Attachment 3

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TECHNICAL EVALUATION REPORT - STRUCTURAL DESIGN ISSUES

LONG-TERM-SERVICE SEISMIC REEVALUATION

SOUTHERN CALIFORNIA EDISON COMPANY

SAN ONOFRE NUCLEAR POWER GENERATING STATION UNIT 1

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1.0 Introduction

1.1 PURPOSE AND BACKGROUND

This report provides technical evaluations of the Licensee's analysis of safety-related structures (including modeling techniques, methods of analysis, and results) with respect to the compliance with the Systematic Evaluation Program criteria, applicable industrial codes, and the applicable Nuclear Regulatory Commission (NRC) criteria. In the event that upgrading was necessary, the upgraded plan's methods and procedures were reviewed for acceptability.

The NRC determined that the modified Housner ground response spectra (a 10% increase in the 0.07 to 0.25 second period range for the horizontal component of acceleration and a 10% increase in 0.05 to 0.15 second period range for the vertical component), anchored at 0.67g was the appropriate ground motion for the seismic reevaluation of the San Onofre Nuclear Generating Station Unit 1 (SONGS 1). This spectrum is referred to as the 0.67g modified-Housner spectrum.

SONGS 1 was shut down from early 1982 to late 1984. Many structures, systems, and components were upgraded during this time by a program known as the Return-To-Service (RTS) program. The plant was then allowed to resume operation, provided that the remainder of the seismic reevaluation program and the resulting plant modifications were completed prior to startup from the current refueling outage.

In a meeting with the NRC staff on February 12, 1985 (Ref. 1.1), and through a letter dated March 12, 1985 (Ref. 1.2), the Licensee proposed their criteria and analysis methodology for the Long-Term-Service (LTS) upgrading. Further meetings and submittals clarified and, in some cases, modified the Licensee's proposals.

Section 2 of this report presents an evaluation of the refueling water storage tank, and Section 3 gives an evaluation of the turbine-building floor-response-spectrum generation. Section 4 evaluates grade beams, electrical duct banks, and the turbine building south extension. Section 5 evaluates the vent stack, Section 6 evaluates the steel-member design calculations, Section 7 presents our conclusions.

1.2 EVALUATION CRITERIA

SONGS 1 is one of the NRC designated Systematic Evaluation Program (SEP) plants, which were not designed to current codes and standards, and to current NRC licensing requirements. Therefore, the NRC has developed a set of criteria and guidelines for use in reviewing these plants. The following documents were used during the review of the SONGS 1 structure design:

a. NUREG/CR-0098, "Development of Criteria for Seismic Review of Selected Nuclear Power Plants," by N. M. Newmark and W. J. Hall, May, 1978.

b. "SEP Guidelines for Soil-Structure Interaction Review," by SEP Senior Seismic Review Team, December 8, 1980.

c. Letter from W. Paulson, NRC, to R. Dietch, the Southern California Edison Company (SCE), "Systematic Evaluation Program Position Re: Consideration of Inelastic Response Using NRC NUREG/CR-0098 Ductility Factor Approach," June 23, 1982.

d. Letter from W. Paulson, NRC, to R. Dietch, SCE, "SEP Topic III-6, Seismic Design Considerations, Staff Guidelines for Seismic Evaluation Criteria for the SEP Group II Plants-Revision I," September 20, 1982.

For cases not specifically covered by these criteria, the following Standard Review Plan (SRP) sections and Regulatory Guides were used:

a. Standard Review Plan, Sections 2.5, 3.7, 3.8, 3.9, and 3.10.

b. Regulatory Guides 1.26, 1.29, 1.60, 1.92, 1.100, and 1.122.

If the Licensee's proposed methodology and criteria deviated from these review criteria and guidelines, we reviewed and evaluated the justification presented by the Licensee, based on our experience and best engineering judgment. We understand that plant-specific deviations may be found acceptable on a case-by-case basis, as long as they reasonably meet the intent of the SEP review guidelines.

1.3 REFERENCES

1.1 Memorandum from E. McKenna to C. I. Grimes, dated February 12, 1985.

1.2 Letter from Mark Medford, SCE, to J. A. Zwolinski, NRC, dated March 12, 1985.

2.0 Evaluation of the Refueling Water Storage Tank

2.1 Introduction

The results of the Licensee's Long-Term-Service (LTS) reevaluation of the refueling water storage tank (RWST) are presented in Ref. 2.1. The results of the Licensee's calculations/analyses were reviewed and audited during the following review meetings: December 10-12, 1985; January 7-9 and 29, and February 6 and 18-19, 1986. To assist the assessment of the Licensee's methodology and seismic responses (including the soil-structure interaction (SSI) analysis of the tank) we performed an independent confirmatory analysis of the tank for the horizontal excitation. The RWST reevaluation was required for the Safety Evaluation Report (SER) issued for the San Onofre Nuclear Generating Station Unit I (SONGS-I) Long-Term Service (LTS) (Ref. 2.2, as Review Item 3.7). The findings and conclusions of our evaluation are summarized below.

2.2 Discussion

Our review of the Licensee's LTS reevaluation of the RWST was performed in three major areas:

a. A modeling of the tank, including the sloshing effect and soil foundation

b. A soil-structure interaction (SSI) analysis, and

c. A reevaluation of the tank and foundation.

These areas are discussed separately in the following three subsections.

2.2.1 Modeling of the Tank - For the horizontal analysis, the tank structure was represented by a three-mass stick model, and the contained water by two additional lumped masses. These two masses included a sloshing mass M_s and a rigid mass M_r. They were determined by using Housner's simplified theory (Ref. 2.3). We found the tank and fluid model for the horizontal analysis to be acceptable.

For the vertical analysis, the tank shell and water were represented by one rigid mass. We found this vertical model of the tank and water also to be acceptable.

2.2.2 Modeling of the Soil Foundation - The Impell version of the CLASSI code was used for the horizontal SSI analys[illegible] the RWST. We found the application of the CLASSI methodology to the hor[illegible] SSI analysis of the RWST to be acceptable. This finding was docum[illegible] in Ref. 2.2. In the Licensee's horizontal SSI analysis, the foundation soil, including the backfill, was assumed to be one uniform material. The shear modulus of 1390 kilopounds per square foot (ksf) for the soil was consistent with that used in the SSI analysis of the reactor building reevaluation, which was acceptable (Ref. 2.4). The soil material damping is taken to be 8%, which is acceptable to the NRC per the SONGS-I LTS SER, Item 3.10.2.2 (Ref. 2.2). We therefore found that the soil properties used in the horizontal SSI analysis of the RWST were acceptable.

For the Licensee's vertical SSI analysis, the soil impedance is represented by a constant vertical soil spring (determined in accord with Ref. 2.5). A 10% composite modal damping was assumed, although the actual computed value (per Ref. 2.1) exceeded 10%. The vertical SSI model is therefore a single degree-of-freedom system having a 10% damping and appropriate soil stiffness. In the vertical SSI analysis, additional conservatism was introduced by taking the spectral peak acceleration from the ground response spectra as the vertical response of the tank. Therefore, we found the results of the vertical SSI analysis to be acceptable.

2.2.3 Analysis Method for, and Results of, the Soil-Structure Interaction Analysis

The five-mass (including two water masses) stick model, with CLASSI-computed soil impedance, was used for the RWST horizontal SSI analysis. This analysis was performed by using Impell's version of the CLASSI code. The vertical seismic response was calculated by using a one-mass model with soil stiffness and a 10% system damping. This analysis was performed by hand calculation. The free-field modified Housner response spectrum for the 0.67g seismic event was used as input for the horizontal SSI analysis; 0.45g was used for the vertical analysis.

The responses of the tank were in-structure response spectra, the maximum base moments and shears, stresses, and overturning moments; all resulting from horizontal excitation. The in-structure peak accelerations were 0.62g, 0.82g, and 1.21g at the base, rigid fluid mass location, and roof of the tank, respectively. (The remaining results are discussed in sections 2.3.)

2.3 Tank and Foundation Evaluations - The tank and foundation were reevaluated by the Licensee, based on the seismic loads generated from the horizontal and vertical analyses. The audit was performed for the following areas: tank shell, anchorage, soil pressure, concrete mat (due to a postulated 1.5" seismically induced settlement of the backfill, and the seismic loads), nozzles and bellows, sliding and overturning stability, and tank roof. They are discussed separately in the following list.

a. Shell buckling was evaluated by comparing the maximum compressive stress in the tank shell with that allowable for buckling, according to the rules of ASME Code Case N-284. The maximum stress accounted for included the dead weight, seismic inertial loads, hydrostatic pressure, and hydrodynamic pressure. The resulting safety factor against buckling was greater than the 1.34 recommended by the code for Level D events. We found the buckling evaluation acceptable.

b. The maximum principal stress in the tank shell was evaluated against the Level-D-event allowable of the ASME B&PV Code, Section III, Division I, Subarticle NC-3800. The maximum principal stress was determined by combining the maximum tensile hoop stress (due to the hydrostatic pressure, hydrodynamic pressure, and vertical seismic load); the maximum tensile longitudinal stress (due to overturning moment from the horizontal seismic analysis); and the maximum shear stress derived from the horizontal seismic analysis. The evaluation criteria and analysis procedures appear adequate. The stress results are within the allowables.

c. The concrete mat foundation was evaluated against the seismic loads. In addition, a postulated 1.5-inch seismically induced settlement of the backfill soil was assumed beneath the northern and western portion of the tank base. The evaluation was done using vertical soil springs to represent the soil. The effects of soil settlement on the basemat were evaluated by using an equivalent beam model for the basemat. For the settlement evaluation, the mat was modeled as being supported by an elastic foundation. 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 displacement of 1.5-inches. A static analysis was then performed.

Another analysis was then performed, which considered the mat as being supported by a uniform elastic foundation, and loaded by the seismic loads and dead weight. The stresses in the concrete basemat from the two analyses were then combined by the algebraic sum method, and evaluated against the ACI-349 Code. The evaluation results appear adequate. In addition, the maximum soil pressure, as generated from the second analysis, was well within the allowable.

d. The anchor bolts were evaluated for tensile and shearing failures against the rules of the ASME Code, Appendix F, for Level D events. The anchor bolts were also evaluated against pullout failure from the concrete. We found the anchor bolt evaluation to be acceptable.

e. Local stresses in the shell at the nozzle locations were evaluated against the allowables from the ASME B&PV Code, Section III, Division I, Summer 1983 Addenda. The evaluation accounted for piping loads and the effect of the postulated 1.5-inch settlement of the backfill soil. We found the nozzle-load evaluation for the shell to be adequate.

f. The sliding stability of the tank and concrete mat was evaluated, based on a 0.59 sliding friction coefficient, because if the seismic base shear exceeded the friction resistance (normal force times friction coefficient), the concrete mat would slide. To determine the maximum distance of a slide, the Licensee performed a nonlinear time-history analysis of the tank using the ANSYS code. The tank was represented by a simplified one-mass stick model. The horizontal and vertical ground motions were simultaneously considered in the analysis, which showed that the maximum sliding distance is on the order of one-half inch. We found the sliding evaluation acceptable, as long as the bellows on the piping attached to the tank can withstand the effect of the tank's sliding, as discussed in Item h.

g. The overturning stability of the tank was evaluated by comparing the kinetic energy induced by the seismic motions to the potential energy required for the tank to tip over at an edge of the concrete mat (Ref. 2.5). The energy comparison showed a safety factor of about ten aginst overturning. We found the overturning evaluation to be sufficient.

h. Bellows and Nozzles - There are four bellows connected to the piping systems near the lower part of the tank. Two of them (G-50A and G-50B, respectively) are located between the tank and the safety injection pump on the two fourteen-inch safety injection lines. One (G-27) is located between the tank and the refueling water storage pump on the eight-inch miscellaneous water line. The fourth one is on the four-inch drain line. These bellows and the corresponding nozzles were evaluated against the seismic loads and soil settlement. The acceptance criteria for the bellows were based on the Standards of the Expansion Joint Manufacturers Association, Inc. The acceptance criteria for the nozzles were based on ASME B&PV Code, Section III, Division I, Summer 1983 Addenda. Bellows on the 4- and 16"-inch lines are being replaced with newly designed ones in order to accommodate the predicted soil settlement and sliding displacement. We found the bellows evaluation and local stresses in the shell at the nozzle loactions to be acceptable.

i. Tank roof - The Licensee did not evaluate the tank roof, based on their belief that a roof failure will not result in a loss of function of the refueling water storage tank. This was found to be acceptable.

