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PROGRESS REPORT

Independent Review of Implementation of Revised Seismic Design Criteria Hanford Waste Treatment and Immobilization Plant (WTP)

Revision D

Prepared for:

U.S. Department of Energy, Office of River Protection

Office of Environmental Management and Office of Engineering and Construction

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Independent Review of Implementation of Revised Seismic Design Criteria
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INTRODUCTION

In December 2000, the U.S. Department of Energy (DOE), Office of River Protection (ORP), awarded Bechtel National Incorporated (BNI) a cost-plus incentive fee contract for the Hanford Waste Treatment and Immobilization Plant (WTP) project. DOE-ORP requested the U.S. Army Corps of Engineers (USACE), Walla Walla District, conduct independent reviews of BNI's implementation of the revised seismic design criteria for the Hanford WTP. The seismic design criteria were revised in response to the development of revised ground motion (RGM) spectra by Pacific Northwest National Laboratory in 2005. The objectives of the USACE Independent Review (IR) Team are to ensure BNI's revised seismic design:

- Complies with code requirements; ensuring safety of the WTP structures, systems, and components (SSCs).
- Maintains efficient design practices to minimize the impact to the cost and schedule of the WTP project without adding undue layers of conservatism.

The WTP project is the largest, most complex radiochemical plant design and construction project undertaken in the United States. Its mission is to treat and immobilize waste from 173 underground tanks containing approximately 53 million gallons of mixed (hazardous and radioactive) liquid waste, generated during production of nuclear materials between 1944 and 1989.

METHODOLOGY

The IR Team concentrated on the seismic analysis and design of SSCs of the two primary WTP facilities, which treat the highest levels of radioactive materials,i.e., Pretreatment (PT) and High-Level Waste (HLW) facilities. These buildings are classified as Performance Category 3 for natural phenomena hazard design by DOE orders and standards and as Seismic Categories (SC) I and II by project-specific criteria.

The IR Team reviewed BNI’s design process, soil-structure interaction (SSI) analysis, structural seismic demand, structural design, and system and component design as well as conservatisms in the project design criteria and in the design performed. The SSI analysis and structural seismic demand analysis were complete. Design calculations for systems and components based on the RGM were limited except for piping and piping supports. There were no calculations available based on the RGM for the structural steel superstructures, control and instrumentation (C&I) components, electrical equipment, cable trays, and ductwork.

In conducting this review, DOE-ORP and BNI were cooperative and helpful in providing comprehensive presentations and supporting documentation. BNI provided unrestricted access to key staff, design engineers, and documentation. The full cooperation and support of DOE-ORP and BNI aided the review.

SUMMARY OF REVIEW

Based on the review, followup discussions of key issues, and satisfactory disposition of comments, the IR Team considers the revised seismic design and analysis in compliance with applicable codes and standards ensuring safety. With the implementation of RGM, BNI has introduced a refined finite element model for structural design and reduced conservatisms from the original analysis. BNI is using efficient design practices such as establishment of project design criteria documents and design guides and standardization of design analysis for reinforced concrete design and piping and pipe supports, which helps to minimize the impact to the cost and schedule of the project. The IR Team did make several findings, observations, comments, and recommendations during this review, and they are presented throughout this report. A summary of this review is presented in the following paragraphs.

Design Process. BNI's design process, including implementation of a graded approach, division of responsibilities, and the assessment of clearances and interfaces is well conceived with strong reliance on an Integrated Safety Management (ISM) program. The issue of dimensional clearances and seismic spatial interactions is being addressed through the use of a comprehensive three-dimensional (3-D) plant model. Additional procedures are being implemented to further assure that these interactions are avoided. These additional procedures should be reviewed upon completion. There are procedures in place defining roles and responsibilities for seismic considerations of the organizations that flow between disciplines including the ISM team, Civil, Structural, and Architectural, C&I, and Plant Design. Due to the limited maturity of the design, a "vertical slice" evaluation of BNI's design process could not be performed. The IR Team recommeded that a "vertical slice" evaluation be performed on a regular basis to assure the ongoing integration of the design between disciplines.

Revised Ground Motion Spectra and Soil Properties. DOE-ORP has developed the RGM spectrum to address Defense Nuclear Facility Safety Board concerns on the site-specific soil characterization and attenuation. In an effort to bound uncertainties in the soil characterization and attenuation models, the RGM spectrum is based on using 85th percentile frequency dependent amplification. It is recognized by DOE-ORP that there are some uncertainties with the soil characterization of the deeper interbeds, which are being addressed by a deep bore drilling program, scheduled to be completed in 2007. As part of this effort, the IR Team recommends reconciliation of the narrow-banded RGM response spectrum shape and lack of amplified frequency content and an independent peer review of the Probabilisitic Seismic Hazard Assessment as well as the soil characterization and dynamic properties developed.

Soil-Structure Interaction Analysis. The SSI analysis of the PT and HLW buildings was performed using the System for Analysis of Soil-Structure Interaction (SASSI) computer code. In terms of overall structural response, resulting soil pressures, and development of in-structure response spectrum (ISRS), the IR Team concluded that the SSI analysis approach meets DOE and industry standards and conforms to standard practice for seismic analysis of DOE nuclear facilities. In addition, the SSI analysis adequately captures the global seismic response of the PT and HLW buildings consisting of in-plane accelerations of walls, slabs, and ISRS from those in-plane accelerations. Local seismic response, in terms of out-of-plane accelerations and corresponding ISRS for concrete floor slabs and some steel members, is captured in SASSI through the use of vertical single degree of freedom oscillators. The IR Team recommended verification analysis be performed to assure accurate vertical response of the oscillators (lollipops) is computed from the large, complex SASSI model. BNI has initiated a sensitivity study to address this issue.

Structural Seismic Demand. The seismic demand for the primary structural concrete and steel superstructure is evaluated in a two-step process using equivalent static loads with a detailed finite element model. Based on tributary areas of the building structure, the maximum response accelerations from the SSI analysis are used to establish equivalent static seismic loads that are applied to a highly detailed finite element model. A comparison of base shears and story shears from SASSI generated accelerations and for selected accelerations used for inertia loads indicate the equivalent static model results are 25 to 30 percent conservative. Validation analysis comparing dynamic analysis results with response from the in-phase static inertial loads demonstrates that the second step static analysis introduces further conservatism. BNI's two-step process is a rational approach to obtain design stresses and deformations; however, the combination of conservative static inertial loads with conservative application of these loads may result an in overly conservative seismic response.

Structural Design

Load Path - The IR Team considered irregularities and discontinuities of floor and roof diaphragms, shear walls, and load collectors. Based on the review of the load paths and force levels outlined in the Structural Summary Report (SSR) and concrete calculations of the HLW building, BNI has demonstrated that adequate load paths are present with adequate strength based on the original ground motion and uncracked section properties. A sensitivity study of cracked and uncracked section properties in the SSR indicates that changes in seismic forces in floor diaphragms are possible. With the updated analysis based on the RGM and cracked section properties, it is recommended that force concentrations or "hard points" be carefully reviewed and addressed for the existing concrete.

The PT building load path is more straightforward as there are far fewer discontinuities and offsets. Review of concrete calculations of the PT building based on the original ground motion and interim seismic criteria (ISC) indicate adequate load paths are present. However in the design of the PT building, BNI has not analyzed the need for load collectors or drag struts.

Concrete Design - Review documents consisted of BNI's concrete methodology spreadsheet with sample shear wall calculations based on the RGM. Other concrete shear wall and diaphragm (slabs) calculations based on original ground motion and ISC were reviewed to gain a better understanding of BNI's design methodology, code compliance, and design margins. Review of calculations for the existing concrete based on the RGM has not been started by BNI. Overall, the concrete calculations reviewed are consistent in approach and meet the American Concrete Institute standards with some exceptions.

Although the RGM increased loads and some redistribution of loading will occur with cracked section properties, concrete design margins should be adequate for existing shear walls and floor diaphragms. This is based on the low demand to capacity ratios of the existing shear walls and floor diaphragms designed with the original ground motion, the ability to use inelastic energy absorption factor ($F\mu$) due to revised project design criteria, and the reduction of conservatism in the original design analysis. The primary concerns are "hard points" within both the HLW and PT buildings.

System and Component Design. The review of seismic design and qualification of systems and components included vessels, piping and piping supports, mechanical and electrical equipment, and miscellaneous steel. Overall BNI is applying DOE, industry, and project standards. A consensus on compliance and design margins of systems and components cannot be made due to the limited number of calculations available; the nature of individual calculation methodologies; variation in complexity of component design, errors, and omissions; and in-progress design issues between vendors and BNI. Numerous piping and piping support calculations were reviewed and found to be complete, comprehensive, and in accordance with good practices, except for a programmatic error in using the square root of the sum of the squares (SRSS) method for modal combination. The following represents findings and recommendations from the system and component design review:

- Seismic design and qualification criteria should adequately tier down into Material Requisitions. One example is the lack of specifying seismic anchor motion as a design condition for systems that are embedded in walls or slabs or extend between multiple support points. The IR Team recommends revising the Material Requisitions and calculations for seismic anchor motion.

- Vessel analyses do not account for the hydrodynamic mass associated with internal components. Vessels are primarily vendor designed and are in an interactive review process between the vendor and BNI. IR Team recommends re-performing analyses and design of vessels to include hydrodynamic mass of internal components and accurately apply load combinations.

- Piping analysis has a programmatic error using the SRSS modal combination instead of the 10 percent grouping method for closely spaced modes specified by the design criteria. BNI is addressing the issue in the Corrective Action Report Program.

- While BNI specifies qualification procedures that are in accordance with DOE orders and standards, some vendors are not closely following the BNI specified procedures. BNI needs increased oversight of seismic qualification by vendors.

Conservatisms. Prior to this IR, DOE-ORP and BNI identified areas of conservatism in the project design criteria and in the design performed with the original ground motion. A number of these conservatisms have been eliminated in the RGM analysis to offset the impact of the RGM. Additional conservatisms remain, which could be reduced if needed.

For seismic structural design, the seismic scale factor (SF) and Fµ are not being used to the extent that is provided for in DOE and industry standards. Conservatism on the order of 5 to 10 percent is introduced by not including the SF provision, permitted by DOE-STD-1020, which is applied to the seismic demand. For the WTP structures classified as SC-I, the use of Fµ is permitted. For SSCs that are ductile, the use of Fµ should be considered as recognized by DOE and industry standards.

For seismic mechanical and electrical equipment qualification, simplicity and conservatism in vendor seismic qualification approaches should be reviewed on an ongoing basis to verify that unnecessary conservatism is not introduced. For equipment to be qualified under the requirements of tailored International Electrical and Electronics Engineers 344, a check should be performed to verify that the condition of operability during and after the earthquake is required and achievable.

SECTION 1.0 - INTRODUCTION

1.1 Background

The scope of the Hanford Waste Treatment and Immobilization Plant (WTP) project is to design, build, and commission a plant to immobilize mixed (hazardous and radioactive) liquid waste stored in underground tanks at the U.S. Department of Energy (DOE) Hanford site near Richland, Washington. The waste accumulated between 1944 and 1989 when the Hanford site produced plutonium and other nuclear materials for national defense. The 53 million gallons of tank waste is Hanford's most serious cleanup problem. The radioactive material is stored in 173 aging steel and concrete underground tanks. The new plant will incorporate the waste in a chemically immobile glass that is stable through a process called vitrification. The elements of the WTP project include:

- Design large-scale facilities to pretreat the waste and separate it into high-level waste (HLW) and low-activity waste (LAW), mix the waste with molten glass in melters, and place it in stainless steel cylinders.
- Construct three nuclear facilities: Pretreatment (PT) facility, LAW facility, and HLW facility for pretreatment and vitrification, along with substantial supporting facilities.
- Commission the facilities to demonstrate that they meet production and efficiency criteria.

1.2 Scope

DOE, Office of Environmental Management, in collaboration with the Office of Engineering and Construction Management, authorized the Office of River Protection (ORP) to prepare a memorandum of agreement and fund the U.S. Army Corps of Engineers (USACE), Walla Walla District, to conduct independent reviews of Bechtel National Incorporated's (BNI's) implementation of the revised seismic design criteria for the WTP. The independent review objectives are to ensure BNI's revised seismic design complies with code requirements ensuring safety of the WTP structures, systems, and components, and, at the same time, ensures efficient design practices are used to minimize the impact to the cost and schedule of the WTP project.

1.3 Independent Review Team

The USACE assembled a team of experienced personnel and nationally recognized experts to review BNI's implementation of the revised ground motion (RGM) spectrum. To accomplish the independent review, USACE assigned Corps personnel to the effort and acquired the services of expert consultants specializing in nuclear facility design and seismic analysis. Biographic summaries for each member are provided in Appendix A, Independent Review Team.

1.4 Review Process

The USACE Independent Review (IR) Team reviewed BNI's revised seismic design and associated supporting documentation in detail focusing on the following key areas: design process, soil-structure interaction (SSI), structural seismic demand, structural design, and systems and components. Generally for each area the design criteria, design analysis, calculations, drawings, and specifications were reviewed. A list of documents reviewed is provided in appendix B. The review meetings were held in BNI's office with BNI providing a formal presentation, access to key personnel, and documentation access. BNI readily provided the IR Team with information needed to perform the reviews. The IR Team documented findings, recommendations, observations, and general comments in a log. The logs were compiled into a Summary Comment Sheet for each area and submitted to DOE-ORP. BNI provided responses to comments through DOE-ORP. The Comment Summary Sheets along with BNI's responses are included in appendix C. In some instances, comments were directed to DOE-ORP and USACE relating to issues such as code of record and soil explorations.

1.5 Revised Ground Motion

In February 2005, Pacific Northwest National Laboratory (PNNL) provided DOE-ORP with an update of the 1996 Hanford Probabilistic Seismic Hazards Analysis (Site-Specific Seismic Site Model for the Waste Treatment Plant, Hanford, Washington; February 2005; PNNL-15089). Based on PNNL's report, DOE-ORP revised the project design basis earthquake (DBE) (also known as the RGM) for Performance Category 3 (PC-3) SSCs in April 2005. The revised seismic ground motion horizontal response spectrum resulted in increased accelerations from little to no increase at low frequencies, to 38 percent at peak response in the region of about 5 Hertz (Hz), and 14 percent at high frequency responses, as shown in figure 1-1. The revised vertical response spectrum is similar to the horizontal response spectrum.

The RGM primarily affected the design of the important-to-safety SSCs of the PT and HLW buildings. Since the RGM increased seismic energy, BNI was required to re-perform the dynamic analysis of the PT and HLW buildings. This re-analysis includes SSI analysis, developing revised in-structure response spectrum (ISRS), re-performing static or dynamic analysis of structures, systems, or components, and re-design or verification of previously designed SSCs for the updated seismic environment.

Due to the lead time required to obtain results based on the RGM and to continue the fast-track design and construction process, an interim seismic criteria (ISC) was developed by BNI and approved by DOE-ORP in April 2005. The ISC increased all seismic accelerations by 40 percent (figure 1-1), established limiting demand to capacity (D/C) ratios, and developed mesh density factors. This was a simple and reasonable approach to conservatively capture the impact of the RGM and the concerns raised about the coarse finite element mesh used in the original analysis. This was an important step to mitigate both the cost and schedule impact of the RGM.

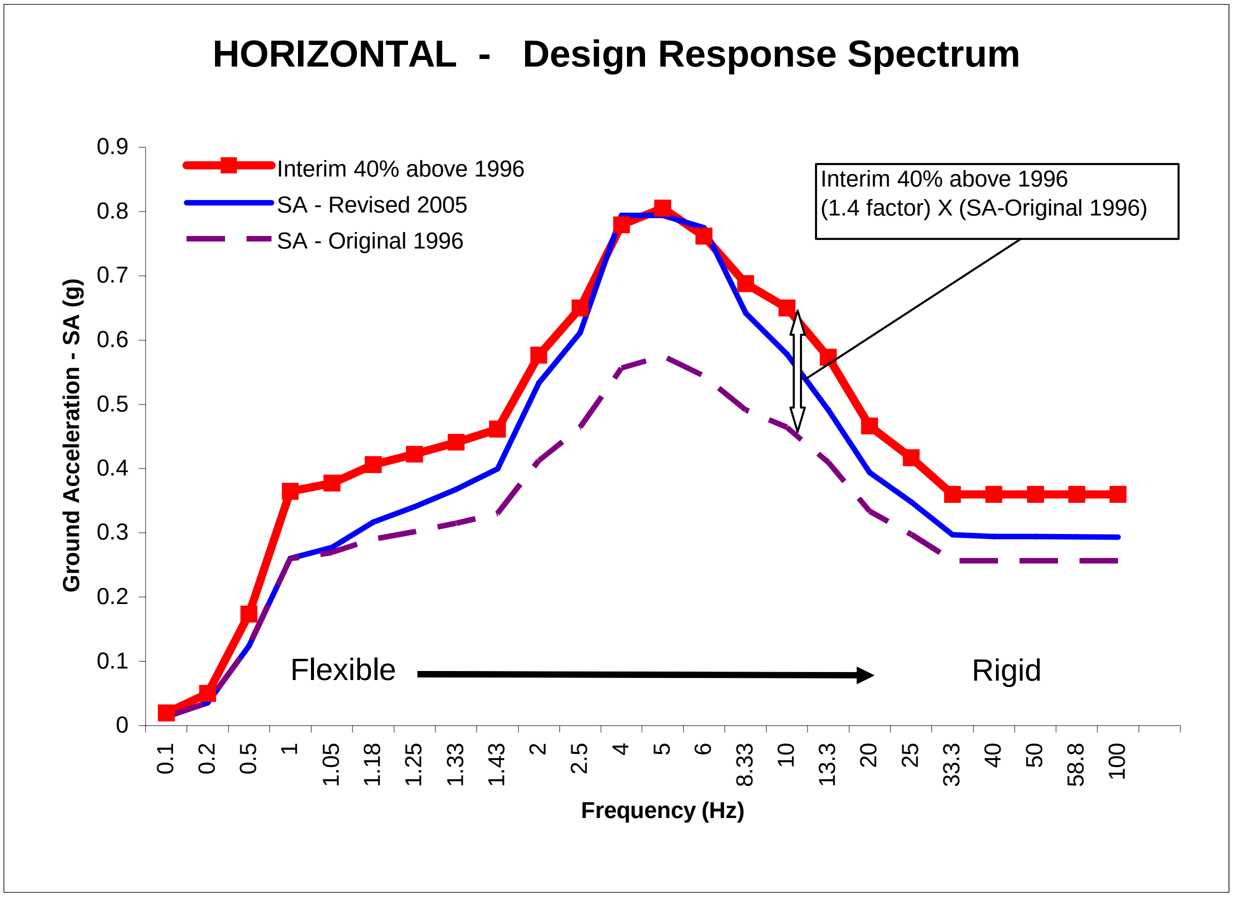

Figure 1-1. Horizontal Response Spectrum Comparison including the April 1, 2005, Interim Seismic Criteria

In an effort to reduce the impact of the RGM, DOE-ORP and BNI prepared a list of conservatisms used in the design criteria and original design analysis. Alpha ratings (A, B, and C) were given to the listed items, based on their ease of implementation and level of agreement. A number of these conservatisms are to be eliminated in the re-analysis and validation. The list of conservatisms and actions are presented in appendix D.

Since the issuance of the RGM, BNI has completed the dynamic analysis, BNI's Structural Design Criteria (SDC) document has been revised to reduce conservatism, and the use of the ISC has been terminated. All designs are currently being performed to the RGM using the revised SDC.

SECTION 2.0 - DESIGN PROCESS

The design process used by BNI for the WTP was reviewed with respect to several areas, which included categorization of SSCs, division of responsibilities among disciplines, clearances and interactions of SSCs, seismic probabilistic risk assessment (PRA), seismic analysis and design criteria (SADC), and conservatisms. IR Team comments are contained in appendix C.1. In summary, the BNI design process is consistent with DOE orders, standards, and guides; however, the review was not able to document that all DOE requirements are currently in place. Observations and concerns are presented below.

2.1 Categorization of Structures, Systems, and Components

DOE orders and standards follow a graded approach for seismic design and evaluation. By this approach, the rigor and conservatism of the design and evaluation criteria is consistent with and appropriate for the safety or mission characteristics and importance of the SSCs being designed. The graded approach is implemented by categorizing SSCs based on their safety or mission importance. Each category has different design and evaluation criteria of varying rigor and conservatism. The process of assigning SSCs to categories and of assuring the appropriate level of performance during potential future earthquakes was reviewed. The seismic categorization system employed for the WTP project is consistent with the safety analysis and DOE performance goals.

SSCs are assigned to categories for design with respect to seismic or other natural phenomena hazards (NPHs). There are four PCs where PC-1 has an NPH design criteria consistent with conventional building codes and PC-4 has an NPH design criteria similar to that used for nuclear powerplants. Based on the hazard levels associated with WTP facilities and operations, BNI determined that Safety Class SSCs shall be designed in accordance with PC-3 critera consistent with DOE standards.

For the WTP project, BNI established a seismic categorization system that accounts for functional requirements of SSCs and seismic interaction with SCs ranging from Seismic Category I (SC-I) through SC-IV. SC-I and SC-II generally follow PC-3 seismic criteria; with the exception that SC-I does not permit reductions in seismic loads to account for inelastic energy absorption capacity of SSCs. SC-II follows PC-3 seismic criteria including such reductions. The IR Team concludes BNI has appropriately implemented the seismic categorizations for the WTP.

2.2 Division of Responsibilities Among Disciplines

System design is generally functional, including redundancy, diversity, and separation of trains and components when necessary. With regard to seismic design, loading conditions that include seismic loads or seismic qualification of SSCs need to be coordinated between disciplines. The division of responsibilities between disciplines, such as engineering (civil, structural, mechanical, electrical, instrumentation and control, and fire protection), procurement, construction, and operations needs to be clearly specified and interfaces should be clearly defined. Interface control is essential to assure smooth implementation of design, construction, and operation of the facility. These aspects are physical (such as load transfers), functional, and operational requirements.

The design process review identified the basic responsibilities of different organizations within the BNI group. Based on the information provided and an independent assessment of the responsibilities of the various organizations, it appears that significant thought and effort have gone into developing a process that defines roles, maintains communications, and controls the design interfaces. However, there has been no verification of the effectiveness of the implementation of the process. The DOE-ORP peer review team attempted a "vertical slice" review in April 2006; however, BNI's design of SSCs based on the RGM was not sufficiently mature to permit this review. Therefore, it is recommended that BNI, or an independent party, perform an evaluation of a "vertical slice" through the process to verify that the initial design and any design modifications are treated appropriately. This vertical slice evaluation should be repeated at 6-month or 1- year intervals to verify the process is being maintained.

2.3 Clearances and Interactions of Structures, Systems, and Components

Facilities, such as the WTP, comprised of complex systems of diverse mechanical, electrical, and structural elements require systems integration criteria to mitigate potential interferences between systems, components, and commodities in the plant. These integration criteria are required for specifically designed SSCs, generically designed and field routed commodities, and commodities installed under good industry practices.

During the design process review, the IR Team itemized numerous approaches to address the potential for "seismic interactions." BNI's discussions were considered incomplete, as they did not recognize this as an issue for normal operating conditions, accident conditions, and for the seismic design, which considers SC-I, SC-II, and SC-III/IV SSCs. The IR Team recommends addressing dimensional interferences and clearances on a continuous basis.

The three-dimensional (3-D) model is a valuable tool to display systems (especially the required dimensional envelopes for conditions of installation), operations, and for design basis accidents. However, not all SSCs and commodities were modeled. Provisions to account for these situations should be in place. There should be verification that the 3-D model or other methods are in place to identify all sources of dimensional incompatibilities for installation, normal operations, and design basis accidents – in particular, the earthquake design basis (RGM). Appendix C.1 observations also note some basic principles that have been shown to be effective in the past for the design of new facilities. In summary, dimensional interferences and clearances needs to be recognized and addressed on an ongoing basis. The IR Team recommends that the vertical slice evaluation discussed above include verification of clearances and interaction of SSCs.

2.4 Seismic Probabilistic Risk Assessment

A seismic PRA for WTP facilities was developed for a limited purpose to verify that no SSCs need to be classified as PC-4 to meet the Radiation Exposure Standards for co-located workers and the general public. Since the purpose was limited, conservative simplifying assumptions were made as first approximations. The seismic PRA meets this limited objective.

To address future issues related to changes in perceived seismic hazard and other developments, a full seismic PRA or seismic margin assessment (SMA) is recommended as a cost effective means to determine seismic margin. A white paper is provided in appendix E. The white paper provides a discussion of the evolution of the SMA and PRA methodologies to the current status of a recognized standard to address many seismic issues that arise after design and/or operations have been initiated. Recommendations on the use of the PRA and SMA for the WTP are presented. The IR Team suggests that any future seismic PRA activities also consider the exposure of the WTP workers as an evaluation criteria.

2.5 Seismic Analysis and Design Criteria

The seismic design criteria used for analysis and design of the WTP facilities are consistent with the code of record and the nuclear and DOE industry standards at the time that the design was initiated. Since that time, some standards have been updated, but the impact to the WTP design is generally not expected to be significant. However, one area of concern is seismic detailing of steel structures.

The steel superstructure, which generally consists of braced frames for both the PT and HLW buildings, are classified as PC-3 SC-I SSCs. BNI's structural design criteria specifies, for SC-I ordinary brace frames, that no credit for inelastic energy absorption be taken (i.e., F_μ of unity). According to DOE-STD-1020, ductile detailing measures shall be employed and the design be allowed to reduce seismic forces by the introduction of a relatively small inelastic energy absorption factor (F_μ). These structures are likely to be in Seismic Category D by recent International Building Code criteria for which ordinary braced frames would not be permitted. The SADC does not reference the American Institute of Steel Construction (AISC) Seismic Provisions that have changed steel design and detailing based on failures seen during the 1994 Northridge and the 1995 Kobe earthquakes. The IR Team recommends that the performance levels, seismic demands, and detailing requirements of AISC 2002 Seismic Provisions and that used for the ordinary braced frames of the PT and HLW buildings be evaluated to ensure that required performace level is achieved with current knowledge.

2.6 Conservatisms

Based on project design criteria, the seismic scale factor (SF) and F_μ are conservatisms that are not being used to the extent that is provided for in DOE and industry standards.

2.6.1 Seismic Scale Factor

DOE-STD-1020 seismic provisions include an SF by which the seismic demand is multiplied. BNI's design criteria does not include an SF. For PC-3 SSCs, the default constant SF value at all frequencies is 1.0. [Note that the WTP design follows DOE-STD-1020-94 (code of record) rather than DOE-STD-1020-2002 in which the hazard probability and SF were modified.] However, DOE-STD-1020 states that, "Variable scale factors, based on the slope of site-specific hazard curves, may be used as discussed in appendix C to result in improved achievement of performance goals." In American Society of Civil Engineers (ASCE) 43-05, SF is referred to as a design factor (DF) and only variable DF values are used. For the original 1996 seismic hazard information, the SF can be shown to be between about 0.9 and 0.95 based on the Hanford seismic hazard curves depending on frequency. Note that the SF has not been evaluated for the new seismic hazard at the WTP facility. Using SF = 1.0 results in seismic demand that is 5 to 10 percent greater than necessary.

2.6.2 Inelastic Energy Absorption Factor

DOE-STD-1020 seismic provisions include an Fμ for PC-3 SSCs. For the WTP structures classified as PC-3, SC-I, the use of Fμ is permitted only for the portion of structures already constructed. For SSCs that are ductile, the use of the Fμ should be considered as recognized by DOE and industry standards for all PT and HLW SSCs.

SECTION 3.0 - SOIL-STRUCTURE INTERACTION ANALYSIS REVIEW

The SSI analysis review considered soil properties, the RGM, SSI analysis, overall structural response, and development of ISRS. Review comments and observations were submitted to DOE-ORP in two separate phases consisting of structural modeling comments numbered SM-1 through SM-73, and the continuation numbered SM-74 through SM-88. Both comment sets along with BNI and DOE-ORP responses are presented in appendix C.2.

The SSI analysis meets industry and DOE standards; however, there are a few issues that were identified and are currently being addressed. These include the ground motion response spectrum (comments SM-5 and SM-29) and SSI analysis for local behavior (comments SM-48, SM-49, and SM-50).

Currently, DOE-ORP is seeking improved subsurface characterization including soil properties by means of additional deep boreholes, laboratory testing, and updating the ground motion response spectrum. The global seismic response from SSI is adequately captured. The IR Team requested additional validation studies on the adequacy of capturing some of the local responses based on the System for Analysis of Soil-Structure Interaction (SASSI) analysis. These issues are included below along with general discussion of the SSI analysis.

3.1 Soil Properties

Extensive subsurface characterization was performed for the WTP project. This included subsurface drilling and sample collection, in-situ seismic velocity testing, geotechnical laboratory testing, and detailed analysis. The subsurface characterization culminated with the submittal of the Geotechnical Investigation report prepared by Shannon and Wilson, dated May 11, 2000, document number WTSC99-1036-42-17 (M1616-51). This report was used as the basis for the revised site response analysis performed by PNNL and as the basis for the material property input parameters for the SSI analysis performed by BNI.

Several issues associated with the soil properties used in the SSI analysis were identified in comments SM-52 through SM-65 presented in appendix C.2. Many of the issues were associated with the technical approach used for the geotechnical investigation including: (1) depth of borings; (2) sample collection methods; and (3) sample testing. Other issues were identified and are associated with the process governing the development of the soil properties presented in the geotechnical investigation report including apparent lack of a quality assurance program and independent review of the work.

Subsequent to issuance of the comments, BNI and DOE-ORP satisfactorily addressed all of these comments. A supplemental site characterization program, previously initiated by DOE-ORP, is underway as of the date of this report. The supplemental program will include drilling of three boreholes, each approximately 1,500 feet below ground surface and will include the collection of samples using specialized methods that will permit dynamic soil testing meeting DOE-STD-1020 and ASCE 4 codes and standards.

As noted in the referenced comments, it was not apparent that the geotechnical investigation, used as the basis for the SSI analysis and development of the RGM, was performed under an approved Nuclear Quality Assurance (NQA) 1 program or that the work was independently reviewed. Based on response by DOE-ORP, the geotechnical subcontractor that performed the work had a qualified NQA-1 program and was on the approved suppliers list for the project at the time the work was performed. In addition, the work of the geotechnical consultant was reviewed and accepted by an independent third party reviewer. No formal documentation to substantiate these process issues was provided to the IR Team.

3.2 Revised Ground Motion Response Spectrum

The revised horizontal DBE response spectrum, referred to as the RGM, has a flat plateau at a peak spectral acceleration of 0.8 g (where g equals acceleration of gravity) in the frequency range of approximately 4 to 6 Hz and peak ground acceleration of 0.3 g (figure 1-1). This is a maximum amplification of about 2.7. DOE-ORP developed the RGM spectrum to address Defense Nuclear Facility Safety Board (DNFSB) concerns on the site-specific soil characterization and attenuation. As indicated in comments SM-5 and SM-29, there were three concerns associated with the RGM:

- In an effort to bound uncertainties in the soil characterization and attenuation models, the RGM spectrum is based on using 85th percentile frequency dependent amplification. This leads to a conservative but unquantified probability level of the ground response spectrum. DOE Standard 1020 states that the seismic hazard should be a mean (M) hazard and that it is undesirable to introduce conservatism in spectral amplification for which the amount of conservatism varies with frequency.

- The RGM spectrum has a narrow banded peak in the range of 4 to 6 Hz that can result in changes in seismic response with relatively minor shifts in structure frequency.

- The RGM spectrum has high amplification at 4 to 6 Hz but no apparent amplification peak at 2 Hz, the frequency of the best estimate soil column.

DOE-ORP decided to perform additional deep boreholes at the site to better define subsurface properties. A Probabilisitic Seismic Hazard Assessment (PSHA) is being initiated to confirm the RGM spectrum meets DOE Standard 1020 requirements. The PSHA and additional soil testing being performed will provide a basis for addressing these issues.

In the development of the median amplification response spectrum based on deep borehole program, the IR Team recommends the following:

- Reconcile the narrow-banded RGM response spectrum shape with the requirements of DOE-STD-1020.

- Reconcile the lack of amplified frequency content in the RGM response spectrum shape in the frequency range of 2 Hz and less, given the SSI analysis soil profiles.

- Engage an independent peer review of the PSHA as well as the soil characterization and dynamic properties developed. This should be performed in accordance with the standard of practice for the nuclear industry.

3.3 Soil-Structure Interaction Analysis

The SSI analysis review of the HLW and PT buildings was considered from three perspectives: details of the implementation in SASSI of the SSI analysis, overall soil-structure response, and local structural behavior. An evaluation, a vertical slice of the PT building, was also performed.

3.3.1 Implementation of SSI Analysis Using SASSI Programs

The SSI calculations provide some details on the SSI analysis of the WTP buildings using the computer program SASSI. The SASSI model development details and assumptions are not described in the calculations; however, additional information is provided in a Structural Summary Report (SSR) prepared for the HLW building. The following discussion only reflects the information contained in these documents and limited discussions with BNI personnel.

SASSI uses the complex frequency response method to solve the dynamic equations of motion associated with the SSI problem. Several characteristics of this method pertinent to the assumptions used in modeling the HLW and PT buildings are summarized below:

1. Generally, it is not feasible to refine the mesh of finite element models of complex structures incorporated into SASSI analysis sufficiently to calculate all structure design related quantities, such as structure member forces and moments. Consequently, for the PT and HLW buildings, a “two-step” analysis approach was taken where the second step of structure design was performed with a detailed Seismic Analysis Program (SAP) 2000 model using seismic response from the SASSI analysis to develop loads for the SAP2000 analysis.

2. The structure model development effort proceeded with a GT/STRUDL (computer code) model for which fixed-base dynamic characteristics were evaluated. The GT/STRUDL model was transformed into a SASSI structure model of the same discretization and benchmarked against the GT/STRUDL model. Generally, the comparisons showed adequate conversion of the models. Subsequent to the benchmarking of the structure models, the SASSI structure model was modified to better capture local response consisting of vertical motion of slabs and some structural steel members, which is discussed later in this section.

3. The SASSI SSI model used equivalent linear (strain compatible) soil properties derived from the free-field site response analysis and, in the case of the material damping values, from the seismic hazard assessment. These properties are valid in the absence of the structure and no account was taken of the secondary effects of the weight of the PT and HLW buildings affecting the soil properties as recommended in ASCE 4. The added stiffness of the soil underlying the structure is assumed to be taken into account through the mechanism of soil property variation.

4. It is recognized that structural damping in SASSI analysis is represented as constant hysteretic damping. DOE criteria specify levels of viscous damping for structures. Any difference in seismic response due to damping assumptions is judged insignificant for the low values of structural damping considered.

5. The highest frequency of accurate analysis is controlled by the Nyquist frequency of the input motion, which in turn is dependent on the time step (dt) of the input motion, $f_{nyquist}$=0.5/dt. For a dt of 0.01 second, the Nyquist frequency is 50 Hz.

   The highest frequency of the analysis may be modified by specifying a frequency cut-off value, which is generally controlled by the soil shear wave velocity and the largest finite element size in the foundation model. Practical considerations for large detailed models such as the HLW and PT buildings may lead to cut-off frequencies that are too low to capture all responses of interest.

   ASCE 4 (Section 3.3.3.5 (b)) states, in part, "The cutoff frequency may be taken as twice the highest dominant frequency of the coupled soil-structure system for the direction under consideration, but not less than 10 Hz. A dominant frequency is defined as being associated with a mode having a modal mass equal to 20% or more of the total structural mass, and shall be obtained from a simplified structural model using the soil springs shown in Table 3.3-1 and 3.3-3."

   In addition to this requirement, the response quantities of interest should be considered. If only overall structure response is sought, lower frequency response dominates and the effects of selecting a cut-off frequency lower than 20 to 25 Hz is negligible. If response quantities of interest are dependent on local responses at higher frequencies, then these response quantities may not be calculated adequately. The result will be to filter out high frequency responses (similar to a low pass filter).

   Significant effort was devoted to understanding and resolving issues related to cut-off frequencies in the SASSI analysis:

- o HLW building - The cut-off frequencies selected for the SSI analysis were 10.5 Hz for the lower bound (LB) soil profile, 17.5 Hz for the M soil profile, and 22 Hz for the upper bound (UB) soil profile.

- o PT building - The cut-off frequencies were 21 Hz for the LB case, 25 Hz for the M case, and 33 Hz for the UB case.

In general, for the calculation of response quantities dependent on higher frequencies, it is desirable to have the cut-off frequency no lower than 25 Hz for typical ground motion frequency content. For overall or global horizontal structure response, it is concluded that the cut-off frequencies are adequate.

The primary purpose of SASSI analysis was to determine in-plane seismic response accelerations of walls and slabs. These accelerations are then used to establish equivalent static loads for detailed SAP2000 structural design analysis or to develop ISRS for input to attached systems and components. However, one area where local accelerations and ISRS are determined from the SASSI analysis is for vertical response of slabs and some steel members. For this purpose, a large number of single degree of freedom (SDOF) oscillators or “lollipops” covering a wide range of frequencies are introduced into the SASSI model. The IR Team recommended that verification analysis be performed to demonstrate that accurate lollipop (oscillator) seismic response is obtained considering the cut-off frequencies used. For this purpose, a sensitivity study is being performed, by BNI to further evaluate this aspect of SSI response calculation.

6. The number of computed frequencies before interpolation of the transfer functions should be adequate to provide a stable solution. This is important in calculating structural response, forces, and stresses in the case of detailed structural models that have large numbers of local modes. Generally, the number of frequency points (40 to 60) selected for explicit calculation of transfer functions is adequate to capture overall soil-structure response. Transfer functions for the remaining calculation frequencies are determined by interpolation. The additional sensitivity study, identified in paragraph 5 above, should provide insight into this issue.

7. For large, complex structures, with many closely spaced or identical modes, careful review of the transfer functions should be performed to verify that at the points of response calculation, adequate frequency discretization, and number of frequencies are selected.

3.3.2 Overall Soil-Structure Response

A two-step approach has been undertaken by BNI in which overall behavior of the structure is determined from the dynamic SSI seismic analysis, and local structural behavior is determined from a detailed finite element model subjected to equivalent static seismic loads represented by accelerations derived from the SASSI analysis (application of the bubble plots of accelerations, see section 4.0 Structural Seismic

Demand). As a result, it is generally the goal of the SASSI analysis to capture the overall seismic behavior of the building and the goal of the equivalent static analysis to capture local behavior for structural design.

Overall soil-structure response includes:

- Seismic response accelerations at floor levels sufficient to determine equivalent static seismic inertial loads for structural design.
- ISRS at selected locations for system, sub-system, component design, and qualification.
- In-plane forces and moments on selected walls, basemat, and diaphragms, generally, those structure elements interfacing with the soil.
- Seismic soil pressures acting on exterior walls.
- Overall soil-structure stability parameters: sliding, uplift, and overturning.

Generally, the SSI models are adequate to calculate overall structure response.

3.3.3 Vertical Slice Analysis for PT Building

An independent evaluation by the IR Team of the PT building for east-west direction earthquake shaking provides some assurance that SSI results are reasonable. The overall objective of independently benchmarking the PT building model and response is generally satisfied through the efforts of this evaluation.

3.3.4 Local Structural Behavior

For out-of-plane behavior of floor slabs obtained directly from the SASSI analysis, it is important to validate that higher frequency response and local modes have been properly taken into account (items 5, 6, and 7 discussed above). Local modes were developed to represent vertical dynamic behavior in the SSI analysis. There is no mention of local modes to represent horizontal behavior of walls or structural steel members where structural or non-structural mass leads to low frequency local modes.

In addition, procedures should be in place to account for the effects of local structure response on the input to SSCs design and qualification. The local effects include dynamic amplification of in-structure responses, such as time histories or ISRS, due to supporting elements having frequencies below 25 Hz and not modeled or modeled approximately in the SASSI model.

3.3.5 Summary

- Cut-off frequencies:
    - o For overall or global seismic response, it is concluded that the cut-off frequencies are adequate for the three soil profiles and the HLW and PT buildings.

- o For local vertical seismic response, in particular related to the ISRS calculated for the "lollipops," it is recommended that further justification should be provided for the HLW building analysis, for the LB soil profile, that the cut-off frequency is adequate for the response quantities of interest, namely those of higher than 10 Hz and subject to local amplification. A sensitivity study is currently being performed to address this issue.

- SASSI was developed to model the overall behavior of soil-structure systems. With the advent of increased computational power more detailed and sophisticated models have been developed. The question remains for these large, complex models with many closely spaced or identical modes, are there adequate procedures in place, with appropriate documentation, that demonstrates that a sufficient number of transfer functions are included in the analysis with regard to calculated vs. interpolated frequencies and with regard to frequency cut-off.

- Generally, it is believed that the SSI models are adequate to calculate overall or global structure response. An independent analysis of the PT building was performed, which generally validated that the overall behavior, as predicted by the SASSI analysis, was reasonable.

- For local structural behavior, validation of the SASSI model behavior for responses and locations that are affected by local dynamic behavior requires additional consideration.

3.3.6 Recommendations

- For local vertical seismic response, in particular related to the ISRS calculated for the "lollipops," demonstrate that the cut-off frequency is adequate to accurately determine the response quantities of interest.

- Document the BNI procedure that assures that SASSI transfer functions are adequate with regard to calculated vs. interpolated frequencies and with regard to frequency cut-off.

3.4 In-Structure Response Spectrum

Acceleration response spectra are computed directly from acceleration time history responses all within the BNI version of SASSI. The resulting spectrum in each direction (i.e., X due to x input, X due to y input, X due to z input, etc.) is combined by the square root of the sum of the squares (SRSS) for each direction of seismic response. Narrow peaks that will be broadened are reduced in accordance with ASCE 4. The spectrum is then broadened to cover uncertainties. A spectrum for each soil profile (LB, M, and UB) is enveloped with manual smoothing to eliminate unwanted spectral dips. To capture the bending response to vertical ground motion of concrete slabs or steel beams or girders, SDOF oscillators are added to represent this behavior and enable response spectrum generation at these locations.

The frequency interval for ISRS generation more than adequately meets the requirements provided in ASCE 4. Overall, the evaluation of ISRS appears to have been performed in an accurate manner using methods consistent with nuclear/DOE industry practice and standards.

Enveloped ISRS are used when the individual ISRS are not exactly at the equipment supports or for equipment with multiple supports (e.g., cranes). The enveloping is accomplished by an Excel macro and by visual inspection, reasonable results are obtained.

SECTION 4.0 - STRUCTURAL SEISMIC DEMAND

This section addresses the analysis approach used by BNI to resolve the seismic demand into the various structural elements of the PT and HLW buildings. The review focused on the two-step analysis method, inertial load, torsion, and structural models. The comments and observations developed as part of this review are presented in appendix C.2 for structural modeling.

4.1 Two-Step Analysis Method for Structural Design

The structural design approach used by BNI is a two-step analysis method in which the first step of dynamic analysis produces nodal accelerations and the second step uses static analysis to evaluate element stresses for the purpose of structural design. The basic premise of this approach is that applying the dynamically calculated accelerations, derived from the SSI analysis to the structure nodal points with mass will provide a static load vector, which will produce load distributions throughout the structure that adequately reproduce the maximum structural element loads calculated by the dynamic analysis. The two-step analysis method is standard practice for DOE nuclear facility seismic analysis.

Seismic forces for design are determined from static analysis of a detailed SAP2000 model with seismic inertial loads developed from floor seismic response accelerations. For the steel superstructure, the design forces are verified by comparison of forces from the SASSI analysis. The use of static analysis with seismic inertial forces is criticized by some because higher mode seismic behavior of some local regions is not captured. BNI judgment that such behavior is not significant for the concrete shear wall portion of these structures was confirmed by the analysis presented in appendix G of the SSR. The SASSI time history check will capture this phenomenon if it were to occur in the steel superstructure.

Several basic questions arise with respect to this two-step analysis method:

- This method is presumed to work best when minimal variations in displacements and accelerations are calculated within a region or sub-region of the basemat, diaphragm, or wall of interest. If significant variations in displacement occur, this is likely an indication that higher modes (i.e., higher than the fundamental frequency) contribute importantly to the response. BNI handled this situation by evaluating variations in nodal accelerations, select regions, or sub-regions that are assumed to behave similarly, and applying the weighted average acceleration for the sub-region to these nodal degrees of freedom.

- The second step equivalent static analysis does not address the question of phase relationships between the nodal displacements (or accelerations) at each of the nodal degrees of freedom. For the SAP2000, equivalent static analysis used for structural design seismic inertial loads are applied in-phase and out-of-phase response is not captured. In appendix G of the SSR, it is demonstrated that application of static inertial loads in phase is a source of significant conservatism for the response of concrete walls and slabs. For the steel superstructure, the member forces calculated by applying equivalent static loads were also shown to be conservative when compared to those calculated in the dynamic analysis.

4.2 Inertial Loads from Dynamic Analysis for Structural Design

The seismic loads calculation present maximum accelerations at each elevation of the HLW building. These accelerations were determined from the bubble plots that give the maximum accelerations at each location of the model from the SASSI seismic analysis. The accelerations that are used to determine seismic inertial loads for the equivalent static seismic analysis of the building are also presented. Comparing acceleration values indicates that conservatism is introduced when the seismic inertial load values are selected.

Base and story shears are provided from the SASSI generated accelerations and for the selected accelerations to be used as inertial loads. In one case reviewed, the base shear is 79,000 pounds of force (79 kips) in the east-west direction and 66 kips in the north-south direction using SASSI accelerations (weighted averaged maximum nodal accelerations). Using the accelerations selected to establish seismic inertial loads, the base shear is 98 kips in the east-west direction and 84 kips in the north-south direction. This comparison demonstrates that conservatism on the order of 25 to 30 percent is introduced in the seismic loads used for determining design stresses and deformations for the case cited.

4.3 Accidental Torsion

Additional forces in shear walls are computed due to accidental torsion in the manner described in the kick-off presentation for topical review meeting No. 1, which is consistent with that described in section 3.1.1 of ASCE 4. Although termed accidental torsion, the introduction of increased seismic forces is intended to account for both response due to accidental eccentricity (due to unforeseen reasons such that it may not be incorporated into the structural model) as well as the effects of non-vertically incident or incoherent seismic waves that can also induce torsional response. Since accidental eccentricities and non-vertically propagating waves are not explicitly included in some manner in the seismic analysis, the approach used for the WTP buildings is reasonable and in compliance with ASCE 4.

4.4 Structural Design Model

The initial structural model of the WTP buildings is developed using the computer program GT/STRUDL. A refined mesh finite element model was developed using the SAP2000 computer program. SAP2000 was used because GT/STRUDL was near its capacity before the mesh refinements could be made. The refined SAP2000 model improved:

- In-plane shear distribution in small wall piers.

- Transverse moments in shears in slabs.

- Areas with offsets in slabs and walls are directly addressed within SAP2000 to eliminate hand calculations required to evaluate additional moments induced by the offsets.

- Significant size openings (i.e., openings with at least one side larger than the thickness) are adjusted to be more precisely represented.

- Use of thick shell element not available in GT/STRUDL to more accurately capture transverse shear stiffness and deformations.

- Cracked bending stiffness to be implemented without modifying the element thickness such that stress results are more readily available for design.

The SAP2000 model greatly improved the representation of the PT and HLW buildings for the purposes of structural design over the GT/STRUDL model.

4.5 Recommendations

- At the structural design stage (reinforced concrete and steel), the IR Team recommends verifying the assumptions implemented in the SAP2000 model, which are the basis for the design load distributions. An example is the general assumption that beam-column connections are not moment resisting and have been modeled with moment releases. Design procedures should include provisions to verify that structural modeling characteristics are appropriate for the load distributions transmitted from the SAP2000 model.

- At the SSC design and qualification stage, the IR Team recommends incorporating potential dynamic amplification of structural elements, not modeled in the SASSI model due to size limitations and confirm procedures are in place to assure proper account is taken for dynamic amplification of structural elements and portions of structures at the design and qualification stages of SSCs. Examples are vertical floor amplification and amplification of frames including the effects of systems and components supported from them.

- Include provisions for installation of seismic instrumentation within PT and HLW buildings to capture strong motion shaking data to aid in post-earthquake evaluation.

SECTION 5.0 - STRUCTURAL DESIGN

The structural design reviews focused on the following:

- Load Path, review of the continuity and strength of the lateral load resisting system from the roof to the basemat with emphasis on the existing concrete elements, appendix C.3.
- Concrete Design, review of BNI's design methodology and design margin of existing concrete, appendix C.4.
- Anchorage and Embedments, a review of BNI's design guide and embedment calculations, appendix C.4.
- Structural Steel Design, review of miscellaneous steel, appendix C.5.

5.1 Load Path

Structural load paths of both the HLW and PT buildings were reviewed. The review considered irregularities and discontinuities of floor and roof diaphragms, shear walls, and load collectors. The walls and floor slabs of the HLW building contain numerous openings, discontinuities, and offsets making it difficult to visualize the building load path. BNI's SSR provided extensive data on seismic force distributions and shear wall discontinuities for the HLW building. The SSR proved to be a valuable tool in understanding the building structural load paths and the extent and detail of this information facilitated the review. Seismic force distributions presented in appendix I of the SSR are based on the GT/STRUDL static analysis with a coarse finite element model. This model assigned uncracked stiffness properties to the shear walls and floor diaphragms for the out-of-plane direction. The SSR was prepared prior to the adoption of the RGM and prior to the conversion to the SAP2000 model. The SAP2000 model includes the seismic demand based on the RGM, effective cracked stiffnesses of 50 percent of the uncracked stiffnesses, and a refined finite element mesh. A study on the impact of the effective cracked stiffnesses is documented in appendix K of the SSR. This study indicates that changes in seismic force distribution in the floor diaphragms are possible.

Based on the review of the load paths and force levels outlined in the SSR and some concrete calculations of force concentrations or "hard points" of the HLW building, BNI has demonstrated that adequate load paths are present with adequate strength based on the original ground motion and uncracked section properties. With an increase in seismic demand from the RGM and potential for redistribution of loads with the use of cracked section properties, "hard points" of the structure should be carefully reviewed and addressed in the final concrete design. Some of these areas are identified in the review comments in appendix C.3.

The PT building load path is more straightforward as there are far fewer discontinuities and offsets. Review of concrete calculations of the PT building based on the original ground motion or ISC indicate adequate load paths are present. However, in the design of the PT building, BNI has not analyzed the need for load collectors or drag struts (CD-12, appendix C.4). There are numerous "hard points" at intersections of diaphragms and shear walls that need to be analyzed for load collectors. The IR Team recommends evaluating the need for drag struts and load collectors in PT building.

5.2 Concrete Design

To assure uniformity in the design of reinforced concrete shear walls and similarly for concrete diaphragms, BNI established a concrete design methodology described in an Excel spreadsheet (24590-PTF-DGC-S13T-00040, Methodology and Example for Shear Wall Analysis). This spreadsheet along with sample shear wall designs based on RGM were reviewed. Revision of calculations for the existing concrete based on the RGM has not been started by BNI. Other concrete shear wall and diaphragm (slabs) calculations based on original ground motion and ISC were also reviewed to gain a better understanding of BNI's design methodology, code compliance, and design margins.

Overall, the concrete calculations reviewed are consistent in approach and meet the American Concrete Institute (ACI) standards with some exceptions:

- The capacity for in-plane moment is an approximate approach and is considered conservative. Simplifying assumptions using the 10 percent of the wall length for the compression zone and 20 percent wall length for the tension zone are used. An accurate analysis would require a stress-strain compatibility analysis to include wall flanges and load demands within the wall flanges. BNI is in the process of updating the calculation to include a stress-strain compatibility analysis.

- The computation to determine total required steel in each face was incorrect. The logic presented for required tension steel did not result in stress-strain compatibility across the section. The appropriate distribution for area of steel in one face is one-half the tension steel plus one-half the in-plane bending moment steel plus transverse bending moment steel. BNI updated the calculation template to incorporate this correction.

Although the RGM increased loads and some redistribution of loading will occur with cracked section properties, concrete design margins should be adequate for existing shear walls and floor diaphragms. This is based on the low D/C ratios of the existing shear walls and floor diaphragms designed with the original ground motion, the ability to use the Fμ due to revised project design criteria, and the reduction of conservatism used in the original design analsysis. The primary concerns are "hard points" within both the HLW and PT buildings that required an analysis to validate D/C ratios.

5.3 Anchorage and Embedments

Anchorage and embedment calculations based on the RGM were limited. Only BNI's *Engineering Design Guide for Embeds and Surface Mounted Plates* (3DG C13 014), PT vessel anchorage calculation and an embedded plate calculation were reviewed.

Both the HLW and PT buildings rely on a large number of steel embedments on walls and slabs for attachment to systems and components within these buildings. The design guide provides a cost-effective means of selecting appropriate embed and surface mounted plates for a range of configurations and loadings within the limitations stated. However, a number of design guide comments were made to which BNI responded by indicating that the guide has not been used for the WTP. Should BNI, in the future, elect to use the design guide on the WTP, the comments should be addressed since the guide contains sections that are deviations from DOE-STD-1020-94.

The Vessel Ring Embedded Plates (24590-PTF-DDC-00001) calculation provide the anchorage design for vessels based on the original ground motion. The load path from the vessel skirt or saddle to the supporting concrete is continuous. Seismic loads on the anchors and ring beams are evaluated in a conservative manner in the determination of the impulsive weight for full tanks and the impulsive and convective (sloshing) weights and their height of application for partially full tanks. In addition, for the vertical cylindrical vessels, the seismic overturning moment and shear is determined by 100-40-40 direction component rule and vector sum of perpendicular components, which is appropriate for shear but conservative for tensile forces due to moment. Observations related to this calculation are presented in appendix C.4.

The embedded plates calculation (24590-PTF-DDC-S13T-00021) for through wall anchorage is based on the RGM. The design calculation was complete and performed in a rational manner.

5.4 Structural Steel Design

The steel superstructure was not available for review due to limited design maturity of the structural steel. However, the review did cover several miscellaneous steel calculations consisting of multi-commodity support beams, structural steel floor framing, and structural steel crane runways. The IR Team comments are provided in appendix C.5. A consensus on the compliance and design margin of miscellaneous steel cannot be made due to the limited calculations and the nature of individual calculation methodologies, which include variation in complexity of the component design, different code applications, level of documentation, and errors and omissions. Responses to review comments from BNI are pending.

5.4.1 Multi-Commodity Support Beams

Based upon the review of a multi-commodity support beam calculation, the following issues were identified:

- Loads on the beams were estimated on a tributary area basis. While this may be appropriate for dead loads, it may be unconservative for seismic loads, particularly piping loads in the axial direction. This assumption needs to be monitored as piping and commodity designs mature. Piping is not typically supported axially at every support and certain support beams might have higher loads than assumed in the calculation. The calculation did not reconcile the assumed loads with the actual loads.

- Pipe friction loads were considered in the static load cases but not in the seismic load cases. Piping transmitting longitudinal seismic reactions to the support beams should also be capable of transmitting friction forces.

- Weak axis bending for static loads is based on the upper half of the beam section, appropriate for dealing with eccentric loads, while for seismic loads it is based on the full section. Since the same eccentricities exist for static and seismic, this could be un-conservative for the seismic load case. BNI should justify or modify the calculation method based on final configuration.

- The 100-40-40 rule for combining directional responses was used incorrectly and un-conservatively. Both dead loads and seismic loads are used in the combination, whereas only seismic loads should have been used, then combined with 100 percent of the dead load.

- Other discrepancies in the calculation were noted in the specific review comments, SD-2 and SD-3.

The IR Team recommends evaluating multi-commodity support beams based on actual commodity loads, appropriate seismic load, and 100-40-40 rule load combinations.

5.4.2 Structural Steel Floor Framing

The structural steel floor framing design calculations reviewed were complete and met project design standards. Design margins reported in the calculations were small (i.e., maximum D/C ratios were close to 1.0). However, the calculations contain many conservatisms. The most significant conservatisms were the use of the full mass of the supported concrete slab versus use of 10 percent of the mass per the design criteria, and the use of the peak of the 4 percent damped floor response spectrum versus 7 percent per the design criteria. Thus, the actual design margins are quite large, on the order of 2 to 3.

5.4.3 Structural Steel Crane Runways

The crane runway calculations reviewed were updated with the RGM. The calculations contained two unconservative inconsistencies with the design criteria. First, the design criteria requires use of peak spectral accelerations when the equipment frequency is less than the peak frequency. However, the calculations in the appendixes used accelerations in the longitudinal and vertical directions at the crane frequency, which is below the peak frequency, rather than peak accelerations. Use of the peak accelerations would increase the calculated stresses. Second, the crane runway frequency calculation did not include the vertical offset of the crane mass. Inclusion of this offset would reduce the calculated lateral frequency, possibly to less than the 33 Hz required by the design criteria.

The reported design margins are not large, with calculated stresses on the order of 85-90 percent of the allowable stresses. The issues discussed above could reduce these margins. However, there are certain conservatisms that, while not significant, would act to increase the available margin by reducing the calculated stresses if taken into account. These conservatisms include the arbitrary 20 percent increase in crane mass and the use of absolute value combination of directional responses versus the 100-40-40 rule permitted by the design criteria.

SECTION 6.0 - SYSTEM AND COMPONENT DESIGN

The system and component reviews focused on the following:

- Vessels, appendix C.6.
- Piping and piping supports, appendix C.7.
- Mechanical and electrical equipment qualification, appendix C.8.

