

**STRUCTURAL ANALYSIS OF ORNL UNDERGROUND  
GUNITE WASTE STORAGE TANKS**

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# STRUCTURAL ANALYSIS OF ORNL UNDERGROUND GUNITA WASTE STORAGE TANKS

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## ABSTRACT

The North Tank Farm (NTF) and the South Tank Farm (STF) located at the Oak Ridge National Laboratory (ORNL)\* in Oak Ridge, Tennessee contain eight underground waste storage tanks which were built around 1943. The tanks were used to collect and store the liquid portion of the radioactive and/or hazardous chemical wastes produced as part of normal facility operations at ORNL, but are no longer part of the active Low Level Liquid Waste system of the Laboratory. The tanks were constructed of gunite. The six STF tanks are 50 ft in diameter, and have a 12 ft sidewall, and an arched dome rising another 6.25 ft. The sidewall are 6 in. thick and have an additional 1.5 in. gunite liner on the inside. There is a thickened ring at the wall-dome juncture. The dome consists of two 5 in. layers of gunite. The two tanks in the NTF are similar, but smaller, having a 25 ft diameter, no inner liner, and a dome thickness of 3.5 in. Both sets of tanks have welded wire mesh and vertical rebars in the walls, welded wire mesh in the domes, and horizontal reinforcing hoop bars pre-tensioned to 35 to 40 ksi stress in the walls and thickened ring. The eight tanks are entirely buried under a 6 ft layer of soil cover. A project in the 1980's removed the majority of the wastes in the STF tanks. Photos and video were also taken of the interior of the eight tanks, and some concrete core samples were removed from the STF domes and tested. Only two of the tanks, W-5 and W-6 in the STF, revealed serious deterioration of the inside wall based on the videos. The present condition of the tanks is not accurately known, since access to them is extremely limited. In order to evaluate the structural capability of the tanks, a finite element analysis of each size tank was performed. Both static and seismic loads were considered. Three sludge levels, empty, half-full, and full were evaluated. The program GTSTRU DL were used for the analyses. Solid brick elements were used to model the tanks. The seismic portion of the analysis was performed using static equivalent loads. In the STF analysis, the effects of wall deterioration and group spacing were evaluated. These analyses found that the weakest element in the tanks is the steel resisting the circumferential (or hoop) forces in the dome ring, a fact verified separately by an independent reviewer. However, the hoop steel has an adequate demand/capacity ratio. Buckling of the dome and the tank walls is not a concern.

## INTRODUCTION

### GENERAL

The Gunite and Associated Tanks (GAAT) project at the Oak Ridge National Laboratory (ORNL) in Oak Ridge, Tennessee, is performing a remedial investigation and feasibility study (RI/FS) to determine more accurately the risks to the general population that the inactive underground gunite storage tank areas (CERCLA site) constitute, and what technologies are required to lower this risk to acceptable levels, and the costs of

implementing these remediation actions. A draft of the RI/FS document for the GAAT was prepared during FY-94, but it became evident that more information was needed about the current condition and contents of the tanks, as well as how practical and cost effective the proposed remediation technologies were before enough confidence could be achieved to present the alternatives to the regulators and to the public. Thus, a "Treatability Study" was initiated to obtain this information. The intent was to assemble and test equipment to successfully remove all the sludge and supernate from at least one of the gunite tanks. A pre- and post- waste characterization

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of the tank(s) is planned to verify the risk reduction obtained. One of the activities needed for these studies is the structural analysis of the tanks, as they currently exist, and also as they are intended to be modified for purposes of the Treatability Study.

## HISTORY

The North and South Tank Farm (NTF and STF) gunite tanks, built in 1943, were originally designed and built to serve for about three years [1]. Gunite is a trade name by the Allentown Pneumatic Gun Company for their product (a mixture of Portland cement, sand, and water in the form of a mortar) sprayed from the nozzle of their cement gun against a form or a solid surface. Shotcrete is the non-trade name for the same product. The original intention was for W-5 and W-6 to receive chemical waste and W-7 to W-10 metal waste. However, due to the changing mission of ORNL, the tank farm piping was modified in 1945 to permit transfer between all six tanks in the STF. Truck pads were installed in 1949 to allow waste from other sites. All of the gunite tanks were removed from service in the 1960s and early 1970s. At that time all contained liquid rad-waste and most also contained varying amounts of radioactive sludge. A recommendation was made that the tanks be emptied and remain in place after being taken out of service. In the interim, the groundwater flow beneath the tanks was to be continuously monitored for any leaks.

A project in 1982-1983 removed the majority of the sludge from the six STF tanks. A sluicing technique was employed to produce a slurry of sludge and supernate which was then treated and pumped to the New Hydrofracture Facility. It is estimated that about 90% of the sludge was removed from the STF during this process. At that time, permanent and moveable steel platforms and footings were installed in the STF in order to support the sluicing operations.

## GENERAL CONSTRUCTION DETAILS

The gunite tanks are physically located in the North and South Tank Farms, which are on either side of Central Avenue at the ORNL. There are six 50 ft. diameter tanks (W5 through W10) located in the STF each having a 170,000 gallon nominal capacity, and two 25 ft. diameter tanks (W3 and W4) in the NTF, the latter each having a capacity of 42,500 gallons. The GAAT is also responsible for four other smaller gunite tanks, two (4800 gallon capacity) located in the NTF, one in the STF (1500 gallon), and the last located elsewhere in the ORNL.

