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UNITED STATES ATOMIC ENERGY COMMISSION

STRUCTURAL EVALUATION OF UNDERGROUND WASTE STORAGE TANKS

By
Edgar F. Smith

June 23, 1955

Hanford Atomic Products Operation
Richland, Washington

Technical Information Extension, Oak Ridge, Tennessee

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STRUCTURAL EVALUATION

UNDERGROUND WASTE STORAGE TANKS

By

Edgar F. Smith

Architectural & Civil Design
Design Engineering Sub-Section
Design Section, Engineering Department

June 23, 1955

Work performed under Contract No. W-31-109-Eng-52

Hanford Atomic Products Operation
GENERAL ELECTRIC COMPANY
Richland, Washington

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STRUCTURAL EVALUATION
UNDERGROUND WASTE STORAGE TANKS

1. INTRODUCTION

Much of the liquid waste from the separation processes at HAPO has been stored in underground tanks which have been constructed in multiple-tank farms at various intervals since the original construction in 1943-44. Over the years process changes and improvements have so changed the contents and character of the wastes that a structural re-evaluation of the older tank farms is in order to determine their suitability to contain present wastes. The present trend of higher specific gravities and temperatures in liquid wastes, in addition to vapor pressures, impose additional loads on our tank structures which were not contemplated at the time the tanks were constructed.

2. OBJECTIVE

It is intended to set forth limiting values of internal vapor pressure and effective liquid specific gravity which will permit the maximum utilization of the existing underground waste storage capacity. These limits are predicated on using increased unit stresses and taking advantage of other phenomena, and will serve as a yardstick of tank operation so that the structural integrity of the tanks will not be violated. It is also intended to describe briefly the elements which could contribute to ultimate failure.

3. SUMMARY & CONCLUSIONS

Present waste storage tanks may be divided into four different design types and each of these types has been analyzed in accordance generally with the rational approach of the Portland Cement Association for circular concrete tanks. Certain basic assumptions were made, which are believed to be reasonable, relating to load conditions, temperature effects and increased allowable unit stresses. The reinforcing steel was permitted to approach a tensile stress of 20,000 psi for sustained hydrostatic pressures and 27,000 psi when transient vapor pressures were imposed in addition to the liquid loading. This compares with a normal design stress of 14,000 psi. On the basis of these assumptions, limiting (or allowable) values of effective specific gravity and vapor pressure within the tanks have been estimated and plotted graphically.

Briefly, the maximum effective specific gravity and simultaneous allowable internal vapor pressure for liquid wastes at elevated temperatures in each of the present tank farm types can be summarized as follows:

Tank Farm Type	Maximum Specific Gravity	Simultaneous Vapor Pressure
241-T, U, B, C, BX	1.9	2.5 psig
241-S, BY, TX, TY	1.2	1.8
241-SX	1.5	4.8
241-A	2.2	6.9

When specific gravity is reduced an increased vapor pressure would be permitted up to a limit of 10 psig beyond which the dome of the tank would be in jeopardy. Although actual structural collapse due to hydrostatic head is difficult to conceive, it is believed that the limiting values presented cannot be exceeded without endangering the integrity of the structure from the standpoint of splitting open and permitting leakage through wide cracks.

4. HISTORICAL BACKGROUND

In the 200 Areas there are 129 underground waste storage tanks grouped in 11 tank farm units of 6 to 18 tanks per farm, having an aggregate capacity of approximately 90 million gallons. The following table summarizes this information for each farm and indicates the date of construction:

Farm	Tanks/ farm	Capacity/tank gallons	Capacity/farm gallons	Year Constructed
T,U,B,C	12	533,000	6,400,000	1943-44
BX	12	533,000	6,400,000	1946-47
TX	18	758,000	13,600,000	1947-48
BY	12	758,000	9,100,000	1948-49
S	12	758,000	9,100,000	1950-51
TY	6	758,000	4,500,000	1951-52
SX	15	1,000,000	15,000,000	1953-54
A	6	1,000,000	6,000,000	1954-55

Although differing in minor details, the general design features of all tanks are similar – vertical reinforced concrete cylindrical tank with an elliptical concrete dome, 75 feet in diameter, with a mild carbon steel plate liner. In an effort to effect more economical storage the tanks were made progressively deeper; the original T,U,B,C tanks had a depth of 17 feet, the liquid depth in SX and A tanks approximated 30 feet. Detail drawings of the composite tank section are included as

Appendix C for all tank types including the proposed SY tank which has a capacity of 1.25 million gallons.

Some of the factors which can influence economical tank proportions are set forth in HW-34860[1], wherein Stivers found that storage cost per gallon could be critically affected by tank depth in tanks of relatively small diameter. For larger diameters the depth was much less important. It will be noted that the SX and A tank depths are slightly less and the proposed SY tank depth is somewhat greater than the predicted optimum.

The design criteria for these tanks did not envision any serious temperature problem, nor was any consideration given to possible internal vapor pressure in the tanks. The liquid waste specific gravity used as a design basis increased from 1.2 for the original tanks to 2.0 for the latest tank farm, as follows:

Tank Type	Specific Gravity
241-T, U, B, C, BX	1.2
241-S, BY, TX, TY	1.25
241-SX	1.35
241-A	2.0

(1) Stivers, H.W. Study To Determine the Economical Tank Size for Radioactive Waste Disposal. HW-34860, February 11, 1955 (Official Use Only)

5. BASIC TANK DESIGN CONCEPTS

Live Load Considerations

It is not proposed to enter into a detailed dissertation concerning the theory of concrete tank design, but rather to point out in a general manner the types of structural action that are encountered in the problem. Unlike tanks with relatively flexible walls, a concrete tank wall is fairly rigid and its action under a hydrostatic head is not limited to a simple concept of lateral forces producing a ring tension stress in the shell. Because the wall is rigid, it can also be considered as a series of vertical beams which help resist the hydrostatic load by beam action.[2] [3] [4]. In general, tank geometry determines the proportional amount of liquid load which is resisted by ring tension and beam action.

The type of restraint at the supports plays an important role in determining the amount of ring tension and wall bending moment (beam action) to be expected in any given tank section. Figure 1 is intended to show the relative magnitude of these forces and an exaggerated picture of the deformation shapes for different types of restraint. The representation is typical for an open tank (free top) with a hydrostatic or triangular loading; internal vapor pressure and restrained top conditions merely alter the shape of the curves -- the

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(2) Portland Cement Association, Circular Concrete Tanks Without Prestressing, Bulletin ST-57.

(3) Gray, W. S. Reinforced Concrete Reservoirs and Tanks. London: Concrete Publications Limited, 2nd Edition, 1942

(4) United States Navy Department, Bureau of Yards and Docks. Standards of Design for Concrete, NavDocsSpec3Yb, Section 12.

RING TENSION

WALL MOVEMENT

STRESS CURVES

HINGED BASE

FIXED BASE

DEFORMATION

FIGURE I

fundamental concept remains unchanged. It is important only to recognize that when the wall cannot deflect freely under a triangular liquid loading the usual forces to be expected by such a loading are altered considerably, and maximum ring tension occurs not at the bottom where the hydrostatic pressure is greatest but at some other point in the wall.

Concrete Characteristics

In addition to the stress produced by the live loading, there are tensile stresses produced within the concrete itself which are independent of other loads. When concrete hardens it has a tendency to shrink or become smaller, but the steel reinforcing bars in the concrete will not permit this shrinkage to take place. In effect, there is an additional tensile force exerted on the concrete because of the shrinkage characteristic; it is this type of action in concrete that produces what are commonly called "shrinkage cracks". On the other hand, most materials will creep or flow plastically under the action of an external force, and concrete follows this same pattern [(5)]. In our case this phenomenon helps the situation since it is believed that plastic flow under load over a period of time will tend to eliminate the shrinkage forces. This means that if we load the tank initially with a hydrostatic load of relatively low specific gravity we should be able, after plastic flow has been accomplished, to increase the specific gravity in such amount that the additional liquid load produces a stress equal to the shrinkage stress

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(5) Magnel, G. Prestressed Concrete. London: Concrete Publications Ltd., 2nd Edition, 1950

which has been eliminated. This philosophy fits in well with the principle of waste self-concentration. Further than that, it can be postulated that the shrinkage stress which has been relieved by plastic flow can be added to the allowable steel stress (taking due account of the difference in the moduli of elasticity)without increasing the tendency of the concrete to crack. This increase would amount to an additional 1500 psi that could be allowed in the reinforcing steel. The rate of plastic flow is rapid at first and then falls off exponentially until at the end of 6 months most of the plastic flow has taken place and at 12 months essentially all has taken place. The chief value of using the plastic flow philosophy is to drastically reduce the amount of concrete required in the walls. It has but little effect on the amount of reinforcing, and still less effect when rating the capacity of an existing tank at higher than normal design unit stresses.

Temperature Effects

Still another factor to be considered is the elevated temperatures which are associated with the waste material to be stored in the tank. These temperatures produce a thermal gradient through the tank wall section with higher temperatures on the inside than on the outside of the wall which indicates that the inner wall is trying to expand more than the outer wall surface. This action tends to produce cracks in the outer tank surface, and steel reinforcing is provided to reduce this tendency. In addition to the temperature effect within the concrete itself the steel plate liner provided in the tank must also be considered. Not only is the steel liner at a higher temperature than the concrete, but it also has a somewhat higher thermal expansion coefficient. The overall temperature in the tank is trying to expand the steel but since it is retained by the concrete shell the result is to increase the tensile forces already imposed on the concrete. The structure as a whole will not experience a simple uniform expansion in all directions under the action of increased temperatures. Neither the dome nor the outer rim of the tank foundation is as hot as the walls. Therefore, the tank is restrained in effect at these points, causing the wall and the tank bottom centers to bulge.

Allowable Unit Stresses

In order to obtain a tank design for given conditions, unit stress values for the materials of construction must be assigned and the tank section proportioned accordingly. The following unit tensile stresses are those recommended by the Portland Cement Association(2):

Reinforcing Steel Stress:	14,000 psi (for ring tension) 20,000 psi (all other)
Concrete Stress:	300 psi (for 3000 psi concrete)

These unit stresses are not unduly conservative; the Navy Department(4) recommends a steel stress of 12,000 psi for ring tension and a concrete

(2) Portland Cement Association, ST-57

(4) US Navy Department, NavDocsSpec3Yb, pg 95

stress of 200 psi for 2500 psi concrete.