At the time of the reviews, there were no calculations based on the RGM for control and instrumentation (C&I) components, electrical equipment, cable trays, and ductwork. A consensus on the compliance and design margin of systems and components cannot be made due to the limited calculations and the nature of individual calculation methodologies, which include variation in complexity of the component design, different code applications, level of documentation, and errors and omissions. Responses to review comments from BNI are pending.

6.1 Vessels

The design criteria for WTP vessels is established by the American Society of Mechanical Engineers (ASME), Section VIII, Division 1, which is a non-nuclear code. BNI specifies that Section VIII, Division 2, Appendix 4, stress analysis and stress allowable criteria be used in order to better evaluate the effects of extreme loading, local discontinuity stresses, and thermal fatigue. Section VIII, Division 2, is also a non-nuclear code but is similar to the nuclear Section III, Division 1, code for Class 1 vessels or Class 2 vessels designed by analysis.

The principal difference between Section VIII, Division 2, and Section III, Division 1, is the allowable primary stress for extreme loads. Section VIII, Divisions 1 and 2, allow a 1.2 increase in the primary stresses when seismic loads are included. In Section III, Division 1, the allowable stress increase for Service Level D, which includes the Safe Shutdown Earthquake (SSE), is nominally a factor of 2.0 with some restrictions. However, the SSE seismic input to nuclear powerplants is equivalent to PC-4 as opposed to PC-3 specified for WTP design. Even so, the WTP criteria will result in designs that are conservative relative to designs in accordance with nuclear codes and standards. This conservatism could be demonstrated in a risk-informed format if there are any seismic induced conditions of overstress that are prohibitive to mitigate by physical modifications to the vessels and their internal components.

Seismic and nozzle load analyses were examined for nine vessels. All of the vessels were vertical cylindrical tanks supported on skirts. Typical vessels included internal assemblies such as pulse jet mixers (PJMs) as shown in figure 6-1. Vessel seismic analysis were typically performed using detailed 3-D finite element models.

Based on reviews of the nine vessels, technical issues were present in several or all of the seismic evaluations. These issues included the following:

- Fluid-structure interaction.
- Treatment of damping.
- Nozzle loads and analysis.
- Skirt buckling.
- Interpretation and combination of vessel stresses.
- Mode combination.
- Earthquake component combination.

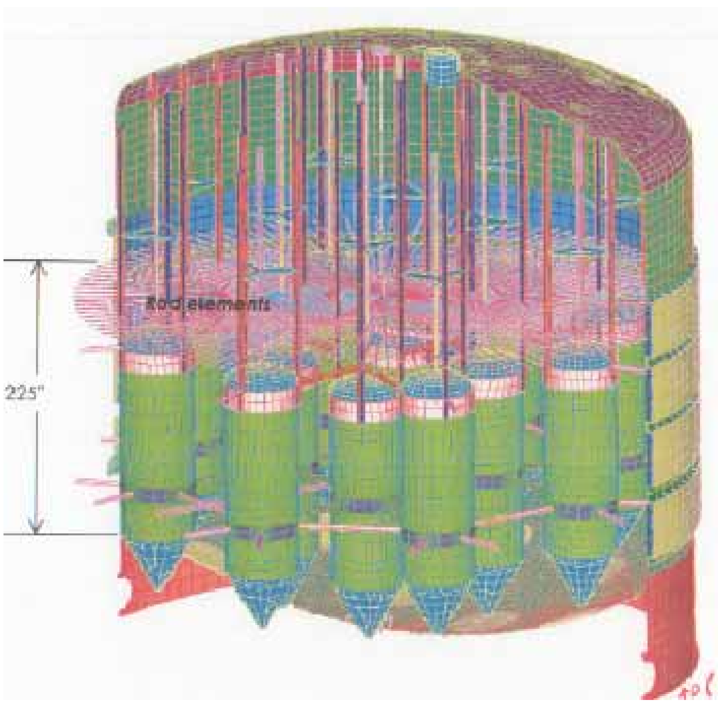

Figure 6-1. Typical Skirt-Supported Vessels and Vessel Internals

BNI agreed with and or reconciled most of the comments made by the IR Team and is working with vendors to correct their analyses for compliance with the vessel specifications and governing codes and standards (appendix C.6). However, there remain a few open issues that, to date, are not completely resolved. BNI acknowledges that they are resolving these open issues with the vendors, but a final resolution has not been transmitted to the IR Team.

The following sections present findings and recommendations.

6.1.1 Fluid-Structure Interaction

The analysis, in general, treated the fluid impulsive and convective mode loading on the vessels in accordance with current guidance for vertical tanks containing fluid; however, most of the vendors did not account for the added mass effect for the submerged internal components. Methods for accounting for this effect are discussed in ASCE 4-98, paragraph 3.1.6.2. The added hydrodynamic mass would increase the reactions in the supports.

BNI conducted an independent analysis of one vessel that addressed the fluid-structure interaction correctly. The analysis used a coupled finite element model consisting of the vessel, the internal PJMs, and fluid elements. The results in some areas were different than the vendor results. From the BNI analysis, it was determined that the vessels and support skirts would not be appreciably affected and would retain adequate margin. However, supports of internal components may experience increased loads and may require modification depending on the current stress levels.

BNI indicated in their response to review comments that they will require the vendors to conduct coupled analysis (appendix C.6, comment VL-7). If the vendor is not capable of performing this analysis, BNI indicated they would assume responsibility for final analysis.

6.1.2 Treatment of Damping

Damping values are specified in BNI specification 24590-WTP-3PS-SS90-T0001 for two levels of response. Since skirt buckling is a critical failure mode, level 1 response is used and the specified damping is 2 percent for the fluid impulsive mode, 0.5 percent for the sloshing mode, and 3 percent for all other equipment. BNI specification 24590-WTP-3PS-MV00-T0002 for vessels, paragraph 7.3.1, states that internal components, supports, and piping systems shall be analyzed the same as the parent vessel unless otherwise noted. This implies that the internal assemblies, their supports, and piping be analyzed for 2 percent damping. This has been the practice of most vendors and is the position taken by BNI. There are currently some overstressed internal assembly supports and nozzles subjected to loads from internal piping. Within the guidance of existing BNI specifications, higher damping could be used for internal assemblies. In cases where instability is the failure mode of internal supports, 3 percent could be used and in cases where stress in supports, vessel walls, or nozzles is the failure mode of concern, 4 percent could be used. The use of different damping in a coupled model is difficult and requires composite modal damping methodology.

For many of the vessel seismic evaluations, damping has been considered in an unnecessarily conservative manner in that 0.5 percent damping has been used for all horizontal modes and 2 percent damping has been used for all vertical modes. The treatment of different damping for impulsive and convective modes can easily be accomplished by using a 0.5 percent damped spectrum up to a frequency just beyond the convective mode frequency and then using 2 percent damped spectrum beyond this point. This defaults to 2 percent damping for internal assemblies and is in compliance with a conservative interpretation of the BNI specifications.

However, where warranted by overstress conditions, BNI should consider more complex analytical methods to use higher damping for internal components to reduce the seismic demand (appendix C.6, comment VL-6 and VL-7).

6.1.3 Nozzle Loads and Analysis

Vendor evaluation of the vessels at the nozzles followed the correct methodology of WRC-107. However, one load combination was missed, which did not affect the design of the vessels checked but has the potential to cause overstress. This needs to be rectified.

In some of the analyses reviewed, vessels were overstressed locally due to nozzle loads. BNI originally provided generic nozzle loads and criteria for evaluation of nozzles in specification 24590-WTP-3PS-MV00-TP001. Subsequently, BNI replaced these nozzle loads with more plant specific loads resulting from piping analysis. There are still some issues with nozzle load overstress and this is an ongoing iteration between BNI and the vendors. In most cases, nozzle loads are controllable by addition or modification of pipe supports or changes in pipe routing.

Generally, the vendors use WRC-107 methodology for evaluating the stresses in the vessel wall due to nozzle loads, and analysis procedures are correctly applied using software that conducts the WRC-107 anayses. Two load combinations have been evaluated. In the first case, primary stresses were evaluated for a load combination of weight, seismic, and operating pressure. In the second case, secondary through-the-wall bending stresses were evaluated for a load combination including restraint of thermal expansion loads combined with the primary loads. The two load combination cases are in accordance with BNI specification 24590-WTP-3PS-MV00-TP001, paragraph 3.7, but conflict with the requirements of ASME Section VIII, Division 2, Appendix 4-138. Appendix 4-138 requires that nozzle loads are to include all external loading due to weight, seismic, and restraint of thermal expansion. This single load combination should be used to compute primary and secondary stresses. The vendors have correctly analyzed the primary-plus-secondary load combination but have not evaluated primary stresses for this load combination. The reviews did not observe any cases where the primary stresses would be above the allowable stress but this may not always be the case.

6.2 Piping and Supports

There were 6 pipe stress calculations reviewed along with 54 associated pipe support calculations. Pipe stress calculations covered various sizes of pipe. Pipe support calculations included both standard designed supports and custom designed supports. In general, the pipe stress and pipe support calculations meet DOE and industry standards and practices except for the method used for modal combination. Review comments are presented in appendix C.7.

6.2.1 Pipe Stress Calculations

Other than the issues addressed below, all calculations reviewed were complete/comprehensive and adhered to good quality assurance (QA) practices (e.g., statement of codes and standards employed, checking/signature/approvals, and document control). Design codes and standards, design inputs, and pipe stress analysis techniques are appropriate.

- All piping stress calculations reviewed have a programmatic error. They do not call out a modal combination method in BNI's piping stress program ME-101 (computer code) input; thus, the default for modal combinations is used. In ME-101, the default is the SRSS method of modal combination. Closely spaced modes are not considered as required by Design Criteria 24590-WTP-DC-01-001 and by industry practice (e.g., ASCE 4). Design Criteria 24590-WTP-DC-01-001 requires the 10 percent grouping method.

- One calculation reviewed was revised due to a revision to the ME-101 Report Writer modules. Report Writer is a program language in ME-101 that allows the analyst to create standard report modules. Some of the modules used, perform calculations such as friction forces on supports, corroded pipe stress calculations, etc. These modules are not part of the general ME-101 verification. The modules that perform calculations must be verified.

6.2.2 Pipe Support Calculations

All pipe support calculations reviewed were complete, comprehensive and adhered to good QA practices [e.g., computer code verification and validation, statement of codes and standards employed, checking/signature/approvals, and document control]. Design codes and standards, design inputs, and pipe support analysis techniques are appropriate. Pipe support loads are derived from the pipe analysis. Due to the programmatic error of using the default modal combination of SRSS described above, the loading on supports will increase with closely spaced modes. A detailed review or reanalysis is required.

6.3 Mechanical and Electrical Equipment Qualification

There were only a limited number of calculations available at the time of the review. The majority of completed calculations were for the mechanical handling equipment. However, there were a sufficient number of examples to conduct a comprehensive review of the seismic qualification process for mechanical and electrical equipment.

In most cases, there is significant interaction between disciplines required to achieve seismic design and qualification of SSCs. Systems design defines performance requirements of systems and their components. Mechanical; electrical; C&I; heating, ventilation, and air conditioning; fire protection; and others design the functional systems and their support systems. Integrated Safety Management (ISM) coordinates the various activities. With respect to seismic design and qualification, Civil, Structural, and Architectural (CS&A) provides the following:

- ISRS for analysis and testing of systems and components.
- Reviews Material Requisition (with regard to seismic requirements).
- Reviews vendor submittals (proposal and final design package) for acceptable seismic analysis and testing and compliance with SADC and Safety Requirements Document (SRD).
- Receives seismic anchorage loads for inventorying and verification that embeds or other anchorage systems are adequate.
- Receives other loading conditions to be included in the structural element design.

The number of interactions and the stages of design at which interaction is required amongst all of the above-mentioned disciplines requires close coordination for project success.

The IR Team provided the following observations and findings:

- The requirements for seismic qualification of equipment with regard to input from supported structure response, effects of multiple supports, and reduced inelastic energy absorption capacity are recognized in seismic criteria for DOE-STD-1020 and from the BNI SADC for the WTP project. However, not all of these requirements have been tiered down to the Material Requisitions for the seismic qualification for equipment, tanks, and for control and electrical systems. The general specifications do not include any considerations for seismic anchor motion (i.e., relative displacements between multiple support points) as required in section 2.4.1 of DOE-STD-1020 and in the BNI SADC.
- The engineering specifications permit four seismic qualification methods: (1) analysis; (2) test; (3) combined analysis and test; and (4) past qualification in a nuclear installation. The specifications provide detailed requirements for past qualification by test. Similar detailed requirements are needed for past qualification by analysis.
- Large in-wall and in-slab components, such as doors, hatches, and other large penetrations should be designed and qualified to a RGM seismic environment that includes ISRS at the support locations and relative motions of the wall or slab. For inclusions that are stiff, relative to the supporting wall or slab, a combined analysis including the wall or slab and the relatively rigid inclusion may be required to adequately account for the interaction between the two. For inclusions that are relatively flexible, relative to the supporting wall or slab, the displacements at the cutout or penetration should be superimposed on the inclusion to account for

relative motions. For small inclusions, relative motions of wall or slab are likely not of concern.

- In a number of calculations, steel members were to be qualified using the nuclear structures steel code, AISC N690. Implementation of AISC N690 appeared to be difficult for many vendors, as the calculations were somewhat confusing and, in several instances, excessively conservative. For example, in one calculation, stresses were held at 1.4 times the allowable stress capacity. This limit is appropriate for shear, but it was applied for all behavior modes. In the case of shielded process doors, allowable stress capacities were used and apparently not increased by 1.33 for seismic conditions or 1.6 (1.4 for shear) per AISC N690 provisions.

- There were cases where anchorage to concrete followed ACI 349-01 as appropriate, but in another calculation, allowable stress capacities were used. Another example of excess conservatism was for hatches where a detailed finite element analysis demonstrated rigid behavior, but the peak of the ISRS was used instead of the peak ground acceleration.

- Based on BNI's presentation, there may be some items in black cells that are SC-III or lower. Items in the black cells should be categorized SC-I or SC-II given that entry and inspection is not possible after an earthquake of any size occurs.

- The SRD specifies tailoring of applicable standards. For International Electrical and Electronics Engineers (IEEE) 344, the SRD states, “The Scope, section 1.0, of IEEE 344 applies to equipment that needs to function during and after an SSE for a Nuclear Power Generating Station. For WTP the equipment that needs to function during and after a DBE is SDC/Safety Design Significant/Safety Class/Safety Significant (SDS/SC/SS) equipment which must be qualified to SC-I.” The requirement of function during the earthquake may be too restrictive in some cases and exceptions due to timing required to switch from normal power to emergency power may be taken into consideration. The requirement of the tailored IEEE 344 to require all SDC/SDS/SC/SS equipment to be tested to remain functional during and after the earthquake should be verified as necessary. If so verified, all Material Requisitions should highlight this requirement to the bidders.

6.4 Recommendations

The IR Team provided the following recommendations:

- Re-perform analysis and design of vessels to include hydrodynamic mass of internal components and accurately apply load combinations.

- Ensure vendors performing vessel analysis and design have sufficient experience and training to produce accurate design analysis and code interpretation.

- Re-evaluate piping and pipe support analysis with closely spaced modes requirement.

- Validate Report Writer modules added to ME-101.

- Re-evaluate fire protection system, seismic qualification of deluge valves, sprinkler heads fusible links, integrity of mechanical couplings, etc., to address interaction criteria.

- Implement procedures that ensure BNI seismic qualification requirements tier down to engineering specifications, vendor requirements, and vendor submittals.

- Sample vendor packages showed significant conservatism was introduced due to lack of understanding of the seismic qualification requirements or procedures (i.e., extremely conservative simplified methods were used). BNI should evaluate vendor submittals for conservatism and its impact on cost and maintainability.

- Equipment and other items located in areas inaccessible after startup should be classified as SC-II as a minimum to preclude the need for inspection if an earthquake occurs and re-starting of the facility is desired. An example is the black cells.

- Provide internal seminars or workshops by knowledgeable BNI engineers or consultants from the CS&A organization describing the seismic analysis, design, and qualification procedures for WTP for engineers responsible for SSCs.

- A review of the tailoring of IEEE 344 should be performed to verify that the requirements function during and after the earthquake for SDC/SDS/SC/SS equipment is necessary. This requirement could be unnecessarily conservative and unachievable.

SECTION 7.0 - FINDINGS AND RECOMMENDATIONS

This section provides a summary of findings and recommendations reached to date. Areas of conservatism, areas of potential unconservatism, and recommendations to reduce design-related risk are also summarized. They are based on the material provided, discussions, and meetings with BNI, DOE-ORP, and their consultants, as well as, the responses received to written comments. Comment resolution is ongoing and scheduled to be completed by the end of 2006. The current status of the project, with the incorporation of the RGM, is in the early stages of implementation with a limited number of calculations and documents available for review. The recommendations contained herein are offered to assist in the completion of ongoing calculations and future work.

Some of the recommendations are already being addressed and implemented by DOE-ORP and BNI. As an example, DOE-ORP started a deep boring program to augment subsurface characterization and verify the RGM conservatism.

7.1 Design Process

BNI's design process, including implementation of a graded approach, division of responsibilities, and the assessment of clearances and interfaces is well conceived with strong reliance on an Integrated Safety Management (ISM) program. The issue of dimensional clearances and seismic spatial interactions is being addressed through the use of a comprehensive three-dimensional (3-D) plant model. Additional procedures are being implemented to further assure that these interactions are avoided. These additional procedures should be reviewed upon completion. There are procedures in place defining roles and responsibilities for seismic considerations of the organizations that flow between disciplines including the ISM team, Civil, Structural, and Architectural, C&I, and Plant Design. Due to the limited maturity of the design, a "vertical slice" evaluation of BNI's design process could not be performed. The IR Team recommeded that a "vertical slice" evaluation be performed on a regular basis to assure the ongoing integration of the design between disciplines.

7.2 Revised Ground Motion Spectrum and Soil Properties

DOE-ORP has developed the RGM spectrum to address DNFSB concerns on the site-specific soil characterization and attenuation. In an effort to bound uncertainties in the soil characterization and attenuation models, the RGM spectrum is based on using 85th percentile frequency dependent amplification. It is recognized by DOE-ORP that there are some uncertainties with the soil characterization of the deeper interbeds, which are being addressed by a deep bore drilling program, scheduled to be completed in 2007. As part of this effort, the IR Team recommends reconciliation of the narrow-banded RGM response spectrum shape and lack of amplified frequency content and an independent peer review of the PSHA as well as the soil characterization and dynamic properties developed. This should be performed in accordance with the standard of practice for the nuclear industry.

7.3 Soil-Structure Interaction Analysis with SASSI

The seismic analysis of the PT and HLW buildings was performed using SSI analysis with SASSI. In terms of overall structural response, resulting soil pressures, and development of ISRS, the majority of reviewers conlcuded that the SSI analysis approach meets current state of practice for nuclear-related structures. In addition, the SSI analysis adequately captures the global seismic response of the PT and HLW buildings consisting of in-plane accelerations of walls, slabs, and ISRS from those in-plane accelerations. Local seismic response in terms of out-of-plane accelerations and corresponding ISRS for concrete floor slabs and some steel members is captured in SASSI through the use of vertical SDOF oscillators or lollipops. The IR Team recommended that verification analyses be performed to assure accurate vertical response of the oscillators is computed from the large, complex SASSI model. BNI has initiated a sensitivity study to validate the oscillator response.

7.4 Facility Design Margins, Areas of Conservatism, and Areas of Potential Unconservatism

It is extremely difficult to assess at this time the adequacy of the facility design margins. Although the SSI analysis and two-stage equivalent static analysis is complete, the design based on the RGM is limited. The IR Team assessment of the seismic design margin, areas of conservatism, and areas of potential unconservatism are summarized in the following sections.

7.4.1 Structural Seismic Design Margin

The seismic demand for the primary structural concrete and steel superstructure is performed in a two-step process. For concrete design based on the RGM, only BNI's concrete methodology spreadsheet with sample shear wall calculations were available. Other concrete shear wall and diaphragm (slabs) calculations based on original ground motion and ISC were reviewed to gain a better understanding of BNI's design methodogy, code compliance and design margins. Although the RGM increased loads and some redistribution of loading will occur with the introduction of cracked section properties, concrete design margins should be adequate for existing shear walls and floor diaphragms. It is recommended that force concentrations or "hard points" be carefully reviewed and addressed for the existing concrete. This is based on the low D/C ratios of the existing shear walls and floor diaphragms designed with the original ground motion, the ability to use Fμ due to revised project design criteria, and the reduction of conservatism used in the original design analysis.

7.4.2 System and Component Design Margin

A consensus on compliance and design margins of systems and components cannot be made due to the limited number of calculations available; the nature of individual calculation methodologies; variation in complexity of component design, errors, and omissions; and in-progress design issues between vendors and BNI. Areas of focused concern have been captured in the reviewer comments. The observed small margins in vessel nozzle loads, missing hydrodynamic mass from vessel internal components, and the effects of un-conservative use of SRSS combinations in pipe stress analysis are examples of issues that are in the process of being resolved and have direct bearing on design margins.

7.4.3 Areas of Conservatism

- Scale Factor - Conservatism, approximately 5 to 10 percent, is introduced by not including the SF in the project design criteria as permitted by DOE-STD-1020. The SF is applied to the seismic demand.

- Inelastic Behavior - For SSCs classified as SC-I, the use of Fµ is permitted for existing structures. For SSCs that are ductile, the use of the Fµ should be considered as recognized by DOE and industry standards (comment SM-9, appendix C.1).

- Inertial Loads - The seismic demand for the primary structural concrete and steel superstructure is evaluated in a two-step process using equivalent static loads with a detailed finite element model. Based on tributary areas of the building structure, the maximum response accelerations from the SSI analysis are used to establish equivalent static seismic loads that are applied to a highly detailed finite element model. A comparison of base shears and story shears from SASSI generated accelerations and for selected accelerations used for inertia loads indicate the equivalent static model results are 25 to 30 percent conservative. Validation analyses comparing dynamic analysis results with response from the in-phase static inertial loads demonstrates that the second step static analysis introduces further conservatism.

- Vessel Anchors - Seismic loads on the anchors and ring beams are evaluated in a conservative manner in the determination of the impulsive weight for full tanks and the impulsive and convective (sloshing) weights and their height of application for partially full tanks. In addition, for the vertical cylindrical vessels, the seismic overturning moment and shear is determined by 100-40-40 direction component rule and vector sum of perpendicular components, which is appropriate for shear but conservative for tensile forces due to moment.

- Vessel Seismic Loads - For many of the vessel seismic evaluations, damping has been considered in an unnecessarily conservative manner in that 0.5 percent damping has been used for all horizontal modes and 2 percent damping has been used for all vertical modes. The analysis should use a horizontal response spectrum of 0.5 percent up to about 0.5 Hz and 2 percent above that because the sloshing frequency is typically less than 0.5 Hz. This could be done with a single response spectrum that should significantly decrease the seismic stresses in the vessel and skirt.

- Steel Members - In a number of calculations, steel members were to be qualified using the nuclear structures steel code, AISC N690. Implementation of AISC N690 appeared to be difficult for many vendors, as the calculations were somewhat confusing and, in several instances, conservative.

- Equipment Qualification - Typical equipment seismic qualification by vendors often demonstrated that simplicity and, consequently, conservatism dominated the seismic qualification.

- Functionality - Tailoring of IEEE 344, which states that, the requirements function during and after the earthquake for SDC/SDS/SC/SS equipment is necessary. This requirement could be unnecessarily conservative and unachievable.

7.4.4 Areas of Potential Unconservatism

- The lack of amplified frequency content in the RGM response spectrum shape in the frequency range of 2 Hz and less could be unconservative as evidenced after completion of the pending PSHA.

- ISRS calculated for areas of local amplification within the PT and HLW buildings that may be affected by cut-off frequencies used in SASSI.

- Multi–Commodity Support Beams: Several areas of potential unconservatism were noted. First, loads on the beams were estimated on a tributary area basis, appropriate for dead loads, but may be unconservative for seismic loads. Second, pipe friction loads were considered in the static load cases but not in the seismic load cases. Third, weak axis bending for static loads is based on the upper half of the beam section, appropriate for dealing with eccentric loads, while for seismic loads it is based on the full section. Finally, the 100-40-40 rule for combining directional responses was used incorrectly and un-conservatively.

- Crane Runways - Calculations in the appendixes used accelerations in the longitudinal and vertical directions at the crane frequency, which is below the peak frequency, rather than peak accelerations. Second, the crane runway frequency calculation did not include the vertical offset of the crane mass, which would reduce the calculated lateral frequency, possibly to less than the 33 Hz required by the design criteria.

- Vessel Fluid-Structure Interaction - The vessel vendors have not properly evaluated the fluid-structure interaction effects for cases where the vessel contains internal assemblies. The vendors did not account for the added mass effect for the submerged internal assemblies. Methods for accounting for this effect are discussed in ASCE 4-98, paragraph 3.1.6.2. The added hydrodynamic mass would increase the reactions in the supports.

- Pipe Stress Calculations - A programmatic error in the pipe stress calculations exists. Piping stress calculations reviewed do not call out a modal combination method in BNI’s piping stress program ME-101 input; thus the default for modal combinations is used, which is the SRSS method of modal combination. Closely spaced modes are not considered and the 10 percent grouping method is not used as required by Design Criteria 24590-WTP-DC-01-001 and ASCE 4. This is being addressed by BNI's Corrective Action Report Program.

- Seismic Anchor Motion - The lack of equipment specification requirements of seismic anchor motion for the design and qualification of components embedded in structural elements, such as hatches, penetrations, and systems that span multiple levels and locations within the buildings.

- Post-Earthquake Evaluation - From an ongoing mission standpoint, the installation of non-SC-I or SC-II SSCs in inaccessible areas, such as black cells, is not recommended given that entry and inspection is not possible after an earthquake of any size occurs.

7.5 Recommendations to Reduce Design Related Risks

The following is a listing of recommendations to reduce design related risk associated with the implementation of revised seismic design criteria:

- Evaluate the response spectrum and dynamic soil properties based on the deep borehole program.

- Complete sensitivity study to validate that the ISRS for areas of local amplification (e.g., "lollipops") is accurately calculated from the large complex SASSI model of the HLW building.

- Revise Material Requisitions and calculations for seismic anchor motion.

- Evaluate the need for drag struts and load collectors in PT building.

- Include provisions for installation of seismic instrumentation within PT and HLW buildings to capture strong motion shaking data to aid in post-earthquake evaluation.

- Design procedures should include provisions to verify that structural modeling characteristics are appropriate for the load distributions transmitted from the SAP2000 model.

- Evaluate multi-commodity support beams based on actual commodity loads, seismic loads, and 100-40-40 rule load combinations.

- Provide internal seminars or workshops describing the seismic analysis, design, and qualification procedures for WTP for engineers responsible for SSCs.

- Re-perform analysis and design of vessels to include hydrodynamic mass of internal components and accurately apply load combinations.

- Ensure vendors performing vessel analysis and design have sufficient experience and training to produce accurate design analysis and code interpretation.

- Re-evaluate piping and pipe support analysis with closely spaced modes requirement.
- Validate Report Writer modules added to ME-101.
- Use seismic PRA or SMA to control impact to the project design from future hazard increases instead of redesigning to the higher demands.
- Expand 3-D interference model to consider all equipment and components.
- Perform a vertical slice review on a scheduled basis to ensure proper design integration.
- Consider lessons learned on steel design and code improvements (AISC Seismic Provisions) based on past strong motion earthquakes such as Northridge and Kobe.
- Reevaluate fire protection system, seismic qualification of deluge valves, sprinkler heads fusible links, integrity of mechanical couplings, etc., to address interaction criteria.
- While evaluating seismic interaction, focus on the integrity of source rather than protecting the target.
- Account for local structural response for SSC design and qualification to include the dynamic amplification of the local supporting structure.
- Increase BNI oversight of seismic qualification of equipment by vendors.

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SECTION 9.0 - ACRONYMS AND ABBREVIATIONS

ACI	American Concrete Institute
AFB	Air Force Base
AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
BNI	Bechtel National Incorporated
C&I	control and instrumentation
CDF	Core Damage Frequency
CERL	Construction Engineering Research Laboratory
CLASSI	Continuum Linear Analysis for Soil Structure Interaction
CS&A	Civil, Structural and Architectural
DBE	design basis earthquake
D/C	demand to capacity
DF	design factor
DNFSB	Defense Nuclear Facility Safety Board
DOE	Department of Energy
dt	time step
EPRI	Electric Power Research Institute
ERDC	Engineering Research and Development Center
ESM	Engineering Standards Manual
Fµ	inelastic energy absorption factor
g	acceleration of gravity
GT/STRUDL	computer code
HCLPF	High Confidence of Low Probability of Failure
HLW	high-level waste
Hz	Hertz
IAEA	International Atomic Energy Agency
IEEE	International Electrical and Electronics Engineers
IR	independent review
ISC	interim seismic criteria
ISM	Integrated Safety Management
ISRS	in-structure response spectrum
kips	1,000 pounds of force
LANL	Los Alamos National Laboratory
LAW	low-activity waste
LB	lower bound
LERF	Large Early Release Frequency
M	mean
ME-101	computer code
NATO	North Atlantic Treaty Organization
NPH	natural phenomena hazard
NQA	Nuclear Quality Assurance
NRC	Nuclear Regulatory Commission

ORNL	Oak Ridge National Laboratory
ORP	Office of River Protection
PC	Performance Category
PF	Plutonium Facility
PG&E	Pacific Gas and Electric
PJMs	pulse jet mixers
PNNL	Pacific Northwest National Laboratory
PRA	probabilistic risk assessment
PSHA	Probabilistic Seismic Hazard Assessment
PT	pretreatment
QA	quality assurance
RGM	revised ground motion
RLE	Review Level Earthquake
SADC	seismic analysis and design criteria
SAP2000	Seismic Analysis Program
SASSI	System for Analysis of Soil-Structure Interaction
SC	Seismic Category
SC	Safety Class
SDC	Safety Design Class
SDOF	single degree of freedom
SDS	Safety Design Significant
SF	scale factor
SMA	seismic margin assessment
SRD	Safety Requirements Document
SRSS	square root of the sum of the squares
SS	Safety Significant
SSC	structures, systems and components
SSE	Safe Shutdown Earthquake
SSI	soil-structure interaction
SSR	Structural Summary Report
STERI	seismic evaluation of replacement items
UB	upper bound
UFC	Unified Facilities Criteria
USACE	United States Army Corps of Engineers
WTP	Waste Treatment and Immobilization Plant
ZPPR	Zero Power Physics Reactor
3-D	three-dimensional

PROGRESS REPORT

Independent Review of Implementation of Revised Seismic Design Criteria
Hanford Waste Treatment and Immobilization Plant (WTP)

Revision D

Appendix A

Biographic Summaries of Independent Review Team Members

Paul D. Baughman, P.E.

Education

M.B.A., Northeastern University, Boston, MA
M.S., Civil Engineering, Northeastern University, Boston, MA
B.S., Civil Engineering, Northeastern University, Boston, MA

Professional Registrations

Professional Engineer, State of Massachusetts
Structural Engineer, State of Massachusetts

Professional Background

Mr. Baughman has 35 years of professional engineering and project management experience with industrial and power plant structures, systems, and equipment. He has held a variety of positions encompassing structural and mechanical design, safety and risk evaluations, and regulatory interface. He has actively participated in research into the effects of earthquakes on structures, piping, and equipment. Mr. Baughman is a subject matter expert for the Electric Power Research Institute (EPRI) Seismic Qualification Utility Group (SQUG), which sponsors the application of earthquake experience data for seismic verification of nuclear power plant piping and equipment. He has also participated in post-earthquake reconnaissance at several earthquake sites, and in development of the electronic earthquake experience database. He has acted as project manager for seismic probabilistic risk assessments (PRA) and seismic margin assessments at Indian Point 2, Three Mile Island, Oyster Creek, Calvert Cliffs, Pickering A and Bruce A nuclear power plants, and was a peer reviewer for the Oconee, Vermont Yankee and Maine Yankee plants. International projects have included seismic margin reviews of Pickering A and Bruce A in Canada, Kozloduy in Bulgaria, Paks in Hungary and Bohunice in Slovakia.

Piping assessments have included seismic analyses of reactor primary coolant piping, vessels and components at Pickering A, Bruce A, TMI Unit 2 and Seabrook, and well over a dozen other major piping assessment projects. Additionally, he has performed mechanical equipment seismic evaluations for Boston Edison, Maine Yankee, Public Service of New Hampshire, Consolidated Edison, Gulf States Utilities, Rochester Gas and Electric, Southern Electric International, Virginia Power, Ontario Hydro, Public Service Electric and Gas, and GPU Nuclear; electrical equipment seismic evaluations for Vermont Yankee, Boston Edison, Maine Yankee, GPU Nuclear, Philadelphia Electric, Virginia Power, Rochester Gas and Electric, and Consolidated Edison; and piping seismic evaluations for Vermont Yankee, Tennessee Valley Authority, Ontario Hydro, Princeton Plasma Physics Laboratory, Westinghouse Savannah River, Rochester Gas and Electric, Public Service Electric and Gas, American Electric Power, Northeast Utilities, and Mesquite Lake Resource Recovery Center.

Mr. Baughman has performed seismic verifications of cable tray, conduit, instrument tubing, and ductwork for Southern Nuclear, Princeton Plasma Physics Laboratory, Tennessee Valley Authority, Public Service of New Hampshire, Consolidated Edison, GPU Nuclear, and Rochester Gas and Electric. Lastly, he has prepared procedures for seismic evaluation of replacement items (STERI) for Bruce Power, Maine Yankee, GPU Nuclear and Virginia Power, and presented training in STERI at Virginia Power, GPU Nuclear and Rochester Gas and Electric. He is a SQUD trainer for the SQUD New and Replacement Equipment course.

William G. Bolte, E.I.T.

Education

M.S., Structural Engineering, University of Missouri - Rolla
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Professional Registrations

Engineer in Training, State of Missouri

Professional Background

Mr. Bolte is presently a junior structural engineer with the U.S. Army Corps of Engineers (USACE), Kansas City District. He is responsible for the preparation of the designs and technical drawings for structural elements of military and civil projects. Additionally, he is responsible for the review of shop drawings and the approval of submittals for work-by-others to ensure compliance to criteria. Mr. Bolte's project work includes: analyzing existing hydraulic structures (pump plants, gatewells, floodwalls, outlet structures, and box culverts) for the Kansas City and Topeka Levees; structural design on the Fort Riley Fiscal Year 2005 Barracks Complex; and structural project engineer for Hillsdale and Melvern Dams in Kansas.

Robert D. Campbell, P.E.

Education

B.S., Mechanical Engineering, University of Washington
M.S., Mechanical Engineering, University of Southern California

Professional Registrations

Mechanical Engineer, State of California
Professional Engineer, State of Colorado
Professional Engineer, State of Tennessee

Professional Background

Mr. Campbell has over 40 years experience in engineering design, analysis, risk assessment, and management. His expertise focuses on mechanical systems of nuclear power reactors, hazardous chemical facilities, high-temperature liquid metal facilities, and propulsion systems. Since 1980, his emphasis has been on probabilistic risk assessment of nuclear power plant equipment and subsystems. As part of this work, he has been involved in the detailed walkdowns and evaluations of over 50 nuclear power stations in 20 countries. He was a principal participant in the development and application of seismic margin criteria in an EPRI-sponsored research program and has participated in the development of methodology for seismic probabilistic risk assessment and its application to over 30 nuclear power stations. He was the principal author of the EPRI "Fragility Applications Guide" (EPRI 1002988).

He has supported the International Atomic Energy Agency (IAEA) for many years in developing criteria for seismic assessment of Soviet designed reactors and has participated in several IAEA training programs to train engineers in the former Soviet block countries in seismic evaluation of existing nuclear power plants.

Recent U.S. Department of Energy (DOE) projects include a top-level review of seismic evaluation of piping and equipment in the DOE High Flux Isotope Reactor at Oak Ridge National Laboratory (ORNL) and support to Argonne National Laboratory to provide training in seismic evaluation and review of seismic assessments of the Armenia Nuclear Power Plant.

Select Publications

August 1987. "Walkdown Criteria for Evaluating Seismic Margin in Nuclear Power Plants." with R.P. Kennedy and R.P. Kassawara. Paper K11-2 presented at Structural Mechanics in Reactor Technology Conference.

With R. P. Kennedy and R. P. Kassawara. August 1987. "An Overview of Seismic Margin Assessments for Nuclear Power Plants." Paper M11-1 presented at Structural Mechanics in Reactor Technology Conference.

With R. H. Sues and P. J. Amico. August 1987. "Contributions of Earthquake Initiating Events to Nuclear Power Plant Public Risk." Paper M2-2 presented at Structural Mechanics in Reactor Technology Conference.

John Connor, P.E.

Education

B.S., Civil Engineering, Iowa State University

Professional Registrations

Professional Engineer, State of Kansas

Professional Background

Mr. Connor is a senior structural engineer with the USACE, Kansas City District. He has over 11 years of structural design experience with vertical construction for military projects. He has designed for a variety of buildings including multi-story barracks, vehicle maintenance facilities, aircraft hangars, munitions igloos, and range support facilities. His design projects include buildings of various structural systems and material types such as steel frames, concrete and masonry shear walls, and wood and cold-formed steel construction. His experience with bridges includes inspections, load ratings, and maintenance and repairs.

Mr. Connor also facilitates and instructs technical seminars including “Special Inspection of Seismic Resisting Systems” and “Seismic Design for A/M/E Components.”

Professional Affiliations

American Society of Civil Engineers
American Institute of Steel Construction
Society of American Military Engineers
Structural Engineers Association of Kansas and Missouri

Philip S. Hashimoto, P.E.

Education

M.S., Civil Engineering, University of California - Berkeley
B.S., Civil Engineering with Highest Honors, University of California – Berkeley

Professional Registrations

Civil Engineer, State of California
Civil Engineer, State of Idaho

Professional Background

Mr. Hashimoto has over 25 years experience in civil/structural and earthquake engineering, with proven capabilities in project management and business development. He has directed project staff in a diversity of technical areas, including structural/seismic analysis and design, construction support and management, equipment seismic qualification, risk assessment, and computer software development for client including DOE, Nuclear Regulatory Commission (NRC), commercial nuclear utilities, and commercial interests.

Mr. Hashimoto directed and executed the seismic evaluation of the High Flux Isotope Reactor located at the DOE ORNL for current Performance Category (PC)-3 seismic criteria specified by DOE-STD-1020-2002. Mr. Hashimoto performed a seismic/structural scoping study for the Zero Power Physics Reactor (ZPPR) located at the Idaho National Laboratory. In support of a decision to re-start the ZPPR, the objective of this study was to provide opinions on whether the facility could be demonstrated to comply with current DOE seismic criteria. In addition, Mr. Hashimoto has managed projects at several DOE sites; services included structural/seismic analysis and evaluation, soil-structure interaction (SSI) analysis, strengthening design, program and criteria development, and peer review.

Select Publications

With S.P. Harris and R.L. Stover, “Seismic High Wind, Tornado, and Probabilistic Risk Assessments – The High Flux Isotope Reactor, Oak Ridge National Laboratory”, presented at the Tenth Conference on Structural Mechanics in Reactor Technology, Los Angeles, California, August 1989.

With P.G. Prassinos, C.Y. Kimura, D.B. McCallen, R.C. Murray, Lawrence Livermore National Laboratory, and M.K. Ravindra, R.D. Campbell, A.M. Nafday, W.H. Tong, EQE Engineering, Inc., “Seismic Failure and Cask Drop Analyses of the Spent Fuel Pools at Two Representative Nuclear Power Plants”, prepared for Division of Safety Issue Resolution, Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission, Washington, D.C., January 1989.

With H.T. Tang and L.W. Tiong, “Earthquake Experience Data Relevant to Nuclear Plant Vertical Storage Tanks”, presented at the Second Symposium on Current Issues Related to Nuclear Power Plant Structures, Equipment and Piping with Emphasis on Resolution of Seismic Issues in Low Seismicity Regions”, Orlando, Florida, December 1988.

James J. Johnson, Ph.D., P.E.

Education

B.C.E., Civil Engineering, University of Minnesota
M.S., Civil Engineering, University of Illinois
Ph.D., Civil Engineering, University of Illinois

Professional Registrations

Civil Engineer, State of Alabama
Civil Engineer, State of California

Professional Background

Dr. Johnson has more than 30 years of experience in risk analysis for natural and man-made hazards. His expertise includes the development, implementation, and teaching of seismic risk and seismic margin assessment methodologies. He has participated in seismic PRAs of over 20 nuclear power plants as well as the development and application of best estimate or median-centered response procedures to over 60 nuclear facilities. He was responsible for several portions of the U.S. NRC Seismic Safety Margins Research Program, and he participated in the development of the US Utilities Requirements Document for U.S. utility specifications for new nuclear power plant design. Currently, Dr. Johnson participates in two research projects concerning U.S. DOE and nuclear power plant seismic design and evaluation process. He has played a significant role in the development of general and plant-specific seismic evaluation procedures to include criteria for assessing equipment and component functionality and structural integrity, seismic systems interaction, anchorage, and other issues.

Dr. Johnson has extensive theoretical and practical experience in the SSI analysis of major facilities and has written a comprehensive assessment of the state-of-the-art of SSI. Most recently, Dr. Johnson authored “Soil-Structure Interaction,” Chap. 10, and co-authored “Loss Estimation,” Chap. 30, Earthquake Engineering Handbook, 2003. He lectured for the North Atlantic Treaty Organization (NATO) Advanced Study Institute on Developments in Dynamic SSI, and he was principal investigator for EQE on the SSI modeling, predictive analysis, and resolution of measured and predicted response for the combined EPRI/NRC Lotung, Taiwan scale model project. He has performed SSI analyses of a wide variety of surface and embedded structures using simplified to sophisticated substructure methods and linear and nonlinear finite element techniques. Dr. Johnson was a consultant to the U.S. NRC concerning revisions to the Standard Review Plan for seismic analysis and design.

Professional Affiliations

American Society of Civil Engineers
- Dynamic Analysis Committee
- Committee on Nuclear Standards, Seismic Analysis of Safety Class Structures, Author of ASCE 4-98, “Seismic Analysis of Safety-Related Nuclear Structures And Commentary.”

Earthquake Engineering Research Institute

Larry T. Nicholson

Education

B.S., Mechanical Engineering, University of California, Berkeley

Professional Background

Mr. Nicholson has 29 years of engineering and group leader experience in the power industry and 1 year in aerospace design and engineering. He is presently a consultant with ARES Corporation in its Structural Mechanics Group. His responsibilities include design and evaluation of piping, pipe supports, and equipment for conventional and nuclear power plants.

Previously, Mr. Nicholson was a Project Engineer with Altran Corporation where he performed static and dynamic analysis on piping in accordance with American Society of Mechanical Engineers (ASME) Section III and American National Standards Institute (ANSI) B31.1 for Wolf Creek, DC Cook, and Diablo Canyon nuclear power plants. He used ME-101 and PDSTRUDL (computer code) for these analyses. He performed quality assurance on programs used for structural analysis (ANSYS, SAP2000, PDSTRUDL). Mr. Nicholson generated spectra for non-load bearing walls at Diablo Canyon nuclear power plants. He worked on drop analysis of canisters of vitrified glass used for storage and transport at the Hanford site.

While with Bechtel Power Corporation, Mr. Nicholson had 13 years seconded to Pacific Gas & Electric (PG&E) in the Diablo Canyon piping group. His primary responsibilities were spectra generation, perform static and dynamic analysis on piping in accordance with ASME Section III and ANSI B31.1, assisted PG&E in the use of Bechtel piping programs. He wrote, documented, and maintained programs for use on PG&E computers. He also recommended computer hardware and software to be purchased by PG&E. He setup and maintained computer software and hardware. He was the system administrator for the piping group's IBM RS6000 mini computer.

Mr. Nicholson spent 4 years in Bechtel and PG&E integrated design and construction project for Diablo Canyon Nuclear Power Plant. He worked as an Assistant Piping Stress Group Leader for 1 year. He worked as Subgroup Leader in charge of spectra generation for 3 years. Mr. Nicholson set up programs on Bechtel computers to combine spectra per the Diablo Canyon criteria. He also trained stress engineers to use Bechtel piping programs.

Also during his time at Bechtel, Mr. Nicholson spent 7 years in the Susquehanna Nuclear Power Plant piping stress group, 5 years as Assistant Group Leader, 1 year as group leader. He was responsible for all types of piping stress analysis, static and dynamic per ASME Section III and ANSI B31.1. He was involved in containment LOCA and Main Steam relief valve hydrodynamic loading analysis. He was responsible for setting schedules, estimating manpower requirements, and man-hours for new work.

Lastly, while with United Aircraft, Mr. Nicholson designed solid rocket motor hardware, which included specifying materials, dimensions, design processes, and supervised creation of design drawings. He reviewed designs for feasibility of manufacture, ease of assembly, inspection and maintenance, and weight and size reduction.

John S. North, P.E., G.E.

Education

M.S., Geotechnical Engineering, University of California, Berkeley
B.S., Civil Engineering, University of California, Berkeley

Professional Registrations

Civil Engineer, State of New Mexico
Civil Engineer, State of California
Geotechnical Engineer, State of California

Professional Background

Mr. North is a Senior Principal Professional for Kleinfelder New Mexico operations with 20 years technical and project management experience in geotechnical consulting. Mr. North has design and construction experience with state and Federal highway projects, tunneling projects, dam evaluation and rehabilitation, nuclear facilities, slope stability analysis and landslide mitigation, earthquake engineering, commercial developments, water and wastewater treatment facilities, transfer stations and landfill closures, and urban and waterfront development.

Mr. North currently serves as the project manager for the geotechnical/seismic investigation for the Chemistry and Metallurgical Research Replacement project at Los Alamos National Laboratory (LANL). This new PC-3 facility will support the adjacent Plutonium Facility (PF) #4 in the overall mission associated with Stockpile Stewardship and certification of the stockpile. Mr. North has managed numerous geotechnical engineering projects at LANL including the TA-18 Relocation Project, the Advanced Hydrotest Facility project, and the TA-50 Pump House and Influent Tanks project.

Mr. North's technical experience includes deep foundation design including driven end-bearing and friction piles; mini-piles and drilled caissons; excavation support; slope stability analysis and slope stabilization; seismic analysis and earthquake engineering; excavation support systems and groundwater control; SSI analysis; slurry walls; tension foundations; ground improvement methodologies; tunneling and trenchless technologies; seepage analysis and modeling; and geotechnical instrumentation design. He also has extensive experience with shallow foundation systems including spread-type footings and slabs-on-grade, post tensioned slabs, mats, and structural slabs and rafts.

Stephen A. Short, P.E.

Education

M.S., Structural Engineering, University of California - Berkeley
B.S., Civil Engineering - Berkeley

Professional Registrations

Civil Engineer, State of California

Professional Background

Mr. Short has 37 years of professional experience in conceptual design, response evaluation, and criteria development for a variety of structures and equipment for extreme loads in support of the energy and defense industries. Primary areas of expertise are in structural engineering and structural dynamics, which have been applied in diverse engineering projects involving; 1) engineering for natural phenomena hazards 2) evaluation of structures and equipment subjected to extreme loads; and 3) blast resistant design and evaluation. Mr. Short participated with a technical committee at LANL for the preparation the 2004 edition of the structural chapter of the LANL Engineering Standards Manual (ESM). He is currently developing training materials for the structural chapter of the LANL ESM for a one-half day overview course intended for program managers, safety analysts, and other interested parties and a full day detailed course intended for structural engineers.

Mr. Short participated on an ASCE working group preparing the Standard ASCE 43-05, "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities and Commentary." This standard is expected to eventually replace DOE-STD-1020 for DOE facilities as well as to provide seismic design and evaluation criteria for other nuclear facilities, such as fuel fabrication and processing facilities. Previously, he participated on the ASCE working group that prepared ASCE Standard 4-98, "Seismic Analysis of Safety-Related Nuclear Structures and Commentary." He is currently leading a group focusing on seismic analysis methods for the update to ASCE 4. Mr. Short conducted a seismic/structural evaluation of a massive concrete canyon building located at Hanford and to be used for nuclear waste storage. He performed dynamic analyses of this building for earthquake loads accounting for significant SSI for this massive concrete structure embedded in relatively soft soil. Mr. Short has conducted seismic analyses of massive concrete process cells on a soil site at the Pantex Plant using the computer program, System for Analysis of Soil-Structure Interaction (SASSI). He has conducted soil-structural-interaction analyses of heavy reinforced concrete cells at ORNL using the computer program, Continuum Linear Analysis for Soil Structure Interaction (CLASSI).

Select Publications

Mr. Short has authored and co-authored many publications on seismic design criteria, seismic evaluation of nuclear and lifeline structures, and blast resistant design.

Siv Sivakumar, P.E.

Education

B.S., Civil Engineering (2nd Class honours, 1st Division), University of Westminister, London, England.
M.S., Civil Engineering - Continuing program at University of Kansas, Edward Campus

Professional Registrations

Associate Member of the Institution of Civil Engineers, England
Professional Engineer, Alberta, Canada
Professional Engineer, State of Kansas

Professional Background

As Chief, Structural Section, Design Branch, USACE, Kansas City District, Mr. Sivakumar presently supervises and directs a team of 12 Structural engineers and two technicians for Military, Civil works, and Operation and maintenance work projects totaling over $150 million for which he develops budgets and schedules for work consisting of 18 dams and 30 bridges. Previously, Mr. Sivakumar was lead structural engineer for the following USACE military construction and Civil works projects: Airmen’s Dormitory (masonry structure) at McConnell Air Force Base (AFB), Central fire station addition (masonry) at McConnell AFB, Three Company fire station (steel frame) at Ft. Leonard Wood, flexible pile cap foundation design for several rolling gates and stoplog gaps using finite element computer program for L-385 flood control project, and designed and detailed gatewell structures.

While at TapanAM Associates, Mr. Sivakumar served as the manager of the Structural Department and lead engineer for the design of several new multi-purpose buildings in structural steel, masonry, concrete, and timber as well as the retrofit design of existing facilities. He was a lead structural engineer for the following military projects: Central Fire Station at McConnell AFB, temporary lodging facility (timber structure) at McConnell AFB, Communication center (masonry) at Whiteman AFB, hazardous storage facility at Fort Riley, expansion of company officer’s mess at Fort Leonardwood, and expansion of hospital entry at Fort Leonardwood. He also successfully managed and completed the following military projects: Consolidated supply facility (200 feet span pre-engineered metal building) at Fort Riley, seismic retrofit design of a three-story concrete flat-slab rolling pin barracks building at Fort Riley, and seismic retrofit design of a historic structure at Fort Riley. He also served as a lead structural engineer for the following non-military projects: Two-story glue-lam beam/timber structure (women’s dormitory) for DOE, three elementary schools, one high school, 100-foot diameter counter poise steel-framed antenna design for the Federal Aviation Administration, 60 feet high steel framed support for three 60-ton flour bins at Panama City, Panama, and numerous retrofit projects for General Services Administration and Environmental Protection Agency. Other duties included project coordination, detail structural analysis, and quality control review during the design phase of the project.

During his tenure at Stone and Webster Canada Limited, Mr. Sivakumar was a structural engineer in design of pipe support structural steel frames, using a mainframe computer for a nuclear power plant in Texas, including checking of local stresses in run pipe at point of welded attachment. He also designed welded and bolted connections and anchor bolt and base plate assemblies.

Mark Summers, P.E.

Education

M.S., Civil Engineering, Oklahoma State University
B.S., Civil Engineering, Walla Walla College, Washington

Professional Registrations

Civil Engineer, State of Washington

Professional Background

Mr. Summers has 27 years of design experience as a structural engineer on hydroelectric projects, fish bypass facilities, fish hatcheries, and military projects. He is presently a senior structural engineer with the USACE, Walla Walla District, State of Washington, working on civil works and military projects. His work projects support the DOE, Puget Sound Naval Shipyard, Bureau of Reclamation, Forest Service, Northwest Division Seismic Center, and the Seattle, Portland, Kansas City, and Omaha Districts. His design experience includes buildings for the Combined Arms Collective Training Facility at Fort Lewis; three-story steel framed office building (170,000 square foot) in Jacksonville, Florida; the McNary screen rehabilitation facility (20,000 square foot); spillway deflectors at Ice Harbor Dam; sheet pile cutoff walls for the Wyckoff/Eagle Harbor (1,900 linear feet) and Milltown Superfund (6,800 linear feet) sites; dock facility at the Port of Benton for unloading 2,500 ton reactor core packages; and seismic upgrade of Lucky Peak intake tower service bridge. Investigations include seismic evaluations of powerhouses and high-risk buildings; hydraulic steel structures for serviceability and fracture critical members; and fatigue life of Lower Monumental navigation lock gate. Reviews include the Hanford Vitrification Plant ($800 million/per year project); Tuttle Creek intake tower; and Lower Monumental spillway deflectors and stilling basin repairs. His designs efforts used simple hand analysis, detailed structural models, finite element modeling, response spectrum analysis, and current building criteria and codes. He employed creative and innovate designs such as shear friction attachment of spillway flow deflectors, stabilizing guide wall extension with post tensioning system, and using post-tensioned supports for the Milltown sheet pile wall.

Mr. Summers' past assignments include Chief of the Structural Design Section in which he performed technical and administrative supervision of structural engineers, architects, and technicians (16 personnel). Significant projects designed by the section included the new navigation lock lift gate for Ice Harbor and the Lower Granite floating fish collector. Additionally, he served as Technical Manager for the Columbia River Juvenile Fish Mitigation Program ($30 to $40 million annual design and construction program). The main projects were the McNary, Ice Harbor, and Lower Monumental fish facilities. He managed design teams producing design memorandums, contract drawings and specifications, and providing engineering during construction. He developed schedules and design costs, coordinated the design with the user, monitored progress, resolved conflicts, and conducted review meetings. He also served as a project engineer for the design of the McNary Juvenile Fish Facility ($16 million design and construction budget), and as a structural engineer, he designed the office/laboratory buildings for the McNary, Little Goose and Lower Granite Juvenile Fish Facilities and the 1,600-foot long reinforced concrete fish collection channel and dewatering structures at McNary. He also developed preliminary and final structural designs for civil and military projects. Designs included culvert tainter valves; overflow and non-overflow sections, spillway gates and outlet structure; steel-framed buildings; and masonry buildings and various reinforced concrete structures.

Mansour Tabatabaie, Ph.D., P.E., G.E.

Education

Ph.D., Earthquake Engineering, University of California - Berkeley
M.S., Geotechnical Engineering, University of California - Berkeley
B.S., Civil Engineering, Tehran University, Iran

Professional Registrations

Civil Engineer, State of California
Geotechnical Engineer, State of California

Professional Background

Dr. Tabatabaie has 26 years of diversified experience in geotechnical and earthquake engineering, soil dynamics, seismic hazard and ground motion evaluations, cold-region engineering, numerical modeling, and computer programming. Dr. Tabatabaie is the principal author of numerous computer programs in soil dynamics and earthquake engineering including SASSI, which evaluates the dynamic response of soil structure systems in three dimensions. Dr. Tabatabaie has advanced the substructuring methods used in SASSI to solve large and complex SSI problems such as those involving large pile groups, fluid-structure interaction, structure-soil-structure interaction, general seismic wave fields including inclined propagating body waves and surface waves, incoherent ground motions, dynamic loads, and ambient vibrations. He has also been involved in the development and application of non-linear soil models for large seismic deformation analysis of slopes and excavations.

Select Project Experience

Smart-1 Array ¼ Model Reactor; Lotung, Taiwan

Participated in blind seismic response prediction study for SMART-1 array in ¼ model reactor experimental project. The team's predicted response came within 5 percent of actual recorded motions in the structure, which was rated as the best among all participants.

Savanna River Nuclear Power Plant; Savannah River, Georgia

Participated in 3-D soil structure interaction analyses for seismic re-evaluation of Savannah River Nuclear Power Plant. The work included development of three-dimensional finite element SSI model using computer code SASSI and interpretation of the analysis result and performing Quality Assurance studies in accordance with NRC requirements.

Duke Nuclear Power Plant; Omaha, Nebraska

Participated in 3-D seismic SSI analyses of reactor building supported on large pile group foundation for Duke Nuclear Power Plant. As part of this study, a new procedure was developed and implemented in SASSI code to enable partitioning of the large SSI problem into the superstructure and substructure models that could be analyzed separately, and the results combined to obtain final response.

Eric Walton, E.I.T.

Education

B.S., Civil Engineering, University of Idaho

Professional Registrations

Engineer in Training, State of Washington

Professional Background

Mr. Walton is a civil engineer in the Structural Design Section of the U.S. Army Corp of Engineers, Walla Walla District, in the State of Washington.

His civil work project experience includes:

- Design of concrete stop logs and repair of floating bulkhead for Ice Harbor.
- Emergency repairs of tainter valve and trash shear boom for Lower Granite.
- Participation in a rehabilitation study for Lower Monumental Lock and Dam Navigation Lock System. Work included creating, modifying and interpreting a finite element model of the lock gate as well as evaluation of the fatigue life of the gate and proposal of ways to increase the longevity of the gate. Also, he performed inspections of hydraulic steel structures for serviceability and developed repairs when required.