A limited amount of information is available concerning the details of tank construction. Some photos taken during the construction are available. A couple drawings are available. Despite the size differences, there are many similarities between the tanks. They all sit on or very close to bedrock, with a concrete mat sitting on bedrock. The base of the tanks is 3 in. thick lightly reinforced gunite. The height to the overflow level

for each of the tanks is close to 12 ft. Both sizes have a thickened ring at the top of the sidewall where the dome joins the wall, though the larger tanks have a larger ring with more hoop steel area. Both tanks have 60 ksi yield hoop steel in both the sidewall and the thickened ring, pre-stressed to about 35-40 ksi., all other steel is 40 ksi yield steel. The domes are reinforced with a wire mesh mat and radial steel. The minimum compressive strength of the gunite is specified to be 5000 psi at 28-days. The gunite tanks discussed in this paper are all buried, having a 6 ft earth fill cover.

## GENERAL CONDITION OF THE TANKS

During the 1982/83 sluicing operations for the STF gunite tanks, a number of black and white still and video photos of the interior of the STF tanks were obtained. Additional color footage was taken in 1992, which also included tank W-3 in the NTF [2]. In addition, some core samples of the domes were also obtained and structurally tested. The soil cover has never been removed, except for small areas in order to attach risers for various projects. The exterior wall of the tank has not been observed since the soil covering was placed back in 1943. The videos of tanks W-7 to W-10 and also W-3 indicate that the interior walls and dome undersides of these tanks are in good condition. The inside walls of tanks W-5 and W-6, which are known to have held caustic materials, shows areas where wire mesh is visible and possibly areas of deeper deterioration. It is very unclear from the photos/videos, just how deep the deterioration has progressed. The underside of the domes of these two tanks appear to be in good condition. The cylinder tests of the dome cores showed an average strength of 10,000 psi.

## STRUCTURAL ANALYSES OF TANKS

Structural analysis of the gunite tanks (50 ft diameter) located in the South Tank Farm was performed by MMES in 1986. The main purpose of the 1986 analysis was to show that the tanks should not be used in the future for waste storage, and this was fairly easy to conclude. This first study was limited in scope and, as a result, a simplified finite element model (FEM) using plate elements to model a quarter tank was used. The 1994 RI/FS study requested that the 50 ft STF tanks be evaluated for collapse given various loading conditions, and SAIC was selected to perform a more comprehensive analysis, which also would include seismic [3]. A similar evaluation of the 25 ft NTF gunite tanks was made in-house.

## HAZARD CATEGORIZATION

As a result of initial hazard screening, the North Tank Farm was placed in Hazard Category 2 (PC-3). This level was later downgraded to a Hazard Category 3 (approximately PC-2). The South Tank Farm was initially given a Hazard Category 3 (PC-2)

rating, and is currently a Radiological Facility, which could either be a PC-2 or a PC-1 depending on the existence of safety class items. Both analyses were run assuming a PC-2 classification. DOE-STD-1020-94 [4] requires that a PC-2 category structure be able to withstand an earthquake with a return period of 1000 years with peak ground acceleration,  $p_g$ , equal to 0.13g (on rock), which was used for both analyses. The Oak Ridge site-specific bedrock  $p_g$  study for a 1000-year return period resulted in  $p_g = 0.08g$  [5], so there is almost a 60% conservatism by using 0.13g (the site-specific study was incomplete at the time this work started). The tanks therefore are analyzed for seismic loads induced by a 0.13-g horizontal and a 0.09-g vertical ground accelerations (vertical acceleration is assumed to be 2/3 of horizontal). Earthquake induced dynamic soil and fluid pressures were treated as static loading conditions. Soil-structure interaction was not included in the FEM. SAIC did consider group interaction effects in the STF, but group effects was not considered in the NTF since only two tanks are involved.

### **EVALUATION CRITERIA**

The allowable tensile stress in reinforcing steel is 24 ksi (for Grade 60 steel) per the American Concrete Institute (ACI) 318-89 (R92) code [6], and the tensile stress limit for concrete cracking is 300 psi using as an estimate for the uniaxial tensile stress the formula  $4\sqrt{f_c}$  [7] with  $f_c = 5,000$  psi. In general, the allowable tensile stress in concrete, per all codes, is zero, but the uniaxial tensile stress value is being used to infer a bound for initial cracking. This formula gives 283 psi, which was rounded up to 300 psi (the actual value could be as high as 400 psi) since the true gunite compressive strength is probably higher than the 5000 psi minimum. ACI 334R.1-64 [8], ACI 344R-70 [9], and the 1988 ACI 344 Report [10] also provided guidance on the gunite evaluation. ACI 344R-70 gives an allowable compressive (flexural and axial) stress of  $0.38f_c = 1900$  psi for gunite. The Maximum Stress criterion was selected to screen elements for further evaluation. If any maximum principle or algebraic stress exceeded the 300 psi tensile stress limit, then that element was further evaluated as having the potential to form a tension crack.

