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River Protection Project - Seismic Analysis of Vitrification Buildings

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ABSTRACT

The construction of the vitrification facility of the River Protection Project (RPP), Waste Treatment Plant (WTP), in the Hanford Reservation in the State of Washington is progressing at this time. The facility is planned to vitrify nearly 205 million liters of radioactive and chemical wastes currently stored in 177 underground tanks at the Hanford site some of which date back to World War II. The vitrification process mixes waste into sturdy glass. The glass will be stored partly at the site and partly off-site.

The facility consists of several large complex structures that are categorized as Seismic Category I, following the US Department of Energy (DOE-1020, [1]) guidelines. The High Level Waste (HLW) and Pre-Treatment (PT) buildings are the two main structures with a composite steel framing and concrete shear wall structural system inter-connected at multi-story levels. The process areas in these structures consist of multiple cells and caves connected by transfer tunnels and shielded doors designed to meet confinement and shielding requirements. The shielding requirements stipulate concrete walls up to 4-ft thick and concrete slabs that range in thickness from 1 ft to 3 ft. The footprint of each of these buildings is about 300 ft by 500 ft, rising above grade level from 70 ft to120 ft. The embedment for these buildings varies from 20 ft to 50 ft, with the depth of embedment varying with plan location. The large size of the structures and composite nature of the design result in a complex dynamic behavior of low frequency steel components interacting with the higher frequency concrete members causing local amplifications of motion. To capture the dynamic behavior of the structure, a detailed finite element model is constructed and the modal properties are extracted.

At the location of the facility, the site is a very deep soil site consisting primarily of dense to very dense sand and gravel layers. A comprehensive geotechnical investigation program with emphasis on multiple geophysical testing techniques was executed to develop the best estimate as well as the range of dynamic soil properties needed for seismic soil-structure interaction (SSI) analysis.

The composite design and extensive size of the structures stipulate a detailed model carefully discretized for seismic SSI analysis. Such analyses are required for the purpose of predicting the seismic loads for structural design and generating representative in-structure response spectra for design and qualification of an extensive array of equipment, piping, electrical and control systems.

In this paper, a brief summary of the site conditions and design motions is first presented. Subsequently, the main points concerning development of the SSI models and some of the SSI results are presented and discussed.

KEY WORDS: Soil-Structure Interaction, SSI, RPP, WTP, Vitrification, Seismic Design, SASSI2000

INTRODUCTION

The River Protection Project (RPP) consists of the following major parts: 1) waste-site remedial action and disposal facility, 2) decontamination and decommissioning, 3) ground water and vadose zone management, 4) facility surveillance and maintenance. Waste Treatment Plant (WTP) Facility (formerly referred to as "TWRS-P facility") is part of the RPP and involves design and construction of waste treatment facilities using vitrification to mix waste into a sturdy glass. Nearly 205 million liters of radioactive and chemical wastes are currently stored in 177 underground tanks at Hanford near the Columbia River, some of which date back to World War II. The RPP-WTP is the largest vitrification facility in USA. The general arrangement plan is shown in Figure 1. Some of the major structures in the plant include

- Pretreatment (PT) Building
- High Level Waste (HLW) Vitrification Building
- Low Activity Waste (LAW) Vitrification Building
- Administration Building
- Melter Assembly Building
- Laboratory Building.
- Other supporting small buildings.

The most critical and largest structures categorized as Seismic Category I following the DOE-1020 guidelines are the PT and HLW buildings. Other buildings are either simple small structures or are categorized as Seismic Category III or IV. Seismic Category I structures correspond to Performance Category 3 (PC3) following the DOE-1020 definitions. The buildings accommodate an extensive array of systems and components, piping and all piping related equipments, tanks and pumps. Over one million feet of piping will be used in the facility. Therefore, it is necessary to develop a reasonable estimate of in-structure response spectra for design and qualifications of the equipments.

DESIGN APPROACH

For seismic design and soil-structure interaction analysis of the facility, a document titled Seismic Analysis and Design Approach (SADA) was prepared. This document was reviewed by the oversight committee and other regulatory groups and was used as a basis for seismic analysis and design. The design basis earthquake (DBE) for Seismic Category I structures is the 2000-year earthquake following the DOE-1020 guidelines. The DBE is based on the earlier report by Geomatrix Consultants [2] for the Hanford reservation. The horizontal and vertical acceleration response spectra of the DBE defined at the ground surface level are shown in Figure 2. Using the DBE spectra, a set of 3-component acceleration time histories were developed to match the spectra in accordance with the NRC and DOE spectral-matching requirements, including spectral fitting criteria, independence among the three components, and adequacy of the power and duration of the time histories. The acceleration time histories were subsequently used for both the free-field and SSI analysis.

