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

Prepared for

Southern California Edison Company

Prepared by

Impell Corporation
350 Lennon Lane
Walnut Creek, California 96598

Impell Report No. 01-0310-1392
Revision 0

April 1985

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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, and results of this evaluation are documented in Reference [1]. 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 [2]. 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. 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.

The SASSI computer code was used to perform the soil-fluid-structure interaction analysis of RWST. SASSI was selected based on its ability to consider varying soil properties beneath the tank. However, as discussed in Section 2.1.2, the dynamic properties of the soil were found to be essentially uniform, therefore this capability was not used in the analysis.

2.1.1 Tank/Fluid Model

The dynamic model of the RWST was developed using the analysis procedures of Haroun [5]. Haroun's procedure divides the mass of the tank into three distinct parts:

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

2. Mass associated with deformation of the tank shell (i.e., tank flexibility) relative to the base.

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

These masses, their location along the axis of the tank, and the frequency response of the tank's flexibility and convective masses were determined using the equations and the graphs in [5].

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 and the tank flexibility masses were attached to the shell with springs such that the frequencies of vibration of the spring mass systems were equal to the frequencies predicted using Haroun's analysis model.

Since the RWST is symmetrical, a two-dimensional model was used to represent the three-dimensional response.

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 nonuniformities 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.

For this tank, it was assumed that a stiffer soil-structure interaction system would result in higher base loads. Stiffer soil properties (i.e. shear modulus) would increase the SSI frequency and resulting loads. Therefore, upper bound soil properties were developed which are approximately 20 percent larger than the calculated values. The SSI evaluation of the RWST in the horizontal direction was performed for the following set of soil properties:

- Upper bound soil properties*
- Upper bound plus 50 percent soil properties

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 0.1 percent principal soil strain.

The upper bound 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. A second evaluation was performed using upper bound plus 50 percent soil properties to account for other uncertainties. Table 2.2 summarizes the soil properties used in this evaluation for the two cases. The assumption that stiffer soil increases the base shears and moments is verified in Section 2.3.

2.1.3 Control Motion

The SONGS-1 DBE 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

* Note that the shear modulus was increased and the soil density was decreased to maximize the shear wave velocity.

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 appliedin 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 SASSI (System for Analysis of Soil-Structure Interaction) computer program. SASSI is a three-dimensional equivalent linear analysis program developed by Professor John Lysmer at the University of California, Berkeley. It employs a frequency domain solution and uses a substructure approach. A description of SASSI is included in Appendix A.

The 2-D fixed-base model described in Section 2.1.1 was adapted for use in SASSI. The SASSI model consisted of:

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

The basemat was modeled using beam elements, with properties equivalent to those of the actual basemat. The tank model was connected to the ends of the basemat, using rigid beam elements to realistically model the attachment of the tank to the basemat. The SASSI model is shown in Figure 2.1. Two horizontal SASSI analyses were performed, one for the upper bound and one for the upper bound plus 50 percent soil properties.

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].

Both upper bound and upper bound plus 50 percent 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 SASSI 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. Table 2.4 shows that the base shears and moments are larger for the SSI analysis with upper bound plus 50 percent soil properties. Thus, these values are conservatively used for the tank evaluations.

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 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.

3.1 Shell Evaluation

Table 3.1 shows a comparison of the compressive stresses calculated for the shell and the allowable compressive stresses. The table lists two values of allowable compressive stress. The first value is 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. A safety factor of 1.0 was applied to the values listed in the table. The actual safety factor is found by dividing the allowable stress by the calculated stress.

The second allowable compressive stress is the value 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 3.1 show that all safety factors are greater than 1.0. The minimum factor of safety using the uncorrected CC-N-284 allowable is 1.37. This value is greater than the Code recommended safety factor of 1.34 for Level D events. When the conservatisms of the

analysis, including using the peak vertical response and upper-bound plus 50 percent soil properties, are considered, a factor of safety of 1.37 is more than adequate to demonstrate the integrity of the tank.

The minimum factor of safety against tank buckling increases to greater than 2.0 when the stabilizing effect of internal pressure is considered. The increase in allowable, which was calculated using the procedures in the AWWA standards [8], is in accordance with CC-N-284.

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 pressure resulting from vertical seismic excitation plus gravity loads. 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 3.4. As shown in the table, all stresses are less than the allowables, and the minimum safety factor is 1.47.

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

3.2 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 the equivalent beam model of the basemat developed for the SSI evaluation. 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. 3.1). 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 3.3). Based on the allowable loads of ACI-349, the safety factors were 1.48 for the basemat in bending and 2.32 in shear.

3.3 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 twice, the first calculations using a conservative estimate of the frictional resistance between the tank bottom and shell, and the second calculations using the extremely conservative assumption that frictional resistance to the shear force is negligible.

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 3.4) of .88 was obtained by conservatively neglecting the friction between the tank and foundation. The more realistic calculations considering the frictional resistance show an interaction of .28, a safety factor of 3.56.