### **HOOP STEEL PRE-TENSION LOSSES**

In both the NTF and the STF tanks, the hoop steel in the wall and the upper ring was pre-tensioned to either 35 ksi or 40 ksi during the construction of the tanks, using turnbuckles. ACI-344 recommends that, unless precise methods are used to determine prestress losses, that long-term losses be estimated at 32 ksi (this value includes losses due to shrinkage, plastic and elastic shortening of concrete, and relaxation in steel). This means that between 80 and 90% of the original prestress is lost, not including frictional losses. Thus, the analyses for both set of tanks assumed that all the initial prestress had been lost.

## **ANALYSIS OF 50 FT STF GUNITE TANKS**

### **DESCRIPTION**

The South Tank Farm is located south of Central Avenue inside the main fence of ORNL, about three blocks from the west end of the plant. It contains the six large 50 ft diameter gunite tanks (W-5 to W-10) and a much smaller one, W-11, having only a 1500 gallon capacity. The six large tanks are located at 60 ft centers in a 2 x 3 matrix. Each tank has an inside diameter of 50 ft, a 12 ft vertical sidewall, 6 in. thick, a 1½ in. gunite inner liner for much of the height, and a spherical domed roof made up of two 5 in. thick layers rising another 6 ft - 3 in. at the center of the tank having a radius of 53'-1½". There is a thickened ring and haunch at the dome-wall juncture. The wall thickness is 9 in. at the ring, while the haunch, starting 24 in. from the top of the ring goes out in a 45-degree angle 30 in. to the underside of the dome. The tank floor is nominally 3 in. thick except at the juncture with the wall, where the thickness increases to 9 in.

The floor and the dome are reinforced with one layer of welded wire mesh and reinforcing rods placed in the radial direction. The sidewall is reinforced with three layers of welded wire mesh, vertical ½ in. rods, and 21 horizontal rebar hoops. The thickened ring is reinforced with 17 horizontal rebar hoops.

Penetrations in the tank domes were made at different times in the life of the tanks. The size, number, and location of the penetrations varies from tank to tank. The tanks were fabricated with one 30 in. diameter manhole (located about 3 ft from the edge of the dome) and a small center 4 ft vent hole. In one tank, five 24 in. and one 30 in. penetrations now exist.

### **FINITE ELEMENT MODEL**

A three dimensional FEM [3] of the tank was constructed from the 8-node solid elements using the GTSTRUDL program. Since the local stress conditions around the various openings in the tank dome were desired, the entire 360° tank was modeled, thus the model consisted of 104 identical pie segments. Because of its uncertain structural strength and integrity, the inner liner was not included in the model. The dome was modeled as being two layers thick with the bottom of the upper layer connected to the lower layer via rigid space trusses (a gap element that only transfers compressive axial force to the lower dome layer). The dome was modeled with two layers because a second 5 in. layer was added to the original 5 in. thick dome after construction had begun, such that a cold joint probably existed between the two layers. The FEM consists of 8424 elements, 2392 rigid links, and 16,120 joints. A cut-away view of the model is shown in Figure 1. The model is axi-symmetric except for the seven openings in the dome. The tank model is restrained in all translational directions at nodes along the bottom face of the tank floor. All other nodes are free to translate in three directions.

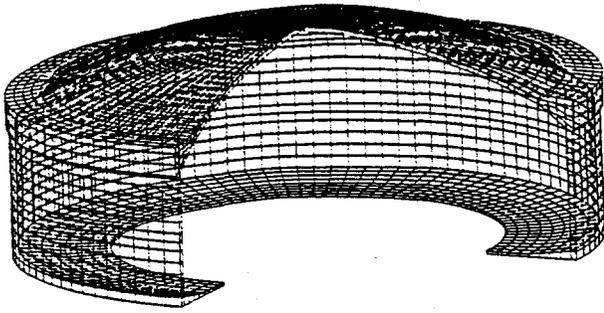


Figure 1. Model of 50 ft Gunite Tanks

The material property of the finite element represents the uncracked concrete strength. The strength of the reinforcing steel was not included in the element but was considered in hand calculations after the stresses in the finite element were obtained. The tank floor, wall, and dome elements are assumed to be homogenous and isotropic. Wall elements are modeled 6 in. thick, the dome elements 5 in thick (each layer), and the floor elements as being 3 in. thick, except that floor elements within a 4 ft band of the wall thicken linearly in the radial direction from 3 in. to 9 in. at the inside face of the tank wall. The confinement ring at the edge of the dome was represented by a cross section of 15 finite elements. The selection of the 8-node solid elements over the 20-node solid elements was based on a balance of computer capacity, computing time, accuracy of results, and effort of interpretation of the results. The FEM assumed seven penetrations in the dome: one 24 in. diameter hole in the center, four 24 in. diameter holes along a 20 ft radius circle, one 30 in. diameter hole and one 12 in. diameter hole along a 22 ft radius circle. The actual penetrations are reinforced with concrete pads, but this additional concrete thickness was not included in the model.

### STRUCTURAL LOADINGS

Gravity and seismic loadings are considered in this analysis. Seismic loading is applied to the structure as equivalent static loading. The gravity loads included the dead load of the tank, the soil pressure, and the fluid loads. The earthquake loads were made up of the tank inertial loads, the seismically induced soil loads, and seismically induced fluid loads. Miscellaneous loads such as equipment on the superstructure, occasional live loads, etc. were not included in this analysis. The effects of uplift from buoyancy had been previously analyzed and determined to be unlikely.

