Analysis of Underground
Waste Storage Tanks 241 - SY
at Hanford, Washington

October 1974

prepared for
Atlantic Richfield Hanford Company
Richland, Washington

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URS

John A. Blume & Associates, Engineers

San Francisco

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WASTE STORAGE TANKS 241-SY
AT HANFORD, WASHINGTON

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1. INTRODUCTION

1.1 Purpose

High level radioactive wastes are generated by the chemical separations plant at Hanford, Washington. These wastes are stored for long periods in large underground tanks on the Hanford Project in accordance with the Waste Management Program being carried out by the Atlantic Richfield Hanford Company (ARHCO). The 241-SY tank farm is being built to expand the waste storage capacity at Hanford. The 241-SY tanks have to be designed to withstand all credible load conditions during their use and maintain integrity such that no leakage to the surrounding soil occurs. URS/John A. Blume & Associates, Engineers conducted a Phase I review of the tanks and presented the results of this review in a report (ARHCO Waste Storage Tanks, 241-SY Tank Farm, Phase I Review) dated February 26, 1974. The report presented a qualitative analysis of the tank structure, reviewed the applicability of the seismic analysis of 241-AZ tanks that are identical to the 241-SY tanks, and recommended a detailed (Phase II) analysis required for the 241-SY tanks. URS/Blume was then contracted by ARHCO to make a detailed engineering analysis study as recommended in the Phase I review report, of the proposed 241-SY tank structure for long-term dead, live, and thermal loads plus the Safe Shutdown Earthquake (SSE) ground motions. The purpose of this report is to describe and present the results of this study.

1.2 Description of Tanks

The proposed 241-SY tank farm consists of three cylindrical, dome-roofed tanks (tanks 101, 102, and 103) and is located in the 200 West Area near the 242-S Evaporator Crystallizer building. The three tanks are essentially identical to each other and to the 241-AZ tanks¹ in the 200 East Area, located approximately 5 miles east of the 200 West Area. Figures 1 and 2 show the layout of the tanks and a vertical section through the tanks and the surrounding soil. Each tank is 56 feet high overall with a minimum height of overburden of 6-1/2 feet above its dome. The outside diameter of each tank is approximately 83 feet and there is a minimum of a 24-foot clear separation between the walls of adjacent tanks.

A half vertical cross section of one of the tanks is shown in Figure 3. The tank provides dual containment for the liquid waste. The primary containment is provided by a primary steel tank with plate thickness varying from 3/8 inch in the dome region to 7/8 inch in the base knuckle region. The secondary containment is provided by a reinforced concrete tank with a steel plate liner on the inside face. The concrete tank wall is 18 inches thick, and the thickness of the base slab is generally 11-1/2 inches with a increase to 24 inches in the circumferential and central areas. The secondary steel liner is 3/8-inch thick. The concrete cylindrical wall-to-base slab connection is of the sliding type with a curb on the exterior side of the base slab limiting the motion of the tank wall. The concrete dome is an ellipsoid, having a major diameter of 80 feet and a minor diameter of 30 feet on the inside face. The thickness of the concrete dome varies from approximately 30 inches near the wall to 15 inches at the crown. There are several penetrations in the dome roof of the tank, ranging in diameter from 4 inches to 42 inches, required for various purposes. The primary and secondary tanks are separated by a 30-inch annulus of air in the cylindrical region and by an 8-inch-thick layer of insulating concrete in the base region. In the dome region, the primary steel tank is directly attached to the secondary concrete tank forming a liner of the latter. Further details of the tank structure may be obtained from Reference 2 drawings.

1.3 Scope of Investigation

The basic purpose of the present investigation was to determine the combined effects of long-term dead, live, and thermal loads and the SSE ground motions on the proposed 241-SY Waste Tank Structure. To analyze the structure for each of the above load conditions, an axisymmetric finite element model of the tank, with or without the surrounding soil, was used. The axisymmetric finite element model of the tank with the surrounding soils consists of shell and solid elements. Although the soil extends infinitely around the tank, the boundaries of the model were taken only sufficiently far from the tank as to affect the results of the analysis in and near the tank minimally. The model was used for the dynamic analysis of the tank-soil interaction during an earthquake as well as for the static analysis under gravity loads.

The primary steel tank alone, modeled by axisymmetric finite elements, was used for hydromechanical analyses. Stresses in the primary steel tank due

to thermal loads were also computed using this model.

The seismic analysis was conducted in two parts. In one part the interaction of the empty tank with the surrounding soil was considered; in the other the effect of the sloshing liquid on the tank was examined. It was assumed that the responses of the tank to these two earthquake-induced phenomena are not coupled and may be computed separately. The hydrodynamic effect of sloshing fluid was considered approximately by a quasistatic approach. The problem of interaction of tank and soil during earthquake motion was treated as a dynamic problem using a time-history approach. In this dynamic analysis, the time-history of responses was computed for the soil-tank finite element model subjected to a time-history of earthquake motion at its base. The design earthquake (SSE) has been specified as a time-history of free-field ground motion at the surface. The equivalent SSE motions at the base of the soil-tank model were computed by a deconvolution procedure.

A gravity load analysis was also carried out with the same soil-tank model used in the seismic analysis with some minor changes. The gravity load condition includes the dead load of tank and soil, live load (surcharge) and weight of tank contents.

A separate nonlinear thermal-creep analysis of the concrete tank was carried out by our consultant Dr. Y. R. Rashid. A finite element model of the concrete tank and dome, including the secondary steel liner, was used for this analysis. Gravity load (overburden and lateral soil pressures) was also included as the initial mechanical load on the tank. Dr. Rashid's report on this analysis is included as Appendix A of this report.

1.4 Safe Shutdown Earthquake (SSE)

The 241-SY tanks are located in the 200 West Area, approximately 5 miles west of the 241-AZ tanks in the 200 East Area. A site seismicity study for the 241-AZ tanks is reported in the Appendix of Reference 1. Due to the proximity of the two sites, the SSE (referred to as Design Earthquake in Reference 1) proposed for the 241-AZ tanks was considered applicable to the 241-SY tank site. The design response spectrum of the SSE for 5% damping is shown in Figure 4. A synthetic free-field ground acceleration time-history correlated to the SSE design response spectrum has been developed by URS/John

A. Blume & Associates, Engineers, and is shown in Figure 5. The 5% damping response spectrum of the synthetic time-history is compared with the corresponding SSE design spectrum in Figure 4.

2. SOIL PROPERTIES

A subsurface geophysical investigation was conducted at the 241-SY tank site to determine the dynamic soil properties. The results of that study are reported in Reference 3. Soil properties to a depth of 150 feet from the surface were obtained. Data beyond that depth were developed from other available geologic data.⁴ From these data, it is believed that the surface of the Ringold formation is at a depth of approximately 155 feet. The Ringold formation consists of cemented and very dense conglomerate whose stiffness is estimated to be almost 20 times that of the soil layers (silty sand) just above. Thus the Ringold formation may be interpreted as bedrock for the purpose of analysis and was taken as the base of the soil-tank model.

It has been established by many investigators⁵ that soil behavior under earthquake forces is nonlinear, i.e., the stress-strain relations are not linearly proportional. However, a direct nonlinear dynamic analysis of the soil-tank structure was beyond the scope of this investigation. Instead, an equivalent linear dynamic analysis approach⁶ was used. In this approach, soil moduli are assumed to depend on the average maximum strain levels occurring over the duration of earthquake motion. Thus, an initial guess is made at these strain levels, and the soil properties based on them are used in the first analysis. If the assumed strain levels are much different from the computed values, the latter are then used as the basis for soil properties for the next analysis, and so on. Such an approach has been shown to lead to results comparable to those obtained from direct nonlinear analysis.⁶

The shear modulus data given in Figure 5 of Reference 3 are obtained from geophysical tests where very low strain levels ($10^{-4}\%$) are involved. Figure 6 of Reference 3 gives the variation of shear modulus, in a non-dimensional form, with shear strain levels. The data on variation of damping with strain levels for sandy material given in Reference 5 were also used in the present analysis.

3. COMPUTATION OF BASE MOTION (DECONVOLUTION)

The SSE has been specified in terms of free-field spectra (Figure 4) and was represented by synthetic free-field acceleration time-history (Figure 5). To evaluate the response during the SSE of an underground tank and the soil surrounding it, the input motion must be applied at the base of the soil-tank model. This base motion should be equivalent to the specified surface motion.

Such a computation of the motion at the base of soil layers that will produce a given surface response motion is the reverse of the usual problem of computing the surface response of soil layers subjected to a given base motion. It is difficult if not impossible to solve such a problem in the time domain. However, for stable linear elastic systems, there exists a simple relationship between the output motion (motion at the surface or any intermediate point) and the input (base) motion in the frequency domain:

$$Y(\omega) = H(\omega) \cdot X(\omega)$$

where $Y(\omega)$ and $X(\omega)$ are the Fourier transforms of the output motion $y(t)$ and input motion $x(t)$, respectively, and $H(\omega)$ is the frequency response function of the system and is solely dependent on the system properties. Here, $Y(\omega)$ can be computed because the surface motion $y(t)$ is known. $H(\omega)$ is simply the Fourier transform of the surface response of the soil layers to unit impulse input at the base. Thus, $X(\omega)$ may be computed from the above equation and then $x(t)$ may be obtained by inverse Fourier transform of $X(\omega)$.

The 155-foot deep soil layers down to the Ringold formation (which was taken as the bedrock base of the soil-tank model as explained in Section 2) were modeled by a soil column of unit area for the deconvolution analysis described above to obtain the base motion time-history at the surface of Ringold formation. The acceleration time-history at the surface of the soil column was recomputed using the computed base motion as input. The acceleration response spectra of the recomputed surface motion and the original free-field motion are compared in Figure 6.

The soil properties used in the deconvolution analysis are shown in Table 2. As explained in Section 2, the soil properties are strain dependent; consequently, an iterative elastic approach was used. The strain levels are also shown in Table 2. The computed base acceleration time-history is shown in Figure 7.