2.4 CONFIRMATORY ANALYSIS

To assist in assessing the Licensee's methodology and seismic response of the soil-structure interaction effects, we performed an independent confirmatory analysis of the tank, in the horizonal direction. The vertical responses of the tank are much smaller than those in the horizontal direction, and were estimated conservatively by hand calculation. Therefore, we considered the confirmatory analysis performed in the horizontal direction adequate for assessing the methodology and modeling technique used by the Licensee.

The tank was modeled using 3-dimensional beam elements of the SAP4 computer code (Ref. 2.6). Masses were lumped along the axis of the tank at the appropriate heights, with two masses representing the contained fluid. The dynamic fluid model was developed using the analysis procedures of Housner

(Ref. 2.3). The mass of the fluid is divided into two parts in this procedure: mass associated with the first sloshing mode of the fluid (convective mass), and mass associated with the ground motion —(rigid or impulsive) mass. The rigid mass of the fluid was lumped with the tank shell at the calculated height, according to the procedure. At the same time, the convective mass was connected to the shell with a spring, so that the vibrational frequency of this mass-spring system equaled the sloshing frequency predicted by the Housner procedure.

We calculated the foundation impedance functions of the the tank by using two methods: the CLASSI approach (frequency-dependent impedance), Ref. 2.3, and the constant-soil-spring method, Ref. 2.2. For both methods, a uniform elastic half-space medium was assumed for the soil under the tank.

The constant soil-spring methodology is acceptable to the NRC, and has been used as a reference for evaluating other SSI methodologies.

The dynamic modal properties of the fixed-base structure were calculated by using the SAP4 code. For the constant-spring approach, the composite modal damping values were calculated by using the COMDAMP code. The LLNL versions of the computer codes, CLASSI and RESPNS, were used to generate structural responses for the CLASSI and constant-soil-spring approaches, respectively. CLASSI is a computer code for simultaneously analyzing the soil-structure effects and computing structural responses.

With the CLASSI approach, the peak accelerations were calculated as being 1.22g, 0.83g, 0.62g at the roof, the rigid fluid mass, and the base, respectively. For the constant-spring method, the peak accelerations were calculated as being 1.39g, 0.91g, and 0.65g, at the corresponding locations, respectively. The maximum difference between these two methods is about 14%. (Appendix A describes the confirmatory analysis.) Therefore, the CLASSI approach is acceptable.

2.5 COMPARISON OF LLNL AND IMPELL RESPONSE RESULTS

In order to assess the acceptability of the Licensee's methodology and results for generating the response of the tank, we compared responses generated by a comparable method: the CLASSI approach. As a result, we saw differences only between LLNL's and Impell's results in the second place after the decimal point. We therefore concluded that the Licensee's methodology and the results for the structural response of the refueling water storage tank are acceptable.

2.6 CONCLUSIONS

Our evaluation found the LTS reevaluation of the RWST to be acceptable.

2.7 REFERENCES

2.1 Impell Report, San Onofre Nuclear Generating Station, Unit 1, Evaluation of the Refueling Water Storage Tank for Long Term Service, Impell Corporation, Walnut Creek, California, Impell Report No. 01-0310-1392, Revision 2, 1986 (two volumes). Transmitted by letter dated 3/31/86.

2.2 Safety Evaluation Report by the Office of Nuclear Reactor Regulation, Long Term Service Plan - SEP Seismic Reevaluation, Criteria and Methodology, San Onofre Nuclear Generating Station Unit No. 1., Docket No. 50-206, September 19, 1985.

2.3 Nuclear Reactors and Earthquakes, TID-7024, August 1983: "Chapter 6 - Dynamic Pressure on Fluid Containers."

2.4 O. R. Maslenikov, et al., "SMACS - A System of Computer Programs for Probabilistic Seismic Analysis of Structures and Subsystems", Lawrence Livermore National Laboratory, Livermore, California, UCID 20413, Vol. I, March 1985.

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

2.6 S. J. Sackett, User's Manual for SAP4 (A Modified and Extended Version of the U. C. Berkeley SAPIV Code), Lawrence Livermore National Laboratory, Livermore, California, UCID-18226 (1979).

3.0 Evaluation of Turbine Building Floor-Response-Spectra Generation

3.1 INTRODUCTION

The floor response spectra generated by the Licensee for the turbine building (T/B) were audited during the review meetings held on July 1-2 and December 10-12, 1985. This audit was required by the Safety Evaluation Report (SER), Section 3.10.4 (see Ref. 3.1). The Licensee also provided a report on September 24, 1985 (Ref. 3.2).

3.2 DISCUSSION

The structural analysis model, including soil foundation flexibility for the T/B, is based on a model previously developed by Bechtel. A three-dimensional finite-element structural model was used to represent the T/B structure. The Bechtel SSI model uses frequency-independent soil springs, in accordance with the Licensee's proposed methodology for an SSI analysis of the T/B during the Systematic Evaluation Program (SEP) phase of the seismic reevaluation (Ref. 3.3). This approach was considered acceptable by NRC staff during the RTS evaluation (Ref. 3.5). For the given model, the Licensee previously proposed to generate the floor response spectra for the Long-Term-Service (LTS) phase using the direct generation technique implemented in Impell's FLORA computer code. The FLORA wide-band solution approach has been accepted by the NRC (Ref. 3.1). However, the Licensee changed the methodology to use a modified time-history-analysis technique in lieu of the direct generation technique.

For the modified time-history-analysis technique, a time-history modal analysis of the T/B model was performed with the Impell code EDSGAP, to generate the floor response spectra at required locations. The floor spectra from the time-history analyses were then multiplied by certain "correction factors" to obtain modified floor spectra, to account for the difference between the input synthetic time-history spectra and the modified Housner spectra. The correction factor varies with the structural frequency. At each frequency, the correction factor represents the ratio of the floor spectrum generated from the input (a synthetic time history) to the corresponding floor spectrum directly generated from the modified Housner spectra. The direct generation is based on the FLORA methodology, but the actual computation of the correction factors was done with another computer code, FACTOR, a special version of FLORA developed by Impell. The modified floor spectrum so generated excludes the interaction between the piping systems and the T/B structure, and is equivalent to using a zero value for the modal interaction mass, m_{ik}, when the floor spectrum is directly generated using the FLORA code. The Licensee indicated during the review meeting that they may consider using a non-zero m_{ik} wherever the piping system and structure interaction effect is deemed significant enough to warrant using it.

The modified time-history-analysis technique just described appears acceptable. However, we were concerned that the correction factor may be sensitive to whether the FLORA narrow-band or wide-band solution is actually used. We also felt it necessary to review the magnitudes of the correction factors at representative locations in the T/B, and to review the ordinates of the floor spectra at these same locations those that would have been directly generated from the wide-band solution of FLORA, based on either the modified Housner or the synthetic-time-history spectra being input. In addition, noting that the current floor spectra for the LTS typically are lower than the ones for the SEP, we felt that it was necessary to qualitatively compare the current uncorrected floor spectra to the corresponding spectra previously generated by Bechtel (during the SEP phase of the seismic reevaluation), particularly where a large discrepancy occurred between the two. Consequently, as a result of the July 1-2, 1985 audit review, we requested the Licensee to provide the following additional information:

a. Correction factors generated from both the narrow- and wide-band solutions for the 2% damping spectra at these locations in the T/B:

Direction	Elevation	Location	Node Numbers
N-S	42'	Area 2 Deck (A-53)	580, 586, 611
Vertical	35.5'	Area 6 Deck (A-60)	29, 71, 86

E-W	35.5'	Area 6 Deck (A-61)	29, 71, 86

b. The 2%-damping, floor-response spectrum was obtained by enveloping the three floor-response spectra at the corresponding three locations specified in Item a above, which were directly generated using the FLORA wide-band solution and based on either the modified Housner or synthetic time-history ground spectrum. For example, the floor response spectrum at the area-two deck (A-53) at an elevation of 42' is the envelope of three spectra (at nodes 580, 586, and 611). Contributions from the three earthquake components were to be combined with the SRSS technique for each of the three specified locations.

c. A comparison of the 2%-damping Bechtel-design floor spectra to the uncorrected floor spectra, as generated from the Impell time-history-analysis of the SSI model, at the three locations specified in Item a.

The Licensee provided Item c in Ref. 3.2, and Items a and b during the audit of December 10-12, 1985. The information provided appears sufficient to resolve our concerns on the acceptability of the T/B LTS floor spectrum.

3.3 CONCLUSIONS

Based on the outcome of the two audit review meetings and the additional information subsequently provided by the Licensee, we conclude that:

a. The structural modeling methodology appears acceptable because it is consistent with that used previously in the SEP phase of the reevaluation.

b. The correction factor approach for generating the modified (corrected) floor response spectrum from the time-history-generated spectrum appears acceptable for the case of $m_{ik} = 0$.

(c) The LTS floor spectra are qualitatively consistent with those previously generated by Bechtel during the SEP phase and, hence, appear adequate.

3.4 REFERENCES

3.1 Safety Evaluation Report by the Office of Nuclear Reactor Regulation, Long Term Service Plan - SEP Seismic Reevaluation, Criteria and Methodology, San Onofre Nuclear Generating Station, Unit No. 1, Docket No. 50-206, September 18, 1985.

3.2 "SONGS-1 Responses to NRC Request for Information, Turbine Building Response Spectra," Enclosure three to letter from M. D. Medford, SCE, to J. A. Zwolinski, NRC, September 24, 1985.

3.3 "Balance of Plant Structures Seismic Reevaluation Program, Turbine Building and Turbine Generator Pedestal, San Onofre Nuclear Generating Station, Unit 1,"Enclosure two to letter from K. P. Baskin, SCE, to D. M. Crutchfield, NRC, April 30, 1982.

3.4 "Balance of Plant Structures Seismic Reevaluation Criteria, San Onofre Nuclear Generating Station, Unit 1," Enclosure to letter from K. P. Baskin, SCE, to D. M. Crutchfield, NRC, February 23, 1981.

3.5 Safety Evaluation Report, by the Office of Nuclear Reactor Regulation, Return to Service, Criteria and Methodology San Onofre Nuclear Generating Station, Unit No. 1, Docket 50-206, Plan November 21, 1984.