Three fluid levels were considered: empty, half-full, and full. The results for hydrodynamic forces acting in-phase and out-of-phase (sloshing effects) with the direction of the earthquake were

also considered. Thus, eight load cases (combinations) are necessary:

- Case 1: static loads, empty tank
- Case 2: static loads, full tank
- Case 3: static loads, half-full tank
- Case 4: static + earthquake, empty tank
- Case 5: static + earthquake, full tank (in-phase)
- Case 6: static + earthquake, full tank (out-of-phase)
- Case 7: static + earthquake, ½-full tank (in-phase)
- Case 8: static + earthquake, ½-full tank (out-of-phase)

## RESULTS FOR THE 50 ft STF GUNITE TANKS

### STRESSES

The basic structural segments of the tank are the dome, the upper thickened ring, and the wall. The empty tank during earthquake (load case 4, with no internal hydrostatic pressure to provide counterbalancing force to resist the external pressure from the soil) produced the highest stresses in the tank. Table 1 compares the gravity load stresses to the gravity plus seismic load stresses. A brief discussion of the stresses in the various tank areas follows.

Table 1. Summary of Stress Results for STF Tanks

Stress Component	Gravity Stress Cases (psi)	Gravity + Seismic (psi)
Syy, top of wall	+489	+549
Sxx average in ring	---	+220
S1 near dome hole	---	+316
S1 in wall, top	+621	+689
S3 in dome	-597	-642

#### Dome top surface

The center portion of the dome is in compression. The maximum principal compressive stress is 642 psi, which is about one-third of the allowable compressive stress of 1,900 psi, and about one-tenth of the ultimate. The perimeter of the dome is in tension with stresses less than 150 psi, about half of the cracking level of 300 psi.

#### Dome ring

The dome ring has principal stresses ranging from less than

200 psi near the top surface to almost 700 psi at the bottom of the ring wall juncture. The highest maximum principal stress of 690 psi occurs at a very localized spot at the very top of the wall where the thickened ring starts. The stresses decrease very rapidly at locations below this point. These large principal stresses are due primarily  $S_{yy}$  (vertical) stresses. The hoop stresses in this area are smaller, but may be more critical because of the limited amount of hoop steel. The hoop stress ( $S_{zz}$ ) distribution in the upper part of the wall and the dome-ring is shown in Figure 2. The average in this zone is about 230 psi.

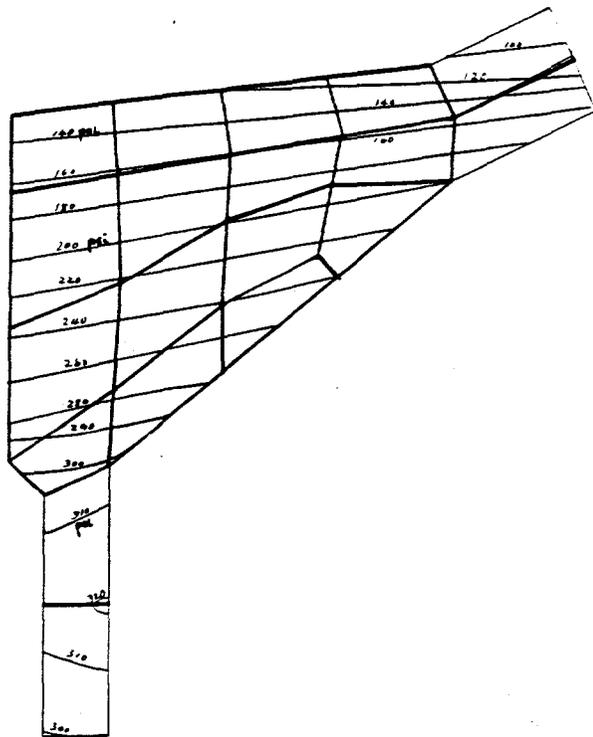


Figure 2. Hoop Stresses in Dome-Ring-Wall, Load 5

### Tank wall

Except for a few feet near the ring and near the base, most of the wall is in compression. At the top, the tensile stresses rapidly drop from the already mentioned 690 psi to 250 psi within about 18 inches. The wall junction near the floor has some small tensile stresses, less than 150 psi, due to flexure caused by lateral earthquake motion.

### Dome bottom surface

The center portion of the dome is in compression. At locations closer to the edge of the dome the stresses become tensile, albeit small. The maximum principal stresses increase to about 200 psi, still below the cracking limit.

### Penetrations on dome

The stresses are intensified at and near the edges of the penetrations. The stress increases around the hole intensify more rapidly at holes located closer to the dome edge ring. The stresses range from less than 100 psi to about 316 psi. The stress intensification around the hole only influences areas within approximately 1.5 hole diameters. Thus, when additional holes are drilled, the impact on dome structural integrity can be minimized if the holes are spaced at least three diameters away from nearby hole centers and one diameter away from the dome ring.