Mr. Walton's military experience includes working as a designer for a Fort Lewis battalion building and multiple three-story barracks buildings. The design involved sizing members for gravity and seismic loads, including eccentrically braced frames, and timber shear walls.

Mr. Walton's previous work experience also includes inspecting timber bridges for Boise National Forest Service.

Daniel J. Weinacht, Ph.D., P.E.

Education

B.S., Mechanical Engineering, SD School of Mines & Technology, 1984
M.S., Mechanical Engineering, University of Illinois at Urbana-Champaign, 1986
Ph.D., Mechanical Engineering, University of Illinois at Urbana-Champaign, 1992

Professional Registrations

Professional Mechanical Engineer, State of New Mexico

Professional Background

Dr. Weinacht has approximately 18 years of diverse experience as a mechanical engineer. His areas of expertise include mechanical engineering design and analysis for specialty scientific equipment (including pressure vessels and nuclear-certified hardware), seismic analysis, fatigue and fracture analysis, materials behavior, computational mechanics, constitutive model development, project management, and risk management. Dr. Weinacht currently holds the position of Southern California Projects Manager with ARES Corporation. In this role, he provides leadership and technical direction to project personnel assigned to his division. Previous to this, he served in the following capacities:

Lead Manager for ARES Team supporting the National Ignition Facility Project at Lawrence Livermore National Laboratory; Manager of ARES Los Alamos Area Operations during which he managed an office of 35 engineering and project management specialists including 16 project controls (cost and schedule) personnel providing on-site support at LANL for complex projects and programs including Pit Manufacturing and at the LANL TA-55 PF. Additionally, as Manager he provided senior technical leadership and peer review for engineering design activities in the mechanical design, specialty equipment design and fabrication, and seismic evaluation areas in ARES-Los Alamos Office. Dr. Weinacht also evaluated the impact of abnormal mechanical environments on structures, systems and equipment and served as an independent peer reviewer on numerous projects per relevant Department of Defense and DOE standards. He supported ARES hazard, safety, and risk analysis work for the Defense Threat Reduction Agency including the nuclear Weapon System Safety Assessments for the B-2A and the F-15/F-16 Dual Capable Aircraft. Risks of plutonium dispersal were the primary focus of these studies. In particular, the seismic analysis of aircraft shelters and weapon storage facilities was peer reviewed by Dr. Weinacht. Lastly, as Manager he served as the ARES Principal Investigator for the Hazard Analysis for Weapons Storage and Storage Blast Effects Testing and Analysis programs. These studies involved computational and experimental characterization of the hazards posed to U.S. nuclear weapons stored in the U.S. Air Force Weapons Storage Vaults when subjected to conventional munitions blasts; Finally, he served as both a Project Leader and Principal Investigator with LANL on various DOE projects.

Professional Affiliations

American Society of Mechanical Engineers
Society of Automotive Engineers
American Society for Precision Engineering

James Wilcoski, P.E.

Education

B.S., Civil Engineering, University of Illinois at Urbana-Champaign
M.S., Civil Engineering, University of Illinois at Urbana-Champaign

Professional Registrations

Professional Engineer, State of Illinois

Professional Background

Mr. Wilcoski is a research structural engineer with the U.S. Army Engineering Research and Development Center (ERDC), Construction Engineering Research Laboratory (CERL), in Champaign, Illinois. His previous appointments with the ERDC-CERL include: Principal Investigator, Engineering Division; Principal Investigator, Structural Engineering Team; Principal Investigator, Engineering Team; Associate Investigator, Engineering Team; Associate Investigator, Materials Application Team; and Associate Investigator, Engineering & Materials Division. Mr. Wilcoski has been participating in the structural and nonstructural review of the DOE, Hanford Waste Treatment Plant since 2005. His other related activities include:

- Development of equipment support motions for fragility testing of equipment, which will be used in the ATC-58 Protocol Shake Table Testing, for the experimental quantification of the seismic fragilities of architectural, mechanical, electrical and nonstructural building components and systems that are permanently located in a building structure. These motions are narrow-band random sweep motions, created by sweeping high- and low-pass filters across a random signal. The band width and sweep rate are selected to create records that provide comparable levels of equipment excitation as typical far field earthquakes, plus provide a direct correlation between, time of failure (time stamping) and the center frequency of failure, so that frequency dependent fragility envelopes can be developed. The records were scaled to provide a relatively smooth spectral acceleration between 2 and 32 Hertz, and a relatively smooth spectral displacement between 0.5 and 2.0 Hertz. These motions were based on the above referenced U.S. Army CERL TR-97/58.

- Participation in National Science Foundation Sponsored reviews of the Network for Earthquake Engineering Simulation Program equipment reviews as part of the construction management team in 2000 and again in 2002.

- Development of the Performance Test Requirements for the Triaxial Earthquake and Shock Simulator during commissioning and to define the performance envelopes. Oversaw the conduct and documentation of these tests in 1995.

- Conducted sessions on cold-formed steel, wood shear panels and nonstructural components in the USACE Proponent Sponsored Engineer Corps Training, PROSPECT Course, Seismic Design for Buildings, in 2001 through 2004.

- Primary author on the non-structural portions of the Department of Defense, Unified Facilities Criteria (UFC) Seismic Design for Buildings, UFC 3-310-04. This document should be published in 2006.

James Wilcoski, P.E. (continued)

Professional Publications and Honors

Mr. Wilcoski has published extensively on seismic design, including the following most closely related to the proposed project:

Wilcoski, J., Hall, R.L., Matheu, E.E., Gambill, J.B., Chowdhury, M.R., "Seismic Testing of a 1/20 Scale Model of Koyna Dam," ERDC TR-01-17, November 2001.

Wilcoski, J., Gambill, J.B. and Smith, S.J., "The CERL Equipment Fragility and Protection Procedure (CEFAPP)," U.S. Army CERL TR-97/58, March 1997.

Wilcoski, J. and Smith, S.J., "Fragility Testing of a Power Transformer Bushing," U.S. Army CERL TR-97/57, February 1997.

Wilcoski, J. Trovillion, J.C., AN/ALQ-99 TJS Pod Vibration Survey Test Results," ERDC/CERL TR-05-20, September 2005.

Notohardjono, B. D., Wilcoski, J., Gambill, J. B., Design of Earthquake Resistant Server Computer Structures, Journal of Pressure Vessel Technology, ASME, Vol 126, February 2004.

Additionally, Mr. Wilcoski has received several professional honors, including the ASME Pressure Vessels and Piping Division, Outstanding Technical Paper in Seismic Engineering, "Design of Earthquake Resistant Computer Structures," and the ERDC Award for Outstanding Team Effort, Pentagon Rebuild Retrofit Program Study.

Professional Affiliations

American Society of Civil Engineers
Earthquake Engineering Research Institute
American Iron and Steel Institute, Committee of Framing Standards, 1998-present

Thomas D. Wright, P.E.

Education

B.S., Civil Engineering, University of Missouri, Columbia
M.S., Civil Engineering, University of Missouri, Columbia

Professional Registrations

Civil Engineer, State of Washington
Professional Engineer, State of Missouri

Professional Background

As a senior structural engineer, USACE Kansas City District, Mr. Wright provides structural engineering advice, review and criteria development for civil works projects. His projects include Kansas City Levees Feasibility Study, Topeka Levees Feasibility, Turkey Creek Channel, Tuttle Creek Dam Tainter Gate Strengthening, Harlan Co Dam Safety Assurance, ITR for Perry Dam Seismic Safety Review, and numerous periodic dam safety inspections. He developed Unified Facilities Criteria Masonry Design for Buildings for nation-wide use in design of military buildings, and developed the Masonry Design for Buildings training course. He is also a member of Field Advisory Group for updating TI 809-04, Seismic Design of Buildings to Unified Facilities Criteria.

Mr. Wright's past appointments include Chief, Structural Section, Kansas City District, during which he supervised the Structural Section consisting of a staff of 14 structural engineers in structural analysis and design of military and civil works projects. During his tenure, he supervised the structural design of over 50 military buildings including structural steel, reinforced concrete, and masonry construction for barracks, company administration buildings, munitions storage units, family housing, air traffic control facilities, and long-span aircraft hangers. He further supervised analysis and design of structures for levees, channels, and dams including outlet structures, floodwalls, pump plants, gatewells, closure structures, analysis of Tainter gates, and slide gates. Projects included the Brush Creek Channel and river walk, Blue River paved reach, Blue River Dodson Industrial District Levee, Missouri River Levee System L-385, and Tuttle Creek Dam Safety Assurance Study. Lastly, he supervised dam safety inspections, reporting, and Operations and Maintenance design work for 18 Kansas City District dams.

As a structural engineer with the Kansas City District, Mr. Wright served as Project Coordinator for the Harry S Truman Dam and Reservoir, a flood control and hydropower dam project. He coordinated the overall development of plans, real estate requirements, relocation matters, cost estimates and budgetary submissions. Additionally, he served as structural Project Design engineer on Blue Springs and Longview dams. He developed structural layout and design, and developed plans & specifications. Lastly, he developed designs for dam features on several Kansas City District dam projects including, Smithville, Hillsdale, Clinton, Long Branch, and Harry S Truman.

Professional Activities

American Society of Civil Engineers
Structural Engineers Association of Kansas and Missouri
Masonry Standards Joint Committee
Masonry Society

PROGRESS REPORT

Independent Review of Implementation of Revised Seismic Design Criteria
Hanford Waste Treatment and Immobilization Plant (WTP)

Revision D

Appendix B

Documents Reviewed

Appendix B

Documents Reviewed

Topical Area	Document Number and Title
1	• 24590-WTP-DC-ST-04-001, Rev. 3, Seismic Analysis and Design Criteria • WTP Structural Modeling Review, Presentation, Abdul et al., 01/09-10/2006 • 24590-WTP-SRD-ESH-01-001-02, Project Safety Requirements Document, Vol. II • 24590-WTP-RPT-ST-03-001-B, Summary Structural Report, Rev B, HLW Vitrification Building, Volume IIA, Appendix C, Assessment of the Dynamic SSI Model Parameters and Results • WTSC99-1036-42-17 (H1616-51), Final Report Geotechnical Investigation, May 11, 2000, Shannon & Wilson, Inc. • ORP Letter 05-WTP-082, R. J. Schepens to J. P. Henschel, “Contract No. DE-AC27-01RV14136 – Dynamic Soil Properties for the Waste Treatment and Immobilization Plant (WTP),” included in Refs. 1 and 2 • Rohay, A. C. and S. P. Reidel, “Site-Specific Seismic Site Response Model for the Waste Treatment Plant, Hanford, Washington, prepared by the Pacific Northwest National Laboratory for the US Department of Energy, Office of River Protection, under Contract DE-AC05-76RL01830, March 2005 • 24590-WTP-S0C-S15T-00002, Generation of DBE Time Histories, No. 2 • 24590-HLW-S0C-S15T-00001, Rev 2, 1, 0, High Level Waste Vitrification Building: Free Field Analysis • 24590-HLW-S0C-S15T-00006, Rev. D, HLW Verification of Building Seismic Analysis-Structural Model • 24590-HLW-S0C-S15T-00007, High Level Waste Vitrification Building Seismic Analysis: SSI Analysis • 24590-HLW-SOC-S15T-00008, High Level Waste Building Seismic Analysis – Seismic Loads • 24590-HLW-S0C-S15T-00009, In-Structure Response Spectra • 24590-HLW-S0C-S15T-00025, Structural Model with Equipment Seismic Loads • 24590-HLW-S0C-S15T-00091, HLW Facility Structural Analysis with Refined Structural Model • 24590-PTF-S0C-S15T-00001, PTF Seismic Analysis: Free Field Analysis, Rev. 2 • 24590-PTF-S0C-S15T-00002, Structural Model • 24590-PTF-S0C-S15T-00003, Pretreatment Building Seismic Analysis: SSI Analysis • 24590-PTF-SOC-S15T-00004, Pretreatment Building Seismic Analysis –

Topical Area	Document Number and Title
	Seismic Loads • 24590-PTF-S0C-S15T-00005, In-Structure Response Spectra • 24590-PTF-S0C-S15T-00010, Rev.1, Pretreatment Building – Soil Springs • 24590-PTF-S0C-S15T-00046, Rev. 0, Comparison of SAP2000 PT Building Model with GT/STRUDL Model • ENG-DECS-05-066, Consideration of Thermal and Seismic Loads in Reinforced Concrete Buildings
2	• 24590-WTP-RPT-ST-03-001-B, *Summary Structural Report, Revision B*, Volumes I, IIA, IIB • 24590-HLW-DGE-13T-00045 • Drawing Nos. 24590-HLW-P1-P01T-00001 to -00011 • 24590-PTF-SOC-S15T-00047, Pretreatment Facility Structural Analysis with Refined Structural Model • 24590-PTF-P1-P01T-00001 thru 00006 (General Arrangement Plans) • 24590-PTF-DB-S13T-x (Structural Concrete Wall Sections)
3	• 24590-PTF-DDC-S13T-00001, Vessel Ring Embedded Plates • 3DG C13 014, *Revision 3, Engineering Design Guide for Embeds and Surface Mounted Plates*, September 2004 • 24590-PTF-DGCS13T-00012, Design of Walls at Col Lines 25.5, 27, 28.5, 30, B, D, E, H, J, K, L & M from EL 28 to 56 and Dowels up to floor at EL 28' • 24590-PTF-DGC-S13T-00016, Design of Walls at Col Lines 28.5 and 30, and Col Lines B, E, H & L Bounded by Col Lines 24 and 30 from el 56' to 77' • 24590-PTF-DGC-S13T-00040, Excel Spreadsheet Methodology and Example of Shear Wall Analysis, and Excel template "Wall at column line 4, cut 4:B-E el 56'" • 24590-PTF-DGC-S13T-00022, Rev No B, Design of Slab at 56 ft Elevation
4	• Vessels: PWD-VSL-00015 and 00016; 24590-QL-POB-MVA0-0001-06-00027 drawing • 24590-QL-POB-MVA0-0001-03-00005 and 00006, 264" I.D. Acidic/Alkaline Effluent Vessel calculation • Equip Tag #: 24590-PTF-MV-CXP-VSL-00005, PO#: 24590-QL-POA-MVA0-00014, Seismic Analysis of 60" ID C.S. Reagent Storage Vessel • 24590-QL-POA-MVA0-00010-03-01, Rev. 00E, Design Calculations, UFP-VSL-0002A/B • 24590-QL-POA-MVA0-00010-03-00005, Rev. 00E, Nozzle Load Calculations, UFP-VSL-0002A/B • 24590-QL-POA-MVA0-00010-09-00001, Rev. 00A, Seismic Analysis, UFP-VSL-0002A/B • 24590-QL-POA-MVA0-00010-09-00002, Rev. 00A, Fatigue Analysis, UFP-VSL-0002A/B • 24590-QL-MRC-MVA0-B0002-S0011DC • 24590-QL-MRC-MVA0-B0002, Rev. 001

Topical Area	Document Number and Title
	• 4590-QL-POC-MVA0-00001-19-02, Rev. 00C, Seismic Data Report, HLW-VSL-00022 • 24590-QL-POA-MVA0-00010-19-08, Rev. 00A, Fatigue Analysis Report, HLW-VSL-00022 • 24590-QL-POA-MVA0-00010-03-21, Rev. 00F, Nozzle Loading Calculations, HLW-VSL-00022 • 24590-QL-MRB-MVA0-00001-S0017 • 24590-QL-MRB-MVA0-00001-S0019 • 24590-QL-POD-MVA0-00001-09-00024, Rev. 00A, Finite Element Seismic Calculations, PWD-VSL-00044 • 24590-QL-POD-MVA0-00001-06-07, Rev. 00D, Nozzle Loading Calculations, PWD-VSL-00044 • 24590-QL-POD-MVA0-00001-09-06, Rev. 00C, Vessel Code Calculations, PWD-VSL-00044 • 24590-QL-POD-MVA0-00001-09-00023, Rev. 00A, Finite Element Fatigue Calculations, PWD-VSL-00044 • 24590-QL-POC-MVAO-00001-03-19-00006, Seismic Report for CXP-VSL-00001 • 24590-QL-POC-MVAO-00001-03-17, External Nozzle Loads Analysis for CXP-VSL-00001 • 24590-QL-SRA-MTE5-0001-47-00001, FRP-VSL-00002A/B/C/D, Waste Feed Receipt Vessel and Pulse Jet Mixers, Dec. 8, 2005 • CCN: 136987, Independent Review of the Waste Feed Receipt Vessel FRP-VSL:-00002 A, B, C, D, March 8, 2006 • 24590-QL-POA-MVAO-00006-08-00002, Rev 00B, Seismic Analysis of 180” Cesium Exchange Treated Law Collection Vessel CXP-VSL-00026 A/B/C • 24590-QL-POA-MVAO-00006-04-03 Nozzle Analysis • 24590-QL-POC-MVAO-00001-03-18 HLP-VSL-0027A • 24590-QL-POC-MVAO-00001-03-19 HLP-VSL-0027B • 24590-QL-POC-MVAO-00001-03-20 HLP-VSL-0028
5	• WTP Design Process Review (with a Seismic Focus), April 3, 2006 • 24590-WTP-SED-ENS-03-002-02, PT Safety Envelope Document • 24590-WTP-SRD-ESH-01-001-02, PT Safety Requirements Document • Rich Smith presentation on the seismic probabilistic risk assessment
6	• 24590-WTP-DC-PS-01-002, Revision 3 - Pipe Support Design Criteria • 24590-WTP-GPG-ENG-005, Revision 2 - Engineering Design Guide for Pipe Supports • USACE Piping Presentation, April 17, 2006 by John Minichiello • 24590-PTF-PHC-FRP-50001; Engr Support Calc for PTF-FRP-H25087 OCC: 1.2 w/ RGM; Engr Support Calc for PTF-FRP-H25081 OCC: 1.2 w/ RGM; Engr Support Calc for PTF-FRP-H25091 OCC: 1.2 w/ RGM; Engr Support Calc for PTF-FRP-H25171 OCC: 1.2x1.5 w/o RGM

Topical Area	Document Number and Title
	• 24590-PTF-PHC-FRP-50004; Engr Support Calc for PTF-FRP-H25088; Engr Support Calc for PTF-FRP-H25170 • 24590-PTF-PHC-FRP-50005; Engr Support Calc for PTF-FRP-H25101, H25095, Engr Support Calc for PTF-FRP-H25101, H25095, Engr Support Calc for PTF-FRP-H25017 • 24590-PTF-PHC-FRP-50006; Engr Support Calc for PTF-FRP-H25086 • 24590-PTF-PHC-FRP-50007; Engr Support Calc for PTF-FRP-H25089 • 24590-PTF-PHC-FRP-50008; Engr Support Calc for PTF-FRP-H35035, H35028, H35030, H35031, H35033, H35037, H35039, H35006; Engr Support Calc for PTF-FRP-H35021, H35024, H35019, H35017, H35060 • 24590-PTF-PHC-FRP-50016; Engr Support Calc for PTF-FRP-H25108 & H25109 • 24590-PTF-PHC-FRP-50017; Engr Support Calc for PTF-FRP-H35040, H35032, H35036, H35038; PTF-PWD-H35008 & H30003; Engr Support Calc for PTF-FRP-H35016, H35018, H35020, H35023, H35003; PTF-PVP-H35062 & H35059; PTF-PWD-H30022 & H30007 • 24590-PTF-PHC-FRP-50018; Engr Support Calc for PTF-FRP-H25090 & H25097 • 24590-PTF-PHC-FRP-50068; Engr Support Calc for PTF-FRP-H35029, H35027, H35034 • 24590-PTF-PHC-FRP-50069; Engr Support Calc for PTF-FRP-H25110 • 24590-PTF-PHC-FRP-50070; Engr Support Calc for PTF-FRP-H25111 • 24590-PTF-PHC-FRP-50071; Engr Support Calc for PTF-FRP-H25112, H25104 • 24590-PTF-PHC-FRP-50072; Engr Support Calc for PTF-FRP-H25113
7	• Julyk, John L., USACE Equipment Design and Seismic Qualification Requirements Review Presentation, May 8, 2006 • 24590-WTP-3PS-SS90-T0001, Rev. 1, Engineering Specification for Seismic Qualification of SC I/II Equipment & Tanks • 24590-WTP-3PS-JQ06-T0003, Rev. 4, Engineering Specification for Seismic Qualification of SC I Control and Electrical Systems and Components • 24590-WTP-LAW-3PS-M000-T0002, Rev. 0, Engineering Specification for Master Slave Manipulators for PTF, HLW, LAW & LAB • 24590-QL-POA-MJW0-00003-09-00003, Rev. G, Model RE-T Telemanipulator, Seismic Analysis • 24590-WTP-3PS-ADDC-T0002, Rev. 1, Engineering Specification for HLW/PT System Transfer Hatches, Hatch Drives, Hatch Pushrod Assemblies, and Floor Penetration Liner • 24590-QL-POA-ADDH-00003-08, Rev. B, Seismic Qualification Report, Hatch • 24590-HLW-3PS-MX00-T0001, Rev. 2, Engineering Specification for QL Shielded Personnel Access Doors • 24590-QL-POA-ADDB-00001-09-33, Rev. F, Design File, Shielded Personnel Access Doors, Bechtel Hanford, River Protection Project-Waste Treatment

Topical Area	Document Number and Title
	Plant
8	• 24590-HLW-SSC-S15T-00137, Design of Multi-Commodity Support Beams Above El. 37'-0" • 24590-HLW-SSC-S15T-00055, Filter Cave Crane/Power Manipulator Runway
9	• Structural Summary Report, Chapters 1 through 8 and Appendices B through H, J, and K

PROGRESS REPORT

Independent Review of Implementation of Revised Seismic Design Criteria
Hanford Waste Treatment and Immobilization Plant (WTP)

Revision D

Appendix C

Comment Summary Sheets

C.1 Design Process Review Comments
C.2 Structural Modeling Review Comments Phase I & II
C.3 Load Path Review Comments
C.4 Concrete Design Review Comments
C.5 Structural Steel Design Review Comments
C.6 Vessels and Nozzles Review Comments
C.7 Piping and Piping Supports Review Comments
C.8 Equipment Review Comments

C.1 Design Process Review Comments				
No.	**Document**	**Comments and Observations**	**BNI Remarks/ Resolutions[1]**	**Follow-up by USACE Review Team**
DP-1	1. 24590-WTP-DC-ST-04-001, Rev. 3, Seismic Analysis and Design Criteria. 2. BNI Personnel, "WTP Design Process Review (with a Seismic Focus)," Presentation, April 3, 2006.	Reviewer: Jim Johnson/Steve Short 1. In many multi-disciplinary projects, the division of responsibilities for design and qualification of structures, systems, and components for seismic DBE conditions is unclear. Often this situation is termed a silo effect, where each discipline is focused on their individual responsibilities and interfaces do not receive appropriate attention. 2. Division of responsibilities here is meant to include: a) By disciplines, such as civil, mechanical, electrical, fire protection, and systems engineering. b) By components such as piping, equipment qualification (mechanical and electrical), other commodities (conduit, cable trays, HVAC duct, etc.), field routed commodities, etc. 3. Interface control is essential to assure smooth implementation of design, construction, and operation of the facility. 4. Reference 2 provided presentation material identifying the basic responsibilities of different organizations within the BNI group. Based on the information provided and an independent assessment of	Observations	

[1] "Observation" is for information only - a response is not required.

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		the responsibilities of the various organizations, following observations are made: a) Integrated Safety Management (ISM) process is the key initiating point and the focal point for on-going modifications to the design. Multi-disciplinary teams implement the process. b) For seismic issues, CS & A plays a major role for design and qualification of SSCs. CS & A is responsible for overall seismic analysis of the SC-I structures providing input (Bubble acceleration plots) for structure design and input (In-Structure Response Spectra) for system and component design and qualification. For seismic equipment qualification, CS & A provides: in-structure response spectra for analysis and testing of systems and components; reviews Material Requisitions (with regard to seismic requirements); reviews Vendor submittals (proposal and final design package) for acceptable seismic analysis and testing and compliance with SADC and SRD; receives seismic anchorage loads for inventorying and verification that embeds or other		

C.1 Design Process Review Comments				
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		anchorage systems are adequate; and receives other loading conditions to be included in the structural element design. For commodities (HVAC, cable trays and conduit), CS & A performs seismic analyses and/or designs commodities and their supports. For piping, Plant Design [performs seismic analyses and CS & A designs supports and their anchorage. For C & I, CS & A designs instrument racks as necessary. For seismic systems interaction, it is assumed that CS & A will be the lead for the evaluation of interaction hazards during the design process and the in-plant verification before initial operation. c) Control and Instrumentation (C & I) plays a major role in the integration process since it touches many safety, process, and support systems. d) Plant Design, as the responsible entity for the 3D Model, also, plays a major role.		
DP-2	24590-WTP-DC-ST-04-001, Rev. 3, Seismic	Reviewer: Jim Johnson/Steve Short	BNI	

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No.	Document	Comments and Observations	BNI Remarks/ Resolutions[1]	Follow-up by USACE Review Team
	Analysis and Design Criteria. BNI Personnel, "WTP Design Process Review (with a Seismic Focus)," Presentation, April 3, 2006.	For success, it is essential that the various organizations and disciplines involved in the seismic analysis, design, and qualification of SSCs maintain appropriate channels of communication – avoid tendencies for silo type behavior.		
DP-3	WTP Design Process Review (with a Seismic Focus), April 3, 2006 24590-WTP-SED-ENS-03-002-02, PT Safety Envelope Document 24590-WTP-SRD-ESH-01-001-02, PT Safety Requirements Document Rich Smith presentation on the seismic probabilistic risk assessment	Reviewer: Jim Johnson/Steve Short DOE facilities are designed within a graded approach where the seismic design criteria used has conservatism and rigor that is consistent and appropriate for the safety and mission importance of the SSC being designed. Different criteria for different SSC characteristics are implemented by the assignment of NPH performance categories. At the onset of this project, it was unclear whether the NRC or DOE would regulate the WTP design. As a result, Seismic Categories ranging from I to IV were assigned to SSCs. These seismic categories are then related to the DOE NPH performance categories as the basic seismic criteria are from DOE-STD-1020-94. The seismic categories have been developed in a reasonable and rational manner following an integrated safety management process using basic principles of hazard identification, hazard evaluations, and hazard control development. Further, the basic	Observations	

C.1 Design Process Review Comments

No.	Document	Comments and Observations	BNI Remarks/ Resolutions[1]	Follow-up by USACE Review Team
		principles of conducting safety analyses in accordance with DOE-STD-3009 are applied to assess whether the hazards affect the public or on-site workers. Multi-discipline engineering, operations, and safety specialists participate in the hazard analysis process that leads to the assignment of seismic categories.		
DP-4	WTP Design Process Review (with a Seismic Focus), April 3, 2006 24590-WTP-SED-ENS-03-002-02, PT Safety Envelope Document 24590-WTP-SRD-ESH-01-001-02, PT Safety Requirements Document Rich Smith presentation on the seismic probabilistic risk assessment	Reviewer: Jim Johnson/Steve Short On Slide 21 of the presentation, it is noted that the hazard evaluation considers three affected populations (public, co-located worker, and facility worker), In other documents and in the seismic PRA, it seems that the facility worker is not considered. How are facility workers considered in categorization of SSCs and what is the distinction made from the co-located workers?	BNI	
DP-5	WTP Design Process Review (with a Seismic Focus), April 3, 2006 24590-WTP-SED-ENS-03-002-02, PT Safety Envelope Document 24590-WTP-SRD-ESH-01-001-02, PT Safety Requirements Document	Reviewer: Jim Johnson/Steve Short Are there requirements for redundancy or defense in depth that affect seismic categorization of SSCs? What is the process for assigning redundant hazard barriers (i.e., not primary barriers) to a seismic category?	BNI	

C.1 Design Process Review Comments				
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	Rich Smith presentation on the seismic probabilistic risk assessment			
DP-6	WTP Design Process Review (with a Seismic Focus), April 3, 2006 24590-WTP-SED-ENS-03-002-02, PT Safety Envelope Document 24590-WTP-SRD-ESH-01-001-02, PT Safety Requirements Document Rich Smith presentation on the seismic probabilistic risk assessment	Reviewer: Jim Johnson/Steve Short Certain SSCs are identified as APC or RRC and assigned to SC-IV and PC-1. Do all other items that are not APC or SC-1, II, or III have any seismic requirements?	BNI	
DP-7	WTP Design Process Review (with a Seismic Focus), April 3, 2006 24590-WTP-SED-ENS-03-002-02, PT Safety Envelope Document 24590-WTP-SRD-ESH-01-001-02, PT Safety Requirements Document Rich Smith presentation on the seismic	Reviewer: Jim Johnson/Steve Short Seismic design of fire protection piping is designed in accordance with NFPA following provisions of DOE-STD-1066-99, Fire Protection Design Criteria. What is the Seismic Category and NPH performance category for fire protection systems? In accordance with Section 7.3.2 of DOE-STD-1066, sway bracing is to be designed for a horizontal force, Fp = K*Wp, where K is consistent with DOE-STD-1020-94 determined by an engineer qualified in seismic analysis. For WTP fire protection piping, how is	BNI	

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	probabilistic risk assessment	the factor K determined? What are design considerations for fire protection piping in withstanding differential displacements and survival of fittings and joints?		
DP-8	WTP Design Process Review (with a Seismic Focus), April 3, 2006	Reviewer: Jim Johnson/Steve Short Page 16 of the presentation identifies SSCs making up the PT seismic control strategy as including: (1) ultafiltration feed vessels and associated piping; (2) C5 ventilation systems including HEPA filters; (3) others; and (4) power, controls, and instrumentation as required to allow the SC-1 SSCs to perform their safety functions. I assume that building envelope confinement is another important part of the seismic control strategy. What does "others" refer to? In the case of leakage from vessels or piping are there drainage, collection, and retrieval systems that are in SC-1? Are there monitoring and alarm systems that are in SC-1? Do the feed vessels and piping mentioned above include all vessels and piping in the PT facility containing hazardous materials including input from the tank farm, processing and storage in PT, and transfer out to the HLW facility?	BNI	
DP-9	WTP Design Process Review (with a Seismic Focus), April 3, 2006	Reviewer: Jim Johnson/Steve Short Page 55 of the presentation should include displacements for seismic anchor motion of multiply supported systems as key design input provided by the building dynamic seismic analysis in	BNI	

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		addition to seismic loads and ISRS.		
DP-10	1. BNI Personnel, "WTP Design Process Review (with a Seismic Focus)," Presentation, April 3, 2006. 2. S. Vail, "Preventing Seismic Interactions," Presentation, April 3, 2006. 3. 24590-WTP-DC-ST-04-001, Rev. 3, Seismic Analysis and Design Criteria	Reviewer: Jim Johnson/Steve Short 1. For facilities comprised of complex systems of diverse mechanical, electrical, and structural elements, systems integration requirements and procedures should be in place to prevent dimensional interferences. These systems integration provisions can be implemented in many forms, e.g., specifically calculated displacements for SSCs, dimensional envelopes specified - around which no other SSCs can be placed, exclusion areas defined, such as compartments where only SSCs of particular safety classes can be located, etc. The Project 3D model is a very valuable tool to display systems, especially the required dimensional envelopes for conditions of installation, operations, and for design basis accidents. 2. This issue applies to: a) All relevant loading conditions (normal operating conditions, non-seismic accident conditions, and Design Basis Earthquake - including RGM). b) Specifically designed SSCs, generically designed SSCs, field routed commodities under WTP-specific guidelines, and field routed commodities installed under good industry	Observations	

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		practice. 3. For the Design Basis Earthquake or RGM, the principal issue is systems interaction, including the phenomena: a) ***Impact*** of non-seismic category I or Seismic Category I SSCs on adjacent Seismic Category I SSCs – this impact due to close proximity of SSCs is potentially damaging to operability of vibration-sensitive SSCs or to failure of brittle SSCs, such as glass tubing like level indicators. These phenomena are sometimes referred to as ***proximity*** issues. b) ***Falling*** of non-seismic category I SSCs impacting Seismic Category I SSCs. c) ***Spray*** or ***flood*** hazards due to failure or leaking of liquid transmitting commodities, such as piping, or failure or leaking of components containing liquids. 4. For the Design Basis Earthquake or RGM, these potential interactions include: a) Seismic Category I SSCs on Seismic Category I SSCs. b) Seismic Category II SSCs on Seismic Category I SSCs. c) Seismic Category III & IV SSCs on Seismic Category I and II SSCs. The potential impact of SC-III and IV SSCs on SC-II SSCs refers to the possible initiation of a chain reaction of		

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		failure of SC-II and subsequently SC-I SSCs. 5. Reference 2 is part of Ref. 1 and itemizes numerous approaches to addressing the potential of "Seismic Interactions." a) Seismic categorization. Identify non-Seismic Category I SSCs that are potential hazards to SC-I SSCs and re-categorize them or their supports to be SC-II. b) Separation of SC-I SSCs from non-SC-I SSCs. This can be accomplished through exclusion areas, Zone of Influence (ZOI) concepts, etc. c) Re-routing of non-SC-I SSCs. d) Providing SC-II shields. e) Permitting potential systems interaction recognizing that the interaction will not cause damage to the SC-I SSC. Examples are small diameter conduit impacting large diameter SC-I conduit, small diameter piping impacting large diameter SC-I piping, etc.		
DP-11	BNI Personnel, "WTP Design Process Review (with a Seismic Focus)," Presentation, April 3, 2006. S. Vail, "Preventing Seismic Interactions," Presentation, April 3,	Reviewer: Jim Johnson/Steve Short The Project 3D model is a very valuable tool to display systems, especially the required dimensional envelopes for conditions of installation, operations, and for design basis accidents. There should be recognition that not all SSCs and commodities are modeled. Provisions to	BNI	

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	2006. 24590-WTP-DC-ST-04-001, Rev. 3, Seismic Analysis and Design Criteria	account for these situations should be in-place. Verify that the 3D model or other methods are in-place to identify all sources of dimensional incompatibilities for installation, normal operations, and design basis accidents – in particular, the earthquake design basis (RGM).		
DP-12	BNI Personnel, “WTP Design Process Review (with a Seismic Focus),” Presentation, April 3, 2006. S. Vail, “Preventing Seismic Interactions,” Presentation, April 3, 2006. 24590-WTP-DC-ST-04-001, Rev. 3, Seismic Analysis and Design Criteria	Reviewer: Jim Johnson/Steve Short Some basic principles have been shown to be effective in the past for the design of new facilities: a) Address the issue at the source rather than the target whenever possible. Utilize 5a-c above to the extent possible. Specifically, support, anchor, and locate non-SC-I SSCs to prevent impact, falling, spray, and flood rather than relying on shields, ZOI concepts, or non-damage impacts. i. Identify compartments to only contain SC-I and -II SSCs where appropriate. One example of this approach is Black Cells where access after an earthquake to verify the condition of SC-III	BNI	

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		and –IV SSCs is not feasible. ii. Increase anchorage to assure falling of non-SC-I SSCs will not occur. iii. For non-SC-I or –II piping systems that contain liquids, implement design provisions that minimize the likelihood of spray or flood. Implementing a source-based program minimizes configuration control requirements for construction and operations. b) Establish procedures for field routed commodities taking into account potential seismic systems interaction issues. c) Field walkdowns should be performed area-by-area and/or SC-I component-by-component depending on the hazard. 1. Verify that a source-based program is in-place to minimize the need to utilize target-based arguments for the lack of damage due to seismic systems interactions. 2. Verify that procedures are in-place to assure that field-routed commodities will not adversely affect the established dimensional envelopes for SC-I and –II SSCs		

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		during installation, operations, and design basis accident conditions.		
DP-13	BNI Personnel, “WTP Design Process Review (with a Seismic Focus),” Presentation, April 3, 2006. S. Vail, “Preventing Seismic Interactions,” Presentation, April 3, 2006. 24590-WTP-DC-ST-04-001, Rev. 3, Seismic Analysis and Design Criteria	Reviewer: Jim Johnson/Steve Short Fire protection piping should be routed implementing earthquake engineering concepts to avoid potential hazards due to spray and leakage. Dynamic analysis need not be performed, but basic routing and support principles should be implemented, especially for header-branch line locations, proximity of sprinkler heads to possible impact surfaces, etc. Verify that fire protection piping is being routed and supported taking into account the lessons learned from past earthquakes avoiding earthquake related vulnerabilities.	BNI	
DP-14	BNI Personnel, “WTP Design Process Review (with a Seismic Focus),” Presentation, April 3, 2006. S. Vail, “Preventing Seismic Interactions,” Presentation, April 3, 2006. 24590-WTP-DC-ST-04-001, Rev. 3, Seismic Analysis and Design Criteria	Reviewer: Jim Johnson/Steve Short Configuration control procedures for operations, maintenance, and new systems installation should be in-place. Configuration control procedures include permanent and temporary items, such as scaffolding, ladders, equipment carts, etc. Verify that Configuration Control procedures are in-place for operations, maintenance, and installation of new systems.	BNI	
DP-15	1. 24590-WTP-DC-ST-04-001, Rev. 3,	Reviewer: Jim Johnson/Steve Short	Observations	

C.1 Design Process Review Comments

No.	Document	Comments and Observations	BNI Remarks/ Resolutions[1]	Follow-up by USACE Review Team
	Seismic Analysis and Design Criteria 2. 24590-WTP-SRD-ESH-01-001-02, Project Safety Requirements Document, Vol. II 3. 24590-WTP-GPG-SANA-005, Rev. 2, “Guide: Seismic Probabilistic Risk analysis Methodology,” 03/08/04. 4. R.I. Smith, Meeting and Presentation, “Seismic PRA,” 4 April 2006.	1. Section 10.5a of Ref. 1 states that a seismic probabilistic risk assessment (PRA) was performed. The assertion is made that the results of the seismic PRA demonstrated the seismic design of the facility has adequate margin to preclude the need for any SSCs to be designated PC-4. Specifically, that the seismic design is adequate assurance of compliance with the radiation exposure standards of Ref. 2. 2. During the week of April 3, 2006, meetings were held with R.I. Smith to understand the seismic PRA that was performed for the WTP. The seismic PRA was developed for a limited purpose, i.e., to verify that no Structures, Systems, and Components (SSCs) need to be classified as PC-4 or higher to meet the Radiation Exposure Standards (RES) for co-located workers and the general public. Since the objective was limited, conservative simplifying assumptions were made as first approximations. The meetings and documents (Refs. 3 and 4) explained the objectives, methodology, and results. It is agreed that the seismic PRA satisfactorily addresses the specific issue related to PC-4 SSC classification.		

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SM-1	24590-WTP-DC-ST-04-001, Rev. 3, - Seismic Analysis and Design Criteria	Reviewer: Stephen A. Short The basic seismic criteria are DOE-STD-1020-94. The more recent version of this standard is DOE-STD-1020-2002. These versions of STD-1020 are essentially equivalent for PC-3 SSCs, but may be significantly different for PC-1 and PC-2 SSCs. The later version of STD-1020 uses the IBC and ASCE 7 for seismic provisions while the earlier version uses the UBC for seismic provisions. DOE-STD-1020-2002 specifically notes that the IBC seismic criteria are not equivalent to the UBC seismic criteria. For the Pre-Treatment and High Level Waste (PT and HLW) facilities, the PC-3 provisions are expected to be comparable between DOE-STD-1020-94 and DOE-STD-1020-2002.	Observation DOE-ORP accepted comment to be a Code of Record issue.	Closed For PC-2 structures, systems, and components within the HLW and PT facilities as well as other parts of the WTP, the use of the 97 UBC is potentially unconservative in light of the USGS ground motion used as input for the IBC and ASCE 7. There is further potential unconservatism from the revised PC-3 ground motion that would also affect the PC-2 levels. It is my understanding that BNI or DOE-ORP checked the PC-2 ground motion used for design against the recent increases in ground motion and determined that what is being used is greater than what is required by more recent ground motion estimates. It is recommended that this check be documented in the seismic criteria.
SM-2	24590-WTP-DC-ST-04-001, Rev. 3, - Seismic Analysis and Design Criteria	Reviewer: Stephen A. Short There is no mention of the seismic scale factor, SF from DOE-STD-1020 in the Bechtel Seismic Analysis and Design Criteria (SADC). For PC-3 SSCs, the default SF value is 1.0. However, DOE-STD-1020 states that “Variable scale factors, based on the slope of site-specific	BNI BNI will take comment into consideration as a Category A Conservatism.	Closed

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		hazard curves, may be used…..” For the original 1996 seismic hazard information, the scale factor can be shown to be between about 0.9 and 0.95 based on the Hanford seismic hazard curves. Using SF = 1.0 results in seismic demand that is 5 to 10 percent greater than necessary. Consider use of the variable seismic scale factor.		
SM-3	24590-WTP-DC-ST-04-001, Rev. 3, - Seismic Analysis and Design Criteria	Reviewer: Stephen A. Short Test data has demonstrated that low-rise concrete shear walls with a height to width ratio less than 2.0 such as the PT and HLW buildings have their shear strength significantly underpredicted by the ACI strength equations. Section 4.2.3 of ASCE 43-05 specifies an equation for the shear strength of such walls that eliminates this conservatism. For many of the walls of the PT and HLW buildings, this shear strength equation may permit a significant reduction in conservatism. Consider the use of the ASCE 43 shear wall capacity equation	BNI BNI will take comment into consideration as a Category A Conservatism.	Closed It is recommended that this potential conservatism be considered. However, even though this shear wall capacity is published in ASCE 43, it is our understanding that the test data is being reviewed at this time by ACI. Category B or C conservatism may be more appropriate until the ACI review is completed and resolved.
SM-4	24590-WTP-DC-ST-04-001, Rev. 3, -	Reviewer: Stephen A. Short	BNI	Closed

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	Seismic Analysis and Design Criteria	By these criteria, $F\mu$ of unity is used for crack control as the reinforced concrete walls are one of the confinement barriers. For constructed walls the use of $F\mu$ values from DOE-STD-1020 are permitted. This raises the question as to whether such walls can function as adequate confinement barriers. In addition, during the presentations it was shown that concrete stresses averaged over some distance are used instead of peak concrete stresses. The use of average stresses implies redistribution of concrete stresses during limited inelastic behavior. The use of average stresses does not seem consistent with $F\mu$ of unity.	This question identifies two items: adequacy of confinement and the use of average stress and its ramification on F_{μ}. It is recognized the concrete cells in PTF and HLW are not perfectly watertight, for example, note that water stops have not been provided at construction joints as would be in water tight concrete construction. However, this does not invalidate their ability to meet the requirements for secondary confinement documented in the safety analysis. These confinement requirements relate primarily to confinement of radioactive aerosols that may be formed in the cells during normal operations or as a result of design basis events. Provided seismic caused through-wall cracking is not substantial enough that the C5V system is not able to maintain a negative pressure (with respect to atmosphere), the concrete structure has fulfilled its primary confinement function. In regard to liquid confinement, the safety analysis only requires that the structure remain sufficiently tight following a seismic event such that leakage of contaminated liquids from the cells into adjacent areas can be “minimized for at least 24 hours” following the event. Minor through-wall cracking that may occur in localized areas as a result of the basis earthquake is not expected to compromise this function since the slabs are sloped to a	

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			sump/drain system that keeps the liquid away from the walls. Also, the great majority of the high-active liquid waste is stored inside the cells in seismically qualified vessels and would not be available to contribute to leakage. The response to PSAR question PT-PSAR-221 provides a detailed description of how confinement is maintained. On the second item, the ACI 318-99 code allows a design to be based on average stresses. For example, the design of a shear wall is based on a shear force, which is analogous to using an average shear stress. Another example is the design of two-way slabs. ACI 318-99 allows this design to be based on the bending moment averaged over a width of a "column strip" that is 1/2 of the smaller bay spacing of the columns. It should also be noted that for the HLW and PT facilities the design approach was to determine and document for each structural element a D/C (demand over capacity) ratio based on the full seismic loads rather than dividing the seismic loads by F_{μ}.	
SM-5	24590-WTP-DC-ST-04-001, Rev. 3, -Seismic Analysis and Design Criteria	Reviewer: Stephen A. Short The revised horizontal DBE response spectrum has a flat, but narrow plateau at peak spectral acceleration of 0.8g at 4 to 6 Hz and	DOE-ORP/USACE 1st Response: Carl Costantino, see attached Response to SM-5 and SM-29.	Open 1st Follow-up: The reviewer still needs to understand the basis for the narrow banded input spectrum. Are uncertainties in

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		peak ground acceleration of 0.3g corresponding to 84 percentile amplification. DOE-STD-1020 specifically specifies that median amplification is to be used with no conservative bias because the introduction of conservatism in spectral amplification is frequency dependent. Use of spectral conservative amplification has also lead to an undesirable design spectral shape that has relatively steep slopes below 4 Hz and above 6 Hz. Steep slopes in the design spectra mean relatively large changes in structural input with variation in structural frequencies.	2nd response: Ivan Wong The review comments by Short and Johnson on the shape of the design spectrum are valid comments from a regulatory perspective. However, Carl Costantino points out correctly the site-specific reasons for the design spectral shape and so there will need to be some sort of negotiated agreement on the final design spectrum.	the modeling and analyses adequately captured when this spectrum shape is used? In a subsequent review, the effect of the spectrum slope on seismic response is demonstrated where small differences in soil-structure frequency lead to significant differences in seismic response in each direction of shaking. 2nd follow-up: JJJ – The specific comments related to broadband vs. narrow band ground response spectra and the use of the use of spectral amplification values at non-exceedance probabilities other than median values are correctly identified as regulatory issues. The question of the lack of amplified frequency content in the 2 Hz. and less frequency range remains open.
SM-6	24590-WTP-DC-ST-04-001, Rev. 3, - Seismic Analysis and Design Criteria	Reviewer: Stephen A. Short In Section 7.2.1.5 of the SADC, permissible Response Level 1 and 2 damping levels are provided (Actually, Response Level 1 damping is used. Both DOE-STD-1020-94 and ASCE 43-05 permit larger values of structural damping in accordance with Response Level	BNI Previous parametric studies performed on Rev. 0A of HLW building showed that the calculated structural forces (in terms of weighted maximum accelerations at each floor level) are essentially the same using either Level 1 or Level 2 structural damping, with the results using Level 1 damping are slightly on the conservative	Closed

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		3. For analyses where response is limited to elastic levels, Response Level 2 damping values are most appropriate. However, f where inelastic behavior is permitted, Response Level 3 damping values may be used.	side. Thus using only Level 1 damping in the analysis is judged appropriate since this practice will results in faster execution without producing over-conservative design parameters. Per SADC stipulation, the SSI analyses for both HLW and PTF buildings are limited to elastic analysis only. Every element in the analysis model must remain elastic. Therefore, Level 3 damping is not used. Inelastic behavior may be permitted for selected structural components only later in the design stage.	
SM-7	24590-WTP-DC-ST-04-001, Rev. 3, -Seismic Analysis and Design Criteria	Reviewer: Stephen A. Short ASCE 4 states that the potential for reduced lateral soil support of the structure should be considered when accounting for embedment effects. Section 7.2.1.7 of the SADC states “Monitoring the soil pressure behind the embedded walls of the structure. Parametric studies shall be performed to include the effect of soil-wall separation up to a depth that seismic soil pressure exceeds the in-situ static stress on the walls.” There is no discussion as to how these parametric studies will be performed.	BNI 1st Response: A parametric study was performed on the wall-soil separation issue in detail for the HLW Rev. 0A model, upper bound soil case. This study shows clearly that, except for very shallow depth (about 2 ft.), the separation between the structural walls and the surrounding soil media during earthquake shaking, even at the most critical locations, is of very limited nature in both spatial locations and temporal durations, and is unlikely to have any significant effect on the overall behavior of the buildings Four figures from that study are attached. (These figures are located	Closed 1st Follow-up: There are still a few questions on this issue. It is a clear requirement in ASCE 4 to account for reduced lateral soil support of embedded structures. The BNI argument is that there is no significant reduction of lateral soil support because the calculated soil pressures do not significantly exceed the static earth pressures for significant lengths of time. It was good to see that this requirement was explicitly considered. In the SSI analyses, the soil and structure are welded together for the entire embedment.

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			following the end of this resolutions table.) Figure SM-7-1 shows the overall layout of HLW model Rev. 0A at grade. Figures SM-7-2, -3, and -4 show the soil-pressure time histories at selected embedment depths at the three most critical locations, i.e., the soil columns NW-Y, NE-Y and SW-Y where there is no floor mat at grade. In these figures, the "Static Envelope" line is the calculated static soil pressure at the elevation. The magnitude and duration of dynamic soil pressure exceeding the static envelope are clearly illustrated in the figures. It should be noted that the above conclusion was reached for the Rev. 0A structure and the embedment was modeled conservatively such that the 6 ft. thickness of the foundation mat at El. –21' was ignored. Later modifications of the structure design, including the additional grade floor mat on the southern side of HLW building, have the effect of increasing the size of the static envelope and therefore reducing the potential separation between soil and structure. Therefore, the conclusion is still valid. 2nd Response: This response is to clarify the additional comments made by the reviewer.	The reviewers should be able to review the entire calculation for it to be convincing that there is no significant reduction in lateral soil support. Is this analysis documented in the structural summary report? If not, the calculations should be made available to the reviewers. It is not clear how the static soil pressure is computed at 1.25 feet below grade. The value of 0.27 ksf (scaled from Fig. SM-7-2) is much greater than obtained from 0.11 kcf * 1.25 feet * ko=0.5 that equals 0.07 ksf. By the same approach, the static pressure at 17.5 and 19.75 feet below grade is about 30% higher than the values on the figures. The peak soil pressures at 1.25, 17.5, and 19.75 feet below grade scaled from Figures SM-7-2 thru SM-7-4 are 0.85, 1.0, and 1.72 ksf, respectively. The same values from the recent SSI calculation (Figure 7-48) are 2.3, 1.4, and 3.5, respectively. As a result of the differences in static and dynamic soil pressures and because only a small part of the calculation is presented, there is

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			The static lateral earth pressure values as shown in Figures SM7-2 through SM7-4 were calculated as the combination of the following components: (Figure SM7-5) • Insitu lateral soil pressure due to soil weight as recommended in the WTP Geotechnical Report. • A large portion of the side walls are adjacent to extended surface basemat foundation with an average overburden pressure of 1.0 ksf, generating additional lateral pressure of 0.36 ksf. • Underground structure was constructed by open-pit excavation. Backfill soil adjacent to the sidewalls was compacted, generating additional compaction-induced lateral soil pressure on the sidewalls. This pressure varies with depth and reaches a maximum value of 0.52 ksf. at 3 ft. depth, then reduces to zero at about 15 ft. depth (as recommended in the Geotechnical Report) The additional comments regarding the differences between the results from the parametric study, which is the basis of the figures SM7-1 through SM7-5 and is based on the HLW Rev. 0A model, and the results of the latest HLW Rev. 0D model, will be answered in response to	not a convincing story that there is no significant lateral soil support. 2nd Follow-up: The at-rest lateral coefficient appears to be 0.36 rather than 0.5 as assumed in the comment. However, there are additional static lateral pressures due to compaction and overburden. This seems reasonable. This closes the comment from the standpoint of the static pressures except that it would have been desirable for this to have been documented in the SSI calculation. The dynamic pressures questions have been resolved in response to SM-85.

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			SM-85.	
SM-8	24590-WTP-DC-ST-04-001, Rev. 3, - Seismic Analysis and Design Criteria	Reviewer: Stephen A. Short Section 7.3.5 of the SADC addresses seismic interaction of lower category SSCs with higher category SSCs. It is not entirely clear how the lower category SSCs are designed.	BNI If the failure of a SC-III or SC-IV system/component under a seismic event could adversely affect the safety function of a SC-I SSC, the system/component is categorized as SC-II and designed accordingly. In the WTP Project, seismic categorizations of SSCs are determined through an integrated safety management process (ISMP).	Closed The statement in the resolution note should be placed in Section 7.3.5 of the seismic criteria to make what is done clear.
SM-9	24590-WTP-DC-ST-04-001, Rev. 3, - Seismic Analysis and Design Criteria	Reviewer: Stephen A. Short The steel superstructure for both the PT and HLW buildings are ordinary braced frame structures with no credit for inelastic energy absorption taken (i.e., $F\mu$ of unity). The philosophy of DOE-STD-1020 is that ductile detailing measures shall be employed. Combined with this requirement, the design is allowed to reduce seismic forces by the introduction of a relatively small inelastic energy absorption factor, $F\mu$.	BNI BNI will take comment into consideration as a Category A Conservatism.	Closed The point of the comment was not that BNI should use $F\mu$ but that ductile detailing is encouraged or required by DOE seismic criteria. BNI designs SC-I and SC-II steel structures by 1997 UBC Section 2213 that employs Seismic Zone 3 and 4 detailing. This is in compliance with DOE-STD-1020.
SM-10	24590-WTP-DC-ST-04-001, Rev. 3, - Seismic Analysis and Design Criteria	Reviewer: Stephen A. Short The SADC does not reference the AISC seismic provisions or the FEMA 350 series of reports that	DOE-ORP. DOE-ORP accepted comment to be a Code of Record issue.	Open The use of an older Code of Record is acceptable as long as there is not a know problem with

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		have changed steel design based on failures seen during the 1994 Northridge and the 1995 Kobe earthquakes. What assurance do we have that lessons learned from these earthquakes are not being overlooked? Assure that important recent steel provisions are not overlooked		that code. In the case of steel design, recent earthquake experience demonstrated failures and the steel codes have been modified to prevent future failures. I believe that the AISC seismic provisions must be considered for the design of these facilities.
SM-11	WTP Structural Modeling Review Presentation	Reviewer: Stephen A. Short This presentation was a comprehensive view of the criteria used and calculations performed for the WTP design. The presentation was clearly delivered by the personnel that had actually performed the work such that they were very knowledgeable and able to answer all questions posed to them. As a result, the presentations greatly facilitated the review process and were certainly appreciated. During the course of the presentation, it became clear that Bechtel personnel had performed many side calculations to check the accuracy of their assumptions and methodology. It was learned during the meetings at the Bechtel offices that these side calculations have been documented in a structural summary report prepared for the HLW building. It is anticipated that	Observation	Closed The structural summary report is currently being reviewed.

