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SAN ONOFRE NUCLEAR GENERATING STATION, UNIT 1
EVALUATION OF THE REFUELING WATER STORAGE TANK FOR LONG TERM SERVICE

Volume I

Prepared for:

Southern California Edison Company

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IMPELL CORPORATION

REPORT APPROVAL COVER SHEET

Client: Southern Califonria Edison

Project: SONGS-1 Job Number: 0310-058

Report Title: Evaluation of the Refueling Water Storage Tank for Long Term Service

Report Number: 01-0310-1392 Rev. 0

The work described in this Report was performed in accordance with the Impell Quality Assurance Program. The signatures below verify the accuracy of this Report and its compliance with applicable quality assurance requirements.

Prepared By: Ward Chief / DePondya Date APRIL 26, 1985

Reviewed By: Walter Bak / M Saatta Date: April 26, 1985

Approved By: [illegible] Date: April 26, 1985

REVISION RECORD

Rev. No.	Prepared	Reviewed	Approved	Approval Date	Revision
1	WR Bak	M Saatta	Hai Tung Hing	3/4/86	Reanalysis
2	WR Bak	M Saatta	Kenneth R. Baker	3/21/86	SCE Comments

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1.0 INTRODUCTION

The Refueling Water Storage Tank (RWST) has been evaluated on two previous occasions. Both analyses concluded that the RWST was adequate for its intended function. These conclusions were based on a comparison of the compressive stress in the wall of the tank with modified Code allowables. The allowables were based upon the Code Level D allowables, and they were increased to account for the effects of bending vs. axial stress distributions, internal pressure effects, and safety factors smaller than the Code recommended values.

The RWST was first evaluated for the 0.67g modified Housner event. The tank was modeled with a fixed base, and procedures developed in Reference [5] were used to account for the effects of fluid sloshing and tank shell flexibility. At the time this evaluation was performed, the soil parameters below the tank had not been completely defined, and therefore the effects of foundation settlement were not considered.

A second evaluation was performed later as part of the Return to Service (RTS) effort where the RWST was reevaluated for the effects of a 0.5g modified Housner event. This was based on the previous 0.67g evaluation, but included the effects of foundation settlement on the concrete basemat.

Recently, Impell has reevaluated the tank using more sophisticated analysis methods to develop a realistic safety factor.

Impell's approach was to perform a soil-structure interaction (SSI) analysis of the tank-and-soil to obtain more realistic base shears and moments in the tank for the 0.67g seismic event. Impell has also considered the effects of in-situ backfill soil and performed an evaluation to determine the tank movement or sliding due to the modified Housner event. The results of the reanalysis of the tank are contained in this report.

2.0 SOIL-STRUCTURE INTERACTION

2.1 Analysis Approach

A general analysis approach was taken where a computer model was developed to represent the RWST and its foundation. Elements were included to represent the soil, the tank shell, and the fluid. These elements are described in the following sections.

The tank model was then used in a soil-fluid-structure interaction analysis to compute realistic base moments and shear forces. These loads, in addition to dead weight, vertical seismic, and hydrostatic pressure loads, were then used to perform evaluations of the tank shell, anchorage, and foundation.

2.1.1 Tank/Fluid Model

The dynamic model of the RWST was developed using the analysis procedures of Housner [1]. Housner's procedure divides the mass of the tank into two distinct parts:

1. Mass associated with the first sloshing mode of the fluid, commonly referred to as the convective mass.

2. Mass associated with the ground motion, commonly referred to as the impulsive, or rigid mass. 2

These masses, along with their location along the axis of the tank, and the frequency response of the tank's flexibility were determined using the equations and the graphs in [1] and [5]. Reference [5] was used to determine the frequency of the tank shell and rigid mass accounting for the tank flexibility. The shell was modeled with appropriate EI values to generate the calculated frequency. 2 2 2

The tank was modeled using beam elements (Fig. 2.1). Mass corresponding to the vertical response of the system was lumped along the axis of the tank at the appropriate heights (Table 2.1). The mass corresponding to the horizontal response of the sloshing fluid was attached to the shell with springs such that the frequency of vibration of the spring mass system was equal to the frequency predicted using Haroun's analysis model. 1

The frequency response of the model was verified by performing a fixed-base mode/frequency analysis using the EDSGAP computer code [10]. The results of the EDSGAP analysis show that the model responded at the predicted frequencies. (See Table 2.1.)

The development of the fluid-structure model is described in more detail in [2].

2.1.2 Soil Parameters

The soil conditions at the SONGS-1 site are extremely uniform, resulting in very small areal and depth non-uniformities in the soil properties. The soil is uniformly dense San Mateo sand extending to about 1000 feet below site grade, with the absence of significant layering. However, there are regions around major structures which are composed of backfill soil with varying relative compaction.

As shown in Figure 2.2, approximately 40 percent of the RWST is founded on native San Mateo sand, with the remaining 60 percent founded on shallow (up to 8-foot depth) Category B backfill soil [13]. The backfill is San Mateo sand with a minimum of 92 percent relative compaction. As indicated in Reference 13, only small differences exist between the shear modulus values of native San Mateo sand and that of backfill sand with 92 percent relative compaction, for the strain levels associated with a DBE-type event (approximately 10 percent). In addition, the difference in soil densities is less than 10 percent. As a result, both the native San Mateo sand and the backfill under the RWST will exhibit similiar response to seismic loadings. For this reason, the underlying soil is modeled as a uniform half-space. The average soil properties for the site were used on the SSI evaluation of the RWST. ⚠

For horizontal excitation, the two main soil parameters influencing SSI are the soil shear modulus (or shear wave velocity) and damping of the soil material. These properties were determined by considering the range of strain levels expected at the site (given in [13]). Strain-compatible soil properties were then derived in accordance with Figure 2.3. The soil hysteretic (material) damping was limited to the damping value at 0.05 percent principal soil strain. ⚠

The soil properties were selected so as to use a maximum value for the shear modulus and a minimum value for the soil density to predict conservative in-structure responses. Table 2.2 summarizes the soil properties used in this evaluation.

2.1.3 Control Motion

The SONGS-1 artifical time history was used for the SSI analysis. The response spectrum corresponding to this time history envelopes the horizontal 0.67g modified Housner response spectrum (Fig. 2.4). For the SSI analysis in the horizontal direction, the control motion was assumed to consist of vertically propagated shear waves. The RWST basemat was assumed to be surface-founded. The control motion is applied in the free-field at the ground surface level. This is consistent with the NRC Standard Review Plan 3.7.2.

2.2 Soil-Structure Interaction Evaluations

The soil-structure interaction analyses were performed using the CLASSI computer program. CLASSI is a three-dimensional equivalent linear analysis program. It employs a frequency domain solution and uses a substructure approach. A description of CLASSI is included in Appendix A.