### **CRACKING STRESSES AND REINFORCEMENT**

For all loading cases, concrete at the top of the wall has a high potential to form horizontal cracks on the exterior surface. The vertical reinforcement at the top of the wall is not in compliance with current ACI codes for resisting moments. However, reinforcement is adequate to withstand the existing hoop tensile forces in the wall, dome ring, and in the dome. Another area not fully in compliance with the reinforcing requirements of the ACI codes is a 3 ft band at the edge of the dome; however, the concrete tensile stress in this area is relatively low and cracking is not expected to occur.

### **DISPLACEMENTS**

All the displacements are small. The maximum vertical displacement was found to be 0.093 in., occurring at the top of the dome, and the maximum horizontal displacement at the top of the wall was 0.023 in.

### **ADDITIONAL DOME LOADINGS**

Additional soil or equipment load on the dome may cause horizontal cracks on the exterior surface of the wall in a narrow band just below the dome ring. However, stresses exceeding the cracking strength of the concrete only exist in localized areas. The stability of the tank is not threatened since the stresses in the remainder of the tank wall and the dome ring are not high.

### **REDUCTION IN SECTION THICKNESS**

The 1½ in. gunite liners on the interior surface of the wall in Tanks W-5 and W-6 show some deterioration where patches of welded mesh wires are exposed. Video inspection did not show any deterioration in other tanks. The integrity of the gunite becomes uncertain when exposed to chemical attack. Calculations using the FEM results indicate that no reduction of the wall thickness in the top 2 ft is acceptable; a reduction of 1 in. is acceptable for the portion of the wall 2 ft below the top; and for sections located 3 ft or more below the top of the wall, a 2 in. reduction in the wall thickness is acceptable. Thus, at the mid-height of the wall, a 2 in. reduction, about 33%, in the wall

thickness is acceptable from a structural stability standpoint. It may not be acceptable, however, from a leakage or containment standpoint.

## STABILITY

This analysis shows that the dome shell has a safety factor of 51 against buckling, considerably larger than the safety factor of 4 to 6 recommended by ACI 344R-70. The structural stability of the tank dome is, however, dependant on the peripheral confinement provided by the dome ring steel. The dome ring is primarily subjected to tensile hoop loading which is resisted by the embedded 60 ksi reinforcement. This analysis shows the dome ring reinforcement exhibits a capacity/demand ratio of 1.4 with respect to the recommended ACI 318-89 working stress tensile allowable of 24 ksi for prestressed reinforcement (reinforcement is assumed to resist all the tension in the ring/haunch cross section). With respect to yield, the ring reinforcement steel exhibits a capacity to demand ratio of 3.5. The tank wall exhibits a safety factor of 44 against buckling.

## GROUP BEHAVIOR OF TANKS

The 50 ft tanks in the South Tank Farm are located 60 ft apart, center-to-center, and thus the clear distance between tanks is slightly less than 10 ft. The response of a single tank is impacted during an earthquake by nearby tanks, especially if the clear spacing between the tanks is on the order of a tank radius. This analysis performed a limited investigation based on a study of the behavior of a group of steel tanks [11]. The study found that the maximum stress increase due to tank-to-tank interaction is about 3% over the combined stress of a single tank which does not really affect the overall structural assessments. However, it is suggested that this phenomenon, especially for concrete tanks, be further investigated.

## ANALYSIS OF THE 25 ft NTF GUNITE TANKS

### DESCRIPTION

The North Tank Farm is located north of Central Avenue and just north of the South Tank Farm. It contains four steel tanks and four gunite tanks, all buried. The two larger gunite tanks (W-3 and W-4) are located next to each other with 10 ft clear spacing and 6 ft of soil cover. These tanks have an inside diameter of 25 ft, a 12 ft vertical sidewall 5 in. thick, and a spherical domed roof having a nominal thickness 3½ in. (single layer) with a dome rise of 3'-1½" and a dome radius of 26'-6¾". There is a thickened ring and haunch at the dome-wall juncture. The wall thickness is 6 in. at the ring, while the haunch, starting 18 in. from the top of the ring goes out in a 45-degree angle almost 20 in. to the underside of the dome. The tank floor is nominally 3 in. thick except at the juncture with the

wall, where the thickness increases to 9 in. The floor has one layer of welded wire mesh. The dome is reinforced with one layer of welded wire mesh and 5/8 in. diameter radial reinforcing rods. The sidewall is reinforced with three layers of welded wire mesh, vertical ½ in. rods spaced at 12 in. centers, and sixteen 5/8 in. diameter horizontal rebar hoops (made in three sections, threaded together, and connected with ¾ in. turnbuckles). The thickened ring is reinforced with six 7/8 in. diameter horizontal rebar hoops.

Penetrations in the dome of the two gunite tanks in the North Tank Farm either exist or are planned as part of waste removal projects. The tanks were fabricated with one 30 in. diameter manhole (located about 3 ft from the edge of the dome). The size, number, and location of the new penetrations varies between the two tanks. This analysis assumed three symmetrical penetrations, which included a 24 in. diameter hole in the center and two 24 in. diameter holes located 180° apart about three feet from the edge. The actual penetrations are semi-reinforced with concrete pads. For simplicity the FE model did not include the thickened pads. A second FE model was run which included no penetrations in order to understand the increased stress effects on the tanks due to the penetrations.