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		this structural summary report will be reviewed in detail at a later time.		
SM-12	WTP Structural Modeling Review Presentation	Reviewer: Stephen A. Short Uncertainties in Soil Properties and SSI Analyses - Page 23 of the presentation presents soil profiles for lower bound, best estimate, and upper bound properties based on the updated soils data determined when the revised ground motion was developed. The variation of shear wave velocity with depth is much less than the minimum variation specified in ASCE 4 as shear wave velocity lower bound is only 1.14 less than the best estimate meaning lower bound shear modulus is only 1.29 less than the best estimate. The shear modulus should cover a range of at least 1.5. The actual SSI analysis was performed using the envelope of the original soil profile and the updated profile where the updated profile only had an influence very close to the ground surface where the variation in shear modulus was a factor of about 1.6.	BNI Reviewer pointed out that ASCE 4 requires variation of soil properties in SSI analyses. The variation of shear modulus of soil should be as least 1.50. Refer to plots of shear wave velocities of Upper bound, Mean, and Lower bound soil profiles in slide #23, #24, and #26 of the presentation, given to the USACE on 01.09.06. (For reference, a copy of these slides is located following the end of this resolutions table.) The updated shear wave velocities of the three soil profiles are presented in plot in slide #23 where the variation of shear modulus between different soil profiles are less than the 1.50 specified in ASCE 4. The updated and the original shear wave velocities of the three soil profiles are plotted together in slide #24. The original Upper bound soil profile has higher shear wave velocities than the corresponding updated soil profile. The original Lower bound soil profile has lower shear wave velocities than the corresponding updated soil profile except at shallow depth where the updated lower bound soil profile has lower values.	Closed

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			To ensure that the bounding soil properties are considered in the SSI analyses of HLW and PTF, bounding soil profiles are used in SSI analyses. The bounding soil profiles used in the SSI analyses consist of (1) the upper bound profile is the maximum enveloped of the original and updated upper bound soil profiles, (2) the lower bound profile is the minimum enveloped of the original and updated lower bound soil profiles, and (3) the Mean profile is the original soil profile. Since the original soil profiles satisfy the ASCE 4 required minimum variation of shear modulus by a factor of 1.5, and since the final soil profiles used in the current SSI analyses (shown in slide #26 of presentation) have equal or larger variation than the original soil profiles, this ASCE 4 requirement is considered satisfied.	
SM-13	WTP Structural Modeling Review Presentation	Reviewer: Stephen A. Short <u>Comparison of GT/STRUDL and SASSI Structural Models - </u>Validation of the model conversion from GT/STRUDL to SASSI is performed by comparing the fundamental frequencies of fixed base GT/STRUDL model to peaks of the transfer function as determined from a SASSI structure model on a	Observation 1st Response: BNI is confident in the conversion check performed. 2nd Response: The reviewer's suggestion is well received. It is our experience that SASSI transfer function results at low frequency will be significantly impacted even if one node is inadvertently fixed. Our work process is	Closed 1st Follow-up: It is our experience that it is easy to inadvertently constrain degrees of freedom in the House module of SASSI. Our solution to check such an occurrence is to evaluate transfer functions at the 1st frequency (i.e., near zero). For this long period motion, all

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		SASSI hard rock site. Normally comparison of only fundamental frequencies would not be sufficient. Since the SASSI structural model was developed directly from the GT/STRUDL model such a comparison probably does demonstrate that a significant error has not been introduced during the conversion process.	to visually inspect all plotted transfer functions. We pay special attention to the beginning portion of the curves, i.e., the portion at very low frequency range. In addition, the bubble diagrams of maximum accelerations are generated for all nodes in the model. The results confirm that no node has "zero" acceleration, which would be the response obtained from SASSI for a fixed node. The inspection of the transfer functions and results from bubble plots confirm that there is no artificial fixity in the model.	transfer function values should be very near unity. We export the values and sort them by amplitude to assure that there are no inadvertent constraints. What does Bechtel do to assure that the SASSI model has not inadvertent constraints? 2nd Response: Closed
SM-14	WTP Structural Modeling Review Presentation	Reviewer: Stephen A. Short <u>Accidental Torsion -</u> Additional forces in shear walls are computed due to accidental torsion in the manner described on Page 69 of the presentation. This approach is consistent with that described in Section 3.1.1 of ASCE 4. It was mentioned at the meeting that these provisions are sometimes not implemented when there are detailed finite element models such as those used for the WTP buildings but they are included for this design. I believe it is generally not the practice to include accidental torsion with detailed modes. Hence, the approach used f is conservative and in compliance with ASCE 4.	Observation BNI acknowledges the observation.	Closed

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SM-15	WTP Structural Modeling Review Presentation	Reviewer: Stephen A. Short <u>Reduction of Spectral Peaks -</u> Comparison of raw and final spectra on Pages 53 and 54 and Pages 73 and 74 indicate the broadening, enveloping, reduction of peaks for ISRS. Page 53 is a spectrum with no narrow peaks for which reduction is not appropriate. There was no peak reduction in the final spectrum as shown on Page 73. The spectrum on Page 54 has a narrow peak at about 3.5 Hz that has been reduced in the final spectrum as shown on Page 74. It is interesting to note that this peak is as broad as it can be and still justify reduction. The frequency range at 80 percent of the peak is just below 30 percent of the peak frequency per the criteria.	Observation The point of the observation is unclear to BNI.	Closed
SM-16	WTP Structural Modeling Review Presentation	Reviewer: Stephen A. Short <u>Seismic Loads for Design -</u> Pages 88 and 97 note that seismic forces for design are determined from static analysis of a detailed SAP2000 model with seismic inertial loads developed from floor seismic response accelerations. For the steel superstructure, the design forces are verified by comparison of forces from the SASSI analysis. The use of static analysis with seismic	Observation BNI acknowledges the observation. The observation is in agreement with what *is* BNI's process, in place.	Closed. It should be noted that this approach was not acceptable to the DNFSB for the Pit Disassembly and Conversion Facility at Savannah River. The Board suggested and the designer (WGI) agreed to perform a dynamic analysis of the detailed structural model using foundation response spectra determined from SASSI

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		inertial forces is criticized by some because higher mode seismic behavior of some local regions is not captured. In this case, it is concluded that such behavior is not significant for the concrete shear wall portion of these structures. This conclusion seems reasonable to me. The SASSI time history check will capture this phenomenon if it were to occur in the steel superstructure.		analyses. This issue is addressed in Attachment G of the structural summary report that is currently being reviewed.
SM-17	Reductions of Conservatisms in the Analysis	Reviewer: Stephen A. Short This subject is described as Agenda Item 6 in the presentation (Pages 111 through 119) as well as other documents received after the Richland visit by the reviewers. The WTF facilities were designed with more conservatism than a normal design because the construction so closely followed the design. Due to the increased ground motion at the site, a list of potential reductions in conservatism was considered and some of these reductions were implemented in the design ranging from Group A, least controversial to Group C, most controversial.	Observation BNI acknowledges the observation.	Closed

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SM-18	Reductions of Conservatisms in the Analysis	Reviewer: Stephen A. Short #1.3A - Use of Response Level 2 damping of 7% for reinforced concrete vs. Response Level 1 damping of 4% would normally be expected to be of some consequence. I assume that structural material damping has low effect on response because there is high radiation damping due to soil-structure interaction.	Was BNI; changed to Observation. See response for SW-6.	Closed
SM-19	Reductions of Conservatisms in the Analysis	Reviewer: Stephen A. Short #2.9A - Reducing conservatism in below grade wall design by using SASSI wall pressures is being implemented. This is an area that requires further review as I do not know the original procedure for evaluating and designing below grade walls.	USACE Review of below grade walls performed in Feb 27 to March 9 review	Closed per M. Summers Comments on this issue are provided in the Feb 27 to March 9 review.
SM-20	Reductions of Conservatisms in the Analysis	Reviewer: Stephen A. Short #1.5B - Consideration of horizontal spatial variation of input ground motion due to incoherency may be especially beneficial in reducing vertical ISRS for equipment seismic qualification. Consider incorporating incoherency effects	BNI BNI will take comment into consideration as Category B Conservatism. Note: use of coherency models is addressed in the Seismic Analysis and Design Criteria (24590-WTP-DC-ST-04-001 Rev. 3).	Closed

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SM-21	Reductions of Conservatisms in the Analysis	Reviewer: Stephen A. Short #2.7C - If it is beneficial to use actual measured concrete strength data rather than minimum specified levels, this measure that has been rejected would be easy to implement and there should be sufficient data available.	BNI BNI will take comment into consideration as Category B Conservatism.	Closed
SM-22	Reductions of Conservatisms in the Analysis	Reviewer: Stephen A. Short #4.3 – The use of past earthquake experience data for equipment seismic qualification per DOE/EH-0545 as currently permitted by ASCE 43-05 has been dropped from consideration at this time. This approach may be very beneficial in terms of cost and schedule for certain equipment items and should not be completely forgotten. Keep open the possibility of using earthquake experience data	BNI BNI will take comment into consideration as Category B Conservatism.	Closed
SM-23	24590-PTF-S0C-S15T-00005, Rev. 0B, - PTF Seismic Analysis: In-Structure Response Spectra (ISRS) 24590-PTF-S0C-S15T-00045, Rev. B, -	Reviewer: Stephen A. Short Acceleration response spectra are computed from acceleration time history responses and the resulting spectra in each direction (i.e., X due to x input, Y due to x input, Z due to x input, etc.) are combined by SRSS for each direction of seismic input.	Observation BNI acknowledges the observation.	Closed

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	PTF Seismic Analysis: Enveloped In-Structure Response Spectra	Narrow peaks that will be broadened are reduced in accordance with ASCE 4. The spectra are then broadened to cover uncertainties. Spectra for each soil bound (lower, best estimate, and upper) are then enveloped with some manual smoothing to eliminate any unwanted spectral dips. The SASSI model does not capture the bending response to vertical ground motion of concrete slabs or steel beams or girders. As a result, single degree of freedom oscillators are added to represent this behavior and enable response spectra generation at these locations.		
SM-24	24590-PTF-S0C-S15T-00005, Rev. 0B, - PTF Seismic Analysis: In-Structure Response Spectra (ISRS) 24590-PTF-S0C-S15T-00045, Rev. B, - PTF Seismic Analysis: Enveloped In-Structure Response Spectra	Reviewer: Stephen A. Short On Sheet 5, reduction of narrow spectral peaks by 15% is described. This is done to compensate for conservatism introduced by peak broadening. Sheet 5 states that this is conservatively only done for narrow peaks. In fact, it is only appropriate for narrow peaks and would be unconservative for broader peaks. The R.P. Kennedy approach for prescribing appropriate peaks for broadening should be added to ASCE 4 to clarify this point in the future.	Observation BNI acknowledges the observation.	Closed
SM-	24590-PTF-S0C-	Reviewer: Stephen A. Short	BNI	Closed

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25	S15T-00005, Rev. 0B, - PTF Seismic Analysis: In-Structure Response Spectra (ISRS) 24590-PTF-S0C-S15T-00045, Rev. B, - PTF Seismic Analysis: Enveloped In-Structure Response Spectra	Broadening of the lower bound spectra by +15% and the upper bound spectra by -15% as described on Sheet 5 is a means of eliminating unwanted valleys between the peaks of the spectra for the 3 different soil cases. Use of +30% for lower bound and -30% for upper bound may be even more effective and eliminate some of the manual operations required to produce final ISRS.	Reviewer suggested to increase the spectra broadening from +15% and -15% for lower and upper bound to +30% and -30% could save some of the manual operations in the generations of the ISRS for HLW and PTF. This is a good suggestion. BNI try this suggested procedure when generating ISRS in the future.	
SM-26	24590-PTF-S0C-S15T-00005, Rev. 0B, - PTF Seismic Analysis: In-Structure Response Spectra (ISRS) 24590-PTF-S0C-S15T-00045, Rev. B, - PTF Seismic Analysis: Enveloped In-Structure Response Spectra	Reviewer: Stephen A. Short The frequency interval for in-structure response spectra generation more than adequately meets the requirements provided in ASCE 4.	Observation BNI acknowledges the observation.	Closed
SM-27	24590-PTF-S0C-S15T-00005, Rev. 0B, - PTF Seismic Analysis: In-Structure Response Spectra (ISRS) 24590-PTF-S0C-	Reviewer: Stephen A. Short Vertical oscillators to capture local behavior of concrete beams and steel beams are discussed on Sheet 11. The oscillators are developed to match the frequencies of these structural components as computed	Was BNI; changed to Observation. In discussions with DOE-ORP on 02.15.06, USACE agreed to change their comment to 'observation'. BNI acknowledges the observation.	Closed

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	S15T-00045, Rev. B, - PTF Seismic Analysis: Enveloped In-Structure Response Spectra	in Reference 5 of this document. These frequencies were determined by GTSTUDL modal analyses for typical slab and beam configurations. The development of the oscillator properties is actually shown in Reference 4, the "Seismic Loads" calculation. There are methods for developing single degree of freedom oscillators for multi-degree of freedom structural systems such as two-way slabs and beams. The vertical SDOF oscillators added to the SASSI model to obtain slab and beam vertical ISRS seem to have been developed in a reasonable manner.		
SM-28	24590-PTF-S0C-S15T-00005, Rev. 0B, - PTF Seismic Analysis: In-Structure Response Spectra (ISRS) 24590-PTF-S0C-S15T-00045, Rev. B, - PTF Seismic Analysis: Enveloped In-Structure Response Spectra	Reviewer: Stephen A. Short Enveloped ISRS are used when the individual ISRS are not exactly at the equipment supports or for equipment with multiple supports (e.g., cranes). The enveloping is accomplished by an Excel macro and by visual inspection reasonable results are obtained.	Observation BNI acknowledges the observation.	Closed
SM-29	24590-WTP-DC-ST-04-001, Rev. 3, Seismic Analysis and Design Criteria	Reviewer: James J. Johnson Figures 1 and 2 (Ref. 1) and Slides 17 and 18 (Ref. 2) present the	DOE-ORP 1st Response: Carl Costantino , see attached Response to SM-5 and SM-29.	Open 1st Follow-up: An additional review during the week of 27

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	WTP Structural Modeling Review, Presentation, Abdul et al., 01/09-10/2006	revised DBE ground response spectra for the horizontal and vertical directions. 1. The horizontal DBE response spectra (5% damped) are anchored to a PGA of 0.3g with peak amplification to 0.8g in the frequency range of approximately 4 Hz. to 6 Hz. – amplification factor of 2.67. Due to the relatively narrow frequency range of the peak amplified region, this horizontal DBE response spectrum is narrow-banded. One source of this narrow-banded character of the horizontal DBE response spectra is likely to be local site amplification. However, Slides 23 and 24 (Ref. 2) show soil profiles used in the SSI analyses, specifically shear wave velocity vs. depth to a depth of 350 ft. At a depth of 250 ft., a significant impedance discontinuity exists; shear wave velocities are about 2000 fps and less for soils at depths less than 250 ft. and greater than 3000 fps (for some Ringold formation assumptions greater than 4000 fps) at depths greater than 250 ft. A simple calculation of principal site period or	2nd Response: Ivan Wong: The review comments by Short and Johnson on the shape of the design spectrum are valid comments from a regulatory perspective. However Carl Costantino points out correctly the site-specific reasons for the design spectral shape and so there will need to be	February to 3 March including Rohay, A . C. and S. P. Reidel, "Site-Specific Seismic Site Response Model for the Waste Treatment Plant, Hanford, Washington, prepared by the Pacific Northwest National Laboratory for the US Department of Energy, Office of River Protection, under Contract DE-AC05-76RL01830, March 2005. Figure 3.4.4 compares the horizontal design response spectra developed in 1996 and revised in 2005. Overplotting the CR0098 median horizontal ground response spectra anchored to 0.3g PGA on Fig 3.4.4 demonstrated conservatism in the frequency range of approximately 2.8 Hz. to 8 Hz. when compared to the 2005 revised ground motion. However, in the frequency range below 2.8 Hz., the 2005 revised ground motion "steeply decreases" with frequency and falls below the CR-0098 median, soil ground response spectra. This is important for buildings whose important soil-structure frequencies lie in this range. The PTF building has frequencies of interest in this range.

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		frequency would suggest a site principal frequency of less than 2 Hz. No significant amplification of 2 Hz. motion is observed in the horizontal DBE. The expectation is that: a) If the DBE response spectra shape is narrow banded, it would have significant frequency content at and below 2 Hz. b) If full uncertainty of site characteristics is taken into account, the amplified frequency range is expected to be broader than the 4 – 6 Hz. observed in the figure. 2. The vertical DBE response spectrum (5% damped) is anchored to a PGA of 0.21 g with peak amplification to 0.47 g in the frequency range of 5 – 13 Hz. – amplification factor of 2.24. The frequency-content of this broad-banded spectra near 5 Hz. appears compatible with the lower frequency of the horizontal DBE response spectra of about 4 Hz. The broad-banded nature of this spectrum is expected. Since the revised DBE has been extensively reviewed, there should exist an explanation for the narrow-	some sort of negotiated agreement on the final design spectrum.	In light of this further observation and the expectation that the frequency content of the 2005 revised ground motion should contain amplification in the frequency range around 2 Hz. (see comment 1a), additional explanations should be presented. 2nd Follow-up: JJJ – The specific comments related to broad-band vs. narrow band ground response spectra and the use of the use of spectral amplification values at non-exceedance probabilities other than median values are correctly identified as regulatory issues. The question of the lack of amplified frequency content in the 2 Hz. and less frequency range remains open.

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		banded nature of the horizontal DBE response spectra and the frequency-content whereby the 2 Hz. and below frequencies are lacking.		
SM-30	24590-WTP-DC-ST-04-001, Rev. 3, Seismic Analysis and Design Criteria 24590-WTP-SRD-ESH-01-001-02, Project Safety Requirements Document, Vol. II	Reviewer: James J. Johnson 1. Section 5 of Ref. 1 states that RPP-WTP Facility structures, systems, and components (SSCs) will be categorized as Seismic Category I, II, II, IV, and V based on the failure consequence of their safety functions. Further, Sec. 5 refers to Ref. 2 Appendix A, “Implementing Standards for Safety Standards and Requirements Identification” for hazard evaluation and safety function definition supplemented by engineering analyses. The criteria and implementation of safety and seismic classification should be reviewed. A more detailed review of RPP-WTP Facility structures, systems, and components (SSCs) itemizing their safety and seismic classifications should be performed. Reference 2 and other relevant information should be available. 2. Section 10.5a of Ref. 1 states that a seismic probabilistic risk assessment (PRA) was	Observation BNI will accommodate the reviewer at a time, requested by the reviewer.	Closed Two topics are addressed in this summary item SM-30: (i) Safety and Seismic Classification of Structures, Systems, and Components (SSCs). *This issue will be addressed in the review meetings of 3 – 7 April 2006.* (ii) The performance, parameters, and results of the seismic PRA, which provided the basis for design decisions. *This issue should similarly be addressed in the on-site meetings of 3-7 April 2006.*

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		performed. The assertion is made that the results of the seismic PRA demonstrated the seismic design of the facility has adequate margin to preclude the need for any SSCs to be designated PC-4. Specifically, that the seismic design is adequate assurance of compliance with the radiation exposure standards of Ref. 2. The seismic PRA model and results should be reviewed. 3. During this review period, time was not available to review criteria, implementation, and results of safety and seismic classification of SSCs. Agreement is to schedule time within later scheduled reviews, e.g., equipment qualification review, to cover this topic.		
SM-31	24590-WTP-DC-ST-04-001, Rev. 3, Seismic Analysis and Design Criteria 24590-WTP-SRD-ESH-01-001-02, Project Safety Requirements Document, Vol. II	Reviewer: James J. Johnson Review criteria (Ref. 2), implementation, and results of safety and seismic classification requirements for SSCs. Within Seismic Category I and II systems and components, review the performance requirements of systems and components during and after the shaking. Reference 2 and other relevant information should be available.	USACE	Closed Same as Item SM-30 above.

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		Schedule an independent meeting, or time during a future meeting, to review safety and seismic classification. Schedule time to review criteria (Ref. 2), implementation, and results of safety and seismic classification requirements for SSCs. Within Seismic Category 1 and 2 systems and components, review the performance requirements of systems and components during and after the shaking. Review methods implemented to assure systems and components achieve performance requirements.		
SM-32	24590-WTP-DC-ST-04-001, Rev. 3, Seismic Analysis and Design Criteria 24590-WTP-SRD-ESH-01-001-02, Project Safety Requirements Document, Vol. II	Reviewer: James J. Johnson Review the seismic PRA model, assumptions, and results. Perhaps, the argument that the seismic design provides adequate margin for greater than DBE earthquakes and for population exposure is based on the absence of population in close proximity to the RPP-WTP facility. The statement was made during the plant visit that the facility is designed to walk away from if the DBE occurs. Reconcile the “walk away” philosophy with the results of the seismic PRA. The PSAR and other relevant documents should be available for review.	USACE.	Closed Same as Item SM-30 above.

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		Review the seismic PRA performed – model, assumptions, and results. Review the decisions made based on the results.		
SM-33	24590-WTP-DC-ST-04-001, Rev. 3, Seismic Analysis and Design Criteria	Reviewer: James J. Johnson 1. For facilities comprised of complex systems of diverse mechanical, electrical, and structural elements, systems integration requirements to prevent dimensional interferences are required. 2. These considerations are necessary for specifically designed SSCs, for generically designed and field routed commodities, and for commodities installed under good industry practices. 3. Loading environments include normal operating conditions, non-seismic accident conditions, and DBE conditions. 4. For non-seismic category 1 SSCs, the principal issue is systems interaction, often referred to as II/I, including phenomena such as proximity (impact), falling (impact), spray and flood hazards, etc. due to response and/or failure of non-category 1 SSCs. 5. Procedures to assure no	Observation BNI acknowledges the observation.	Closed This issue will be addressed in the review meetings of 3-7 April 2006.

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		adverse consequences of interactions between SSCs during normal operating conditions, accident conditions, and DBE conditions should exist.		
SM-34	24590-WTP-DC-ST-04-001, Rev. 3, Seismic Analysis and Design Criteria	Reviewer: James J. Johnson During this review period, time was not available to review systems integration aspects of assuring no adverse consequences of interactions between SSCs of all seismic categories. Agreement should be to schedule time within later scheduled reviews to cover this topic. During future meeting, schedule time to review systems integration aspects of assuring no adverse consequences of interactions between SSCs of all seismic categories and for loading combinations that include the DBE.	USACE	Closed This item is the same as SM-33.
SM-35	24590-WTP-DC-ST-04-001, Rev. 3, Seismic Analysis and Design Criteria	Reviewer: James J. Johnson 1. In many multi-disciplinary projects, the division of responsibilities of design and qualification of structures, systems, and components for seismic DBE conditions is unclear. Often this situation is	Observation BNI acknowledges the observation.	Closed This issue will be addressed in the review meetings of 3-7 April 2006.

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		termed a silo effect, where each discipline is focused on their individual responsibilities and interfaces do not receive appropriate attention. 2. Division of responsibilities here is meant to include: a) By disciplines, such as civil, mechanical, electrical, fire protection, and systems engineering. b) By components such as piping, equipment qualification (mechanical and electrical). Other commodities (conduit, cable trays, HVAC duct, etc.), field routed commodities, etc. 3. Interface control is essential to assure smooth implementation of design, construction, and operation of the facility.		
SM-36	24590-WTP-DC-ST-04-001, Rev. 3, Seismic Analysis and Design Criteria	Reviewer: James J. Johnson During this review period, time was not available to review division of responsibilities (as described above) and interface control, i.e., basically, systems integration considerations. Agreement should be to schedule time specifically to review these topics or within later scheduled reviews. During future meeting, schedule time	USACE	Closed This item is the same as SM-35.

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		to review division of responsibilities, interface control, and systems integration approaches for the project.		
SM-37	24590-WTP-RPT-ST-03-001_B, Summary Structural Report, Rev B, HLW Vitrification Building, Volume IIA, Appendix C, Assessment of the Dynamic SSI Model Parameters and Results.	Reviewer: Mansour Tabatabaie This section presents a series of parametric studies performed on HLW Vitrification building using reduced model of the structure to demonstrate adequacy of the methodology used for the dynamic SSI analyses of the HLW building as well as provide some insight into the overall dynamic behavior of the system. The results of the reduced models are compared with those of the detailed structural model used in SASSI production runs. All results are obtained for the original structure configuration (Rev 0B FE model) and original ground motions.	Observation BNI acknowledges the observation.	Closed
SM-38	24590-WTP-RPT-ST-03-001_B, Summary Structural Report, Rev B, HLW Vitrification Building, Volume IIA, Appendix C, Assessment of the Dynamic SSI Model Parameters and Results.	Reviewer: Mansour Tabatabaie Fixed-Base Results: Comparison of fixed-base results in terms of computed in-structure maximum accelerations is presented for models S2 (STRUDL) and F4 (SASSI) but do not include the same results for model S2 (CE933). Model S2 (STRUDL) uses 4% Rayleigh damping anchored to frequencies of 0.5 and 20 Hz (Fig C-	BNI The purpose of this parametric study of different stick and finite element models of HLW building as shown in the referenced document is to provide a general validation for the methodology of analysis and to provide a sanity check for the possible variation of the results. In this regard, all models adopted in this study are simplified, especially the stick models are quite crude, and certain	Closed - within the context of the stated goal exercised on a crude and simple stick model of structure.

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		37), and model F4 (SASSI) uses 4% constant hysteretic damping ratio. Because model S2 (CE933) would use modal damping, it will be useful to assess the difference in dynamic response for different types of damping assigned to the structure. Include similar results for Model S2 (CE933) in Figures C-9 and C-10.	aspects of the structure characteristics are ignored. No attempt was made to fine-tuning the parameters in order to reach a better match among the models for any particular results. Given the assumptions and the simplified models, results of the documented study show that the adopted methodology for the SSI analysis is reasonable and the results of SASSI analysis are within reasonable range, thus the purpose of the study is served.	
SM-39	24590-WTP-RPT-ST-03-001_B, Summary Structural Report, Rev B, HLW Vitrification Building, Volume IIA, Appendix C, Assessment of the Dynamic SSI Model Parameters and Results.	Reviewer: Mansour Tabatabaie SSI Results for Surface Foundation: Comparison of SSI results for model H1 (SASSI), S2 (CE933) and S2 (STRUDL) are shown in Figures C-38 through C-42 for E-W and C-43 through c_47 for N-S direction at different points, respectively. SASSI and CE933 both use same frequency-dependent foundation impedance input and the structure models are comparable. The maximum in-structure accelerations computed from CE933 are consistently higher than those of SASSI at all points by up to 20% for both E-W and N-S directions. The spectral accelerations for SASSI and CE-933 show good agreement for all locations in the E-W direction. However, in the N-S direction, there	BNI See response to SM-38, above.	Closed - within the context of the stated goal exercised on a crude and simple stick model of structure, as stated in SM-38.

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		is large difference in computed spectral accelerations between 2 and 7 Hz. Provide explanation for the large differences observed between SASSI and CE933 model results for surface foundation.		
SM-40	24590-WTP-RPT-ST-03-001_B, Summary Structural Report, Rev B, HLW Vitrification Building, Volume IIA, Appendix C, Assessment of the Dynamic SSI Model Parameters and Results.	Reviewer: Mansour Tabatabaie SSI Results for Embedded Foundation: Comparison of SSI results for various models with embedded foundation are shown in Figures C-48 through C-52 for the E-W direction and C-53 through C-57 in the N-S direction, respectively. The S2 (CE933) results reasonably match those of H1 (SASSI) and F4 (SASSI) both in terms of computed spectral accelerations and maximum accelerations at various levels in the structure. This is in contrast to large differences shown in the results of surface foundation between SASSI and CE933, as discussed in Item 2 above. Provide explanation for improved results going from surface to embedded foundation. This will be significant e.g. in the case of Pretreatment Facility, which is supported at ground surface for the most part.	BNI See response to SM-38, above.	Closed - within the context of the stated goal exercised on a crude and simple stick model of structure, as stated in SM-38.

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SM-41	24590-WTP-RPT-ST-03-001_B, Summary Structural Report, Rev B, HLW Vitrification Building, Volume IIA, Appendix C, Assessment of the Dynamic SSI Model Parameters and Results.	Reviewer: Mansour Tabatabaie General Comment: It will be useful to assess the effect of using different damping ratios, as stipulated for the response level 1 and 2 in Seismic Analysis and Design Criteria (24590-WTP-DC-ST-04-001, Rev 3) on the dynamic response of the structure using the models presented in this section.	BNI See response to SM-38, above.	Closed - within the context of the stated goal exercised on a crude and simple stick model of structure, as stated in SM-38.
SM-42	24590-HLW-S0C-S15T-00001, Calculation Cover Sheet, High Level Waste Vitrification Building: Free Field Analysis 24590-PTF-S0C-S15T-00001, Calculation Cover Sheet, Pretreatment Building Seismic Analysis: Free Field Analysis	Reviewer: Mansour Tabatabaie These calculation documents provide the basis for selecting strain compatible soil shear wave velocity and damping ratios for SSI analyses. DOE has provided revised strain compatible soil properties at the WTP site in conjunction with the revised DBE spectra. Review of the data provided by DOE is not part of the scope of our review work. BNI has performed one-dimensional site response analysis using the original soil profile and properties with the revised ground motions to develop new strain compatible shear wave velocity and damping ratio profiles. These results are compared against those provided by DOE and previously produced by BNI using original design motions. Based on	Observation BNI acknowledges the observation.	Closed

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		these comparisons, strain-compatible properties are selected for SSI analyses.		
SM-43	24590-HLW-S0C-S15T-00001, Calculation Cover Sheet, High Level Waste Vitrification Building: Free Field Analysis 24590-PTF-S0C-S15T-00001, Calculation Cover Sheet, Pretreatment Building Seismic Analysis: Free Field Analysis	Reviewer: Mansour Tabatabaie Although the new ground motions are significantly more energetic than the original ground motions provided by DOE, there is no noticeable difference in the computed strain compatible shear wave velocity profile from two ground motions reported by BNI, as shown in Figure 7-1 in both documents. On the other hand, the strain compatible soil damping ratio show noticeable increase in the top 100 foot soil layer due to larger soil shear strains induced by the new ground motions. This effect should be explained in light of the soil shear moduli and damping ratio versus soil shear strain relationship used for the WTP site in the SHAKE analyses.	Was BNI; changed to BNI/DOE-ORP BNI Response: Comparing the strain-compatible soil data between the original and the updated soil profiles, reviewer is concerned that although the increases in soil strain are relatively small, the increases in soil damping are much larger. From the relative values in the acceleration response spectra of the input ground motion, the Revised Ground Motion has more energy than the original Ground Motion. Therefore, for the same initial low strain soil profile, the calculated strain compatible shear wave velocity is expected to be lower while the corresponding soil damping is expected to be higher using the Revised Ground Motion. These are confirmed by the comparison of shear wave velocity in Figure 7.1 and the comparison of soil damping in Figure 7.2 of the referenced Free Field Analysis calculations for HLW and PTF where Free Field analyses by SHAKE program were performed using the original low strain soil data with the Revised Ground Motion as well as the Original Ground Motion.	Closed

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			Furthermore, at the strain level of these Free Field analyses, the rate of change (slope in the degradation curves) for soil damping is much higher than that for shear modulus. For example, compare the following strain compatible results from two SHAKE analyses with the Original H1 and the Revised H1 Ground Motions as input. At the 2nd soil layer for Lower Bound case (depth at mid-layer = 7.75'), the strain compatible values are: Input Ground Motion Original H1 Revised H1 Uniform Shear Strain 0.00293 0.00386 Shear Modulus 4,095 ksf 3,933 ksf Damping 0.028 0.033 Comparing the above SHAKE results, damping increases by 18% from 0.028 to 0.033, while the shear modulus (G) decreases from 4,095 to 3,933 and the shear wave velocity decreases by the square root of the ratio of G by only 2%.	

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			As the reviewer pointed out, this effect is more pronounced at the top 100 feet of the soil profile. Since these soil profiles are very competent, the reduction in shear wave velocity and the increase in soil damping are small. However, since the absolute values of the soil damping are small, they are plotted in magnified scale in Figure 7.2, and thus appear to have large increases. With closer inspection, the magnitudes of these increases in soil damping are actually all very small.	
SM-44	24590-HLW-S0C-S15T-00001, Calculation Cover Sheet, High Level Waste Vitrification Building: Free Field Analysis 24590-PTF-S0C-S15T-00001, Calculation Cover Sheet, Pretreatment Building Seismic Analysis: Free Field Analysis	Reviewer: Mansour Tabatabaie The large difference between strain compatible damping profiles provided by DOE and those computed by BNI using the new ground motions should be explained. In addition, the rationale for selecting the higher damping values for near surface material in the SSI analyses should be explained in light of the fact that the time histories used by BNI in their SHAKE analyses is spectra compatible and, as a result is expected to be more energetic that those considered by DOE; thus resulting in higher rather than lower damping as compared to DOE results.	BNI Reviewer addressed two issues: Comparing the strain-compatible soil damping between those provided by DOE and those calculated using SHAKE program. Provide rationale for selecting the set with the higher damping (near ground surface) values to be used in SSI analyses. There are many differences between the Free Field Analyses performed to calculate the soil data for DOE and those performed in BNI Free Field calculations that contributed to the difference in soil damping. Some of the known differences are listed below: • Seismic motion in DOE	Closed

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			analyses used convolution technique while BNI used deconvolution. • DOE analyses used multiple input motions while BNI used a single set of seismic motion. • Shapes of ARS of DOE input motions are much different from those BNI input motions. • Soil degradation curves as well as soil layer thickness used in the iterative solutions are different between analyses performed for DOE and by BNI. Based on the differences in the input data listed above, the calculated strain compatible soil damping values are expected to be different. However, these damping values are very small relative to the overall damping within the SSI analyses so that they are expected to have minor effect on the calculated SSI responses. Also, since the method used to calculate the set of soil damping from DOE is consistent with that used in generating the Revised Ground Motion (RGM), the slightly higher damping values from DOE are used along with the RGM in the SSI analyses of HLW and PTF.	
SM-45	24590-HLW-S0C-S15T-00007,	Reviewer: Mansour Tabatabaie	Observation	Closed

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	Calculation Cover Sheet, High Level Waste Vitrification Building Seismic Analysis: SSI Analysis 24590-PTF-S0C-S15T-00003, Calculation Cover Sheet, Pretreatment Building Seismic Analysis: SSI Analysis	This document provides some details on the SSI analyses of the HLW building using computer program SASSI. The SASSI model development details and assumptions are not described herein. Therefore, the comments provided below only reflect the information regarding SASSI analysis results presented herein and some limited discussions with Thomas Ma, BNI project engineer, at this time.		
SM-46	24590-HLW-S0C-S15T-00007, Calculation Cover Sheet, High Level Waste Vitrification Building Seismic Analysis: SSI Analysis 24590-PTF-S0C-S15T-00003, Calculation Cover Sheet, Pretreatment Building Seismic Analysis: SSI Analysis	Reviewer: Mansour Tabatabaie Soil Profiles: The SSI model used the strain compatible shear wave velocity and damping ratio calculated from the free field site response analyses. These properties are only valid in the absence of the structure. The High Level Waste (HLW) and Pretreatment (PT) Facility buildings are relative large structures that will impose an additional net uniform vertical bearing pressure after completion. For example HLW will impose an additional pressure on the order of 3 ksf over the entire footprint of the structure (256 ft by 326 ft) after completion causing further consolidation and increase in effective confining pressure to a large depth. The additional	BNI The following details in the site response analysis address the concerns of the reviewer in this comment: 1. The site soil profile does not start from the grade surface. As stated in the Rev. 1 of the free-field analysis calculation, “the loose Dune Sand at the top portion of the profile was assumed removed and the grade level was assumed to be at the top of the Upper Hanford Sand”, for the WTP site the thickness of this removed loose layer is around 10 – 15 ft. 2. The initial shear wave	Closed. See additional comments below: 1. Item 1 can reduce the effect of additional net pressure imposed by buildings by about 1.1 to 1.65 ksf. 2. The residual confining pressure of 8.9 ksf may not be appropriate within the context of the formula presented. 3. The argument in Item 3 does not apply to pretreatment (PT) facility building, which is mainly a surface supported structure.

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		confining pressure will increase the low strain shear modulus that will result in higher soil shear wave velocity, in particular in the upper 30 ft. This effect which is more predominant right below the foundation basemat will increase the soil stiffness and decrease the effect of soil radiation damping, thus allowing more energy of seismic waves to be imparted into the structure and less energy to be dissipated as a result of structure feedback into the foundation soil; thus, causing higher structural response in particular at higher frequencies. This effect has not be considered and/or addressed in the SSI analyses. Perform analyses, as appropriate to assess the magnitude of increase in soil stiffness due to the presence of the HLW and PTF structure. If the increase in soil stiffness is found significant, the soil properties below the foundation should be properly adjusted in the SSI analyses to account for this effect.	velocity profile used in the free-field analysis is defined with a number greater than 0 at grade even though the top portion of the profile is classified as "sand." During the analysis process, this number is not adjusted to be compatible with the confining pressure at any particular depth. This implies that in the analysis model a certain amount of residual confining pressure is retained in the soil and the soil skeleton has been pre-consolidated under this pressure. Any additional overburden that is less than or equal to this pre-consolidation pressure would not likely cause additional compression of the soil. For example, in the original lower bound soil profile, determined per ASCE 4-98, Vs = 1143 ft/sec. for the top 26 ft. soil. Using the formula $G = 1000\ K_2\ (\sigma_m')^{1/2}$ and assuming $K_2 = 61$, $K_0 = 0.4$ and $\gamma = 110$ psf, this Vs value corresponds to the	The reviewer, however, believes that this effect may be accounted for within +/-50% variation of low strain shear modulus used in the SSI analyses.

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			pre-consolidation pressure of about 8.9 ksf., or about 81 ft. of overburden soil. The Vs values for mean and upper bound soil profiles would imply even higher pre-consolidation pressure values. 3. As pointed out by the reviewer, The HLW building imposes an average of 3 ksf of overburden pressure on the soil layers beneath the basemat. However, this pressure is largely compensated by the removal of the top 27 feet of soil (21 ft. for the design depth of the structure and 6 ft. for the thickness of the basemat), thus the overall increase of confining pressure for the underlying soil profile is negligible. Consider the above facts, it is judged that the increase of shear modulus of soil due to additional confining pressure imposed by the buildings is insignificant, and would have only minimal effect on the overall SSI response of the structures.	
SM-	24590-HLW-S0C-	Reviewer: Mansour Tabatabaie	BNI	Closed. See additional

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47	S15T-00007, Calculation Cover Sheet, High Level Waste Vitrification Building Seismic Analysis: SSI Analysis 24590-PTF-S0C-S15T-00003, Calculation Cover Sheet, Pretreatment Building Seismic Analysis: SSI Analysis	Hard Rock Analysis of the Converted Structural Model: In comparing the hard rock predominant modes of SASSI versus fixed-base modes of STRUDL, it should be clarified whether these modes refer to undamped or damped frequencies of the system. For example, the SASSI modes appear to be damped while the STRUDL modes are expected to be undamped as is the general case in the modal analysis. Damped modes are generally higher than their undamped counterparts. In the case of PT, the damped SASSI modes (fx=8.3 Hz and fy=5.0 Hz) are shown to be slightly higher than the STRUDL undamped modes (fx=8.2 and fy=4.9 Hz), which indicates correct trend. However, for the HLW structure, the trend is reversed. Not explicitly clarified in the write up. Clarify the type of reported structural modes in terms of being damped or undamped. In addition, discuss the magnitude of any frequency shift effect in comparing damped versus undamped modes from SASSI and STRUDL, respectively.	Reviewer is concerned with any differences in damping that may exist and any resulting frequency shift between the SASSI Hard Rock model and the GT/STRUDL Fixed-Base model for HLW and PTF. First, a fixed-base finite element model of HLW for dynamic analysis is developed using GT/STRUDL computer code. Then this GT/STRUDL finite element model is converted into SASSI finite element model, mainly by changing each node and each element from GT/STRUDL format into SASSI format. To simulate the fixed boundary condition of the GT/STRUDL model, (very stiff) Hard Rock soil properties are used in this SASSI model. Then, transfer functions that are calculated from the SASSI Hard Rock model and compared with the fixed-base modal frequencies calculated from the GT/STRUDL model. For the fixed-base finite element model and for the Hard Rock SASSI model, most of the model damping is from material damping where it is set at uniform 4% damping in both models. At this low magnitude of damping, it is expected that damping has very minor effects on structural frequencies. The minor difference between the modal	comments. 1. From the response provided it is understood that the GT/STRUDL frequencies correspond to damped modal frequencies and, therefore, can be directly compared with damped frequencies derived from SASSI transfer functions. 2. Because the selected frequencies from fixed-base GT/STRUDL and hardrock SASSI analyses for comparison can not be associated with the same mode and/or behavior of the system in the two models, the frequency shift and/or difference can not be explained.

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			frequencies of the fixed-base model and the frequencies at the peaks of the transfer functions of the Hard Rock model are likely caused by the following reason: HLW facility is large in horizontal dimension relative to its vertical dimension. At each major floor slab elevation, major reinforced concrete slabs spread out horizontally are supported by shear walls. Since at various horizontal locations, these shear walls have different dimensions and thus locally have different natural frequencies. Therefore, the HLW model has many local frequencies. However, since different locations of HLW can be linked together by reinforced concrete slabs; horizontal natural frequencies at any one location on the slab can be influenced by natural frequencies at many other locations. When choosing a natural frequency for the fixed-base finite element model of HLW to compare with Hard Rock SASSI model, a single frequency in each of the two horizontal directions is chosen based on the mode that has the largest mass participation. This is over simplified since as stated above, multiple frequencies are expected at a single location.	

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			For the same reason, a single location in the Hard Rock SASSI finite element model for HLW also is expected to have influence from multiple frequencies. Therefore, transfer functions contain wide peaks and/or multiple peaks, which indicate more than a single frequency. The purpose of comparing a single modal frequency with the peak of the transfer function at the center of slab from the Hard Rock model is to validate the model conversion from GT/STRUDL into SASSI. Some differences in frequencies are to be expected. The same argument is also applicable to PTF.	
SM-48	24590-HLW-S0C-S15T-00007, Calculation Cover Sheet, High Level Waste Vitrification Building Seismic Analysis: SSI Analysis 24590-PTF-S0C-S15T-00003, Calculation Cover Sheet, Pretreatment Building Seismic Analysis: SSI Analysis	Reviewer: Mansour Tabatabaie Hard Rock Analysis of the Converted Structural Model: No ARS comparison of the hardrock SASSI versus fixed-base STRUDL model is provided. This comparison is important to show the effect of different types of structural damping used in the SASSI and STRUDL models. Not presented and/or discussed. Provide comparison of ZPA and in-structure ARS response of hardrock	BNI 1st Response: Reviewer is concerned with any differences in damping that may exist between the SASSI Hard Rock model and the GT/STRUDL Fixed-Base model for HLW and PTF can result in discrepancy in the calculated acceleration responses. The conversion of the GT/STRUDL finite element model (FEM) into SASSI finite element model mainly involved the reformatting of the nodes and elements within the model. Each element of the GT/STRUDL FEM is converted into one	Open 1st Follow-up: The reviewer believes such comparison will be useful to show that the dynamic response of the structure is adequately modeled using frequency domain method used in SASSI. Further elaborations follow: Section 7.3.1 of the calc documents (“Hard Rock Analyses of the Converted Structural Model”) states that the converted SASSI structural

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		SASSI and fixed-base STRUDL to assess the effect of using constant hysteretic damping on the high frequency response of the structures.	identical element in SASSI FEM. This conversion is verified by checking the input data, by inspecting the un-deformed geometry plots of the model, and also by comparing the fixed-base modal frequencies calculated from GT/STRUDL FEM with the transfer functions calculated from the Hard Rock SASSI FEM, which is discussed in SM-47. However, agreement or disagreement from comparison of response accelerations in terms of ZPA and ARS between the GT/STRUDL FEM Fixed-Base analyses and the Hard Rock SASSI analyses do not affect the accuracy of the SSI responses. Therefore, these comparisons are not performed.	model has essentially the same <u>dynamic characteristic</u> as the GT/STRUDL structural model, confirming the correctness of the model conversion. The conclusion is only based on comparing global predominant frequencies of the two systems in the x, y and z directions. Other effects such as structural damping, other global and local modes, interaction between different modes, etc. are not reflected in the above comparison. These are important factors that can affect the response of the structure. Such effects are not addressed in the SSI methodology validation, as discussed in Items 38, 39 and 40 because of the simple structural model used. The reviewer understands that the accuracy of the finite element structural model conversion from GT/STRUDL to SASSI has been confirmed by comparing the fixed-base (or hard rock) frequencies of the two models. But the point here is to go beyond the model conversion and actually look at the adequacy of the modeling assumptions and frequency

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			2nd Response: As discussed in the teleconference between the Project and the reviewers (on May 11th), in the development of the SASSI dynamic models, GT/STRUDL is used essentially as a development tool with interactive graphics and databases to show model geometries and properties. The GT/STRUDL program and the GT/STRUDL dynamic model are not used for any seismic analysis to generate any responses for structural design. Therefore, a detailed comparison of dynamic characteristics of the GT/STRUDL and SASSI models is not necessary since this comparison would serve no purpose for the structural design. The reviewer agreed to re-visit the structural summary report and make an assessment of the SASSI results against those developed using the beam stick model and lumped SSI parameters.	response solution for a very detailed structural model used in SASSI where the factors listed above become important. This is very different than analyzing a simple structural stick model. Because fixed-base (or hardrock) structural models mainly reflect on the structural response without including SSI effects, this is a good place where the GT/STRUDL and SASSI detailed models can be compared to assess the effect of the factors listed above. However, this can only be done by comparing the ZPA and in-structure response spectra (ISRS) at numerous points in the structure that include global in-plane as well as out-of-plane and local modes responses. This comparison will directly affect the accuracy of the SSI response later on. 2nd Follow-up: The reviewer has revisited Appendix C of the SSR to further assess whether the documented results are adequate to verify the accuracy of the SASSI procedure in modeling a detailed structure such as that of HLW. As stated

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				before, this assessment is not simply looking at the correct conversion of the structural finite element model from GT/STRUDL to SASSI format. The goal is to know whether the dynamic response of the complex structural system in terms of the response quantities sought and used for design is adequately captured using complex frequency response analysis method. The reviewer acknowledges that extensive verification of the structural model parameters and foundation impedance and scattering characteristics have been performed and documented in a well organized manner in Appendix C. This provides confidence that the structural model has been properly translated into SASSI finite element model and the frequency-dependent foundation impedance and scattering properties are correctly modeled in the SSI analysis using SASSI. It is further shown that the SSI response in terms of global, maximum horizontal acceleration and horizontal ISRS obtained at and/or near mass center of

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				various floors reasonably agrees between detailed finite element SASSI model of the HLW structure and those using stick models of the structure. In reviewer's opinion as much as Appendix C reflects on the correct model translation and foundation dynamic stiffness and damping characterizations for SASSI SSI analyses, it does not provide the assurance that the general dynamic response of the detailed structural model is adequately captured by SASSI using complex frequency response procedure. Based on the results presented in Appendix C, it can only be concluded that the detailed SASSI model of HLW structure adequately predicts the global, maximum horizontal acceleration and horizontal ISRS at the mass center of various floors. The question as to whether a detailed comparison of dynamic characteristics of the GT/STRUDL and SASSI models are or are not necessary depends on the type of response quantities obtained from SASSI and used for design. In

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				response to SM-50, BNI states that only global SASSI responses have been used for design and therefore, the comparisons of global responses of detailed SASSI model with those obtained using stick model, as provided in Appendix C of SSR should suffice. However, the reviewer has observed that the floor accelerations at all nodal points of flexible slab have been obtained from SASSI and used to develop bubble acceleration plots that are subsequently used for floor slab, shear wall and roof framing design. Vertical ISRS of local slabs have also been calculated from SASSI and used for system and equipment qualifications. In addition, to better capture the vertical response of the flexible floor slabs in SASSI analyses, SDOF systems (oscillators) have been developed and added to detailed finite element models of the HLW and PTF buildings. All of these necessitate the need for better understanding of the modeling issues in complex frequency response analysis of detailed structural model, as done in SASSI.

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SM-49	24590-HLW-S0C-S15T-00007, Calculation Cover Sheet, High Level Waste Vitrification Building Seismic Analysis: SSI Analysis 24590-PTF-S0C-S15T-00003, Calculation Cover Sheet, Pretreatment Building Seismic Analysis: SSI Analysis	Reviewer: Mansour Tabatabaie Modification of SASSI Structural Model to SSI Model: SASSI Run of the Modified SSI Models: The cut-off frequency selected for the SSI analyses are 10.5 Hz for the LB case, 17.5 Hz for the mean case and 22 Hz for the UB case for the HLW structure. For the PTF structure the cut-off frequencies are 21 Hz for the LB case, 25 Hz for the mean case and 33 Hz for the UB case. In general, it is desirable to have the cut-off frequency no lower than 25 Hz to ensure that all significant modes of the structure are captured in the SSI analyses. This may also be significant for calculation of stresses and forces in the structural elements and for the response of the secondary systems and equipment. Not addressed and/or discussed in terms of its impact on the analysis results. Perform sensitivity analyses to assess the impact of using low frequency cutoff on the dynamic response of the HLW structure, systems and equipment (SSE). Alternatively, the SASSI foundation model may be refined to allow the analyses to be carried out to	BNI 1st Response: This issue is more critical in the HLW building since a lower cutoff frequency is used than the one used in the PTF building for each of the three soil cases. Therefore only the HLW model will be discussed. Based on the modal analysis results, the cumulative mass participation for the fixed-base HLW model are: 90% at 13.8 Hz, 95% at 20.6 Hz, and 98% at 26.8 Hz. In addition, the overall SSI responses of the building are dominated by the SSI frequencies which are much lower than the predominant fixed-base modal frequencies of the building. Therefore, even with the lower cutoff frequencies used in the HLW SSI analysis, it is believed that the calculated results have captured the responses of the structure model sufficiently. 2nd Response: As agreed in the teleconference between the Project and the reviewers (on May 11th), acceleration response spectra at 4 SDOF oscillators, which have natural frequencies of 7, 12, 15 and 20 Hz, are calculated for all soil cases from the PTF building analysis. Figures SM49-1 through SM49-4 show the results of comparison. In the figures, the original 5% damped ARS for LB, M	Open 1st Follow-up: The reviewer understands the rationale provided for using a low cutoff frequency in the SSI analyses. But in reviewer's opinion the acceleration response of higher frequency modes in the structure (say above 13.8 Hz) may not necessarily be controlled by the fact that 90% of cumulative mass participate in response of the structure below 13.8 Hz to justify using a low cutoff frequency in the SSI analyses. Similarly, a cut-off frequency of 10.5 Hz in SASSI may not capture the acceleration response of modes that are excited at frequencies above 10.5 Hz. It seems reasonable that some sensitivity studies re needed to ensure that the use of such low cut-off frequencies are not affecting the dynamic response of the HLW structure, systems, and equipment (SSE). 2nd Follow-up: The reviewer has looked at the comparison of vertical ARS curves calculated for the oscillators with 7, 12, 15

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		frequencies of 25 Hz or higher, if deemed necessary.	and UB soil cases with cutoff frequencies at 21, 25 and 33 Hz, respectively, are plotted together with the envelope of the three curves. In addition, the ARS for the LB soil case is recalculated with a cutoff frequency at 10.5 Hz. This last curve is also plotted in the figures. It is shown clearly from the figures that in the ARS of all four SDOFs with different natural frequencies, the LB results are never dominant in the frequency range higher than 6 Hz. Therefore, the cutoff frequency at 10.5 Hz for LB soil case does not affect the final ISRS results.	and 20 Hz frequencies, which are shown on Figures SM49-1, SM49-2, SM49-3 and SM49-4, respectively. As expected, the maximum acceleration and the shape of response spectra are not affected for low cut-off frequency of 10.5 Hz for LB soil case when the frequency of the oscillator is below 10.5 Hz. However, when the frequency of the oscillator is higher than the cut-off frequency, both the maximum acceleration and shape of spectral response are significantly affected (with the exception of maximum acceleration in Figure SM49-4). The reviewer expects that the same trend would hold for the UB and M soil cases. The cut-off frequency selected for the SSI analyses of HLW building are 10.5 Hz for the LB soil case, 17.5 Hz for the M soil case and 22 Hz for the UB soil case, respectively. If the acceptance criteria for using a low-cut off frequency for LB soil case is to be tied to the enveloped results of all three soil cases (i.e. UB, M and UB) rather than looking at each soil case separately as stated by BNI in

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				the teleconference between the project and reviewers on May 11th, then the effect of the selected cut-off on all soil cases need to be examined. It is, therefore, suggested that BNI should produce similar results for the UB and M soil cases for the PTF building using cut-off frequency of 22 and 17.5 Hz, respectively. These results should be plotted against the previous results for comparison. Furthermore, the reviewer would like to see more sample results for oscillators with higher dominant frequency. Calculation document “24590-HLW-S0C-S15T-00015, HLW Floor Slab and Roof Beam Vertical Frequencies for SSI Analysis, No. 0C” shows about 100 oscillators developed for each cracked and uncracked slab section. About 30% of these oscillators have predominant frequency between 20 and 33 Hz. In general, selecting one sample oscillator result at 20 Hz is not representative of the collection especially when it shows that the dominant frequency is more at 9.5 Hz rather than 20 Hz. It is

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				suggested that BNI produces 5 additional ARS results of oscillator responses with dominant modes above 17 Hz with at least 3 with dominant modes above 20 Hz for further evaluations. These results should be presented with the above frequency cutoffs for further assessment of the effect of frequency cut-off on response of these oscillators.
SM-50	24590-HLW-S0C-S15T-00007, Calculation Cover Sheet, High Level Waste Vitrification Building Seismic Analysis: SSI Analysis 24590-PTF-S0C-S15T-00003, Calculation Cover Sheet, Pretreatment Building Seismic Analysis: SSI Analysis	Reviewer: Mansour Tabatabaie The calculations do not provide detailed information on the SASSI structural model. For example, • Were trailing zeros added to the input motion for SASSI analyses. • How many frequencies are calculated in the SASSI analyses before interpolation. • How many local modes exist in the SASSI model after adding SDOF models. • How are local mode responses adequately captured by number of computed frequencies to provide stable results. • Are closely spaced modes	BNI 1st Response: The comments are well taken and more details of the analyses will be addressed in the next revision of the calculation. For reviewer's information, the following details are used in the analysis: • Trailing zeros are added to the input time histories. All input time histories are generated for 2048 points and the analyses are all performed for 4096 points. • The calculated frequencies in all SASSI analysis are in general determined as following: (1) Every 0.5 Hz from 0 to 10-15 Hz, starts from 0.024 Hz;	Open 1st Follow-up: The reviewer is mainly concerned that for the level of structural detail used in the SASSI model and analyses, adequate number of frequencies is computed at proper spacing with proper frequency resolution to reasonably capture the dynamic response of the structure. To get such assurance, it will be necessary to demonstrate the stability of the response transfer functions at numerous points in the structure. In addition, a comparison of the structural response using SASSI and GT/STRUDL or SAP2000 may be necessary to confirm the adequacy of the computed

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		accurately calculated with computed frequencies and frequency resolution. Not found in the calcs Provide information and demonstrate stability of the results.	(2) Every 1 Hz from end of (1) to the cutoff frequency; and (3) After calculating at all of the above frequencies, combine the results and inspect the transfer functions at major floors and critical locations of the buildings. Then calculate at additional frequencies as needed. In general, the SSI responses of the models are calculated at 40 – 60 frequencies for different soil cases. At the frequency range of major structural responses, the frequency increment is usually 0.25 Hz or less. • It is not possible to determine all local modes in the SASSI models since the DOFs of both HLW and PTF models exceed 30,000. However, careful inspection of transfer functions at control points and major SDOFs of the building indicates that the number of calculated frequencies is sufficient to avoid omission of any major structural modes and to remove most of the interpolation-induced spikes.	frequencies and stability of the transfer functions. In general, it is the reviewer's opinion that 40 to 60 computed frequencies before interpolation may not be adequate to properly capture all the local structural modes of the HLW and PT SSI systems in the SASSI analyses performed. 2nd Follow-up: With respect to the sample transfer functions

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			2nd Response: The SASSI model is developed for the purpose of generating the seismic responses required for structural design. These responses are nodal accelerations and in-structure response spectra for equipment design. The model is not intended to develop local responses for detail design. With these objectives, the SASSI model is considered adequate for SSI analysis. Seismic response of a massive concrete shear wall structure is controlled by a few structural modes as expected and as evident from the transfer function results obtained from SASSI. Many nuclear structures have been and continue to be analyzed using the beam stick model that captures the major modes of the structure and thus the global response of structure adequately. The refinement of the model and inherent inclusion of many modes of vibration does not imply the global responses are controlled by all modes of vibration. The response at every nodal point is controlled only by a few modes of vibration. Comparison of the SSI results with simple lumped parameter model in the structural summary report provides the assurance that the global response of the structure is captured adequately. As for the questions of stability of transfer functions, the proper spacing of calculated frequency points, and whether	presented in Figures SM50-1 and SM50-2 of the response provided, it appears that the mean (M) soil response governs at the frequency of the oscillator. One may have thought that the upper bound (UB) soil case would have governed since it would have driven the oscillator closer to a fixed-base condition. The reviewer would like an explanation as to whether this is caused by the ground response spectral shape or interpolation effects (the same phenomenon is observed in ISRS plots shown in Figures SM49-1 and SM49-2 in response to SM-49). The fact that there may be two closely-spaced peaks near the resonant frequency of the oscillator is also obvious from the LB transfer function shown in Figure SM50-1. The reviewer is wondering if one or two additional frequency points were computed near the peak of the transfer function for the mean or UB soil case and added to the response, it would not alter the results (in other words, the transfer function would remain stable and the peak is not missed for the UB soil case).

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			the number of calculated frequency points is sufficient, it is best to illustrate through an example as shown in Figure SM50-1, the ZZ-responses at Node 16995, an oscillator with natural frequency 7 Hz. It is shown clearly from the figure that the predominant responses at this node for all soil cases occur at the frequency range of 5-10 Hz. As explained in previous response to the comment, the calculated frequency points at this range are at every half Hz, i.e., at 5, 5.5, 6, 6.5, 7, 7.5, 8, 8.5, 9.0, 9.5, 10.0 Hz for all three soil cases. Figure SM50-2 shows expanded view of this frequency range for Figure SM50-1. It can be observed clearly from the figure that the calculated frequency numbers are sufficient and are spaced appropriately to capture all major variations of the transfer functions.	BNI has further noted that local responses from SASSI analyses are not used for design. The reviewer has observed that maximum accelerations at all nodal points of the flexible floor slabs have been obtained from SASSI and used to develop bubble acceleration plots, which are subsequently used for floor slab, walls and roof beam designs. Vertical ISRS of local floor slabs have also been calculated from SASSI and used for systems and equipment qualifications. In addition, to better capture the vertical response of the flexible floor slabs in SASSI analyses, SDOF systems (oscillators) have been developed and added to detailed finite element models of the HLW and PTF buildings. The question with respect to accuracy of the SASSI model to capture the vertical acceleration of the structural nodes to be used for design of floor slabs and roof framing, and the vertical response of the floor slabs using SDOF systems to be used for equipment qualifications still remain open and require further elaborations and

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				demonstrations.
SM-51	24590-WTP-S0C-S15T-00002, Calculation Cover Sheet, Generation of DBE Time Histories, No. 2	Reviewer: Mansour Tabatabaie This calculation documents the generation of Design Basis Earthquake (DBE) time histories for the revised ground motions for seismic analyses of the WTP buildings. The fit is performed such that the generated time histories meet ASCE Standard 43-05, "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities."	Observation BNI acknowledges the observation.	Closed
SM-52	24590-HLW-S0C-S15T-00001, Rev 2, 1, 0, High Level Waste Vitrification Building: Free Field Analysis	Reviewer: John North Original Issue calculation and subsequent revisions were reviewed. Revision 2 represents the modifications made as a result of the revised seismic design criteria provided by the Office of River Protection (ORP). Revision 2 utilizes the modulus degradation and damping ratio relationships provided in Revision 1 of this calculation. Revision 1 of this calculation indicates that the modulus degradation and damping ratio relationships recommended in the Shannon & Wilson Geotechnical Report (WTSC99-1036-42-17) for sand and rock soil strata was used.	Observation BNI acknowledges the observation.	Closed

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		This same Revision 1 indicates that for the gravel layer, the modulus/damping relationships as recommended by Rollins et al were utilized for the analysis.		
SM-53	24590-HLW-S0C-S15T-00001, Rev 2, 1, 0, High Level Waste Vitrification Building: Free Field Analysis	Reviewer: John North The modulus degradation and damping ratio relationships utilized in the Free Field Analysis (FFA) for sand were based on those recommended in the Shannon & Wilson Geotechnical Report (SWGR). These values are those recommended for Generic ENA sites (EPRI Figures 7.A-18 and Figure 7.A-19). While these curves represent generic, industry-accepted values they may not be representative of site-specific values. Governing codes and regulations (ASCE 4-98, DOE STD 1020, and DOE STD 1022) for Performance Category (PC) 3 facilities like the HLWF state that shear modulus and damping curves should be developed based on site-specific testing results and supplemented as appropriate by published data for similar soils. These same documents indicate, “Sufficient laboratory test data should be obtained to allow for reasonable assessments of median values of soil properties and their	DOE-ORP/USACE The Seismic Design Review Panel is recommending a program of dynamic laboratory testing of the overburden soils, the Hanford sands and gravels, Ringold Formation, and basalt and interbeds. The testing will consist of resonant column and torsional shear testing of 3 Hanford sand, 10 interbed and 2 basalt samples; large-diameter resonant column testing of 5 Hanford gravel and 2 Ringold samples, and 50 free-free resonant column testing of 50 samples. The purpose of the testing is to provide data to develop site-specific shear modulus reduction and damping curves.	JSN Followup – Reviewer acknowledges approach proposed by DOE-ORP/USACE and believes the proposed testing will provide the required site-specific dynamic properties. The newly acquired site-specific dynamic properties should be evaluated and compared to the generic properties used in both the site response analysis and the SSI analysis; this is critical for determining the acceptability of these two analyses. Comment is closed.

