. 1770

SCALE 3/4\"/>

AS BUILT
N-2-3550

STEEL PLATE LINER BEARING

SHELL PLATE: HORIZONTAL SEAMS TO BE DOUBLE BEVEL GROOVE WELD JOINTS
VERTICAL SEAMS TO BE DOUBLE BEVEL GROOVE WELD JOINTS
BOTTOM PLATE: ALL SEAMS TO BE DOUBLE BEVEL GROOVE WELD JOINTS
JOINTS FOR COILING SHALL BE DOUBLE BEVEL GROOVE WELD JOINTS
SHALL BE SINGLE BEVEL GROOVE WELD JOINTS AGAINST A BACK-UP STRIP. ALL BOTTOM PLATE WELDING SHALL BE BY SUBMERGED ARC WELDING PROCESS.
ALL JOINTS SHALL HAVE 100% PENETRATION OF WELD METAL.

NOTE:
CONTRACTOR SHALL SUBMIT SHOP DETAILS OF ALL STEEL WORK FOR APPROVAL BY THE COMMISSION PRIOR TO FABRICATION.

SECTION
N-2-3550

SCALE 3/4\"/>

AS BUILT

APPROVED

NO.	DESCRIPTION	DATE	BY	CHECKED
1	DESIGN	10-1-58	J. E. HARRIS	J. E. HARRIS
2	CONSTRUCTION	10-1-58	J. E. HARRIS	J. E. HARRIS
3	AS BUILT	10-1-58	J. E. HARRIS	J. E. HARRIS
4	REVISION	10-1-58	J. E. HARRIS	J. E. HARRIS
5	REVISION	10-1-58	J. E. HARRIS	J. E. HARRIS
6	REVISION	10-1-58	J. E. HARRIS	J. E. HARRIS
7	REVISION	10-1-58	J. E. HARRIS	J. E. HARRIS
8	REVISION	10-1-58	J. E. HARRIS	J. E. HARRIS
9	REVISION	10-1-58	J. E. HARRIS	J. E. HARRIS
10	REVISION	10-1-58	J. E. HARRIS	J. E. HARRIS

75 FT. STORAGE TANKS
COMPOSITE SECTION
WASTE DISPOSAL FACILITY
2-1-58

ADDITIONAL WASTE DISPOSAL FACILITY
2-1-58

FIG. 15

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Fig. 15. Dwg. No. H-2-39511

Tank Farms: 241-SX
Number Constructed: 15 Tanks
Nominal Capacity: 1,000,000 Gallons
Inside Diameter: 75'-0"
Steel Tank Depth: 32'-3-7/8"
Concrete Tank Height: 46'-5-5/8"

Tank Characteristics: Composite structure consisting of an open-top, cylindrical, all-welded, carbon steel tank with a dish-shaped bottom tee-connected to the shell plates and entirely enveloped by a cylindrical conventional reinforced concrete tank with an elliptical dome. Membrane waterproofing with protective cement grout applied on tank base.

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Fig. 16. Dwg. No. H-2-55911

Tank Farms: 241-A
Number Constructed: 6 Tanks
Nominal Capacity: 1,000,000 Gallons
Inside Diameter: 75'-0"
Steel Tank Depth: 32'-4"
Concrete Tank Height: 47'-9-3/4"

Tank Characteristics: Composite structure consisting of an open-top, cylindrical, all-welded carbon steel tank with a flat bottom tee-connected to the shell plates and entirely enveloped by a cylindrical conventional reinforced concrete tank with an elliptical dome. Membrane waterproofing with protective cement grout applied on tank base and membrane waterproofing without protective shot-crete applied on concrete dome.

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DATE

FILMED

7/19/94

END