## FINITE ELEMENT MODEL

The analysis of the NTF tanks also uses the GTSTRUDL code, using three-dimensional finite elements, mostly 20-node solid bricks. The LMES GTSTRUDL version runs on a RISC-6000 machine. The 20-node solid elements have eight corner nodes plus mid-side nodes, whereas the 8-node elements used in the earlier STF analysis have only the eight corner nodes. Because of memory limitations from using the 20-node elements on the computer it was not possible, nor necessary, to model the entire tank, and only a quarter model was used (Figure 3). Boundary conditions were used to reflect the other portions of the tank. The quarter tank model was initially assembled using a generation scheme in cylindrical coordinates, and then the penetration were added by removing the appropriate nodes and elements. The model without penetrations had 2190 elements and 13032 joints. The model with the penetrations had 2094 elements and 12869 joints. The elements representing the bottom of the tank and the extreme top of the dome were 8-node solid elements; all the rest were the 20-node solid elements, except that variable node transitional elements were utilized in between the 8-node and the 20-node elements.

## STRUCTURAL LOADINGS

Gravity and seismic loadings are considered in this analysis. The seismic loading is applied to the structure as an equivalent static loading. The static loads included the dead load of the tank, the static soil pressure, and the static hydraulic load. The earthquake loads were made up of the tank inertial loads, the seismically induced soil loads, and seismically induced hydraulic loads. Miscellaneous loads were not included in this analysis.

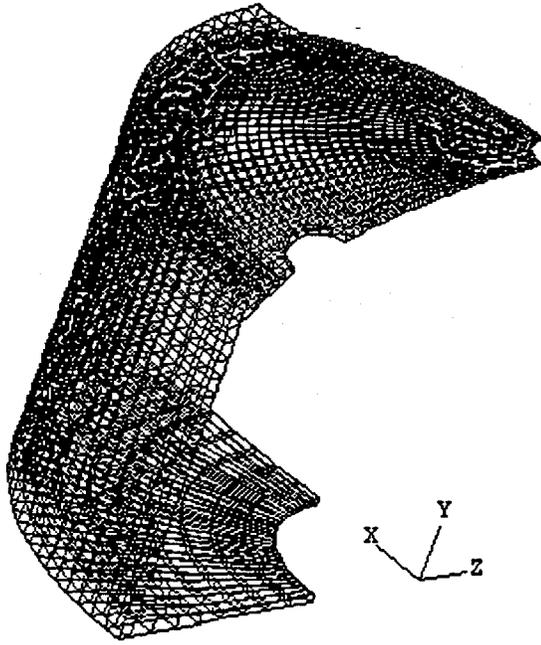


Figure 3. Model of NTF Gunite Tank

A close-up view of the dome-ring-wall area of the model is shown in Figure 4.

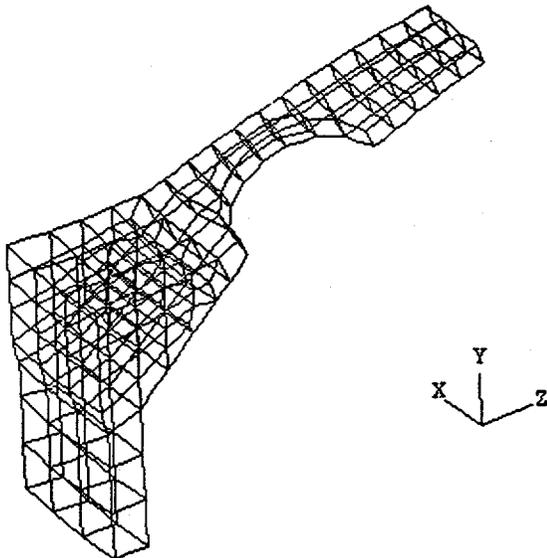


Figure 4. Enlargement of Dome-Ring-Wall Area

The static loads were symmetrical in nature and were applied to the quarter model using symmetrical boundary conditions. The resulting stresses and displacements are valid for all four quadrants. The loads due to vertical seismic are also symmetrical and were handled in a similar manner. However, the dynamic loads due to the seismic input are anti-symmetrical in nature and require anti-symmetry boundary conditions. The center line of the tank in the direction of the earthquake is still a symmetry boundary, but the boundary between the two halves perpendicular to the earthquake vector requires anti-symmetry boundary conditions. Thus, the dynamic loads will produce different results between these two halves, which are labeled the left and the right sides. The earthquake was assumed to act from right to left. Since three fluid levels are being considered, empty, half-full, and full, nine load cases (combinations) are necessary.

- Case 1: static loads, empty tank
- Case 2: static loads, full tank
- Case 3: static loads, half-full tank
- Case 4: static + earthquake, empty tank, left side
- Case 5: static + earthquake, full tank, left side
- Case 6: static + earthquake, 1/2-full tank, left side
- Case 7: static + earthquake, empty tank, right side
- Case 8: static + earthquake, full tank, right side
- Case 9: static + earthquake, 1/2-full tank, right side

## RESULTS: 25 FT NTF GUNITE TANKS

### STRESSES

As for the STF gunite tanks, the basic structural segments of the tank are the dome, the thickened ring, and the wall. While the NTF and the STF have many similarities, and while the locations of high stresses are about the same in both, there are some rather dramatic differences. For one thing, it is clear that the stress concentrations around the penetrations are more pronounced for the 25 ft tanks than for the larger 50 ft tanks, even though these are extremely localized.