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		potential variability." (DOE STD 1022 5.5.1.2). Consider the feasibility of conducting site specific testing to characterize modulus degradation and damping ratio relationships for all soil, gravel, and rock at the site.		
SM-54	24590-HLW-S0C-S15T-00001, Rev 2, 1, 0, High Level Waste Vitrification Building: Free Field Analysis	Reviewer: John North The modulus degradation and damping ratio relationships utilized in the FFA for gravel were based on those recommended by Rollins et al. The leading experts in the subject field believe that the Rollins' curves misrepresent the strain-dependent behavior of gravels and depict them, as being excessively linear because the date used by Rollins did not adequately establish the low-strain modulus. Use of these curves will lead to erroneous results. In addition, governing codes and regulations (ASCE 4-98, DOE STD 1020, and DOE STD 1022) state that shear modulus and damping ratio relationships should be developed based on site-specific testing results and supplemented as appropriate by published data for similar soils. The same codes and regulations indicate that for coarse geologic materials, like those being discussed in this comment, that	DOE-ORP/USACE See SM-53 resolution	Comment closed per SM-53 resolution

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		special testing equipment and facilities should be used. The codes further say that only if it is not feasible to collect samples and perform these special tests, that dynamic properties from published data should be estimated. Considering the developing state of the practice, a re-evaluation should be made as to the feasibility of collecting site specific samples for use with specialized large-diameter testing equipment and procedures, with the ultimate goal of generating site specific modulus degradation and damping ratio relationships for site gravels. Compare site-specific test results to those used in the free-field ground motion development and soil structure interaction analysis. If site-specific tests results are enveloped by the range considered than previous analysis results should be considered satisfactory.		
SM-55	24590-HLW-S0C-S15T-00001, Rev 2, 1, 0, High Level Waste Vitrification Building: Free Field Analysis	Reviewer: John North The Rev 1 FFA calculation refers to the SWGR for modulus degradation and damping ratio relationships for rock soil strata. There were no statements found in the report indicating relationships for rock soil strata. Figures 5-2 and 5-3 of the	DOE-ORP/USACE See SM-53 resolution	Comment closed per SM-53 resolution

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		Rev 1 FFA provide modulus degradation and damping ratio relationships for rock. Where did these relationships come from? If site-specific data cannot be obtained, the EPRI (1993) curves for gravel, perhaps modified in accordance with more recent studies conducted at the University of Texas, are to be preferred to the Rollins curves.		
SM-56	WTSC99-1036-42-17 (H1616-51), Final Report Geotechnical Investigation, May 11, 2000, Shannon & Wilson, Inc.	Reviewer: John North Development of subsurface static and dynamic soil properties. The methodology used to develop these properties should be evaluated for consistency with the published standards, should be in accordance with the standard of practice for similar types of facilities, and in accordance with the governing regulations. Some of the properties provided in the Geotechnical Report are based on simplified methods. These same properties form the basis for the majority of structural analysis and design. The importance level of the proposed PC-3 structure implies that the current state of practice methods of developing these properties be utilized. Use of other than site-specific and project-specific	Observation	All but last two sentences of review comment - Comment closed per SM-53 resolution. Last two sentences of review comment – Comment remains Open. There remains lack of evidence that the work presented in the geotechnical report was performed in accordance with an approved NQA-1 program. The reviewer has received notification that some 3rd party review was

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		developed properties could lead to different outcomes for the ground response analysis, which forms the basis for the structural design. Without site-specific properties it cannot be determined if the ground response analysis is accurate or possibly over- or worse yet, under-conservative. It also cannot be determined where the site specific properties fit within the range considered in the evaluation or if they are outside of the range considered. There were no statements found in the report indicating that the work was performed in accordance with an approved quality assurance program. There were no statements found in the report indicating that a third-party peer reviewer or review team reviewed the work for acceptance with standard practice.		performed during the PNNL study. The reviewer has also received documentation prepared by BNI endorsing the original seismic report by Geomatrix and the geotechnical report by Shannon and Wilson. However, it is not clear that these reviews meet the NQA-1 requirements for a documented independent 3rd party review.
SM-57	WTSC99-1036-42-17 (H1616-51), Final Report Geotechnical Investigation, May 11, 2000, Shannon & Wilson, Inc.	Reviewer: John North The site characterization performed as part of the geotechnical investigation is detailed in its spatial distribution of subsurface investigations.	Observation	Closed
SM-58	WTSC99-1036-42-17 (H1616-51), Final Report Geotechnical Investigation, May 11,	Reviewer: John North The depth to which the deeper borings were drilled was insufficient	Observation	It is the reviewer's understanding that additional deep borings will be drilled to characterize the subsurface stratigraphy at the

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	2000, Shannon & Wilson, Inc.	to define a complete characterization down to the basement rock, which forms the input depth for ground response analysis.		site, to collect samples to resolve comment SM-53, and to allow for full column, in-situ, seismic velocity testing to be performed. It is recommended that any new site work performed should be done so under an approved NQA-1 program and be reviewed by an independent 3rd-party reviewer or review team. Comment open awaiting confirmation of this understanding and response to this recommendation..
SM-59	WTSC99-1036-42-17 (H1616-51), Final Report Geotechnical Investigation, May 11, 2000, Shannon & Wilson, Inc.	Reviewer: John North The sampler type used in the borings did not afford an opportunity to collect less disturbed samples for undisturbed index and geomechanical testing.	Observation	It is assumed that specialized drilling methods and sample collection systems will be employed during the work associated with comments SM-53 and –58. The methodologies proposed should be reviewed for acceptance and or alternatives. Comment open awaiting confirmation of this assumption and response to this recommendation.
SM-60	WTSC99-1036-42-17 (H1616-51), Final Report Geotechnical Investigation, May 11, 2000, Shannon & Wilson, Inc.	Reviewer: John North The frequency of using the "relatively undisturbed" sampler (i.e. ring-lined barrel sampler) seems low leaving some doubt in the certainty and confidence of the laboratory results that were developed based	Observation	As material unit weight is a direct input into both the site response analysis and SSI analysis, it is prudent that appropriate means and methods be employed to measure this property. It is recommended that the future drilling, sampling, and testing

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		on testing of samples recovered with this method.		programs discussed in SM-53 and –58 be established to facilitate the measurement of this property with confidence and certainty. Comment open awaiting response to this recommendation.
SM-61	WTSC99-1036-42-17 (H1616-51), Final Report Geotechnical Investigation, May 11, 2000, Shannon & Wilson, Inc.	Reviewer: John North The Standard Penetration Test (SPT) values noted in the report were collected using variable methods including two different types of hammers and two different types of drive samplers. One of the samplers, the 2" O.D. split-spoon sampler is in compliance with ASTM Test Method D1586. However, the other sampler, the 3" O.D. split-spoon sample/ring sampler is not in compliance with D1586. The two different hammers used for sampling likely produce variable dynamic striking energies (this point was actually noted in Section 4.1 of the report) that can influence the SPT blow counts (N-values) measured during the test. Section 7.7 of the report states that statistical analysis was performed to evaluate correlations between Seismic Cone Penetrometer Studies (SCPT) and SPT $(N_1)_{60}$ values, and that the correlations were based on five pairs of SCPTs and borings performed.	Observation	The reviewer's comment stands and remains Open

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		The report indicates that borings BD-8, -23, -35, -47, and –54 were used for the correlation. The report notes that the "SPT N-values measured in the borings were converted to $(N_1)_{60}$ values using Equation 7.2." It is not clear if 1) all N-values provided in the report, including those measured in borings other than the BD borings indicated in Section 7.7, were converted to $(N_1)_{60}$ values, and 2) if the N-values reported on <u>all</u> boring logs are corrected values. Also in Section 7.7, the conclusion that the energy delivered by the two different hammer systems used was approximately equal (this contradicts earlier statements made in Section 4.1 of the report noted above) and that the assumption that the hammer efficiency ratio was 60 percent, may be overstated and thus could be misleading. Published literature provides very specific means to assign hammer energy efficiencies for the different types of hammers used. This appears to have not been considered. In summary, the N-values (or $(N_1)_{60}$ values) reported and any subsequent related correlations could be misleading or erroneous.		
SM-62	WTSC99-1036-42-17 (H1616-51), Final Report Geotechnical	Reviewer: John North The geophysical data while being	Observation	Open – See follow-up for last two sentences of SM-56.

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	Investigation, May 11, 2000, Shannon & Wilson, Inc.	extensive appears to not have been reviewed by an independent 3rd-party peer reviewer. This type of data is highly influenced by the methods used, the completion of the borehole casings installed, and the interpretations made. This data forms much of the basis for the dynamic properties presented in this report. As such the characterizations made using this data generally require expert review and evaluation.		
SM-63	WTSC99-1036-42-17 (H1616-51), Final Report Geotechnical Investigation, May 11, 2000, Shannon & Wilson, Inc.	Reviewer: John North The modulus degradation and damping ratio relationships recommended in the report for sands are those recommended for Generic ENA sites (EPRI Figures 7.A-18 and Figure 7.A-19). While these curves represent generic, industry-accepted values they may not be representative of site-specific values. Also, these curves are shown based on depth ranges, presumably below ground surface. Considering the relatively low unit weight of the site soils indicated in the report, these curves may be even more unrepresentative of the true site-specific values. Governing codes and regulations (ASCE 4-98, DOE STD 1020, and DOE STD 1022) state that modulus	DOE-ORP/USACE See SM-53 resolution	Comment open awaiting resolution of comment SM-53.

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		degradation and damping ratio relationships should be developed based on site-specific testing results and supplemented as appropriate by published data for similar soils. These same documents indicate, "Sufficient laboratory test data should be obtained to allow for reasonable assessments of median values of soil properties and their potential variability." (DOE STD 1022 5.5.1.2). Lack of site-specific modulus degradation and damping ratio relationships for site soils. The impact to the design process could be significant as this issue affects the ground motion definition as well as the soil structure interaction analysis.		
SM-64	WTSC99-1036-42-17 (H1616-51), Final Report Geotechnical Investigation, May 11, 2000, Shannon & Wilson, Inc.	Reviewer: John North The modulus degradation and damping ratio relationships recommended in the report for gravels are those recommended for Gilroy 2 (EPRI Figure 6.B-3). While these relationships likely represent a best estimate based on published data, they may not be totally representative of site-specific values. Governing codes and regulations (ASCE 4-98, DOE STD 1020, and DOE STD 1022) state	DOE-ORP/USACE See SM-53 resolution	Comment open awaiting resolution of comment SM-53.

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		that modulus degradation and damping ratio relationships should be developed based on site-specific testing results and supplemented as appropriate by published data for similar soils. The same codes and regulations indicate that for coarse geologic materials, like those being discussed in this comment, special testing equipment and facilities should be used. The codes further say that if it is not feasible to collect samples and perform these special tests, that dynamic properties from published data should be estimated. Considering the developing state of the practice, a re-evaluation should be made as to the feasibility of collecting site specific samples for use with specialized large-diameter testing equipment and procedures with the ultimate goal of generating site specific modulus degradation and damping ratio relationships for the gravels at the site. Lack of site-specific modulus degradation and damping ratio relationships for site soils. The impact to the design process could be significant as this issue affects the ground motion definition as well as the soil structure interaction analysis.		
SM-	WTSC99-1036-42-17	Reviewer: John North	DOE-ORP/USACE	

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65	(H1616-51), Final Report Geotechnical Investigation, May 11, 2000, Shannon & Wilson, Inc.	Site specific testing for modulus degradation and damping ratio relationships have not been performed in accordance with DOE and industry standards or as appropriate for a project of this importance and magnitude. Recommend site specific testing.	See SM-53 resolution	Comment open awaiting resolution of comment SM-53.
SM-66	24590-HLW-S0C-S15T-00008, Seismic Loads 24590-PTF-SOC-S15T-00004, Seismic Loads	Reviewer: Thomas D. Wright The HLW bubble plots on sheets 9, 11, 41, and possibly others have accelerations up to 2.7g with adjacent bubbles indicated as low as 0.65g. This seems like a very large change in acceleration for adjacent nodes. After investigation, it appears the reason for this large difference is that some of the acceleration nodes are at the mid-point of individual members. After discussion and consultation with Bechtel personnel, it was concluded that this is reasonable. PT bubble plot sheets 8, 9, and 12 have similar large variations in accelerations.	Observation BNI acknowledges the observation.	Closed
SM-67	24590-HLW-SOC-S115T-0006, HLW Vitrification Building Seismic Analysis - Structural Model.	Reviewer: Thomas D. Wright Selection of Poisson's ratio may result in conservative or unconservative estimates of shear displacements. Sheet 11. The shear modulus, G, is	Observation BNI acknowledges the observation.	Closed

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		sensitive to the Poisson's ratio and therefore the shear displacements are sensitive to this value. The value used (0.17) is based on the value given in ASCE 4-98. It is assumed that this is appropriate but there is no discussion of this.		
SM-69	24590-WTP-DC-ST-04-001 Seismic Analysis and Design Criteria.	Reviewer: Thomas D. Wright Para 8.2 The ACI 530-99 code is unconservative since does not include a maximum reinforcing steel ratio, has unconservative lap splice length requirements, and has out of date minimum seismic detailing requirements.	BNI As use of masonry is no longer foreseen, BNI will revise the Seismic Analysis and Design Criteria (24590-WTP-DC-ST-04-001) to remove masonry design requirement. After such a revision, this document will then align to the latest Structural Design Criteria (24590-WTP-DC-01-001) revision. Estimate revision will be issued 04.05.06.	Closed
SM-70	24590-WTP-DC-ST-01-001, Rev. 10, Structural Design Criteria ENG-DECS-05-066, Consideration of Thermal and Seismic Loads in Reinforced Concrete Buildings 24590-WTP-DC-ST-04-001, Rev. 3, Seismic Analysis and Design Criteria 24590-HLW-S0C-S15T-00006, Rev. D,	Reviewer: Thomas D. Wright The reviewed documents address the temperature range that is to be used for design and the effects of this temperature on reinforced concrete, including cracking. The papers do not address the design for steel expansion and contraction. The normal operating thermal movements for this 520+ feet long structure are about 1.75 inches, assuming half of the expansion each direction from the center. This amount of movement cannot be	BNI 1st Response: The actual thermal growth of the PT facility is much less than 1.75-in., and thermal growth effects on structural steel are accounted for in the steel design. The documents listed as reviewed as part of the comment SM-70 do not explain how thermal expansion is considered for steel design. First, the 1.75-in. expansion of PT facility concrete stated in the reviewer's comment appears to be based on a temperature delta of 43º F (difference	Open. 1st Follow-up: See response to resolution notes below.

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	HLW Verification of Building Seismic Analysis-Structural Model	easily accommodated in the design. If an attempt is made to restrain this movement (which is not a good idea) then the analysis must include these loads.	between the nominal base slab temperature of 113º, which is the normal operating temperature, and 70º base temperature). Over 520-ft, a 43º F delta temperature equates to 1.75-in. However, the actual thermal growth of the structure is much less than 1.75-in for two reasons: 1. The nominal temperature of the base mat (bulk thermal effect) used in the finite-element model (FEM) of PT is not 43º; the bulk temperature is approximately 27º. The basemat top surface is approximately 113º, but the soil temperature is 80º. The average mid-line slab temperature is 97º, which results in a temperature input to the FEM of 27º (97 - 70 base temp.). 2. The 1.75-in. expansion is based on an unrestrained slab. However, the concrete is restrained by the horizontal soil springs under the basemat and restrained by walls and diaphragms. Of course, it is the restraints on free expansion of the concrete that results in thermal stresses.	The comment was directed towards structural steel expansion, not concrete expansion. The upper part of the structure is structural steel. Structural steel supports the roof, which will be subject to a 43° temperature variation. Roof temperatures in a summer sun can approach 150° and may be subject to sub freezing temperatures in the winter. The comment was intended to address structural steel expansion and contraction, not the concrete. The comment was intended to address structural steel expansion and contraction, not the concrete.

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			A review of the PT facility FEM shows maximum normal operating concrete expansions of approximately 0.8-in., with most of the structure having expansions much less than 0.8-in. Second, thermal growth of the concrete will be reflected in structural steel design because the steel model is attached to the concrete FEM. The FEM will provide design loads for steel design, including thermal expansion of the concrete. The steel beams that support concrete slabs are typically not included in the FEM, but these beams are used to support weight of wet concrete until the concrete cures. After curing, the concrete slabs take the large percentage of the additional live loads, etc, and the steel beams take only a small percentage. The beams that support concrete slabs for construction range from 18 to 54-ft in length. Concrete expansion over these lengths has minimal impact on steel design. 2nd Response: add BNI response (still outstanding)	This does not directly address the reaction forces caused by steel expansion and contraction. Your response says that the FEM does not include the design loads for steel thermal expansion. The issue here is that steel responses to temperature changes much faster than does concrete because steel is a better conductor and has less mass than concrete. On the drawings that I saw, members were connected together with no provision for expansion or contraction. The accumulative effect is and effective member length of over 500 feet, not 54 ft. 2nd Follow-up: add TW response TBD
SM-71	24590-WTP-DC-ST-04-001 Seismic Analysis and Design Criteria.	Reviewer: Thomas D. Wright Para 8.2. This provision does not require the use of reinforcing steel for masonry design.	BNI As use of masonry is no longer foreseen, BNI will revise Seismic Analysis and Design Criteria (24590-WTP-DC-ST-04-001) to remove masonry design requirement. After such a revision, this	Closed

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			document will then align with the latest Structural Design Criteria (24590-WTP-DC-01-001) revision. Estimate revision will be issued 04.05.06.	
SM-72	24590-PTF-SC-S15T-00002, Rev 7, PTF-Structural Model for SSI Analysis	Reviewer: Thomas D. Wright In Appendix C of the PTF document, partition loads have been included through out the structure. It seems unlikely that partitions will be used as this is not office or living space. The total weight of the partition loads in the building is about 5.6 million pounds. This is a small percentage of the building weight but it is a source of conservatism when the vertical distribution of the load is considered.	Observation. BNI will take comment into consideration as Category B Conservatism.	Closed
SM-73	24590-HLW-S0C-S15T-00006; Structural Model 24590-HLW-S0C-S15T-00007; SSI Analysis 24590-HLW-S0C-S15T-00008; Seismic Loads 24590-HLW-S0C-S15T-00009; In-Structure Response Spectra 24590-PTF-S0C-S15T-00002;	Reviewer: John Connor In general, the seismic analysis is well documented and straightforward. The assumptions used are reasonable and it appears that the designers have taken great care to validate assumptions and results. The structural model is very large and the post-processing output will be enormous. It appears that many analysis engineers will be reassigned to other projects. With	BNI A calculation is presently being generated which documents the format of the design forces and moments used for concrete design. This format has been coordinated with the BNI concrete design team and the standard reinforced concrete design tools. For every section cut, a table is generated showing these values for each basic load. These basic loads will be combined and factored in the standard reinforced concrete design tools in accordance with the design criteria. A calculation is also being	Closed

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	Structural Model 24590-PTF-S0C-S15T-00003; SSI Analysis 24590-PTF-S0C-S15T-00004; Seismic Loads 24590-PTF-S0C-S15T-00005; In-Structure Response Spectra	key analysis engineers gone, it will be difficult for design engineers to read the output effectively, and mistakes may occur during interpretation of the output. Is there a formal procedure that instructs designers on how to access the analysis output and how to interpret the data? Examples that should be considered: how to access data, how to filter data, will actual results be used or will designer use the largest value for all designs?, Will info be provided to the designer from the analyst, or will designer need to filter through data on there own? Will the designer need to apply further load factors and load combinations, or does the SAP output already consider that?	generated which validates these design tools.	

Figure SM-7-1

HLW Vitrification Building -- Finite Element Model for Dynamic Analysis - Plan View at El. 0'

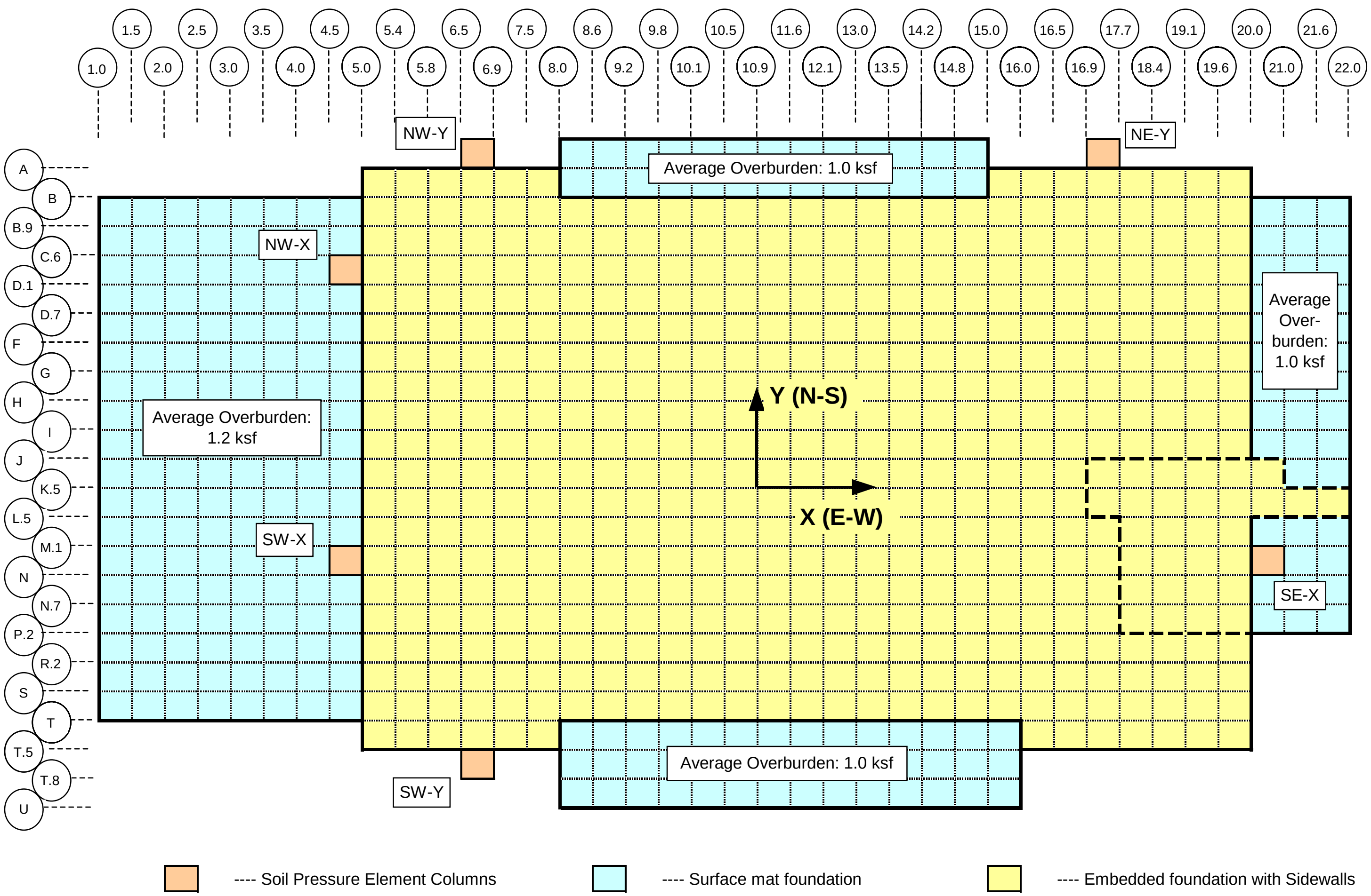


Figure SM-7-2

N-S (Y) Soil Pressure Time History. NW-Y Column, El. -1.25

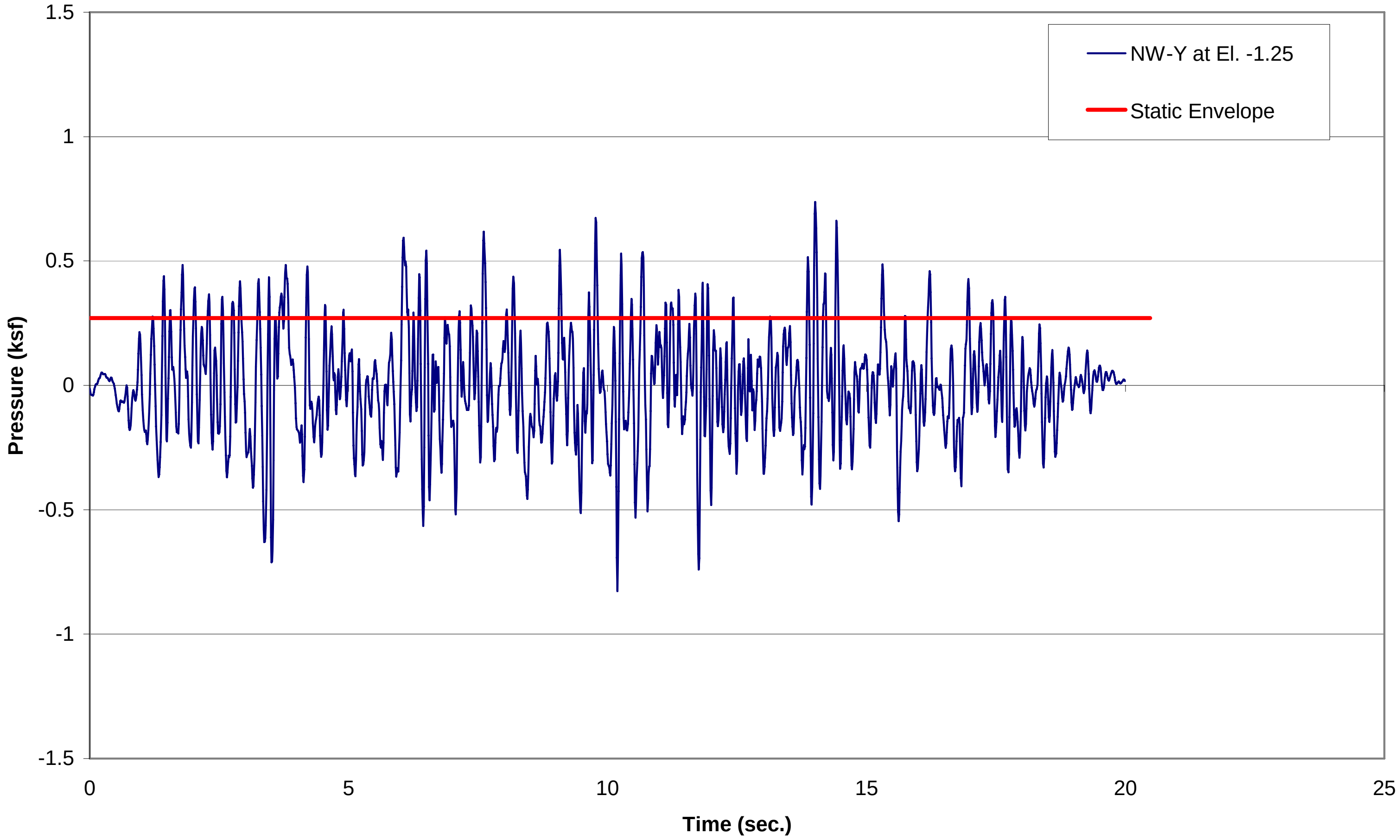


Figure SM-7-3

N-S (Y) Soil Pressure Time History. SW-Y Column, El. -17.25

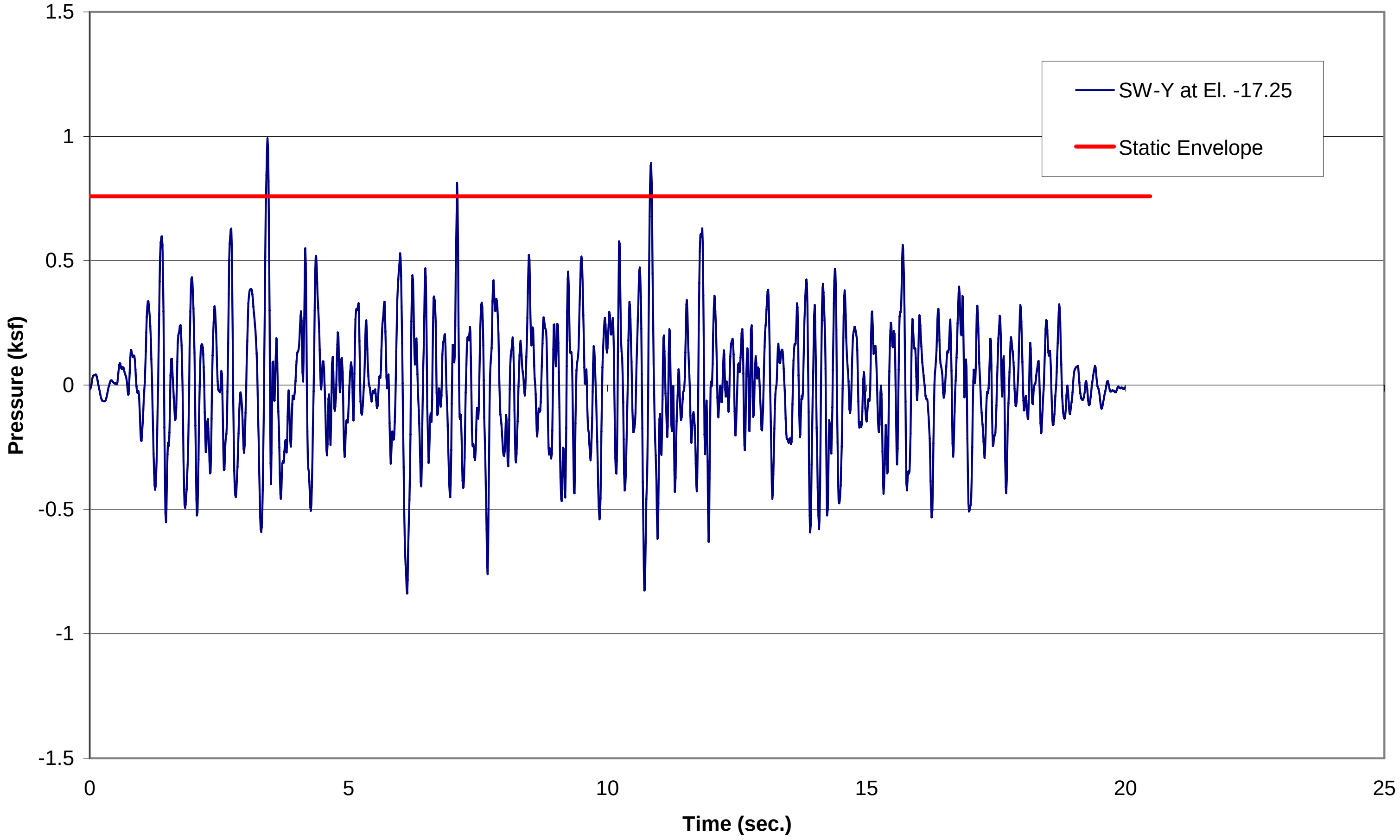
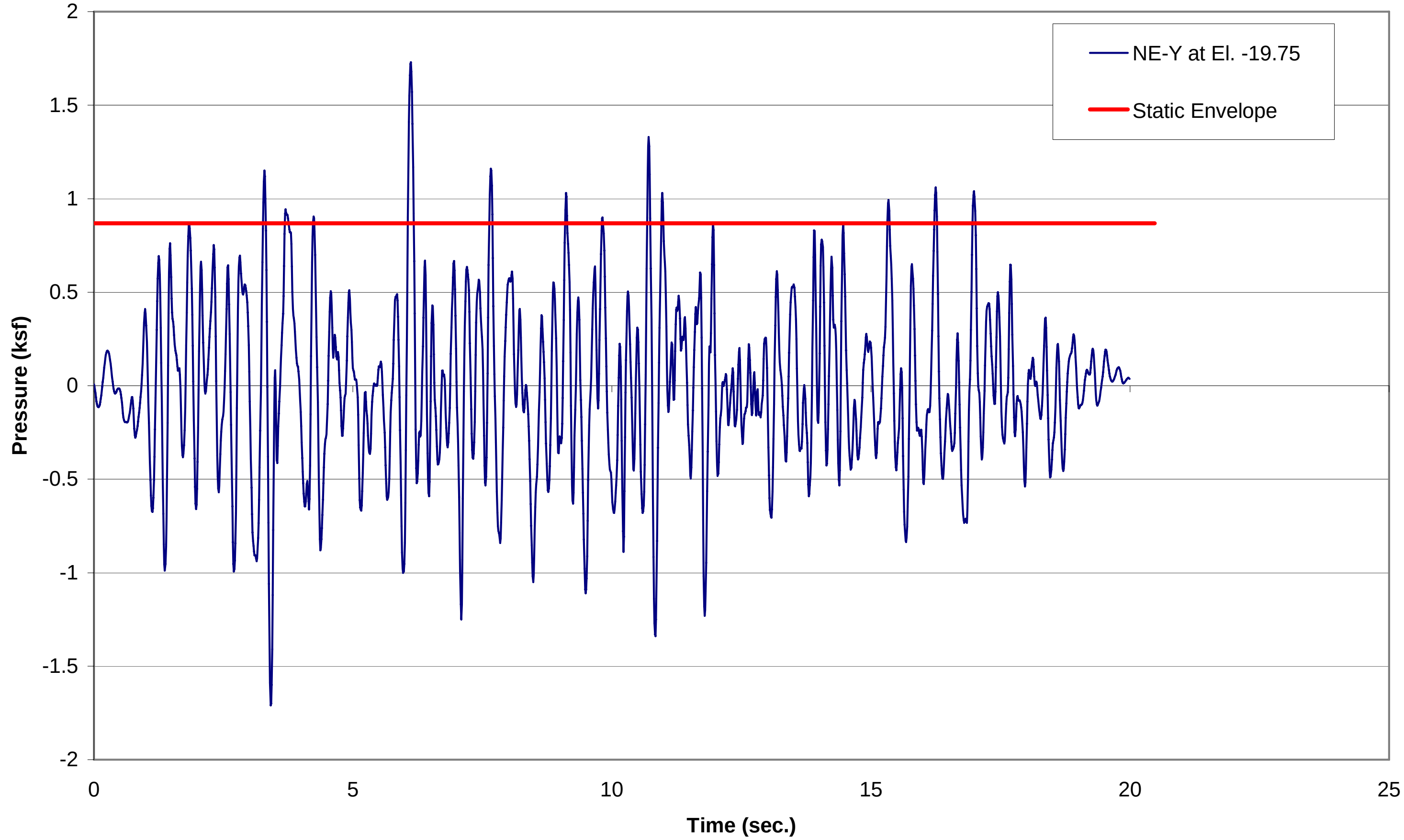
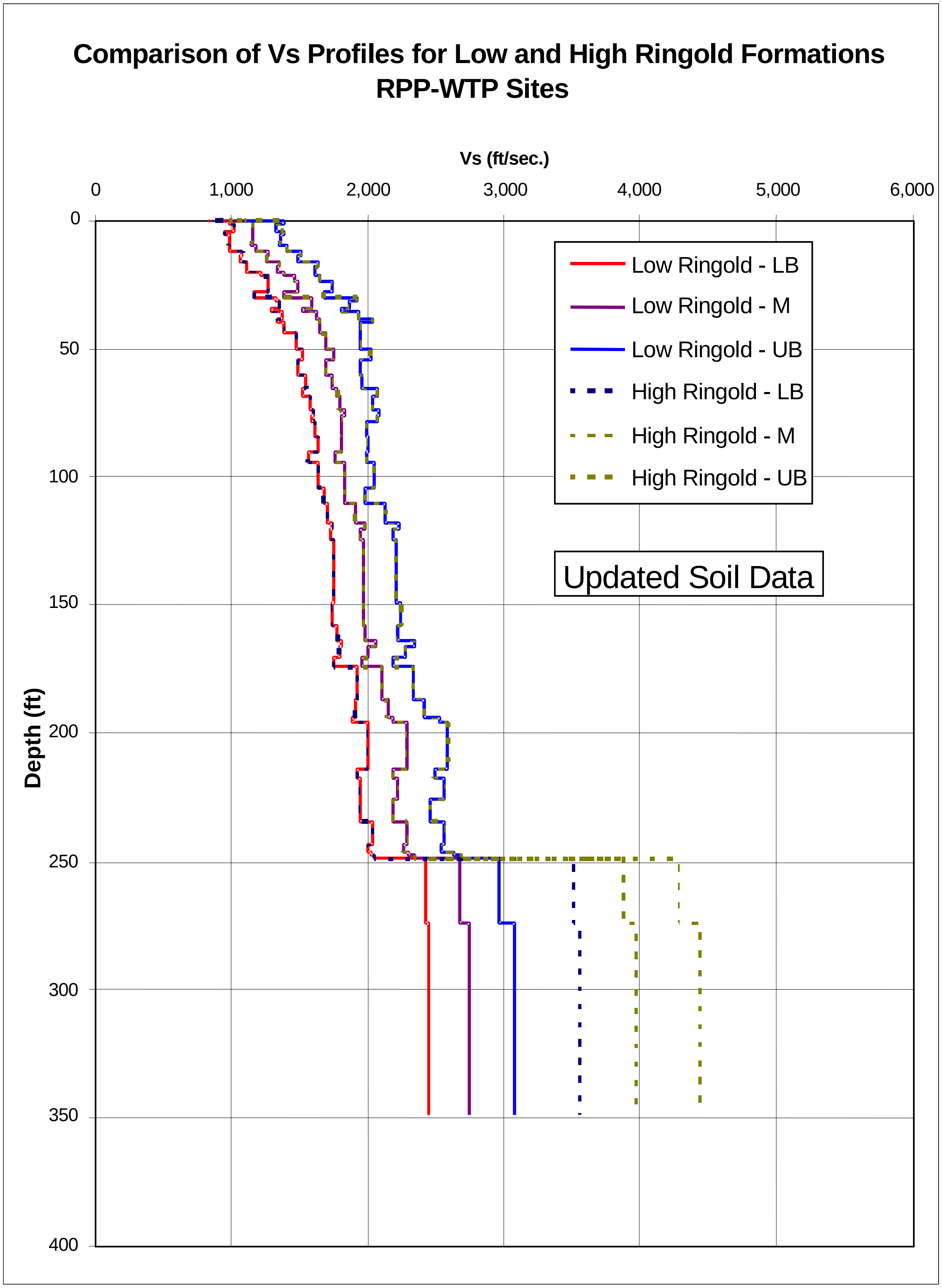


Figure SM-7-4

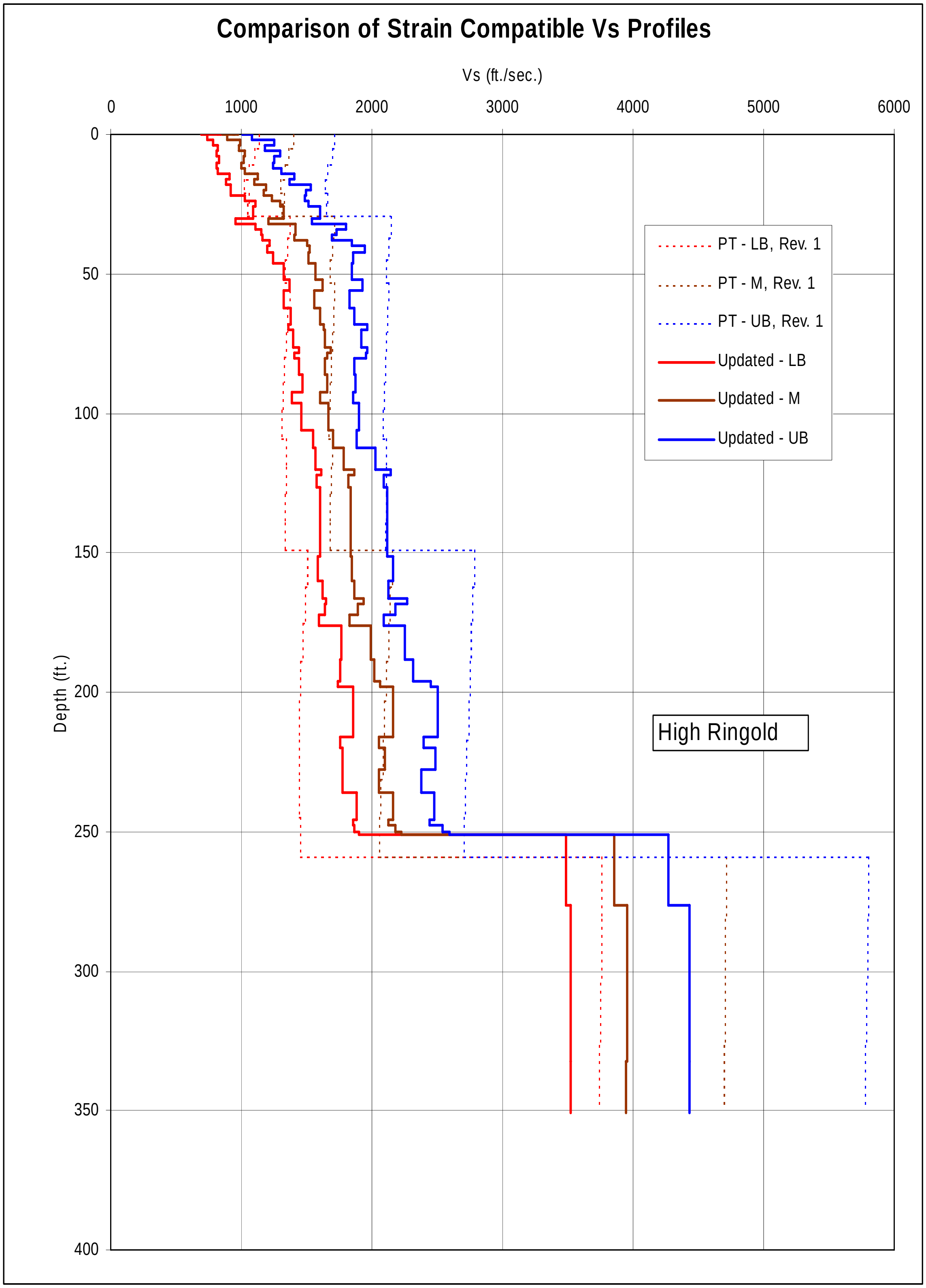
N-S (Y) Soil Pressure Time History. NE-Y Column, El. -19.75



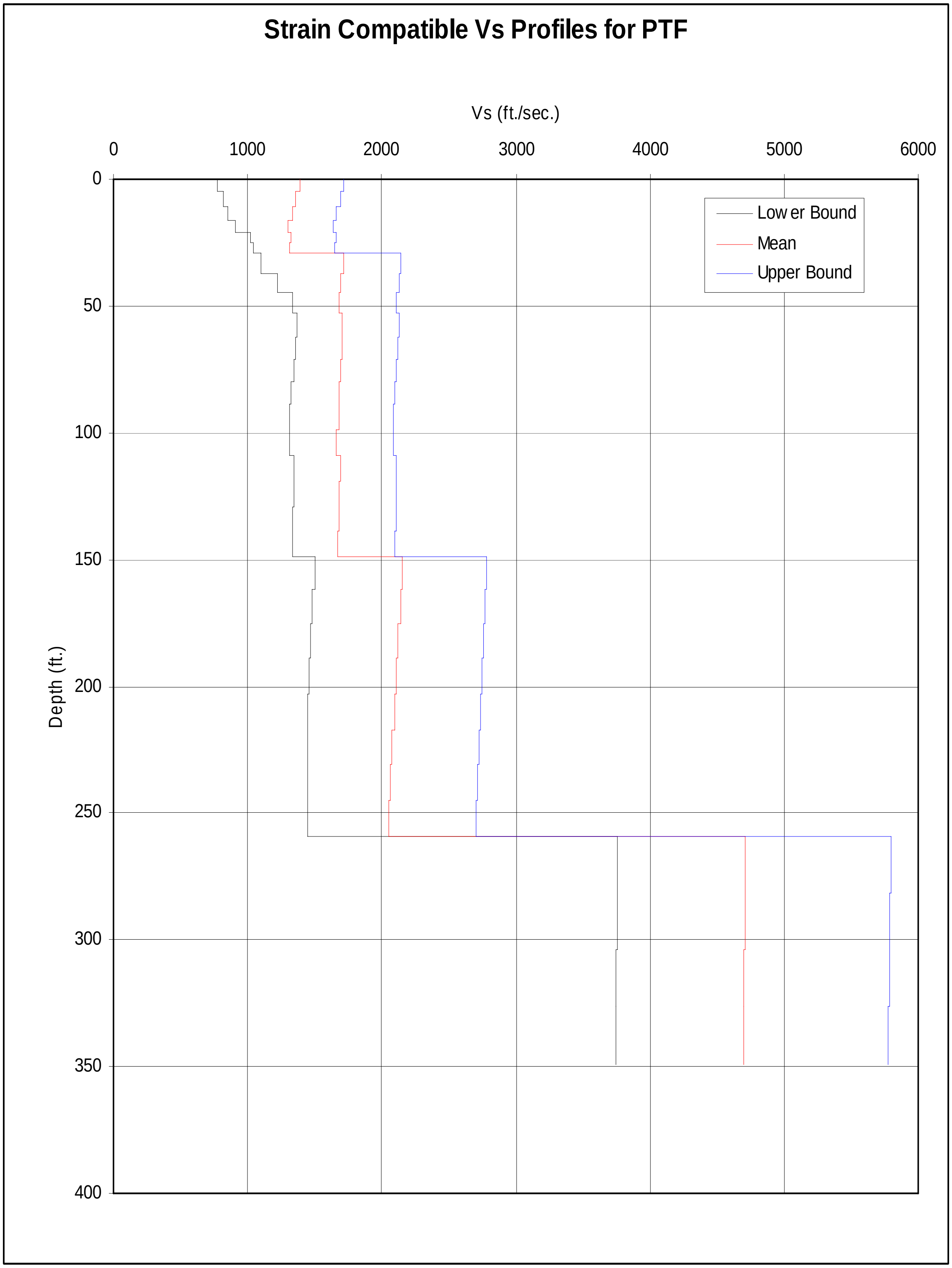
Slide 23 of the January 9, 2006 presentation (refer to SM-12)



Slide 24 of the January 9, 2006 presentation (refer to SM-12)



Slide 26 of the January 9, 2006 presentation (refer to SM-12)



Response to SM-5 and SM-29
C. J. Costantino

1. COE Comment:

The comments provided are as follows:

1. The horizontal DBE response spectra (5% damped) are anchored to a PGA of 0.3g with peak amplification to 0.8g in the frequency range of approximately 4 Hz to 6 Hz, - amplification factor of 2.67. Due to the relatively narrow frequency range of the peak-amplified region, this horizontal DBE response spectrum is narrow-banded. One source of this narrow-banded character of the horizontal DBE response spectra is likely to be local site amplification. However, Slides 23 and 24 (Ref 2) show soil profiles used in the SSI analyses, specifically shear wave velocity vs. depth to a depth of 350 ft. At a depth of 250 ft., a significant impedance discontinuity exists; shear wave velocities are about 2000 fps and less for soils at depths less than 250 ft. and greater than 3000 fps (for some Ringold formation assumptions greater than 4000 fps) at depths greater than 250 ft. A simple calculation of principal site period or frequency would suggest a site principal site frequency of less than 2 Hz. No significant amplification of 2 Hz motion is observed in the horizontal DBE. The expectation is that:

a) If the DBE response spectra shape is narrow banded, it would have significant frequency content at and below 2 Hz.

b) If full uncertainty of site characteristics is taken into account, the amplified frequency range is expected to be broader than the 4 - 6 Hz observed in the figure.

2. The vertical DBE response spectrum (5% damped) is anchored to a PGA of 0.21g with peak amplification to 0.47g in the frequency range of 5 to 13 Hz – amplification factor of 2.24. The frequency content of this broad-banded spectrum near 5 Hz appears compatible with the lower frequency of the horizontal DBRE response spectra of about 4 Hz. The broad-banded nature of this spectrum is expected.

4. The revised horizontal DBE response spectrum has a peak spectral acceleration of 0.8g and peak ground acceleration of 0.3g. This is a maximum amplification of about 2.7, which appears to be conservative (i.e. 84-percentile amplification). The revised horizontal DBE response spectrum appears to have the same frequency content as the original ground motion (i.e. peak amplification at about 5 Hz) but be generally 10 to 20 percent greater at all frequencies. Then the conservative amplification has been superimposed on the new spectrum to create a relatively narrow banded design spectrum. DOE-STD-1020 specifies that median amplification be used with no conservative bias. The reason for this specification is that the introduction of conservatism in spectral amplification is frequency dependent with greater conservatism in amplified regions and no added conservatism away from the amplified regions. Use of spectral conservative amplification has also lead to an undesirable design spectral shape that has relatively steep slopes below 4 Hz and above 6 Hz. There is considerable uncertainty in seismic analysis of structures including the structural

stiffness, mass and resulting stiffness. Steep slopes in the design spectra mean relatively large changes in structural input with variation in structural frequencies. To avoid this situation, DOE-STD-1020 specifies that design spectra shall be smooth and broad-banded

3. Response:

The responses to these comments address several different issues and may be responded to as follows. First, by way of (minor) clarification, the PGA associated with the revised surface ground response spectrum is 0.293g, not 0.3g. The original PGA recommended for the site was 0.257g.

Current procedures for developing ground response spectra for facility design have undergone significant change since the original development of the DOE-STD-1020 generic guidance. The ground response spectrum shown in Figure 2-2 of DOE-STD-2002 is a median shaped NUREG/CR-0098 developed by evaluating limited information available in the 1970's from the WUS empirical database. The guidance recommended attaching a median shaped spectral amplification factor to the mean estimate of site PGA to arrive hopefully at a mean estimate of design spectra.

Figure 1 shows both median and 85th percentile spectral shapes associated with the 0098 spectra recommended for either rock or soil sites. Appended to the plot is a spectral shape currently being recommended by both NRC and DOE for CEUS hard rock sites. These eastern shapes are much more skewed to high frequencies and clearly are very much steeper in the frequency ranges of general structural interest. Figure 2 presents the same data plotted on a log-linear plot. The issue of peakedness of these shapes is clearly subjective. The important issue to keep in mind is that the design spectra must be developed to capture the primary responses associated with the potential effects of site characteristics. The use of very broad-banded spectra, such as RG 1.60 shapes, are no longer considered appropriate for design since they clearly do not generally capture the potential important effects of site conditions. The comment on steepness of slopes of design surface spectra is not one that is currently considered in guidance documents prepared for NRC, DOE or ASCE. The general tone of these documents is to generate spectra appropriate for given sites, capturing the effects of these characteristics in a suitably conservative fashion.

The original surface response spectrum recommendation, generated circa 1996, was based on empirical data available at that time together with some modifications evaluated to account for potential effects of site characteristics known from a restricted site data set. The hazard calculations made use of attenuation functions appropriate for deep California soil sites convolved with locally appropriate source definitions. Comparison of potential site effects from generic California and Hanford sites indicated that amplification effects were comparable. However, upon further detailed evaluations and significant review, it was found that these effects were not necessarily comparable, particularly in and around the WTP site.

Figure 3 presents a comparison of the low strain shear wave velocity profile associated with a Northern California (NCA) generic soil site with the best estimate profile at the WTP site. The existence of stiffer rock at depth as well as stiffer sediments in the upper 350' of the WTP profile indicated a surface response that would be expected to be significantly higher than at the generic site. Similar conclusions were drawn for comparison with the Southern California (SCA) generic profile. In addition, the more linear sediment properties anticipated for the WTP sediments as compared to the CA generic models indicated that site amplification would be larger at WTP than at the generic sites. An issue of significant uncertainty concerned the impact of the properties and detailed configuration of the basalt/interbeds on site amplification.

These effects were evaluated by performing an extensive logic tree analysis of the effects of WTP site properties on amplification effects to try to capture this epistemic uncertainty. In these calculations, the attempt was not to recalculate the seismic hazard at the site, but to recalculate the potential differences between the NCA/SCA site responses from those of the WTP site, accounting for the uncertainty in these properties. Figure 4 presents a typical result of the "Relative Amplification Factor – Hanford Site/CA Site" that would then be applied to the previously available MEAN ground surface spectrum. Therefore, the current modified spectrum is not an 85th percentile spectrum, as is often stated, but what is hopefully considered to be a conservative estimate of the MEAN surface spectrum. Recalculation of the seismic surface spectrum following revised site response procedures, as recommended in NUREG/CR-6728 and ASCE 43-05, is a task still needed to properly define the MEAN surface site response spectrum.

As can be noted in Figure 4, there is no significant frequency shift between the mean relative amplification factors, Hanford/CA, as compared with the 85th percentile response. In addition, the relative amplification factor around 2 Hz is about the same, mean to 85th, while the ratio is larger at about 5 Hz and above. This characteristic is an indicator of the relatively linear response characteristics of the site; that is, no significant nonlinear material behavior was noted in these response calculations. One of the large contributors to the range of responses around 5 Hz was due to the uncertainty in characteristics of the basalt/interbed sequence. With better definition of these properties, the recommended relative amplification factor that may be used with confidence may in fact be lower.

Figure 5 presents a comparison of the original site-specific surface response spectrum developed in the original hazard analysis with the current recommendation. The process followed was to apply the 85th relative amplification factor computed from the logic tree analysis to the original design spectrum and then smooth over frequency gaps in the resulting response spectrum. This spectrum is labeled "SA-Recommended" in Figure 5. The value at the peak is somewhat lower than that resulting from the 85th relative amplification. The spectrum at the peak was then broadened somewhat at the crown to capture typical effects of probabilistic spectra calculations. The spectrum labeled "SA-Broadened" is then considered a conservative estimate of the MEAN surface spectrum that can be used for design. The results determined from the proposed site-drilling program can be used to better define the influence of the basalt/interbed sequence on the proposed design spectrum. The performance of the hazard calculations following the revised recommended procedures, starting from bedrock definition of the outcrop spectrum, also will lead to a more consistent evaluation of ground motion.

Finally, Figure 6 presents a comparison of the various spectra discussed in Figure 5 with the generic NUREG shapes. The original spectral shape recommended for the site is compared with the median NUREG shape scaled to the same PGA and using V/A of 36 in/sec/g and AD/V^2 of 6. As can be seen, the spectra are comparable in the constant acceleration range, although the fit at low frequency can be improved by modifying the assumed parameters. A similar comparison is provided for the modified recommended spectrum and scaled to the revised PGA. The width of the spectrum is again very comparable to the NUREG shape indicating that the “peakedness” comment of the recommended spectrum is inappropriate. The amplification in the range between about 2.5 Hz and 8 Hz is greater than indicated by the NUREG shape due primarily to the differences in characteristics between the CA generic profiles and the WTP site-specific profile.