An allowable steel stress in ring tension as low as 10,000 psi has been used by some designers on the basis that low stress is necessary to minimize the width of cracks that may develop in the wall. And it cannot be disputed that lower steel stress will result in a smaller crack when considered by itself. However, lower steel stress results in larger steel areas that must be provided to withstand any given ring tension, and this in turn will increase the shrinkage stress in the concrete which helps to produce cracks. Furthermore, the bonding force between the steel and concrete is important. Consider two cases of crack formation. If the bond resistance is high in one case and low in the other, we can expect smaller cracks with the higher bond resistance. A lower steel stress may well require larger size bars, and since the bond resistance is a function of the amount of surface contact between the concrete and steel, this will result in less surface contact area per unit of cross sectional steel area provided and, therefore, less bond resistance which can produce larger cracks.

The value of 14,000 psi, then, can be considered a compromise between the two factors, one indicating high stress and the other low stress. A concrete tensile stress limitation of 200 psi has been used by some designers, particularly in England, but it is believed that the American practice of using 300 psi is reasonably conservative when shrinkage forces are included and ring tension is determined by a carefully conducted analysis.

Roof Design

No attempt will be made here to elaborate on the basic principles of dome roof design. Instead, the reader is referred to Appendix A wherein is reproduced an unplublished memorandum dated April 1, 1954, entitled, Basis of Dome Design - Purex Tank Farm. This memorandum was prepared for another purpose, but is believed more than adequate for this report. Although it deals specifically with the 241-A tank dome, the domes on all tanks are similar, the chief difference being that a redistribution of reinforcing steel was made in the A tank dome (and some slight overall reduction in tonnage) to reduce the heavy concentration of steel at the periphery.

Soil Pressures

Since the tanks are buried in the ground with a minimum average of 6 to 8 feet of earth cover over the top of the dome, soil pressure effects must be reviewed. Soil pressures are considered when proportioning the wall to withstand the external soil load when the tank is empty. However, it is common practice to ignore the effect of soil pressure assistance in supporting any part of the hydrostatic loading. The literature is quite specific in setting forth this philosophy. Quoting from the U.S. Navy Department, Bureau of Yards and Docks, NavDocsSpec3yb, November 15, 1929, "Standards of Design for Concrete", we find:

> "12-02 (a) -----Where the tank is buried in the earth no allowance is made for the reduction of internal pressure on account

of the external earth pressure. This is because a relatively small deflection will permit undesirable cracking.......

"12-02 (g) External pressure -- Where the tank is buried in the earth, provision should be made for the external earth pressure when the tank is empty...... When the tank is full, the internal (pressure) may be considered as compensating (for the external pressure". As stated above, no reliance should be placed on the external earth pressure in designing the tank for internal fluid pressure......."

Although the consideration of earth pressure is not considered good practice when designing for internal loadings, it does, nevertheless, exist to some degree, except perhaps after a tank has expanded against the soil under the influence of increasing temperatures and then later shrunk away from the soil during periods of decreasing temperature. Recognition of this soil pressure has entered into the justification of higher unit stresses used to evaluate our existing tanks, as explained later in section 6 of this report.

6. APPLICATION OF BASIC CONCEPTS

Some Structural Unknowns

Lest it be considered that an underground tank of this nature lends itself to an exact analysis, some of the unknowns are listed which must be considered in the design. More especially must they be considered when tanks are re-evaluated at unit stresses higher than customarily used in an original design problem.

There is the problem of construction errors. It is possible that steel bars have been left out of concrete forms. Weather conditions may require compromises to be made. Even the substitution of materials without the designer's approval is not uncommon. Therefore, there can be no real assurance that the structure has been constructed exactly according to plans and specifications.

Figure 1 on page 10 indicates the influence on stress distribution of the type of restraint that is assumed between the base and the walls. It probably can be said without fear of contradiction that the base is not free to slide, but whether it is hinged or fixed cannot be determined precisely. Probably some intermediate condition between these extremes more closely represents the actual restraint picture.

The value of effective specific gravity (as distinguished from average specific gravity) causing hydrostatic loading on the wall is uncertain. Average weight per unit of volume does not necessarily reflect the true value. Heavy particles in suspension may have but little effect on the lateral hydrostatic pressure produced by the liquid. The settling of such solids to the bottom of the tank would have a similar effect in reducing the apparent specific gravity acting upon the wall.

When temperature is considered, many indeterminants come to mind. The actual magnitude of temperature differentials in the tank section is unknown. Likewise, little is known of the possible effects of thermal expansion and/or contraction of the backfill surrounding the tank. The heavy concrete ring at the base and the dome may remain relatively cool and this tends to restrain the tank at those points. In all probability this results in a differential movement (or bulging) of the wall and base slab with respect to the rest of the section, thereby contributing to secondary stresses which cannot be calculated precisely. High temperatures near the bottom of the tank in themselves could conceivably so crack the concrete base that major leakage might be detected were it not for the steel liner. It is obviously impossible to observe this condition under operating conditions.

The passive resistance of soil is a direct function of the applied load. In our case such load can be applied by the tendency of the tank to deform outwardly, but little is actually known of the magnitude of this phenomenon. Soil test tables at 100-K Area indicated a deflection or settlement of 1/16 inch under a load of 4 tons psf, and the soil returned to its original position when the load was removed. When the tank walls expand against the soil during thermal expansion it is probable that this elastic range of the soil has been exceeded, so that when subsequent contraction takes place as a result of lower temperature, the wall actually pulls away from the soil eliminating any semblance of soil pressure on the wall.

There is little data which can be applied to the problem of whether or not shrinkage stress can be eliminated in its entirety by plastic flow.

These are some of the conditions that must be evaluated, but which cannot be calculated precisely. Rational approaches have been made but they cannot be considered entirely adequate when we attempt to increase allowable unit stresses (reduce the safety factor). An attempt was made to predict the temperature differential to be expected[6], but again, the result is based on assumptions that may not be entirely valid. A more exact determination of these several factors would require test data which is not now available. Such a test program would include the widespread usage of such instrumentation as thermocouples, strain gages and pressure cells. At this time, it is only by realizing the limitation of the calculations and attempting to evaluate the unknowns that higher unit stresses can be applied to underground tanks. The term 'estimate' has been defined as "a carefully considered computation of some quantity, the exact magnitude of which cannot be determined". Any attempt to determine the magnitude of unit stresses in the tank can, at best, be said to be an estimate, taking full cognizance of the definition.

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(6) Cook, M. W. Temperature Drop in Waste Tank Walls. HW-36403-RD, April 25, 1955 (Secret)

Failure Considerations

Any limiting value regarding structural strength must imply some indication of impending failure. Failure can be defined in many ways, but we will limit the definition to actual structural collapse of the tank structure. Since no vacuum forces are present the tank cannot collapse inwardly. Too high a vapor pressure and/or hydrostatic head conceivably could expand the concrete to the point where cracking would render the wall ineffective to contain liquid. But it is difficult to envision any circumstance which could expand the wall to the point where the wall is sheared from the dome thereby dropping the dome and the earth above it into the tank. In all cases the liquid level is more than 20 feet below the ground surface, and although the passive resistance of the soil will probably not prevent cracking of the concrete walls at these depths it would exert tremendous external pressures against the tank tending to hold the walls substantially in place. As long as the steel liner remains tight the liquid contents cannot leak to the ground, and actual structural collapse of the tank is very remote.

The dome, however, could very well be lifted from the tank wall if the internal vapor pressure became too great. Since very little resistance (anchorage) is exerted by the wall to hold the dome in place against an upward force, it has been assumed that the maximum vapor pressure under any circumstance should not exceed the weight of the dome and the earth above it. The data in Appendix A indicates 10 psig to be the limiting value; and internal pressures in excess of 10 psig should not be permitted without realizing that the structural integrity of the dome may be in jeopardy.

The aspect of failure from an external source, such as earthquake or enemy bomb action, should not be overlooked. The tank meets the earthquake requirements of the Uniform Building Code and can be considered earthquake resistant. Although the dome is a good shape structurally to resist bomb blast, the tank cannot be considered immune to a direct hit particularly if nuclear weapons were employed.

Mechanical Strength

Excluding all consideration of higher temperatures and the presence of vapor pressure, we have seen that the original design criteria regarding specific gravity was low when compared to present wastes. It follows then, that unless allowable unit stresses are increased lower allowable specific gravities will result from the increased temperatures and still lower gravities if pressure is to be tolerated in the tank. We have also briefly enumerated the unknown factors, some of which help the case and others do not. Certainly the temperature problem cannot be ignored. However, soil pressures will definitely help the case and any uncertainties as to effective specific gravity versus actual specific gravity can result only in a lower value which reduces the hydrostatic head.

Although the application of temperature to the problem is based on rational methods, it is believed that the results are indicative and can be applied. For the purpose of the analysis a temperature differential of 23° F per foot of wall thickness was used(6). It is realized that

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(6) Cook, HW-36403-RD

this value is based on boundary conditions that are probably more severe than will ever be experienced in any tank, but it is not considered prudent to use lesser values.

It is a well recognized practice in certain structural fields to assume higher unit stresses when the capacity of an existing structure is to be rated. To withstand long term hydrostatic head an allowable unit stress of 20,000 psi was used in the analysis for ring tension reinforcing steel which is 43% greater than the customary design value of 14,000 psi. This stress may be rationalized on the basis that it is no more than the usual design stress for steel in other types of concrete structures and it is within the percentage increase by certain codes in rating existing structures(7). It is recognized that any soil pressure load that is present will reduce the unit stress, and, in addition, the passive resistance of the soil will resist ultimate collapse of the wall.

The transient nature of the vapor pressure within the tank is somewhat analogous to wind and earthquake loadings in other structures; and all building codes, including the Uniform Building Code, under which we operate for applicable structures, permit an increase in unit stress when such loadings are applied. Therefore, a further increase was taken and a permissible unit stress of 27,000 psi in the steel was used when transient vapor pressure loading was included with the hydrostatic load

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(7) American Railway Engineering Association. Specifications for the Design and Construction of Steel Railway Bridges & Concrete Railway Structures

already existing. It is admitted that there is no real basis for taking a double increase in unit stress for ring tension values resulting in a final stress almost double the 14,000 psi which would be used in an original design problem. A stress of 27,000 psi is very close to the estimated yield point (approximately 30,000 psi) of the steel reinforcing. However, it is opinionated that this stress will never actually be reached since the vapor pressures are present at a time when the passive resistance of the soil is bearing on the tank wall due to elevated temperatures within the tank.