Qualification of the remaining parts of the anchorage was based on a comparison of the loads calculated during the evaluation of the anchor bolts and the loads used for evaluation of the tank anchorage in [1]. As the current loads are much smaller than those used in [1], the bolt chairs (including stiffening rings and welds) and anchor bolt pullout are acceptable by comparison.

3.4 Nozzle Evaluation

Nozzle loads were previously evaluated using Welding Research Council Bulletin 107 (WRC 107) in [1]. Nozzle loads were also addressed in [12]. Both previous evaluations showed local stresses in the shell at the nozzle locations to be below the allowables of the ASME Code [11]. Therefore, nozzles are qualified based on [1] and [12] and were not reevaluated.

4.0 CONCLUSIONS AND RECOMMENDATIONS

The evaluation of the SONGS-1 RWST, as described in this report and in References [1], [2], and [3], 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.

5.0 REFERENCES

1. "Evaluation of SONGS-1 Refueling Water Storage Tank," Impell Report No. 04-0310-0061, Revision 1, June 1983.

2. "RWST Fixed Base Model," Impell Calculation No. RWST-1, Rev. 0, April 18, 1985, Impell Job No. 0310-058.

3. "RWST SSI Analysis," Impell Calculation No. RWST-SSI-01, Rev. 0, April 16, 1985, Impell Job No. 0310-058.

4. "Stress Evaluation of SONGS-1 RWST," Impell Calculation No. RWST-2, Rev. 0, April 22, 1985.

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.

5.0 REFERENCES

12. "RWST Evaluation," Impell Calculation No. EQ-22, Rev. 0, November 3, 1984, Impell Job No. 0310-036.

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).

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	1360 $\frac{lb \cdot s^2}{ft}$	27.3 ft	17	13	.298 Hz
Tank Flexibility	4502 $\frac{lb \cdot s^2}{ft}$	18.1 ft	15	12	7.12 Hz
Rigid Fluid	1876 $\frac{lb \cdot s^2}{ft}$	15.4 ft	11	11	Rigid

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)
Upper Bound Properties	1000	527	11.0	0.35	0.116
Upper Bound plus 50 percent Properties	1500	646	11.0	0.35	0.116

Notes:

(1) Based on major principal strain of 0.10 percent.

Table 2.3

Peak Accelerations from SSI Evaluations

SASSI Node	Peak Accelerations (g)	
	Upper Bound	Upper Bound Plus 50 Percent Soil
17	0.41	0.49
15	0.21	0.25
11	0.19	0.23

Table 2.4

Base Shears and Moments from SSI Evaluations

Analysis Case	Base Shear (k)	Base Moment (k-ft)
Upper Bound	511	11,100
Upper Bound Soil Plus 50 Percent	654	13,450

Table 3.1

Compressive Stresses in Tank Shell

Section	Soil Conditions	Maximum Compressive Stress (1)	Allowable Stress (2)	Safety Factor	Allowable Stress(3)	Safety Factor
t = .25	Upper Bound	2.83 ksi	4.52 ksi	1.60	7.95 ksi	2.81
	Upper Bound Plus 50 Percent	3.31 ksi	4.52 ksi	1.37	7.95 ksi	2.40
t = .329"	Upper Bound	3.21 ksi	5.95 ksi	1.85	9.85 ksi	3.07
	Upper Bound Plus 50 Percent	3.86 ksi	5.95 ksi	1.54	9.85 ksi	2.55

Notes:

(1) Compressive stresses calculated using peak loads from 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 stresses are based on Code Case N-284 with a safety factor of 1.0. No increase was taken for internal pressure effects, axial vs. bending stress distribution, transient effects, or other conservatisms.

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

Table 3.2

Maximum Principal Stress in Shell

Section	Soil Properties	S_1 (1)	S (2)	Safety Factor
t = .25"	Upper Bound	16.9 ksi	25.4 ksi	1.50
	Upper Bound Plus 50 Percent	17.3 ksi	25.4 ksi	1.47
t = .329"	Upper Bound	15.8 ksi	25.4 ksi	1.61
	Upper Bound Plus 50 Percent	16.1 ksi	25.4 ksi	1.58

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 3.3

Summary of Results for the Concrete Basemat

	Calculated Loads	Allowable Loads	Safety Factor
Moment	110 k-ft/ft	163 k-ft/ft	1.48
Shear	46.5 psi	108 psi	2.32

Table 3.4

Summary of Results for Anchor Bolts

Soil Condition	Tensile Stress	Shear Stress	Interaction	Safety Factor
Upper Bound(1) Plus 50 Percent	17.1 ksi	5.07 ksi	.28	3.6
Upper Bound	10.9 ksi	2.16 ksi	.10	9.8
Upper Bound(2) Plus 50 Percent	17.1 ksi	17.5 ksi	.88	1.13

Notes:

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

(2) Calculated neglecting frictional effects.