4. ANALYSIS APPROACH

4.1 General Assumptions

Structural analysis of underground tanks with liquid contents subjected to long-term gravity, thermal, and creep loads, as well as the transient earthquake ground motion is a complex problem. The recent development of the finite element method of analysis of structures has made such analyses possible. However, a few simplifying assumptions regarding the structure and the loads were necessary to make such analyses practicable within reasonable budget limits. Two basic assumptions made in the present analysis are outlined below. Several other assumptions will be mentioned in the remainder of this section.

Each tank was assumed to be axisymmetric. The nonaxisymmetric features of the tank were either ignored or modeled approximately by equivalent axisymmetric elements. The soil surrounding the tank was also assumed to extend axisymmetrically for a large radius (large in relation to tank size). The effects of neighboring tanks and nonaxisymmetric soil profile were ignored. It is difficult to estimate the effect of this assumption on the computed responses, especially in the case of earthquake forces. When considering gravity loads, it seems that ignoring the existence of adjacent tanks will lead to conservative results with regard to lateral soil pressures on the concrete tank walls; effects elsewhere should be minimal. Also, as will be observed from the results presented later in this report, the earthquake-generated stresses are generally much smaller than those caused by static loads. The computed results should, therefore, be conservative.

The hydrodynamic problem of liquid pressures on the tank due to oscillating liquid contents was solved separately from the soil-tank interaction problem. That is, the coupling between the liquid motion and the tank motion was assumed to be negligible because the frequency differences of the two motions are large. Hydrodynamic forces computed on the basis of this assumption will be quite conservative.⁷

4.2 Finite Element Models

The tank and the surrounding soil were modeled as an assemblage of axisymmetric, thin, conical shell and toroidal solid finite elements. The shell element has, in general, the shape of a conical frustum, but can also represent a segment of a vertical cylindrical shell and a flat, horizontally oriented plate at extreme orientations. The solid elements are toroids of triangular or quadrilateral cross section. A shell element and a quadrilateral solid element are shown in Figure 8 along with the global reference axes (Z , R , and ϕ) and the degrees of freedom involved at each nodal point. For a description of the finite element method, see Reference 8. Reference 9 develops the finite element method for axisymmetric structures under axisymmetric and nonaxisymmetric loading, the particular area of interest in the present investigation.

The entire soil-tank finite element model is shown in Figure 9. Detail near the tank region is shown in Figures 10a and 10b. The concrete tank, including the dome and the base slab, was represented by thin shell elements. The secondary steel liner attached to the concrete tank was ignored, because it will contribute very little to the stiffness of the shell. The primary steel tank was included in the model and was also represented by thin shell elements. The primary steel tank was assumed to be rigidly attached to the concrete shell in the dome region. In the base slab region, the layer of insulating concrete between the primary steel tank and the concrete tank was modeled by solid elements. The primary tank was assumed to rigidly attached to the insulating concrete. This will be the behavior in reality because of self weight of the tank and the weight of the contents.

The concrete tank dome, wall, and slab were represented by shell elements at their middle surfaces. To maintain actual geometry, the nodal points of the shell elements on the middle surface and the nodal points of the adjacent solid soil elements at the soil-tank boundary were connected by thin shell elements of appropriate length (usually half the thickness of the concrete wall or slab), stiffness, and weightless material. Similar solid connector elements were used in the cantilever portion of the wall footing to maintain its actual geometry. For dead load analysis, the shell connectors between concrete walls and adjacent soil were assumed to be pin-connected to elimi-

nate the drag-down effect (transfer of weight of soil) on the concrete wall from the surrounding soil. This assumption closely simulates actual conditions, because the drag-down effect due to consolidation of the fill around the tanks diminishes with time after initial construction.

Except for the materials used to define the properties of the cracked concrete elements in the dome for the dynamic analysis, all materials, i.e., soil, concrete, insulating concrete, steel, etc., were assumed homogeneous and isotropic for a given analysis run. As indicated in Section 2, the soil properties were assumed to be dependent on expected strain levels. The properties used in the analysis for various materials are given in Table 3.

Results of the thermal-creep analysis by Dr. Rashid show some cracking near the center and in the haunched portion of the dome. The cracking is due to tensile forces in the circumferential direction. Consequently, in the dynamic analysis, the cracked concrete was modeled by material having equivalent orthotropic properties with reduced stiffness of only the hoop reinforcement in the circumferential direction and the stiffness of the concrete in the other directions.

Only a finite extent of soil around the tank could be included in the model. The bottom boundary of the model was assumed to be 155 feet below the ground surface. The vertical boundary was assumed to be at a radius of 240 feet from the axis of symmetry of the soil-tank model. The bottom boundary was at the surface of the Ringold formation, which, as explained in Section 2, consists of a much stiffer material than the soil above and thus forms a natural boundary. The vertical boundary was somewhat artificial; however, it was sufficiently far from the tank region that the stresses and displacements in this region were little affected by the artificial boundary. Furthermore, to simulate continuity at the vertical boundary, appropriate boundary conditions were assumed: lateral support but freedom of vertical movement under vertical loads, and exactly the reverse under lateral loads. Also, in the earthquake analysis, dampers were used at the vertical boundary to simulate radiation damping¹⁰ (loss of energy due to radiation of wave energy into soil beyond the boundary). The bottom boundary was assumed to be fixed for all load cases.

No geophysical test data for the tank site were available at the start of the present investigation, so initially the soil-tank finite element model was made to extend down to 200 feet below the surface. The model had been finalized and the input data cards had been punched when the geophysical investigation results were obtained. The very dense and cemented Ringold formation is believed to be 155 feet below the surface. It was decided, as explained in Section 2, that the surface of this formation forms a natural bottom boundary. To avoid delaying the present investigation, instead of developing a new model, all nodal points below the 155-foot level were assumed fixed. The soil-tank model given in Figure 9 shows the model down to the 155-foot level only.

A separate model of the primary steel tank was used for analysis under hydro-mechanical pressures that act directly on the primary tank and have very little effect on the secondary tank. Also, a more refined model of the primary tank was made possible by analyzing it independently. The model used in hydromechanical analysis is shown in Figure 12, with the tank liner plate represented by axisymmetric shell elements. The thermal stress analysis was carried out with a separate computer program (see Section 4.4) whose limitations made it necessary to model the tank liner plate by axisymmetric solid elements (5 layers across the thickness).

A separate model of the concrete tank wall and dome was used in thermal-creep analysis conducted by Dr. Rashid and is described in Appendix A.

The wall-to-base slab joint in the concrete tank is of the sliding type. The wall was therefore assumed to be free to move laterally with respect to the base slab under long term loads such as gravity, lateral soil pressure and thermal loads and under creep effects. However, for the transient earthquake ground motion loading, which generally lasts for less than a minute, the wall and the base slab were assumed to be pin-connected, i.e., free to rotate but prevented from sliding with respect to each other.

4.3 Load Conditions for Analyses

In addition to the two dynamic load cases, i.e., hydrodynamic loads and earthquake ground motion, the gravity (i.e., dead and live), hydrostatic, thermal,

and vapor pressure (in the primary tank vapor space) loads must also be considered to obtain the proper combined stress for checking the steel and concrete tank sections. A separate thermal-creep analysis of the concrete tank was conducted by Dr. Y. R. Rashid.

4.3.1 Gravity Loads

Under gravity loads, the weight of the tank and the soil in the model were considered. The weight of the 1,000,000 gallons of liquid waste (specific gravity of 1.7) was also considered as was the live load at the ground surface, which consists of a uniform load of 40 psf and a concentrated load of 50 tons. The 50-ton concentrated load was applied only in the central region at the axis of symmetry. Its application elsewhere makes it a nonaxisymmetric load which cannot be easily modeled by equivalent axisymmetric loads. The soil-tank model was used in the analysis for gravity loads.

4.3.2 Hydromechanical Loads

The liquid waste to be contained in the primary steel tank will exert hydrostatic pressures on that tank. Vapor in the space above the liquid surface will also exert pressures on the tank dome. These pressures are long-term in nature. The motion of the tank under seismic excitation will cause sloshing of the liquid in the tank, resulting in hydrodynamic pressures on the tank. Such hydrodynamic pressures will be exerted only for a short time during an earthquake.

The hydromechanical pressures act directly on the primary steel tank, which then transfers them to the surrounding soil through connections to the concrete tank dome and base slab. The effect of these loads on the concrete tank and soil is expected to be minimal. Thus, the primary tanks were analyzed under such loads under the assumption that the dome and base slab of the concrete tank provide it rigid supports. An analysis of the entire soil-tank model under hydrostatic loads confirmed this assumption.

In computing the hydrostatic pressures, the specific gravity of the liquid contents was assumed to be 1.7. Vapor pressure has been specified as ranging between -6 inches of water to +60 inches of water. Vapor pressure of +60

inches of water acts in the same direction as the hydrostatic pressures and the weight of liquid, and was therefore used in the analysis simultaneously with the hydrostatic pressures.

Hydrodynamic pressures caused by the sloshing motion of the liquid contents were computed by a method developed in Reference 7 that is essentially similar to that developed by Housner.¹¹ This method assumes that the tank is rigid; the liquid is nonviscous, incompressible, and homogeneous; the displacements, velocities, and slopes of the free surface of the liquid are all small; and the flow field is irrotational. From the expression for hydrodynamic pressure developed in Reference 7, using only the first sloshing mode and assuming that the spectral acceleration value at the first mode period (which is greater than 5 seconds) equals half the maximum ground acceleration, $\ddot{u}_g$, the pressure on the tank wall at radius R is computed to be

$$p(z, \phi) = \rho R \ddot{u}_g \{1 - f_1(R, z)\} \cos \phi$$

where

$$f_1(R, z) = \frac{\cosh 1.84 \frac{H-z}{R}}{2.38 \cosh \frac{1.84H}{R}}$$

In the above expression, H is the height of liquid surface from the base of the tank, z is the coordinate of depth measured from the surface of the liquid (positive downward), ρ is the mass density of the liquid, and ϕ is the circumferential coordinate (angle) measured from the direction of application for the base motion.

The model of the primary steel tank used in the hydrostatic and hydrodynamic analyses is shown in Figure 12. The hydrodynamic pressures on the tank computed by the above procedure are given in Table 4 along with the hydrostatic pressures.