Using unit stress values of 20,000 psi for hydrostatic head and 27,000 psi for hydrostatic head plus vapor pressure loadings, each of the tank types was analyzed to determine the maximum specific gravity and simultaneous allowable vapor pressure for each of two cases. First, the tank was assumed to be at an elevated temperature, and second, the tank was assumed to be cold, i.e. a temperature approaching normal ground conditions. The following values obtained:

MAXIMUM LOADINGS FOR FULL WASTE TANKS

Tank Farm	Maximum Specific Gravity		Simultaneous Vapor Pressure, psig	
	Hot	Cold	Hot	Cold
241-U, T, B, C, BX	1.9	2.1	2.5	2.5
241-S, BY, TX, TY	1.2	1.4	1.8	1.8
241-SX	1.5	1.8	4.8	5.3
241-A	2.2	2.4	6.9	7.5

Actually, the vapor pressure for a cold tank means little because no condition is envisioned which would result in a vapor pressure if the tank were cold. If tanks are operated at gravities lower than the maximums, the vapor pressure can be increased and a graph is presented in Figure 2 which indicates this relationship for each of the tank types. Because of the nature of the unit stress limits which were discussed above, the graphical information should not be extrapolated beyond the maximum specific gravity indicated. In other works, reduction or elimination of vapor pressure will not increase the maximum specific gravities listed above. Neither should the extrapolation be extended beyond a maximum vapor pressure of 10 psi which is the limiting pressure on the dome as discussed earlier.

Since the maximum permitted gravities may be lower than a desirable operating level, it should be recognized that higher gravities are permissible if a limit is placed on the maximum depth to which a tank is filled. An arbitrary specific gravity of 2.2 was assumed and the following limiting depths and simultaneous vapor pressures were calculated:

Tank Farm	Max. Fill Depths* @ Sp. Gr. 2.2, ft.	Simultaneous Vapor Pressure, psig	Tank Depth* to Overflow, ft.
241-U, T, B, C, BX	15	3.0	17
241-S, BY, TX, TY	16.5	3.0	23.5
241-SX	21.5	5.5	31

* Depths are measured from the low point of tank bottom

VAPOR PRESSURE VS. EFFECTIVE SPECIFIC GRAVITY IN FULL WASTE STORAGE TANKS
(USING UNIT STRESSES HIGHER THAN NORMAL DESIGN PRACTICE)

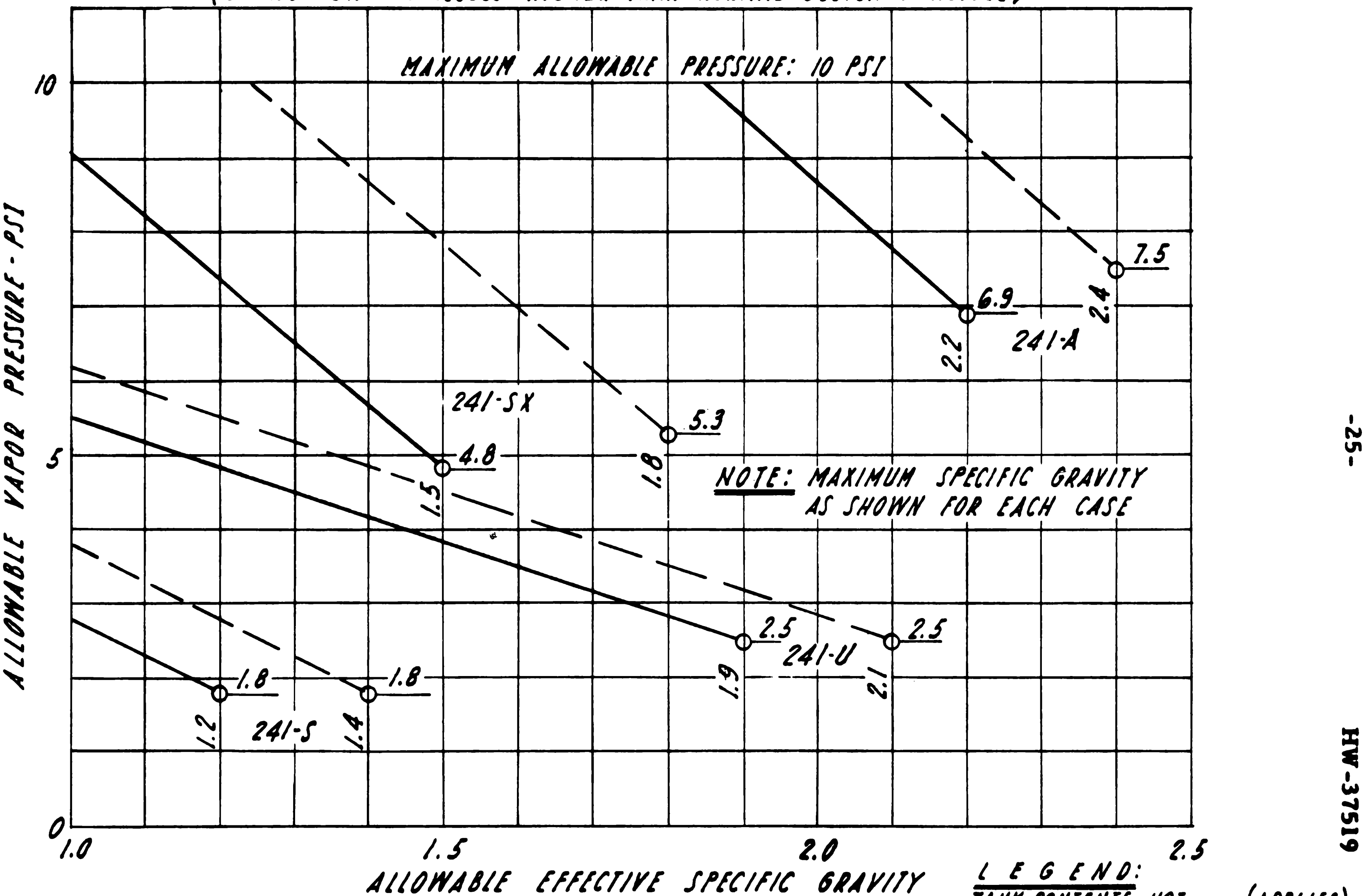


FIG. 2

The reader is cautioned against making extrapolations between the values tabulated above and those presented in Figure 2, without a careful investigation since the section at which critical stress is reached will vary with the fill depth in each case.

It will be noted that little has been said about stresses other than ring tensions. However, the analysis indicated that the ring forces were the critical ones and the limiting values of specific gravity so calculated resulted in a very small increase over normal design stress for the beam action in the wall. Therefore, the reader will not be burdened with a discussion of wall bending moments other than to point out that such forces exist.

7. CONCLUSION

Although the limitations on tank operation presented herein have been calculated on rational methods and with many significant factors unevaluated precisely, it is believed that waste tanks subjected to the specific gravities and vapor pressures quoted will not present an undue structural hazard. Unquestionably, the higher allowable unit stresses will permit more and wider cracks than would be the case if the original lower (and usual design) unit stresses were used. The higher unit stresses change the degree of cracking that is permitted. However, the values are believed to be such that the structural stability of the tank is not endangered. As long as the integrity of the steel plate liner is not violated there need be but little concern about waste leakage to the sub-surface strata. Actual structural collapse due to hydrostatic overloading seems extremely remote when one considers that although the passive resistance of the soil will not prevent cracking of the concrete it will prevent marked outward tank deformation.

C O P Y

PUREX TANK FARM
Basis of Dome Design

Radioactive wastes are stored in underground tanks for indefinite periods requiring suitable protection of personnel from harmful radiation originating in the wastes. As a matter of economy these structures are shielded by being buried with a soil cover, the minimum depth being in the range of 6 to 8 feet. The underground tanks are 75 ft. diameter reinforced concrete structures with steel plate liner, of varying depths depending on the capacity desired.

The dome type of roof structure was selected for the following reasons:

1. A clear span structure would result thereby eliminating any complications of interior supports.

2. A comparison with other types of clear span roof structures, such as beam and slab construction, indicated the dome to be the most economical type of construction under the particular design conditions.

3. The dome shape is more resistant to bomb blast.

In general, the dome design followed the recommendations of the Portland Cement Association as set forth in ST-55, "Design of Circular Domes". The elliptical shape was used in order to obtain a better distribution of reinforcing steel, and to eliminate the need of a heavy edge member to carry ring tension as would be required in the case of a discontinued spherical shape. The dome rise of 12 feet is perhaps small, but was balanced economically against the additional excavation and backfill which would be required if a greater rise was employed. The fact that the dome is not truly elliptical, but is composed of segments of circular arcs, can be attributed to a consideration of the problems of form construction. To this end, four circular arcs form the interior surface of the dome roof; to/wit, arcs of radii 95 ft., 60 ft., 10 ft. and 2 ft. $2\frac{1}{2}$ in. This series of circular arcs very closely approximate the locus of a true ellipse.

Domed roofs, as usually employed in the design of structures, are exposed and therefore subjected to relatively light loads - dead load, wind and/or snow load. Rarely do we find a dome which is required to carry the heavy soil loading as in this particular case. The value of allowable compressive stress for concrete in domes is not covered in the A.C.I. code, but past experiences have shown that high compressive stresses do not obtain in the usual dome. Such is not the case here. Certainly an average load of approximately 1600 psf. (see fig. 2) is not conducive to low stress. In this particular case the compressive stress of about 275 psi is higher than the 150-200 psi values usually encountered in domes but considerably less than the American Concrete Institute allowable compressive stresses in other types of members. This condition is believed reasonable and results in a dome thickness of 15 inches. The cross section of the dome near the outer edge is thicker in order to accommodate the required amount of reinforcing steel.

In the detailed calculations three lines of dome action were considered as follows:

1. A true ellipse with the major and minor axes to the interior of dome surface.

2. A true ellipse with the major and minor axes to the center line of dome thickness.

3. A random curve approximating the center line of the actual dome thickness.

These three cases are illustrated in figure 1.