#### Plain model (no penetrations)

The maximum principal stress (tension) is 387 psi (load case #4) and occurs at the outside junction of the wall and the bottom of the tank (Node #20). There is also another high tensile region at the top outside of the wall where the ring starts to thicken. The highest principal tensile stress there (node #49) is about 350 psi (load cases #4, 5, 6, 7, and 9). Those are the only two locations where the principal tensile stress exceeds 300 psi. In both cases, the actual stress is in the vertical direction,  $S_{yy}$ , such that a horizontal crack running around the circumference of the tank might open. The maximum hoop stress,  $S_{xx}$ , for all the loadings is only 196 psi, occurring on the inside face of the haunch near the wall. The average tensile hoop stress ( $S_{xx}$ ) in ring/haunch

zone ranges between 138 psi and 167 psi. These stresses are not high enough to induce cracking.

The highest compressive stress, S3, is 724 psi (load case #6), occurring at the point where the underside of the dome joins the thickened haunch; another "high" compression stress zone occurs at the inside corner where the wall meets the base, the highest stress in this region being 665 psi.

The maximum S1 stress without seismic is 316 psi and occurs at node 49 (for both load cases 1 and 3), while the S1 stress at node 20 is now only 209 psi (load 1). Thus, the seismic loads increase the maximum principal stress at node 20 by almost 85% (387 vs. 209), but only by 11% at node 49 (350 vs. 316). The maximum compressive stress under gravity loads is 625 psi, while with seismic it is 724 psi, an increase of 15%.

Model with Penetrations

In this model, in contrast with the larger 50 ft STF tanks, the inclusion of penetrations in the dome plays a very major role in determining the stress profile of the tank. Removed from the holes, the stress pattern is similar to that of the tank with no penetrations, except that only the node 49 location principal stress, S1, exceeds 300 psi (342 psi maximum), whereas the node 20 location now has a maximum value of only 271 psi. For the node 20 locations which are in the same or adjacent radial rows to an edge hole, the maximum tensile stresses is as high as 396 psi (load case 4). Again, this stress is predominantly an Syy stress (vertical).

However, at holes near the tank edge, the stresses have increased by a factor of two or three, to 644 psi. There exist a number of node locations, all on or within 6 in. of the hole boundary, which have maximum principal stresses in excess of 300 psi, most of them over 400 psi. These occur both on the bottom and on the upper surface of the dome. These are, however, very localized stresses, which decrease very rapidly with distance from the hole boundary, as can be seen in Figure 5. In general, within 8 in., or one-third of the hole diameter, the stresses have fallen below the 300 psi level. The maximum gravity S1 stress around the holes is 520 psi, so the seismic increases the stress by 23%. Table 2 compares the gravity load stresses with the gravity plus seismic load stresses for this tank model.

The average hoop stress (Sxx) in the area of the ring/haunch ranges from 142 to 174 psi in the vicinity of an edge holes, and decreases 10 to 20 psi in regions distant from the holes. Again, no cracking is expected in the hoop direction. Hoop stresses for loading 4 are shown in Figure 6. The ring/haunch area is about 296 in.<sup>2</sup>, thus the stress in the hoop steel (area steel = 3.6 in.<sup>2</sup>) is 14.3 ksi (0.174 ksi x 296 in.<sup>2</sup>/3.6 in.<sup>2</sup>), which is below the 24 ksi allowable steel tensile stress. The maximum compressive principal stress is 1280 psi (load case 6), which occurs on the

bottom of the dome on the edge of the hole nearest the thickened ring. There is only a 4% difference in this stress between load cases 4 through 9.

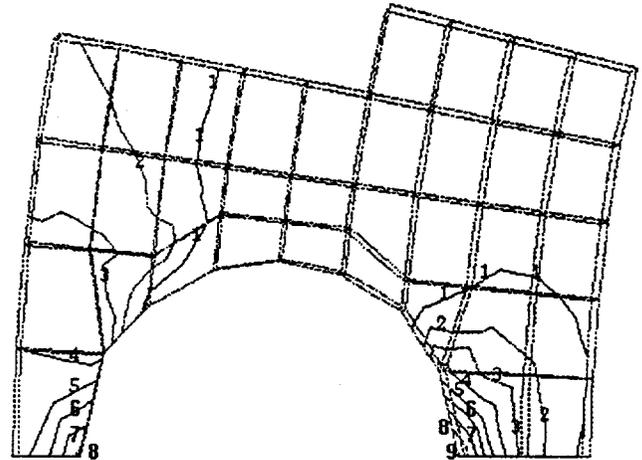


Figure 5. Maximum Principal Stresses at a Dome Edge Hole, Loading 4, n = 70 psi, S1max = 643 psi.