The 3-D fixed-base model described in Section 2.1.1 was adapted for use in CLASSI. The CLASSI model consisted of:

- The fixed-base stick model
- Rigid basemat
- Underlying soil medium modeled as a uniform half-space.

The basemat was modeled using as a rigid circular plate to model the geometry of the concrete mat. The tank model was connected to the center of the basemat. The CLASSI model is shown in Figure 2.1. A horizontal CLASSI analysis was performed.

The vertical seismic response was estimated using a lumped parameter model and the free-field modified Housner response spectrum for the 0.67g seismic event (Figure 2.5). The frequency of the tank/soil system was calculated by modeling the system as an undamped single-degree-of-freedom (SDOF) oscillator.

The mass of the tank, fluid, and basemat were lumped on a vertical soil spring. The stiffness of the spring was calculated using equations in Table 3-2 of [6]. Average soil properties were considered. The frequencies calculated for the oscillator were greater than the frequency corresponding to the maximum spectrum peak (4 Hz) therefore, the vertical acceleration was conservatively assumed to be the peak value.

The damping value for the vertical soil response was calculated using methods in both Table 3-2 and Appendix H of [6]. In addition, damping values were calculated using [14]. In each case, the soil's vertical damping was estimated to be much higher than 10 percent. However, the system's damping was conservatively limited to 10 percent. This assumption is consistent with the values listed in [7].

2.3 Soil-Structure Interaction Results

The critical terms in the evaluation of the RWST are the overturning moments resulting from horizontal excitation of the tank. The stresses resulting from vertical seismic excitation are much smaller than the stresses resulting from the horizontal excitation. Therefore, a computerized SSI analysis was performed for the horizontal direction while conservative hand calculations were used to estimate the vertical response.

Peak accelerations at the in-structure mass locations were generated from the SSI analyses. A summary of peak accelerations for horizontal excitation, obtained from the SSI analyses, as well as the maximum base moments and shears from the time history analyses are presented in Tables 2.3 and 2.4, respectively.

The loads due to vertical seismic excitation, axial compression, and hydrostatic pressure were conservatively computed using the peak of the free-field vertical response spectrum (Figure 2.5).

3.0 SLIDING EVALUATION

The RWST foundation was originally designed for a lower seismic input than the 0.67g modified Housner event. The existing foundation is not capable of completely resisting motion due to sliding. Therefore, an evaluation is performed to define the maximum sliding distance of the tank due to the seismic event. The approach and results of the sliding evaluation are presented in Reference 12 and summarized in the following paragraphs. ⚠

3.1 Analysis Approach

The sliding evaluation is performed by using a single degree-of-freedom model of the tank and by modelling the interface between the tank and soil. A time-history analysis is then performed using vertical and horizontal input motions representative of the modified Housner input.

3.1.1 Model

The model used for the sliding analysis is shown in Figure 3.1. The tank is modelled as a single degree-of-freedom system. The mass of the tank and its contents is lumped at the center of gravity of the system and the beam properties are established to correctly predict the fundamental frequency of the tank. The model frequency is 7.12 Hz for the tank as described in Section 2.1.1 of this report.

The model also includes the interface between the tank foundation and soil. The interface element includes a vertical and horizontal component. The vertical component is a gap element which has a very high stiffness when contact exists between the tank and soil, and zero stiffness when the tank uplifts. The horizontal spring represents the frictional force between the soil and tank foundation. The coefficient of friction used to establish this stiffness is 0.59 based on soil data in the FSAR for SONGS 2 and 3.

3.1.2 Control Motion

Horizontal and vertical time histories were simultaneously applied to the tank model. The horizontal time-history has a maximum acceleration of 0.67g and the vertical time-history has a maximum acceleration of 0.44g. The input motions were applied in the following manner:

- horizontal motion plus vertical motion ⚠
- horizontal motion plus vertical motion (with reverse sign)

The vertical motion was applied in two signs to ensure that the maximum sliding results were obtained. The EDSGAP computer program (Reference 10) was used for the analysis.

3.2 Sliding Results

The sliding analysis results are displayed in Figure 3.2. The sliding motion "walks out" and produces a maximum tank displacement of 0.68 inches. This displacement is assumed to occur in any direction. The results of this evaluation impact the evaluation of piping attached to the tank, and the sliding displacement will be used in piping and tank nozzle qualification. ⚠

4.0 TANK EVALUATION

The loads generated using the methods described in the previous sections were used to evaluate the tank. The evaluations were performed in [4] and are summarized below.

The tank shell is evaluated for the effects of pressure, gravity and seismic loads. The functionality of the tank is based on the structural integrity of the tank shell. The roof will not be evaluated since local buckling of the roof or upper shell will not result in a loss of function of the RWST. ⚠

4.1 Shell Evaluation

Table 4.1 shows a comparison of the compressive stresses calculated for the shell and the allowable compressive stresses. The table lists the allowable calculated using the rules of Code Case N-284 [9]. The CC-N-284 allowable is based on the theoretical critical buckling stress factored by a capacity reduction factor (for the RWST) of .207. The allowable is compressive stress from Code Case N-284, increased to reflect the stabilizing influence of the internal hydrostatic pressure. CC-N-284 specifically addresses the effects of internal pressure as:

> "The influence of internal pressure on a shell structure may reduce the original imperfections, and therefore higher values of capacity reduction factors may be acceptable."

The increased allowable used in the evaluation of the RWST is based on the procedures of the AWWA Standard for Welded Steel Water Storage Tanks [8]. The increase was calculated using the minimum hydrostatic pressure, which occurs during vertical seismic motion, at each section of the tank. The increase in critical buckling stress was calculated assuming a safety factor of 1.0 and was added to the CC-N-284 allowable described above. The resulting allowable was then divided by the calculated stresses to determine the safety factor. ⚠

The results of the stress comparisons of Table 4.1 show that all safety factors are greater than 1.0.

The minimum factor of safety is 1.44. This value is greater than the Code recommended safety factor of 1.34 for Level D events. When the conservatisms of the analysis are considered, a factor of safety of 1.44 is more than adequate to demonstrate the integrity of the tank.

The maximum principal stresses in the shell were calculated and compared to the Level D allowables of Subarticle NC-3800 of the Code [11]. The principal stresses were determined by combining the maximum tensile hoop and maximum tensile longitudinal stresses with the maximum shear stress in the shell. Hoop stresses were calculated based on the maximum hydrostatic and hydrodynamic pressures resulting from horizontal and vertical seismic excitation plus gravity loads. The hydrodynamic pressure was calculated using the procedures of Reference 16. Longitudinal stress results from the overturning moment and the weight of the shell under gravity and vertical seismic loads. The results of this stress check are summarized in Table 4.2. As shown in the table, all stresses are less than the allowables. ⚠

The evaluation shows that the tank shell is qualified without modification for the 0.67g Housner event.