4.0 Evaluations of the Grade Beam Design, and the Buried Electrical Duct Bank, and the As-Built Reevaluation of the Turbine Building South Extension.

4.1 Introduction

The Licensee's results with regard to the design of grade beams for the Auxiliary Feedwater Pump foundations, the reevaluation of the buried electrical duct banks, and the as-built reevaluation of the Turbine Building south extension are presented in Refs. 4.1 and 4.2. The results were reviewed by LLNL and its subconsultant, and an audit, was conducted at the Norwalk Office of the Bechtel Power Corporation on December 13, 1985. Our findings from reviewing Refs. 4.1 and 4.2, and from the audit meeting, are summarized in Sections 4.2 and 4.3.

4.2 DISCUSSION

Grade Beam for Auxiliary Feedwater Pump Foundation - Two new grade beams were designed and constructed in June, 1984 to address the effect of seismically induced settlement of the backfill soil supporting the Auxiliary Feedwater Pump foundations. Each of the two new grade beams are in turn supported on new concrete piers at the ends. The design is based on the assumption that the backfill soil does not provide any support to the pump foundations, and that the new grade beams and concrete piers provide the necessary support. The two ends were assumed to be a pin-pin connection: a conservative assumption. The analysis of the grade beams took into account all applicable dead loads, seismic loads, jet impingement loads, and pipe reaction loads during normal operation or in the shutdown condition. Hand calculations were used for the analysis. The design is based on ACI 318-1977. Both the analysis and the design appear to be adequate.

Buried Electrical Duct Banks - The buried electrical duct banks were reevaluated to address the effect of a seismically induced settlement of the backfill soil. In the reevaluation, the duct banks were assumed to have no support from the backfill where they traversed the backfill areas. With the backfill soil discounted, the duct banks were analyzed as beams having simply supported end conditions at the native soil/backfill interface, or of having fixed end conditions where the duct banks are embedded in concrete. In determining the moment capacities of the duct banks, the embedded conduits were taken as being reinforcements in addition to the actual reinforcements in the beams, if any. The modeling and methodology for analysis appear to be acceptable. The evaluation criterion is ACI 318-77. During the audit, we examined the design of the South and North Ducts, and the East-of-Pump-Well Duct (calculations LBC-CC 2.0, p.p. 7/60 and 8/60, 4/4/83; LBC-CC 2.0, p. 12/60, 5/6/83; and LBC-CC 2.0, p. 44/60, 4/1/83, respectively). The reevaluation results appear to be adequate.

As-Built Reevaluation of Turbine Building South Extension - The as-built modifications to the Turbine Building (T/B) South Extension are identified in Ref. 1. The evaluation is based on Ref. 4.3. The analysis is based on a finite-element structural model consisting of the South Extension, the turbine generator and pedestal, and the gantry crane. Structural damping is assumed to be 7% and soil material damping, 13%. Soil springs were used to represent the soil-structure interaction effect, and a maximum composite modal damping of 20% was used whenever the computed value exceeded 20%. Apparently, the soil material damping, used for the evaluation of the South Extension, exceeds the 8% value specified in the SONGS 1 LTS Safety Evaluation Report (Ref. 4.4). However, soil radiation damping is usually much higher than soil material damping. In addition, the licensee already conservatively limited the composite modal damping, which is a combination of structural damping and soil material and soil radiation damping values, to 20%. Based on the above mentioned reasoning and our sound engineering judgement, we concluded that the 13% of the soil material damping for the evaluation of the South Extension is acceptable. The BSAP computer code was used for the analysis. The analysis model and methodology appear to be adequate. Reference 4.2 provides the results of Licensee's additional evaluation of the structure for the effect of the shifting crane weight to the intact leg, and for the effect of impact when the uplifted leg is lowered onto the support rail. The addition evaluation indicates a safety margin of 1.63, which appears sufficient.

4.3 CONCLUSIONS

Based on our review of the information presented in Refs. 4.1 and 4.2, and the outcome of the audit, our conclusions are as follows:

a. The design of the two new grade beams, and their supporting concrete piers, are sufficient to address the concern about the seismically induced settling of the backfill soil.

b. The reevaluation of the four buried electrical duct banks appears adequate to address the concern about the seismically induced backfill soil settlement.

c. The reevaluation of the T/B South Extension including the gnatry crane appears sufficient.

4.4 REFERENCES

4.1 Letter from M. O. Medford of SCE to J. A. Zwolinski of the USNRC, dated September 24, 1985.

4.2 Letter from M. O. Medford of SCE to G. E. Lear of the USNRC, dated March 27, 1986.

4.3 Safety Evaluation Report by the Office of Nuclear Reactor Regulation, Return-to-Service Plan, Criteria and Methodology, San Onofre Nuclear Generating Station, Unit No. 1, Docket 50-206, November 21, 1984.

4.4 Safety Evaluation Report by the Office of Nuclear Reactor Regulation, Long-Term-Service, Criteria and Methodology, San Onofre

Nuclear Generating Station, Unit No. 1, Docket 50-206, September 18, 1985.

5.0 Evaluation of the Vent Stack

5.1 INTRODUCTION

During the December 10-12, 1985 review meeting held at Impell, we reviewed the evaluation results for the vent stack. The review was conducted by auditing the Calculation No. DC-1663 of SCE, dated 7/20/84. Our findings are summarized in Sections 5.2 and 5.3.

5.2 DISCUSSION

The stack is constructed of A36 steel plate. It is tapered from 4.5 feet in diameter at the top to 8 feet in diameter at the base, which is anchored with ASTM A193 anchor bolts (Grade B7, F_y = 105 ksi) to a 20 foot-diameter octagonal concrete foundation. The vent stack was evaluated for 0.67g modified Housner earthquake loads. The analysis model consisted of an 11-mass stick model run on the SAPIV code. A fixed-base analysis was made, which was sufficient, judging by the low first-mode frequency.

For the seismic evaluation, the allowables are 1.6 times the AISC code allowables. The seismically induced stresses at the base of the stack are within the allowables by a large margin. We reviewed the stress in the 48 one-inch diameter anchor bolts, which are pre-tensioned to 10 ksi during installation. The anchor bolts have a large margin of safety against seismic loads. We also audited the duct-opening stress condition, the stability of the vent stack against sliding and overturning, the maximum soil bearing pressure, and the anchor bolt pull-out capacity. Sufficient safety margins exist in all of these audited areas.

5.3 CONCLUSIONS

The vent-stack analysis for seismic loading indicated that there is a large safety margin against the allowables. Our audit found the vent stack to be sufficient for the seismic condition.

6.0 EVALUATION OF STEEL BEAMS SUPPORTING PIPING SYSTEMS

6.1 INTRODUCTION

According to the preliminary results of the LTS seismic reevaluation, the Licensee identified twenty eight of the steel beams supporting the safety-related piping systems as having exceeded the AISC-code elastic limits. They included two beams in the reactor building and twenty six in the turbine building, as listed in Table 6.1. Of these twenty eight steel beams, seven in the turbine building were shown to be within the elastic limits, as based on the final reevaluation results. The remaining twenty one beams have been (or are being) upgraded to meet the LTS elastic limit criteria, as identified in Table 1. We conducted an on-site inspection of the steel beams on April 14, 1986 and followed up with an audit at Impell's offices in Walnut Creek, California on April 16 - 18, 1986. Our findings are summarized in Sections 6.2 and 6.3.

6.2 DISCUSSION

Site inspection - The site visit was to inspect the as-built conditions, and the status of the modification design with regard to meeting the intended purpose of the modification. We also inspected most of the steel beams that had been upgraded prior to the LTS, as listed in Table 6.2.

Most of the modifications were accomplished using any one or a combination of the following types of modification design:

a. Reinforcing the beams for loading in the major axis by adding cover plates at the top and /or bottom flanges, adding web stiffener plates, or adding a structural T-section to the bottom flange.

b. Reinforcing the beams for loading in this minor axes by adding lateral bracings, and for loading causing torsional stresses by adding torsion-resisting assemblies.

c. Reinforcing the moment-resisting capacity at the end connections of the beams with additional welding.

In addition to upgrading the steel beams, five new columns, four of which are laterally braced, are being installed to provide additional structural support to the steel beams of the north extension mezzanine. The modification designs appear to be reasonable for serving their intended function.

Audit - The audit evaluated the modeling technique, method of analysis, and whether the calculated stress met the LTS elastic limit criterion, for those beams with (or without) modification. This criterion is that the stresses, in both the beam member and end connections, induced by the LTS loads must stay within 1.6 times the allowable stresses that are specified in the AISC code, Part 1.

a. Beams Requiring No Modification - For the seven steel beams that were qualified as elastic, and hence required no modification, we audited the Impell calculations for the beam member qualification, and the Bechtel calculations for the end connection qualification.

For the beam member qualifications, hand calculations were used for the analysis. All beams were considered as one-span, except for Beam No. EHP-B22 which was analyzed as a two-span beam. The beam ends were assumed to be simply supported, except for Beam Nos. NE-B4.4 and NE-B4.8, for which one end was assumed to be fixed (because of the moment connection design). The stresses from the building seismic loads were combined with those from the pipe support seismic loads using the square-root-of-the-sum-of-the-squares (SRSS) rule. The total seismic stresses were combined with the static load stresses using the absolute sum (ABS) rule. The stress induced by the pipe support seismic loads from the same (or different) piping systems were combined using the ABS rule. The safety factor, defined here as the margin of the LTS elastic limit allowe over the LTS load-induced maximum stress in the member, is listed in Table 6.1 for the seven beams in question. The results indicated very narrow margins.

For the end-connection qualifications, according to the Impell calculations, the end reactions from the LTS loads are smaller than the corresponding reactions previously used by Bechtel in their design. We audited the Bechtel design calculations in Calculations Nos. IPTC-CC-03.2 to CC-03.4, which showed that the existing end connections were within the elastic limit under the LTS loads.

b. Beams Requiring Modification - On a sampling basis, out of the twenty-one steel beams requiring modification, we audited the modified designs of sixteen of the beams contained in Bechtel's Calculation MODB--CC. Hand calculation was used in the analysis. The analytical method was the same as that applied to the seven beams requiring no modification. Primarily, the modification designs appear acceptable for the intended funct n. The modification designs primarily used one (or a combination) of the three methods previously described.

6.3 CONCLUSIONS

Based on the results of the on-site inspection and the subsequent audit review, we found that the LTS reevaluation of the twenty eight steel beams (listed in Table 6.1) appear to be reasonable. However, the extensive modifications of the Turbine Building North Extension, including the mezzanine, may substantially change the LTS seismic loads in the steel beams. In view of the very narrow margins in the seven beams not currently requiring modification, and the possible increase in the structural frequencies, the Licensee was required to evaluate the impact of the North-Extension structural modifications on the piping and on the building LTS seismic loads in the steel beams. Therefore, the acceptability of the steel-beam evaluation for the Turbine Building North-Extension, is contingent upon the outcome of the north extension reevaluation of the final modified configuration. For the steel beams in the Reactor Building, the Turbine-Building East-Heater Platform, and the Turbine-Building West Heater Platform, the modification is much less extensive and the current evaluation/modification appears to be acceptable.

TABLE 6.1 Twenty-eight former inelastic beams.