4.3.3 Thermal Loads

The liquid contents in the tank will remain at temperatures of around 250°F over a long period. The steel and concrete tanks are constructed and remain at normal atmospheric temperatures (assumed to be 70°F) until the introduction of the hot liquid waste contents. The base plate and most of the cylindrical wall region of the primary steel tank that is in direct contact with the liquid will reach and remain at the same temperature as that of the liquid. Temperatures in the remainder of the steel tank and those in the secondary concrete tank were computed by ARHCO using heat transfer analysis.

Concrete is subject to creep under sustained loading, especially under elevated temperature conditions. The thermal-creep analysis of the concrete tank that was conducted by Dr. Y. R. Rashid used the temperature data developed by ARHCO. The temperatures were assumed to rise gradually over a period of 30 days to the design levels and then remain steady. The analysis was continued to further steps in time well beyond the time necessary to establish self-limiting cracking and creep deformations. The soil overburden load, live load of 40 psf, and internal vapor pressures in the tank were included in the analysis as initial mechanical loading on the tank. This detailed report on the thermal-creep analysis is included as Appendix A.

The detached portion of the primary steel tank (Figure 12) was analyzed for thermal stresses due to temperature change. Simultaneously, concrete tank displacements at the dome and the base slab regions where the two tanks are connected were applied to the steel tank. The displacements of the concrete tank in the dome region were obtained from the results of the thermal creep analysis. The displacements in the base slab were computed by the formula

$$\Delta = \alpha \cdot R \cdot \Delta T$$

where Δ is the radial displacement at radius R , α is the coefficient of expansion for concrete (6.38×10^{-6} in./in./°F) and ΔT is the change in temperature.

4.3.4 Earthquake Ground Motion

Ground motion at a given point due to an earthquake may be represented by two horizontal components and one vertical component. The rotational components are negligible. The soil-tank structure is assumed to be rigid in the vertical direction, and the responses due to vertical ground motion are obtained by scaling the gravity load responses by a factor of 0.167 (the design SSE maximum vertical ground acceleration specified is $2/3 \times$ maximum horizontal ground acceleration, $0.25g = 0.167g$).

The response of the soil-tank structure to a horizontal component of SSE input base motion time-history was computed. This input base motion was determined as described in Section 3. The equations of motion for the soil-tank model were solved by a step-by-step method, and the entire time-histories of the responses were computed. Only the maximum values of the response parameters were recorded (with the time of occurrence), mainly because of the large number of response parameters of interest. Also, instead of computing the responses of the system to two independent, mutually perpendicular horizontal components, it was assumed that they cause the same maximum responses, though they may occur at different times.

4.4 Computer Programs

The main computer program used for the analyses reported herein was AXIDYN, a computer program for the static and dynamic analysis of axisymmetric structures by the finite element method. The program, written in Fortran IV, was developed at the Department of Civil Engineering of the University of California, Berkeley,⁹ under the direction of Professor E. Wilson, and was used in all the finite element analyses reported here.

The structure to be analyzed by AXIDYN can consist of axisymmetric shells, axisymmetric solids, or a combination of the two. The axisymmetric shells are represented by conical-frustum-shaped shell elements, whereas the axisymmetric solid body is represented by an assemblage of toroids of quadrilateral or triangular sections. The two types of elements are shown in Figure 8 with the degrees of freedom considered in the analysis for such elements. The nodal points on a vertical section are seen in Figure 8 to be nodal circles.

The program will handle static or dynamic forces that are axisymmetric or are such that they can be developed into Fourier series form along the nodal circles, i.e., as functions of ϕ . AXIDYN accepts five types of loading cases:

- Dead loads
- Arbitrary static loads
- Arbitrary dynamic loads
- Horizontal earthquake accelerations
- Vertical earthquake accelerations

Dead loads, arbitrary static loads, and horizontal earthquake accelerations are the load cases considered in the present analysis.

The program has the capability of performing dynamic analyses both by the modal superposition approach and by direct integration of the coupled equations of motion. The latter approach was used in the current investigation. A new subroutine, developed for the step-by-step integration of equations by a procedure¹² that is more accurate and stable than the procedure originally developed for the CDC 6600 computer system, was modified to handle large finite element models encountered in the present analyses on the CDC 7600 computer system. Such a system at the Lawrence Berkeley Laboratory of the University of California at Berkeley was used for all computer analyses.

Other computer programs were also used for preparation, checking, and plotting of data, and for the deconvolution analysis described in Section 3. Among these is a program named MATRAN, which performs matrix operations on data in matrix form, including structural analyses and time-series analyses. The latter capability was particularly useful for the deconvolution analysis. MATRAN is a substantially expanded version of program SMIS. The latter program was also developed at the Civil Engineering Department of the University of California, Berkeley.

Another program, SAP IV, also developed by the Civil Engineering Department of the University of California at Berkeley,¹³ was used in the thermal analysis of the primary steel tank. SAP IV is a general three-dimensional struc-

tural analysis program and was used instead of AXIDYN because the latter does not handle thermal loads.

4.5 Load Combinations and Computation of Stresses

As explained in Section 4.2, the tanks were entirely modeled by shell elements. The computer output of forces on shell elements is in terms of stress resultants, longitudinal force and moment, circumferential force and moment, and in-plane shear force and torsional moment. These stress resultants are shown in Figure 13 for a vertically oriented shell element, which represents tank walls. Tensile forces and moments producing tension on the outside face of a shell element are considered positive. This convention is followed in the figures and tables of this report. For a horizontally oriented shell element, representing the base slab of the tank, the stress resultants shown in Figure 13 are still valid except that the longitudinal forces may be interpreted as the radial forces. Note that no transverse shear (i.e., shear across the shell thickness) is output by the computer program; it may be approximately computed from the longitudinal and circumferential moments.

Stresses from the three sets of stress resultants -- longitudinal force and moment, circumferential force and moment, and shear and torsional moment -- are computed on appropriate sections of the elements. First, though, the stress resultants from various loads were combined and the following considerations were taken into account in this combination process.

Among the load cases considered here (Section 4.4), two loads -- hydrodynamic pressures and horizontal earthquake ground motion -- are nonaxisymmetric. The hydrodynamic pressures act normal to the container wall, and the magnitude varies over the circumference as a cosine function of the angle ϕ from the direction of applied ground motion (see Section 4.3.2). A component of horizontal earthquake ground motion may be represented by radial and tangential loads (in the horizontal plane) varying over the circumference as cosine and sine functions, respectively, of the angle ϕ measured from the direction of applied ground motion. For both of these nonaxisymmetric loadings the longitudinal and circumferential stress resultants vary cosinusoidally, whereas the in-plane shear and torsion vary sinusoidally, from the direction of applied ground motion. The other three loads -- gravity load, hydrostatic

pressures, and vertical earthquake ground motion -- are axisymmetric. Stress resultants on an element due to these loads are constant over the circumference, and in particular the in-plane shear and torsion are zero everywhere.

The seismic design criteria specify that the horizontal ground motion shall be represented by two mutually perpendicular components acting simultaneously. The maximum values of a stress resultant in an element due to the two mutually perpendicular horizontal components will occur at 90° angles and probably at different times. Let us assume that the magnitudes of a stress resultant due to the two components are equal. If we combine the particular response due to the two components by the root mean square method, it is clear that at no point along the circumference will the combined value of the response be greater than the response due to one component at a section parallel to that component. The same argument holds for hydrodynamic loads because they are generated by horizontal earthquake ground motion. Therefore, in obtaining the combined stress resultant from dynamic and static loads, one needs to consider only one component of the horizontal ground motion. Furthermore, according to the argument presented in the preceding paragraph, in combining the stress resultants from axisymmetric and nonaxisymmetric loads included in the present analysis, the circumferential variation of these stress resultants need not be considered.

For the static loadings (dead load and hydrostatic pressures) the direction of the action of stress resultants is known from the computer output. However, for dynamic loadings (vertical and horizontal earthquake ground motions and hydrodynamic pressures) only the maximum values of the various stress resultants are known. These may act in either direction and may occur at different times during the earthquake. Hence, algebraic summation was used in combining stress resultants from the static load cases, whereas absolute summation was conservatively used in combining stress resultants from the dynamic load cases. However, in combining the total static and the total dynamic forces and moments, directions (or signs) were assigned to the dynamic forces and moments independently to produce the worst possible stresses.

In determining the stresses in the primary steel tank, the results of our analyses considering the various load conditions, namely dead, surcharge,

thermal, hydromechanical loads, and earthquake ground motion are considered. The thermal-creep analysis by Dr. Y. R. Rashid, which used a nonlinear method of analysis, included the long-term loads, consisting of dead, surcharge, and vapor pressure at the inner surface of the dome, as initial mechanical loading on the tank. The results of this thermal-creep analysis and the results of our earthquake analysis of the tank as a whole were combined to check the stresses at the concrete tank. Effects of the hydrodynamic loads (which act directly on the primary steel tank) on the concrete tank are small and are not considered.

5. RESULTS OF ANALYSIS

5.1 Primary Steel Tank

The primary steel tank was analyzed as a part of the overall soil-tank model (Figure 9) for gravity loads and earthquake ground motion. It was analyzed separately for hydrostatic and hydrodynamic loads (Figure 12) as well as thermal loads.

Internal forces (i.e., both forces and moments) in the primary tank caused by the hydromechanical loads are plotted in Figures 15 and 16. For convenience in plotting, the top knuckle and roof portion and the base knuckle and base plate portion have been developed vertically in these figures. The hydrostatic pressures and corresponding internal forces are constant over the circumference of the tank. The hydrodynamic pressures, which are caused by earthquake motion, and the corresponding internal forces vary along the circumference with the cosine of the angle measured from the direction of applied motion. The values of hydrodynamic internal forces shown in Figures 15 and 16 are maximum values acting on a section parallel to the applied motion. The hydrodynamic internal forces are generally much smaller than the hydrostatic internal forces.

The longitudinal (or meridional) and circumferential (or hoop) moments due to hydromechanical loads are significant only at and near the top and bottom knuckles. The longitudinal forces are generally of smaller magnitude in comparison to circumferential forces. The latter are maximum near the base of the tank for hydrostatic load and are essentially constant over the cylindrical portion of the tank for hydrodynamic load. Note that the hoop forces caused by hydrostatic pressures are generally tensile, whereas the hoop forces due to hydrodynamic pressures may act either way.