4.2 Foundation Evaluation

Based on the soil conditions under the RWST, Reference [13] (Section 5.17, p. 5-10) postulates that the potential exists for up to 1-1/2 inches of seismically induced settlement under the northern and western portions of the tank (see Figure 2.2).

The effects of soil settlement on the basemat were evaluated using an equivalent beam model of the basemat. For the settlement evaluation, the mat was modeled as supported on an elastic foundation. This was done by replacing the elastic half-space used for the SSI evaluation (Fig. 2.1) with vertical springs along the basemat (Fig. 4.1). A uniform load, equal to the dead weight of the tank and fluid, was applied to the basemat. The stiffness of the equivalent soil springs under one-half of the tank was reduced until the maximum displacement in the model was equal to the predicted settlement of 1.5 inches. When the displacement of 1.5 inches was reached, the maximum moment and shear force in the concrete were compared

to the allowable values (see Table 4.3). Based on the allowable loads of ACI-349, the safety factors were 1.48 for the basemat in bending and 3.90 in shear.

The settlement loads produce an downward bending in the concrete slab. The seismic motion will result in a upward bending shape in the concrete mat. The concrete mat was evaluated for seismic loads by determining the equivalent soil bearing pressure and applying this load to a model of the concrete mat considering the deadweight and seismic contribution of the tank and its contents. The results of this evaluation are reported in Table 4.3.

Soil bearing pressure is also evaluated and the resultant bearing pressure of 14.6 ksf is less than the allowable bearing pressure of 25.0 ksf for the site. The allowable bearing pressure is described in Reference 17. The factor of safety in bearing is 1.71. ⚠

4.3 Tank Anchorage

The anchorage of the RWST consists of 32 1-5/8 inch diameter anchor bolts embedded in the concrete basemat [15] with a steel base ring, stiffening ring, and stiffening plates welded to the tank shell.

The tensile and shear forces in the anchor bolts were calculated using the base moments and shears from [3]. The tensile bolt loads were calculated considering shell uplift due to overturning, vertical seismic loads, and dead load. The shear bolt loads were calculated by distributing the base shear equally over the 32 bolts. The shear calculations were performed using a conservative estimate of the frictional resistance between the tank bottom and shell.

The stresses in the anchor bolts were calculated and combined in accordance with the rules of the ASME Code, Appendix F, for level D events ([11], F-1335). The results of the stress check shows that all stresses are below the allowables. The maximum value of the stress interaction (see Table 4.4) of 0.81 was obtained. ⚠

Qualification of the remaining parts of the anchorage was based on standard analytical techniques and are

described in detail in Reference 4. All components are within acceptable industry allowable values.

4.4 Nozzle Evaluation

Nozzle loads were previously evaluated using Welding Research Council Bulletin 107 (WRC 107). The 16", 8" and 4" nozzles associated with piping lines SI-04, SI-05, MW-01 and MW-11, respectively, were evaluated. All other small nozzles at the upper portion of the tank were not considered since the tank shell and small bore attached piping is flexible to accomodate associated pipe movements. The 16", 8" and 4" nozzles have piping runs with flexible bellows to minimize nozzle loads at the tank. The evaluations showed local stresses in the shell at the nozzle locations to be below the allowables of the ASME Code [11]. The results are documented in Reference 4. Additionally, the existing bellows on the 8" line was demonstrated to be structurally adequate under the applied loads. Bellows on the 4" and 16" lines are being replaced with newly designed bellows in order to accommodate the settlement and sliding displacements. 1 2

5.0 CONCLUSIONS AND RECOMMENDATIONS

The evaluation of the SONGS-1 RWST, as described in this report and in References [2], [3], and [4], has shown that the tank and its foundation are qualified according to the criteria of the ASME and ACI Codes. No modification to the tank or foundation is required.

6.0 REFERENCES

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2. "RWST Fixed Base Model," Impell Calculation No. RWST-1, Rev. 0, September 26, 1985, Impell Job No. 0310-058.

3. "RWST SSI Analysis," Impell Calculation No. CL-RWST-01, Rev. 1, January 18, 1986.

4. "Stress Evaluation of SONGS-1 RWST," Impell Calculation No. 0310-117-01, Rev. 0, March 4, 1986. ⚠

5. Haroun, Medhat A., "Vibration Studies and Tests of Liquid Storage Tanks," from Earthquake Engineering and Structural Dynamics, Vol. 11, No. 2, March-April 1983, John Wiley and Sons, New York.

6. "Topical Report: Seismic Analyses of Structures and Equipment for Nuclear Power Plants," Report No. BC-TOP-4-A, Rev. 3, November 1974, Bechtel Power Corp., San Francisco, California.

7. "Development of Soil-Structure Interaction Parameters, Proposed Units 2 and 3, San Onofre Nuclear Generating Station," prepared for SCE by Woodward-McNeill and Associates, Los Angeles, California.

8. "AWWA Standard for Welded Steel Tanks for Water Storage," ANSI/AWWA Standard D100-79, American Water Works Associated, Denver, Colorado.

9. ASME Boiler and Pressure Vessel Code, Code Case N-284, "Metal Containment Shell Buckling Design Methods, Section III, Division 1, Class MC," Approved: August 25, 1980, Reaffirmed: May 25, 1983.

10. EDSGAP Computer Program, Version 3/1/80, Impell Corporation, San Francisco, California.

11. ASME Boiler and Pressure vessel Code, Section III, Division I, "Rules for Construction of Nuclear Power Plant Components," 1983 Edition, Summer 1983 Addenda.

12. "RWST Tank Sliding Response," Impell Calculation No. TANK-01, Rev. 0, January 21, 1986. ⚠

13. San Onofre Nuclear Generating Station Unit 1, Backfill Soil Coditions Report, including Addendum 2.

14. Richart, F.E., Woods R.D., and Hall J.R., "Vibrations of Soils and Foundations," Prentice-Hall, Englewood Cliffs, New Jersey, 1970.

15. SONGS-1 Drawing No. 567783-10, "Foundation and Trench Details, Plans and Sections" (RWST Foundation Details).

16. Wozniak and Mitchell, "Basis of Seismic Design Provisions for Welded Steel Oil Storage Tanks," API Proceedings, Refining Dept., 43rd Midyear Meeting, Toronto 1978 (Vol. 57).

17. "Development of SSI Parameters to Proposed SONGS 2 and 3," by Woodward-McNeill and Associates; January 31, 1974. ⚠

Table 2.1

Mass Distribution and Frequency Response of Tank Model

	Mass	Elevation	Node Point (2) Horizontal	Node Point (2) Vertical	f_n(3)
Sloshing Fluid	9505 $\frac{lb \cdot s^2}{ft}$	27.3 ft	15	13	.298 Hz
Rigid Fluid	50760 $\frac{lb \cdot s^2}{ft}$	14.75 ft	12	12	7.12 Hz

Notes:

(1) Elevation above the tank base.