Location	Beam No.	Reevaluation Status	Acceptance
	RTS-B11	modified	yes
Reactor Building	RTS-B18	modified	yes
	NE-B4.3	modified	yes**
	NE-B4.4	elastic (FS=1.05)*	yes**
T/B North Extension	NE-B4.7	modified	yes**
	NE-B4.8	elastic (FS=1.04)	yes**
	NE-B5.2	modified	yes**
T/B North Extension	NEM-B2.9	modified	yes**
	NEM-B4	modified	yes**
	NEM-B2.10	modified	yes**
	NEM-B2.11	modified	yes**
Mezzanine	NEM-B2.2	modified	yes**
	NEM-B2.4	modified	yes**
	NEM-B2.5	modified	yes**
	NEM-B2.8	modified	yes**
	NEM-B5	modified	yes**
	NEM-B6	modified	yes**
	EHP-B5	elastic (FS=1.01	yes
	EHP-B6.2	elastic (FS=1.7)	yes
	EHP-B7	modified	yes
T/B East Heater Platform	EHP-B24	elastic (FS=1.24	yes
	EHP-B2	modified	yes
	EHP-B4	modified	yes
	EHP-B22	elastic (FS=1.01)	yes
	EHP-B3	modified	yes
T/B West Heater Platform	WHP-B4.1	elastic (FS=1.00)	yes
	WHP-B23.1	modified	yes
	WHP-B6	modified	yes

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* FS= safety factor of maximum LTS stress in beam member against 1.6 AISC Part I allowable.

** Acceptance is contingent upon the outcome of the reevaluation of the modified configuration.

TABLE 6.2 Additional Steel Beams Inspected During Site Visit

Location	Beam Number
Reactor Building	RTS-B4 RTS-B8 W18x96 (NEAR RTS-B19) RTS-B19 RTS-B57 RTS-B57 RTS-B72
T/B North Extension	NE-B5.1 NE-B5.4
T/B North Extension Mezzanine	NEM-B1.1 NEM-B1.2 NEM-B2.3 NEM-B2.6 NEM-B2.7 NEM-B2.12 NEM-B3.1 NEM-B3.1 NEM-B100 NEM-B105
T/B West Heater Platform	WHP-B5 WHP-B7
T/B	TB-B103 E-5

7.0 CONCLUSIONS

A detailed review was performed to provide technical evaluations of the structural upgrading design, analysis, and load generation at the San Onofre Nuclear Generation Power Station, Unit 1. The structures reviewed in this report include the refueling water storage tank, turbine building, buried grade beams, electrical duct banks, turbine building south extension, vent stack, and steel members. Reviews of the Licensee's criteria, methodologies, and results, along with additional information provided by the Licensee led to the following conclusions.

a. Refueling Water Storage Tank:

- For the tank and soil models, the CLASSI method of analysis for soil-structure for interaction effects appears to be acceptable.

- The evaluation of the tank shell, concrete mat, anchor bolts, bellows, and nozzles appear to be adequate.

b. Turbine Building Floor Response Spectrum Generation:

- The SSI modeling methodology appears acceptable.
- The correction-factor approach for generating the modified floor-response-spectra from the time history generated spectra appears acceptable.

c. Grade Beams inside the Turbine Building:

- The evaluation of the buried beams appears to be acceptable.

d. Electrical Duct Banks:

- The evaluation of the electrical duct banks appears to be acceptable.

e. As-Built Turbine Building South Extension:

- The evaluation of the Turbine Building South Extension appears to be acceptable.

f. Vent Stack:

- The evaluation of the vent stack appears to be acceptable.

g. Secondary Steel Beams Supporting Piping Systems:

- The evaluation of the secondary steel beams appears to be acceptable. However, due to the extensive modification of the Turbine Building North Extension (including the mezzanine) and the very narrow margins in the seven beams not currently requiring modification, the Licensee was required to evaluate the impact of the modification on both the piping and the building seismic loads in the beams.

APPENDIX A

CONFIRMATORY SOIL-STRUCTURE INTERACTION ANALYSIS

OF THE REFUELING WATER STORAGE TANK

FOR

LONG-TERM SERVICE

SAN ONOFRE NUCLEAR GENERATING STATION, UNIT 1

A.1.0 INTRODUCTION

This appendix provides the results of an independent soil/structure/fluid interaction analysis of the refueling water storage tank (RWST), using Licensee supplied input motions for the San Onofre Nuclear Generating Station (SONGS), Unit 1, Long-Term-Service (LTS) seismic reevaluation. It describes the methodology, the development of the structured and fluid models, the input motions, and the results for the independent analysis. Finally, a comparison between the independent results and the Licensee's results for the responses is presented.

Two methods for modeling the soil/structure interaction were used. The first one is the CLASSI methodology, which was also the one adopted by the Licensee. The second is the constant impedance methodology. Both have been used as our technical evaluation basis in a previous Technical Evaluation Report for the SONGS-1 LTS program (Ref. 2.2).

A.2.0 STRUCTURAL-FLUID MODELS

The SONGS-1 refueling water storage tank (RWST) is a surface-founded cylindrical steel shell with a conical roof (Fig. 1). Of the basemat's 35.5 foot diameter, 40% sits on native San Mateo sand, and 60% on the shallow (up to 8-foot depth) backfill soil (Fig. 2). The tank is anchored with 32 1-5/8 inch diameter anchor bolts embedded in the concrete basemat to a steel base ring, stiffening ring, and stiffening plates which are welded to the tank shell. There are four piping lines with bellows connection to the lower part of the tank. The tank was assumed to be full of water.

The tank was modeled using 3D beam elements of the SAP4 computer code (Ref. 2.6, Section 2.7). Mass was lumped along the axis of the tank at the appropriate height.

The dynamic fluid model was developed using Housner's analytical procedures (Ref. 2.3 of Section 2.7). The mass of the fluid is first divided into two parts: a mass associated with the first sloshing mode of the fluid — a convective mass; and a mass associated with the ground motion — rigid (or impulsive) mass. Following Housner's proceedure, the rigid mass of the fluid was lumped with the tank shell at the calculated height. The convective mass is connected to the shell with a spring, so that the frequency of vibration of this mass/spring system becomes equal to the frequency predicted by the procedure. This fixed-base tank/fluid mathematical model is shown in Fig. 3.

A.3.0 CALCULATION OF FOUNDATION IMPEDANCES

A.3.1 Soil Profile

The soil at the SONGS-1 site is the uniformly dense San Mateo sand extending to about 1000 feet below site grade. However, 60% of the soil (up to depth of 8 feet) under the RWST is backfilled with San Mateo sand (at a relative compaction of 92%), with the remaining 40% of the foundation being on

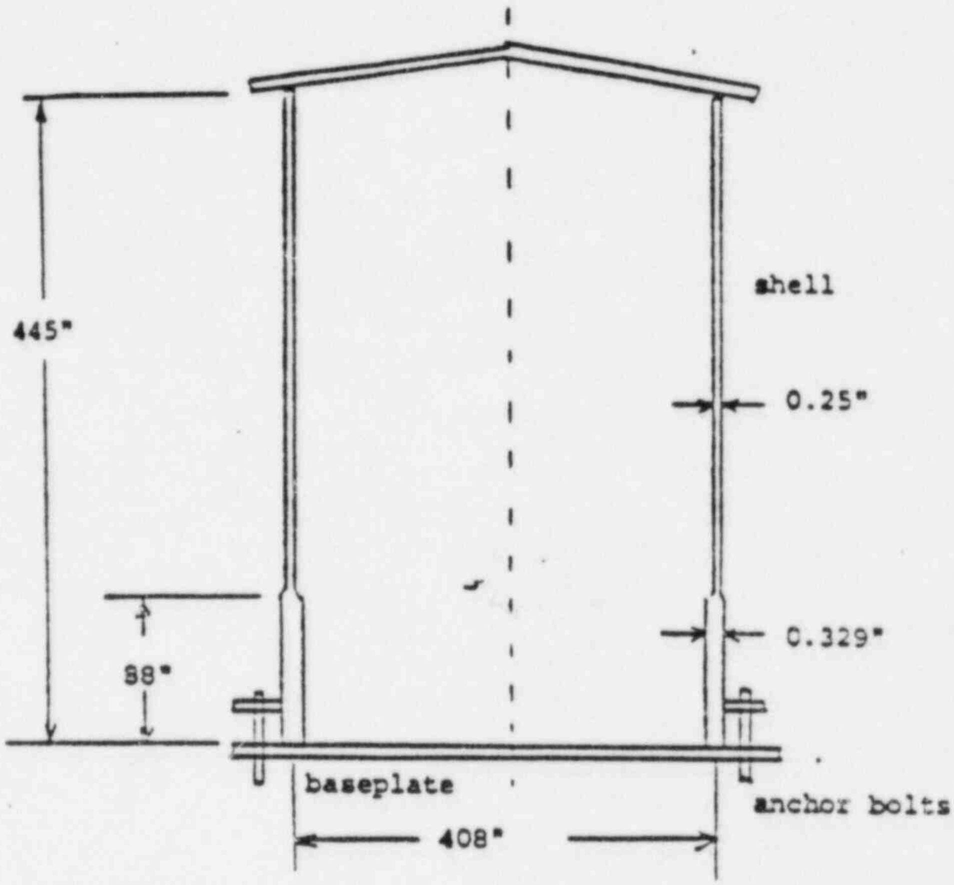
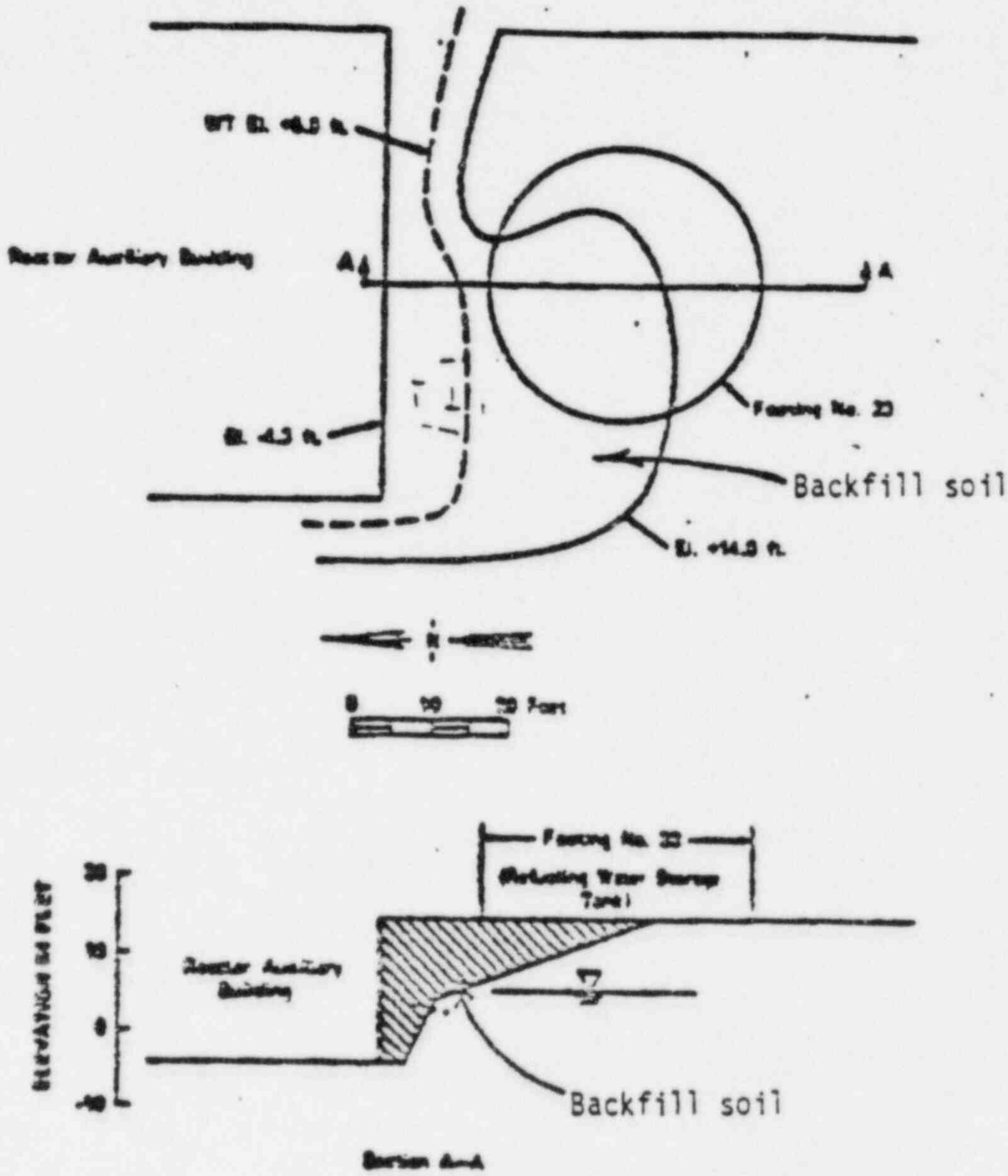