The internal forces in the primary tank caused by gravity loads and soil-structure interaction due to earthquake ground motion are generally much smaller than those caused by hydromechanical loads, and are listed in Tables 5 and 6. The moments and forces are of any significance only at and near the base knuckle and to a lesser extent near the top knuckle. The moments

are more significant in the longitudinal direction and the forces are more significant in the circumferential direction. Note that the gravity loads and corresponding internal forces are constant along the circumference, whereas the earthquake-caused internal forces vary with the cosine of the angle measured from the direction of applied motion, and the values given in Tables 5 and 6 are the maximum values. Also note that the earthquake-induced internal forces may act in either direction.

Due to the nature of the model and computer program used, only the stresses on the primary tank for thermal loads were available. The stresses are insignificant everywhere except in and near the base and top knuckles of the tank. In the upper knuckle, the maximum longitudinal and circumferential stresses are, respectively, 9.6 ksi (compressive) and 3.9 ksi (tensile). For the bottom knuckle, the corresponding maximum stresses are 4.0 ksi and 1.6 ksi (both tensile).

Stresses due to gravity and hydromechanical loads and earthquake ground motion were also computed. The circumferential stress is dominated by hoop tensile force caused by hydrostatic pressures and is maximum at nodal point (NP) 235, where the thickness of tank liner plate is 1/2 inch. Under long term loads (gravity + hydrostatic + thermal) the maximum hoop stress at NP 235 is 15.3 ksi, and addition of seismic load stresses boosts it to 20.1 ksi. The longitudinal stress under long term loads is maximum at NP 133 (at the base of the base knuckle, see Figure 12) having a value of 13.1 ksi, which increases to 19.3 ksi when seismic load stresses are added.

The primary tank steel is specified as ASTM 516 Grade 65. According to ASME Boiler & Pressure Vessel Code Section III Division I,¹⁴ the allowable stress for such steel at temperatures around 250°F is 21 ksi (and the yield stress is 31.5 ksi). Thus, the maximum computed stresses in the primary steel tank are less than this allowable value.

The radial displacements of the primary steel tank under hydromechanical pressures are shown in Figure 14. Maximum displacements of 0.165 inches and 0.104 inches under hydrostatic and hydrodynamic pressures, respectively, occur near NP 15.

5.2 Concrete Tank

The secondary concrete tank was analyzed for gravity loads and soil-structure interaction (due to earthquake ground motion) as a part of the overall soil-tank model (Figure 9) using computer program AXIDYN. The dome and cylindrical wall portion of it was separately analyzed for thermal loads and creep by Dr. Rashid using the computer program SAFE-CRACK (Appendix A). This thermal-creep analysis considered the nonlinear behavior of concrete, and therefore the gravity loads (including the active lateral soil pressure on the tank wall) were included as initial mechanical loading in the thermal-creep analysis.

Hence, for the concrete tank dome and wall, the SAFE-CRACK analysis results were considered valid for the effect of long term loads (gravity, thermal, and creep) and AXIDYN analysis results valid for earthquake ground motion loading. However, for the base slab, only AXIDYN results were available for all loadings and are used herein. Thermal stresses in the base slab are expected to be small because the connection between the wall and the slab is of the sliding type. Furthermore, creep will tend to relieve the thermal stresses. Thus, the effect of thermal loads and creep on the base slab is considered negligible and is ignored.

The effect of hydromechanical loads, which act directly only on the primary steel tank, was found to be negligible on the secondary concrete tank and was ignored.

The lateral displacements in the tank region of the soil-tank model under earthquake motion are maximum at approximately the same time, 6.56 to 6.57 seconds after the start of earthquake motion. The maximum lateral displacements at the top and bottom of the concrete tank wall (nodal points 359 and 191, respectively) are 1.121 inches and 0.985 inches, respectively, giving a relative lateral displacement of 0.136 inches between the top and bottom of the tank wall.

The maximum lateral accelerations due to the earthquake motions are 0.15g at the base of the tank, 0.21g at the roof of the tank, and 0.295g at the soil surface close to the right vertical boundary.

The longitudinal (or meridional) forces and moments and the circumferential (or hoop) forces and moments on the concrete tank wall, roof slab, and base slab from the earthquake analysis are given in Table 7. The corresponding plots of these forces and moments are shown in Figures 17 and 18. The earthquake response plots in these figures are envelopes because maximum responses at different points in the structure may occur at different times and have either positive or negative values. The earthquake responses are plotted for a section parallel to the applied horizontal earthquake motion and are maximum. They vary circumferentially as the cosine of the angle measured from the direction of applied motion. The internal forces that develop in the concrete tank under earthquake loading are generally much smaller than the corresponding responses under dead load.

The results of thermal-creep analysis, including the effect of gravity loads of the concrete tank dome and wall, are discussed in Appendix A. The forces and moments on the base slab due to gravity loads, as obtained from AXIDYN analysis, are shown in Figure 19. The longitudinal moment, which results mainly from the vertical point load imposed by the tank wall, is maximum near the tank wall and is the most significant of all internal slab forces due to gravity loads.

The results from SAFE-CRACK analysis under initial mechanical loading (i.e., gravity and soil pressure) indicate an upward displacement of 0.03 inches at the dome crown and an inward horizontal motion of 0.12 inches of the base of the wall. Under thermal loading, just after the temperature of the contents has reached its maximum value (30 days after heating has started), the displacement at the crown is reversed and is now 0.51 inches downwards; similarly, the displacement at wall base has also reversed and is now 0.41 inches outward. Due to long-term creep effects, the displacement at the crown of the dome stabilizes to 0.23 inches upwards and at the base of slab to 0.34 inches outwards.

The results of SAFE-CRACK analysis indicate that under the gravity loads (initial mechanical loading), there is no cracking in concrete and the maximum compressive stress in concrete is less than 1000 psi. Maximum stress in meridional and hoop steel reinforcement is less than 7 ksi compressive and

1 ksi tensile. During heating, the analysis indicates, cracking in concrete begins, and by the time temperatures reach their peaks, radial and meridional cracking has occurred in regions near the dome crown and haunch. The maximum compressive stress in concrete is less than 1600 psi, except in a very localized region at the dome crown soffit, where it is 2600 psi. Maximum stresses in steel reinforcement are less than 24.2 ksi compressive and less than 7 ksi tensile. The analysis further indicates that after long-term creep, when stable conditions are established, the cracking spreads somewhat but remains in the same general region. The maximum compressive stress in concrete is now less than 1000 psi, whereas stresses in steel are less than 24.2 ksi compressive and 4.0 ksi tensile. The high compressive stress (24.2 ksi) in reinforcement after heating and long-term creep effects is confined to a very small area near the dome crown, being less than 13 ksi elsewhere.

The SAFE-CRACK analysis indicates that principal strains in regions of radial cracking and meridional cracking near the haunch (where diagonal reinforcement exists) are less than 1 percent, indicating adequate reinforcement. However, principal strains approach 10 percent in the meridionally cracked region near the dome crown, where no diagonal reinforcement exists, indicating that these cracks may increase in an unstable manner unless diagonal reinforcement is provided.

The seismic analysis of the tank by AXIDYN computer program was an elastic analysis, though regions of cracked concrete were modeled by appropriate elements with reduced stiffness, as explained in Section 4.2. Thus the results of seismic analysis cannot be directly combined to those from the nonlinear SAFE-CRACK analysis for long term loads. However, an indication of additional stresses due to seismic ground motion may be obtained by computing the stresses due to such loading independently. Such computations indicate maximum stresses (tensile or compressive) of 5.4 ksi in hoop steel in the cracked concrete region near the dome haunch. The maximum stresses in the uncracked region of the dome are less than 120 psi compressive in concrete and 1.7 ksi tensile or compressive in steel. In the tank wall, maximum compressive stress in concrete is less than 400 psi and maximum stress in reinforcement steel is 11.5 ksi (indicating some flexural cracking in concrete at the base of the wall in the longitudinal direction.

Stresses in the base slab for long-term and seismic loads were computed from the AXIDYN analysis results. The longitudinal stresses in a localized region near the tank wall location under long-term (viz. gravity) loading are quite high: 35.8 ksi tensile in bottom reinforcement steel (indicating some flexural cracking at the bottom face) and 2000 psi compressive in concrete. Addition of seismic stresses boosts these values to 42.7 ksi and 2475 psi, respectively. Elsewhere in the base slab, the stresses are substantially smaller.

The specified ultimate compressive strength (f'_c) of the tank concrete is 4500 psi. The allowable compressive stress in concrete, according to the old ACI 318-63¹⁵ code, are 2025 psi for long term loads and 2700 psi when seismic loads are also considered. Thus the compressive stresses in concrete are within allowable values everywhere in the tank except in a very localized region at the dome crown soffit for a short period during and after heating.

Reinforcement steel is specified as ASTM 615-68 Grade 60 with minimum yield strength of 60 ksi and allowable stresses of 24 ksi for long-term loads and 32 ksi when seismic stresses are included. The computed stresses in reinforcement steel exceed these allowable values in a couple of localized regions at the dome crown and in base slab near the tank wall, though they are well within the yield strength of the steel.

In-plane shear stresses caused by seismic ground motion result in high nominal shear stresses in the tank wall ranging from 252 psi to 395 psi, the latter occurring near the base of the wall. According to ACI 318-63, the allowable stress is 446 psi, assuming that longitudinal and circumferential steel reinforcement help resist the in-plane shear.

6. SUMMARY AND CONCLUSIONS

The underground 241-SY tank structure, which consists of a primary steel tank and a secondary concrete tank and is proposed to provide leakproof containment for radioactive waste liquids, has been analyzed for gravity and thermal loads, as well as for Safe Shutdown Earthquake ground motion. The secondary concrete tank has also been analyzed for long-term creep effects.

The tank structure was analyzed by the finite element method using an elastic analysis approach, except for the thermal-creep analysis of concrete tank which involved a nonlinear approach. The surrounding soil was included in the model for the seismic analysis.

An evaluation of the results indicates that the stresses in the primary steel tank are within their allowable values. The stresses in concrete and reinforcement steel of the concrete tank are also within their respective allowable values with the exception of a few localized areas. The stresses in concrete and steel reinforcement in a very small region at the dome crown exceed slightly their allowable values. The stress in bottom steel reinforcement in the base slab near its junction with the tank wall also exceeds the allowable value though it is well within its yield strength.