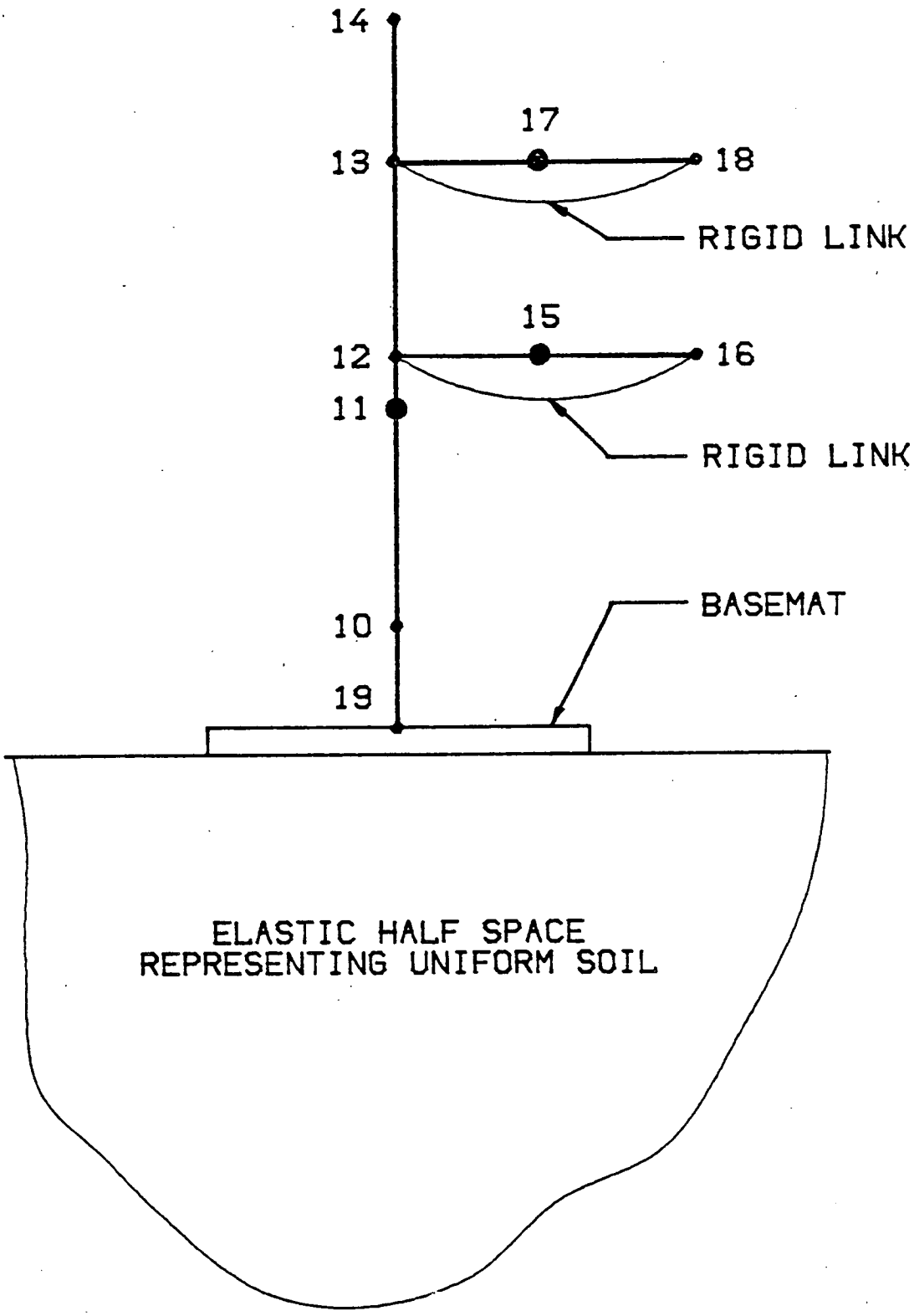


FIGURE 2.1 Soil Structure Interaction Model for RWST Evaluation

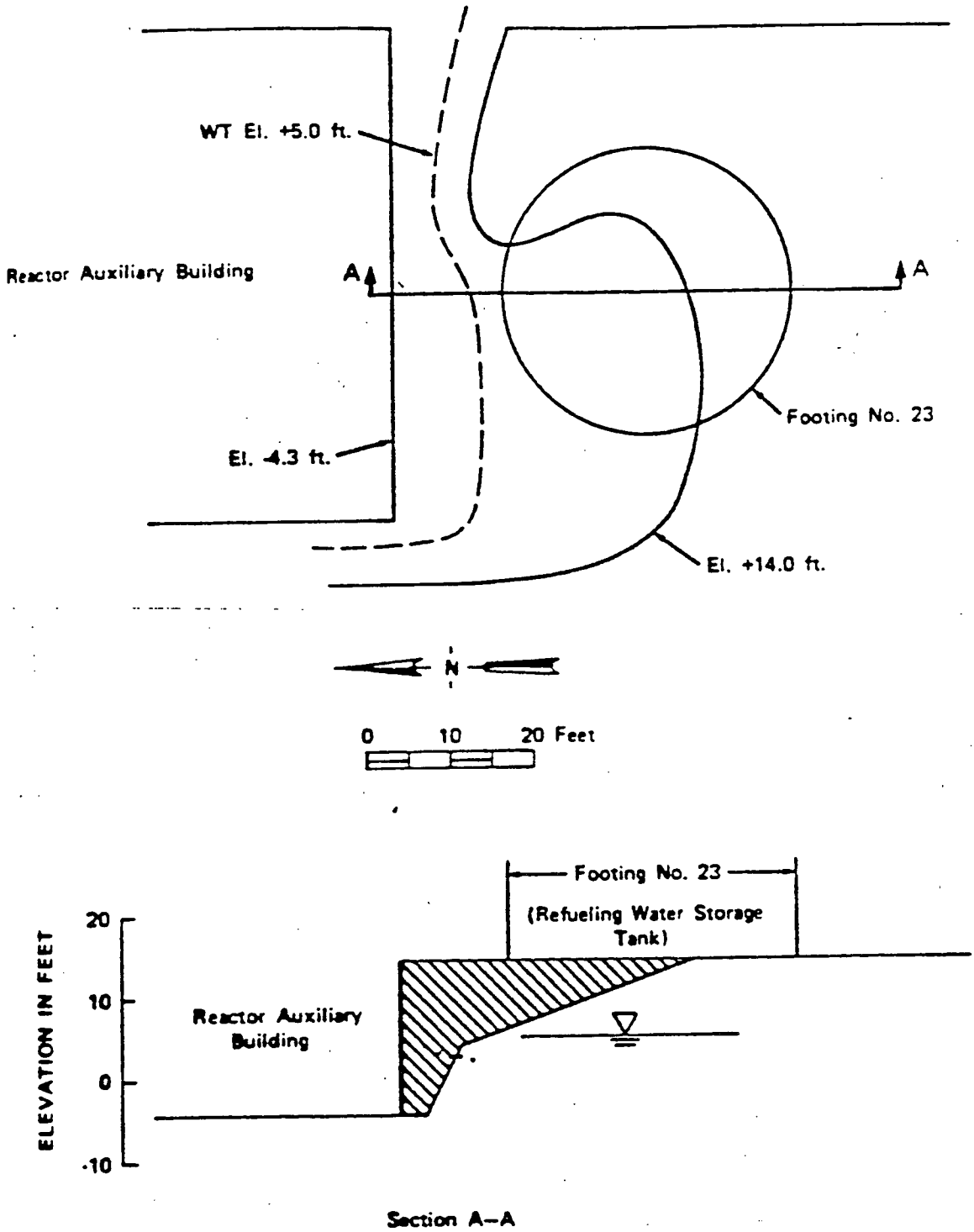