Figure 1. The refueling water storage tank.



Settlement = 1½"

LOCAL SOIL CONDITIONS UNDER
REFUELING WATER STORAGE TANK

Figure 2. Local soil conditions under the refueling water storage tank.

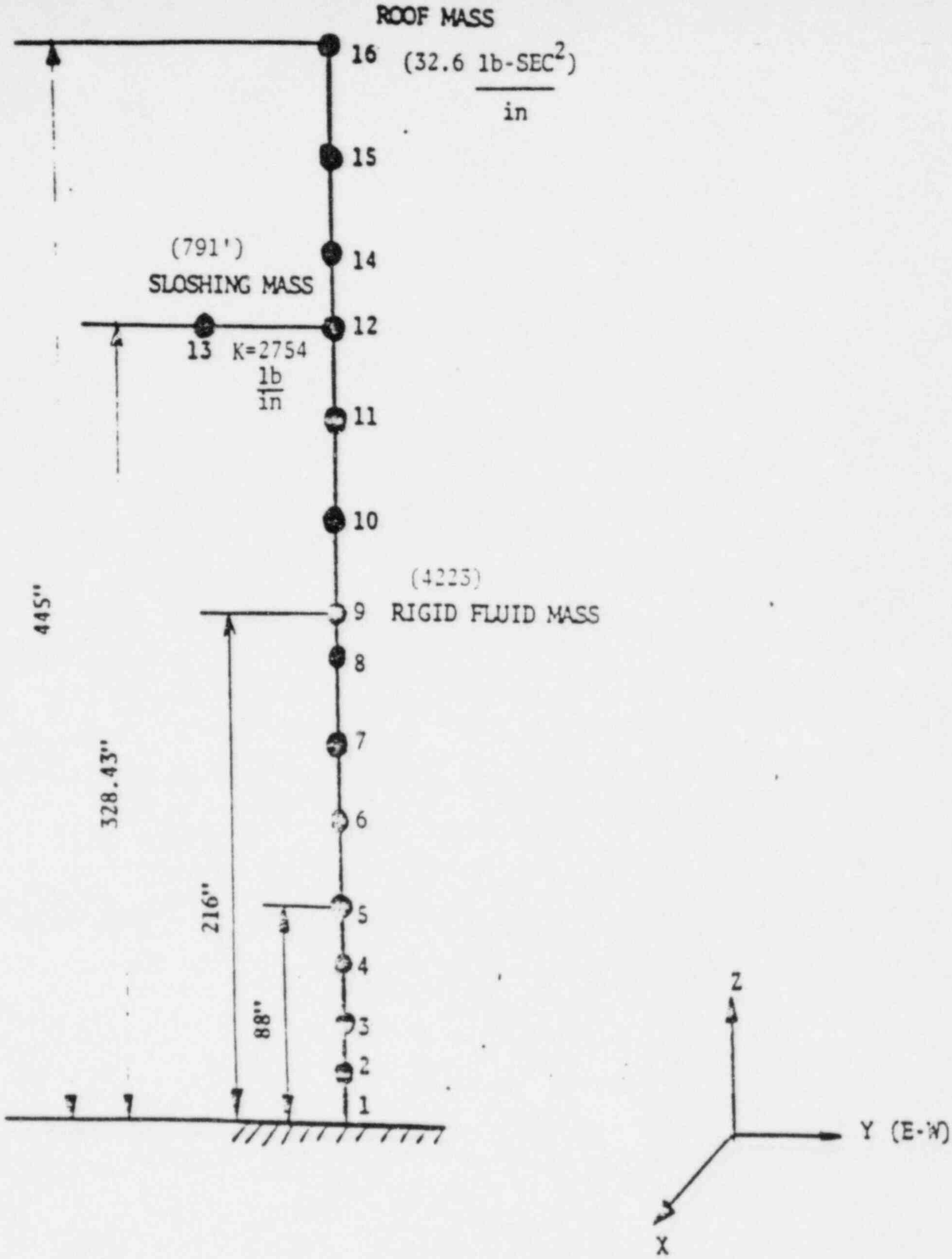


Figure 3. Mathematical model of the refueling water storage tank.

native soil. The difference of properties, e.g., shear modulus, at the strain level of 0.67g earthquake, is small, as indicated in Ref. 2.1. Therefore, a uniform elastic half-space medium was assumed for the analysis.

The soil properties used for the analysis were:

Shear modulus (kip /ft)	1203
Shear wave velocity (ft/sec)	578
Material damping (%)	8.0
Poisson's ratio	0.35
Weight density (kip/ft)	0.116

The average dynamic soil properties are compatible with shear strains at the site induced by the 0.67g design motion at the ground surface. These soil properties were used by the Licensee for their SSI Analysis.

A.3.2 Foundation Impedances

The diameter of the tank circular foundation is 35.5 feet [and 2 feet and 4-1/4 inches in thickness.] We calculated impedance functions for the refueling water storage tank by using two methods: the CLASSI approach (Ref. 2.4) and the constant impedance method (Ref. 2.2).

CLASSI is a computer code for simultaneously analyzing the soil-structure effects and computing structural responses. This code uses a three-step substructure approach — a determination of the foundation input motion, and then the foundation impedance, followed by an anlysis of the coupled soil-structure system. Since the control motion is specified directly at the foundation, foundation input motion does not need to be determined. The foundation impedances are calculated by using a continuum method. In general, for a linear elastic or viscoelastic material and a uniform or horizontally stratified soil deposit, each element of the impedance matrix is complex-valued and frequency dependent. The real part of the matrix represents the stiffness of the soil and the imaginary part represents the damping. The final step in the substructure approach is the actual SSI analysis. The foundation input motion and the foundation impedances are combined with a dynamic model of the structures for the soil-structure system. The response analysis is then performed in the frequency domain. Fourier transform techniques are applied to obtain the time history of the response.

Figures 5 through 7 show the impedance calculations by the CLASSI approach, while the impedances calculated by using the constant spring method are 1.0-E5 kip/ft for translational stiffness, 2.76E7 kip-ft/radian for rocking stiffness, 2.07E3 kip-sec/ft for translational damping, and 2.12E5 kip-ft-sec/rad for rocking damping, respectively.

A.4.0 <u>INPUT MOTION</u>

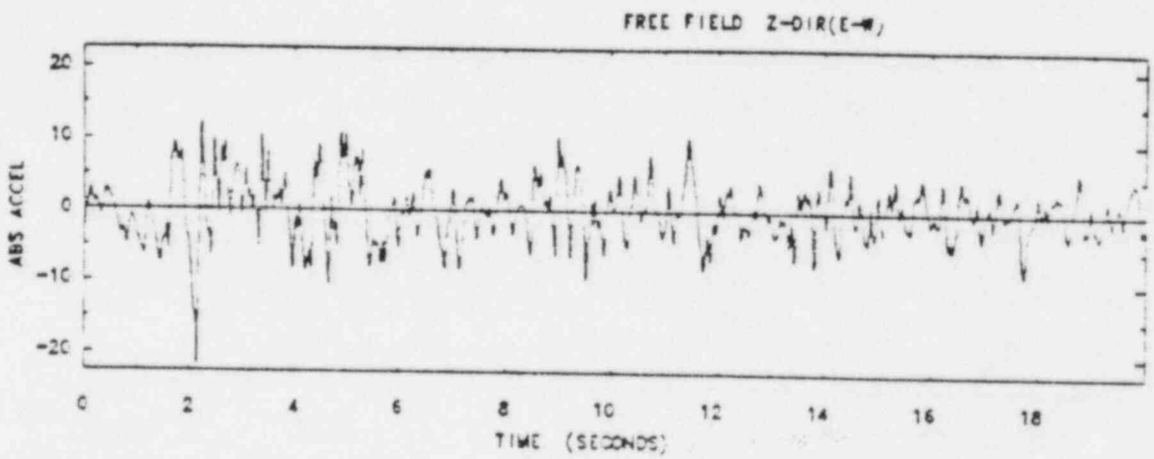


Figure 4a. The Input motion time history supplied by Impell.