(2) Node points are shown on Figure 2.1

(3) Natural Frequencies in Hertz.

Table 2.2

Soil Properties Used in the SSI Evaluation

Case	Shear Modulus (ksf)	Shear Wave Velocity (ft/sec)	Material Damping (%) (1)	Poisson's Ratio	Weight Density (kcf)
Average Properties	1203	578	8.0	0.35	0.116

Notes:

(1) Based on major principal strain of 0.05 percent.

⚠

Table 2.3

Peak Accelerations from SSI Evaluations

Location	CLASSI Node	Peak Accelerations (g)
Roof	14	1.211
Sloshing Mass	15	0.572
Rigid Mass	12	0.824
Base	10	0.622

Table 2.4

Base Shears and Moments from SSI Evaluations

Analysis Case	Base Shear (k)	Base Moment (k-ft)
Shell evaluation (base of 0.25" shell)	1467	12553
Shell evaluation (base of tank 0.329")	1472	23347
Foundation	1472	28400

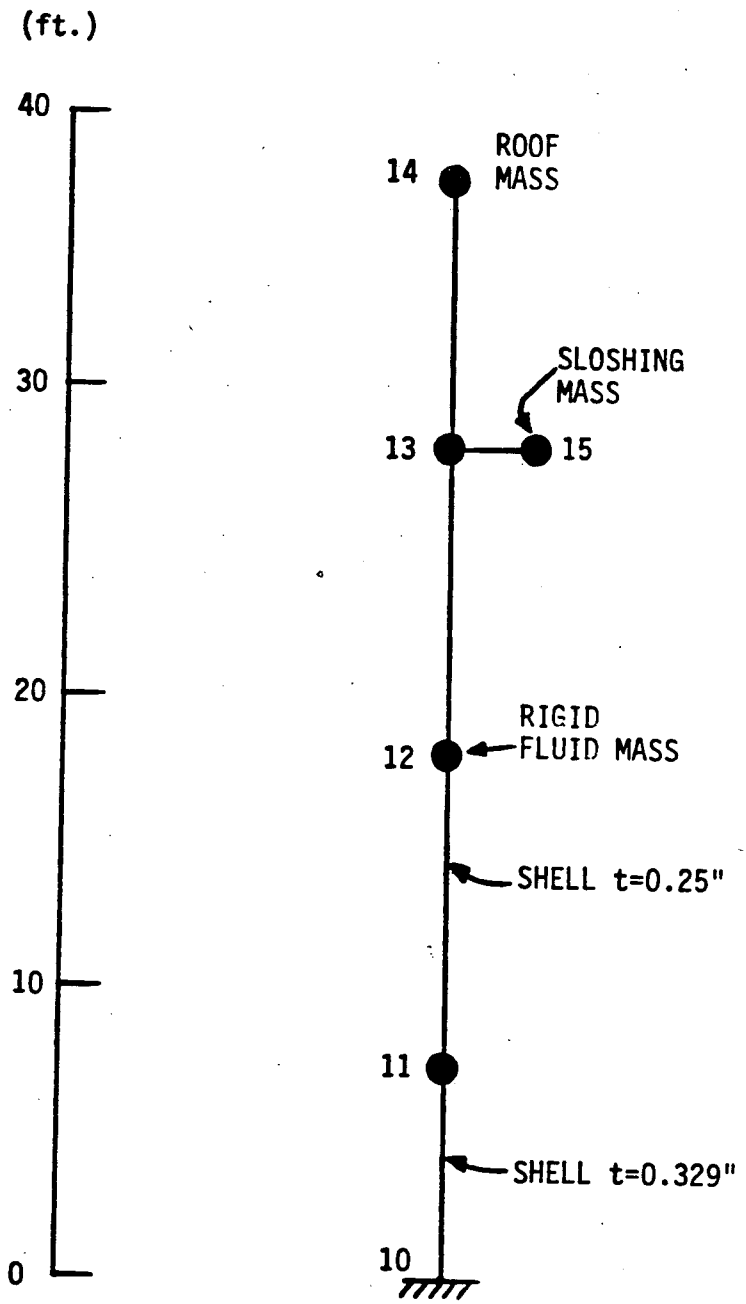


FIGURE 2.1 Soil Structure Interaction Model for RWST Evaluation

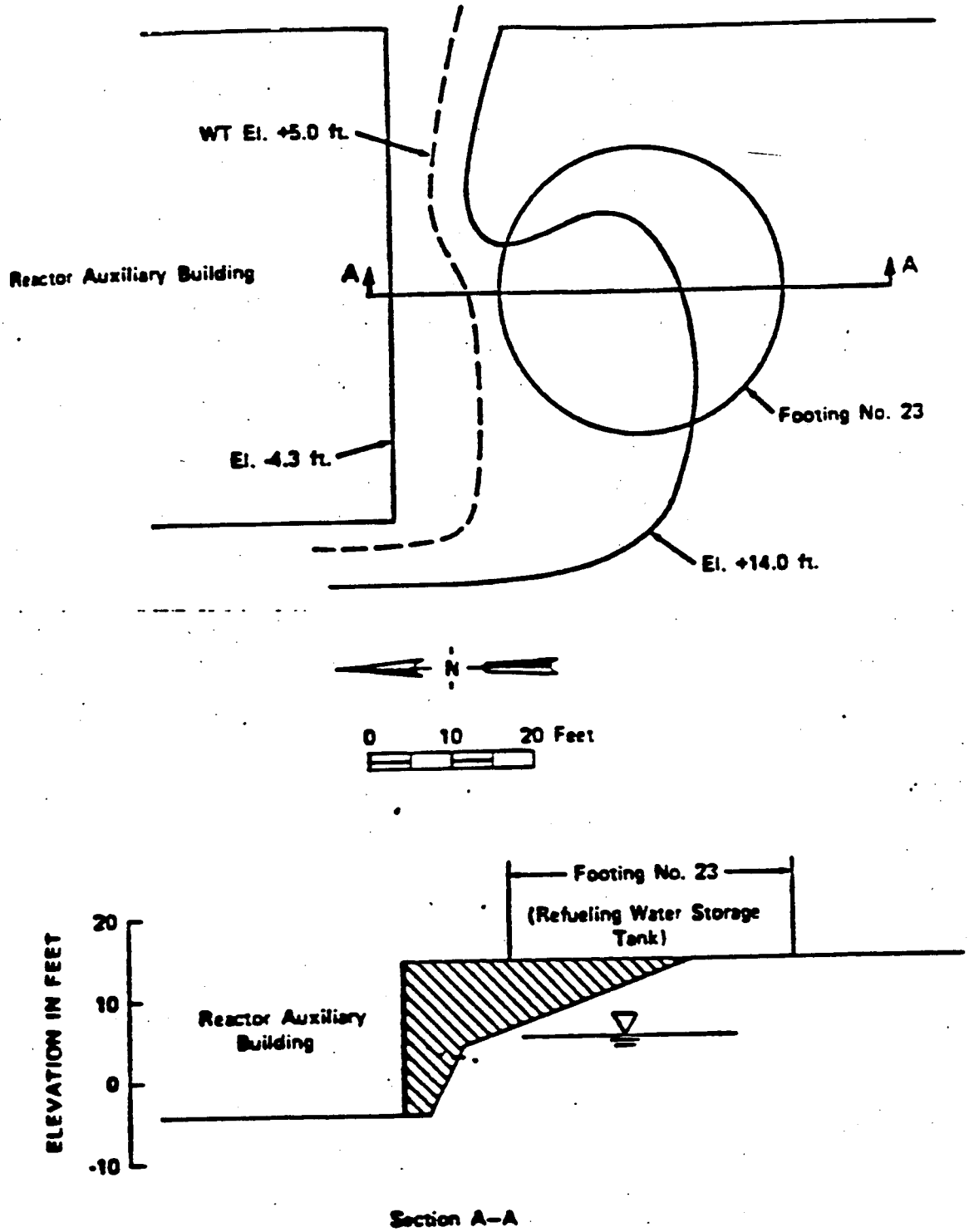