FIGURE 2.2 Local Soil Conditions Under Refueling Water Storage Tank, Item No. 23

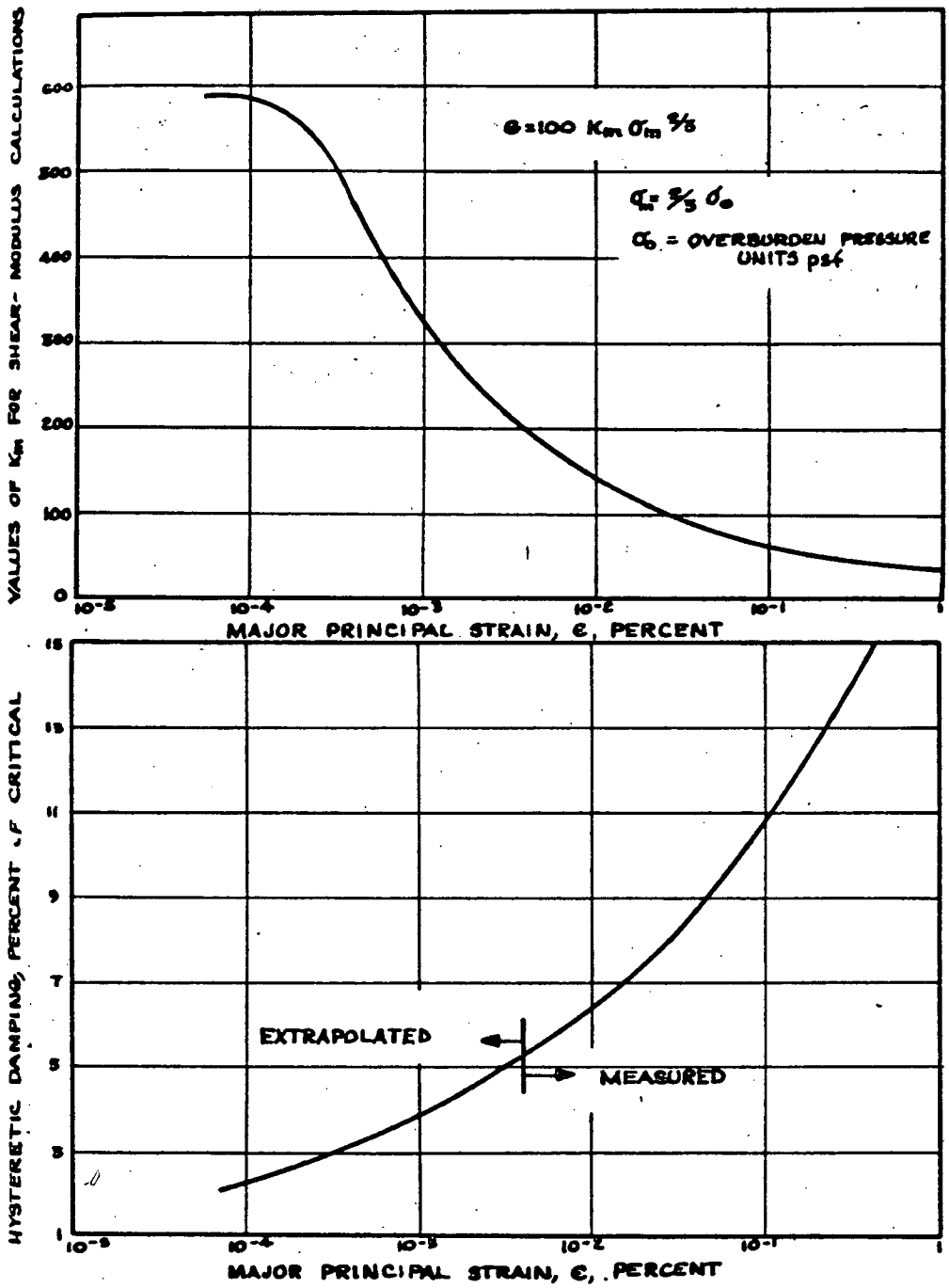


FIGURE 2.3 Modulus and Damping vs