To characterize the subsurface conditions, a comprehensive geotechnical field and laboratory investigation was performed by Shannon & Wilson [3]. Since it was known from other projects at the Hanford site that the foundation soils are very competent for bearing purposes, the emphasis was placed on geophysical measurement to develop the dynamic soil properties for SSI analysis. In addition to the extensive boring and sampling using the Standard Penetration Test (SPT), the site was probed by the Seismic Cone Penetrometer (SCPT) including the seismic velocity measurement. The geophysical properties were also measured by the seismic refraction and down-hole techniques. The results were synthesized and a range of soil shear wave velocity profiles including the upper bound, best estimate and the lower bound profiles along with other geotechnical parameters were established. Figure 3 shows the range of the initial measured shear wave velocities as well as the strain-compatible shear wave velocity obtained for the free-field de-convolution analysis using the computer program SHAKE2000 [4]. The granular nature of the materials at the site and the high shear wave velocity confirms other field-measured data and indicates a dense to very dense granular site condition. The estimate for the settlement of the buildings indicated a small and short-term settlement (< 0.5 inches).

To characterize the complex structural behavior of the buildings, the fixed base model of the buildings were constructed using the Computer Program GTSTRUDL [5]. As indicated above, the structures are composed of concrete shear walls and slabs along with steel framing and roof. To illustrate the composite design of the structure, Figure 4 shows the concrete and steel models encapsulating the concrete part for the PT building. For such composite structures with large dimensions, development of the traditional simple beam stick model is nearly impossible since the simple model cannot adequately capture the dynamic behavior of the structure. The finite element model constructed consists of 3,994 concrete plate elements and 4,464 steel beam elements. The fixed-base analysis of the buildings shows the horizontal frequencies in the range of 8 Hz to 10 Hz for the concrete core and a range of 4 Hz to 6 Hz for the steel members. The difference in the vertical frequency is much more pronounced. The distinct vibration frequencies between the concrete and steel members and the interaction among these members complicate the motions of the structures causing local amplifications at numerous locations on each floor. For equipment design, a slab-to-slab evaluation of the vertical frequencies of the slabs and the interaction with the equipment supported by the slab was performed separately and appropriate modeling of the secondary system was used in the total structural model following ASCE 4-98 [6] guidelines. A detailed discussion of the method employed and the advantage of such methods in the global SSI model of the structures in presented by Ostadan et al. [7].

For SSI analysis, the Computer program SASSI2000 [8] was used. The new version of this program makes it possible to handle a large finite element model for SSI analysis using a high-end Windows-based workstation. In the case of the PT building, the structure is embedded for a small portion at the center to a depth of 51 ft below grade and rises to Elevation 119 ft above the ground surface. Over 74% of the weight of building is due to the weight of concrete members, with the remaining weight due to steel members, equipment, and other systems and components. Nearly 50% of the weight is at and below grade elevation. Given the total weight of 565,600 kips and the footprint of 558 ft. by 250 ft., the foundation pressure is about 4 ksf suggesting a much lighter pressure as compared to the foundation pressure for a major building in a typical nuclear power plant. In spite of the light foundation pressure, the SSI effects were found to be significant and very beneficial in reducing the seismic loads. It should be noted that even with the large size of the SSI model, a typical run time using the subtraction method in SASSI2000 on a high-end Windows-based workstation is about 120-160 hours for each soil case.

SSI RESULTS

To illustrate the SSI effects, the acceleration response spectra at a key location in the PT building for the fixed base, upper bound, mean and lower bound soil cases are shown in Figures 5. The results clearly show the SSI effects in shifting the response frequency and reducing the amplitude of the response. The acceleration response spectra were subsequently enveloped for the three soil cases after incorporating the coupling responses for each earthquake and soil case. To illustrate the complexity of the response of the composite concrete and steel structures, the horizontal response motions at two locations at Elevation 77 ft of the PT building are compared in Figure 6a. The two points correspond to a wall location (center point) and a steel column location (East edge point). As depicted in this figure, even though the floor slab integrates the concrete and steel structure, there is still a pronounced difference in the floor motion at the same elevation due to the variation of local structural properties and floor discontinuities. The difference in the response is even more pronounced in the vertical direction. Figure 6b shows the vertical motions at the same two locations. This observation supports the modeling techniques employed in that a detailed finite element model is more appropriate for such complex and composite structures rather than a simple beam stick model.