FIGURE 1
TYPICAL SPECTRAL
SHAPES
File: NUREG0098.CRD

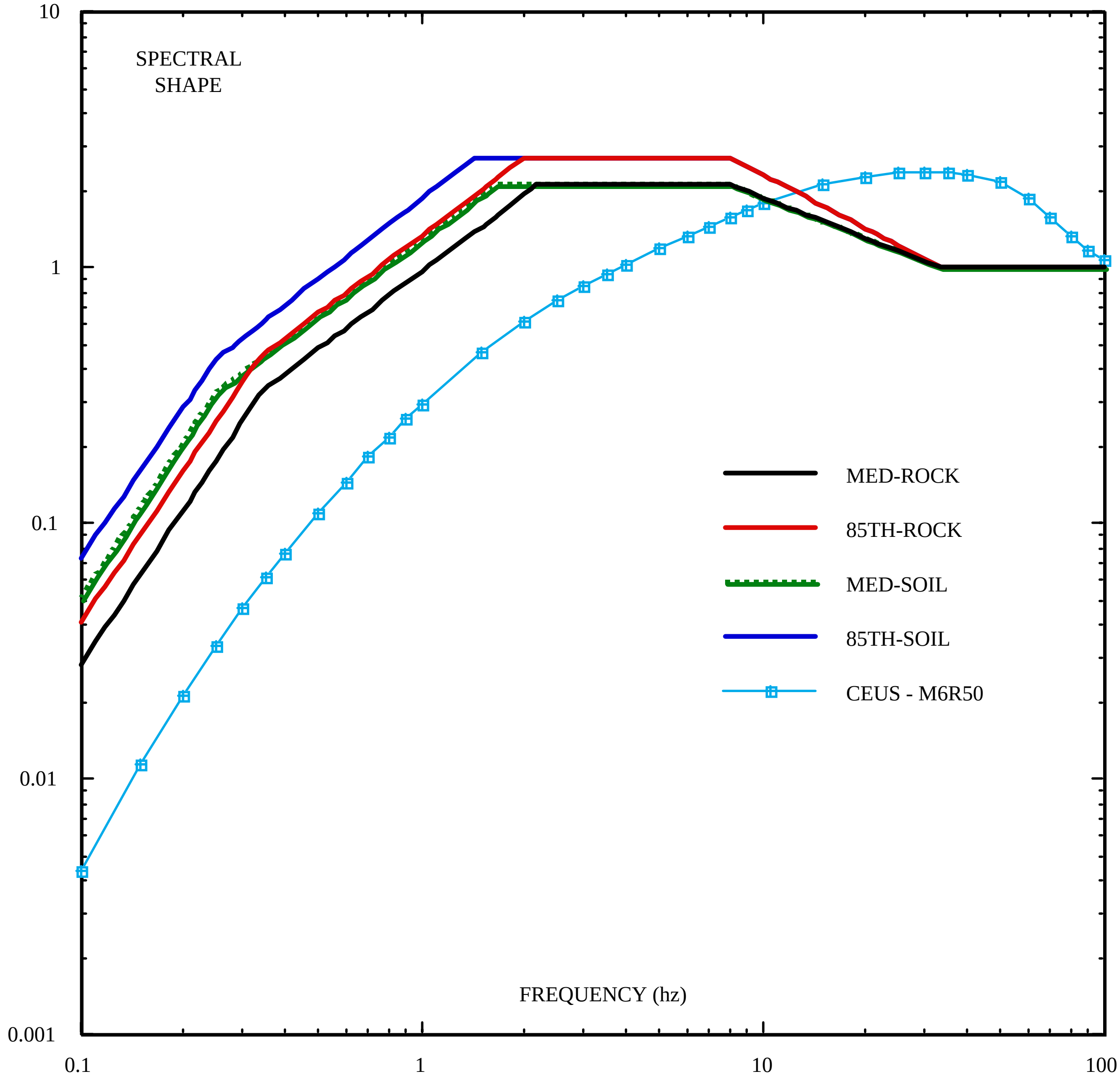


FIGURE 2
TYPICAL SPECTRAL
SHAPES
File: NUREG0098.CRD

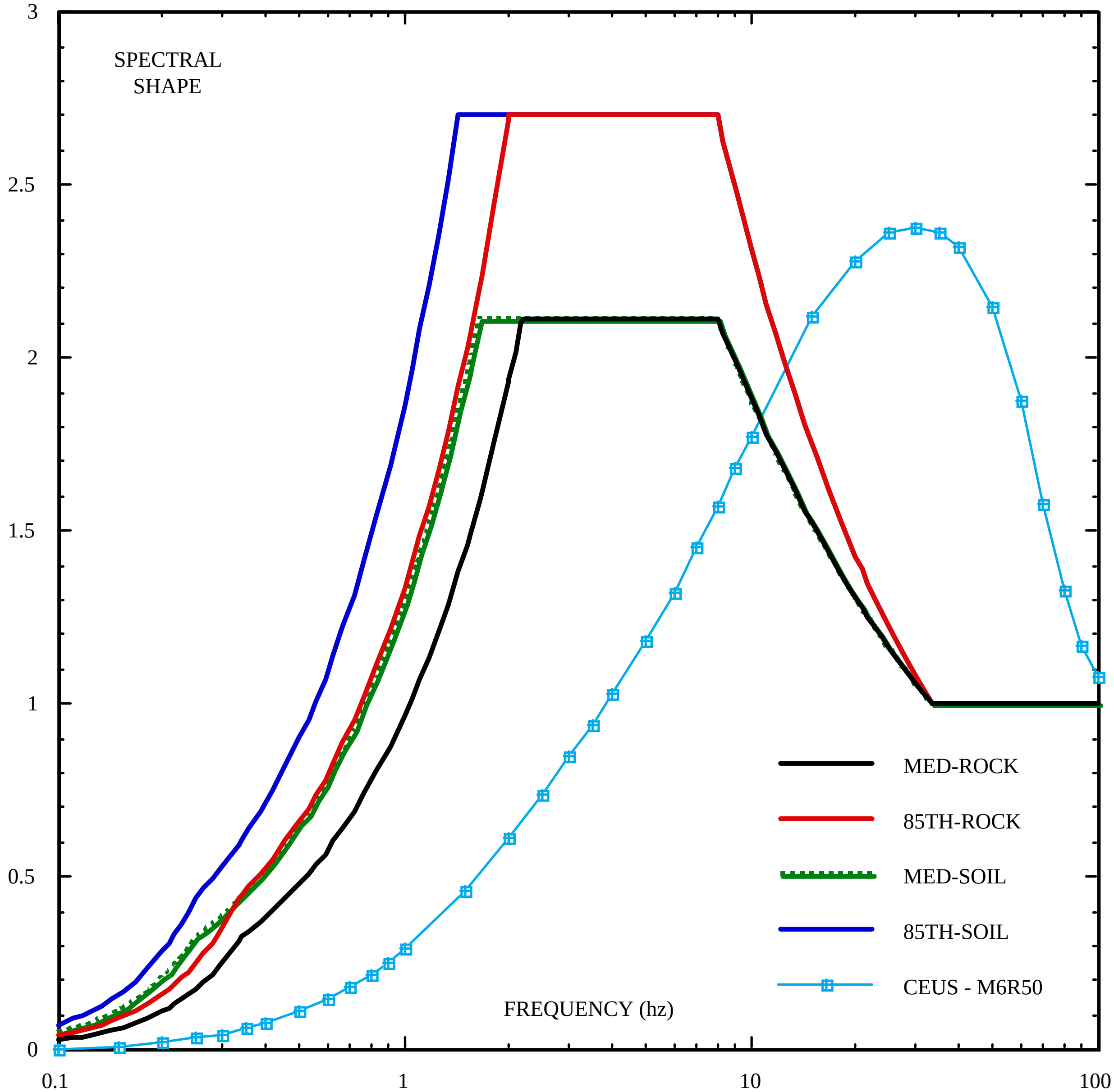


FIGURE 3
LOW STRAIN SHEAR WAVE PROFILES
File: VELOCITY.CRD

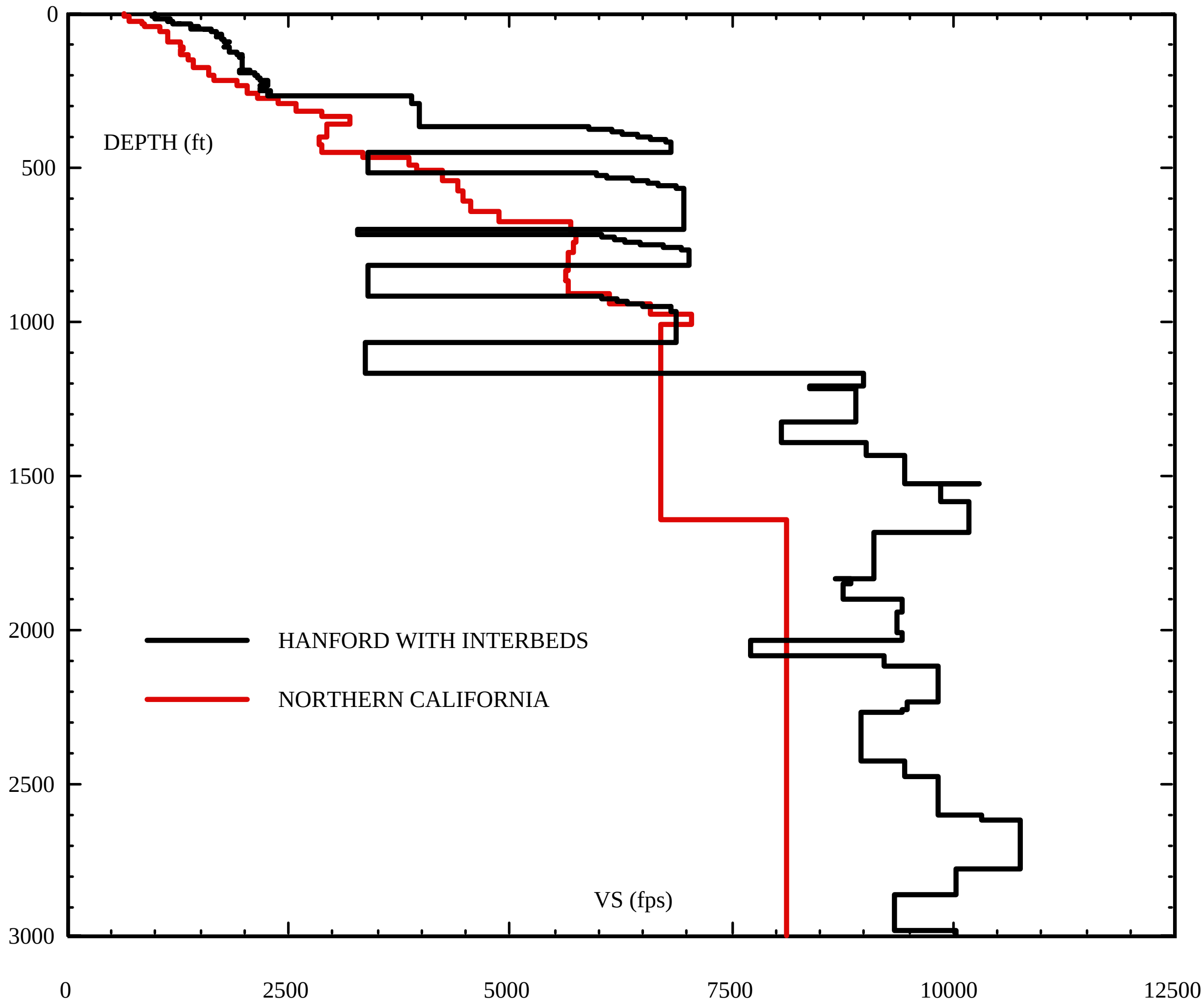


FIGURE 4
COMPARISON OF 85TH SAF's TO MEAN SAF's
File: TREEOUT-R2-SUM.CRDATA

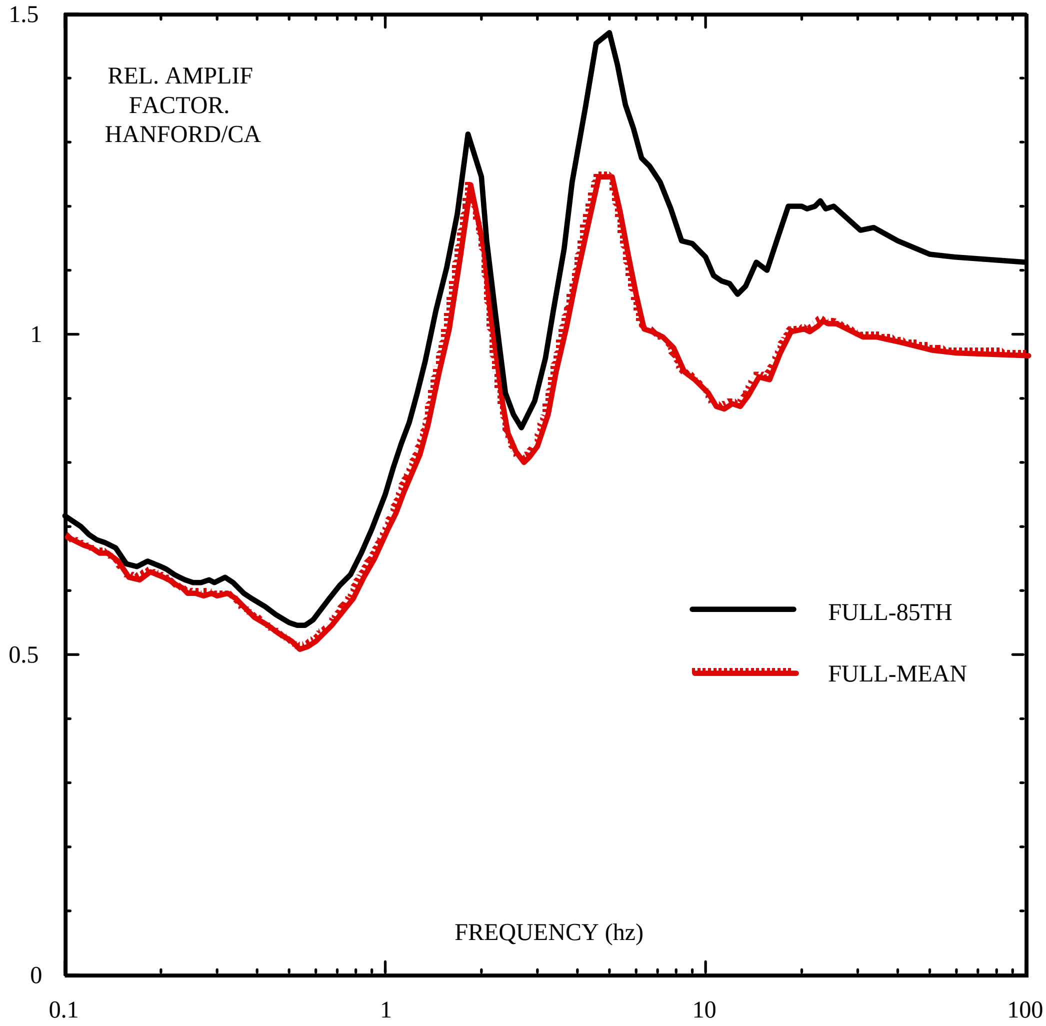


FIGURE 5
MODIFIED SPECTRA
File: SPECTRA.CRD

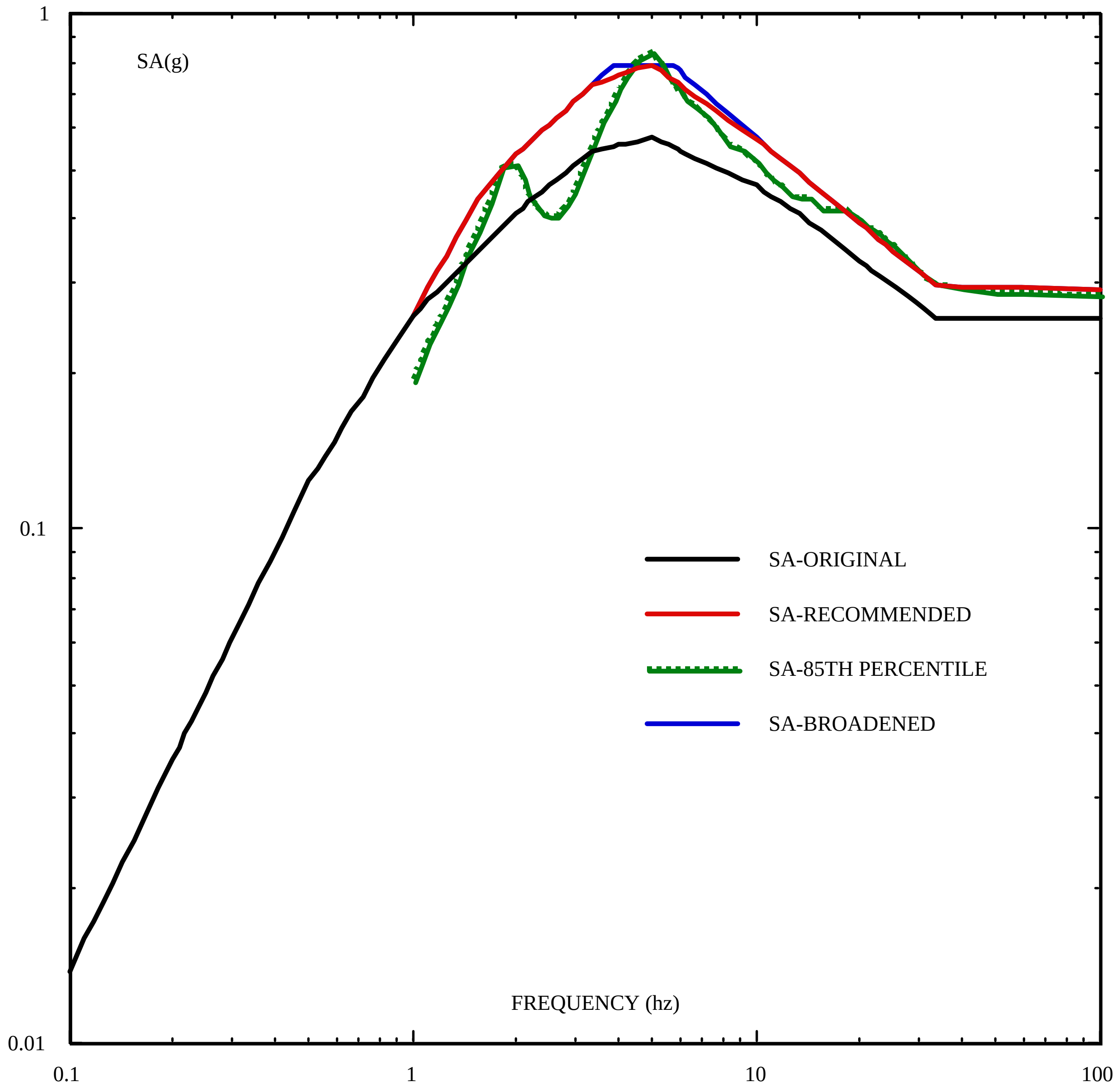
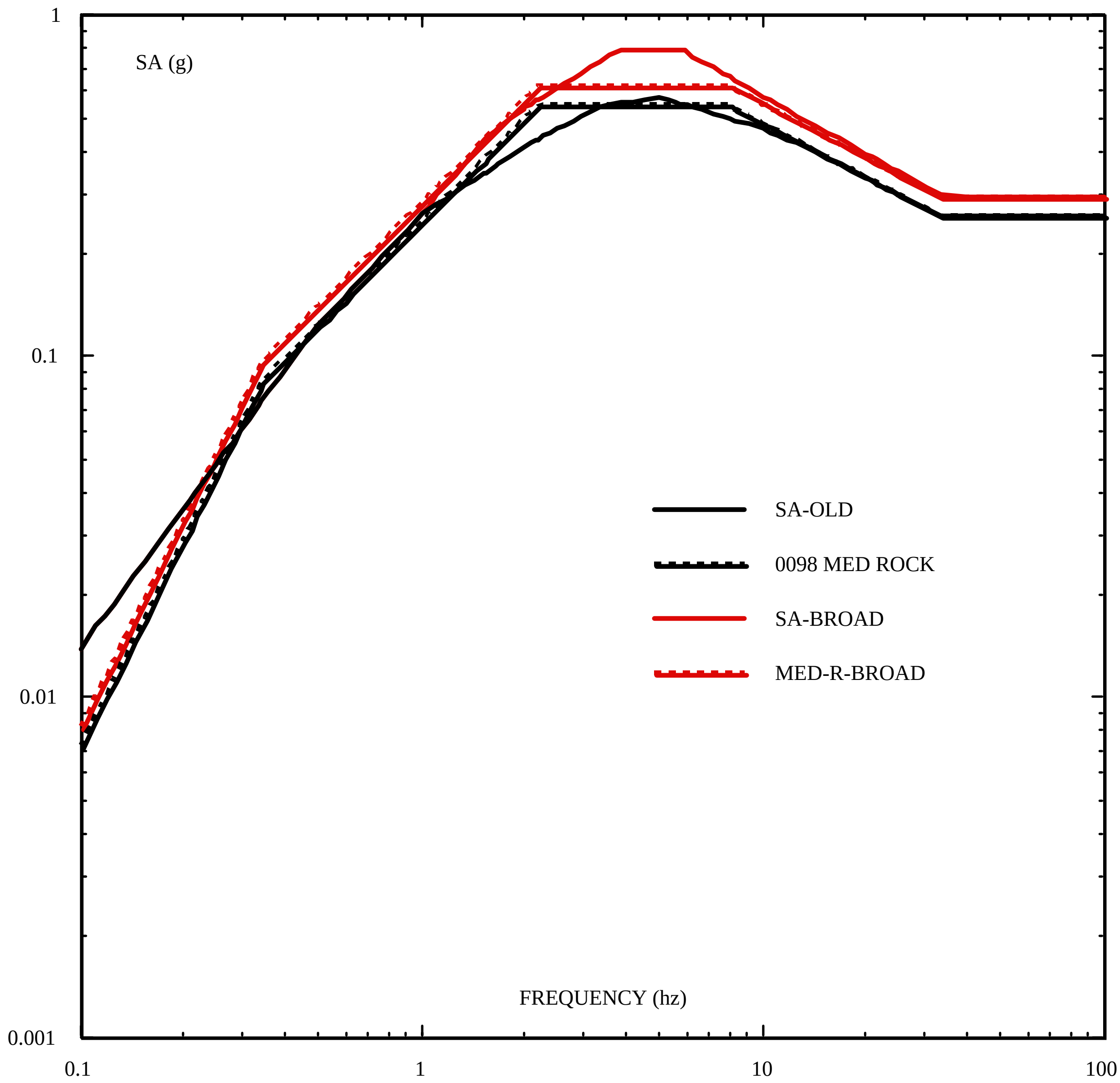


FIGURE 6
COMPARISON OF SITE
SPECTRA TO NUREG SHAPES
File: FITS.CRD



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SM-74	24590-WTP-DC-ST-04-001, Rev. 3, Seismic Analysis and Design Criteria	Reviewer: James Johnson Deformation is not accounted for in the SADC from Nonlinear Behavior of Structures, Systems, and Components (SSCs) a) The use of nonlinear behavior in the design of buildings, non-building structures, systems, equipment, and components is permissible as stated in Ref. 1 (with reference to other industry standards). Identification of the specific cases where nonlinear behavior is taken into account for the design, qualification, and/or evaluation of these items should be done; specifically the use of Fμ factors to adjust seismic loading conditions. b) In cases where the use of Fμ is formally implemented or in other cases where structures, systems, and components (SSCs) are expected to behave nonlinearly, the additional deformation associated with this nonlinear behavior should be considered. Consideration may influence the requirements for interfaces and clearances between SSCs and other commodities to prevent impact	BNI/McConnel, Khabir: For major structures, this issue is addressed in the Seismic Analysis Design Criteria, 24590-WTP-DC-ST-04-001, Section 8.7.1. SC-I and SC-II Structures of Section 8.7 Building Separation where it states in the 3rd paragraph: "In the event that a portion of existing SC-I structure was qualified using and inelastic energy absorption factor, then and evaluation or judgment will be made to determine if that portion of the structure contributes significantly to the Δ_{E1} or Δ_{E2}. It the inelastic contribution increases Δ_{E1} or Δ_{E2} by more than 10%, then it shall be included when determining Δ_{E}." For Structures (other than major structures), Systems and Components, WTP is in process of developing a Design Guide 24590-WTP-GPG-ENG-033, titled "*Identification and Evaluation of Seismic II/I Structures, Systems and Components, and Seismic Interaction*." It is forecasted to issue this guide for project for use by 4th quarter 2006.	(a) Closed (b) I assume the new Design Guide will deal with (b) and (c) - Closed

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		or other potentially adverse consequences. Consideration may influence the calculated story drift values. Acknowledgement of these situations should be done to verify that they have been considered. c) For SC-III and IV SSCs, the potential adverse effects of nonlinear dynamic response of these components on SC-1 and II SSCs should be considered.		
SM-75	WTP Structural Modeling Review, Presentation, Abdul et al., 01/09-10/2006 24590-PTF-SOC-S15T-00004, Pretreatment Building Seismic Analysis – Seismic Loads 24590-HLW-SOC-S15T-00008, High Level Waste Building Seismic Analysis – Seismic Loads	Reviewer: James Johnson Application of Bubble Acceleration Values in SAP2000 Static Model. The approach does not address the question of the phase relationships between the nodal displacements (or accelerations) at each of the nodal degrees of freedom. This question should be addressed. First, for each soil case, do the peak values at the nodal degrees of freedom occur at the same or close to the same time? Second, when enveloping over the soil cases, how does the phase question enter into the analysis? Are the majority of the peak values from one soil case and, hence, no new phase questions are raised? It would seem that taking the envelope of the three soil cases and ignoring	BNI/Ma: The concern that when using SAP2000 (or GT-Strudl) to calculate element stresses for structural design is that horizontal seismic loads are applied statically in the same direction such that any out of phase seismic motions, which potentially can cause higher stresses, are ignored. This concern was raised previously by the Defense Nuclear Safety Board and BNI has provided responses. The BNI responses are documented in Appendix G of Summary Structural Report for HLW Vitrification Building, document #24590-WTP-RPT-ST-03-001, revision B, December 22, 2004. Selected Figures and Tables, taken from Appendix G of the referenced report are included following this table.	Recognize that the stresses calculated with SASSI are not necessarily accurate in these members – otherwise there would be no motivation to develop a 3 x 3 model with SAP 2000 – so the argument that the stresses calculated with the static analysis are greater than the SASSI results does not hold. In the concrete portions of the structure and for the steel braced, it is reasonable to assume that the static stresses are reasonable and probably conservative, but the SASSI calculated stresses are not the justification. Closed.

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		phase considerations would be conservative when evaluating overall response, such as base shear or overturning moment. However, for design of diaphragms or walls where discontinuities due to penetrations, stiffness changes, intersecting structural elements, etc. are present, considering the potential for out of phase behavior may produce greater demand. For sub-structures, such as the structural steel upper stories, this was expected and the member forces calculated applying the static loads were reported to be conservative when compared to those calculated dynamically. The absence of these situations within the concrete structure should be verified.	For the reinforced concrete portion of HLW, the major structural members that resist horizontal seismic forces are in-plane shear in major walls and slabs. Major walls are relatively rigid in the in-plane shear direction such that each wall has its own natural frequency. Shear stress time histories at various slab elevations of each selected major wall are shown to be in-phase by time history plots in appendix G of the referenced report. Taken from the referenced report, Figure SM-75-1, attached, demonstrates the in-phase nature of the in-plane shear stress at various elevations of a major wall. Additional time history plots in the referenced report show similar in-phase nature of in-plane shear responses. In addition to wall stresses, the potential out of phase seismic motion can adversely affect the in-plane shear stresses in reinforced concrete slabs such that the adequacy of the statically in-phase application of equivalent seismic accelerations can be questioned. However, since the slabs can twist to relieve its shear stresses, and since the walls are relatively stiff so that the horizontal seismic displacements at wall/slab intersections are small, and thus the shear demand in major slabs can be small due to small	

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			relative displacement between adjacent walls. Table SM-75-1, attached, summarized and compared in-plane shear stresses of stripes of slab at elevation 57' calculated by SASSI SSI analyses and by the statically applied equivalent seismic load. The comparison shows the relatively low shear stresses calculated by static analysis are higher than those calculated by dynamic analysis using SASSI. Figure SM-75-2, attached, shows location of these slab elements. In conclusion, each major shear wall in HLW has its own frequency of vibration in the horizontal in-plane direction, and the statically in-phase application of the equivalent seismic load is conservative. For reinforced concrete slabs that are connecting various shear walls together, horizontal seismic is the only primary contributor to the in-plane shear stresses. Since the statically calculated shear stresses are lower than those from dynamic analyses, and since their absolute magnitudes are small relative to shear capacity of concrete, it is concluded that the static application of seismic loads is adequate in terms of out-of-phase motion in HLW.	
SM-76	WTP Structural Modeling Review, Presentation, Abdul et al., 01/09-10/2006	Reviewer: James Johnson For vertical amplification of flexible floor slabs and structural steel	BNI/Ma: During development of dynamic finite	I assume this response means that the development of the

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	24590-PTF-SOC-S15T-00004, Pretreatment Building Seismic Analysis – Seismic Loads 24590-HLW-SOC-S15T-00008, High Level Waste Building Seismic Analysis – Seismic Loads	members, simplified dynamic models (single degree of freedom (SDOF)) are introduced into the dynamic model to capture local amplification. These SDOF models represent the modal mass at the fundamental frequency of the local structural element. For each soil case, the peak vertical acceleration of the SDOF system is calculated for the three directions of excitation separately and combined by square root of the sum of the squares (SRSS). Hence, at each SDOF there is tabulated three values of peak vertical acceleration. The envelope of the three soil cases is determined and the envelope peak acceleration values times the mass become the equivalent static loads for local member design. This procedure is described with reference to the vertical amplification of slabs and structural steel members. Are there situations where similar local amplification is expected in the horizontal direction, e.g., for structural steel beams or columns spanning large distances and supporting structural or non-structural items that would lead to fundamental frequencies in frequency ranges less than 20 Hz.? Are there situations within the structures where local horizontal	element models of PTF and HLW, the potential of having high local seismic responses at large span steel beams was recognized. Therefore, within the PTF and HLW SSI models, steel beams that have large span between supports, which potentially can result in local fundamental frequencies in the flexible range and high local seismic responses, are ensured to be represented by intermediate nodes to capture these local responses. For example, within the PTF seismic load calculation reviewed (24590-PTF-S0C-S15T-00004, revision 1B), notes 2 thru 9 of Seismic Load Table on page 91 specified higher local horizontal accelerations for steel roof framing, and Exceptions for elevations 117' and 108' of Seismic Load Table on page 93 specified higher local vertical accelerations for steel roof framing.	dynamic structural models considered all situations where local modes less than 20 Hz. were identified and the finite element model was discretized in a manner to deal with these situations. Closed.

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		modes were modeled or should be modeled?		
SM-77	WTP Structural Modeling Review, Presentation, Abdul et al., 01/09-10/2006 24590-PTF-SOC-S15T-00004, Pretreatment Building Seismic Analysis – Seismic Loads 24590-HLW-SOC-S15T-00008, High Level Waste Building Seismic Analysis – Seismic Loads	Reviewer: James Johnson Application of Accidental Torsion to the PTF and HLW a) Generally, it seems that the provisions of industry standards such as ASCE 4, which allow explicit modeling of sources of torsion in the dynamic behavior of structures and in the phenomena associated with the ground motion, should allow an approximate approach like the 5% eccentricity rule, but smaller in amplitude, if some aspects of torsion are accounted for directly in the analysis. For example, a fully three-dimensional model is developed of the building structures of interest. This model represents the best estimate of each source of accidental eccentricity. One phenomena not covered by this model might be the potential effect of non-vertically incident waves. However, to cover this one aspect of unmodeled torsion, it seems overly conservative to require the 5% eccentricity rule. Perhaps, an alternative would be to assume	BNI/Ma: a) Observation b) Seismic loads are specified as equivalent static accelerations in Tables 1 thru 4. For the seismic analysis in the global X, Y, and Z directions, static accelerations given in Tables 1 and 4 may be applied to the SAP2000 finite element model of PTF and HLW. 1. Seismic loads are applied in the global X, Y, and Z directions. 2. All seismic loads are considered cyclical and the static seismic accelerations are reversible. Alternatively, these static analysis results may be treated as (±). 3. Responses of these seismic loads are combined by (100, 40, 40) method. 4. Accidental torsion (M_T) related to EW and NS seismic are applied in separate load cases. 5. M_T is due to either EW or NS input seismic motion. Since M_T is cyclical in nature, its responses are reversible or use (±).	(b) Closed.

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		the 5% eccentricity rule is a minimum, i.e., calculate the best estimate torsional moment from the SSI model. If the value is less than the 5% eccentricity rule, the torsional moment must be increased to the 5% level. A further measure might be to add a 1% or 2% rule to accommodate non-vertically incident waves if not modeled. Hence, for the case at hand, one might argue a smaller value of accidental eccentricity could be applied. b) Application of the equivalent angular accelerations by direction of response and direction of excitation is not clear from the references, including the presentation. Clarification should be provided.	6. Combine responses, due to M_T with the combined seismic response (from step 3 above), with consideration to both $\pm M_T$. 7. Alternatively, it is conservative if responses due to accidental torsion from step 4 are combined with seismic responses from step 2 before performing the (100, 40, 40) load combination in step 3. Using this sequence, responses of accidental torsion in the off-diagonal direction is conservatively included with 0.40 scale factor, the load combination process is simplified, and since the maximum in-plane shear stress is approximately 25 psi, any added conservatism is not significant.	
SM-78	WTP Structural Modeling Review, Presentation, Abdul et al., 01/09-10/2006 24590-WTP-DC-ST-04-001, Rev. 3, Seismic Analysis and Design Criteria	Reviewer: James Johnson Treatment of the soil pressures as design loading conditions in the SAP2000 static model. During presentations of Ref. 1, the methodology of applying the soil pressures was briefly discussed. A summary of the methodology is that soil pressures were extracted from the SSI analyses and transmitted to the SAP2000 static model for the	BNI/Ma, Jeffrey: There are 2 types of soil pressures, which are applied statically to the SAP2000 finite element model, static pressure and seismic soil pressure. Both types of soil pressure are applied only to below-grade walls, in contact with soil. All of the soil pressures are applied in the direction normal to the wall surface and are always compressive.	Conditional on the BNI response to SAS/JJJ comments SM-85 previously responded to below – Closed.

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		design process. For a given direction, the soil pressures on the embedded walls in the perpendicular direction are applied simultaneously. The question remains: on the face where the design soil pressures are tensile, do the soil pressures exceed the static pressures? If they do, how are the seismically induced soil pressures re-distributed to assure that tension is not taken into account in the design process? The evaluation of the seismic soil pressures, calculated in the SSI analysis, and applied to the SAP2000 static model, should be documented. On the tensile-side of the seismically induced soil pressures calculated in the SSI analysis, document whether these stresses exceed the in-situ static soil pressures.	For static soil pressure, the compressive soil pressure is applied to all walls in contact with soil. For seismic soil pressure, the compressive soil pressure is applied to walls that resist the inertia force from the building. For example, if the inertia load of the building is applied in the direction from East to West, then the seismic soil pressure is applied to all of the below grade North-South walls in contact with soil where the soil is located west of the contact surface. If the inertia load of the building is applied in the direction from West to East, then the seismic soil pressure is applied to all of the below grade North-South walls in contact with soil where the soil is located east of the contact surface. Same procedure is applied to NS direction loads for seismic soil pressure. There is no seismic soil pressure in the vertical direction.	
SM-79	1. 24590-PTF-S0C-S15T-00001, PTF Seismic Analysis: Free Field Analysis, Rev. 2 2. 24590-HLW-S0C-S15T-00001, HLW Seismic Analysis: Free Field Analysis, Rev. 2	Reviewer: James Johnson a) One objective of the free field analysis was to generate strain compatible soil profiles of the site for use in the SSI analysis of the PTF building (Ref. 1) and the HLW building (Ref. 2). The focus of this review was on the PTF building. Limited comparisons between the HLW free field analysis and that of the	Observation	

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	3. ORP Letter 05-WTP-082, R. J. Schepens to J. P. Henschel, "Contract No. DE-AC27-01RV14136 – Dynamic Soil Properties for the Waste Treatment and Immobilization Plant (WTP)," included in Refs. 1 and 2. 4. Rohay, A. C. and S. P. Reidel, "Site-Specific Seismic Site Response Model for the Waste Treatment Plant, Hanford, Washington, prepared by the Pacific Northwest National Laboratory for the US Department of Energy, Office of River Protection, under Contract DE-AC05-76RL01830, March 2005. 5. 24590-PTF-S0C-S15T-00010, Rev. 1, Pretreatment Building – Soil Springs.	PTF building were performed. Observations of similarities and differences are noted. b) Given the revised ground motion (RGM) (referred to here as DBE) and the associated sol profiles (stratigraphy, low strain shear modulus, strain compatible shear modulus, and soil damping) generate soil profiles using SHAKE for use in the SSI analysis (SASSI). Three soil profiles denoted "mean," "lower bound," and "upper bound" are generated. c) Input was: site soil profiles (taken from Ref. 1, Rev. 1); soil shear modulus degradation curves and soil material damping curves as a function of shear strain (taken from Ref. 1, Rev. 1); DBE acceleration time histories (taken from 24590-WTP-S0C-S15T-00002, Rev. 2); and dynamic soil properties for WTP (taken from Ref. 3 and it's underlying data and analysis results). d) Reference 3 refers to Ref. 4, March 2005 publication date. Available copy was dated February 2005. Assume no		(f,g,h,i) This comment identifies an error in the calculation and requires a response. The recommendation was to justify the lack of consequences of this error due to the wide range of soil properties considered and commit to re-doing if the analysis is redone at some point. See response below.

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		changes, for the purposes of these observations, between the two versions. e) Reference 1 includes a CD with summary information and SHAKE analyses input and output. These were reviewed to the extent possible. f) For the PTF building, SHAKE analyses were performed for the three soil profiles (LB, M, UB) and for the horizontal component time histories (H1, H2). In all cases reviewed, the ground motion time histories input were at peak ground accelerations of 0.3g. In all cases reviewed, the time histories were immediately scaled to 0.257g. The text states that the "new time histories "were used, which may be true, but the assumption is that the PGA values are at 0.3g to meet the requirements of the RGM or DBE. This does not appear to be the case. g) The approximate fundamental frequencies of the soil columns for the (LB, M, UB) are (1.37 Hz., 1.85 Hz., 2.36 Hz.). This is based on the scaled ground		

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		motion time histories (PGA = 0.257g) and for H1 only. The values for H2 are similar. h) Limited comparisons between Refs. 1 and 2 were performed. SHAKE analyses documented in Ref. 2 for the HLW building were performed for "new time histories" which were at peak acceleration values of 0.3 g and were not scaled to 0.257 g as described above for Ref. 1. The resulting tabulation of soil shear moduli as a function of depth is somewhat softer for all three soil profile cases for the HLW results (Ref. 2) vs. the PTF results (Ref. 1). This is expected. i) The end results of the analyses are comparisons between previous results (presumably Rev. 1 of Ref. 1) and the results provided in Ref. 3, specifically the underlying data. The conclusion was to use soil profiles for LB, M, and UB that envelope the previous profiles and the newly developed profiles (Ref. 3) for soil stiffness (strain compatible shear modulus). This meets the requirements of ASCE 4 and the WTP seismic analysis criteria. The upper 50		

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		ft. of soil for the LB case was modified to take into account new information from Ref. 3. j) Soil material damping was stated to be taken from Ref. 3 for the three soil profiles. These damping values were consistently higher than those from the SHAKE analyses, but were selected as the best estimates based on the ground motion studies. Reference 3 provides a sample of soil profile information for ten ground motion simulations, but not the statistics on the entire sample itself. Reference is made to accompanying CD of data, which was not available. Hence, it is assumed that the damping values used in the SASSI analyses are the average values, layer by layer, for the LB, M, and UB soil cases form the underlying data for Refs. 3 and 4.		
SM-80	1. 24590-PTF-S0C-S15T-00001, PTF Seismic Analysis: Free Field Analysis, Rev. 2 2. 24590-HLW-S0C-S15T-00001, HLW	Reviewer: James Johnson For the PTF building, SHAKE analyses were performed for the three soil profiles (LB, M, UB) and for the horizontal component time histories (H1, H2). In all cases reviewed, the ground motion time	BNI/Ma: (1) Input acceleration time histories in SHAKE analyses documented in reference #1, calculation for Free Field Analysis of PTF, were inadvertently scaled from 0.30g to	(1) Closed.

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	Seismic Analysis: Free Field Analysis, Rev. 2 3. ORP Letter 05-WTP-082, R. J. Schepens to J. P. Henschel, "Contract No. DE-AC27-01RV14136 – Dynamic Soil Properties for the Waste Treatment and Immobilization Plant (WTP)," included in Refs. 1 and 2. 4. Rohay, A. C. and S. P. Reidel, "Site-Specific Seismic Site Response Model for the Waste Treatment Plant, Hanford, Washington, prepared by the Pacific Northwest National Laboratory for the US Department of Energy, Office of River Protection, under Contract DE-AC05-76RL01830, March 2005.	histories input were at peak ground accelerations of 0.3g. In all cases reviewed, the time histories were immediately scaled to 0.257g. The text states that the "new time histories "were used, which may be true, but the assumption is that the PGA values are at 0.3g to meet the requirements of the RGM or DBE. This does not appear to be the case. a) Reconcile SHAKE input time histories between output files and text, i.e., PGA to be used in free field analyses, for the PTF building analyses. If the soil profiles need to be revised for the PTF building, document how this revision will be treated in the existing soil spring calculation (Ref. 5), existing SASSI analysis results, and future soil spring and SASSI analyses. b) Provide PNNL data that shows soil material damping for each of the three soil profiles. Justify the use of these values vs. those derived from the SHAKE analyses.	0.257g PGA. • This does not apply to Free Field Analysis of HLW where the correct 0.30g PGA was used. • This does not apply to SSI analyses where the correct 0.30g PGA was used. • This does not affect soil spring since PGA is not considered in the calculation of soil spring stiffness. • SHAKE analyses for PTF were re-run using the corrected 0.30g PGA. Plots of strain-compatible shear wave velocities and soil damping for Lower Bound, Mean, and Upper Bound soil cases are shown in Figures SM-80-1 and SM-80-2, attached. For comparison purpose, plots of the compatible SHAKE data from reference #1 calculation are shown in Figures SM-80-3 and SM-80-4, attached. • Based on the small variations in shear wave velocity and soil damping (shown in Figures SM-80-1 thru SM-80-4) due to variation in PGA, it is concluded that this change in PGA has very minor effect on these strain compatible data. In addition, since the final shear	(2) I believe the argument is that soil material damping does not have a significant impact on the SSI response of these structures. Hence, the values selected are not important or significant. Basically, there should be compatibility between the material properties of the SSI

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	5. 24590-PTF-S0C-S15T-00010, Rev.1, Pretreatment Building – Soil Springs.		wave velocities and soil damping values used in SSI analyses are based only on data from revision 1 of reference #1 calculation and from data given in reference #3 ORP letter, this change in PGA is concluded to have no impact to the SSI analyses of PTF. (2) Soil damping calculated by SHAKE analyses are shown in Figures SM-80-2 and SM-80-4. They are calculated by de-convolution with input time history motions applied at top of soil. In general, relative to the overall damping in the SSI analyses, these damping values are small and would not have much impact on the calculated SSI responses. Reference #3 ORP letter gives additional sets of soil damping which are calculated with the determination of ARS of input motion at grade where Convolution method was used. The shape of the soil damping of the soil column are different between convolution and de-convolution methods. Since the SHAKE calculated soil damping values are relatively small which would have only small effects on SSI analyses, and since those soil damping given in reference #3	analysis, i.e., shear modulus and damping for strain compatible conditions. This is not the case when damping is extracted and used from the seismic hazard analysis and shear moduli are determined from the SHAKE analyses. On the issue of conservatism, for the embedded structure HLW, higher damping values likely leads to more conservative results due to kinematic interaction and less conservative results due to inertial interaction. For the PTF building, the results are likely to be less conservative due to inertial interaction since there is no embedment of significance (no kinematic interaction effects). One cannot argue conservatism or insignificance effectively without sensitivity studies to verify it. This issue remains open, but is the responsibility of DOE since they specified which soil material damping values the project should use.

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			are consistent with ARS of the RGM, soil damping from reference #3 are used in SSI analyses.	
SM-81	1. 24590-HLW-S0C-S15T-00001 - Free Field Analysis 2. 24590-HLW-S0C-S15T-00007 - SSI Analysis 3. 24590-HLW-S0C-S15T-00025 - Structural Model with Equipment Seismic Loads	Reviewer: Stephen A. Short In the free field analysis, the soil material damping values given in Table 8-1 and used in the seismic SASSI analyses are not the same as the values presented in Reference 3. It appears that the Reference 3 values presented are a subset of the total number of cases considered, but this is not explained in the calculation. There should be a clear path to the damped values used in the analyses.	BNI/Ma: The reasons for selecting the particular soil damping that are used in SSI analyses are stated above in responses to SM-80.	Closed
SM-82	24590-HLW-S0C-S15T-00006 - Structural Model	Reviewer: Stephen A. Short The modulus of elasticity for reinforced concrete is computed on Sheet 10 of the structural model calculations. The equation that E = $33*w^{1.5}$ times the square root of f_c’ where w is taken to be 150 pcf. The ACI equation for normal weight concrete is E = 57,000 times the square root of f_c’. The resulting E used is about 6.4% greater than the ACI value for normal weight concrete. It is my understanding that normal weight concrete is used for all HLW concrete construction.	Observation	Closed
SM-	24590-HLW-S0C-	Reviewer: Stephen A. Short	BNI/Ma:	

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83	S15T-00007 - SSI Analysis	In the SSI analysis, transfer functions for the SASSI hard rock (simulated fixed base) case indicate agreement in frequencies with the GTSTRUDL fixed base analysis. The transfer function amplitude at the fundamental modes are also of interest. In the east-west direction, the amplitude is 5 along the southern portion of the building, 6 in the middle section of the building, and 8 along the northern portion of the building. These relative amplifications are reasonable as there are more resisting walls in the middle and southern parts of the HLW building than there are in the northern portion. In the north south direction, the amplitudes are on the order of 6 to 7 along both the east and west perimeter walls but the amplitude is about 10 in the middle portion of the building. These relative amplifications would be reasonable for behavior of a flexible diaphragm responding between two stiff end walls. The high amplification in the middle portion is hard to understand because of the large number of north south running interior walls within this building. An explanation of the larger middle amplification in the north-south direction is needed.	The HLW has a large basement that is roughly bounded by column lines 5, 20, A & B, and T & T.5, on the West, East, North, and South sides respectively. The top of basemat for area inside these 4 boundaries is at elevation -21' while it is at elevation 0' for the area outside these boundaries. Therefore, in the hard rock case, the fixed boundary conditions are located at elevation -21' inside these boundaries while it is at elevation 0' at and outside these boundaries. Therefore, North-South transfer functions at both the East and West sides are supported at elevation 0' while walls near the center of the building are supported at elevation -21'. The extra 21 feet height for walls near the center of HLW, increased wall height from approximately 57' at both ends to approximately 78' near the center, and also raised the peak of North-South transfer function from the 6 to 7 range to 10.	Closed

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SM-84	24590-HLW-S0C-S15T-00007 - SSI Analysis	Reviewer: Stephen A. Short There needs to be some discussion on the means employed to assure that the transfer functions computed in SASSI are stable and do not change significantly with the addition of new frequencies. The process used should be outlined in the calculation.	BNI/Ma: The questions regarding the adequacy of number of calculated frequency points and the stability of transfer functions were first responded to SM-50 comment (CCN 137417[1]) and reiterated again with additional details and a graphic example in response to SM-50 follow-on comment. The number of calculated frequencies and the spacing distribution were determined and refined through several iterations in the last several revisions of the models, and the transfer functions at all critical model locations were carefully inspected to ensure that the major responses of the structure are adequately captured. However, the suggestions of the reviewer are well received; discussion about the adequacy and stability of transfer functions will be added to the future revisions of the calculations.	Closed
SM-85	24590-HLW-S0C-S15T-00007 - SSI Analysis 24590-HLW-S0C-	Reviewer: Stephen A. Short The evaluation of dynamic soil pressures on embedded walls does	BNI/Ma: The reviewer is concerned by the	Closed

[1] CCN 137417, Letter, from J.P. Henschel, BNI, to R.J. Schepens, ORP, "Transmittal of Responses to Comments on the Structural Models for the Hanford Tank Waste Treatment and Immobilization Plant, Based on the Review or the U.S. Army Corps of Engineers", dated April 6, 2006.

[2] CCN 137417, Letter, from J.P. Henschel, BNI, to R.J. Schepens, ORP, "Transmittal of Responses to Comments on the Structural Models for the Hanford Tank Waste Treatment and Immobilization Plant, Based on the Review or the U.S. Army Corps of Engineers," dated April 6, 2006.

CCN 137083, Letter, from J.R. Eschenberg, ORP, to W.S. Elkins, BNI, "Transmittal of Comments on the Structural Models for the Waste Treatment and Immobilization Plant, Based on the Review by U.S. Army Corps of Engineers", 06-WED-012, dated March 9, 2006.

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	S15T-00008 - Seismic Loads	assume that the soil and structure are welded together over the entire embedment depth, as it must. The resulted soil pressures computed by SASSI are compared with the soil pressures for earth retaining walls as given in ASCE 4, Section 3.5.3.2, elastic solution. The north-south soil pressure loads are much greater than the ASCE 4 elastic solution while the east-west soil pressure loads are much less than the ASCE 4 elastic solution. It is our experience that the elastic solution gives upper bound dynamic soil pressures on basement walls. As a result, the high north-south dynamic soil pressures are surprising. There does not seem to be a physical reason for large differences in the loads in each direction. Overall seismic response of the HLW building is greater in the east-west direction than the north-south direction. The design values for seismic soil pressures on east and west walls are 0.8 ksf from grade to 15 foot depth and 1.2 ksf below. The design values for seismic soil pressures on north and south walls are 1.4 ksf from grade to 15 foot depth and 2.5 ksf below.	differences in distribution patterns and in magnitude among the HLW seismic soil pressure values in both E-W and N-S directions. Since dynamic soil pressure is caused by the interaction between the structure and the adjacent soil strata, the differences in the relative stiffness between the soil and structures, the stiffness distribution along the structural member, and the differences in response motion characteristics can all cause variations in calculated soil pressures. Figure SM-85-1, attached, shows the layout of soil elements and locations of surface slabs. Figures SM-85-2 and SM-85-3, attached, taken from section 7 of calculation 24590-HLW-S0C-S15T-00007, revision 0D, show the magnitude and distribution of the calculated seismic soil pressure in the NS and EW directions. Below, these soil pressures are separated into 4 groups and the distinct characteristic of each group is described. <u>Group 1—East-West soil pressure:</u> The X-direction responses of the building are essentially translational movement. For the NW-X, SW-X and SE-X soil columns, the soil strata immediately adjacent to the sidewalls has additional restrain from	

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		These large differences in loads on the underground walls in each direction need some justification in the calculation. In north-south direction, the SASSI computed soil pressures are presented in Figure 7-48 of the SSI calculation. There are 3 soil pressure elements along the north side of the building and one soil pressure element on the south side of the building. At depths down to 12 feet below grade, the soil pressures on the north wall are a factor of more than 5 greater than the soil pressure on the south wall (comparing NM-UB to SW-UB). The NM-UB and SW-UB soil pressures are relatively constant with depth and the upper bound soil case governs the soil pressures. The large differences in soil pressure from the north side of the building to the south side of the building needs explanation. In the 3 north wall soil pressure elements, the distribution of pressure with depth is very different. As mentioned above, the NM-UB soil pressures are relatively constant with depth. However, for the NW-UB and NE-UB elements, the soil pressures are extremely large at	the surface slabs above where these soil strata are being dragged to move along with the wall. Therefore, the relative movement between the soil strata and the sidewall is reduced by this added restrain. It is worth to point out that the recommended soil pressure distribution in ASCE 4-98 was based on finite element results of free-standing rigid retaining walls. While the soil motion is also pure translational, the surface of soil backfill is free to move without added restraint. The presence of the surface slabs which are connected to the side walls is the main factor that causes the X-direction soil pressure to be smaller than the ASCE 4-98 values. <u>Group 2—North-South soil pressure at South wall (@ column line T.5):</u> Refer to Figure SM-85-1, a surface reinforced concrete slab was added south of column line T.5. Therefore, this soil column is similar to those in Group 1 above. For the same reason as stated above for Group 1, the calculated soil pressure for Group 2 is also relatively low. <u>Group 3—North-South soil pressure at East and West sides of North wall @ column line A:</u> At both of these locations, there is not any surface slab	

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		grade and at 20 foot depth but the soil pressures are very low near 10 foot depth. It would be useful to understand the reasons for these differences in pressure distributions. In all cases, the distributions are different than Figure 3.5-1 of ASCE 4. It is not apparent how the soil pressures presented in Figures 7-47 and 7-48 were evaluated from the SASSI output files given in Attachment F of the SSI calculation. A road map along with intermediate calculations is required to understand these loads.	on top of soil. In the North-South direction, the exterior wall is connected to the interior core of the building by steel struts at elevation 0’ and by reinforced concrete basemat at elevation -21’. These structural connections provide the mean of transferring (1) inertia forces of the massive interior, and (2) NS stiffness of the building, to the exterior wall. The connections at these 2 elevations provide the extra stiffness as well as the driving inertia force to the exterior wall. This created more interaction between the exterior wall and the soil column, thus, higher soil pressure at those 2 elevations. Between elevations 0’ and -21’, the exterior wall is free standing without any structural connections to the interior core of the building. Therefore, the exterior wall is more likely to move with the surrounding soil, and thus lower interaction and lower soil pressure between elevations 0’ and -21’. <u>Group 4—North-South soil pressure at center of North Wall @ column line B:</u> At this location, the exterior wall is connected to a N-S wall between elevations 0’ and -21’. This N-S wall provides relatively “rigid” stiffness to the exterior wall in the out-of-plane (NS) direction. Locally the exterior wall is deformed similar to a rigid body, thus, distribution of the soil pressures are	

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			closely matching a straight line as shown in Figure SM-85-3. In summary, above groups 1 thru 4 point out the reasons of the varying soil pressure as shown in Figures SM-85-2 and SM-85-3. Therefore, the soil pressures shown in these 2 figures are judged to be reasonable and correct. Furthermore, in response to the same reviewer's comment of SM-7 (refer to correspondence CCN 137083 & CCN 137417[2]) that the higher soil pressure peaks in the latest revision due to increased earthquake intensity can possibly increase the chance of soil/wall separation during earthquake shaking, it is worth to point out that all concrete shear walls in the HLW building are designed to go directly from upper portion of the building to the basemat at El. –21', while the exterior concrete walls from ground surface generally support the steel framing on the sides of the building. For this reason, the seismic load path for the structure is primarily from floor slabs to concrete walls to the basemat at Elevation -21 ft. Thus, most of the lateral inertial load is resisted by the base shear. Calculations show that a total of less than 10% of lateral inertial load is resisted by the sidewalls. Furthermore, the latest revision of the design (Rev. 0D) expands the surface	

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			slab to cover the entire length of the southern side of the building (Figure SM-85-1). The effects of both surface slab restraint and the overburden load on the slabs further reduce the possibility of soil/wall separation along the southern side of the building. The only area where soil/wall separation might occur is on the northern side with no surface slab, as identified by NW and NE soil columns. However, this area is designed as elevator wells. The sidewall is designed to stand by itself; it does not carry any load from the building. The potential soil-wall separation in this limited area is not expected to change the response of the buildings for design parameters obtained for design.	
SM-86	24590-HLW-S0C-S15T-00007 - SSI Analysis	Reviewer: Stephen A. Short It is noted on Sheet 11 of the SSI calculations that based on the SASSI computed transfer functions, east-west direction seismic responses appear to be greater than north-south direction seismic responses. This observation is also true in ISRS and base shear results presented elsewhere. The fixed base fundamental frequencies of the HLW are about 9 Hz in the east-west direction and 11 Hz in the north-south direction. It is difficult to assess the frequency of the soil-structure system from the transfer	USACE: “Observation See comment SM-5 & 29 on narrow peak response spectra.” BNI/Khan: USACE refers to SM-5 & 29 in their remarks. Although these two comments were included in informal communiqués to BNI from ORP, these comments were acknowledged by ORP as being ORP’s action and were therefore excluded from the formal transmittal of comments in ORP letter CCN 137083 (06-WED-012	Closed

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		functions generated by SASSI considering the three soil profiles. However, based on ISRS at Elevation 57 in the middle of the building, it appears that the fundamental frequency of the soil-structure systems is about 4 Hz for east-west response and less than 4 Hz for north-south response. The free field ground response spectra has peak spectral accelerations in the 4 to 6 Hz range with relatively steep reductions in accelerations at frequencies above and below this range. Hence, it appears that the narrow peak nature of the input response spectra may be having a strong effect on the computed seismic response. As discussed in earlier review comments, broad banded input spectra are encouraged for seismic design and narrow peaks are undesirable.	of 03.09.06)(refer to footnote on previous page).	
SM-87	24590-HLW-S0C-S15T-00008 - Seismic Loads	Tables 7-1 and 7-2 of the seismic loads calculation present maximum accelerations at each elevation of the HLW building. These accelerations were determined from the bubble plots that give the maximum accelerations at each location of the model from the SASSI seismic analysis. The accelerations that are used to determine seismic inertial loads for the equivalent static seismic analysis	Observation	Closed