Strain, San Mateo Formation Sand

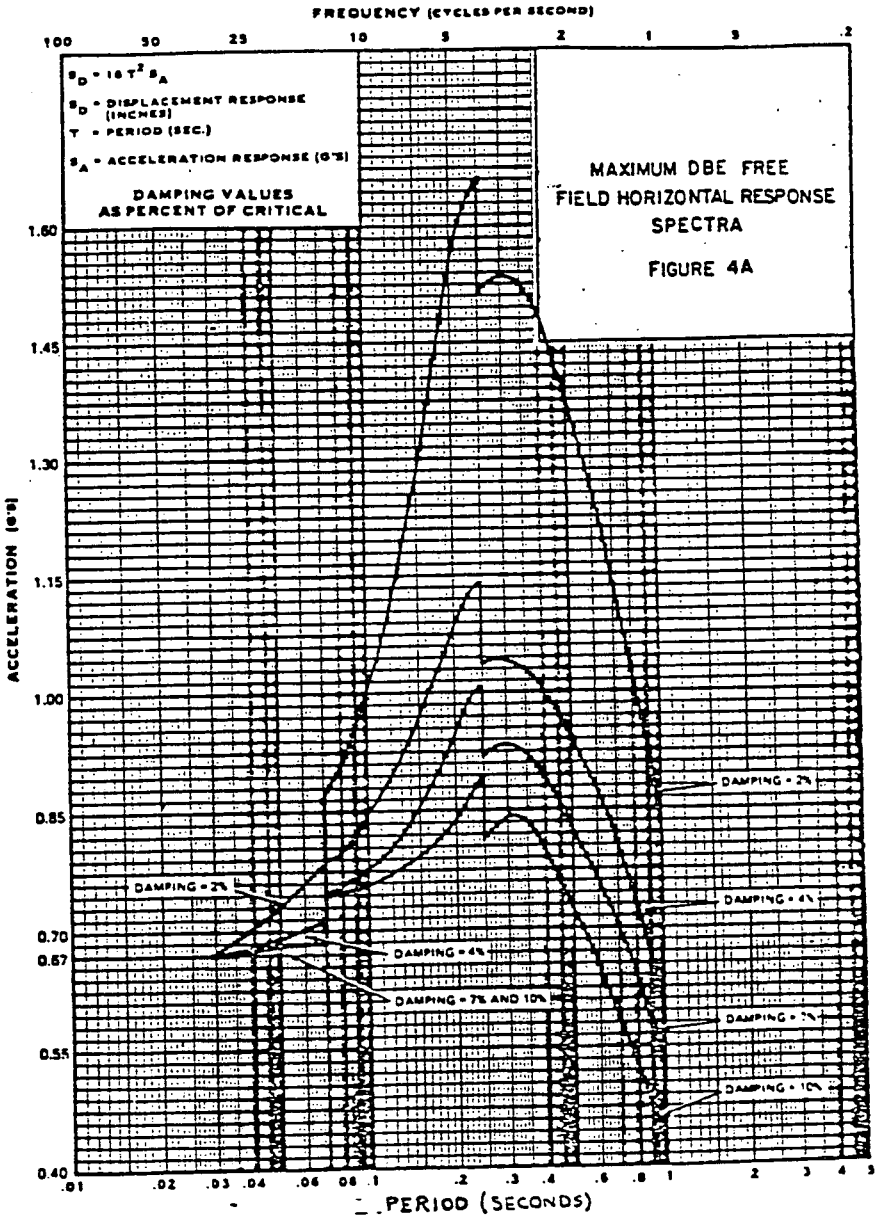


FIGURE 2.4 Free Field Response Spectrum, .67g
Modified Housner Spectrum, Horizontal