Cracking in the dome and upper wall regions of the concrete tank is expected under thermal and creep loads. Satisfactory reinforcement exists to limit this cracking except for certain regions where meridional cracking may increase in an unstable manner. To limit meridional cracking in the concrete tank, it is recommended that diagonal reinforcement be extended throughout the dome region and in the cylindrical wall region up to at least 8 feet below the dome-to-cylinder junction.

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TABLE 1
SOIL PROPERTIES BASED ON GEOPHYSICAL DATA

Depth, ft	Layer No.	Density, pcf	Shear Modulus, ksf	Poisson's Ratio, μ
5	1	108	1,900	0.23
	2	127	2,200	0.18
12	3	108	2,800	0.20
	4	108	3,500	0.28
36	5	108	4,100	0.34
	6	127	4,375	0.38
49	7	127	4,500	0.39
	8	115	4,600	0.41
63	9	115	4,670	0.42
	10	108	4,740	0.43
155	11	108	4,760	0.43
	12	108	4,875	0.44
	13	108	4,920	0.44
	14	108	5,060	0.44
	15	108	5,125	0.44
	16	108	5,240	0.44
	17	108	5,300	0.44
	18	108	5,450	0.42
	19	134	87,500	0.37
	20	142	90,000	0.37

TABLE 2
SOIL PROPERTIES USED IN DECONVOLUTION

Depth	Layer No.	Max. Shear Strain, %	Shear Modulus, ksf	Damping, %
5	1	0.0042	1710	2.8
	2	0.0159	1650	5.6
12	3	0.0289	1736	7.0
	4	0.0347	2012	7.0
36	5	0.0392	2194	7.0
	6	0.0469	2188	7.0
49	7	0.0537	2115	7.0
	8	0.0636	2024	7.0
63	9	0.0711	1985	7.0
	10	0.0787	1943	7.0
155	11	0.0885	1809	7.0
	12	0.0928	1828	7.0
	13	0.0966	1820	7.0
	14	0.0965	1877	7.0
	15	0.0909	1932	7.0
	16	0.0837	2044	7.0
	17	0.0703	2279	7.0
	18	0.0690	2371	7.0

TABLE 3
SOIL-TANK MODEL PROPERTIES

Material	Mass Density, lbs-sec ² /ft ⁴	Modulus of Elasticity, ksf	Poisson's Ratio, μ
Steel	15.217	396.72×10^4	0.300
Concrete	4.658	54.72×10^4	0.150
Insulating Concrete	0.932	2.88×10^4	0.150
Backfill - Layer 1	3.727	4561.0	0.200
Layer 2	3.727	4880.0	0.200
Layer 3	3.727	8527.0	0.200
Layer 4	3.727	12000.0	0.200
Soil - Layer 5	3.540	4755.0	0.230
Layer 6	3.665	5192.0	0.180
Layer 7	3.354	8890.0	0.270
Layer 8	3.944	12510.0	0.390
Layer 9	3.571	13348.0	0.420
Layer 10	3.354	13776.0	0.435
Layer 11	3.354	14976.0	0.440

TABLE 4
HYDROSTATIC AND HYDRODYNAMIC PRESSURES

Soil-Tank Model Node No.	Steel Tank Model Node No.	Coordinates, ft		Hydrostatic Pressure* ksf	Hydrodynamic Pressure ksf
		R	Z		
104	1	31.08	617.76	0	0
	2	32.29	617.76	0	0
119	3	33.50	617.76	0	0
133	4	36.50	617.76	3.521	0.787
147	5	37.00	617.90	3.507	0.808
161	6	37.37	618.26	3.468	0.802
175	7	37.50	618.76	3.415	0.814
190	8	37.50	619.74	3.305	0.813
205	9	37.50	620.83	3.195	0.812
220	10	37.50	622.83	2.983	0.809
	11	37.50	624.58	2.797	0.804
235	12	37.50	626.33	2.612	0.798
	13	37.50	628.08	2.426	0.791
250	14	37.50	629.83	2.240	0.782
	15	37.50	631.54	2.059	0.772
265	16	37.50	633.25	1.878	0.760
	17	37.50	635.25	1.666	0.744
280	18	37.50	637.25	1.454	0.726
	19	37.50	639.75	1.188	0.699
295	20	37.50	642.25	0.923	0.668
	21	37.50	644.25	0.711	0.640
310	22	37.50	646.25	0.499	0.608
	23	37.50	648.01	0.312	0.577
325	24	37.50	649.50	0.312	0
	25	37.50	650.50	0.312	0
340	26	37.50	651.50	0.312	0
349	27	37.50	652.60	0.312	0
358	28	37.50	653.70	0.312	0
366	29	37.40	654.59	0.312	0
375	30	37.11	655.43	0.312	0
383	31	36.63	656.19	0.312	0
391	32	36.00	656.82	0	0
	33	35.00	657.50	0	0
397	34	34.00	658.11	0	0

* Pressure includes 5-foot water pressure in vapor space

TABLE 5

LONGITUDINAL INTERNAL FORCES IN PRIMARY TANK

Soil-Tank Model Node No.	Coordinates		Hydrostatic		Hydrodynamic		Dead + Live Load		Earthquake	
	R ft	Z ft	Mom. kft/ft	Force k/ft	Mom. kft/ft	Force k/ft	Mom. kft/ft	Force k/ft	Mom. kft/ft	Force k/ft
104	31.08	617.76	0.016	4.30	0.000	0.45	-0.014	6.60	0.015	1.65
119	33.50	617.76	0.109	4.24	0.001	0.47	0.055	5.48	0.070	1.31
119	33.50	617.76	0.098	4.02	0.001	1.61	0.046	2.27	0.058	0.47
133	36.50	617.76	-0.597	3.56	-0.003	1.16	-0.530	-0.36	0.653	2.10
147	37.00	617.90	-0.489	2.72	-0.260	1.46	0.427	-2.80	0.244	4.12
161	37.37	618.26	-0.253	2.85	-0.308	1.45	0.766	-2.22	0.650	3.67
175	37.50	618.76	0.156	1.35	-0.125	0.95	0.527	-1.52	0.522	2.12
190	37.50	619.80	0.629	5.18	0.188	2.55	-0.032	-2.09	0.055	1.44
205	37.50	620.83	0.661	5.28	0.224	3.55	-0.113	-3.91	0.064	2.89
205	37.50	620.83	0.022	3.58	0.037	1.94	-0.097	-0.11	0.057	0.53
220	37.50	622.83	0.343	3.64	0.013	3.04	-0.004	-2.34	0.014	1.71
235	37.50	626.33	0.026	1.83	-0.022	3.22	0.004	-1.91	0.005	1.26
235	37.50	626.33	-0.021	4.63	0.003	3.80	0.000	-1.90	0.000	1.02
250	37.50	629.83	-0.053	3.65	-0.020	4.14	-0.001	-1.87	0.001	0.90
265	37.50	633.25	-0.002	3.44	0.000	4.15	0.000	-1.79	0.001	0.78
280	37.50	637.25	0.007	3.64	0.004	4.11	0.000	-1.71	0.001	0.27
295	37.50	642.25	0.000	3.13	0.003	3.54	0.000	-1.61	0.000	0.31
310	37.50	646.25	-0.012	2.96	0.033	3.40	0.000	-1.54	0.001	0.58
325	37.50	649.50	-0.018	4.20	-0.043	0.66	0.000	-1.50	0.001	0.55
325	37.50	649.50	0.012	2.92	-0.020	2.26	0.001	-1.48	0.001	0.74
340	37.50	651.50	0.006	3.60	0.004	1.69	-0.002	-1.56	0.001	0.92
349	37.50	652.60	0.056	3.67	0.015	1.39	-0.022	-1.53	0.018	1.05
358	37.50	653.70	0.025	3.77	-0.004	1.08	-0.004	-0.04	0.007	2.03
366	37.40	654.59	-0.024	3.07	-0.016	0.73	0.022	-1.17	0.021	1.56
375	37.11	655.43	-0.105	2.54	-0.014	0.62	0.043	-1.12	0.057	1.33
383	36.63	656.19	-0.184	3.17	-0.013	0.60	0.045	-1.59	0.056	3.31
391	36.00	656.82	0.254	5.44	0.029	0.60	-0.148	-2.79	0.213	2.90

TABLE 6

CIRCUMFERENTIAL INTERNAL FORCES IN PRIMARY TANK

Soil-Tank Model Node No.	Coordinates		Hydrostatic		Hydrodynamic		Dead + Live Load		Earthquake	
	R ft	Z ft	Mom. kft/ft	Force k/ft	Mom. kft/ft	Force k/ft	Mom. kft/ft	Force k/ft	Mom. kft/ft	Force k/ft
104	31.08	617.76	0.005	1.29	0.000	0.14	-0.005	5.08	0.005	6.15
119	33.50	617.76	0.034	1.55	0.000	- 2.63	0.017	4.85	0.022	6.73
119	33.50	617.76	0.035	1.69	0.000	- 4.37	0.016	6.28	0.025	12.31
133	36.50	617.76	-0.192	1.80	-0.001	- 7.08	-0.175	5.06	0.212	13.29
147	37.00	617.90	-0.164	6.72	-0.080	- 7.18	0.115	9.37	0.058	9.04
161	37.37	618.26	-0.088	25.39	-0.095	- 4.71	0.226	19.92	0.189	4.32
175	37.50	618.76	0.044	53.23	-0.038	2.35	0.158	24.83	0.156	13.04
190	37.50	619.74	0.189	104.22	0.057	21.09	-0.009	15.97	0.017	12.24
205	37.50	620.83	0.198	122.11	0.067	31.46	-0.034	4.12	0.019	4.57
205	37.50	620.83	0.007	104.41	0.011	26.64	-0.028	4.27	0.017	4.50
220	37.50	622.83	0.010	104.25	0.004	30.48	-0.001	0.93	0.003	0.48
235	37.50	626.33	-0.008	89.77	0.000	29.25	0.001	0.08	0.002	0.21
235	37.50	626.33	0.006	90.61	0.001	29.42	0.000	0.08	0.000	0.27
250	37.50	629.83	-0.016	78.58	-0.006	29.57	0.000	0.01	0.000	0.23
265	37.50	633.25	-0.001	63.37	0.000	28.04	0.000	0.01	0.000	0.16
280	37.50	637.25	0.002	45.83	0.001	26.49	0.000	0.00	0.000	0.20
295	37.50	642.25	0.000	25.73	0.001	24.00	0.000	0.00	0.000	0.20
310	37.50	646.25	-0.004	11.00	0.010	21.26	0.000	- 0.01	0.000	0.18
325	37.50	649.50	-0.005	13.74	-0.013	0.39	0.000	0.08	0.000	0.10
325	37.50	649.50	0.004	10.25	-0.006	0.82	0.000	- 0.03	0.000	0.16
340	37.50	651.50	0.002	12.60	0.001	0.09	0.000	- 0.22	0.000	0.47
349	37.50	652.60	0.017	7.64	0.004	- 1.46	-0.007	1.82	0.055	1.23
358	37.50	653.70	0.007	- 4.08	-0.001	- 4.89	-0.001	6.96	0.002	5.45
366	37.40	654.59	-0.008	- 22.52	-0.005	- 8.05	0.007	14.32	0.006	11.15
375	37.11	655.43	-0.032	- 34.20	-0.004	- 8.04	0.013	14.88	0.017	11.89
383	36.63	656.19	-0.053	- 24.18	-0.004	- 5.19	0.012	8.40	0.016	2.71
391	36.00	656.82	0.078	1.84	0.009	- 1.48	-0.044	0.34	0.064	8.79