Table 2. Summary of Stress Results for NTF Tanks

Stress Component	Gravity Stress Cases (psi)	Gravity + Seismic (psi)
Syy, base of wall	+198	+379
Syy, top of wall	+311	+336
Sxx in ring/haunch	+221	+266
Sxx average in ring	+144	+173
S1 in dome	+520	+644
S1 in wall, top	+314	+340
S1 in wall, bottom	+216	+396
S3 in ring or wall	-514	-641
S3 in dome	-1087	-1280

The gravity dead load stress in this area is about 1087 psi, thus seismic increases the stress by about 15%. The maximum compressive stresses (including seismic) at the inside bottom

corner of the haunch and the inside bottom corner of the wall are 661 psi and 444 psi, respectively. Since the code allowable is 1900 psi, there is no concern with the compressive stresses in the tank.

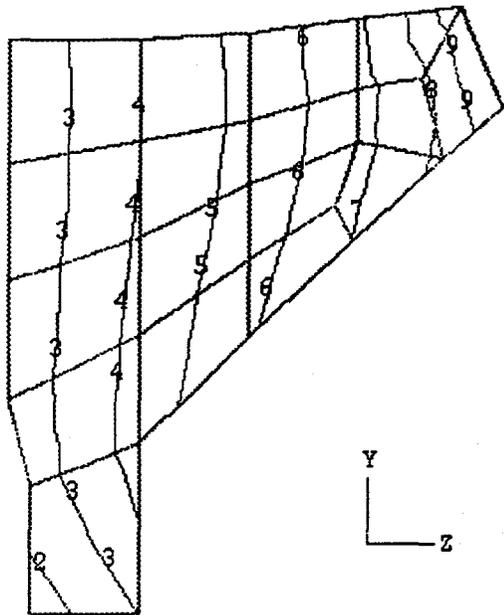


Figure 6. Hoop Stresses ( $S_{xx}$ ) in Dome-Ring  
 $n = 30$  psi,  $S_{xx}(\text{max}) = 300$  psi  
 when  $n = 10$  (upper right)

### STABILITY ANALYSIS

The analysis shows that the dome shell and the tank wall have a margin of safety of 15 and 98, respectively, against buckling, both values being considerably larger than the factor of 4 to 6 recommended by the ACI. The structural stability of the tank dome is, however, dependent on the peripheral confinement provided by the dome-ring steel (Grade 60) reinforcement, which is primarily subjected to tensile hoop loading. As mentioned above, the dome-ring reinforcement is subjected to about 14.3 psi hoop stress, which results in a capacity/demand ratio of 1.68 with respect to the ACI working stress allowable value of 24 ksi. With respect to yield, the ring reinforcement capacity/demand ratio is 4.2. These factors are slightly higher than those found for the 50 ft STF tanks.

### INDEPENDENT REVIEW OF GUNITE TANKS

An independent evaluation of the gunite tanks was performed in early 1995. This evaluation reviewed the ongoing

structural analyses, performed some simple stability calculations for the domes and the walls, reviewed the tank photos and videos, and made some general conclusions related to the integrity of these tanks [12]. The calculations showed that there was ample margin against buckling of both the dome and the wall, but that the capacity was limited by the hoop steel in the dome ring. The calculation showed that this hoop steel was stressed to about 33 ksi for the two sizes of tanks under consideration. This stress value is about twice that obtained using the finite element analysis discussed in this paper. The reason for this difference is that the independent calculation assumed that the dome-ring was infinitely rigid, which will result in an upper bound estimate for the stress. The FE analysis represents more accurately the stiffness of the both the wall and the dome ring and will therefore produce lower stresses in this region. The infinite stiffness assumption, however, is simpler, does not require a finite element solution, and is conservative and is therefore often used for design. The report indicated that (1) the tank designs are more conservative in wall and dome thickness than current tank design standards, (2) there is little evidence of deterioration of any of the domes, a fact upheld by the core sample tests, (3) there is some indication of inleakage around the penetrations, but this problem is more likely the result of inadequate sealing of the penetrations, (4) with the exception of tanks W-5 and W-6, there is virtually no evidence of cracks in the walls of the tanks, and no indication that the circumferential stresses exceed the compression provided by the backfill, and (5) debonding of the inner liner in the STF tanks might occur because of the difficulty of shooting the inner liner, probably in the dark and overhead.

### SUMMARY AND CONCLUSIONS

The finite element analyses, in addition to the independent review, have concluded that the gunite tanks are structurally sound and in no danger of collapse. It was found that the limiting factor against collapse of the tank dome is the hoop steel stress in the dome ring. Table 3 summarizes the three evaluations:

Table 3. Summary of Gunite Tank Margins

Component	FEM STF	FEM NTF	Independent Review	
			NTF	STF
Dome buckling	51	15	---	8.7
Wall buckling	44	98	---	---
Ring steel, hoop allowable stress	1.4	1.7	0.7	0.7
Ring steel, hoop ultimate stress	3.5	4.2	1.7	1.7

As mentioned in the last section, the independent review performed some simplified calculations using quite conservative assumptions, which resulted in the ratios less than one shown in the table. The finite element analyses, by modeling the ring and wall in detail were able to demonstrate that these areas have greater capacity.

Vertical tensile stresses at the top of the wall on the exterior face are high and reinforcement is not fully adequate to prevent some surface cracking. However, the interior face of the wall in this region of the tank wall is in compression and should prevent both in and out leakage of liquids at this level. The tank will not lose its structural stability even if plastic hinges are formed along the top of the wall below the dome ring. Historically, many tanks have been constructed with a structural hinge between the dome and the wall and have performed satisfactorily.

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