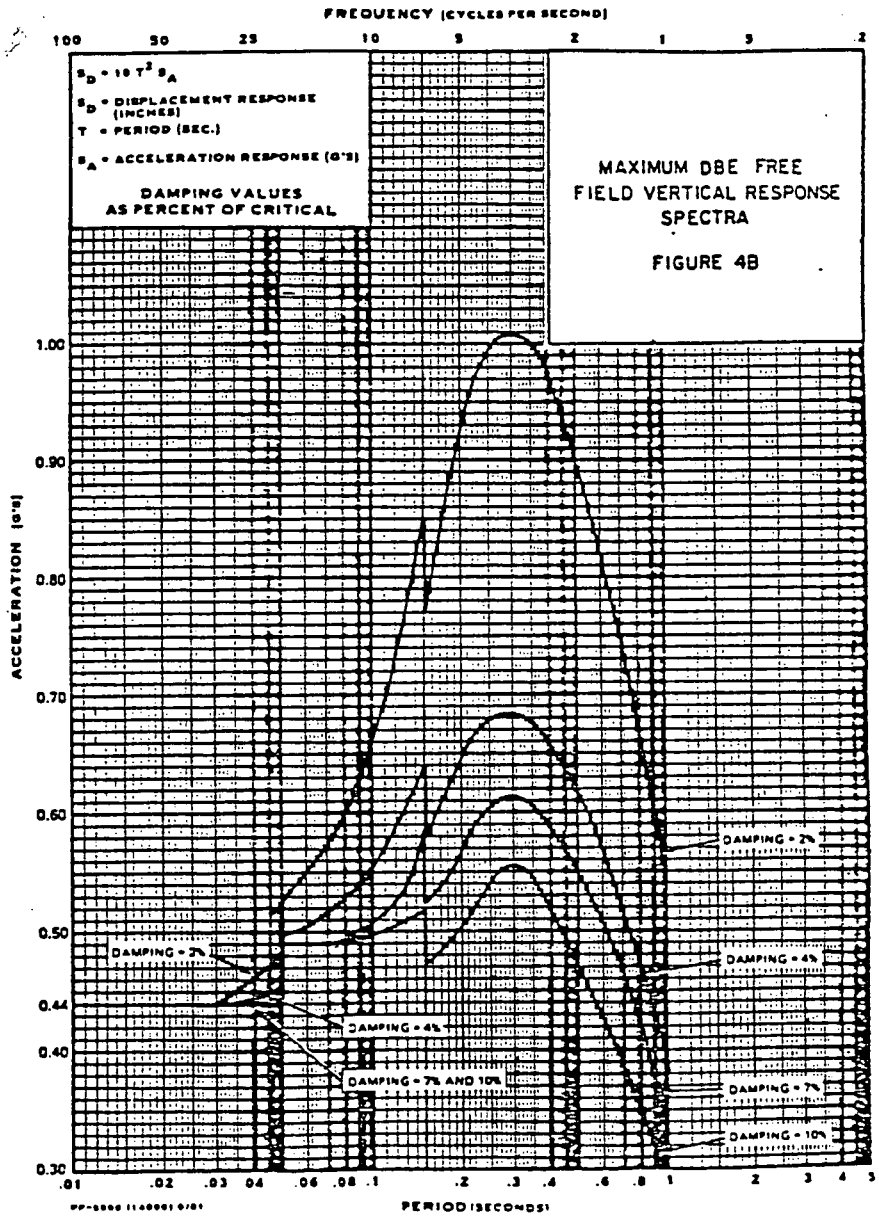
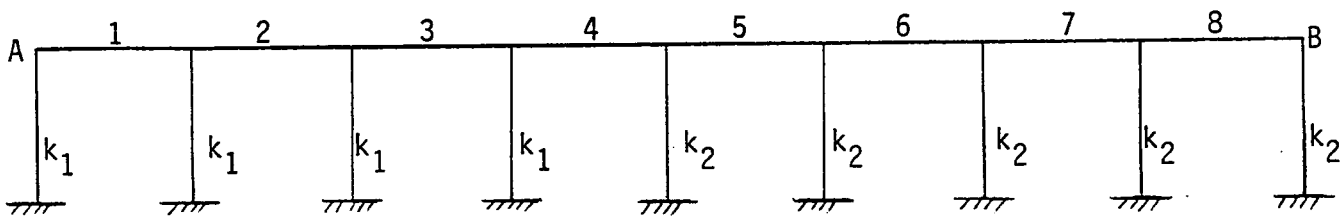


FIGURE 2.5 Free Field Response Spectrum, .67g
Modified Housner Spectrum, Vertical



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

APPENDIX A: Description of Program SASSI

SASSI

Overview

The computer program SASSI (System of Analysis for Soil-Structure Interaction) uses a substructuring method based upon a general finite element formulation. All analyses are performed in the frequency domain using the method of complex response.