Figure 2 indicates the total weight (including soil load, live load, and dead load of dome itself) supported by the dome above any given horizontal circle whose radius is measured from the center of the dome. Using these values and geometric constants as given in ST-55, the hoop stress curve was obtained as shown in figure 3. The hoop tension portion of the curve is of particular interest for it determines the amount of tensile reinforcing steel required.

Using the random curve line of dome action, figure 4 indicates the distribution of the hoop tension along the meridian line of the dome plotted against finite increments of meridian length between the various horizontal radii. Using an allowable tensile stress of 20,000 psi in the reinforcing steel and the area under the curve in figure 4 as a measure of total hoop tension, the required steel area in any given section of the dome is obtained. These values of steel area are plotted in figure 5 together with the steel area actually provided. Also shown in figure 5 is a summary of the increments of steel area provided in the hoop tension portion of the dome.

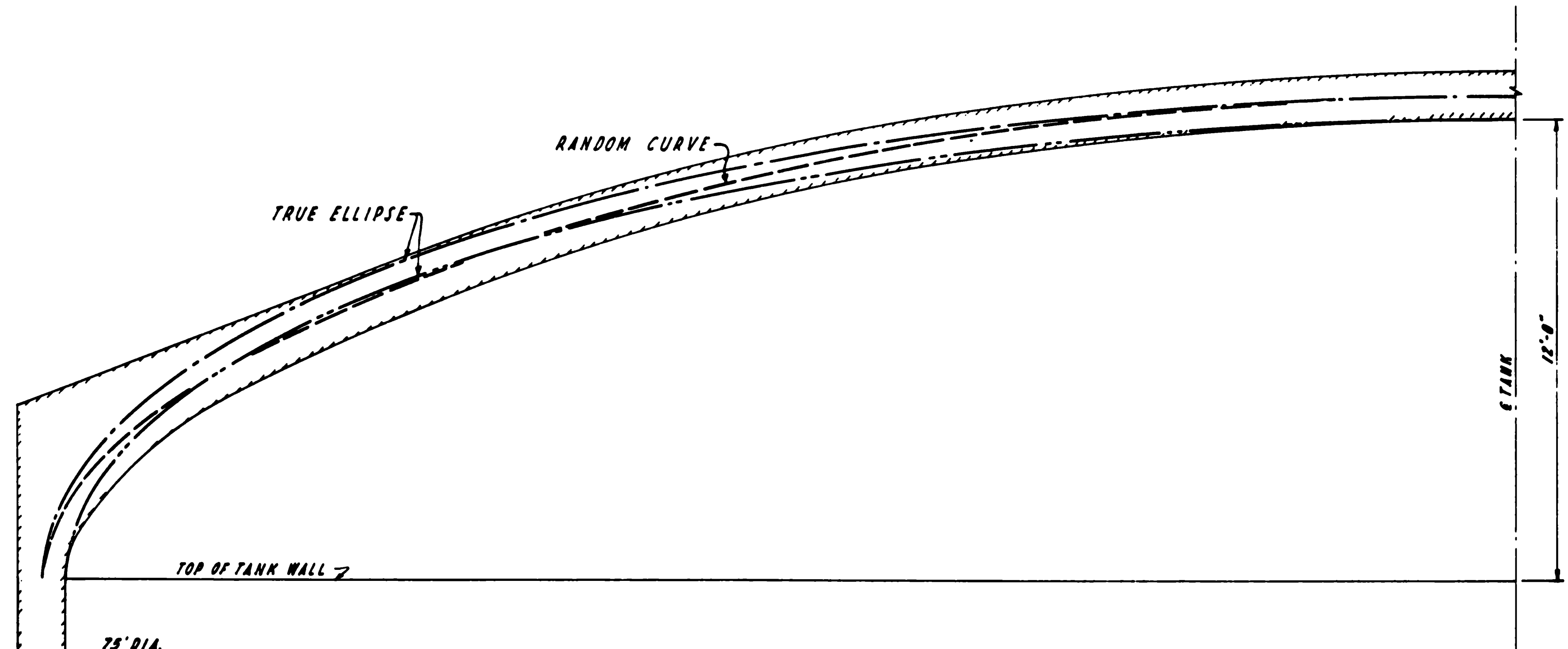
/s/ Edgar F. Smith

Architectural & Civil Design
Design Section
ENGINEERING DEPARTMENT

EF Smith:mm

April 1, 1954

C O P Y



LINES OF DOME ACTION

FIG. 1

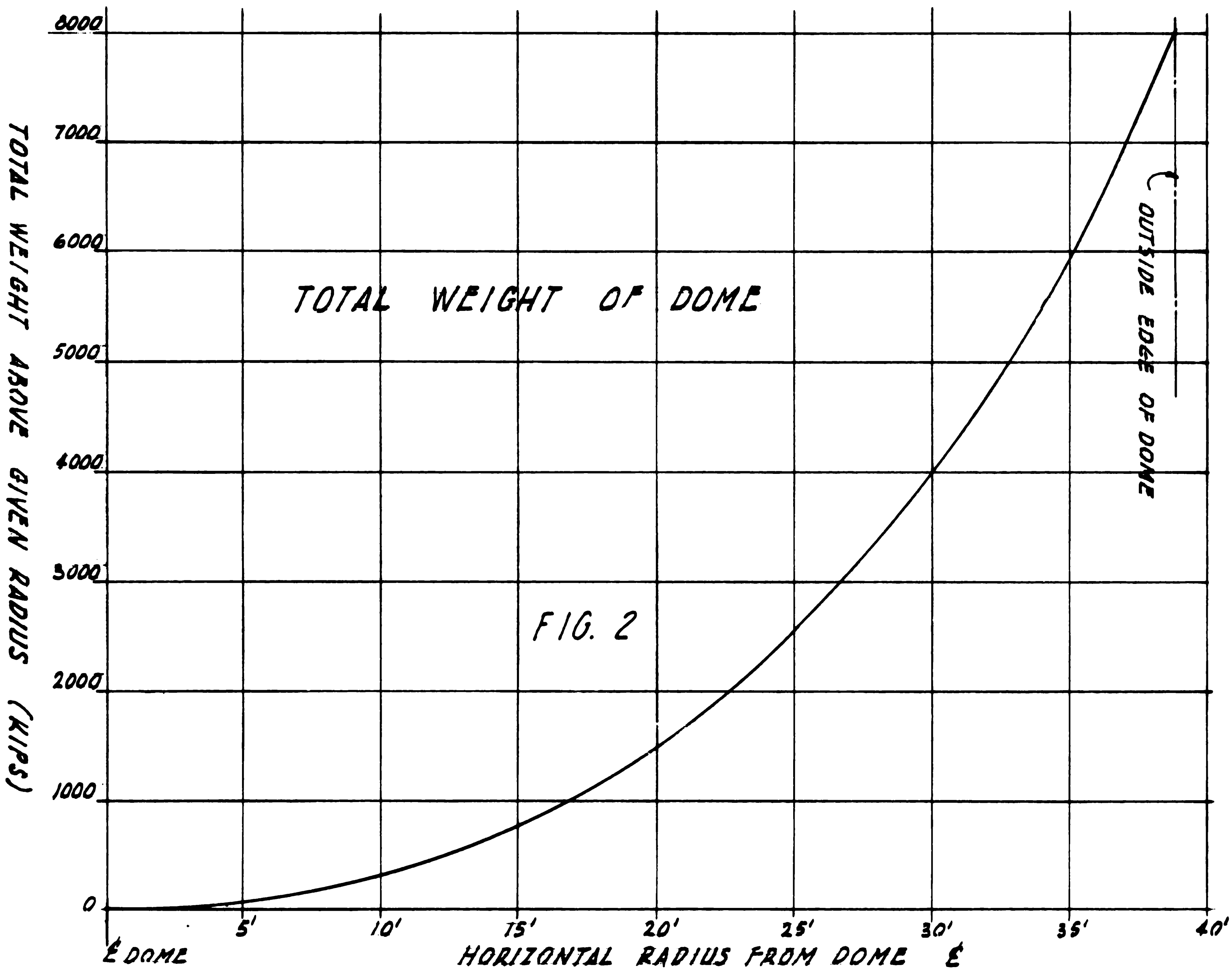

TOTAL WEIGHT OF DOME
TOTAL WEIGHT ABOVE GIVEN RADIUS (KIPS)
8000
7000
6000
5000
4000
3000
2000
1000
0
5'
10'
15'
20'
25'
30'
35'
40'
℄ DOME
HORIZONTAL RADIUS FROM DOME ℄
℄ OUTSIDE EDGE OF DOME