FIGURE 2.2 Local Soil Conditions Under Refueling Water Storage Tank, Item No. 23

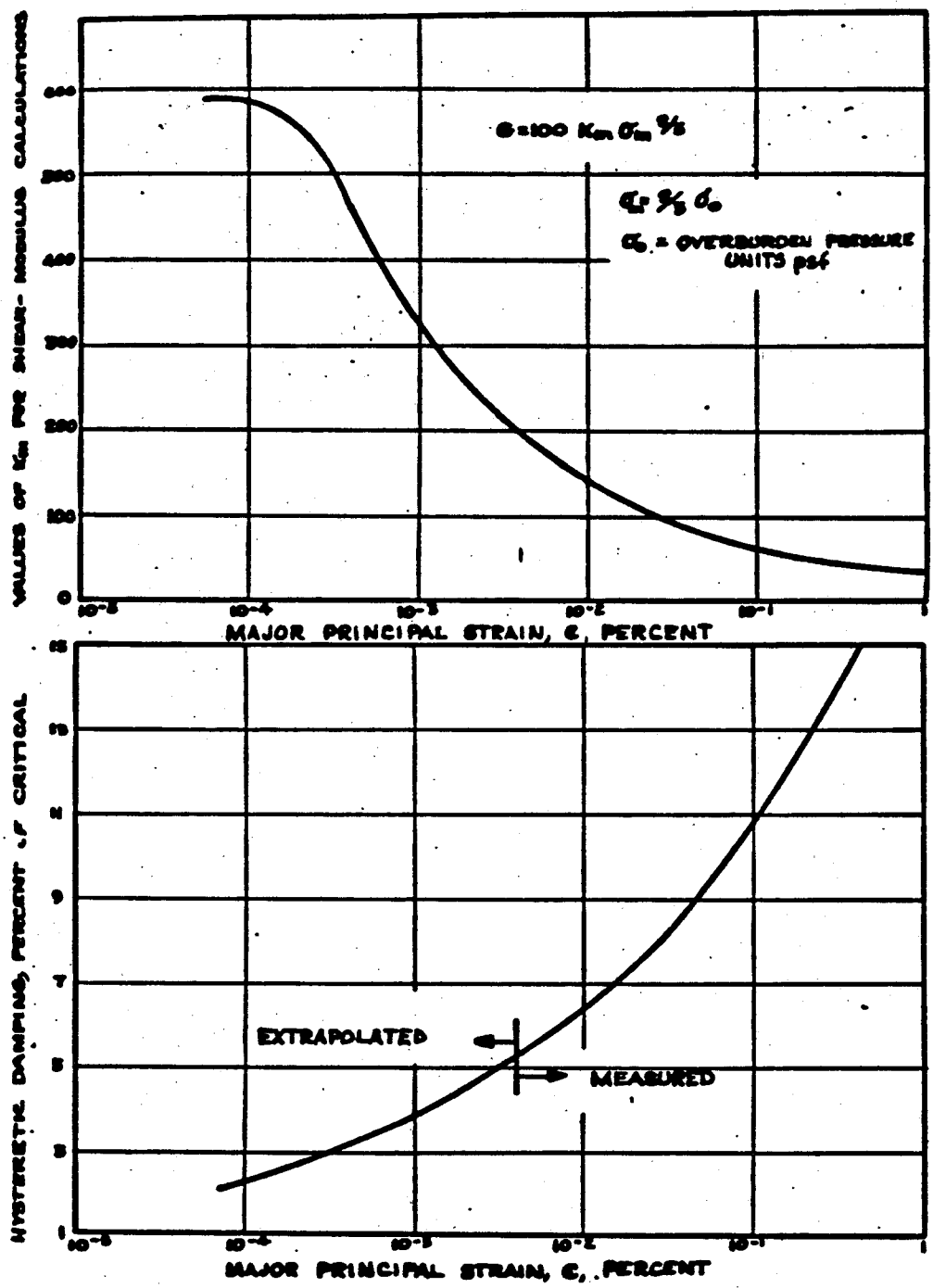


FIGURE 2.3 Modulus and Damping vs Strain, San Mateo Formation Sand

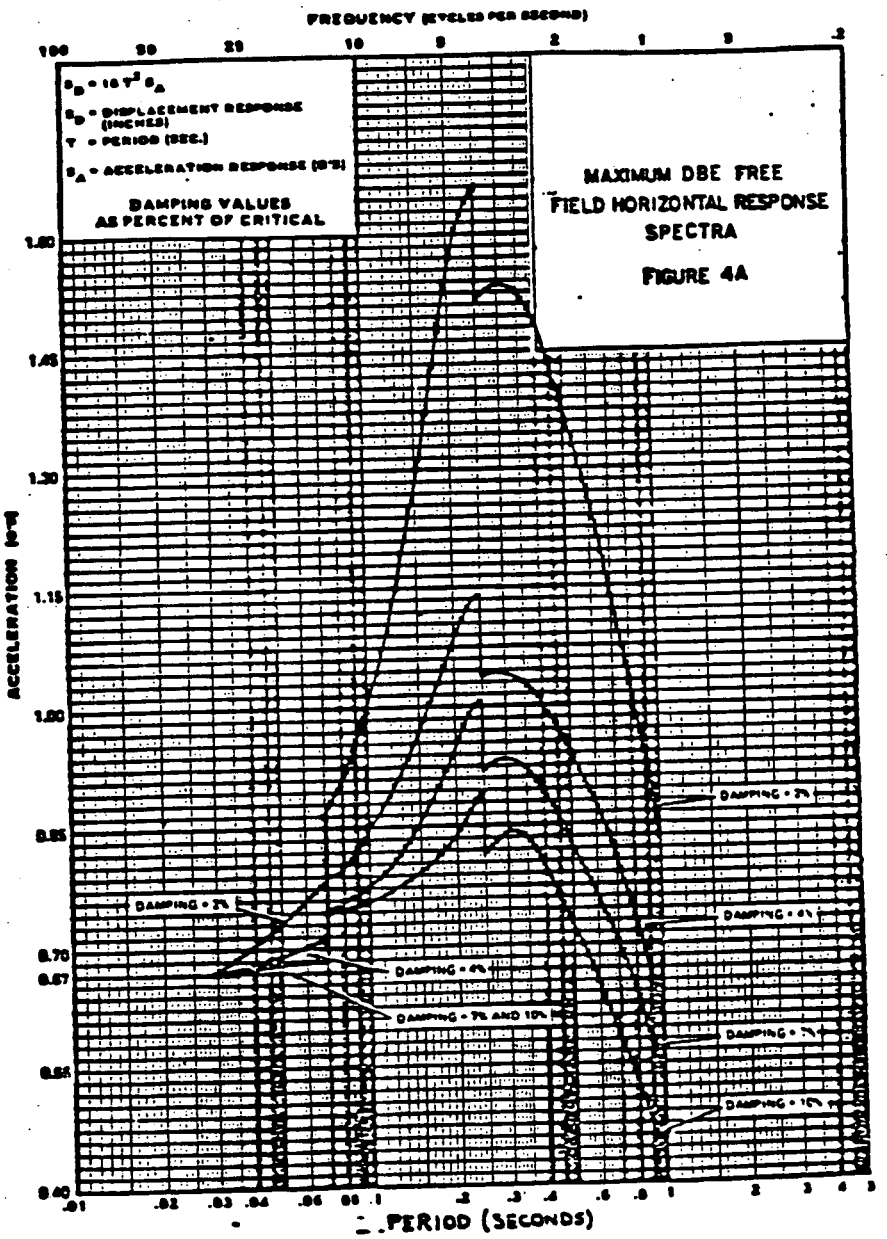


FIGURE 2.4 Free Field Response Spectrum, .67g
Modified Housner Spectrum, Horizontal

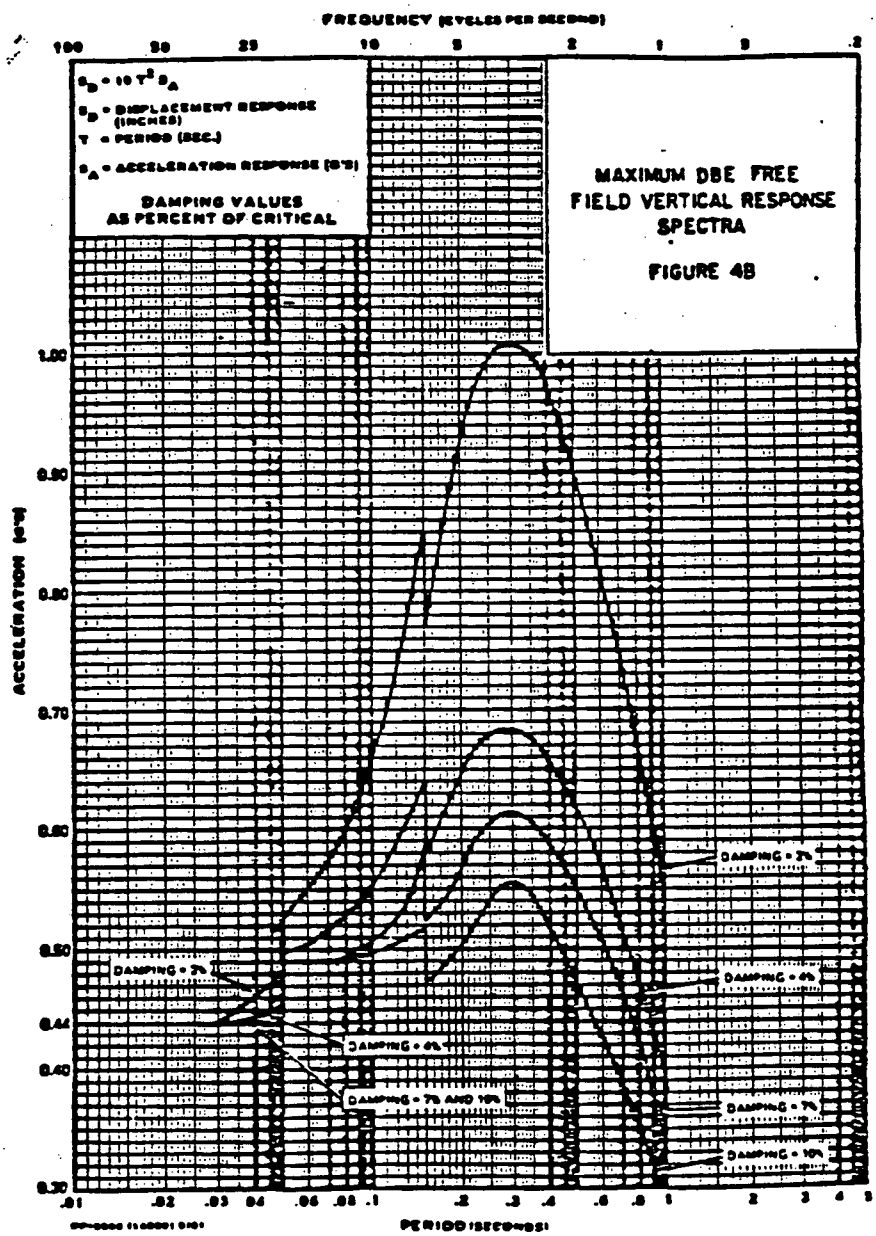


FIGURE 2.5 Free Field Response Spectrum, .67g
Modified Housner Spectrum, Vertical