Some of the features of SASSI include two- and three-dimensional analysis capabilities, consideration of both seismic and nonseismic loads, and the ability to approximately address nonlinear soil behavior. The seismic loads may consist of surface waves, vertical or inclined body waves, or any combination thereof.

Typically, a substructure approach requires that the SSI problem be divided into four constituent parts:

1. Determination of the site response under free-field conditions (i.e., the idealized site with no structures present).

2. Evaluation of wave scattering effects due to the presence of a stiff embedded foundation.

3. Formulation of the subgrade impedance matrix (i.e., the dynamic frequency-dependent subgrade stiffness, including embedment cavity).

4. Analysis of the site-structure system using Steps 1 through 3 to define forcing functions and boundary compatibility conditions.

SASSI uses a substructuring technique called the flexible volume method. This method eliminates the need to explicitly consider the wave scattering effects and simplifies the determination of subgrade impedance matrices. This is accomplished by partitioning the soil and the structure in such a way as to include the soil in the embedded part of the structure in the impedance evaluations. The SASSI substructuring scheme is briefly explained in the next section.

The Flexible Volume Method

The SASSI substructuring scheme is summarized as follows.

The complete system consists of two main substructures: The foundation, which consists of the original site (including the soil to be excavated), and the structure, which consists of the superstructure (region above grade level) and the basement (region below grade level) minus the excavated soil.

According to this method, interaction occurs at all nodes of the embedded portion of the structure; and the mass, damping, and stiffness matrices of the structure are reduced by the corresponding properties of the excavated soil. Therefore, the impedance matrix connects all the nodes in the foundation within the half-space, rather than just the soil-structure interface nodes as is typical of other substructuring methods.

Since the above substructure procedure leads to a regular boundary at the soil surface, the need for an explicit evaluation of the wave scattering effects at the embedded structure boundary is eliminated. Furthermore, the derivation of three-dimensional impedance matrices is reduced to that of determining the response of a layered system to point loads, which is an axisymmetric problem.

The above procedure is implemented in SASSI by formulating an axisymmetric model with axisymmetric transmitting boundaries and a layered half-space representation (whose depth varies with frequency), which may rest on a rigid base or on a viscous boundary. By using this model and successively applying point loads at all interaction nodes, the flexibility coefficients at all interaction nodes are computed. Once the flexibility matrix is computed, the impedance matrix is obtained by inversion of the flexibility matrix.

The SASSI substructuring technique leads to the following equation:

$$\begin{bmatrix} \underline{C}_{ss} & \underline{C}_{si} \\ \underline{C}_{is} & (\underline{C}_{ii} - \underline{C}_{ff} + \underline{X}_f) \end{bmatrix} \begin{Bmatrix} \underline{u}_s \\ \underline{u}_f \end{Bmatrix} = \begin{Bmatrix} 0 \\ \underline{X}_f \, \underline{u}_f' \end{Bmatrix} \qquad (1)$$

from which the final total motions of the structure can be determined. In these equations, s, i, and f refer to degrees-of-freedom associated with the nodes at the superstructure, basement, and excavated soil, respectively. $\underline{C}$ is the complex frequency-dependent stiffness matrix

$$\underline{C}(W) = \underline{K} - W^2\underline{M} \tag{2}$$

where $\underline{M}$ and $\underline{K}$ are the mass and complex stiffness matrices respectively, and W is the frequency of vibration. $\underline{u}$ is the vector of complex nodal displacements, and $\underline{X}_f$ is the frequency-dependent matrix which represents the dynamic stiffness of the foundation at the interaction nodes. $\underline{X}_f$ is usually referred to as the impedance matrix.

SASSI Computational Procedure

Equation (1) is the equation of motion of the full system. As indicated by this equation, it is necessary to compute for each frequency, W, the impedance matrix, $\underline{X}_f$, the load vector, $\underline{X}_f\,\underline{u}_f$, and the dynamic stiffness matrices for the structure, $\underline{C}_{ss}$, $\underline{C}_{is}$, $\underline{C}_{ii}$, and the excavated soil, $\underline{C}_{ff}$.

The SASSI computational procedure can be summarized as follows:

1. Form Dynamic Stiffness of the Structure: This is a complex frequency-dependent stiffness matrix of the form given in Equation (2). It is formed by using the total frequency independent stiffness and mass matrices using standard finite element procedures. The structural dynamic stiffness matrix is of the form:

$$\begin{bmatrix} \underline{C}_{ss} & \underline{C}_{si} \\ \underline{C}_{is} & \underline{C}_{ii} \end{bmatrix} \tag{3}$$

2. Form Dynamic Stiffness of Excavated Soil: This is a complex frequency-dependent stiffness matrix of the form given in Equation (2). It is formed by assembling, for each frequency, the individual

stiffnesses and masses for all elements used to model the excavated soil. The resulting $\underline{C}_{ff}$ matrix is shown in Equation (1).

3. Form Impedance Matrix. This matrix is obtained by inversion of the flexibility matrix developed for the interaction nodes. The frequency-dependent impedance matrix, $\underline{X}_f$, represents the dynamic stiffness of the original site in terms of all the common degrees-of-freedom between the structure and the foundation. The matrix $\underline{X}_f$ in Equation (1) is a square symmetric matrix, whose i-jth term represents the complex force amplitude required at degree-of-freedom i, if a unit displacement $e^{i\ Wt}$ is imposed at degree-of-freedom j, with all other nodes kept fixed.