TABLE 7

INTERNAL FORCES IN CONCRETE TANK DUE TO EARTHQUAKE

Area	Nodal Point	Longitudinal		Circumferential	
		Moment kft/ft	Force k/ft	Moment kft/ft	Force k/ft
Base Slab	16	0.044	3.72	0.061	0.50
	30	0.030	4.70	-0.097	3.68
	45	0.048	5.41	0.322	6.55
	60	0.096	7.09	0.373	10.12
	75	0.242	9.22	0.362	13.60
	90	1.512	11.22	-0.173	19.44
	105	1.762	13.11	0.322	29.41
	120	3.345	15.54	0.488	40.25
	134	4.683	17.45	0.794	46.87
	148	5.444	18.14	1.058	48.47
	162	6.481	12.50	1.316	50.30
	177	0.597	7.04	0.357	50.14
	193	0.005	2.75	0.388	49.29
Wall	191	0.043	5.52	0.196	44.73
	206	4.505	9.68	-0.502	38.78
	221	4.801	10.95	-0.588	30.36
	236	5.568	10.46	-0.785	13.43
	251	5.831	8.51	-0.892	5.73
	266	3.151	7.10	-0.533	12.01
	281	2.825	5.50	-0.514	18.90
	296	2.280	4.09	-0.461	25.31
	311	1.290	3.43	-0.342	33.36
	326	2.352	3.56	-0.516	41.36
	341	3.262	3.85	-1.047	54.13
	350	6.303	4.95	-2.231	77.50
	359	10.600	4.06	-3.755	96.62
Dome	359	10.960	6.98	-0.824	16.46
	367	10.535	6.98	-0.651	15.36
	376	9.469	6.52	-0.384	13.21
	384	7.999	6.19	-0.289	14.34
	391	6.371	6.03	-0.213	15.10
	397	4.922	4.25	0.105	12.03
	403	0.951	2.81	0.174	19.91
	408	0.086	2.39	0.456	24.28
	413	1.011	2.01	0.436	18.52
	418	0.756	1.93	0.360	14.07
	423	0.644	1.49	0.314	10.43
	427	0.547	1.09	0.262	7.18
	431	0.422	0.56	0.186	4.15
	435	0.220	0.49	0.065	1.28

HANFORD WASTE STORAGE
TANKS 241-SY

FIGURE 1

SECTION A - A

SCALE: 1" = 50'

HANFORD WASTE STORAGE
TANKS 241-SY

FIGURE 2

HALF VERTICAL SECTION THRU TANK

HANFORD WASTE STORAGE
TANKS 241-SY

FIGURE 3

COMPARISON OF SSE & SYNTHETIC
TIME-HISTORY SPECTRA
DAMPING = 5%

HANFORD WASTE STORAGE
TANKS 241-SY

FIGURE 5

COMPARISON OF RESPONSE SPECTRA
OF ORIGINAL & RECOMPUTED
SURFACE MOTION
DAMPING = 5%

HANFORD WASTE STORAGE
TANKS 241-SY

ACCELERATION TIME-HISTORY OF
COMPUTED BASE MOTION

FIGURE 7

AXIS OF SYMMETRY
SHELL ELEMENT

(a)

NOTE :

r_i, z_i, t_i & ϕ_i
REPRESENT DEGREES
OF FREEDOM

TOROIDAL ELEMENT

(b)

AXISYMMETRIC FINITE ELEMENTS

HANFORD WASTE STORAGE
TANKS 241-SY

FIGURE 8

FOR ENLARGEMENT
OF THIS AREA SEE
FIGURE 10a

FOR THIS AREA
SEE FIGURE 10b

SOIL - TANK FINITE ELEMENT MODEL

HANFORD WASTE STORAGE
TANKS 241-JY

HANFORD WASTE STORAGE TANKS 241-SY

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URS/BLUME

FIGURE 106

SOIL-TANK FINITE ELEMENT MODEL

ASSUMED EXCAVATION BOUNDARY

NOTE: NUMBERS REFER TO
SOIL LAYERS IN TABLE 1

ASSUMED SOIL LAYERING IN
SOIL - TANK FINITE ELEMENT MODEL

HANFORD WASTE STORAGE
TANKS 241-SY

FIGURE 11

NOTE :

NUMBERS IN PARENTHESIS ARE CORRESPONDING N.P. NUMBERS IN SOIL-TANK MODEL

PRIMARY STEEL TANK MODEL

HANFORD WASTE STORAGE
TANKS 241-SY

FIGURE 12

LEGEND

F_{ss} = LONGITUDINAL FORCE

F_{tt} = HOOP FORCE

F_{st} = SHEAR FORCE

M_{ss} = LONGITUDINAL MOMENT

M_{tt} = CIRCUMFERENTIAL
MOMENT

M_{st} = CROSS MOMENT

a. SHELL FORCES

b. SHELL MOMENTS

AXISYMMETRIC SHELL ELEMENTS FORCES AND MOMENTS

HANFORD WASTE STORAGE
TANKS 241-SY

FIGURE 13

RADIAL DISPLACEMENTS OF PRIMARY STEEL TANK

HANFORD WASTE STORAGE
TANKS 241-SY

FIGURE 14

URS/BLUME

HYDROSTATIC

HYDRODYNAMIC

LONGITUDINAL MOMENTS & FORCES ON
PRIMARY STEEL TANK

----- MOMENT, K-FT./FT.
————— FORCES, K/FT.

HANFORD WASTE STORAGE
TANKS 241-SY

FIGURE 15

**HANFORD WASTE STORAGE
TANKS 241-SY**

FIGURE 16

EARTHQUAKE
LONGITUDINAL MOMENTS & FORCES ON
CONCRETE TANK

----- MOMENTS, K-FT./FT.
————— FORCES, K/FT.

HANFORD WASTE STORAGE
TANKS 241-SY

FIGURE 17

EARTHQUAKE
CIRCUMFERENTIAL MOMENTS & FORCES
ON CONCRETE TANK

----- MOMENTS, K-FT./FT.
 _____ FORCES, K/FT.

HANFORD WASTE STORAGE
TANKS 241-SY

FIGURE 18

LONGITUDINAL

----- MOMENTS, K-FT./FT.
 ———— FORCES, K/FT.

CIRCUMFERENTIAL

GRAVITY LOAD MOMENTS & FORCES
ON BASE SLAB

HANFORD WASTE STORAGE
TANKS 241-SY

FIGURE 19

APPENDIX A

THERMAL-CREEP ANALYSIS OF
241-SY UNDERGROUND REINFORCED CONCRETE TANK STRUCTURE
AT HANFORD, WASHINGTON

by

Y. R. Rashid

APPENDIX A

THERMAL-CREEP ANALYSIS OF 241-SY UNDERGROUND REINFORCED CONCRETE TANK STRUCTURE AT HANFORD, WASHINGTON

PURPOSE AND SCOPE

The 241-SY underground reinforced concrete structure is required to carry sustained soil pressure and thermal loadings with sufficient margin against degradation of its structural integrity during its design life. Under similar time invariant loading conditions, creep and cracking generally develop, initially in transient modes which tend towards self-limiting processes. The main objective of the present study is to determine whether the self-limiting cracking and creep deformations in the tank structure during operation can be verified by appropriate analyses.

The main structural components of the 241-SY tank are:

- a) Concrete, which provides compressive capability and is subject to creep and cracking.
- b) Meridional, hoop, and diagonal reinforcement designed to resist or limit circumferential, radial, and split cracks, respectively.
- c) Secondary steel plate liner, which provides leak-tight seal.

While the steel reinforcement and liner are designed to remain elastic, it is possible that yielding may occur as a result of excessive deformations or due to the formation of wide cracks. Similarly, the concrete is designed to remain stressed well below its ultimate strength, but excessive cracking or misplacement of reinforcement could result in the compressive failure of the concrete in some localized regions. These aspects are qualitatively accounted for in the design; the present analysis, however, provides quantitative assessments of these design considerations.

The analysis is highly dependent on the material properties of the structural constituents, particularly the creep data of the concrete. Because of the relatively high temperature, 250°F, the concrete undergoes thermally activated

creep of much larger magnitude than generally encountered in ordinary concrete structures. Even highly specialized structures, such as prestressed concrete pressure vessels for gas-cooled reactors, are designed to operate at lower temperatures. Therefore, it was of necessity that detailed time-dependent analysis be performed to verify the structural integrity under such severe conditions. Although the operating life of the structure may be several decades, the analysis time need extend only until self-limiting cracking and creep deformations are established and the results can be safely extrapolated to the end of life. In other words, stable deformation and cracking conditions should prevail after only a few years of tank operations, and the analysis time may be restricted to a small fraction of the design life. This premise rests on the assumption that the loadings, which include temperature and soil pressure, remain constant with time.