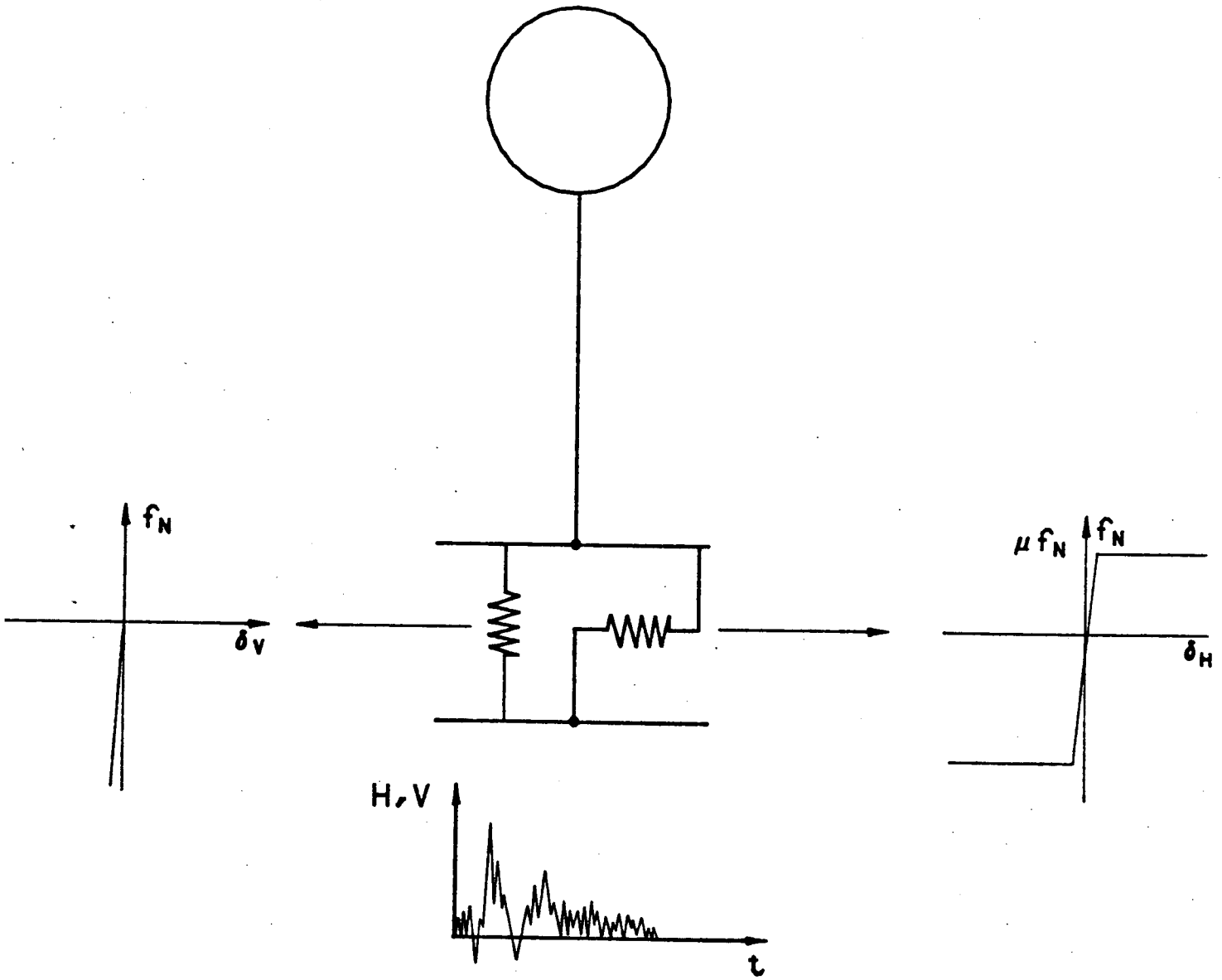


FIGURE 3.1 Sliding Analysis Model

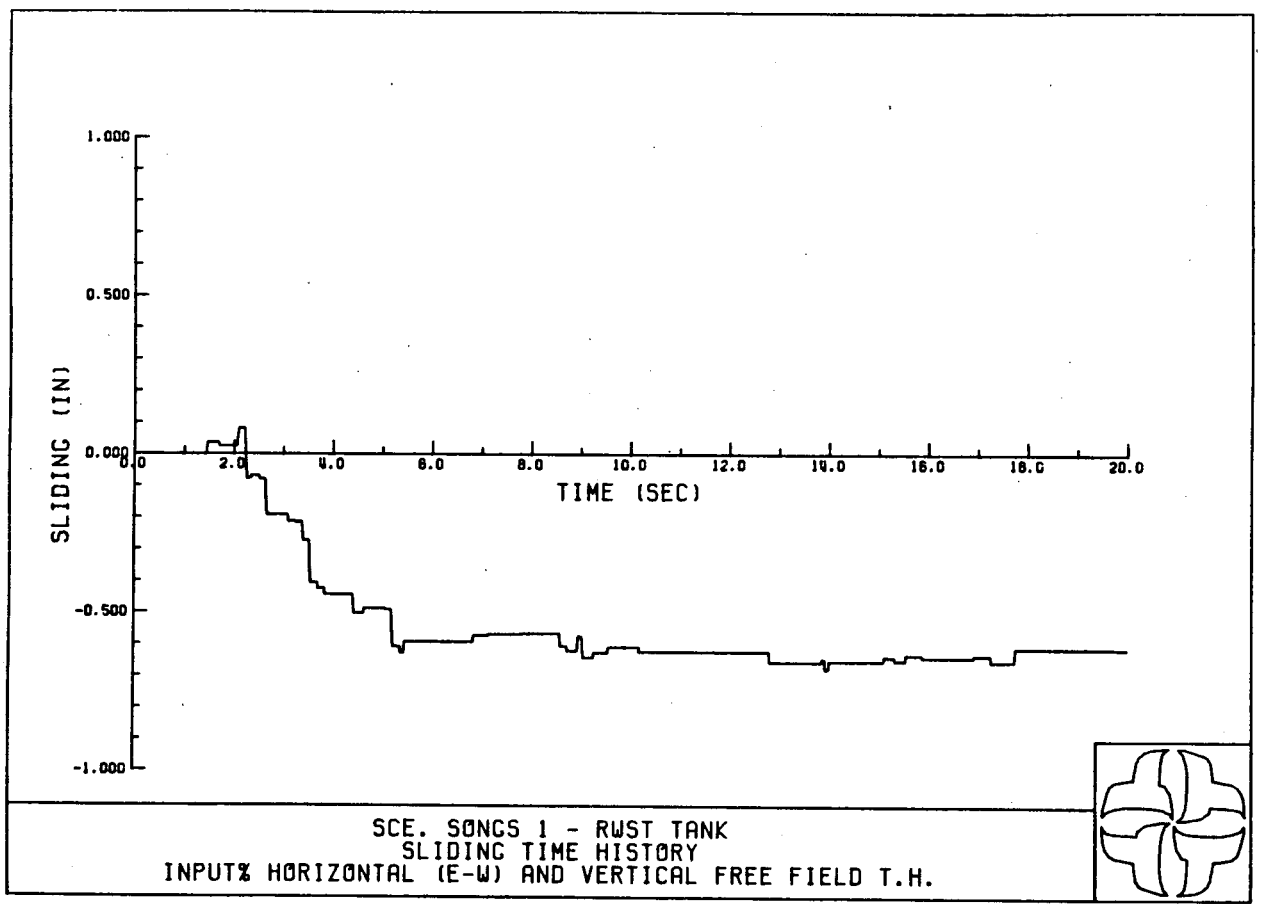


FIGURE 3.2 Sliding Displacement Results

Table 4.1

Compressive Stresses in Tank Shell

Section	Maximum Compressive Stress (1)	Allowable Stress(2)	Safety Factor
t = .25	4.8 ksi	7.85 ksi	1.64
t = .329"	6.7 ksi	9.66 ksi	1.44

Notes:

(1) Compressive stresses calculated using maximum loads produced at a given time point during the time history [3;4]. Maximum bending stresses, resulting from the horizontal excitation, were combined with axial stresses calculated using the spectral peak using SRSS.