To develop the seismic loads for structural stress analysis and design, the peak acceleration responses of the model at the nodes corresponding to the walls and slabs were obtained and enveloped for all three soil cases. The maximum acceleration values were increased to include the calculated accidental torsional effects for each floor and subsequently applied at nodal points as static loads which, after multiplying by the nodal mass, translate to nodal forces resulting in story shear and shear force diagram. The SSI results in terms of shear and moment diagrams are shown in Figures 7a and 7b. It should be noted that the base shear coefficient for part of the building above the basemat at the ground surface level (considering the corresponding weigh of the building) is 0.50. The peak spectral acceleration of the design input motion at 5% damping is 0.58g (see Figure 2). Considering the SSI associated damping (estimated to be about 15%), the spectral peak reduces to 0.4g. Thus, the base shear coefficient of 0.50 is about 15% less than the DBE spectral peak at 5% damping and is about 20% larger than the DBE spectral peak at the associated SSI damping of 15%. This observation shows that the conventional practice of estimating the preliminary base shear force by using 1.5 times the spectral peak of the design motion multiplied by the weight of the building is too conservative for structures that benefit from the SSI effects significantly. It is noted, however, that the response motions of the steel members well exceed the spectral peak of the design motion. On the other hand, the steel members contribute a small fraction to the total shear due to their light weights.

Another noteworthy SSI response for the building is the seismic soil pressure response. For most non-yielding walls, use of ASCE 4-98 dynamic soil pressure recommendation has been considered to be conservative. In the case of the PT building, the pit at the center of the building is subjected to the overburden pressure induced by the foundation mat at the ground surface surrounding the pit. This condition is obviously not consistent with conditions assumed for the ASCE 4-98 dynamic soil pressure solution. For this reason it was not clear if the ASCE4-98 solution for design of the walls of pit would be adequate. Using SASSI, the seismic soil pressure was computed directly from the SSI analysis. The results of the analysis are compared with the ASCE4-98 solution in Figure 8. The recently published solution for seismic soil pressure by Ostadan et al. [9] is also shown in this figure. As depicted in Figure 8, the directly computed results are much less than the ASCE 4-98 and the solution by Ostadan et al. with difference reducing with depth. The reason for such a large difference is due to the fact that the soil mass behind the walls of the pit are well within the pressure bulb of the surface base mat at the ground surface level and tend to move with the building mat. This confinement causes less differential motion between the walls of the pit and the surrounding soil mass thus reducing the seismic soil pressure. It is noted, however, that for this condition there is an additional contribution to the lateral soil pressure caused by the vertical vibration of the building. The additional contribution was found to be relatively insignificant and amounts to only slight increase in the lateral dynamic pressure obtained directly from the horizontal analysis. In the case of the HLW building with embedment to a depth of 27 ft, the exterior walls of the building at the perimeter were in contact with the surrounding soil, a condition consistent with the ASCE 4-98 soil pressure solution. In that case, the seismic soil pressure results were much closer to the ASCE4-98 solution.

SUMMARY

The paper presents the seismic design approach, the SSI modeling and the SSI results for large composite structures. The modeling techniques and the steps taken to develop the seismic loads for stress analysis and equipment design are described. It is shown that for large structures with concrete and steel members, use of a carefully discretized finite element model for SSI analysis is practical and yields more accurate and reasonable responses, particularly for equipment design and qualification. It is also illustrated that with new high end personal computers, such large models can be easily analyzed using the most up to date version of the computer code SASSI2000. The seismic soil pressure results for an embedded part of the building in the center footprint of the building illustrates that ASCE4-98 dynamic soil pressure is too conservative under such condition.