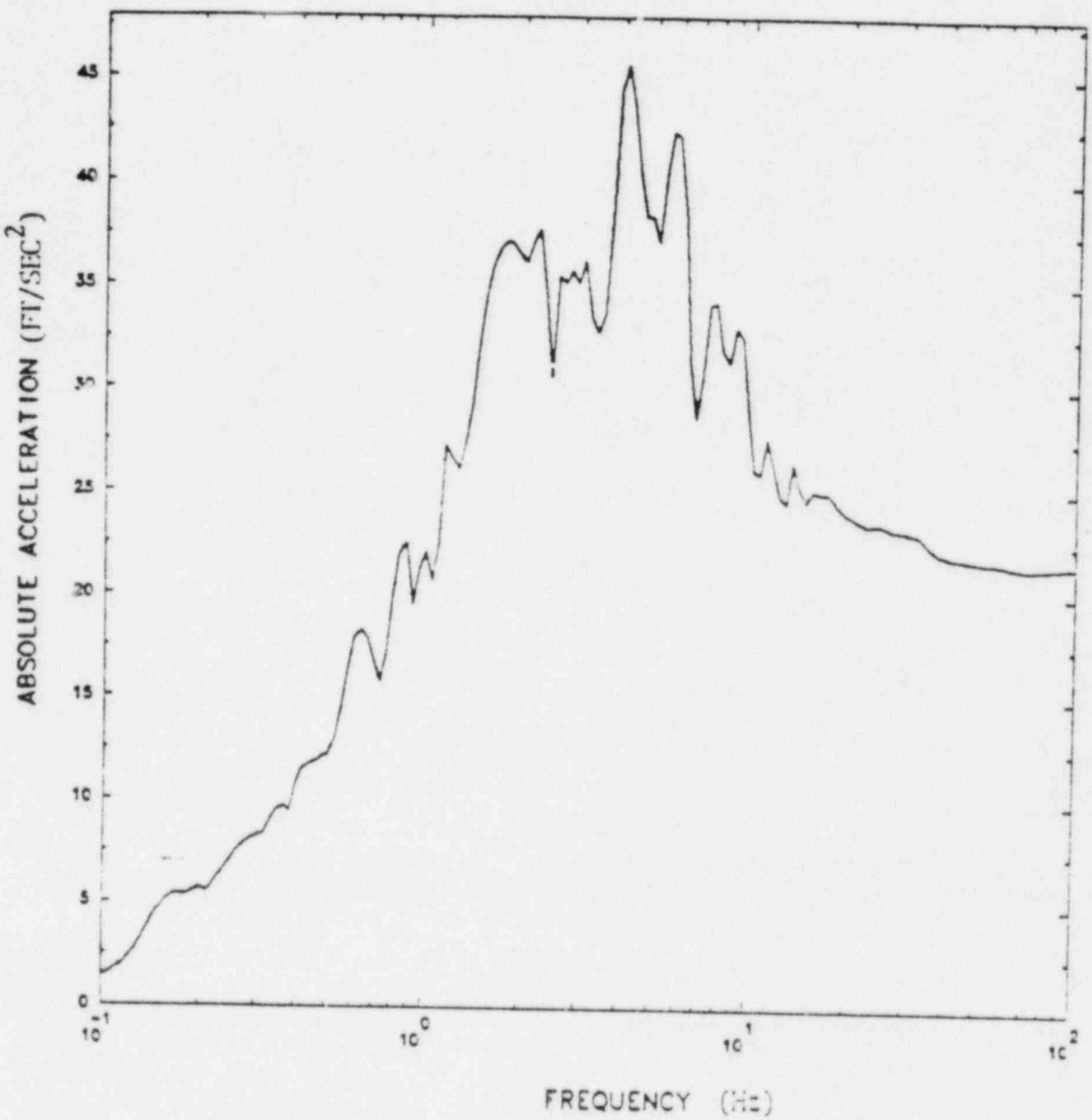
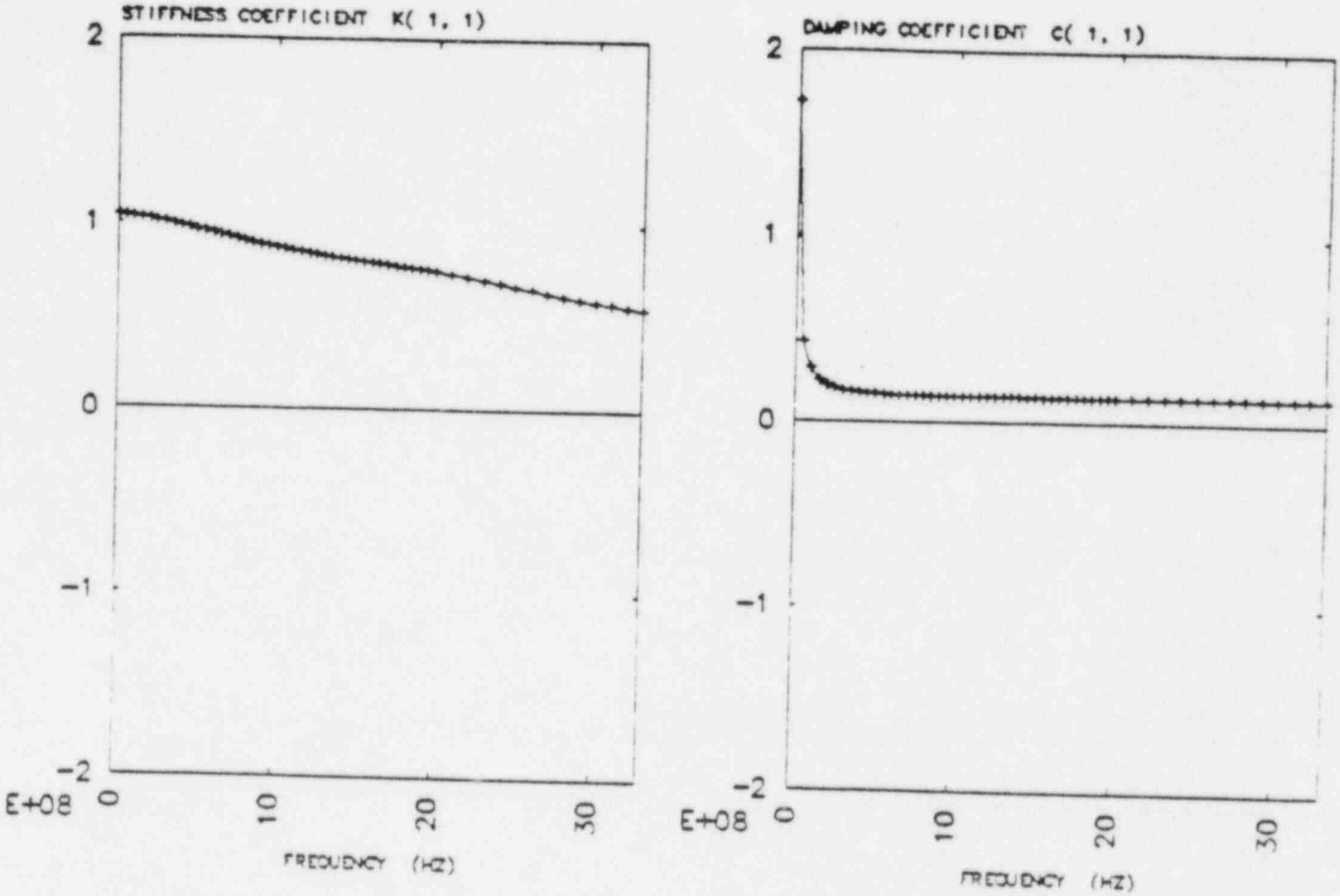


Figure 4b. The SONGS-1 LTS design response spectrum in the east-west direction, with 5% damping.



NOTE: K=LB/FT
C=(LB-SEC/FT)/frequency

Figure 5. The horizontal translational impedances K(1,1) and C(1,1).

The SONGS-1 artificial time history in the east-west direction, which was generated by the Impell Corporation of Walnut Creek, California, was used for this soil/structure/fluid interaction analysis. The response spectrum corresponding to the time history envelopes the horizontal 0.67g modified Housner response spectrum. This time history is shown in Fig. 4a. The corresponding design response spectrum with 5% damping is shown in Fig. 4b. The control motion was applied in the free-field at the ground surface level.

A.5.0 RESULTS

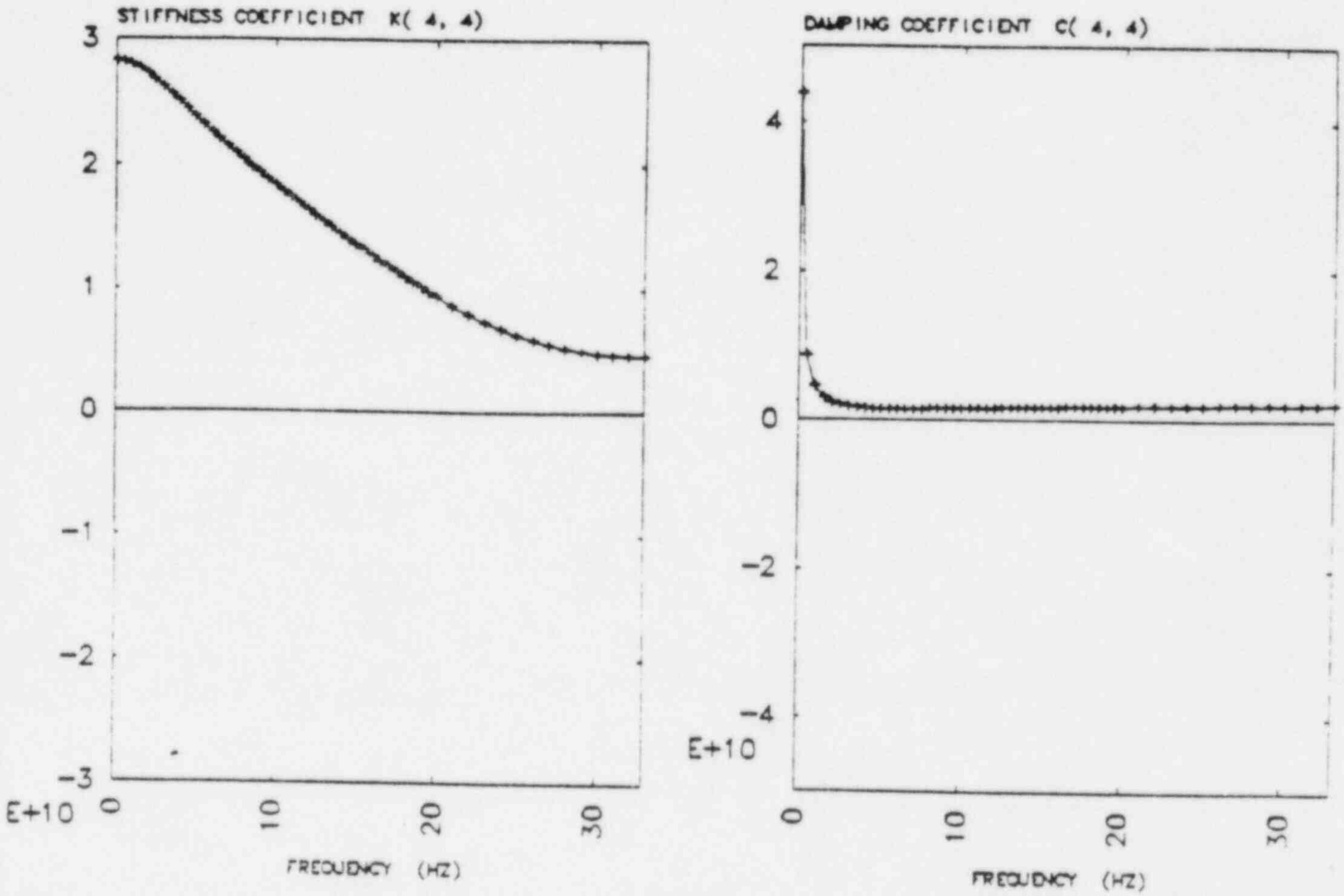
The dynamic modal properties of the fixed-base structure/fluid combination were calculated by using SAP4 code. The results are shown in Table 1. The second and third modes represent the structure's frequencies and the first mode represents the sloshing fluid's frequency. Note that all the results presented here are for the east-west direction only, since the geometry of the tank is axisymmetric and the analysis was performed parallel to the base.

Recall that we used two different methods to calculate the foundation impedances: frequency-dependent impedance (by the CLASSI approach) and frequency-independent impedance (by the constant soil-stiffness method). In the following paragraph, we present the results from the frequency-dependent impedance first, then the results from the frequency-independent impedance calculation. As a point of interest, the results of modal analysis are then presented.

Figure 8 shows that, based on the CLASSI approach the in-structure response spectrum at the foundation level is generally lower than that in the free-field. Figure 9 shows the peak accelerations along the structure elevation. It can be seen from Figure 9 that the accelerations decrease slightly at the lower part of the structure, then increase all the way up to the tank roof.