FIG. 2

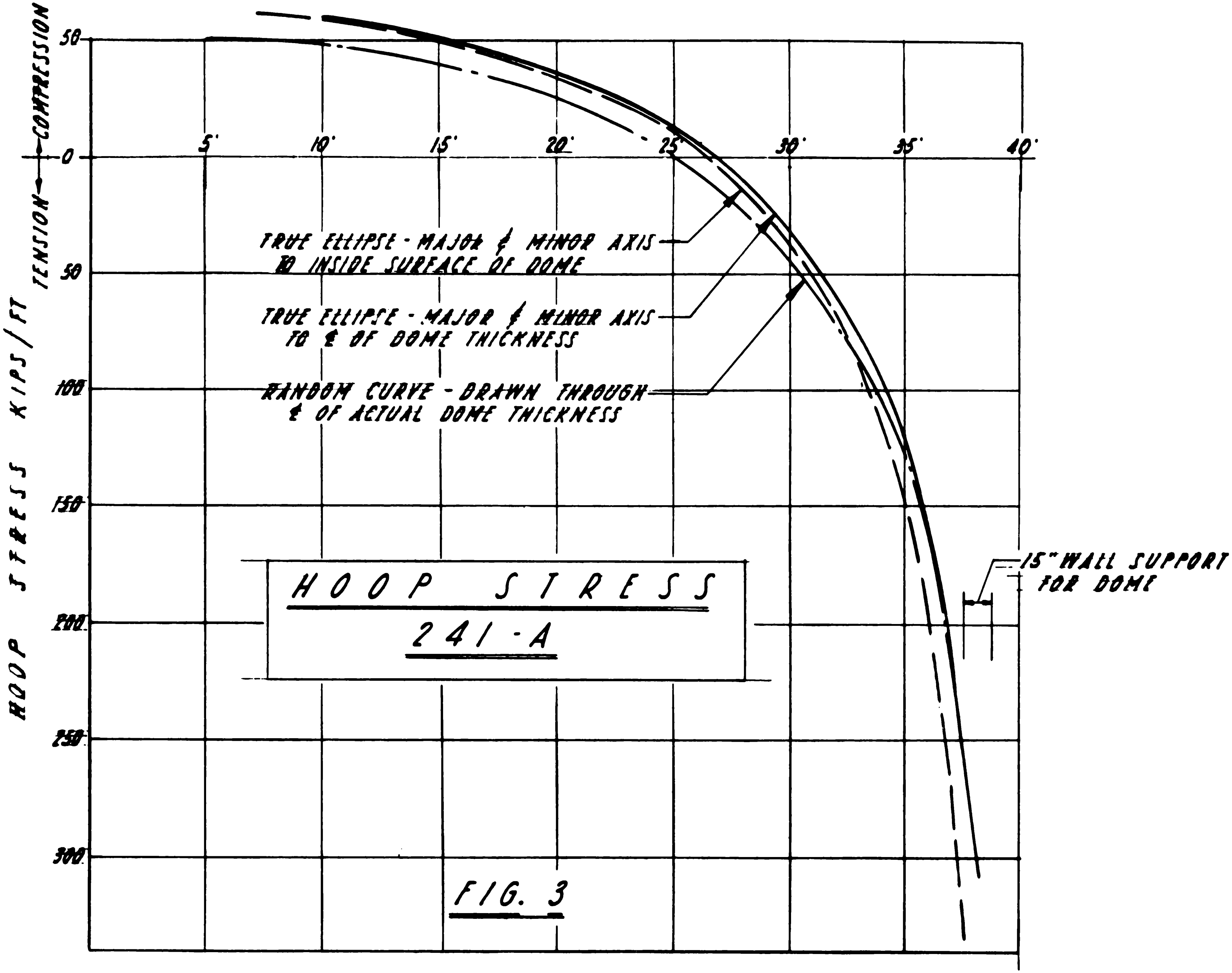
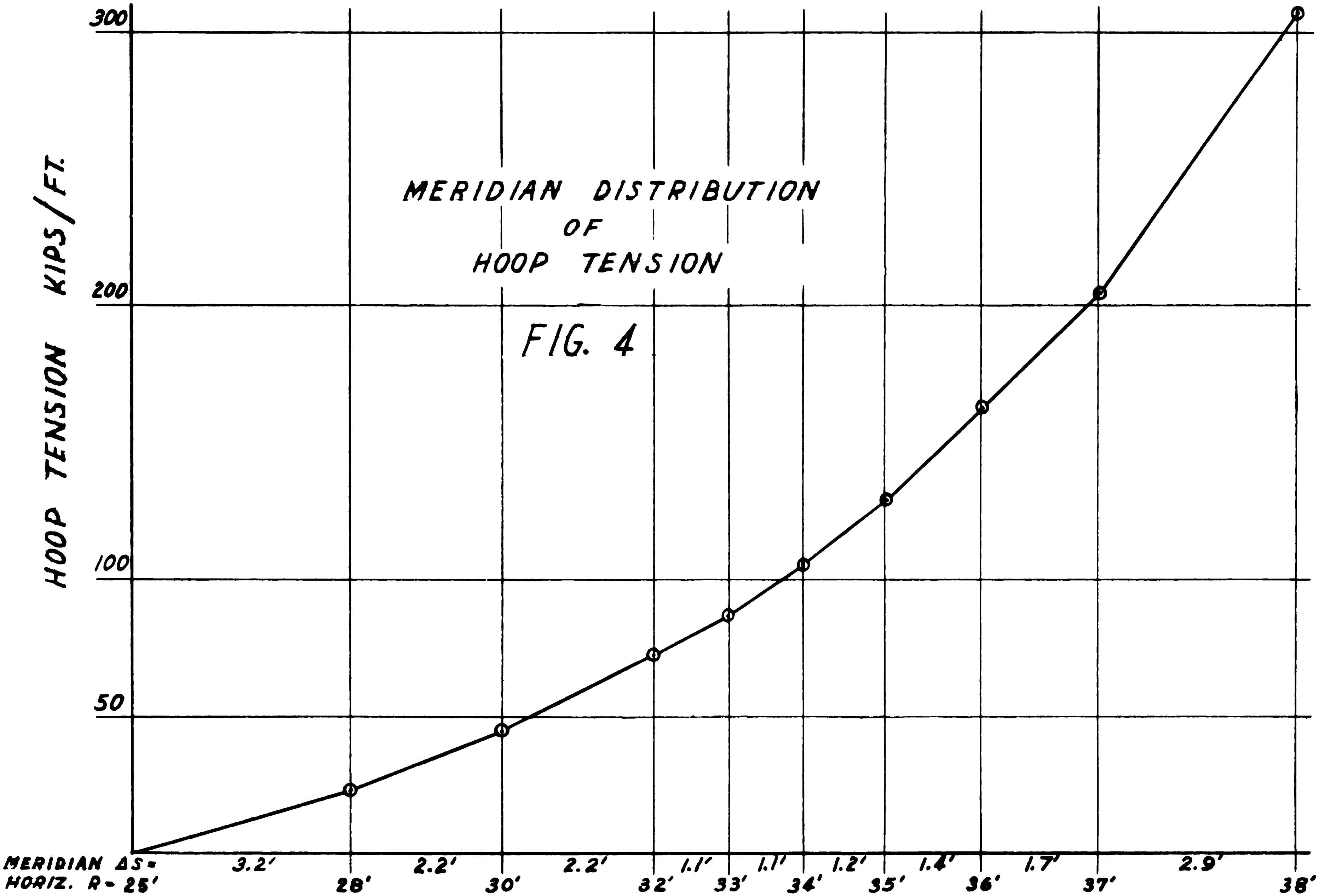
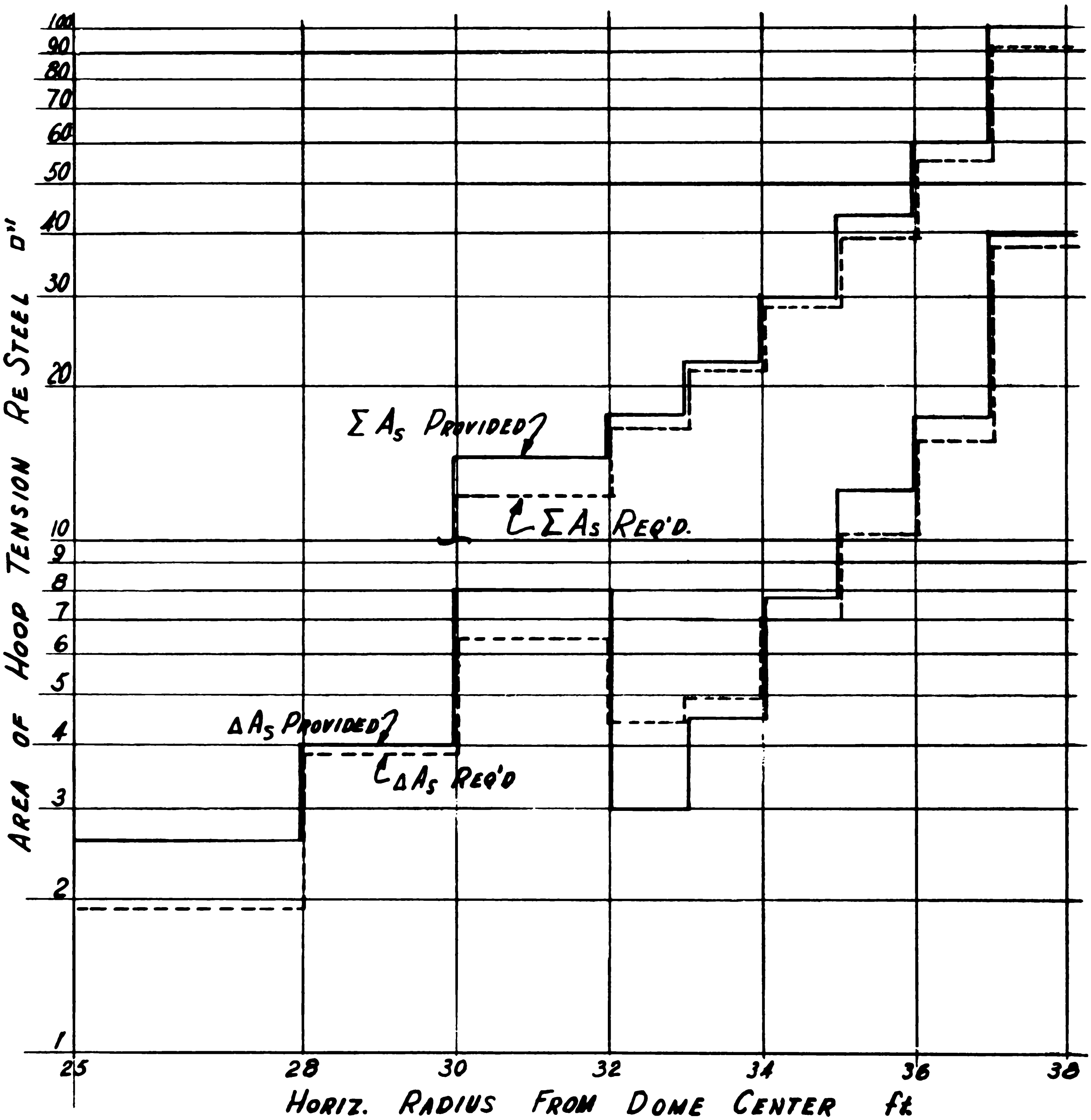

HOOP STRESS
241-A
COMPRESSION
TENSION
HOOP STRESS KIPS/FT
50
0
50
100
150
200
250
300
5'
10'
15'
20'
25'
30'
35'
40'
TRUE ELLIPSE - MAJOR & MINOR AXIS
TO INSIDE SURFACE OF DOME
TRUE ELLIPSE - MAJOR & MINOR AXIS
TO ℄ OF DOME THICKNESS
RANDOM CURVE - DRAWN THROUGH
℄ OF ACTUAL DOME THICKNESS
15" WALL SUPPORT
FOR DOME

FIG. 3



MERIDIAN DISTRIBUTION
OF
HOOP TENSION

FIG. 4



HOOP TENSION REINFORCING STEEL

FIG. 5

C O P Y

June 23, 1955

R. E. Tomlinson
Head, Process Planning Unit
Chemical Development Sub-Section
326 Building

STRUCTURAL EVALUATION 241-S-101 & 104 WASTE TANKS
Ref. a) Tel. conv. ET Merrill & EF Smith, 6-23-55
b) Letter, EF Smith to RE Tomlinson, Waste Storage Tanks, 6-15-55

Based on the current status of liquid wastes in the subject tanks as reported by ET Merrill in the referenced telephone conversation, it is believed that tanks 101 and 104 in 241-S tank farm can be considered satisfactory, and that it is not necessary to lower the liquid level in the tanks at this time.