REFERENCES

1. DOE-STD-1020-94, DOE Standard Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities, U.S. Department of Energy, Washington D.C.
2. Geomatrix Consultants (1996). *Probabilistic Seismic Hazard Analysis, DOE Hanford Site, Washington*, WHC-SD-W236-TI-002, Revision 1a, prepared for Westinghouse Hanford Company, Richland, Washington, October 1996
3. Shannon and Wilson Inc (2000). *RPP-WTP geotechnical Investigation Final Report*, WTPC99-1036-42-17, Report to Bechtel Systems and Infrastructure Company, Richland, Washington, May 2000
4. Bechtel National Inc. (2000). *SHAKE2000 – A Computer Program for Conducting Equivalent Linear Seismic Response Analysis of Horizontally-Layered Soil Deposits*. Revision 1.0. San Francisco, California
5. Georgia Tech Research Corporation (1998) *GTSTRUDL: Integrated CAE System for Structural Engineering Analysis and Design*, Release 25, Atlanta, Georgia
6. ASCE (1998) *Seismic Analysis of Safety-Related Nuclear Structures and Commentary*, ASCE Standard 4, October 1998
7. Farhang Ostadan, Quazi Hossain, and Thomas Ma, Generation of In-Structure Response Spectra Considering Secondary System Mass Interaction, ASME B & PV Conference, Columbus Ohio, August 2003.
8. SASSI2000, Lysmer, J., Ostadan, F, Chin, C, *SASSI – A System for Analysis of Soil-Structure Interaction.*, Geotechnical Engineering, Department of Civil Engineering, University of California, Berkeley. Berkeley, California. December 1999.
9. F. Ostadan, W. H. White, Lateral Seismic Soil Pressure-An Updated Approach, US-Japan Workshop on Soil-Structure Interaction, USGS, Menlo Park, CA, September 22-23, 1998