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		of the building are presented in Table 1 of 4 on Sheet 112. Comparing acceleration values indicate that considerable conservatism is introduced when the seismic inertial loads values are selected. Base shears and story shears are shown in Figures 8-1 and 8-2 for the SASSI generated accelerations and for the selected accelerations to be used as inertial loads, respectively. Using the SASSI accelerations (weighted averaged maximum nodal accelerations), the base shear is 79 kips in the east-west direction and 66 kips in the north-south direction. Using the accelerations selected to establish seismic inertial loads, the base shear is 98 kips in the east-west direction and 84 kips in the north-south direction. This comparison demonstrates that conservatism on the order of 25 to 30% is introduced in the seismic loads used for determining design stresses and deformations.		
SM-88	24590-HLW-S0C-S15T-00091 - HLW Facility Structural Analysis with Refined Structural Model	Reviewer: Stephen A. Short A refined mesh finite element model was developed using the SAP2000 computer program. SAP2000 was used because GTSTRUDL was near its capacity before the mesh	Observation	Closed

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		refinements could be made. The purpose of the refined SAP2000 was to improve: • In-plane shear distribution in small wall piers • Transverse moments in shears in slabs • Areas with offsets in slabs and walls are directly addressed within SAP2000 to eliminate hand calculations required to evaluate additional moments induced by the offsets. • Significant size openings (i.e., openings with at least one side larger than the thickness) are adjusted to be more precisely represented. • use the thick shell element not available in GTSTRUDL to more accurately capture transverse shear stiffness and deformations. • enable cracked bending stiffness to be implemented without modifying the element thickness such that stress results are more readily available for design The SAP2000 model appears to have accomplished these goals and greatly improved the representation of the HLW building for the purposes		

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		of structural design over the GTSTRUDL model		

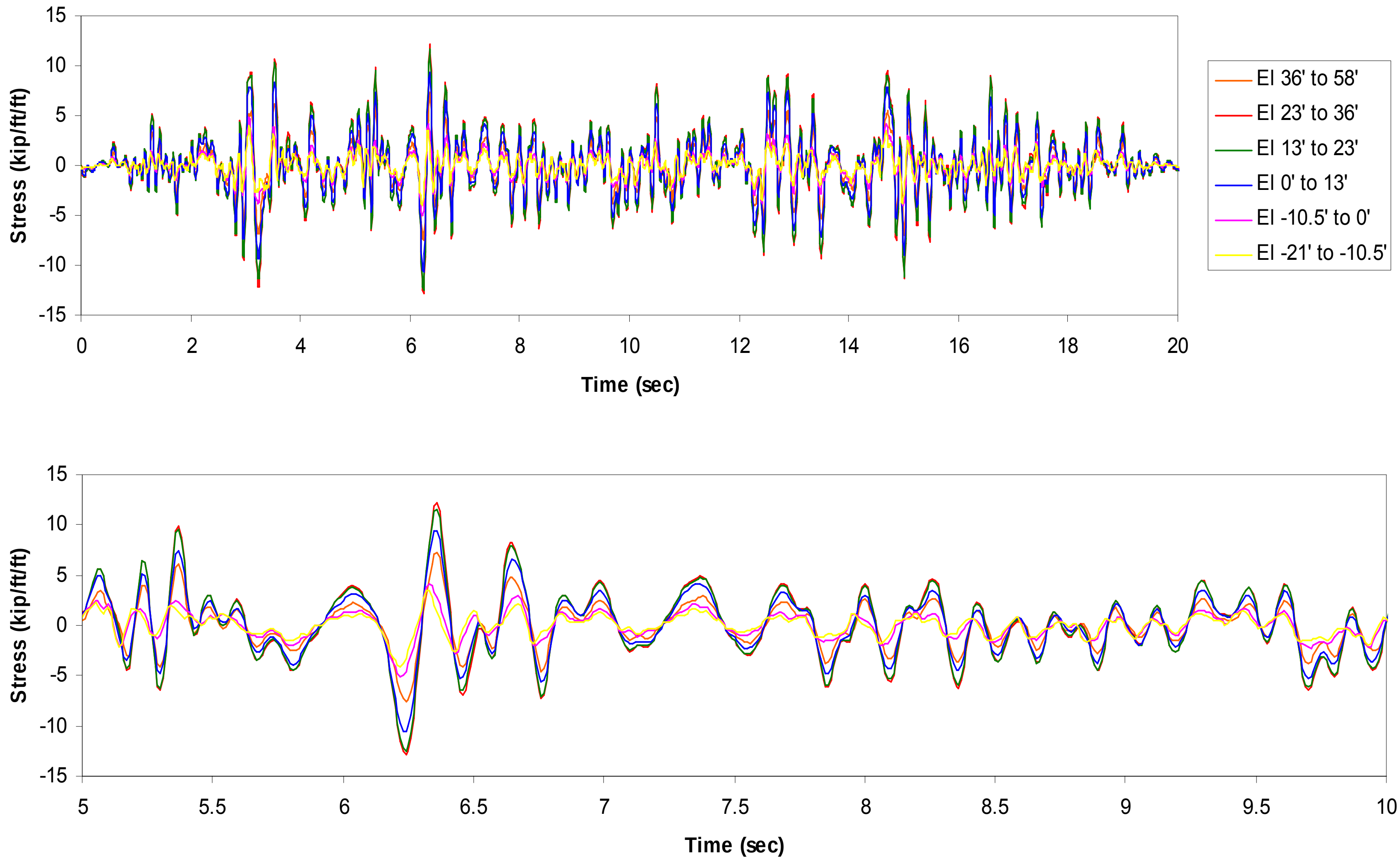
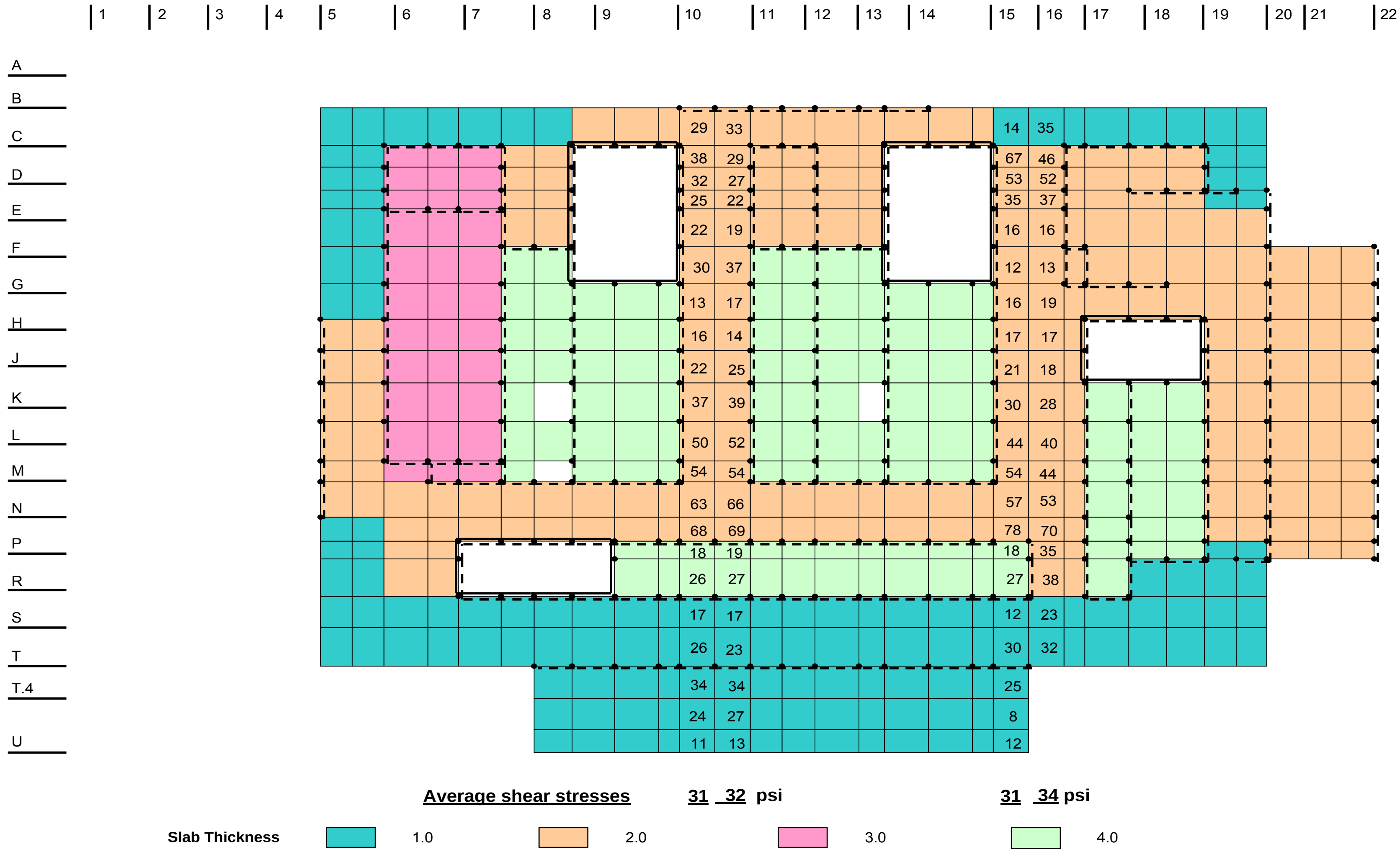


Figure SM-75-1 In-Plane Shear Stresses at Wall at line B of HLW, between 12 & 12.1 due to NS seismic



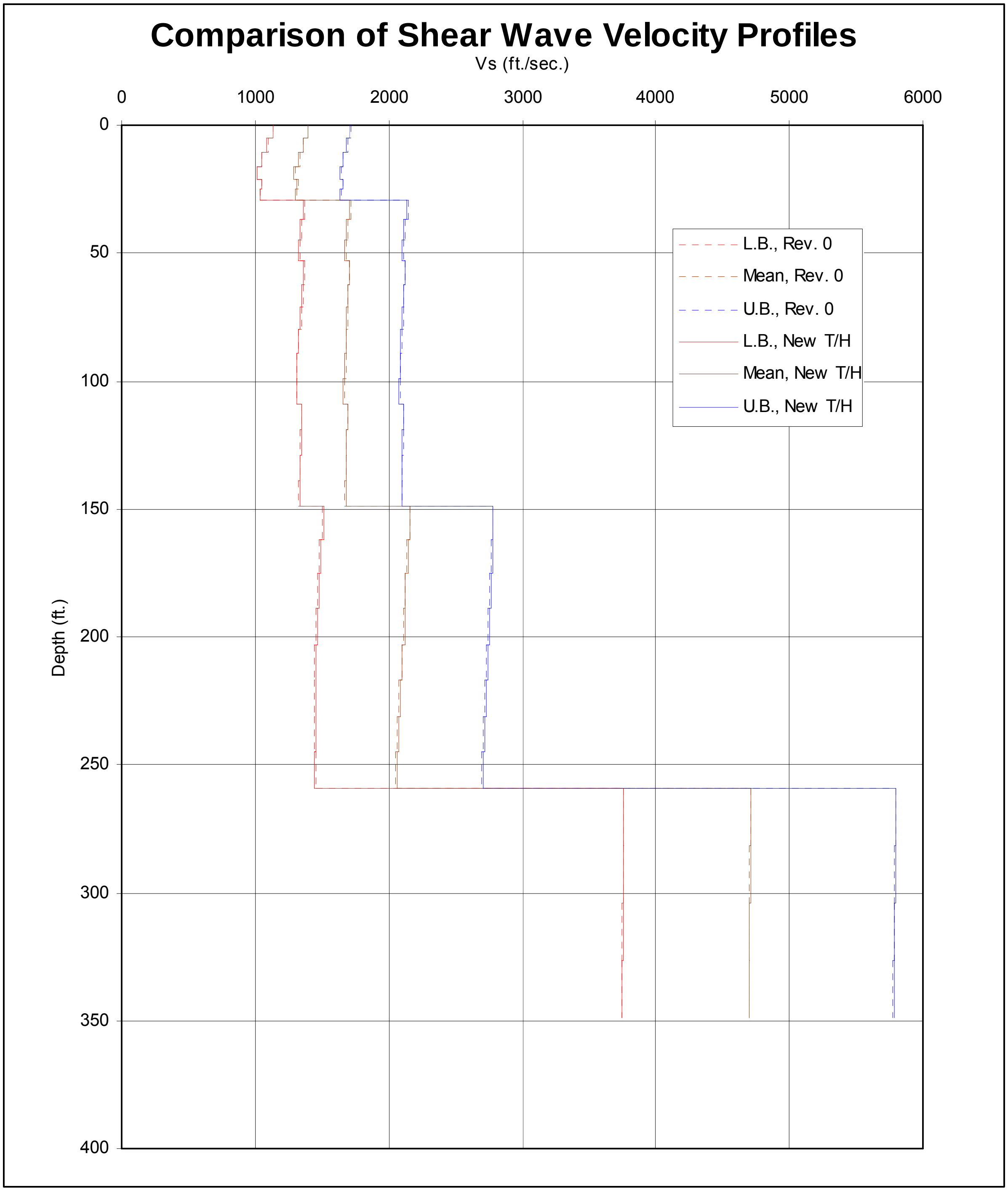
HLW-SSI analysis, rev. 0B (Upper Bound soil), Maximum Shear Stresses at El. 58', due to EW seismic

Figure SM-75-2

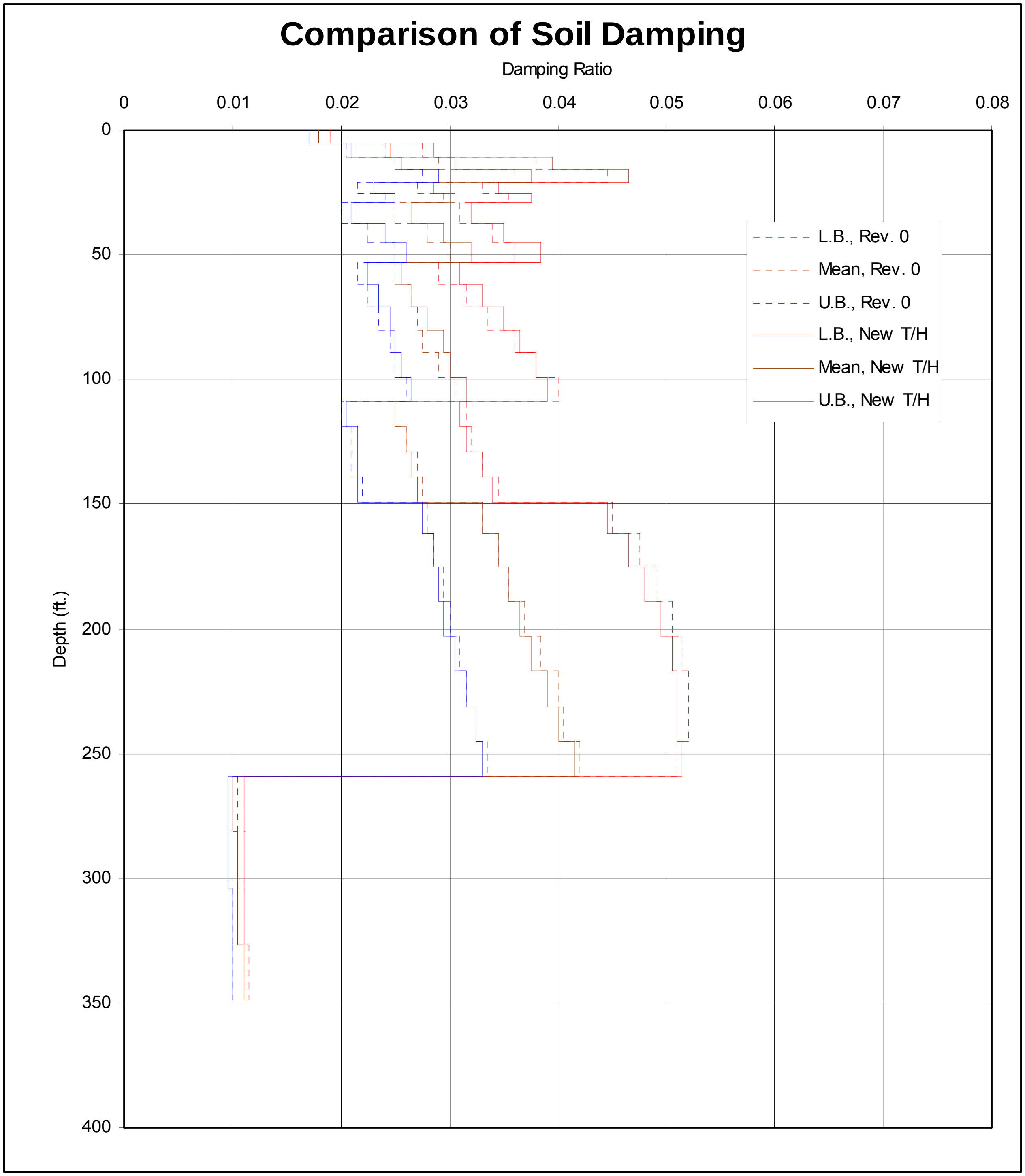
Table SM-75-1 Comparison of In-Plane Shear Stresses between Static and Dynamic Analyses

Shear Stresses at Slab at Elevation 57', due to EW Seismic Motion

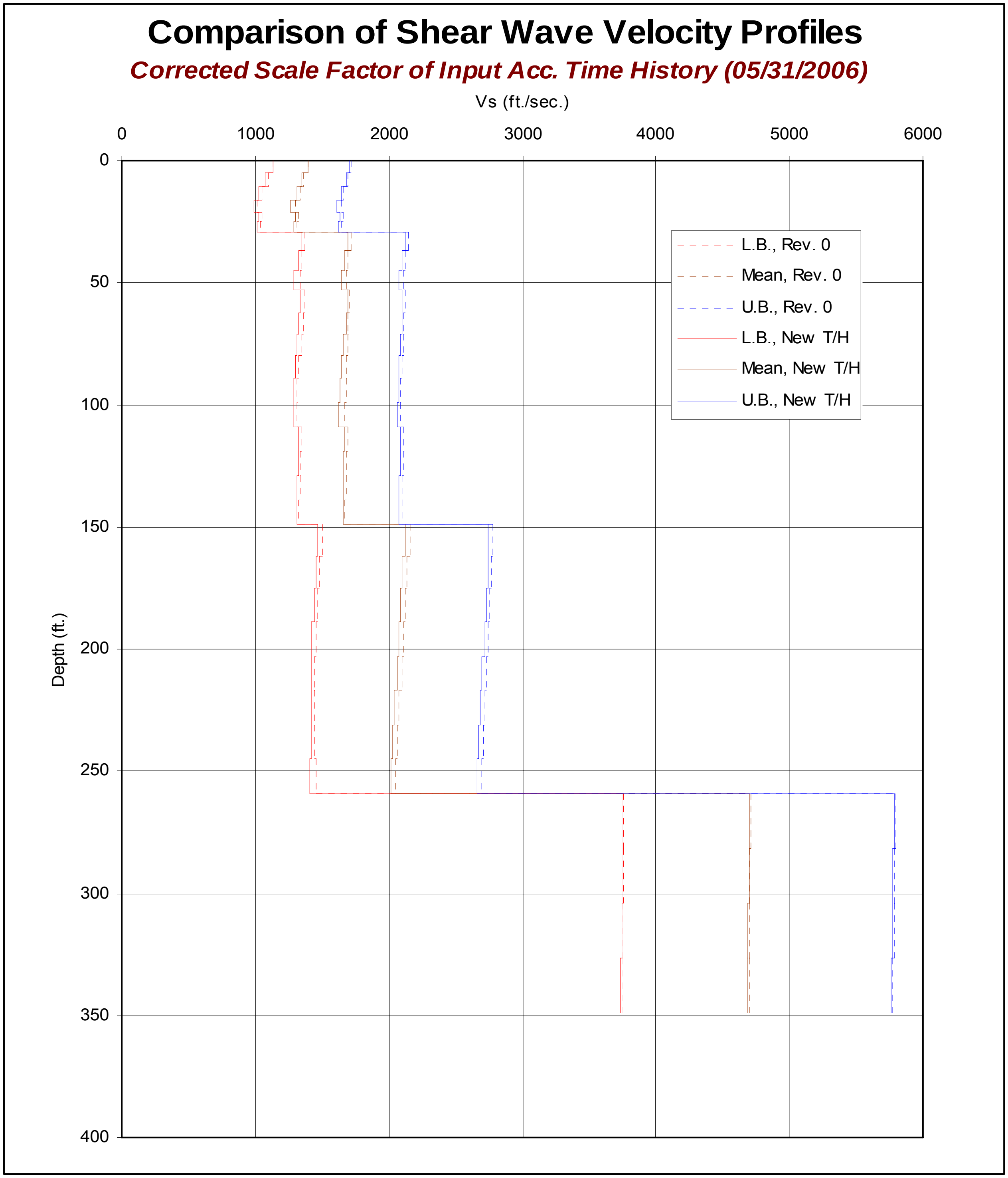
Column Line	Between Column Line	GTStrudl Element No.	SASSI Element No. (Group 8)	**SASSI Shear (psi)**	**GT-Strudl Shear (psi)**	**GT-Strudl / SASSI**	Column Line	Between Column Line	GTStrudl Element No.	SASSI Element No. (Group 8)	**SASSI Shear (psi)**	**GT-Strudl Shear (psi)**	**GT-Strudl / SASSI**
	B-C	7059	12	29	23	0.79		B-C	7068	21	14	5	0.38
	C-D	7099	36	38	49	1.27		C-D	7108	42	67	103	1.53
		7139	57	32	42	1.34			7148	63	53	75	1.40
	D-E	7179	78	25	36	1.44		D-E	7188	84	35	52	1.48
	E-F	7219	99	22	32	1.45		E-F	7228	105	16	21	1.32
	F-G	7259	123	30	47	1.54		F-G	7268	129	12	3	0.25
	G-H	7299	150	13	9	0.72		G-H	7308	159	16	32	2.04
	H-H.8	7339	180	16	23	1.49		H-H.8	7348	189	17	33	1.98
	H.8-J	7379	207	22	47	2.17		H.8-J	7388	216	21	39	1.88
	J-K.5	7419	233	37	76	2.09		J-K.5	7428	241	30	56	1.86
10	K.5-L	7459	262	50	97	1.96	15	K.5-L	7468	271	44	81	1.83
	L-M	7499	292	54	107	1.97		L-M	7508	301	54	94	1.76
	M-N	7539	322	63	145	2.29		M-N	7548	331	57	124	2.17
	N-N.7	7579	352	68	155	2.28		N-N.7	7588	361	78	164	2.10
	N.7-P	7619	378	18	36	2.05		N.7-P	7628	387	18	40	2.25
	P-R.2	7659	404	26	55	2.14		P-R.2	7668	413	27	53	1.97
	R.2-S	7699	431	17	12	0.68		R.2-S	7708	439	12	38	3.25
	S-T	7739	457	26	18	0.69		S-T	7748	466	30	62	2.03
	T-T.5	7779	478	34	78	2.30		T-T.5	7788	487	25	78	3.16
	T.5-U	7819	492	24	52	2.20		T.5-U	7828	501	8	33	4.33
		7859	506	11	23	2.14			7868	515	12	41	3.49
			Average	**31**	**55**	**1.67**				**Average**	**31**	**58**	**2.02**
	B-C	7060	13	33	33	0.98		B-C	7069	22	35	23	0.64
	C-D	7100	37	29	40	1.37		C-D	7109	43	46	65	1.42
		7140	58	27	36	1.32			7149	64	52	83	1.59
	D-E	7180	79	22	28	1.27		D-E	7189	85	37	56	1.52
	E-F	7220	100	19	25	1.30		E-F	7229	106	16	22	1.38
	F-G	7260	124	37	58	1.57		F-G	7269	130	13	13	1.05
	G-H	7300	151	17	15	0.88		G-H	7309	160	19	34	1.80
	H-H.8	7340	181	14	24	1.66		H-H.8	7349	190	17	33	1.88
	H.8-J	7380	208	25	52	2.12		H.8-J	7389	217	18	32	1.82
	J-K.5	7420	234	39	83	2.12		J-K.5	7429	242	28	56	2.03
11	K.5-L	7460	263	52	105	2.01	16	K.5-L	7469	272	40	79	1.99
	L-M	7500	293	54	108	1.99		L-M	7509	302	44	95	2.14
	M-N	7540	323	66	153	2.32		M-N	7549	332	53	118	2.24
	N-N.7	7580	353	69	154	2.22		N-N.7	7589	362	70	152	2.16
	N.7-P	7620	379	19	41	2.10		N.7-P	7629	388	35	82	2.35
	P-R.2	7660	405	27	55	2.05		P-R.2	7669	414	38	76	2.00
	R.2-S	7700	432	17	7	0.38		R.2-S	7709	440	23	15	0.64
	S-T	7740	456	23	15	0.63		S-T	7749	467	32	25	0.80
	T-T.5	7780	479	34	77	2.23		-					
	T.5-U	7820	493	27	58	2.18		-					
		7860	507	13	28	2.21		-					
			Average	**32**	**57**	**1.66**				**Average**	**34**	**59**	**1.64**



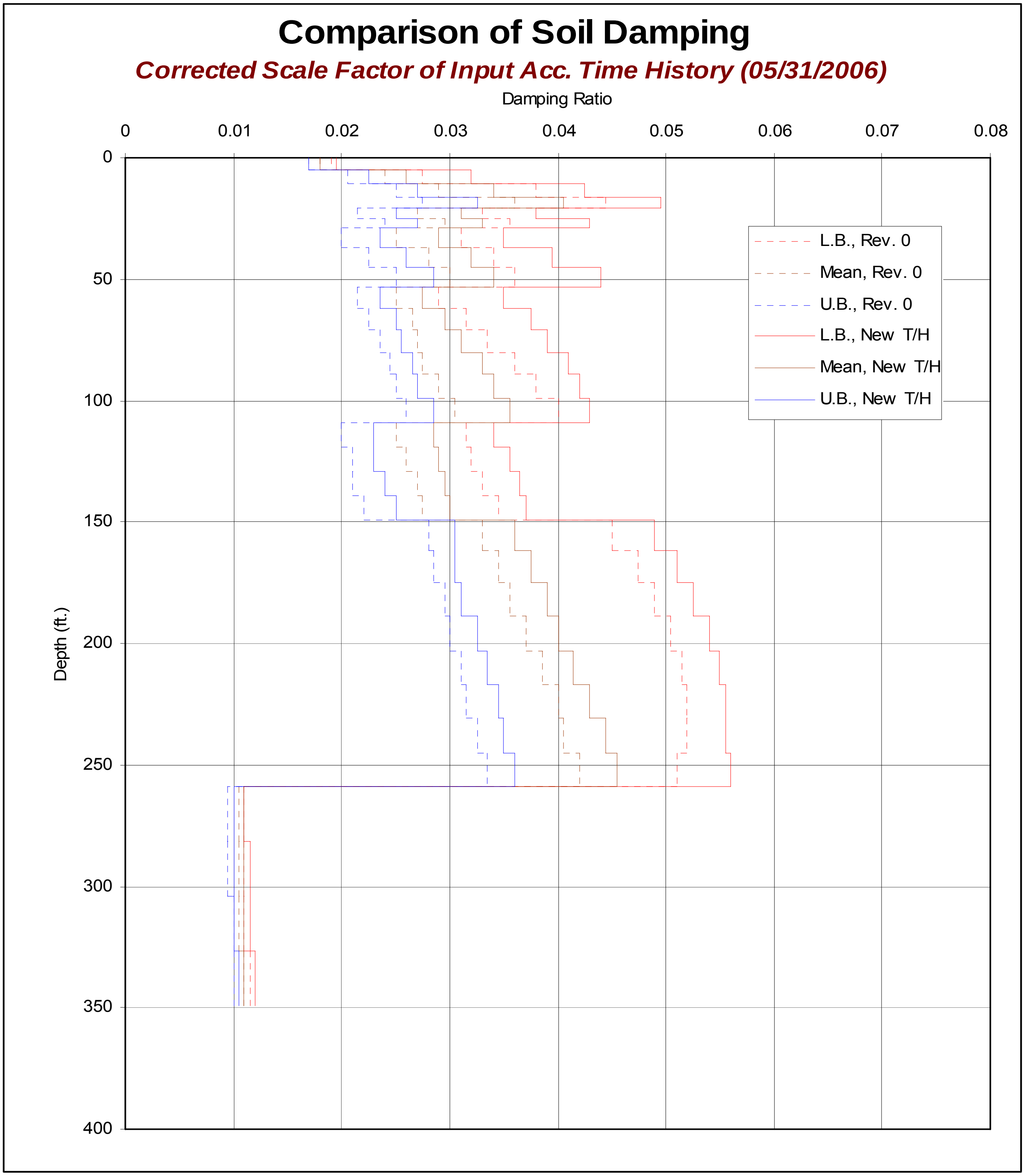
<u>Figure SM-80-1</u> Comparison of Strain Compatible Sv between Revisions 0 and 2 of PTF Free Field Analysis Calculation (#24590-PTF-S0C-S15T- 00001)



<u>Figure SM-80-2</u> Comparison of Strain Compatible Damping Ratio between Revisions 0 and 2 of PTF Free Field Analysis Calculation (#24590- PTF-S0C-S15T-00001)



<u>Figure SM-80-3</u> Comparison of Strain Compatible Sv between Revisions 0 of PTF Free Field Analysis Calculation (#24590-PTF-S0C-S15T-00001) and New SHAKE Analysis Results using the Corrected PGA



<u>Figure SM-80-4</u> Comparison of Strain Compatible Damping Ratio between Rev 0 of PTF Free Field Analysis Calculation (#24590-PTF-S0C-S15T-00001) and New SHAKE Analysis Results using the Corrected PGA

HLW Vitrification Building -- Finite Element Model for Dynamic Analysis (Rev. 0D) - Plan View at El. 0'

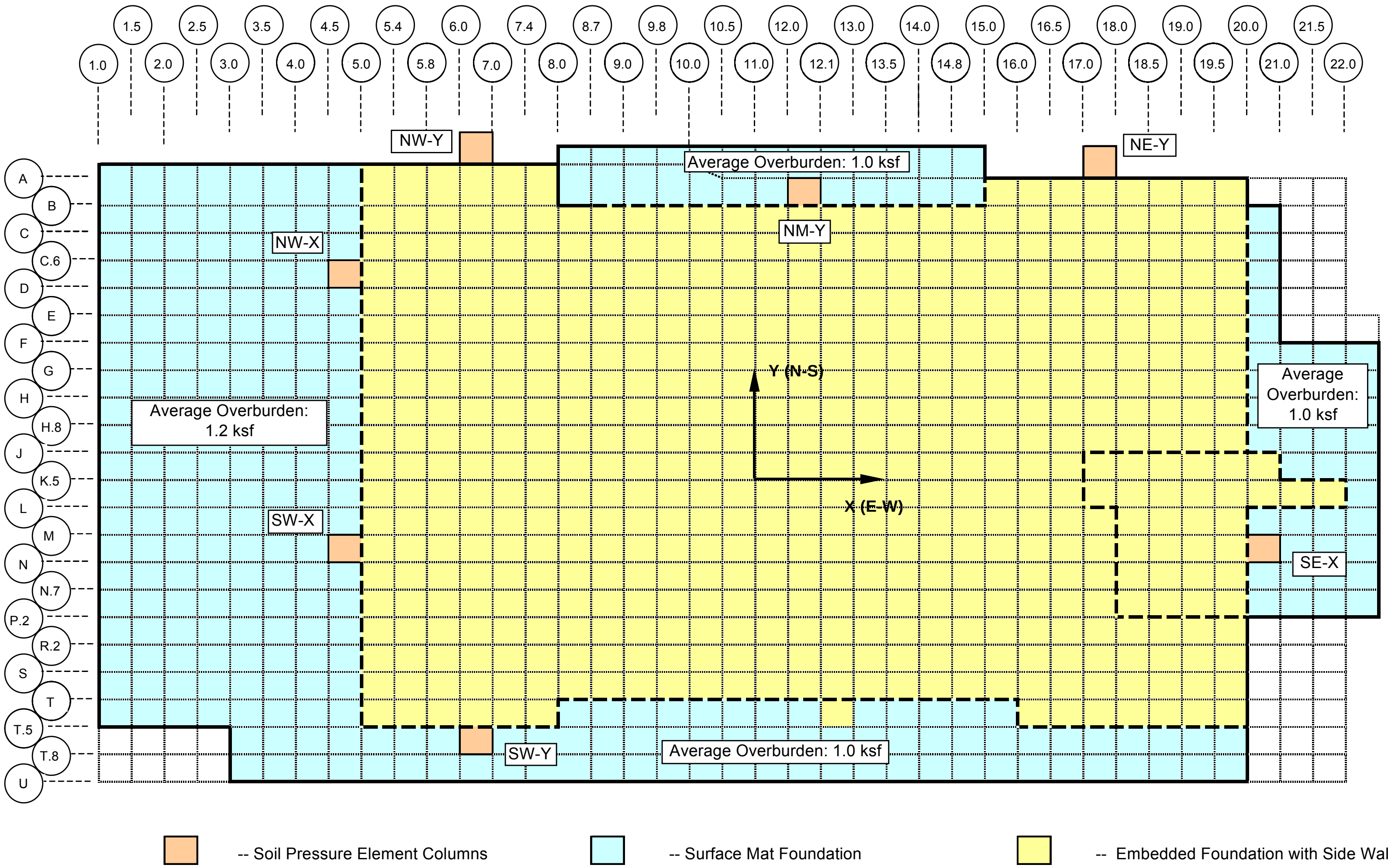


Figure SM-85-1

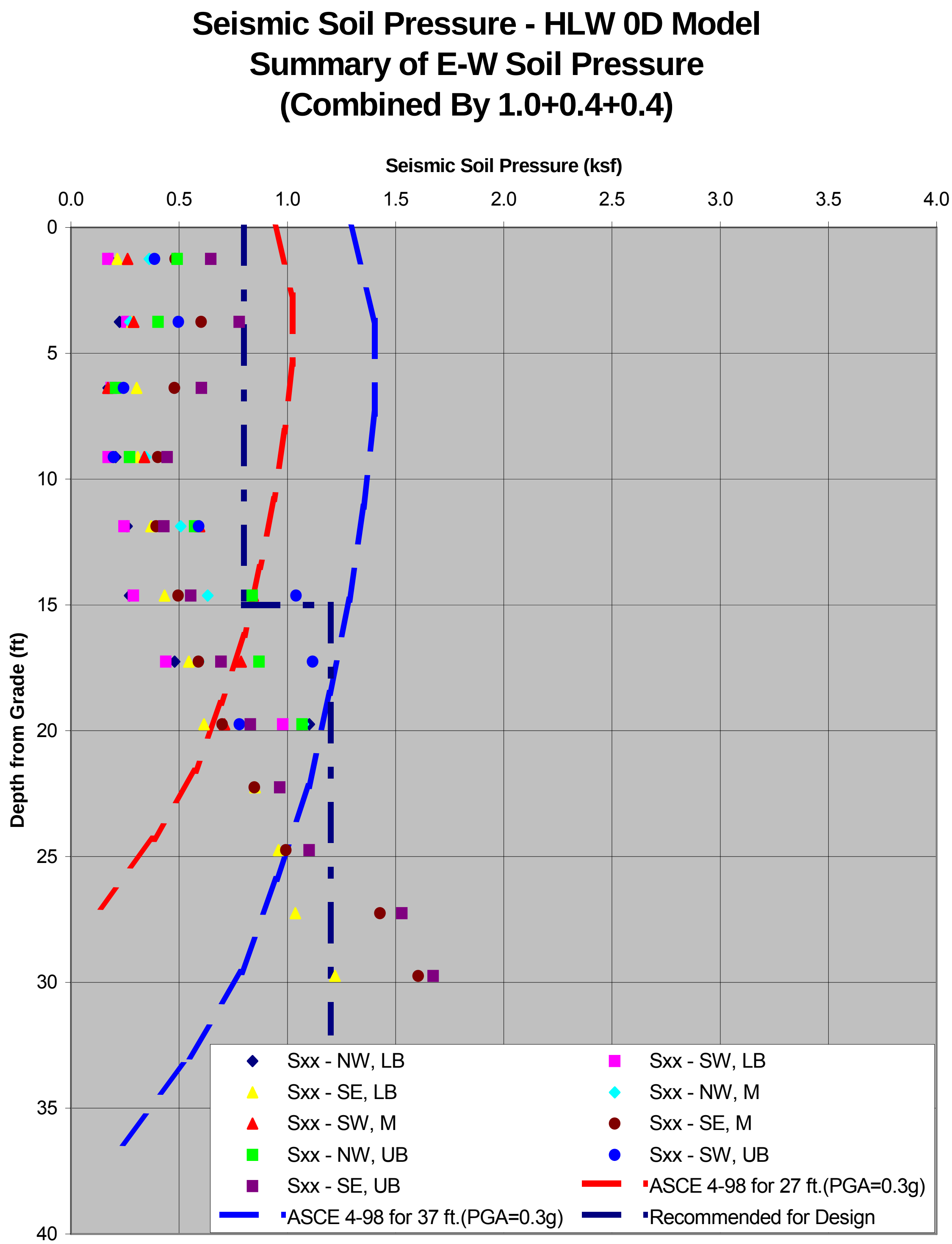


Figure SM-85-2 **Seismic Soil Pressure (figure 1 of 2)**

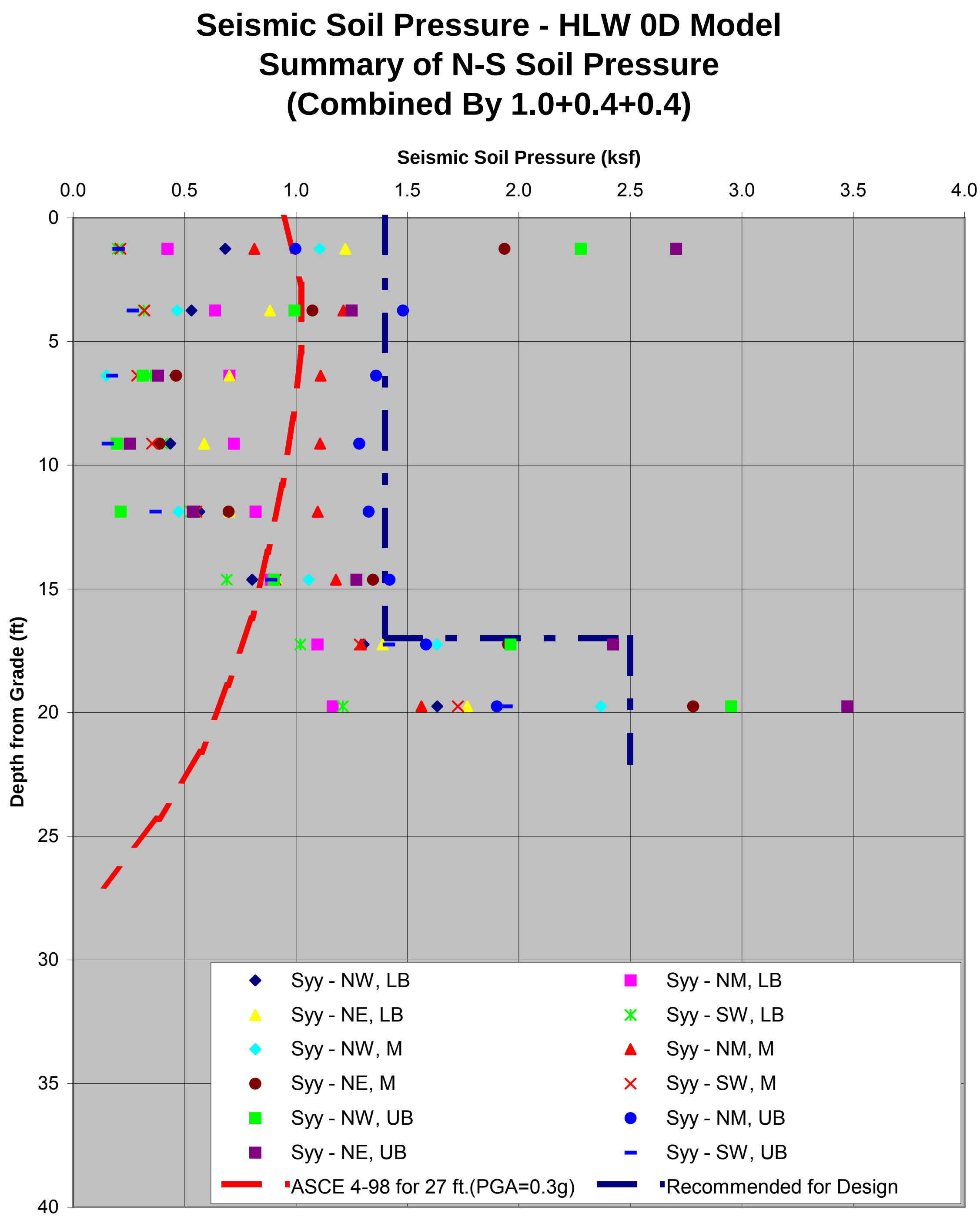


Figure SM-85-3 Seismic Soil Pressure (figure 2 of 2)

C.3 Load Path Review Comments

No.	Document	Comments and Observations	BNI Remarks/ Resolutions	Follow-up by USACE Review Team
LP-1	24590-WTP-RPT-ST-03-001_B, Summary Structural Report, Revision B, Volumes I, IIA, IIB 24590-HLW-DGE-13T-00045 Drawing Nos. 24590-HLW-P1-P01T-00001 to -00011	Reviewer: Philip S. Hashimoto Section 9 of 24590-WTP-RPT-ST-03-001_B, Summary Structural Report, Revision B, provides extensive data on seismic force distributions and review of shear wall discontinuities in the High Level Waste (HLW) Building. The extent and detail of this information significantly facilitated the review. Section 9 of the Summary Structural Report identified several shear wall discontinuities in the HLW Building. These discontinuities are addressed primarily in a qualitative manner. Additional discussion on a few of these discontinuities is provided in Appendix I to the Summary Structural Report. This reviewer surveyed the seismic force distributions and general arrangement drawings, and identified a few additional shear wall discontinuities. These discontinuities have alternative load paths that appear to be capable of transmitting seismic forces to other structural components. (Note: A review of discussion on shear wall discontinuities documented in the Summary Structural Report was performed by another reviewer.)	Observation BNI/White: The forces associated with the discontinuous walls will be updated to reflect the loads from the revised earthquake, the refined finite element configuration and the modified stiffness due to hypothetical/potential cracking. The level of detail will be similar to that provided in Revision B. Additional insight will be provided for the load path as more detailed analysis is preformed.	Closed.

C.3 Load Path Review Comments

No.	Document	Comments and Observations	BNI Remarks/ Resolutions	Follow-up by USACE Review Team
LP-2	24590-WTP-RPT-ST-03-001_B, Summary Structural Report, Revision B, Volumes I, IIA, IIB 24590-HLW-DGE-13T-00045 Drawing Nos. 24590-HLW-P1-P01T-00001 to -00011	Reviewer: Philip S. Hashimoto Seismic force distributions presented in Appendix I of the Summary Structural Report are based on the GTSTRUDL static analysis. BNI has indicated that this information will be updated for the SAP2000 static analysis, although the revision will not be incorporated in the near-term. Update of the Summary Structural Report for the SAP2000 analysis is encouraged, since this information is valuable in terms of understanding the behavior of the structure. Also, the SAP2000 analysis is expected to exhibit changes in the seismic force distributions as noted in the following comment.	BNI/White/Jeffrey: The forces in Appendix I will be updated to reflect the loads from the revised earthquake, the refined finite element configuration and the modified stiffness due to hypothetical/potential cracking. The level of detail will be similar to that provided in Revision B. Summary Structural Report based on GTSTRUDL static analysis will be updated for SAP 2000 static analysis at an appropriate time in the future.	Closed.
LP-3	24590-WTP-RPT-ST-03-001_B, Summary Structural Report, Revision B, Volumes I, IIA, IIB 24590-HLW-DGE-13T-00045 Drawing Nos. 24590-HLW-P1-P01T-00001 to -00011	Reviewer: Philip S. Hashimoto Seismic force distributions presented in Appendix I of the Summary Structural Report are based on the GTSTRUDL static analysis. This model assigned uncracked stiffness properties to the shear walls and floor diaphragms in the out-of-plane direction. It is understood that effective cracked stiffnesses based upon 50% of the uncracked stiffnesses are being used in the SAP2000 static model. A sensitivity study into the impact of the effective cracked stiffnesses is documented in	Observation	Closed.

C.3 Load Path Review Comments				
No.	**Document**	**Comments and Observations**	**BNI Remarks/ Resolutions**	**Follow-up by USACE Review Team**
		Appendix K of the Summary Structural Report. This sensitivity study indicates that significant changes in seismic forces in the floor diaphragms are possible. In-plane forces in the floor diaphragms calculated by the SAP2000 model should be carefully reviewed and addressed in the concrete design. Examples are identified in other comments below.		
LP-4	24590-WTP-RPT-ST-03-001_B, Summary Structural Report, Revision B, Volumes I, IIA, IIB 24590-HLW-DGE-13T-00045 Drawing Nos. 24590-HLW-P1-P01T-00001 to -00011	Reviewer: Philip S. Hashimoto The floor segment at Elevation 37 feet bounded by Column Lines 7, 15, R, and T forms a long, narrow diaphragm segment. In-plane shears and moments at the ends of this diaphragm segment from the GTSTRUDL model are provided in Section 9 of the Summary Structural Report. Out-of-plane shears on the walls on Column Lines R and T are provided in Appendix I of the Summary Structural Report. Section cut forces for the diaphragm and walls from the SAP2000 model were also provided by BNI at the reviewer's request. The GTSTRUDL analysis indicates very large out-of-plane shears in the walls on Column Lines R and T. The GTSTRUDL modeled assigned uncracked stiffnesses to the walls.	BNI/Jeffrey: The finite element model loadings have been reviewed and found to be applied correctly. Results from these cuts will be addressed in design. The two questions addressed here are directly related. The twisting moment on the wall cut is an expression of the change in out-of-plane shear across the cut. When smaller cuts are taken (4) subdividing the cut R1, the twisting moment reduced significantly (~2700ft*k vs. ~450ft*k). Note that within this cut there is a sign change on the out of plane shears. The twisting moment represented in these cuts is the moment due to shears summed about the centroid of the cut, and not the applicable design moment. The moment applicable for design would be obtained from a cut that is taken vertically on the walls for out-of-plane bending.	Closed.

C.3 Load Path Review Comments

No.	Document	Comments and Observations	BNI Remarks/ Resolutions	Follow-up by USACE Review Team
		The wall out-of-plane shears were significantly reduced in the SAP2000 analysis. As expected, the use of effective cracked wall out-of-plane stiffnesses results in reduced out-of-plane shears on the walls. However, the out-of-plane shears on the wall on Column Line R appear to act in the opposite direction from the shears on the wall on Column Line T. An explanation for this unexpected behavior should be provided. In-plane shears on the diaphragm segment from the SAP2000 and GTSTRUDL analyses are comparable. The in-plane moments at the ends of the diaphragm segment at Column Lines 7 and 15 appear to be very low in comparison to the north-south diaphragm load. It appears that most of the moments at the ends of the diaphragm segment are actually being resisted by twisting of the walls on Column Lines R and T. That is, moments about the vertical axis from section cuts through the walls just above and just below the diaphragm are very high. These unexpected analysis results should be reviewed further to confirm they are reasonable and, if so, addressed in the concrete design.	When a long cut across wall R and T is taken, the resulting out-of-plane shear is in the same direction because this area of the building is globally displacing in the south direction from this loading. Linking walls between R and M cause the smaller displacements around column line 9. These smaller displacements can cause a tension in the EL 37 slab because walls R and T are not displacing at the same rate. This in turn will cause reversed out-of-plane shears in the wall cuts along R and T. Additionally, the out-of-plane shear loads found are very near a neutral force position and therefore a force reversal does not carry a high importance (+/- 50 psi). An exaggerated plot of the deflected shape of EL 37 and walls R and T shows that the walls R and T are acting as “Beam flanges” across this opening with EL 37 acting as the “web.” This action adds to the twisting moments represented in the wall cuts. Additionally, plotting of in plane shear on the slab at EL 37 shows a sign reversal near the center of the opening which indicates a torsion due to difference in stiffness between the west and east side of the structure.	

C.3 Load Path Review Comments				
No.	**Document**	**Comments and Observations**	**BNI Remarks/ Resolutions**	**Follow-up by USACE Review Team**
LP-5	24590-WTP-RPT-ST-03-001_B, Summary Structural Report, Revision B, Volumes I, IIA, IIB 24590-HLW-DGE-13T-00045 Drawing Nos. 24590-HLW-P1-P01T-00001 to -00011	Reviewer: Philip S. Hashimoto The shear walls on Column Lines D.7, J.4, and K.7 provide lateral support for the narrow diaphragm segments at Elevation 37 feet. Reactions from the diaphragm segments onto these walls will also be influenced by the use of effective cracked out-of-plane wall stiffnesses. The intersections between the shear walls and diaphragm segments constitute "hard points" that will experience concentrated reactions. These reactions should be carefully reviewed and addressed in the concrete design.	Observation	Closed.
LP-6	24590-WTP-RPT-ST-03-001_B, Summary Structural Report, Revision B, Volumes I, IIA, IIB 24590-HLW-DGE-13T-00045 Drawing Nos. 24590-HLW-P1-P01T-00001 to -00011	Reviewer: Philip S. Hashimoto a) The GTSTRUDL analysis indicates that the east-west wall on Column Line A.0 receives a 1,064 kip force from the diaphragm at Elevation 0 due the original design earthquake ground motion. Review of the drag strut design at this location indicates that 15-#11 bars are required to transmit this force. Thirteen straight bars and 3 "U" bars are provided. The U bars terminate only 2'-5" from the wall-to-floor interface. BNI indicated that the	BNI/Axup: Upon further discussion with the engineers responsible for the original design, it was agreed that, given the final analysis of the structure currently in progress, it would not be necessary to perform calculations to show that concrete shear capacity in the slab/wall joint, in combination with the reinforcement added, would be able to provide adequate transfer of drag strut forces into the slab. As pointed out in the review observation,	Closed.

C.3 Load Path Review Comments

No.	Document	Comments and Observations	BNI Remarks/ Resolutions	Follow-up by USACE Review Team
		force transmitted by the U bars was considered to be in turn transmitted to the north-south wall on Column Line 15 by out-of-plane shear. This design consideration should be documented. b) For the original design ground motion increased by 40%, 19-#11 bars are required to transmit the drag force into the wall. The existing reinforcement is less than that required. However, BNI expects the final drag force for the revised ground motion to actually reduce. The increased finite element mesh refinement in the SAP2000 model is expected to show that more force is transmitted into the wall on Column Line 15, rather than the wall on Column Line A.0. This expectation seems plausible. However, the final analysis and design should carefully review the drag force at this location because of the high force exhibited by previous analyses.	the portion of the slab in question was placed on Hold pending final analysis. Since that time, the new revision of the Structural Design Criteria (SDC) has been issued. The new SDC incorporates provisions for combining thermal and seismic forces in load combinations. Also, a revised analysis of the HLW building is nearing completion which, through the use of a refined mesh, and the use of cracked section properties, now show the drag strut load to be significantly lower than was previously shown in calculation 24590-HLW-DGC-S13T-00045, Rev. A, where the requirement for 15 - # 11 bars was first shown. Preliminary loads taken from the revised model show that the load has reduced from 1,043 kips to approximately 770 kips, a 25% reduction. It is still BNI's intention to maintain the Hold on the portion of the concrete slab until the revised analysis is complete and the new drag forces are incorporated in the detailed design. Based on the new analysis, in conjunction with the new provisions of the SDC, the straight bars originally provided (neglecting the U-bars) will be shown to be sufficient to transfer the drag strut forces in the wall to the slab. These design considerations will be clearly documented in the final version of	

C.3 Load Path Review Comments				
No.	**Document**	**Comments and Observations**	**BNI Remarks/ Resolutions**	**Follow-up by USACE Review Team**
			the calculation. USACE: b) Observation	
LP-7	24590-PTF-SOC-S15T-00047 Pretreatment Facility Structural Analysis with Refined Structural Model 24590-PTF-P1-P01T-00001 thru 00006 (General Arrangement Plans) 24590-PTF-DB-S13T-x (Structural Concrete Wall Sections)	Reviewer: John Connor a) Wall section 4 is shown on sheet A-17 and modeled on sheet G-4. The wall details can be seen on drawing sheet -S13T-00105. Sheet A-17 shows hundreds of sleeve penetrations, some closely spaced and ganged together. Some of the penetrations groupings cover an area comparable to other doors or windows. The model shown on sheet G-4 does not show any openings to represent the ganged pipe penetrations. The model is shown with various colors, which appear to indicate wall thickness changes. The colors appear to correlate with the wall thickness changes rather than equivalent element thicknesses. It appears that an equivalent element thickness was not considered for these ganged openings. b) Wall section 9 is shown on sheet A-19 and modeled on sheet G-10. The wall details can be seen on drawing sheet -S13T-00113. Sheet A-19 shows hundreds of sleeve penetrations, some closely spaced	BNI/Jeffrey: (a) Wall on column line 4 carries a large number of pipe sleeves in the area between col lines H and L and elevations 38' and 56'. These sleeves vary in diameter from 3" to 8" and thickness of material is schedule 40 up to 6" dia and schedule 20 for 8" dia and larger. All sleeves 3" dia or larger are provided with shear studs. Wall detailing is such that no reinforcing bars are cut to accommodate the sleeves, which are approx 12" c/c min. These openings are not shown in the model as penetrations; however, in the above area wall thickness is reduced from 4' to 3'-6" to account for loss of concrete. Note that the pipe sleeves with shear studs restore some strength in the area where concrete is lost due to penetration; however, this strength is not included in arriving at equivalent element thickness. This wall in its entire length carries 13.9 % of the N-S shear. Approx. 1/4 of its length and 1/3 of its height is reduced in thickness from 4' to 3'-6". We have further reviewed this and compared the Moment of Inertia and Shear area at the penetrations on	Closed.

C.3 Load Path Review Comments

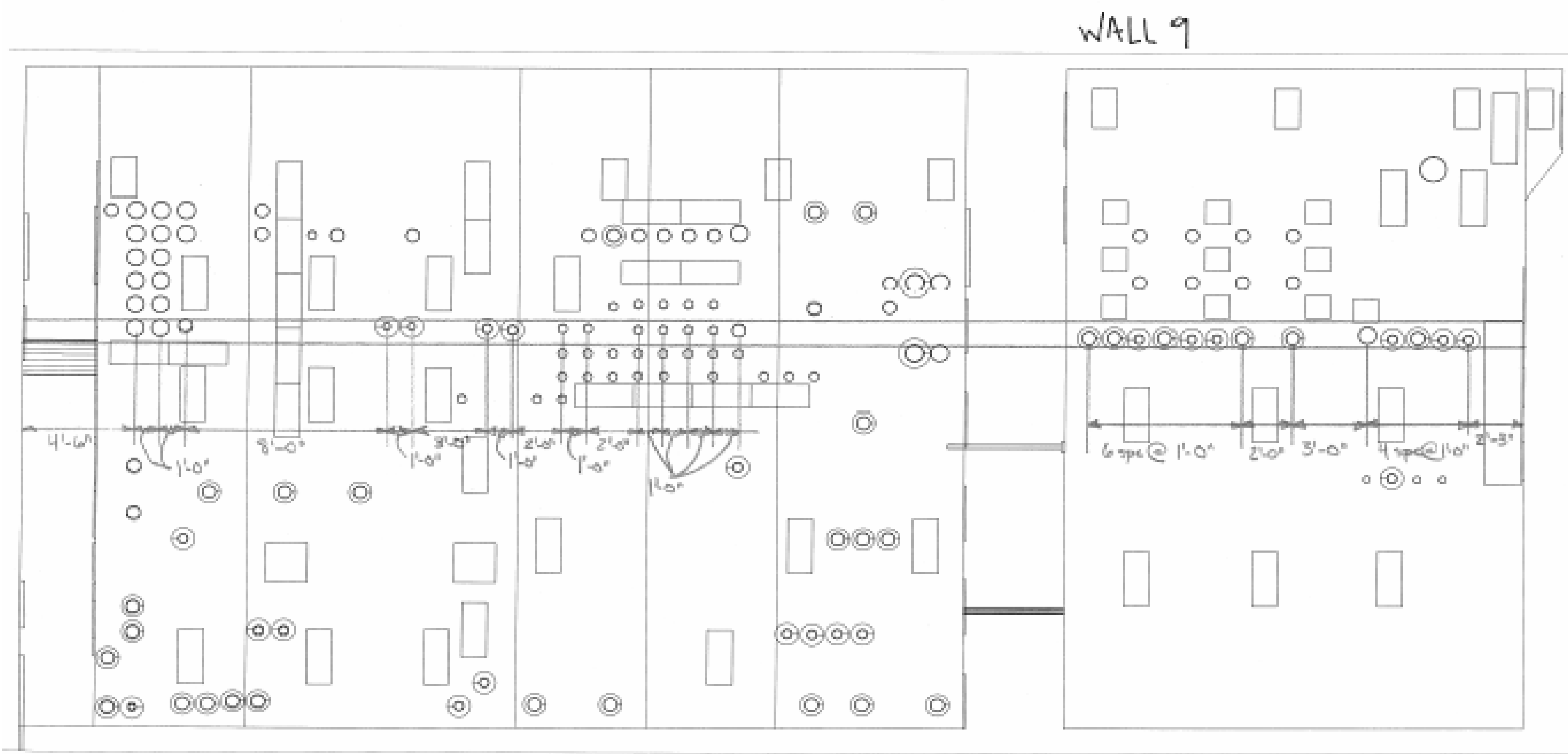
No.	Document	Comments and Observations	BNI Remarks/ Resolutions	Follow-up by USACE Review Team
		and ganged together. Some of the penetrations groupings cover an area comparable to other doors or windows. The model shown on sheet G-10 does not show any openings. The model is shown with 2 colors, which appear to indicate wall thickness changes. The 2 colors appear to correlate with the wall thickness change from 5' to 4'. It appears that an equivalent element thickness was not considered for these ganged openings. c) The load path is not clear, considering that the arrangement and sizes of penetrations varies throughout the wall. It may not be practical to model these ganged pipe penetrations as "holes" in the main SAP building model. Using the equivalent wall thickness approach seems appropriate for the main SAP building model. Walls sections, such as wall 4 and 9, should be modeled separately. A single wall section could be modeled as a stand-alone SAP model file, using forces obtained from the global SAP model file. The single wall section could be modeled with greater detail to account for the ganged penetrations.	attached sketches and spreadsheet (following this table of comment resolutions: Sketches- 3 in total, Spreadsheet - 1 in total). It can be seen that while this wall becomes more representative of its physical properties, the actual redistribution of load is negligible. This is due to (a) reduced concrete thickness is over a small portion of wall, approx 25%, and (b) shear capacity is dominated by reinforcing steel. (b) Wall on col line 9 also carries a large number of pipe sleeves in the area between col lines C and E and elevations 38' to 56'. These sleeves are similar to those described in item (a). These openings too are not shown in the model and wall thickness is reduced from 5' to 4' to account for penetrations. This wall carries only 2.8 % of N-S shear and same reasoning applies here as indicated above in item (a). (c) See LP-8	
LP-8	24590-PTF-SOC-S15T-00047	Reviewer: John Connor	BNI/Jeffrey:	Closed.