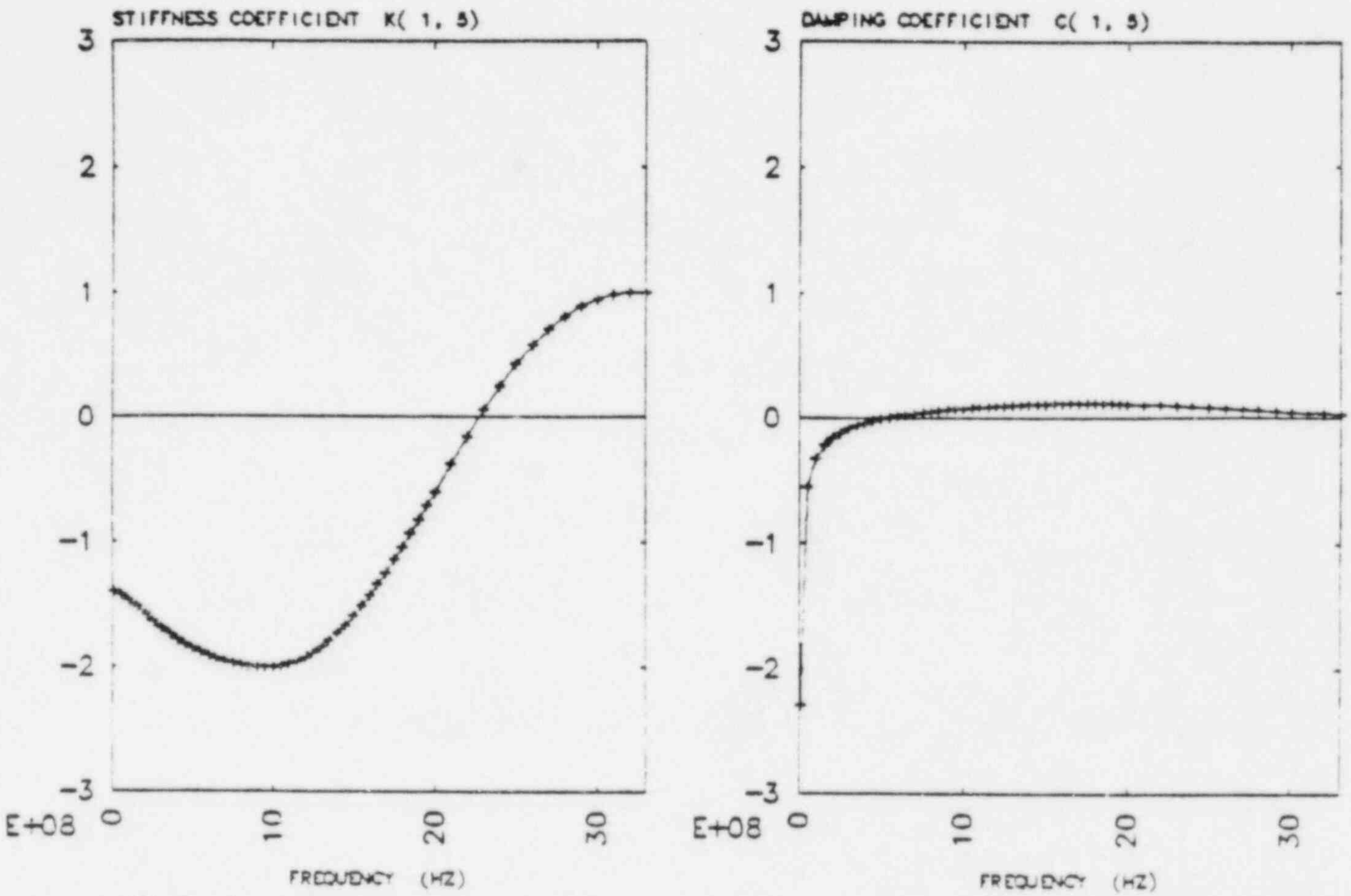
Figure 10 shows the peak accelerations along the structure elevation, based on the frequency-independent impedance calculation. Again, the accelerations decrease at the lower part of the structure, then increase all the way up to the tank roof.

In order to explain the structural response behavior of the soil/structure/fluid system, we computed the modal properties for the case of constant impedances. Table 2 shows the system frequencies and their participation factors. The frequency of the first mode remains 0.297 hertz. This reflects that the first mode's being the local mode representing the sloshing fluid. Table 3 consists of the composite modal damping, ranging from 0.5% to 76.4%. Table 4 presents the mode shapes for the corresponding first three modes. The first mode represents the local mode of the sloshing mass, while the second and third modes are the structural modes. Based on the participating factors and frequencies, it is obvious the second mode dominates the tank response. This verifies the nature of the profile of the maximum accelerator of the tank along the height as described above, both for our confirmatory and the Licensee's analyses, because the acceleration profiles are similar to the shape of the second mode shape.



NOTE: K=LB-FT/RAD
C=(LB-FT-SEC/RAD)/frequency

Figure 6. The rocking impedances K(4,4) and C(4,4).



NOTE: K=LB/RAD
C=(LB-SEC/RAD)/frequency

Figure 7. Translation/Rocking coupling impedances K(1,5) and C(1,5).

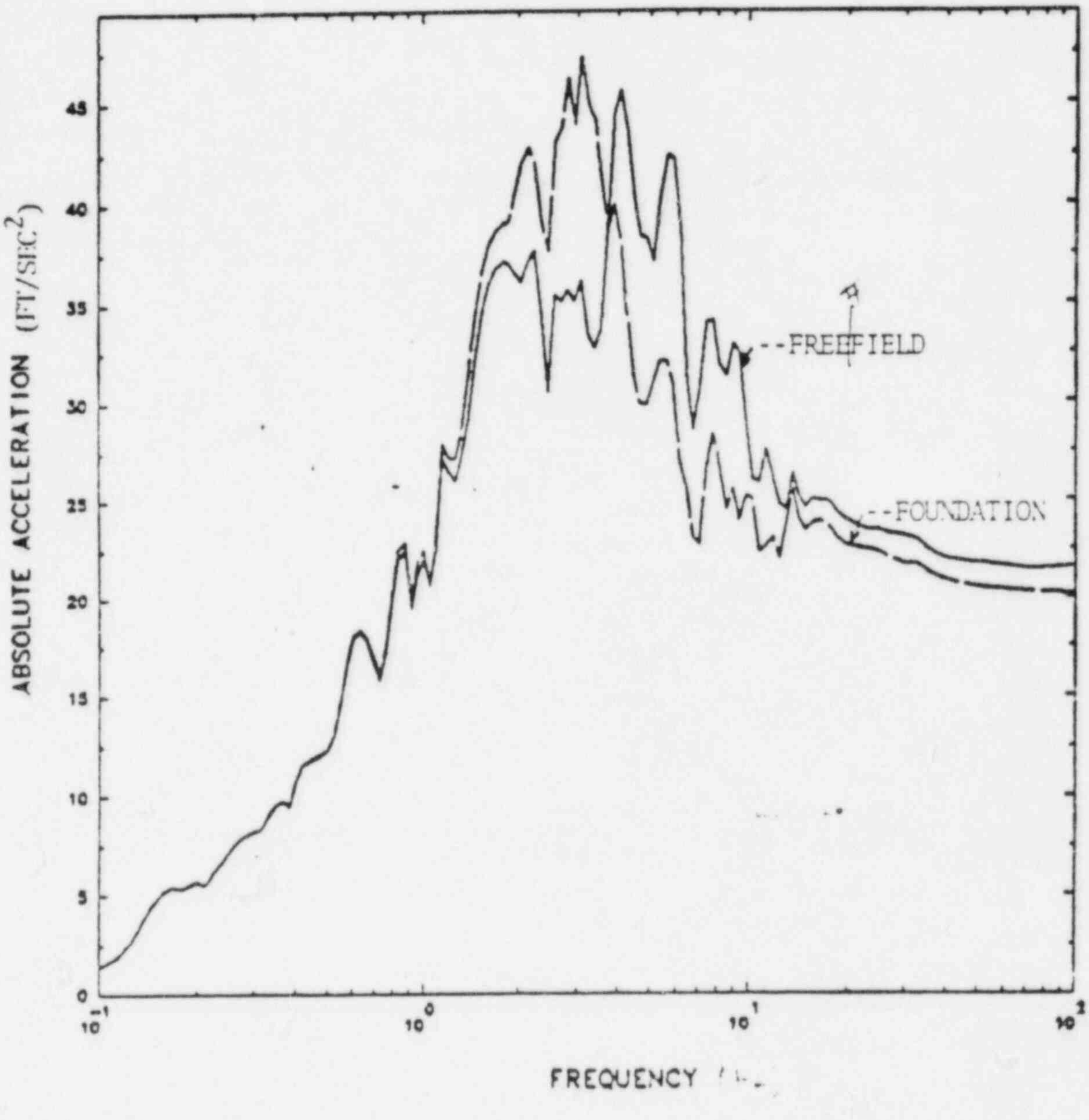


Figure 8. The foundation-level and free-field in-structure response spectra.