4. Form the Total Stiffness of the System: The total stiffness of the system is obtained by adding $\underline{X}_f$ and subtracting $\underline{C}_{ff}$ from the total stiffness of the structure in Equation (3).

$$\begin{bmatrix} \underline{C}_{ss} & \underline{C}_{si} \\ \underline{C}_{is} & \underline{C}_{ii} - \underline{C}_{ff} + \underline{X}_f \end{bmatrix} \tag{4}$$

5. Triangularization: The full matrix in Equation (4) is then triangularized for back-substitution.

6. Form Load Vector: For seismic analyis, the load vector is computed by multiplying the impedance matrix, $\underline{X}_f$, by a vector which contains the free field motions at all the interaction degrees-of-freedom, $\underline{u}_f$. This is the vector

$$\begin{Bmatrix} 0 \\ \underline{X}_f \underline{u}_f \end{Bmatrix} \tag{5}$$

in Equation (1)

7. Solution of the Equations: The transfer functions from the control motion to the final structural motion are obtained by forward reduction and back-substitution of the load vector, using the reduced triangularized matrix obtained in Step 5.

SASSI Program Layout

The SASSI program consists of a series of subprograms to perform the different computational steps described in the previous section.

A brief description of each of the subprograms is provided below.

SASSI Subprograms

1. HOUSE

   This is a standard structural finite element program which contains a library of standard elements, including a three-dimensional solid element, a general beam element, plane strain element, spring element, and shell and plate elements. It computes $\underline{M}$ and $\underline{K}$ for both the structure and the excavated soil. The results are stored on Tape 4.

2. MOTOR

   This computes the complex load amplitudes due to any external forces that might be present, such as impact loads, vibrating machinery, or wind loads. The results are stored on Tape 9. This program is not used for seismic analysis.

3. SITE

   This solves the site response problem for a visco-elastic layered system on a half-space using iterated soil properties. It computes the mode shapes and wave numbers necessary for computing $\underline{u}'_f$ in Equation (1) for a unit control motion specified at any layer interface for a given wave field. It stores the results on Tape 1. This program also provides information required to compute the transmitting boundaries in POINT. This information is written on Tape 2.

4. POINT

   This solves the point load problem for the layered system and computes the mode shapes and wave numbers necessary for the computation of the

flexibility matrix $\underline{F}$ for the interaction nodes for the frequencies analyzed. The results are stored on Tape 3.

5. ANALYS

This is the general driver of the three programs MATRIX, LOADS, and SOLVE. It forms the matrices in Equation (1) and then solves the system to find the final transfer functions, which are stored on Tape 8.

6. MATRIX

This computes for each frequency the impedance matrix, $\underline{X}_f(w)$, and stores it on Tape 5. It also forms the total modified stiffness matrix in Equation (1), triangularizes it, and stores the results on Tape 6.

7. LOADS

This computes the load vector in Equation (1). For the seismic case, the load vector is equal to the impedance times the free field displacements.

8. SOLVE

This reads the reduced total stiffnesses and the load vectors, and performs the back substitution to obtain the total displacement transfer functions. It stores the results on Tape 8.

All frequency-dependent computations in SITE, MOTOR, POINT, and ANALYS are carried out for a selected number of frequencies. The remaining values of the transfer functions are determined by interpolation in the frequency domain.

9. MOTION

This is a deterministic post-processor which reads the transfer functions from Tape 8, performs the interpolation, and computes the final response at a specified number of nodes selected by the user. The program can compute time histories of accelerations (or displacements and velocities for the case of foundation vibration) and acceleration and velocity response spectra.

In its basic mode, the program reads the acceleration time history of the control motion from cards and transforms it to the frequency domain using a Fast Fourier Transform algorithm. It then reads the uninterpolated transfer functions from Tape 8 for selected output motions, performs the interpolation and the convolution with the control motion, and returns to the time domain using the inverse Fast Fourier Transform algorithm. The resulting time histories of acceleration may be output directly or converted to output response spectra.

10. RANDOM

This probabilistic post-processor is in many respects similar to subprogram MOTION. However, instead of using the time history of the control motion it uses a power or response spectrum of this motion. It then evaluates the probabilistic response of the structure. The output consists of mean values and confidence limits on selected design parameters such as maximum accelerations and bending moments. The program can also produce mean values and confidence limits of acceleration response spectra.

11. COMBINE

This combines the transfer functions on Tape 8 with new transfer functions from a second Tape 8. It is necessary to do this if it is found after interpolation that additional frequencies need to be computed.

12. STRESS

This program reads acceleration (displacement) transfer functions and information about elements from Tapes 8 and 4, respectively. For each requested element, it calculates the strain components for each frequency, performs interpolation and convolution with the control motion, and finds the stress and strain time histories by returning to the time domain using the inverse Fast Fourier Transform algorithm.