Briefly, the present status of wastes in these tanks is summarized as follows: (It is recognized that these values have been exceeded heretofore.)

Overall specific gravity	1.50 (liquid and sludge)
Sludge layer thickness	3' at specific gravity 2.5
Sludge temperature	280° F
Liquid temperature	230° F

Using these assumptions a specific gravity of 1.34 has been estimated for the liquid above the sludge layer, assuming that the sludge will remain at the bottom of the tank. This gravity, together with lower temperatures which will continue to recede, is believed to fall within the operating limits of the 241-S tanks as set forth in the graph accompanying the referenced letter.*

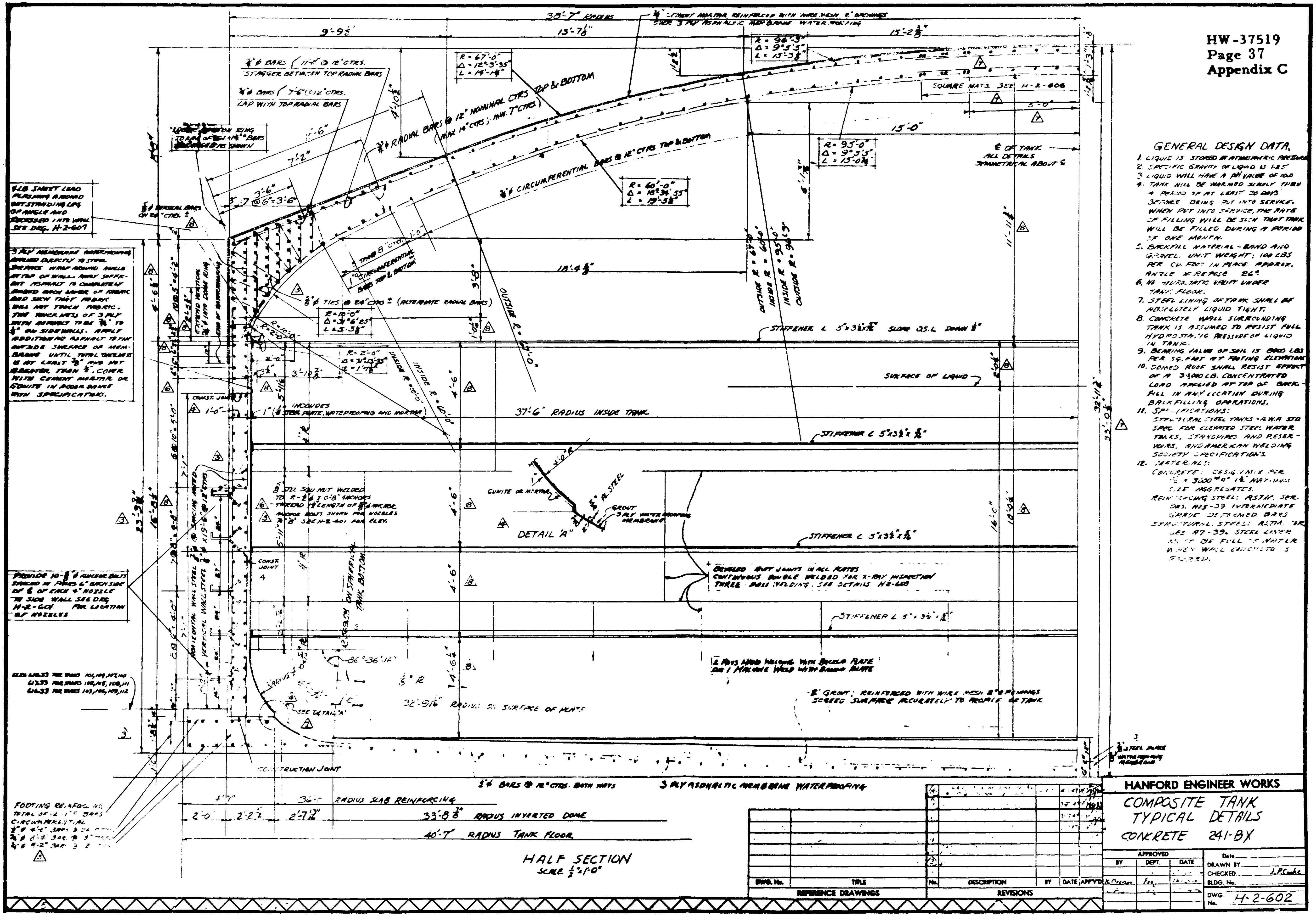
/s/ Edgar F. Smith

Architectural & Civil Design
Design Section
ENGINEERING DEPARTMENT

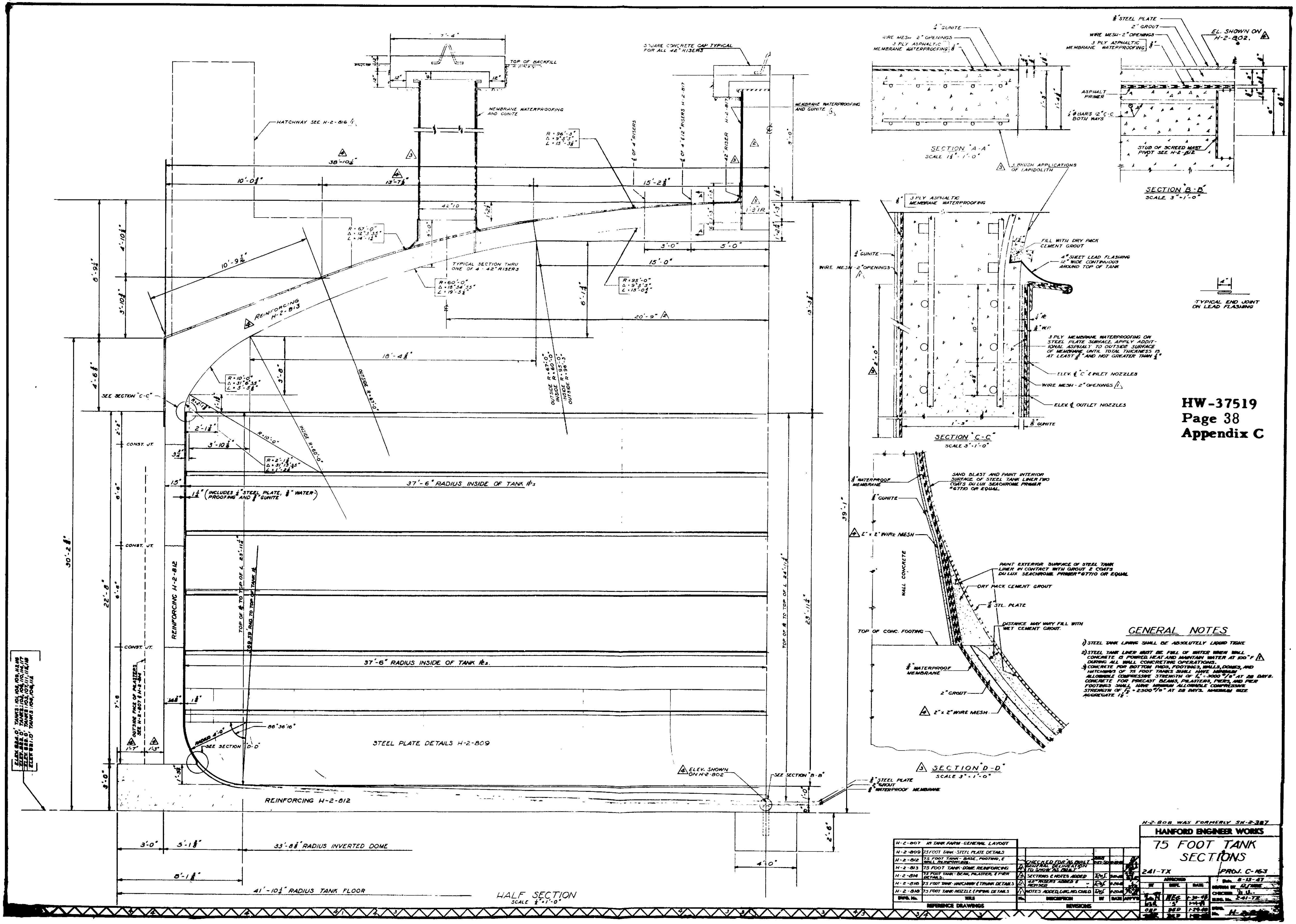
EF Smith:mm

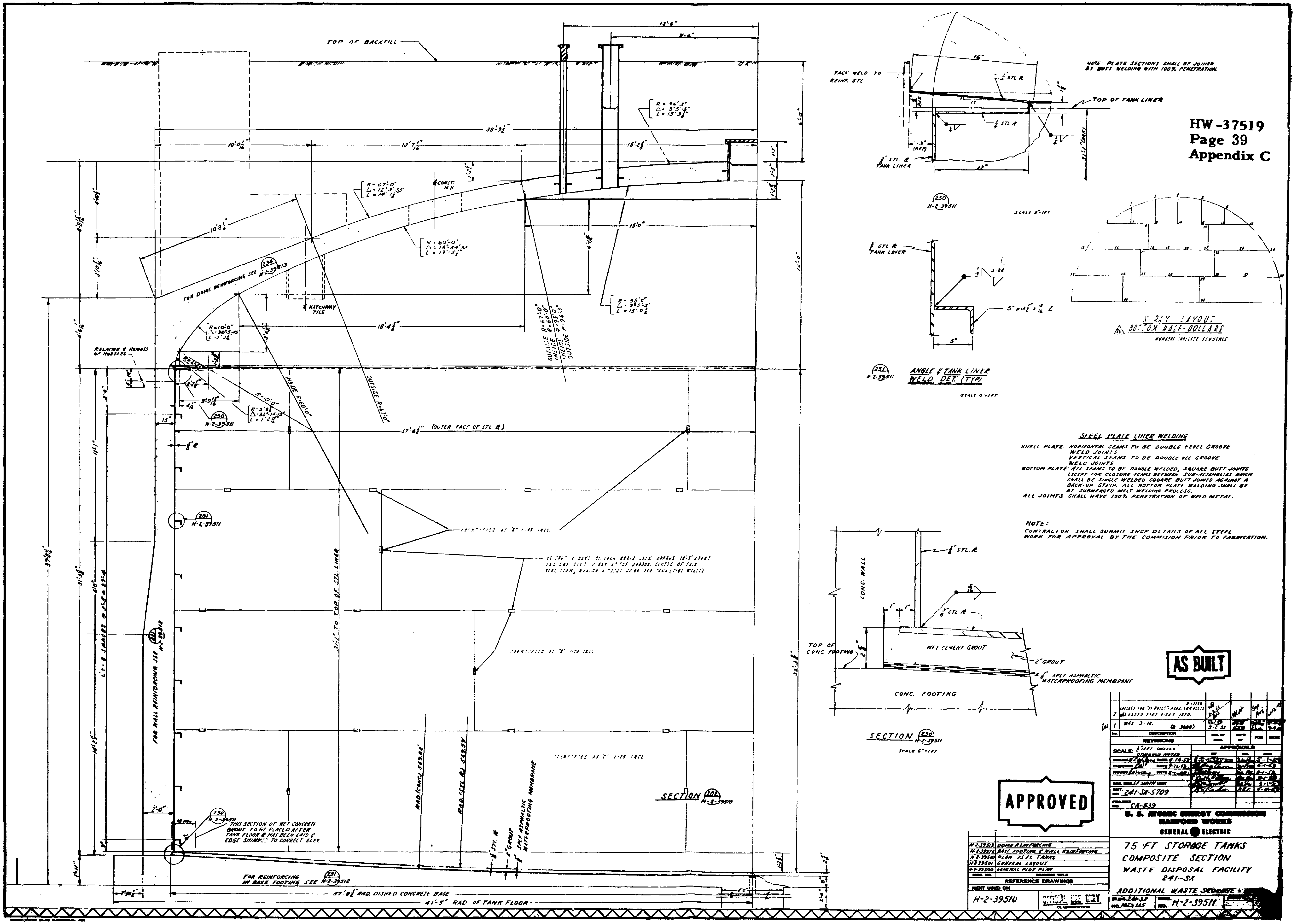
cc: RH Beaton
MW Cook
WM Harty
ET Merrill
HF Peterson
OH Pilkey
CA Rohrmann
MJ Rutherford
FH Shadel
HP Shaw
ED Waters
EF Smith

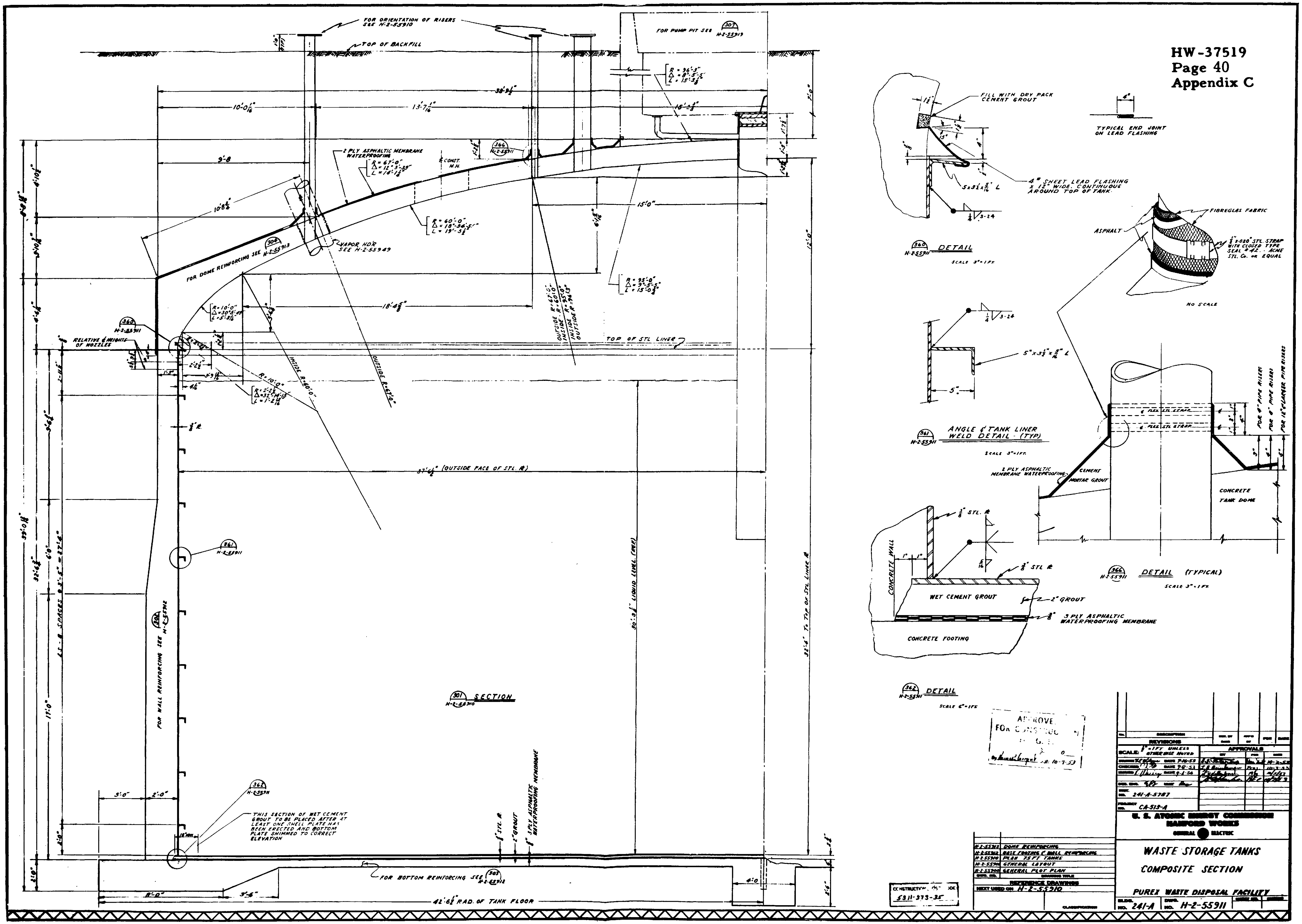
*The graph to which reference is made is identical to figure 2 on page 25.