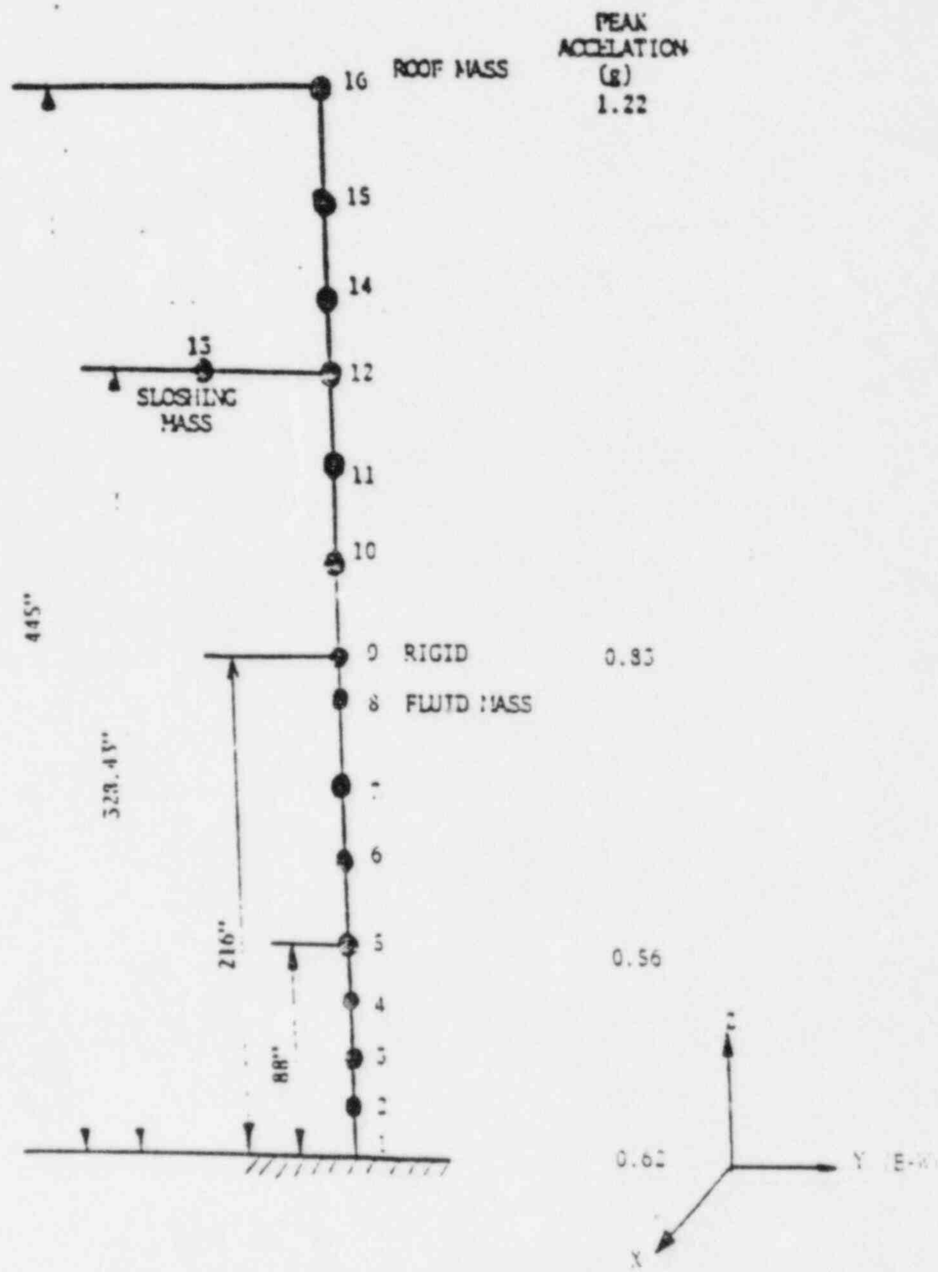


Figure 9. The Peak acceleration response based on frequency-dependent approach (CLASSI) impedance.

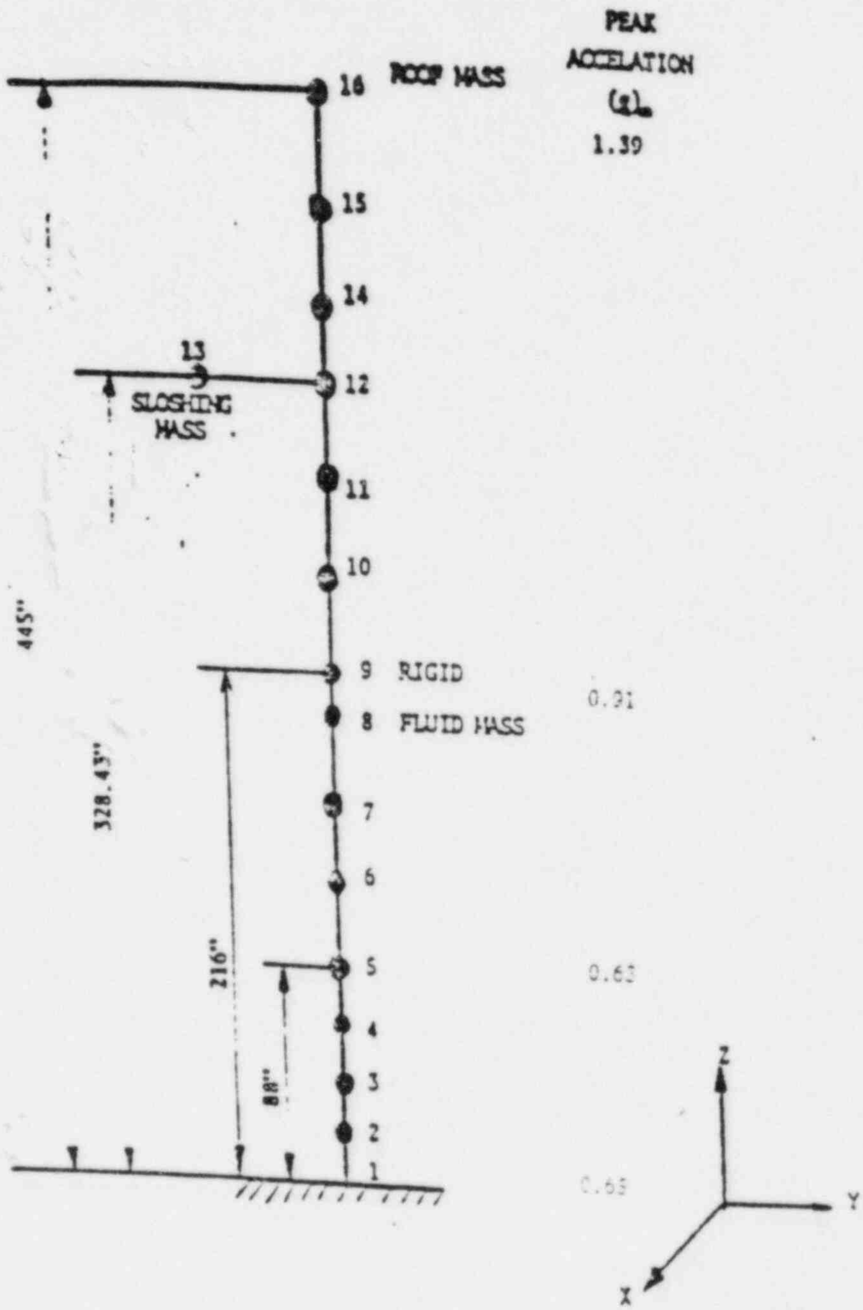


Figure 10. Peak acceleration response, based on the frequency-independent impedance approach (constant spring).

Comparing the responses these two different impedance calculations, indicates that the general response trend is consistent, even though the responses from the frequency-independent approach are higher than those derived by the frequency-dependent method.

A.6.0 COMPARISON OF LLNL AND IMPELL RESPONSE RESULTS

Impell's response results (Ref. 2.2) are shown in Fig. 11. By comparing LLNL's CLASSI and Impell's response results, we see the second decimal place. Accordingly, we conclude that the results of the soil-structure-fluid analysis performed by Impell are acceptable.

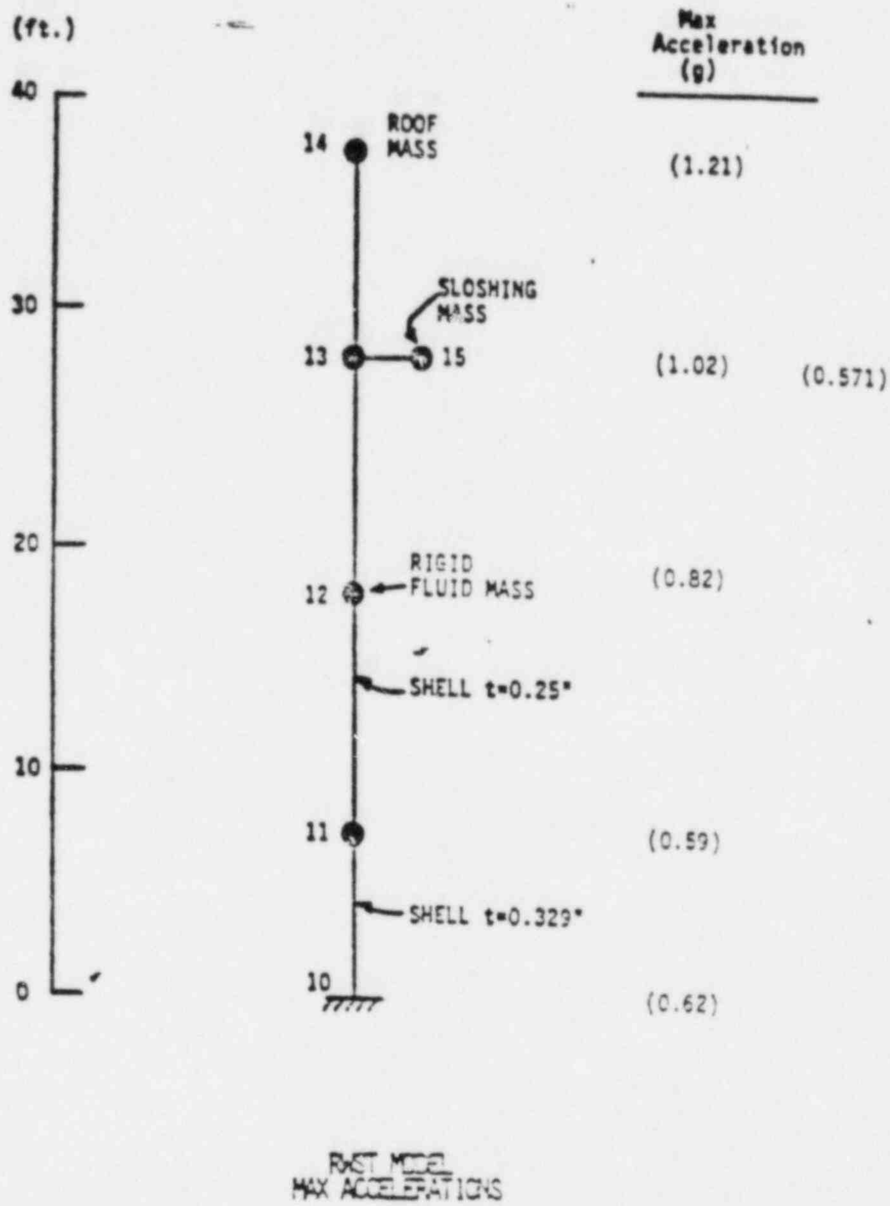


Figure 11. Impell's peak acceleration response, based on the frequency-dependent impedance approach.

Modes	Frequency (Hz)
1	0.297
2	6.9
3	50.5

Table 1. Significant modes of the fixed base-structural-fluid model.

Modes	Frequencies (Hz)	Participation Factor
1	0.297	3.1
2	6.9	7.6
3	50.5	2.4

Table 2. Significant modes of the soil-structure-fluid interaction system (constant impedance).

MODES	DAMPING
1	0.005
2	0.119
3	0.764

Table 3. Composite modal damping of the interaction System (constant impedance).

Mode Shape

Node	Mode 1	Mode 2	Mode 3	
1	1.0431e-04	4.2984e-02	2.7673e-01}	BASEMENT
2	3.0350e-05	1.0248e-02	1.6230e-03	
3	6.2372e-05	2.0969e-02	2.6895e-03	
4	9.5918e-05	3.2106e-02	3.1783e-03	
5	1.3097e-04	4.3666e-02	3.0702e-03	
6	2.1282e-04	7.0572e-02	2.0852e-03	
7	2.9849e-04	9.8331e-02	-1.4014e-03	
8	3.8771e-04	1.2692e-01	-7.4801e-03	
9	4.1593e-04	1.3589e-01	-9.8835e-03}	RIGID MASS
10	4.7620e-04	1.4628e-01	-5.5379e-02	
11	5.3521e-04	1.5638e-01	-1.0031e-01	
12	5.9425e-04	1.6642e-01	-1.4520e-01	
13	3.2458e-01	-9.9611e-04	4.0045e-05}	SLOSHING MASS
14	6.3652e-04	1.7679e-01	-1.9088e-01	
15	6.7714e-04	1.8674e-01	-2.3377e-01	
16	7.1575e-04	1.96228-01	-2.7267e-01}	ROOF MASS

Table 4. Mode shapes of the interaction system (constant impedance).