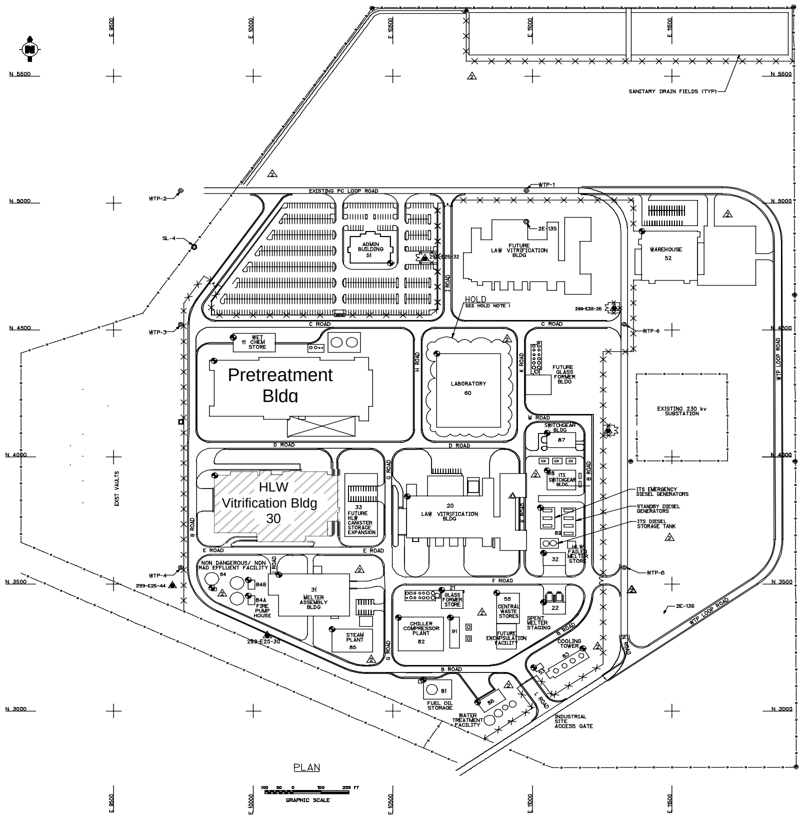


Figure 1 General Plan Arrangement of the RPP-WTP Facility

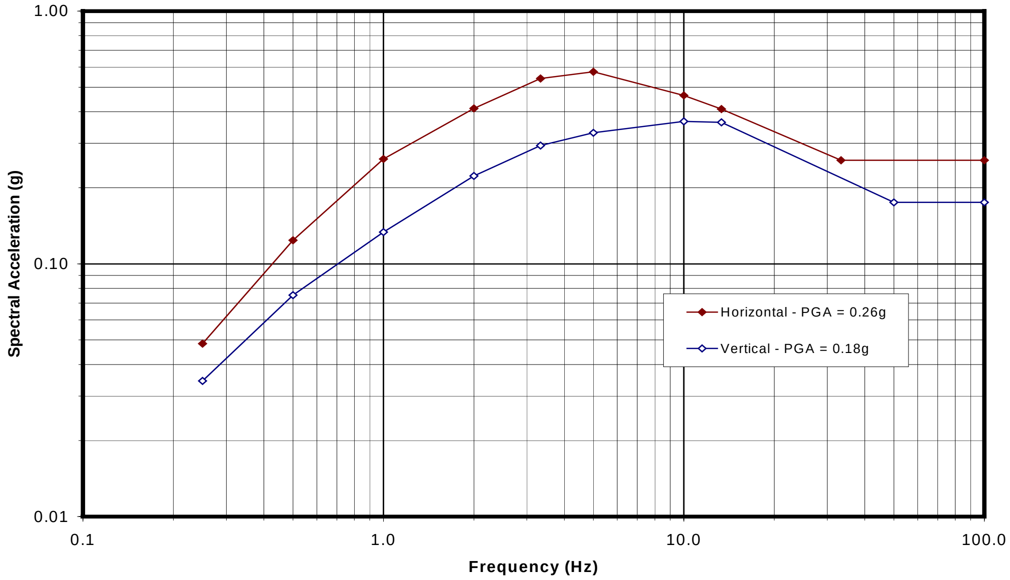


Figure 2 DBE Horizontal and Vertical Response Spectra (5% Damping)

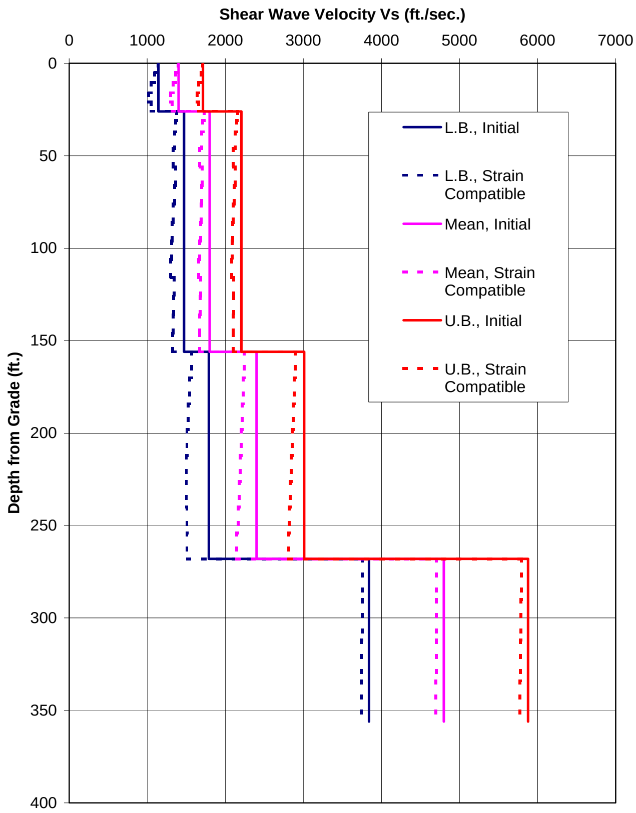


Figure 3 Initial and Strain-Compatible Shear Wave Velocity Profile at the WTP Site