(2) Allowable stress based on Code Case N-284, adjusted to account for the stabilizing influence of internal pressure.

Table 4.2

Maximum Principal Stress in Shell

Section	S_1 (1)	S (2)	Ratio to Allowable
t = .25	22.7 ksi	25.4 ksi	1.12
t = .329"	20.7 ksi	25.4 ksi	1.23

Notes:

(1) S_1 = principal stress calculated using maximum tensile hoop and longitudinal stresses and maximum shear stress.

(2) S = allowable membrane stress for level D loads, Table NC-3821.5-1 of [11].

Table 4.3

Summary of Results for the Concrete Basemat

Case	Load	Calculated Loads	Allowable Loads	Safety Factor
Settlement	Moment	110 k-ft/ft	163 k-ft/ft	1.48
	Shear	42 psi	108 psi	2.57
Seismic	Moment	145 k-ft/ft	163 k-ft/ft	1.12
	Shear	89.9 psi	108 psi	1.20

⚠

Table 4.4

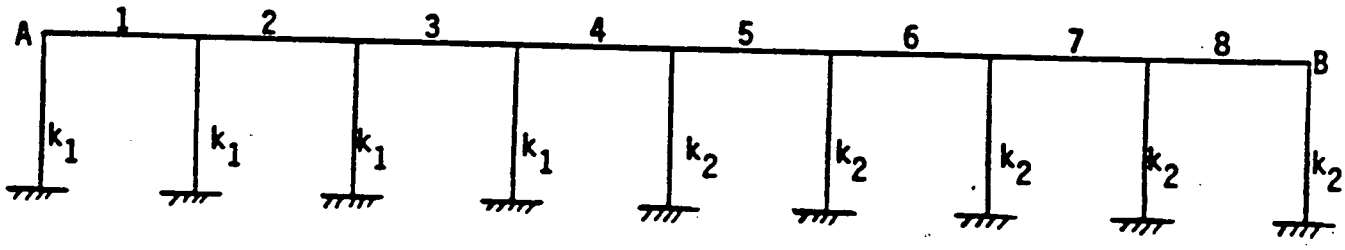
Summary of Results for Anchor Bolts

Soil Condition	Tensile Stress	Tensile Allowable	Shear Stress	Shear Allowable	Interaction	Interaction Allowable
Average	16.8 ksi	36.0 ksi	16.6 ksi	21.6 ks	0.81	1.0

Notes:

(1) Calculated assuming a friction coefficient of 0.55 between the tank base and the basemat.

⚠



Notes: 1. Elements 1 to 8 represent the basemat.

2. A uniform load was applied to the basemat and spring constant k_1 was reduced until the relative displacements between points A and B equalled 1.5 inches.

FIGURE 4.1 Model Used To Evaluate The Concrete Basemat For The Effects Of Soil Settlement

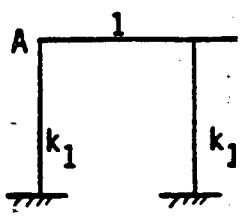
Table 4.4

Summary of Results for Anchor Bolts

Soil Condition	Tensile Stress	Tensile Allowable	Shear Stress	Shear Allowable	Inter-actior
Average	16.8 ksi	36.0 ksi	16.6 ksi	21.6 ks	0.81

Notes:

(1) Calculated assuming a friction coefficient of 0.55 between the the basemat.



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APPENDIX A: Description of Program CLASSI

APPENDIX A: DESCRIPTION OF PROGRAM CLASSI

Performs linear analysis of irregularly shaped rigid solids, with an arbitrary number of structures attached, in a viscoelastic half-space under dynamic loading.

FORTRAN (Extended)

Used to perform continuum analysis of soil-structure interaction problems, considering surface and body waves incident in three dimensions.

Uses a three-dimensional, finite element model of the superstructure, and a continuum model of the soil-rigid foundation system.

DECON, FREAK, RESPEC

CLASSI is a modular computer package with extensive capabilities and applications for the dynamic analysis of three-dimensional, linearly elastic, soil-structure interaction problems. It is used to determine continuum method solutions for these problems. The excitation input may be composed of any combination of surface and body waves at arbitrary horizontal and vertical angles of incidence.

The package consists of a number of interacting subroutines that produce output files directly read by one another. Input data to the pre-processing routines includes material properties, shear wave velocities, and thicknesses for each soil layer. Other routines read the geometry and material information of the finite element mesh defined for the superstructure. Finally, the output files from these routines, consisting of superstructure finite element information, impedance matrices, and driving force vectors, are used to determine the total soil-structure system response.

The methodology of CLASSI is based on a specialized form of the substructuring method. In this method, all the forces due to the dynamic response of the superstructure and foundation, and due to the soil

reactions and incident seismic waves, are balanced at the soil-foundation interface. Basically, the evaluation of the soil-structure interaction response involves:

1. Calculation of the forces at the soil-foundation interface caused by the incident seismic waves, assuming that the foundation does not move.

2. Calculation of the dynamic response of the soil-foundation-superstructure system when it is subjected to the foundation driving force vector.

In the Impell version of CLASSI, these calculations are achieved in the following three steps:

1. Computation of a soil-foundation complex impedance matrix and a driving force vector which characterized the soil-foundation system as it is subjected to the incident waves (Subprograms GHALF, GLAYER, and CLAF).

2. Representation of the superstructure by its mass matrix and fixed-based mode shapes and frequencies; the foundation is represented by its mass matrix. These calculated by any general finite element method program (for example, EDSGAP, IM-SNAP).

3. Combination of the results from the above two steps to form an overall soil-foundation-superstructure complex and frequency-dependent impedance matrix. This matrix is used, together with the foundation driving force vector, to obtain the complete soil-structure interaction response of the system (Subprogram SSI composed of subroutines SC1, SC2, RFFT, and COMSOL).

Output from CLASSI includes response histories for the half-space or soil layers for specified load cases. Structural responses of the superstructure are also output. All output may be saved directly on tape for use with FREAK, RESPEC, or other Impell post-processors. Plotting of output response histories is also possible.

A significant advantage of this package is the use of a condensation method for developing the mass matrix, since the higher modes are expected to respond as rigid masses. This significantly increases the computational efficiency of the program.

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