COMPUTER PROGRAM AND FINITE ELEMENT GRID

The computer program, SAFE-CRACK, applied to the analysis of this tank is based on the finite element method for which the structure is idealized as:

- a) Triangular axisymmetric elements for the concrete.
- b) Uniaxial tension-compression elements for the reinforcement.
- c) Axisymmetric membrane elements for the secondary liner.

The reinforcement is placed along the element edges to ensure bond compatibility between the concrete and reinforcement. Because of the axisymmetric idealization, the meridional reinforcement is assumed to be continuous in the hoop direction but with zero hoop stiffness. Reduction of the elastic modulus of the reinforcement, to account for the concrete and reinforcement occupying the same position, is made appropriately.

The finite element grid consists of 402 nodes, 660 concrete elements, 66 liner elements, 153 meridional and diagonal reinforcement elements and 148 hoop reinforcement elements. The basic grid, namely, the triangular elements are placed five layers (10 elements) across the thickness maintaining an aspect ratio of 4.3 approximately. Based on experience with this type of analysis, this aspect ratio, although larger than desired, is selected to provide good accuracy while maintaining reasonable running time.

MATERIAL PROPERTIES

The material properties and mechanical behavior of concrete and reinforcement are described below:

- a) The concrete is assumed to be elastic-perfectly plastic in compression and elastic-cracked in tension with the following material data:

Compressive Strength	(σ_{ult})	=	4500 psi at 28 days
Modulus of Elasticity	(E)	=	3.8×10^6 psi
Poisson's Ratio	(ν)	=	0.15
Coefficient of Thermal Expansion	(α)	=	6.38×10^{-6} in./in./°F
Tensile Strength	(σ_{crack})	=	450 psi

- b) The liner and reinforcement steels are assumed to be elastic-perfectly plastic with the following average properties:

Liner

Yield Strength	(σ_{yield})	=	35000 psi
Modulus Elasticity	(E)	=	27.7×10^6 psi
Poisson's Ratio	(ν)	=	0.3
Coefficient of Thermal Expansion	(α)	=	6.38×10^{-6} in./in./°F

Reinforcement

Yield Strength	(σ_{yield})	=	60000 psi
Modulus of Elasticity	(E)	=	27.7×10^6 psi
Coefficient of Thermal Expansion	(α)	=	6.38×10^{-6} in./in./°F

- c) Creep data for concrete with properties somewhat different than given above are built into the SAFE-CRACK program. Multiplication by the ratio of the two concrete elastic moduli $\frac{E_{\text{program concrete}}}{E_{\text{tank concrete}}}$ is used to adjust the program creep data for the tank concrete.

The ratio is computed internally by the program as a function of temperature and age. For the case at hand, this ratio is larger than unity, implying that the tank concrete is softer than the built-in properties. The creep data in SAFE-CRACK are somewhat complicated expressions of temperature and age. To provide useful information in this report the adjusted creep curves, including the elastic response, for the temperatures of 70°F, 150°F, 200°F, 250°F and 300°F and for the initial age of 60 days are plotted in Figure 1.

Creep curves for other temperatures can be obtained from Figure 1 by linear extrapolation. The computer program, however, describes the creep data as continuous functions of temperature and age. It should be noted that the temperatures in the tank assumed for this analysis range from 65°F before heating to 250°F at the end of a 30 days heating period. The age of the concrete at the start of the heating period is assumed to be 90 days.

ANALYSIS PROCEDURE

As mentioned earlier, the computer program used is SAFE-CRACK. The basic triangular finite element grid was first established. The steel reinforcement areas were determined from the structural drawings, lumped into uniaxial elements, and placed along the triangular element boundaries as close as possible to their original positions. This ensured full bonding compatibility between the concrete and reinforcement.

The meridional and diagonal (radial) reinforcement, being discontinuous in the circumferential direction, were modeled as continuous shells but with zero hoop stiffness. The hoop reinforcement, being geometrically axisymmetric, was represented exactly in the grid and placed at the nodes closest to their original positions.

The liner elements were modeled as axisymmetric membrane shell elements, i.e. with extensional (no bending) meridional and hoop stiffness. These elements were placed at the inner surface between each pair of nodes. By virtue of the no-bending properties of these elements, the liner and concrete elements remain in intimate contact with each other during deformations, implying that the liner is anchored continuously to the concrete.

The loading, other than thermal, consisted of:

- a) Soil overburden extending 6.5 feet above the crown with unit weight of 120 pcf.
- b) Live load of 40 psf.
- c) Vapor pressure of -6 inches of water at the top inner surface of the dome.

The soil was assumed to be fully saturated and therefore the soil overburden was treated as a hydrostatic load on the outside surface of the tank.

The boundary conditions at the bottom were assumed to be sliding with zero rotation. This type of boundary condition is specified in the design. As will be discussed later, the sliding condition showed no harmful effects on the tank according to analysis results.

In order to simulate the tank structure being heated from 65°F to 250°F for 30 days, the thermal loading was treated as pseudo steady state where the temperature at any time after heating began were obtained by linear extrapolation between the initial (65°F) temperatures and the final temperatures. This was considered a close approximation to the true transient temperature field since the heating rate is quite slow.

The time history of the analysis consisted of 30 time steps. The first step at zero time (corresponding to the age of 90 days) considered only the mechanical loads. Time steps 2 through 11 considered the thermal loading as 10% increments each 3 days. The rest of the time steps covered a period of approximately 5.5 years and extended the analysis much beyond the time necessary to establish that stable conditions will prevail in the tank structure during operation.

The output of the computer program consists of:

- a) Input data printed for reference
- b) The concrete relaxation moduli
- c) The element temperature histories
- d) The finite element grid description
- e) Nodal displacements at each step
- f) Element stresses and strains at each step

The last three items are used to study the response of the tank and are transmitted along with the other output data as a supplement to this report.

DISCUSSION OF THE ANALYSIS RESULTS

Figure 2 shows a plot of the axial displacement at the crown as a function of time. As can be noticed, the initial value, i.e. due to soil pressure alone is (small) positive in spite of the top overburden. Evidently the lateral hydrostatic soil pressure over a projected depth of 55 feet is the cause for net upward displacement at the crown. As heating begins, a negative crown displacement results because the upward and outward displacement due to the free thermal expansion is counteracted by a negative thermal moment. It should be stated that during heating, cracking develops, and part of the thermal forces are therefore dissipated. At the end of 30 days all loadings had reached constant values, and the displacement at the crown followed a rapid transient creep process in the direction of free thermal expansion, i.e., towards the positive direction. This is in agreement with the expected behavior of thermal creep problems where the steady state response of the structure, after thermal stresses relax out, is primarily controlled by the free thermal expansion.

This general behavior is more evident in Figure 3 where the deformed outer surface is plotted at 30 days and 2000 days. As mentioned above, the 30-day deformations show clearly the influence of the thermal moment on the dome. As the thermal moment relaxes out the deformations tend towards the free thermal expansion shape as shown by the 2000-day deformed shape. In contrast with the behavior of the dome, the cylindrical portion of the structure predictably deformed uniformly outwards due to the thermal loading followed by a uniform inward deformation under creep.

The cracking behavior of the structure is presented in Figures 4 through 8. Figures 5 and 6 show the extent of the radial cracks which are caused by the hoop stresses and Figures 7 and 8 show the extent of the meridional cracks which are caused by the maximum principal stresses in the R-Z plane. These figures also show that the majority of the cracks occurred during heating. With the exception of the haunch region, the cylindrical portion of the structure is free of cracking.

The most significant fact which needs to be pointed out is that the meridional cracks are primarily of the split type, i.e., the crack surfaces are in the

general direction of the middle surface of the shell. This type of crack can be controlled by diagonal reinforcement which runs perpendicular to the middle surface of the structure. The original design provides for such reinforcement in the haunch area only. It appears from Figure 5 that the diagonal reinforcement is required throughout the dome and to a distance of at least 8 ft. below the outer dome-to-cylinder junction.

By examining the output, in particular the hoop and the principal strains in the cracked elements, one can easily see that the magnitudes of the hoop strains are quite small, indicating that the circumferential reinforcement adequately provides for limiting the radial cracks. Similarly the principal strains in meridionally cracked elements in the haunch area, where diagonal reinforcement exists, are also limited to about 1%. This is considered small and hence stable. In contrast, however, the principal strains approach 10% in the crown, where meridional cracks dominate and where there are no diagonal reinforcements. Without diagonal reinforcement to limit these cracks, they might increase in an unstable manner.

The stresses in the liner and reinforcement remain well below the yield stresses of the materials. This reinforces the other findings of this analysis that no structural instability is evident.

CONCLUSIONS AND RECOMMENDATIONS

The present analysis indicates the following:

- 1) The structure is stable under the loading conditions given.
- 2) Sliding boundary condition at the base, as provided for in the design, is adequate.
- 3) The liner and reinforcement remain elastic and hence no unbounded deformations occurred. See Stress Table (Table 1).
- 4) The radial crack sizes, i.e., hoop strains in the cracked elements, are quite small indicating adequate hoop reinforcement.
- 5) The meridional crack sizes, i.e., principal strains in the cracked elements, range from about 1% in the haunch area where diagonal reinforcement exists to about 8% in the crown area where diagonal reinforcement is absent. It is recommended, therefore, that diagonal

reinforcement be extended throughout the dome and to a depth of at least 8 feet below the dome-to-cylinder junction (at elevation 654.84ft.)

- 6) The analysis of the primary steel tank and the weld junction connecting the primary tank to the secondary liner was not within the scope of this analysis. Therefore, no evaluation of the particular area of structure can be given in this report.