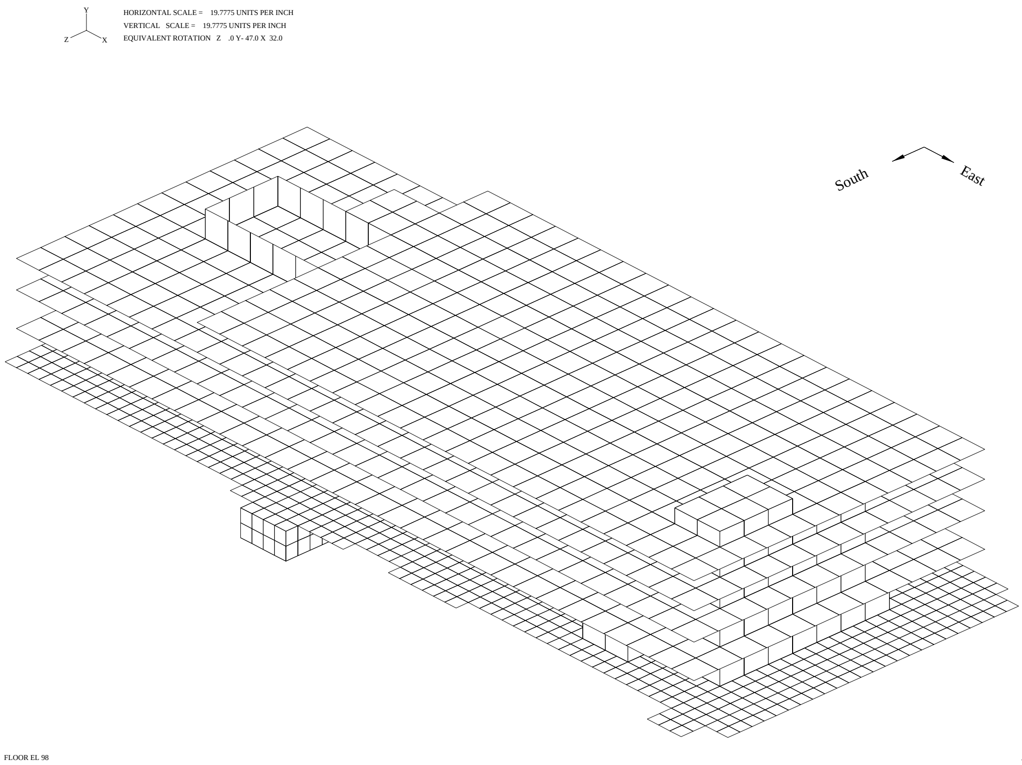


Figure 4a Finite Element Model for PT Building – Concrete Elements Only

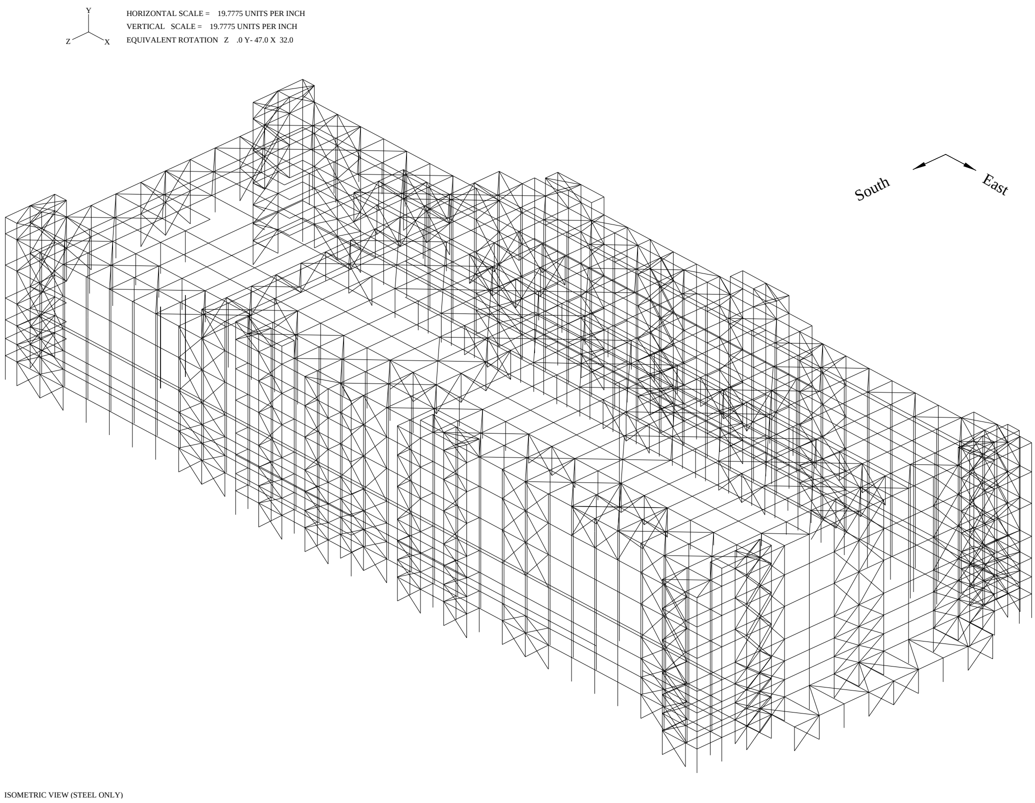


Figure 4b Finite Element Model for PT Building – Steel Elements Only

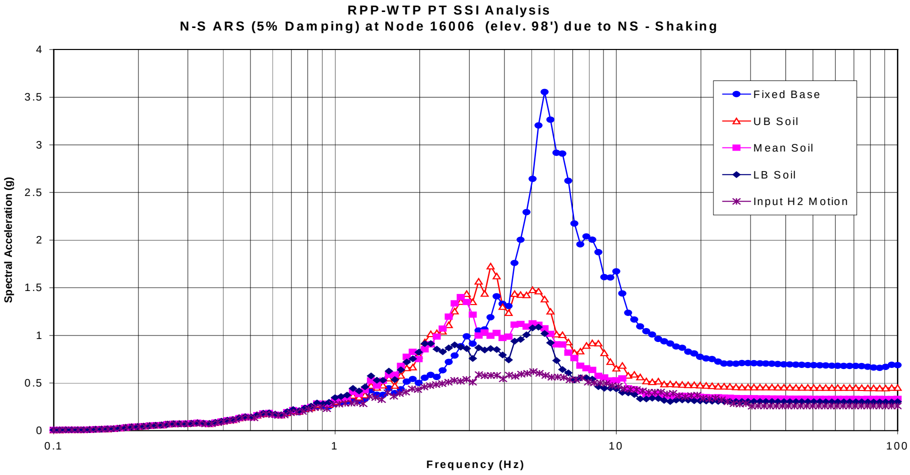