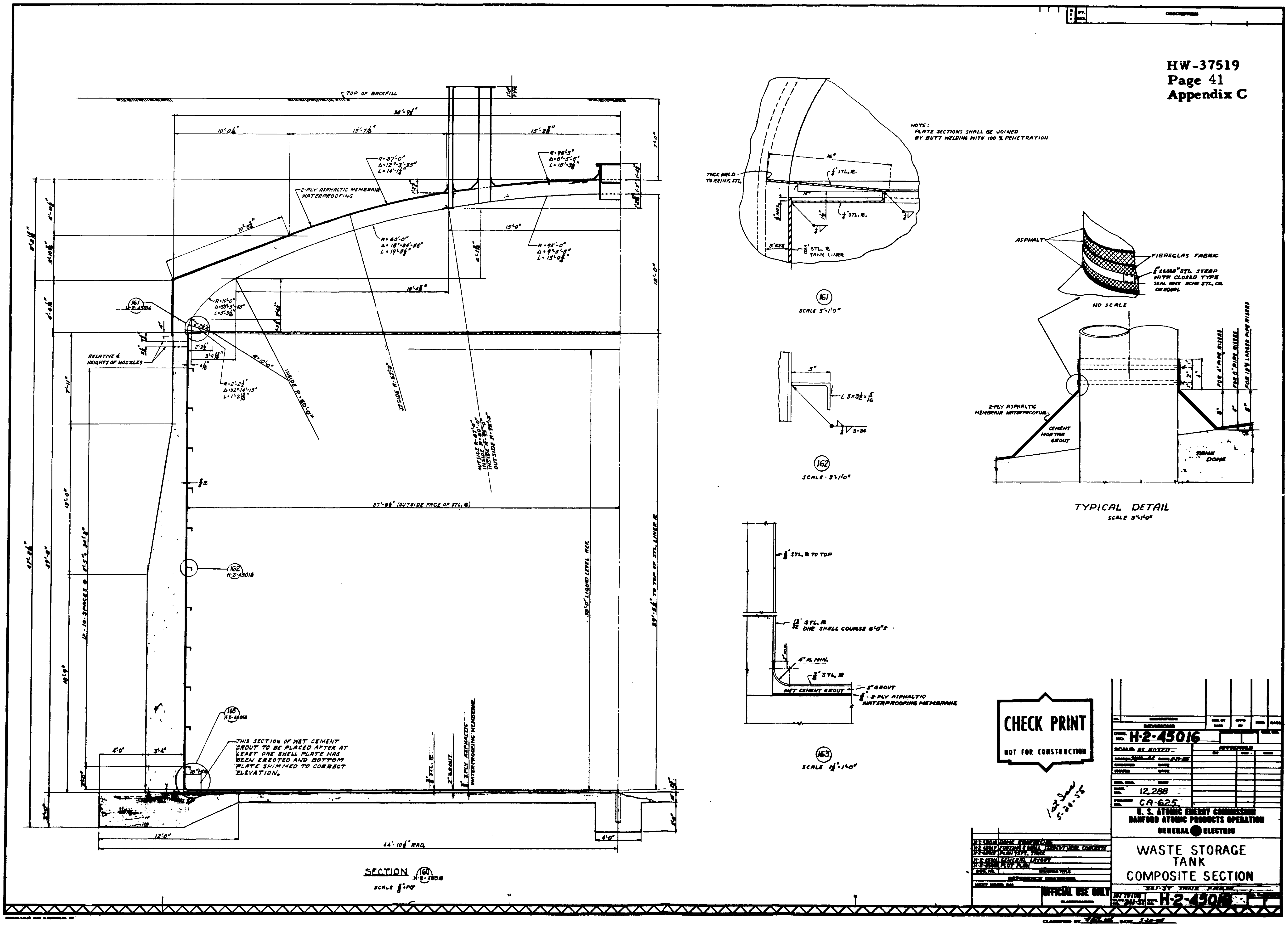

38'-7" RADIUS
9'-9½"
13'-7⅛"
15'-2⅜"
¾" CEMENT MORTAR REINFORCED WITH WIRE MESH 2" OPENINGS OVER 3 PLY ASPHALTIC MEMBRANE WATERPROOFING
R = 67'-0"
R = 95'-0"
R = 60'-0"
SQUARE NUTS SEE H-2-606
15'-0"
℄ OF TANK ALL DETAILS SYMMETRICAL ABOUT ℄
¾" RADIAL BARS @ 12" NOMINAL CTRS TOP & BOTTOM (MAX 14" CTRS; MIN. 7" CTRS)
¾" CIRCUMFERENTIAL BARS @ 12" CTRS TOP & BOTTOM
STIFFENER L 5"x3½"x⅝" SLOPE O.S.L. DOWN ⅛"
SURFACE OF LIQUID
37'-6" RADIUS INSIDE TANK
STIFFENER L 5"x3½"x⅝"
DETAIL "A"
GUNITE OR MORTAR
GROUT
3 PLY WATERPROOFING MEMBRANE
STIFFENER L 5"x3½"x⅝"
BEVELED BUTT JOINTS IN ALL PLATES CONTINUOUS DOUBLE WELDED FOR X-RAY INSPECTION THREE PASS WELDING. SEE DETAILS H-2-603
STIFFENER L 5"x3½"x⅝"
2 PASS HAND WELDING WITH BACKUP PLATE OR 1 MACHINE WELD WITH BACKUP PLATE
2" GROUT; REINFORCED WITH WIRE MESH 2" OPENINGS SCREED SURFACE ACCURATELY TO PROFILE OF TANK
32'-9⅞" RADIUS TO SURFACE OF PLATE
CONSTRUCTION JOINT
¾" ϕ BARS @ 12" CTRS. BOTH WAYS
3 PLY ASPHALTIC MEMBRANE WATERPROOFING
36'-0" RADIUS SLAB REINFORCING
33'-8⅞" RADIUS INVERTED DOME
40'-7" RADIUS TANK FLOOR
HALF SECTION
SCALE ½" = 1'-0"
GENERAL DESIGN DATA.
1. LIQUID IS STORED AT ATMOSPHERIC PRESSURE
2. SPECIFIC GRAVITY OF LIQUID IS 1.25
3. LIQUID WILL HAVE A pH VALUE OF 10.0
4. TANK WILL BE WARMED SLOWLY THRU A PERIOD OF AT LEAST 30 DAYS BEFORE BEING PUT INTO SERVICE. WHEN PUT INTO SERVICE, THE RATE OF FILLING WILL BE SUCH THAT TANK WILL BE FILLED DURING A PERIOD OF ONE MONTH.
5. BACKFILL MATERIAL - SAND AND GRAVEL. UNIT WEIGHT: 100 LBS PER CU. FOOT IN PLACE. APPROX. ANGLE OF REPOSE 26°
6. NO HYDROSTATIC UPLIFT UNDER 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 8000 LBS PER SQ. FOOT AT FOOTING ELEVATION
10. DOMED ROOF SHALL RESIST EFFECT OF A 30,000 LB. CONCENTRATED LOAD APPLIED AT TOP OF BACKFILL IN ANY LOCATION DURING BACKFILLING OPERATIONS.
11. SPECIFICATIONS:
STRUCTURAL STEEL TANKS - A.W.A. STD SPEC. FOR ELEVATED STEEL WATER TANKS, STANDPIPES AND RESERVOIRS, AND AMERICAN WELDING SOCIETY SPECIFICATIONS
12. MATERIALS:
CONCRETE: DESIGN MIX FOR f'c = 3000#/□" 1½" MAXIMUM SIZE AGGREGATES.
REINFORCING STEEL: ASTM. SER. DES. A15-39 INTERMEDIATE GRADE DEFORMED BARS
STRUCTURAL STEEL: ASTM. SER. DES. A7-39. STEEL LINER MUST BE FULL OF WATER WHEN WALL CONCRETE IS POURED.
REFERENCE DRAWINGS
REVISIONS
HANFORD ENGINEERING WORKS
COMPOSITE TANK TYPICAL DETAILS CONCRETE 241-BX
DWG. No. H-2-602

HW-37519
Page 38
Appendix C




TOP OF BACKFILL
FOR DOME REINFORCING SEE H-2-39513
HATCHWAY TILE
RELATIVE & HEIGHTS OF NOZZLES
37'-6 1/4" (OUTER FACE OF STL. R)
OUTSIDE R-67'-0"
INSIDE R-60'-0"
FOR WALL REINFORCING SEE H-2-39512
THIS SECTION OF WET CONCRETE GROUT TO BE PLACED AFTER TANK FLOOR R HAS BEEN LAID & EDGE SHIMMED TO CORRECT ELEV.
FOR REINFORCING IN BASE FOOTING SEE H-2-39512
37'-8 1/2" RAD. DISHED CONCRETE BASE
41'-5" RAD OF TANK FLOOR
SECTION H-2-39510
NOTE: PLATE SECTIONS SHALL BE JOINED BY BUTT WELDING WITH 100% PENETRATION.
TACK WELD TO REINF. STL
TOP OF TANK LINER
TANK LINER
X-RAY LAYOUT
BOTTOM HALF-DOLLARS
ANGLE & TANK LINER WELD DET. (TYP)
STEEL PLATE LINER WELDING
SHELL PLATE: HORIZONTAL SEAMS TO BE DOUBLE BEVEL GROOVE WELD JOINTS
VERTICAL SEAMS TO BE DOUBLE VEE GROOVE WELD JOINTS
BOTTOM PLATE: ALL SEAMS TO BE DOUBLE WELDED, SQUARE BUTT JOINTS EXCEPT FOR CLOSURE SEAMS BETWEEN SUB-ASSEMBLIES WHICH SHALL BE SINGLE WELDED SQUARE BUTT JOINTS AGAINST A BACK-UP STRIP. ALL BOTTOM PLATE WELDING SHALL BE BY SUBMERGED MELT 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 COMMISION PRIOR TO FABRICATION.
CONC. WALL
TOP OF CONC. FOOTING
WET CEMENT GROUT
2" GROUT
5 PLY ASPHALTIC WATERPROOFING MEMBRANE
CONC. FOOTING
SECTION
AS BUILT
APPROVED
H-2-39513 DOME REINFORCING
H-2-39512 BASE FOOTING & WALL REINFORCING
H-2-39510 PLAN 75 FT. TANKS
H-2-39501 GENERAL LAYOUT
H-2-39500 GENERAL PLOT PLAN
REFERENCE DRAWINGS
H-2-39510
OFFICIAL USE ONLY
241-SX-5709
CA-539
U. S. ATOMIC ENERGY COMMISSION HANFORD WORKS
GENERAL ELECTRIC
75 FT STORAGE TANKS
COMPOSITE SECTION
WASTE DISPOSAL FACILITY
241-SX
ADDITIONAL WASTE STORAGE
H-2-39511


FOR ORIENTATION OF RISERS SEE H-2-55910
TOP OF BACKFILL
FOR PUMP PIT SEE H-2-55913
2 PLY ASPHALTIC MEMBRANE WATERPROOFING
FOR DOME REINFORCING SEE H-2-55913
VAPOR HDR SEE H-2-55949
TOP OF STL LINER
RELATIVE ¢ HEIGHTS OF NOZZLES
37'-6" (OUTSIDE FACE OF STL. R)
FOR WALL REINFORCING SEE H-2-55912
SECTION
THIS SECTION OF WET CEMENT GROUT TO BE PLACED AFTER AT LEAST ONE SHELL PLATE HAS BEEN ERECTED AND BOTTOM PLATE SHIMMED TO CORRECT ELEVATION
FOR BOTTOM REINFORCING SEE H-2-55912
42'-6" RAD. OF TANK FLOOR
FILL WITH DRY PACK CEMENT GROUT
4" SHEET LEAD FLASHING x 12" WIDE. CONTINUOUS AROUND TOP OF TANK
TYPICAL END JOINT ON LEAD FLASHING
DETAIL
FIBREGLAS FABRIC
ASPHALT
NO SCALE
ANGLE & TANK LINER WELD DETAIL (TYP)
CONCRETE TANK DOME
DETAIL (TYPICAL)
CONCRETE WALL
WET CEMENT GROUT
2" GROUT
3 PLY ASPHALTIC WATERPROOFING MEMBRANE
CONCRETE FOOTING
DETAIL
APPROVED FOR CONSTRUCTION
U. S. ATOMIC ENERGY COMMISSION
HANFORD WORKS
GENERAL ELECTRIC
WASTE STORAGE TANKS
COMPOSITE SECTION
PUREX WASTE DISPOSAL FACILITY
241-A
H-2-55911
REFERENCE DRAWINGS
H-2-55910



TOP OF BACKFILL
2-PLY ASPHALTIC MEMBRANE WATERPROOFING
RELATIVE & HEIGHTS OF NOZZLES
37'-6¼" (OUTSIDE FACE OF STL. PL.)
THIS SECTION OF WET CEMENT GROUT TO BE PLACED AFTER AT LEAST ONE SHELL PLATE HAS BEEN ERECTED AND BOTTOM PLATE SHIMMED TO CORRECT ELEVATION.
44'-10¼" RAD.
SECTION 160 H-2-45016
NOTE: PLATE SECTIONS SHALL BE JOINED BY BUTT WELDING WITH 100 % PENETRATION
TACK WELD TO REINF. STL.
TANK LINER
SCALE 3"=1'-0"
ASPHALT
FIBREGLAS FABRIC
NO SCALE
2-PLY ASPHALTIC MEMBRANE WATERPROOFING
CEMENT MORTAR GROUT
TANK DOME
TYPICAL DETAIL
WET CEMENT GROUT
2-PLY ASPHALTIC WATERPROOFING MEMBRANE
CHECK PRINT
NOT FOR CONSTRUCTION
H-2-45016
SCALE: AS NOTED
12,288
CA-625
U. S. ATOMIC ENERGY COMMISSION
HANFORD ATOMIC PRODUCTS OPERATION
GENERAL ELECTRIC
WASTE STORAGE TANK
COMPOSITE SECTION
241-SY TANK FARM
OFFICIAL USE ONLY