TABLE I
HIGHEST STRESSES IN THE STRUCTURE

<u>Structural Component</u>	<u>Location</u>	<u>Stress* psi</u>	<u>Stress type</u>	<u>Time Occurred, Days</u>
Concrete	Crown	-3000	radial (Principal)	30
Liner	Crown	26000	radial and hoop	30
	Dome	-13000	meridional	30
Reinforcement	Crown	-25000	hoop	2000 days
	Haunch	7000	meridional	2000 days

*Tension is positive

FIGURE 1 ELASTIC + CREEP SPECIFIC STRAIN
(MATERIAL COMPLIANCE) VS TIME
CONCRETE AGE = 60 DAYS

FIGURE 2 MIDDLE SURFACE DISPLACEMENT AT THE CROWN

FIGURE 3 OUTER-SURFACE DISPLACEMENT

FIGURE 4 EXTENT OF CRACK TYPES 1,2&3 AT 2000 DAYS

FIGURE 5 TYPE 2 (RADIAL) CRACKS AT 30 DAYS

FIGURE 6 TYPE 2 (RADIAL) CRACKS AT 2000 DAYS

FIGURE 7 TYPES 1&3 (MERIDIONAL) CRACKS AT 30 DAYS

FIGURE 8 TYPES 1&3 (MERIDIONAL) CRACKS AT 2000 DAYS

INVESTIGATION TO DETERMINE
DYNAMIC SOIL PROPERTIES
AT THE
241-SY TANK SITE

INVESTIGATION TO DETERMINE
DYNAMIC SOIL PROPERTIES
AT THE
241-SY TANK SITE

prepared for
ATLANTIC RICHFIELD HANFORD COMPANY

October 1974

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INVESTIGATION TO DETERMINE
DYNAMIC SOIL PROPERTIES
AT THE
241-SY TANK SITE

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INVESTIGATION TO DETERMINE
DYNAMIC SOIL PROPERTIES
AT THE
241-SY TANK SITE

INTRODUCTION

This report describes results of a subsurface geophysical investigation carried out at the 241-SY waste storage tank site located in the 200 West area of the Hanford Atomic Energy Commission Reservation near Richland, Washington. The work was conducted by URS/John A. Blume & Associates, Engineers, for Atlantic Richfield Hanford Company (ARHCO) of Richland, Washington. The investigation consisted of drilling and logging one drill hole to a depth of 150 feet and measuring shear and compressional wave velocities in this drill hole. Seismic velocities were also measured by refraction techniques across the ground surface. Dynamic soil properties to be used in seismic analysis of the tanks were derived from geophysical measurements.

The drilling was completed in late February 1974, and geophysical measurements were carried out in early March. The investigation was directed by Andrew Cunningham, who also prepared this report with the assistance of Fred Kintzer.

FIELD INVESTIGATIONS

Drilling

Exploratory drilling at the site was carried out by Hatch Drilling Company of Richland, Washington, who also compiled the driller's log (shown in Appendix A). Drilling was done using cable tool equipment, and samples were collected at 5-foot intervals. Steel casing with an approximate diameter of 6 inches was installed in the hole as the drilling proceeded. Representative soil samples were selected and delivered to a soils laboratory for testing to classify and identify the soils according to ASTM procedures. The location of drill hole 1 at the 241-SY tank site is shown in Figure 1.

 SCALE: 1" = 100'

241-SY TANK FARM

DRILL HOLE 1 AND SEISMIC
REFRACTION LINES 1 AND 2
SITE PLAN

241-SY TANK SITE

FIGURE 1

Geophysical Measurements

Geophysical measurements at the site consisted of uphole determinations of shear and compressional wave velocities and measurements of velocities of horizontally propagated shear and compressional waves on the ground surface by seismic refraction techniques. The seismic refraction data confirm that the velocities measured in the drill hole were comparable to those measured by refraction techniques around the periphery of the drill hole and that the moduli data, derived from drill-hole measurements, can be extrapolated for the area surrounding the drill hole.

Uphole velocity measurements were performed in drill hole 1 at the 241-SY tank site using a triaxial uphole seismometer probe to record the seismic waves produced by an 8-pound sledge hammer at the ground surface. Electric signals from the velocity-sensitive geophones were transmitted to the surface via a multiconductor cable. At the surface, the signals were amplified and recorded on a 6-channel oscillograph recorder. Seismic-wave velocities were measured at 10-foot intervals as the probe was pulled up from the bottom of the drill hole.

Seismic refraction measurements at the ground surface were made using a Bison portable seismograph, model 1570B, and a 6-channel oscillograph recorder in conjunction with a triaxial seismometer array. An 8-pound sledge hammer was again used as the energy source for the surface measurements. Locations of the seismic refraction lines are shown in Figure 1. Seismic refraction line 1 extends 200 feet to the northeast from drill hole 1, and seismic refraction line 2 extends 200 feet in a southeasterly direction. Time-distance plots for the seismic refraction lines are shown in Figures 2 and 3, and a time-depth plot of uphole seismic measurements is shown in Figure 4.

Time-distance and time-depth graphs are constructed by plotting travel time against impact distance. Travel time is the time required for the seismic wave to travel from the energy source, in this case the sledgehammer impact, to the detector. The impact distance along a surface refraction line is the distance along the ground between the detector and the point of impact.

SEISMIC REFRACTION LINE 1

SEISMIC REFRACTION LINE 2

DRILL HOLE 1

SHEAR (V_s) AND COMPRESSIONAL (V_p) VELOCITIES

241-SY WASTE STORAGE TANKS

FIGURE 4

URS/BLUME

Impact distance on a time-depth graph is the vertical distance between the energy source at the ground surface and the detector, which was positioned at 10-foot intervals down the drill hole. The slope of a line connecting a series of points on these graphs is equal to the velocity of the wave passing through the ground. The depths to various soil layers beneath the ground surface can be derived from the surface refraction measurements using known principles of wave propagation and refraction. Depths to soil layers having different seismic velocities can be observed directly on the time-depth graphs.

DYNAMIC SOIL PROPERTIES

Dynamic soil properties were derived for the 241-SY tank site from on-site seismic measurements of shear and compressional wave velocities of vertically propagated seismic waves. Laboratory measurements of moduli on selected samples obtained in drilling were not undertaken because of special problems with soils of the Hanford Reservation: because the soils are unconsolidated, it is not possible to keep samples in an undisturbed condition throughout the sampling and testing procedures, and the relatively high content of gravels makes it impossible to test much of the material.

Calculated values of shear modulus for comparison were obtained by converting blow counts recorded during drilling into estimated soil densities using Gibbs and Holtz's criteria (1957). Using these densities, shear-wave velocities were calculated by means of formulae derived by Hardin and Richart (1963). Values of shear modulus at various depths were then derived from these calculated velocities. Although the reliability of density measurements based on blow counts has been questioned and the data of Hardin and Richart were extrapolated in these calculations, reasonable similarity was observed in the two sets of data. Shear moduli derived from measured shear-wave velocities at the tank site were lower.

The shear modulus for low strain levels, G_{max} , calculated from shear-wave velocities, is shown in Figure 5 as a function of depth below ground surface. Shear modulus is shown in Figure 6 as a decreasing function of strain level. The moduli are expressed as ratios of the maximum values shown in Figure 5.

MAXIMUM SHEAR MODULUS (KSI)

MAXIMUM ELASTIC MODULI OF
FOUNDATION MATERIALS

DESIGN ELASTIC MODULI AS
A FUNCTION OF STRAIN
 (AFTER SEED AND IDRIS, 1970)

Design Modulus Values

The design curves in Figures 5 and 6 are recommended for the 241-SY waste storage tanks. For any depth and shear strain, the appropriate modulus can readily be computed from Figures 5 and 6. For example, the shear modulus corresponding to a shear strain of $2 \times 10^{-2}\%$ at a depth of 50 feet can be evaluated as follows: from Figure 5, $G_{\max} = 31$ ksi; the modulus ratio corresponding to the strain is found from Figure 6 to be 0.65; the desired shear modulus, G , is then $0.65(31 \text{ ksi}) = 20.4 \text{ ksi}$.

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APPENDIX A
DRILLER'S LOG

DESCRIPTION	DEPTH (ft)	SAMPLE		BLOWCOUNT (blows/inches)
		Depth	Type	
Medium, brown, silty, fine-to-medium sand with occasional fine-to-coarse gravel and roots 5.0	0	3.0	D*	
Dense, gray, fine sandy, fine-to-coarse gravel with some silt 12.0	10	7.0	D	
Very dense alternate layers of gray fine-to-medium sand and gray-brown silty fine sand with occasional 2"-lenses of hard, sandy silt 36.0	20	14.0	D	31/12"
		17.0	N**	50/5"
		19.0	D	26/12"
		22.0	N	50/5"
		25.0	N	50/6"
		31.0	N	50/6"
	30	35.0	N	50/2"
Very dense gray-brown, slightly silty to silty, medium-to-coarse sand and fine-to-coarse gravel (42' to 49', very gravelly) 49.0	40	41.0	D	41/12"
Very dense, light brown, slightly silty to silty, fine sand with some fine-to-coarse gravel and medium-to-coarse sand 63.0	50	50.0	N	50/3"
		55.0	D	17/6"
	60	59.0	N	50/3"
Very dense, light brown, silty, fine-to-medium sand with occasional 1/2"-lenses of sandy silt		65.0	N	50/5"
	70	70.0	N	50/4"
		75.0	N	50/4"
		79.0	N	50/5"
	80			
		85.0	D	16/6"
		89.5	D	24/6"
	90	90.0	N	50/5"
		95.0	D	18/6"

APPENDIX A

DRILLER'S LOG

(cont.)

DESCRIPTION	DEPTH (ft)	SAMPLE		BLOWCOUNT (blows/inches)
		Depth	Type	
Very dense, light brown, silty, fine-to-medium sand with occasional 1/2"-lenses of sandy silt	100	100.0	D	22/6"
		100.5	N	50/5"
		105.0	D	21/6"
		109.5	D	22/6"
	110	110.0	N	50/4"
		115.0	D	21/6"
		119.5	D	21/6"
		120.0	N	50/4"
	120	124.5	D	18/6"
		127.0		
Alternate lenses of silt and silty fine-to-medium sand	130	129.5	D	24/6"
		130.0	N	50/3"
		135.0	D	21/6"
	140	140.0	D	17/6"
		140.5	N	50/5-1/2"
		145.0	D	18/6"
	150	149.5	D	17/6"
		150.0	N	50/5"
Bottom of boring				

* D denotes 5"-diameter sampler using 1400 lb hammer, 30" drop

** N denotes standard penetration resistance

FIG.

FIG.

FIG.

FIG.