Figure 5 – SSI Response Motions for PT Building

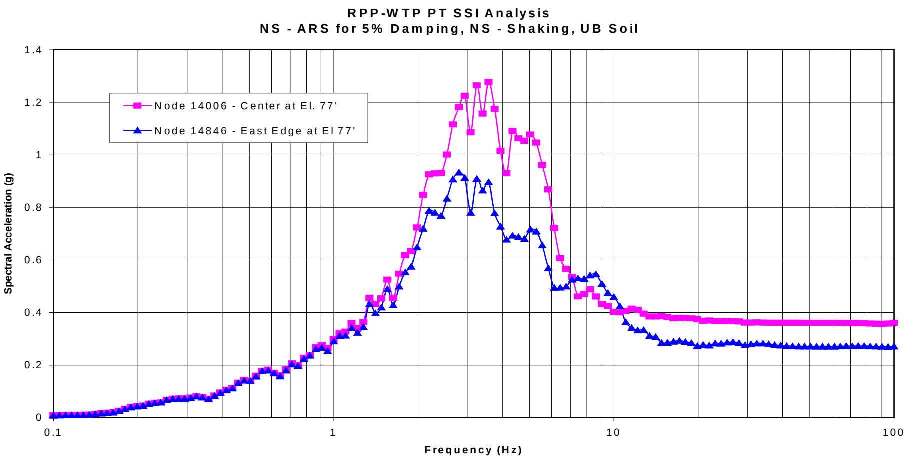


Figure 6a Horizontal Response Motions at Two Locations at El. 77 ft. of PT Building

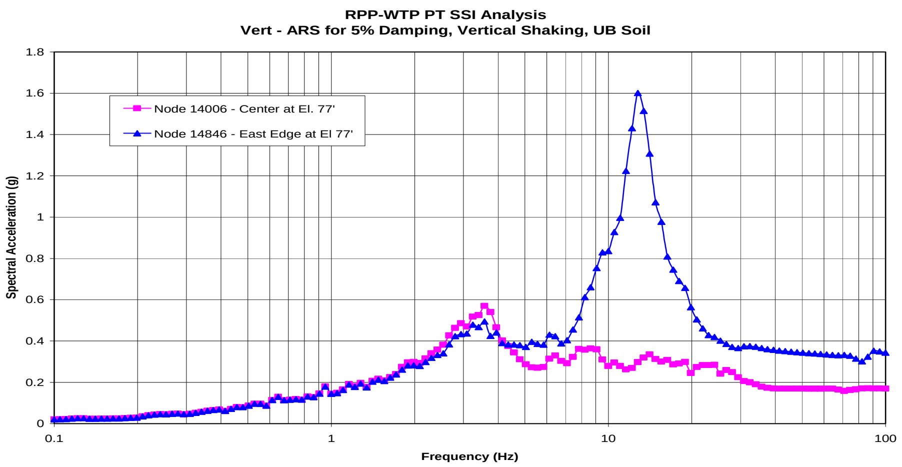
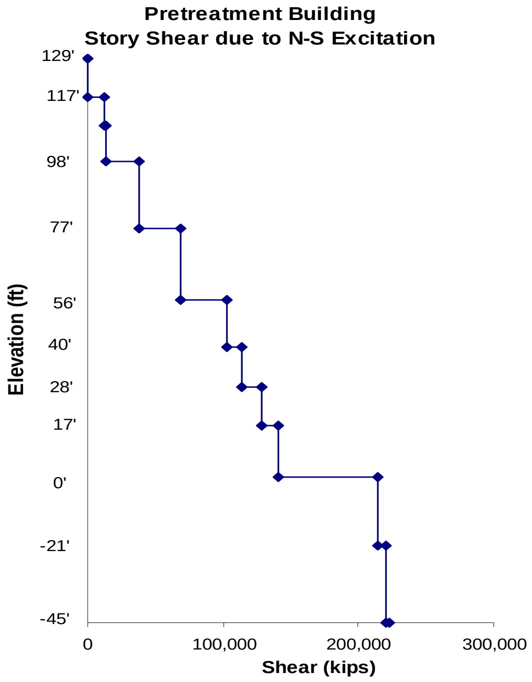


Figure 6b Vertical Response Motions at Two Locations at El. 77 ft. of PT Building



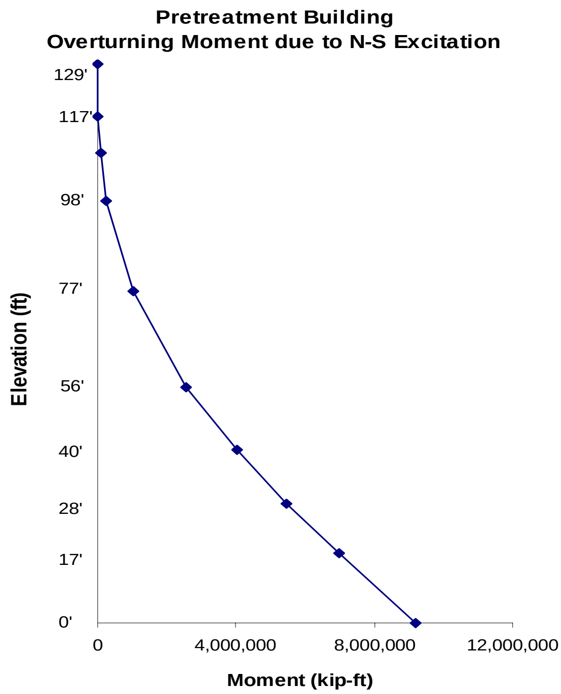


Figure 7a PT Building: Story Shear due to N-S Shaking

Figure 7b PT Building: Overturning Moment due to N-S Shaking

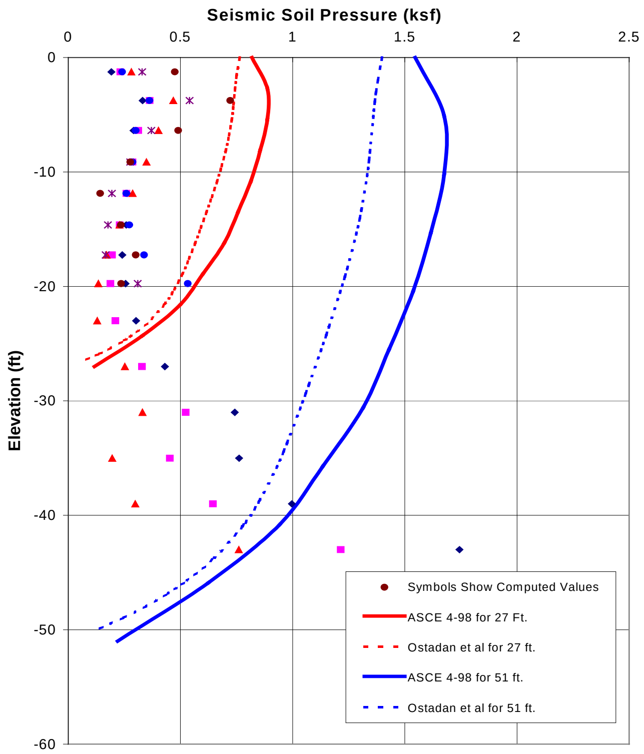


Figure 8 Computed Seismic Soil Pressure versus ASCE 4-98 and Ostadan et al. [9]