C.3 Load Path Review Comments

No.	Document	Comments and Observations	BNI Remarks/ Resolutions	Follow-up by USACE Review Team
	Pretreatment Facility Structural Analysis with Refined Structural Model	The methodology of reducing the element thickness to account for the change in wall stiffness is reasonable for the global building model. However, it is not clear how an equivalent thickness was determined. An example problem or a narrative should be provided to clarify assumptions and approach.	The methodology used to account for penetration is to equally reduce the thickness of concrete in the area of openings to account for the lost concrete. If the openings represent approx. 1/4 of the wall surface area, then wall thickness is reduced by 1/4 in the same area and rounded off. This is done by comparison of openings type and shape using judgment. A narrative to this effect will be added in the calculations methodology to clarify the approach as suggested.	
LP-9	Overview of project from Mark Braccia.	Reviewer: Mark Summers Recommend seismic instrumentation is added to the PT and HLW buildings to record actual accelerations within the buildings. Should an event occur, a record of the floor accelerations could be used to aid in the evaluation of the facility.	BNI/Jeffrey, Braccia: USACE recommendation that seismic instruments be added to the PTF and HLW buildings to record actual accelerations within the buildings will be forwarded to DOE.	DOE's position?

Supporting Information to LP-7 Resolution

<u>Sketches</u> (1st of 3)



$I_{EQ} = 70024$

$I_x = 69704$

$I_R = 0.995$

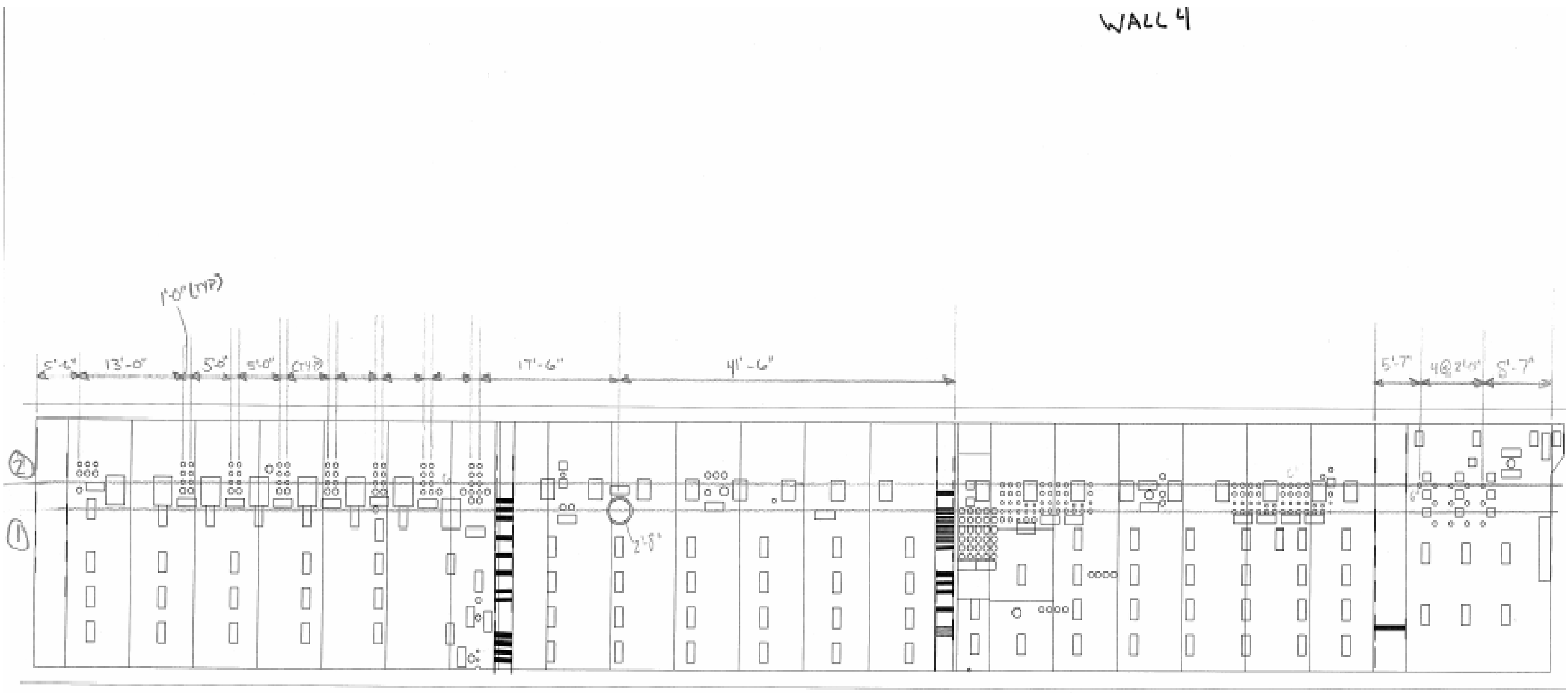
$A_{EQ} = 249\ ft^2$

$A_x = 240\ ft^2$

$A_R = 0.963$

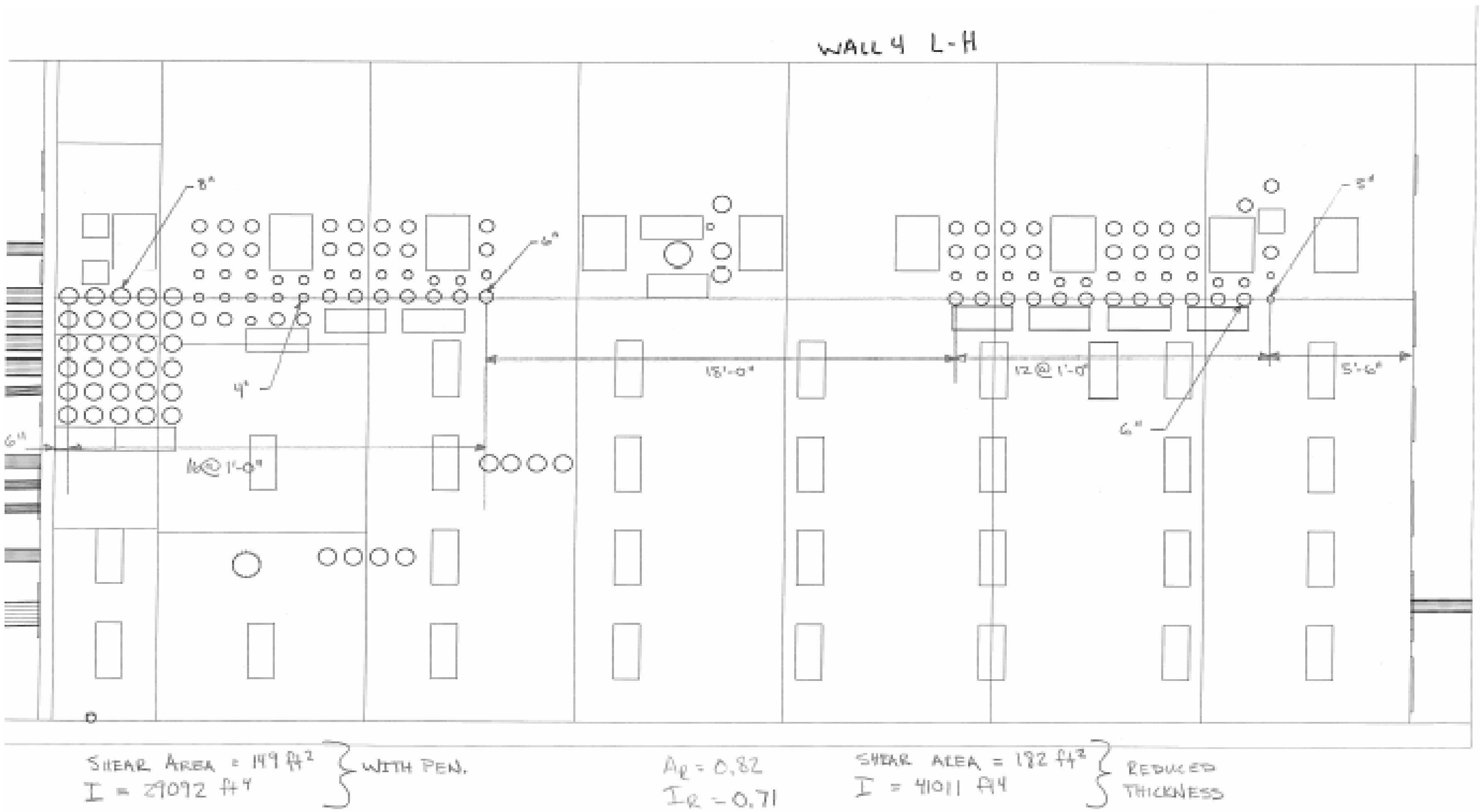
Supporting Information to LP-7 Resolution

<u>Sketches</u> (2nd of 3)



Supporting Information to LP-7 Resolution

<u>Sketches</u> (3rd of 3)



Supporting Information to LP-7 Resolution

<u>Spreadsheet</u> (1 of 1)

Wall 4	Gross Values		Reduced Thickness		Reduced Thick / Gross		Reduced for Penetrations		Reduced Pen / Gross		Shear Capacity Ratio to Gross		Difference between two methods	Remarks
	Area (ft^2)	I (ft^4)	Area (ft^2)	I (ft^4)	Area Ratio	I Ratio	Area (ft^2)	I (ft^4)	Area Ratio	I Ratio	Red Thick	Red Pen		
Line 1 Full Length	813.5	2697766	787.5	2610034	0.97	0.97	741	2518715	0.91	0.93	0.99	0.98	1%	Negligible
Line 2 Full Length	813.5	2697766	787.5	2610034	0.97	0.97	726	2397660	0.89	0.89	0.99	0.98	1%	Negligible

Notes:
For calculation of shear capacity, the Vs remains the same as the gross model because no rebar has been cut for penetrations. A reinforcement ratio of 0.01 was used based on 2 # 11 bars @ 12" E.W. E.F. from issued for construction drawings.

C.4 Concrete Design Review Comments				
No.	**Document**	**Comments and Observations**	**BNI Remarks/ Resolutions**	**Follow-up by USACE Review Team**
CD-1	3DG C13 014, *Revision 3, Engineering Design Guide for Embeds and Surface Mounted Plates*, September 2004	Reviewer: Philip S. Hashimoto Section 9.0 presents an example problem illustrating application of the embed plate load capacity tables. This example problem lacks sufficient detail to enable a user to implement the tables. For example, in the calculation of the equivalent axial load P, it is unclear what the term 1.5/0.25 represents.	BNI/Adediran: The subject comments on Bechtel corporate design guide 3DG-C13-014 are helpful and will be considered when the design guide is issued. At this time, however, it has not been determined if the guide would be used for the WTP project.	Closed.
CD-2	3DG C13 014, *Revision 3, Engineering Design Guide for Embeds and Surface Mounted Plates*, September 2004	Reviewer: Philip S. Hashimoto Appendix A indicates that capacities for concrete expansion anchors are based upon allowable capacities for Ramset/Redhead Trubolt Wedge Anchors in ICBO ER-1372 increased by a factor of three. Relatively high expansion anchor capacities appear to result. As an example, based on Table D-49, the pure tension and pure shear capacities for a 1/2 inch diameter anchors having a minimum embedment of 4.125 inches in concrete having a minimum compressive strength of 4,000 psi both appear to be about 5,000 lbs. The design guide does not provide the data that form the basis for the expansion anchor capacities adopted. Ramset/Redhead technical data indicate that the Trubolt Wedge	BNI/Adediran: See response to CD-1.	The comment is closed if the design guide is not used for the WTP project. However, the comment should be addressed if it is implemented on the WTP project.

C.4 Concrete Design Review Comments				
No.	**Document**	**Comments and Observations**	**BNI Remarks/ Resolutions**	**Follow-up by USACE Review Team**
		Anchor capacities in ICBO ER-1372 represent ultimate capacities divided by a factor of safety of four. The factor of safety used to determine allowable anchor capacities is normally based on the <u>mean</u> ultimate capacities. The expansion anchor capacities used in the design guide may thus approach mean ultimate values. Given the larger uncertainties in expansion anchor capacities due to factors such as installation, the expansion anchor capacities used in the design guide may not be sufficiently conservative for design application. DOE-STD-1020-94 requires that anchorage capacities for design of new installations have a factor of safety of four. A factor of safety of three is permitted for existing anchorage only when detailed inspections and evaluations (e.g., tightness checks, confirmation no concrete cracking) have been performed. The factor of safety of three is implicit in nominal capacities for expansion anchors permitted by DOE/EH-0545, which is intended for application to existing DOE facilities. Table 3 of Bechtel Document No. 3DG-C13-013, *Engineering Design Guide for Concrete Expansion*		

C.4 Concrete Design Review Comments				
No.	**Document**	**Comments and Observations**	**BNI Remarks/ Resolutions**	**Follow-up by USACE Review Team**
		Anchors, provides allowable design capacities for wedge/sleeve anchors. These capacities are based upon a factor of safety of four. 3DG-C13-013 notes that they "are used in conjunction with both ultimate strength design and working stress design methods." For 1/2 inch diameter anchors with an embedment depth of 2.5 inches in concrete having a compressive strength of 4,000 psi, the allowable capacities in 3DG-C13-013 are comparable to those in ICBO ER-1372 (tension capacities of 1,200 lbs from 3DG-C13-013 and 1,275 lbs from ICBO ER-1372 with special inspection, shear capacities of 1,650 lbs from 3DG-C13-013 and 1,190 lbs from ICBO ER-1372). 3DG-C13-013 suggests that the ICBO ER-1372 capacities without an increase factor of three are appropriate. The expansion anchor capacities used by the design guide do not appear to satisfy DOE-STD-1020-94 requirements. They are also in conflict with 3DG-C13-013. If these capacities are to be used, they should be demonstrated to provide sufficient conservatism suitable for design.		
CD-3	3DG C13 014, *Revision 3,*	Reviewer: Philip S. Hashimoto	BNI/Adediran:	The comment is closed if the design guide is not used for the

C.4 Concrete Design Review Comments				
No.	**Document**	**Comments and Observations**	**BNI Remarks/ Resolutions**	**Follow-up by USACE Review Team**
	Engineering Design Guide for Embeds and Surface Mounted Plates, September 2004	Appendix A notes that the load tables for expansion anchors are applicable to all manufacturers with similar bolt material strength and embedment. However, Appendix A also indicates that the factor of three used to obtain ultimate capacities for expansion anchors from service capacities was based upon review of test data from various manufacturers. The load tables should be restricted to those anchor types whose data were used to develop the anchor capacities in the design guide.	See response to CD-1.	WTP project. However, the comment should be addressed if it is implemented on the WTP project.
CD-4	3DG C13 014, *Revision 3, Engineering Design Guide for Embeds and Surface Mounted Plates*, September 2004	Reviewer: Philip S. Hashimoto Appendix B presents the development of an entry in the load table for an embed plate that is loaded concentrically on the anchor bolt patter. To fully validate the load tables, similar calculations should be presented for other anchorage types, specifically embed plates that are not concentrically loaded, and surface mounted plates anchored by both Maxibolts and expansion anchors.	BNI/Adediran: See response to CD-1.	Closed.
CD-5	3DG C13 014, *Revision 3, Engineering Design Guide for Embeds and Surface Mounted*	Reviewer: Philip S. Hashimoto The calculation for an example load table entry in Appendix B, sample calculation for a non-standard	Observation	Closed.

C.4 Concrete Design Review Comments				
No.	**Document**	**Comments and Observations**	**BNI Remarks/ Resolutions**	**Follow-up by USACE Review Team**
	Plates, September 2004	embed plate in Appendix E, sample calculation for a nonstandard surface mounted plate in Appendix F, and sample calculation for a nonstandard embed plate using ACI 349-01 in Appendix H were reviewed and found to be reasonable.		
CD-6	24590-PTF-DGCS13T-00012 Design of Walls at Col Lines 25.5, 27, 28.5, 30, B, D, E, H, J, K, L & M from EL 28 to 56 and Dowels up to floor at EL 28'	Reviewer: William Bolte Wall at column line 30 has high D/C ratios (based on initial wall design calculations) without considering wall openings (other than previously described equivalent thickness method). It is necessary to re-examine with the actual wall configuration with piers between wall openings (previously modeled as reduced thickness) to capture the actual demands. This may be programmatic for other walls (only wall 30 examined in the course of this review). Appendix D - *In-Plane Moment Capacity of a Concrete Wall Subjected to Axial Tension.* Computations fail to take into account wall openings (which will reduce wall overturning resistance).	BNI/Axup: The revised SAP 2000 model has the actual openings for the wall at column line 30. Final confirmation of the concrete design of the wall at column line 30 will use the loads from the SAP 2000 model.	Closed. At issue wasn't so much the fact that the loads may be different. The question arose in calculation of the in plane walls overturning resistance. Calculations were based on a continuous wall, neglecting openings that are present.
CD-7	24590-PTF-DGC-S13T-00016, Design of Walls at Col Lines 28.5 and 30, and Col Lines B, E, H & L Bounded by Col Lines	Reviewer: Eric Walton There is an opening in section "Cut B24-26 El 56". This opening is not accounted for in the calculations.	BNI/Axup: The revised SAP 2000 model includes the opening. Calculation 24590-PTF-DGC-S13T-00016 will be revised to reflect the increase in RGM prior to	Closed

C.4 Concrete Design Review Comments

No.	Document	Comments and Observations	BNI Remarks/ Resolutions	Follow-up by USACE Review Team
	24 and 30 from el 56' to 77'.		placement of concrete and will take into account the effects of the opening.	
CD-8	24590-PTF-DGC-S13T-00040 Excel Spreadsheet Methodology and Example of Shear Wall Analysis, and excel template "Wall at column line 4, cut 4:B-E el 56'"	Reviewer: Eric Walton 1. Suggest adding a flow chart showing the logic of the spreadsheet, this would help in following and understanding the process taken. 2. Part V step 5 and Part VI step 4, f'_c is not linked to input in excel template, The designer should keep this in mind in the event the f'_c should changed. 3. Part IV step 2, $0.2*f'_cA_c$ is not used in excel, as was written up on methodology. This is not a factor unless f'_c becomes less than 4000 psi, the designer should keep this in mind in the event that f'_c should change. 4. Suggest adding reasoning for Vertical Section Cuts in the methodology, i.e. for shear friction between construction joints.	Observation BNI/Axup: 1. Concerned that having a flow chart will add only limited value to the user and will not be worth the effort to produce, or maintain for future deviations of the process. 2. Spreadsheet has been modified to link the concrete compressive strength, f'_c, to input. 3. Good point. Will keep this in mind should a value less than 4,000 psi be used for f'_c. 4. Will take into consideration.	Closed Item 3. May consider making a note in the methodology document.
CD-9	24590-PTF-DGC-S13T-00040 Excel Spreadsheet Methodology and Example of Shear Wall Analysis, and excel template "Wall at column line 4, cut	Reviewer: Eric Walton Sheet B-18, Vertical Section Cuts, Under Special Detailing Results, Item 12, references Part IX Step 4 of the calculation uses a tension steel zone of $0.1l_w$. Part IX Step 4 for Horizontal Section Cut uses $0.2l_w$.	BNI/Axup: Spreadsheet has been changed to use a distance of $0.2l_w$ for vertical section cuts.	Closed

C.4 Concrete Design Review Comments

No.	Document	Comments and Observations	BNI Remarks/ Resolutions	Follow-up by USACE Review Team
	4:B-E el 56'"	Sheet D-17, example for vertical section cut calc. uses $0.2l_w$. Is the Vertical Section Cuts intended for Part IX to use $0.1l_w$ or $0.2l_w$?		
CD-10	24590-PTF-DGC-S13T-00040 Excel Spreadsheet Methodology and Example of Shear Wall Analysis, and excel template "Wall at column line 4, cut 4:B-E el 56'"	Reviewer: Eric Walton Part 1-Section 1b. There is a substitution error when calculating the shear reinforcement capacity (ACI 349-01 11.10.9.1, eq 11-33). For equation (11-33) d is defined as 0.8^*l_w from ACI 349-01 11.10.9.1 and in section 1b of the methodology. When $A_v=p_n{}^*s^*t_w$ is substituted into equation (11-33), d is substituted with l_w. As previously defined d should be substituted with $0.8^*l_{w.}$ This will have a 20% decrease in steel capacity. This caries over to the equations used for calculating nominal shear capacity based on equations 11-31 and 11-32.	BNI/Axup: Spreadsheet has been changed to use $0.8l_w$ for d.	Closed
CD-11	24590-PTF-DGC-S13T-00040 Excel Spreadsheet Methodology and Example of Shear Wall Analysis, and excel template "Wall at column line 4, cut 4:B-E el 56'"	Reviewer: Eric Walton Part XI - Final Review. The Seismic D/C ratio may not be an accurate indicator. The max D/C load case when subtracting the max non-seismic D/C load case does not necessarily use corresponding load cases.	BNI/Axup: BNI believes the seismic D/C, i.e., that portion of the total D/C from the controlling load combination, is accurate and is clarified as follows: The non-seismic load combinations (LCs) are LC 1 through LC 45 as shown on page A-4. Pages A-5 through A-10 have the seismic LCs. The load factor for the seismic LCs, for dead, live, snow,	Closed

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			etc., is always 1.0 with the exception of when the dead is 0.9. By inspection of the non-seismic LCs, it is clear that one of them will always control over the seismic LCs if the seismic load was eliminated or set to zero (0) for the seismic LCs. Therefore, a way to calculate the seismic D/C is to first calculate the controlling D/C using all LCs, and then subtract the D/C value for only the non-seismic LCs (1 thru 45).	
CD-12	24590-PTF-DGC-S13T-00022, Rev No B, Design of Slab at 56 ft Elevation	Reviewer: Mark Summers Based on the review of this document it appears that drag struts and collectors have not been designed or checked, the calculations are only based on section cuts. For the slab at elevation 56, over the hot cell numerous transverse shear walls butt up against the slab. Section cuts typically start and end at these slab/wall intersections. From the excel spread sheet analysis these section cut results typically required additional steel due to high in-plane moments. The high moments in these partial section cuts is due to the non-uniform normal stress across the section cut due to the stress concentrations at the slab/wall intersections. An analysis is needed to validate drag struts and collectors	BNI/Axup: The slab at El 56' is integral with the entire length of the wall, not just attached at the ends where a drag strut analysis would be required.	Disagree. There are several walls that butt up to the slab over the hot cell or butt up with only one edge continuous. To properly transfer the tension or compression stress concentrations (indicated from the SAP2000 analysis) at the wall ends a drag strut/collector analysis is required. <u>This analysis should be performed or checked at all wall to slab stress concentrations within the PTF</u> similar to that performed for the HLW. Otherwise it is assumed that cracking occurs redistributing the tension/compression concentrations into a shear force which is higher than the SAP2000 analysis results that are used for design.

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		are not required.		
CD-13	24590-PTF-DGC-S13T-00022, Rev No B, Design of Slab at 56 ft Elevation	Reviewer: Mark Summers Sheet No. H-7 Part IV, paragraph 1 and 2. Unaware of a requirement to account for transverse shear demand in determining in-plane shear strength. Potential conservatism.	Observation BNI/Axup: Spreadsheet to be revised to take out conservatism.	Closed
CD-14	24590-PTF-DGC-S13T-00012, Rev No A, Design of Walls at Col Lines 25.5, 27, 28.5, 30, B, D, E, H, J, K, L & M from El 28 to 56 and Dowels up to Floor at Elevation 28'	Reviewer: Mark Summers Appendix H, Wall Qualification for RGM Seismic Increase. The load combinations in this appendix do not include seismic combined with thermal. ECCN 24590-PTF-DGE-S13T-00019 cites 24590-WTP-DTD-ENG-05-001 for this deviation. There is no rational as for the deviation, only the statement "For these evaluations the bulk thermal effects need not be combined with seismic." Why the deviation from the Structural Design Criteria load combinations?	BNI/Axup: ECCN was based on the 24590-WTP-ENG-05-001, which justified not combining bulk thermal loads with seismic loads. This is listed as an un-verified assumption of the ECCN, and will be addressed prior to advancement of the calculation to "Confirmed" status in accordance with the latest SDC.	Closed
CD-15	24590-PTF-DGC-S13T-00022, Rev No B, Design of Slab at 56 ft Elevation 24590-PTF-DGC-S13T-00012, Rev No A, Design of Walls at Col Lines 25.5, 27,	Reviewer: Mark Summers Reference 1, Sheet No. H-10, Part VI, paragraph 3. The logic presented for required tension steel does not result in stress/strain compatibility across the section. An appropriate distribution of steel to resist direct tension is half the	BNI/Axup: Will need clarification of comment since it is not clear to which calculation the comment pertains. Neither calculation has a Part VI, paragraph 3 that discusses distribution of total applied tension forces to reinforcement curtains on opposite faces of wall.	References 1 & 2 are in error, should be ECCNs 24590-PTF-DGE-S13T-00024 and 00019, respectively (changes to 24590-PTF-DGC-S13T-00022 and 00012)

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	28.5, 30, B, D, E, H, J, K, L & M from El 28 to 56 and Dowels up to Floor at Elevation 28' 24590-PTF-DGC-S13T-00040, Rev 0, Excel Spreadsheet Methodology and Example for Shear Wall Analysis	required tension steel area should be directly added to the required flexural steel requirement and assume the other half is covered by steel in the opposite face. Reference 2, Sheet No. H-10, paragraph 3. The computation to determine the vertical rebar requirement for net tension is primarily met by using the steel available on the compression side of the section cut. This approach does not account for an additional end moment accounting for the translation of the net tension force from the section centroid to the steel within the compression zone. The appropriate distribution of steel to resist direct tension is half the required tension steel area should be directly added to the required flexural steel requirement and the other half in the opposite face. Reference 3, Sheet No. B-13, paragraph 3. and 4. The computation to determine total required steel in each face is in error. The appropriate distribution for area of steel in one face is 1/2 the tension steel + 1/2 the in-plane bending moment steel + transverse bending moment steel.	However, there is a section VIII, in calculation 00040, that distributes 1/2 of the direct axial tension to the tension in reinforcement caused by bending. Spreadsheet methodology revised.	
CD-16	24590-PTF-DGC-	Reviewer: Mark Summers	BNI/Gurbuz:	Closed

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	S13T-00022, Rev No B, Design of Slab at 56 ft Elevation 24590-PTF-DGC-S13T-00012, Rev No A, Design of Walls at Col Lines 25.5, 27, 28.5, 30, B, D, E, H, J, K, L & M from El 28 to 56 and Dowels up to Floor at Elevation 28' 24590-PTF-DGC-S13T-00040, Rev 0, Excel Spreadsheet Methodology and Example for Shear Wall Analysis	Reference 1 Sheet No. H-9, Reference 2 Sheet No. H-9, and Reference 3 Sheet No. B-11. "Check to determine if the minimum steel ratio, ρ_{min}, required at the ends of the wall is greater than that allowed in accordance with Section 21.6.6.5. If $\rho_{min} > 400/f_y$, provide additional ties." The intent of Section 21.6.6.5 is the <u>actual</u> longitudinal steel ratio provided is to be compared with $400/f_y$, not a calculated ρ_{min}. The actual steel provided may buckle due to cyclic load reversals.	The intent of Section 21.6.6.5 is to preclude buckling of the reinforcement due to cyclic loads as stated in the Commentary. Buckling of the reinforcement has been observed as the section nears ultimate capacity, that is when the maximum concrete strain exceeds 0.003 (0.004 per Ref. 2), regardless of the percentage of axial reinforcement. Recognizing the importance of the concrete strain, the WTP project developed and implemented a strain criterion to limit the maximum concrete strain to 0.002 (Ref. 1). The document calls for addition of a boundary element with appropriate transverse reinforcement if the analysis shows that the concrete strain exceeds 0.002 under the design earthquake loads. In such cases, the provisions of Section 21.6.6.5 will be followed. For the cases where maximum concrete strain in compression is less than 0.002, transverse ties are not required since buckling of the longitudinal reinforcement is unlikely. Using the strain limit of 0.002 rather than 0.003 is intended to provide additional margin. Incidentally, the next revision of ACI 349 is expected to relate the requirement for transverse reinforcement to the exceedance of maximum concrete strain	

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			of 0.002. References: 1. WTP report, CCN 071327, Technical Approach for Boundary Elements in Special Reinforced Concrete Structural Walls, 24590-HLW-RPT-CSA-03-013, November 20, 2003. 2. John W. Wallace and Kutay Orakcal, ACI 318-99 Provisions for Seismic Design of Structural Walls, ACI Structural Journal/July-August 2002.	
CD-17	24590-PTF-DGC-S13T-00022, Rev No B, Design of Slab at 56 ft Elevation 24590-PTF-DGC-S13T-00012, Rev No A, Design of Walls at Col Lines 25.5, 27, 28.5, 30, B, D, E, H, J, K, L & M from El 28 to 56 and Dowels up to Floor at Elevation 28' 24590-PTF-DGC-S13T-00040, Rev 0, Excel Spreadsheet Methodology and Example for Shear Wall Analysis	Reviewer: Mark Summers Reference 1, Sheet No. H-11, Part VII, paragraph 2. The equation presented in this paragraph assumes tied steel. Logic is not provided to flag that ties are required based on use of this equation. Generally the steel is not tied, and axial strength of the element is less than that calculated. Reference 2, Sheet No. H-8 and H-9. The use of the equation from Section 10.3.5.2 for axial compressive strength assumes longitudinal steel in the wall ends/boundary elements are tied. Generally the steel is not tied, and axial strength of the element is less than that calculated.	BNI/Axup: Chapter 14, Section 14.3.6, states that ties in walls are not required where the vertical wall reinforcement is not required to resist compression forces. The check provided in the calculation always shows that compression reinforcement, in accordance with Section 10.3.5.2, is not required.	Closed

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		Reference 3, Sheet No. B-9 and B-10. The use of the equation 10-2 for axial compressive strength assumes longitudinal steel in the wall ends/boundary elements are tied. Generally the steel is not tied, and axial strength of the element is less than that calculated.		
CD-18	24590-PTF-DDC-S13T-00001 - Vessel Ring Embedded Plates	Reviewer: Stephen A. Short a) Seismic loads on the anchors and ring beams are evaluated in a conservative manner. For full tanks, the entire weight of contents is used as the impulsive weight. For partially full tanks, ASCE 4 equations are used to determine impulsive and convective (sloshing) weights and their height of application. The impulsive seismic load is determined from the peak of the 2% damped ISRS. The convective seismic load is determined from the 0.5% damped ISRS at the sloshing frequency computed using ASCE 4 equations. This is a very conservative approach as the impulsive frequency could be determined along with the corresponding spectral acceleration. Also, the use of 4% damping could	Observation BNI/Laughlin: BNI concurs with the observation.	Closed

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		probably be defended. b) Vessel seismic design is by outside vendors. The load path from the vessel skirt or saddle to the supporting concrete is continuous and has been evaluated in a complete manner in this calculation. c) For the vertical cylindrical vessels, the seismic overturning moment and shear is determined by 100-40-40 direction component rule and vector sum of perpendicular components. This approach is appropriate for shear but overly conservative for tensile forces due to moment. At locations where the tension due to moment is maximum in one direction of earthquake shaking, it is zero for the perpendicular direction of earthquake shaking. The 100-40-40 and vector sum combination of moments results in tensile forces that are 8% greater than necessary. d) For the light to moderate loaded vessels, welded head studs anchors embedded 10 inches have been designed. The		

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		design is performed by the CCD method in Appendix B of ACI 349-01. For these anchors, the concrete breakout strength in tension is less than the tensile capacity of the steel stud. As a result, the anchor is non-ductile and the 60% penalty factor is imposed. It is interesting that a non-ductile design would not be permitted for PC-2 structures following Appendix D of ACI 318. It appears that the demand to capacity ratios for these designs is very low for all vessels. e) For heavily loaded vessels, anchors are 3/4 inch Nelson deformed bars embedded 4 feet or #9 reinforcing bars embedded 6 feet. In some cases, steel shear keys or lugs are used to enhance the shear capacity of the anchorage. The design is by shear friction to obtain shear capacity and by achieving full development of the anchor bar to obtain tensile capacity. As a result, a ductile design is achieved by this approach. The only question on the use of the Nelson deformed bars is whether the design should be based on nominal area or stress		

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		area. The nominal area of 0.44 square inches is used in the calculations for the 3/4 inch diameter anchors. The stress area for 3/4 inch diameter Nelson deformed bar anchor studs is 0.41 square inches as specified in ICC Evaluation Report, ER-5217. f) The evaluation of seismic demand on vessel anchors is performed in a very conservative manner. This conservatism could be reduced in a defensible manner if needed.		

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SD-1	Seismic Analysis of Structural Steel	Reviewer: Phillip Hashimoto The seismic analysis approach and results for structural steel at the top of the Pre-Treatment (PT) Building was presented by Bechtel on June 6, 2006. The supporting calculation was not reviewed by the USACE team. Seismic forces for design of the structural steel are obtained by equivalent static analysis using the SAP2000 model of the PT Building. Input to the SAP2000 model was derived from seismic responses calculated by the SASSI dynamic analysis. Seismic forces on structural steel components due to frame action are taken as the envelope of values from the SAP2000 and SASSI analyses. Local forces due to vibration of individual components in between their supports are calculated by supplemental analyses. Seismic loads for the SAP2000 analysis were based upon acceleration profiles in plan and elevation calculated by the SASSI analysis. The seismic loads included torsional effects. Seismic loads due to the three orthogonal earthquake components were combined by the 100-40-40 method. The results of a study to investigate potential out-of-phase motion of the structural steel were presented. Relative displacement time histories in the east-west direction at selected locations at the roof were plotted. The responses at these locations were observed to be in-phase with each other, with the roof responding in a single dominant mode. Ratios of forces calculated by the SAP2000 analysis to forces calculated by the SASSI analysis were determined, tabulated, and examined. The former typically exceeded the	Observations

[1] “Observation” is for information only - a response is not required.

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		latter. Instances in which the SASSI forces exceed the SAP2000 forces are addressed in the design process. Design of a group of similar structural members for bounding forces was shown by example to cover some of the exceedances. Members are designed for the larger of the SASSI and SAP2000 forces. Although the analysis results were not reviewed in detail, they appear to be reasonable. The SAP2000 forces typically exceed the SASSI forces probably due to conservatisms introduced into the equivalent static loads, such as the use of bounding accelerations and adjustment for torsional effects that are not implicit in the SASSI analysis results. It is not readily apparent that this example demonstrates that the equivalent static analysis approach is sufficient to provide conservative forces for structures whose seismic responses are influenced by multiple modes. Information presented seems to indicate that the PT Building structural steel responds primarily to a limited number of dominant modes.	
SD-2	24590-HLW-SSC-S15T-00137, Design of Multi-Commodity Support Beams Above El. 37'-0"	Reviewer: Phillip Hashimoto a) Seismic loads for design of the support beams are typically determined on a tributary basis. For example, the seismic load on a beam is calculated as the product of the peak 4% damped spectral acceleration and the beam tributary mass determined on an area basis. However, the reactions from the supported commodities onto the beams can depend on the manner in which the commodities are attached to the beams and the relative stiffnesses between the commodities and the support beams. The beam seismic loads used for design might be appropriate if the beams are relatively stiff in comparison to the piping and other systems supported. However, preliminary calculations indicate that the	BNI

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		beams can be relatively flexible in weak axis bending. Because the piping is relatively stiff in the longitudinal direction, its reactions on the supports could be different from those considered in the beam design. In addition, the weak axis seismic loads on the beam assume that the piping is rigidly attached to the support beams. It is not known if this is actually the case. If the piping is not rigidly attached to the support beams in the longitudinal direction of the piping, the distribution of piping seismic reactions could be significantly different from those used in the beam design. b) Pipe friction loads are included in the support beam design for static loads. However, the pipe friction loads are not combined with seismic loads. The treatment of the pipe friction loads in the static and seismic load combinations appears to be contradictory. If the piping is capable of transmitting seismic reactions to the beams in the piping longitudinal direction, it should also be capable of transmitting friction loads in this direction. c) Seismic loads are based upon the peak 4% damped spectral accelerations of the applicable in-structure response spectra. These loads should be confirmed to be conservative when they are reconciled with the design/analysis of the commodities that they support (see Comment a) above). d) Weak axis bending stresses due to pipe friction loads are based upon a beam cross-section consisting of only the upper half of the beam. This approach appears to be an approximation that accounts for the eccentricity between the point of load application (at the top flange) and the beam neutral axis. However, weak axis bending stresses due to seismic loads are based upon the entire beam cross-section. If the seismic loads are due to longitudinal piping reactions, a similar eccentricity exists. Stresses on the beam due to weak axis bending and torsional moment resulting from the eccentric loading	

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		should be determined in a more rigorous manner. e) The 100-40-40 method for combining seismic loads in the three orthogonal directions has been applied incorrectly. 100% of the seismic load due to one earthquake component is to be combined with 40% of the seismic loads due to the other two earthquake components. Non-seismic loads should not be reduced. When determining stresses concurrent with 100% of the weak axis bending stress, 40% of the strong axis bending stresses due to seismic load <u>and</u> dead load were used. Reduction of the dead load stress is inappropriate. f) Unbraced lengths of some of the support beams are reduced to account for lateral restraint of the compression flange by columns and connecting beams. The connection details were not available for review. They should be confirmed to provide appropriate flange restraint. g) On Sheet 51, the maximum allowable strong axis bending stress should be limited to the 0.9 Fy value of 45 ksi, rather than the 48 ksi value used. h) On Sheet 76, the allowable column axial stress is based upon an effective length factor of 1.0. This value assumes that the column is laterally braced at the top. The presence of this bracing should be verified, or the effective column length should be revised accordingly. i) The calculation of the brace axial force on Sheet 112 contains two discrepancies, one conservative and one unconservative. The brace force is based upon a tributary area of 10.5 feet by 6.67 feet, or 70 square feet. The total tributary area for the four braces provided is actually 3.33 feet by 52.33 feet, which corresponds to 175 square feet total or 43.6 square feet per brace. The brace force is taken to be [(10.5/2)/10.85] times the horizontal seismic load. The correct factor is the inverse of this value. Considering these two errors, the brace	

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		force is about 2.7 times the value calculated.	
SD-3	24590-HLW-SSC-S15T-00078 Multi-Commodity Support Design for Elev. 23'-0"	Reviewer: Eric Walton The 100-40-40 method for combination of orthogonal earthquake components has not been implemented. There is no weak axis load applied to the beams from its self-weight due to seismic loading. Effects of torsion from pipe hangers were not considered on the beams from a seismic event.	BNI
SD-4	24590-PTF-SSC-S15T-00024 - Miscellaneous Platforms Between El. 28'-0" and El. 56'-0" 24590-PTF-SSC-S15T-00036 - Design of Steel Framing for Leak Detection Shed over South Tunnel and Loading Dock Roof for PT Building.	Reviewer: Bill Bolte May elect to reanalyze the W36x280 incorporated in Platform PP0212A without accounting for F_{μ} or the need arises that the member is properly recorded, per SADC - 8.3.3.2, "Elements where F_{μ} are used for design will be tracked."	BNI
SD-5	24590-PTF-SSC-S15T-00006 Rev. B, *Design E. 77' Steel Framing for PT Building.*	Reviewer: Paul Baughman Rev. B uses RGM. Summary of seismic design approach (Rev. B): • Uses equivalent static analysis, single mode dominant response. • Vertical seismic only (concrete slab takes horizontal seismic). • Uses peak of response spectrum, 4% damping (vs. 7% allowed), 1.4g (vs. 1.0g for 7% damping). • Spectra are Figs. 8E and 9E from 24590-PTF-S0C-S15T-00005 Rev 0B, *Pretreatment Building Seismic*	Observations

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		Analysis – In-Structure Response Spectra (ISRS). • Includes full weight of concrete slab in dead load and seismic (per criteria could use 10%). • Includes loads superimposed on concrete slab (e.g., equipment loads). • Piping and equipment loads appear to be conservative estimates. • For seismic, uses 1.6S and 1.4S for bending and shear, respectively. • Did not appear to use F_{μ} factor. • Max D/C for seismic is 0.96. • Although maximum D/C ratio is 0.96, there is significant conservatism. There is significant margin available.	
SD-6	24590-PTF-SSC-S15T-00028 Rev. C, *Design of Runway for PIH Overhead Mast Power Manipulator with Auxiliary SLEW Hoist.*	Reviewer: Paul Baughman Rev B used the OGM then Rev C, added an attachment that reconciled the design to the RGM. Summary of the seismic design approach (Rev. B): • Used equivalent static analysis, single mode dominant response. • Crane weight increased 20%. • Used peak accelerations of response spectra, 3% damping, for crane forces. • Used spectra Elevations 98' and 77' from 24590-PTF-S0C-S15T-00005 Rev 0A, *Pretreatment Building Seismic Analysis – In-Structure Response Spectra (ISRS)*. • Required supporting structure to be rigid (f > 33 Hz). Not sure why this is required since peak of response spectrum is being used. • Used GTSTRUDL for frequency calculation of runway and supports. Did not appear to include	Observations

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		vertical offset of wheel load (torsion effect) from centerline of runway. Should not have significant effect, but runway frequency is only a small amount over the "rigid" criterion of 33 Hz (33.7 Hz for lateral frequency). • Applied lateral load to one rail only. • Used F_μ = 1.5. • Combined directions absolutely rather than using 100-40-40 rule. • For seismic, used 1.6S and 1.4S for bending and shear, respectively. • Constrained code IR < 0.90. Max IR = 0.86. • Used hand static calculations for loads on runway and support brackets. • Used ME035 for analysis of wall embed plate. Rev. C added Attachment K - evaluates effect of revised seismic loads and crane data (weight not increased 20%). • Calculates ratio of new load to load used in Rev. B. Where new load is higher than design load, effect is evaluated. Only one load is higher (lateral seismic acceleration for ECCN 00053 reconciliation is 3.345g vs. 3.12g). Design meets acceptance criteria. • Uses crane frequencies reported by manufacturer. • Neglects support structure frequency because Rev. B showed support structure is in rigid range (f > 33 Hz). • Uses spectral accelerations at crane frequencies from Figs. C-1, C-2 and C-3 of 24590-PTF-S0c-S15T-00045 Rev. B, *PTF Seismic Analysis – Enveloped In-Structure Response Spectra*. These spectra are from 24590-PTF-S0C-S15T-00005 Rev 0B, *Pretreatment Building Seismic Analysis – In-Structure Response Spectra (ISRS)*. They are RGM	

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		spectra. • Longitudinal (E-W) and vertical frequencies are below peak. 24590-WTP-DC-ST-04-001, Rev. 3, *Seismic Analysis and Design Criteria*, Section 7.2.2.3(a) Equivalent Static Analysis, Single Mode Dominant Response, requires that if the frequency is below the peak of the response spectrum, the peak spectral acceleration must be used. ♦ The longitudinal peak is about 1.1g vs. 0.96g used in the reconciliation, and the vertical peak is about 1.005g vs. 1.005g used in the reconciliation (sheet K-2). ♦ Longitudinal load changes from 15.02 kip to 17.21 kip, still less than design load of 24.17 kip (sheet K-5). ♦ For ECCN reconciliation (sheet K-6) revised longitudinal acceleration 1.1g vs. 0.96g, still less than ECCN acceleration of 1.32g. ECCN evaluation uses longitudinal load but it is limited to friction force. Only changes are in $P_{L(long\text{-}seis)}$ and $P_{(total)}$ on sheet K-11. No change to final loads and conclusion.	
SD-7	24590-PTF-SSC-S15T-00028 Rev. C, *Design of Runway for PIH Overhead Mast Power Manipulator with Auxiliary SLEW Hoist.*	Reviewer: Paul Baughman a) Runway frequency calculation should include vertical offset of wheel load. This could affect Attachment K lateral acceleration (based on support structure frequency > 33 Hz). b) Should re-examine requirement for support structure to be rigid when using single mode dominant response. Since frequency of supported component is not known (worst case assumed), combining with a non-rigid support frequency does not change the approach. c) Reconciliation in Appendix K should use peak spectral	BNI

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		acceleration in longitudinal and vertical directions. Numbers change but conclusion is same. d) Recommend revisiting other crane runway calculations for the above issues.	
SD-8	24590-PTF-SSC-S15T-00032 Rev. B, *Design of Runway for PFH Overhead Mast Power Manipulator with Auxiliary Hoist.*	Reviewer: Paul Baughman Rev A used the OGM then Rev B, added an attachment that reconciled the design to the RGM Summary of seismic design approach (Rev. A): ♦ Used equivalent static analysis, single mode dominant response. ♦ Used peak accelerations of response spectra, 4% damping, for crane forces. ♦ Rev A. uses spectra Elevations 98' and 77' from 24590-PTF-S0C-S15T-00005 Rev 0A, *Pretreatment Building Seismic Analysis – In-Structure Response Spectra (ISRS)*. ♦ Required supporting structure to be rigid (f > 33 Hz). Not sure why this is required since peak of response spectrum is being used. ♦ Used GTSTRUDL for frequency calculation of runway and supports. Did not appear to include vertical offset of wheel load (torsion effect) from centerline of runway. Should not have significant effect, but runway frequency is only a small amount over the "rigid" criterion of 33 Hz (35 Hz lateral). ♦ Applied lateral load to one rail only. ♦ Used F_μ = 1.5. ♦ For seismic, uses 1.6S and 1.4S for bending and shear, respectively. ♦ Used hand static calculations for loads on runway and support brackets. ♦ Used ME035 for analysis of wall embed plate.	Observations

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		♦ Constrained code IR < 0.95. Max IR = 0.94. Rev. B – added Attachment N – evaluates effect of revised seismic loads and crane data. ♦ Calculates ratio of new load to load used in Rev. A. Where ratio is greater than 1.0, effect is evaluated. Some ratios are more than 1.0 but are not high (max 1.155). Applied R to D/C ratio from Rev. A and checked if less than 1.0. All are below 1.0. Max is 0.86. ♦ Uses crane frequencies reported by manufacturer. ♦ Uses spectral accelerations are crane frequencies from Figs. C-4, C-5 and C-6 of 24590-PTF-S0c-S15T-00045 Rev. B, *PTF Seismic Analysis – Enveloped In-Structure Response Spectra*. These spectra are from 24590-PTF-S0C-S15T-00005 Rev 0B, *Pretreatment Building Seismic Analysis – In-Structure Response Spectra (ISRS)*. They are RGM spectra. ♦ Longitudinal (E-W) and vertical frequencies are below peak. 24590-WTP-DC-ST-04-001, Rev. 3, *Seismic Analysis and Design Criteria*, Section 7.2.2.3(a) Equivalent Static Analysis, Single Mode Dominant Response, requires that if the frequency is below the peak of the response spectrum, the peak spectral acceleration must be used. ♦ The longitudinal peak is about 1.7g vs. 1.354g used in the reconciliation, and the vertical peak is about 1.35g vs. 0.998g used in the reconciliation. ♦ Controlling seismic D/C ratio is 0.68 on runway girder (sheet N-7). R_{max} is controlled by longitudinal force. Using peak value gives R_{max} of 0.86 vs. 0.68 in calculation. Structure still meets criterion of D/C < 1.0.	

C.5 Structural Steel Design Review Comments			
No.	**Document**	**Comments and Observations**	**Action By and Response**[1]
SD-9	24590-PTF-SSC-S15T-00032 Rev. B, *Design of Runway for PFH Overhead Mast Power Manipulator with Auxiliary Hoist.*	Reviewer: Paul Baughman a) Runway frequency calculation should include vertical offset of wheel load. This could affect Attachment N lateral acceleration (based on support structure frequency > 33 Hz). b) Should re-examine requirement for support structure to be rigid when using single mode dominant response. Since frequency of supported component is not known (worst case assumed), combining with a non-rigid support frequency does not change the approach. c) Reconciliation in Appendix N should use peak spectral acceleration in longitudinal and vertical directions. Numbers change but conclusion is same.	BNI
SD-10	24590-PTF-SSC-S15T-00035 Rev. C, *Design of Corridor Steel Framing at El 56' for PT Building.*	Reviewer: Paul Baughman Rev. C uses RGM. Summary of seismic design approach (Rev. C): • Uses equivalent static analysis, single mode dominant response. • Vertical seismic only (concrete slab takes horizontal seismic). • Uses peak of response spectrum, 4% damping (vs. 7% allowed), 1.4g (vs. 1.0g for 7% damping). • Spectra are Figs. 13E and 14E from 24590-PTF-S0C-S15T-00005 Rev 0B, *Pretreatment Building Seismic Analysis – In-Structure Response Spectra (ISRS).* Sheet 3 gives seismic coefficients. The coefficient for horizontal is 1.5g. But looking at Fig 13E, the coefficient should be 1.6g. The coefficient is not used in the calculation. • Includes full weight of concrete slab in dead load and seismic (per criteria could use 10%). Adds from 25%	Observations

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No.	Document	Comments and Observations	Action By and Response[1]
		to 200% depending on area. • Includes loads superimposed on concrete slab (e.g., equipment loads). • Piping and equipment loads appear to be conservative estimates. • For seismic, uses 1.6S and 1.4S for bending and shear, respectively. • Did not appear to use F_{μ} factor. • Max D/C for seismic is 0.96.	

C.6 Vessels and Nozzles Review Comments				
No.	**Document**	**Comments and Observations**	**BNI Remarks/ Resolutions**	**Follow-up by USACE Review Team**
VL-1	Vessels: PWD-VSL-00015 and 00016 24590-QL-POB-MVA0-0001-06-00027 drawing 24590-QL-POB-MVA0-0001-03-00005 and 00006 - 264" I.D. Acidic/Alkaline Effluent Vessel calculation	Reviewer: Jim Wilcoski a. The fluid is included in the model as solid elements so that the effects of impulsive and sloshing should be accounted for directly. Three analysis cases are defined, but it is unclear if the effects of gravity are included in any or all of these. Therefore, it is unclear if gravity loads are included in the stress results. b. On page 2 of the calculation reports, it appears they conservatively used 0.5% damping based response spectra for all modes, not just sloshing modes. This is inaccurate and overly conservative. They could have rather defined the response spectra that would equal the 0.5% values for frequencies at or below the highest sloshing mode, and then use 2% damping based response spectra for frequencies above that level. c. Page 45 lists the calculated frequencies, with the two lowest having values of 1.39 Hz.	BNI/Eaton: a. Inputs for gravity loads were not included in the vendor analysis; this will be included in the vendor analysis revision. * b. Agree - The use of 0.5% damping values for both sloshing and impulsive modes is a conservative approach and was acceptable at the time of this analysis. Since larger loads (RGM) are being applied we are requiring the vendors to use 2% damping for the impulsive modes to reduce the loading on the building structure. All new RGM reports will utilize the 2% damping curves above the sloshing modes. c. Agree with the comment. The vendor calculation reviewed was to the baseline seismic curves. The vendor will be notified of the comment and will be incorporated into the analysis revision.* d. Agree with comment. Vendor will be notified and the comment will be included in the next submittal for the	Closed a. BNI plans are acceptable, but the results should be reviewed. b. BNI plans are acceptable, but the results should be reviewed. c. BNI plans are acceptable, but the results should be reviewed. d. BNI plans are acceptable, but the results should be reviewed.

C.6 Vessels and Nozzles Review Comments

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		However, when ASCE 4-98, section C3.5.4.3.2 is used to calculate the sloshing modes, values of 0.369 Hz are found. These values based on ASCE 4-98 are well below the smallest values listed on page 45, calling into question the accuracy of the modeling of the fluid that was used to calculate the sloshing modes. If the sloshing modes were not calculated correctly, the impulsive loading and gravity load of the fluid may also have not been properly represented in the model. d. Page 3, 2nd paragraph, indicates that the number of modes included in the analysis were those which fell within the parameters of Figures 61, 62 and 63 (i.e., within the frequency range of the in-structure response spectra). Pages 45 through 49 lists the 1st 200 modes (up to 48.6 Hz), and pages 50 through 61 lists their participation factors, plus the modal mass and cumulative mass for modes with large participation factors. However, the cumulative mass is never compared with the total mass of the vessel. The number of	revised analysis. e. Agree with comment. Specification requires absolute summation, (24590-WTP-3PS-MV00-T0001, 6.1.3). Comment will be included in RGM analysis. f. Agree with comment. Vendor will be notified and the comment will be included in the next submittal for the revised analysis. * g. Agree. Discuss with NWC on the basis for combining before final response to this comment. This appears to be a conservative approach	e. BNI plans are acceptable, but the results should be reviewed. f. BNI plans are acceptable, but the results should be reviewed. g. BNI plans are acceptable, but the results should be reviewed.

C.6 Vessels and Nozzles Review Comments				
No.	**Document**	**Comments and Observations**	**BNI Remarks/ Resolutions**	**Follow-up by USACE Review Team**
		modes used in the analysis should generally be the number necessary to reach 90% of the total mass of the structure. This may not be realistic, but at least the comparison should be made between the participating mass and total mass of the vessel. e. Page 3, 2nd paragraph, indicates all closely spaced modes are calculated using the SRSS method. However, design requirements indicate that the contributions of modes within 10% of each other should be summed. f. Pages 27 through 44 plots the stresses from case 3 for each of the six stress components. The bottom of page 3 then shows how the peak values of these are combined using the square root of the sum of the squares of these peak values. These plots show that the peak of all stresses occur on either one of the pulse jet mixer (PJM) inlet nozzle junction or an 18 in. charge vessel inlet nozzle junction. Page 3 indicates the stress intensity, SI is computed by hand for these two high stress areas, using a square	for combining all peak stresses, which is not incorrect. Removal of conservatism may be of benefit though when applying RGM and increased overblow loads.* *BNI estimates to have response, including vendor input by 06.29.06.	

C.6 Vessels and Nozzles Review Comments				
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		root of the sum of the squares, SRSS. This is not the proper way to combine these stresses. The 1998 Section VIII, Division 2, Appendix 4, Mandatory Design Based on Stress Analysis, Article 4-1, paragraph 4-120, Derivation of Stress Intensities provides the proper method of calculating stress intensity. When principal stresses acting in opposite direction are combined as shown in this paragraph, the stress intensity will be greater than those based on SRSS. g. The bottom of page 3 shows that the peak stress values for either the PJM inlet nozzle junction or 18" CV Inlet Nozzle junction are combined, but it appears that these peak values from different locations near these nozzles may be combined.		
VL-2	Equip Tag #: 24590-PTF-MV-CXP-VSL-00005 PO#: 24590-QL-POA-MVA0-00014 Seismic Analysis of 60" ID C.S. Reagent	Reviewer: Jim Wilcoski a. Typographical error. Page 39 lists the frequencies and participating mass of the primary modes and page 43 identifies two modes. The note at the bottom of page 43 should read "the next significant mode is at	BNI/Eaton: a. Typographical error exists and confirmed with the vendor. Analysis will be corrected and resubmitted	Closed a. BNI plans are acceptable, but the results should be reviewed.

C.6 Vessels and Nozzles Review Comments				
No.	**Document**	**Comments and Observations**	**BNI Remarks/ Resolutions**	**Follow-up by USACE Review Team**
	Storage Vessel	30.1 Hz (Mode 7). Mode 8 is a companion mode of Mode 6.” rather than “the next significant mode is at 31.0 Hz (Mode 8). Mode 7 is a companion mode of Mode 6.” This should have no impact on the results, as the contributions should have been automatically incorporated by the software. b. Pages 49 and 50 present the calculations of the Horizontal Impulsive Mode and Horizontal Sloshing Mode effective fluid weight, W, height from the bottom of the cylindrical shell to the centroid of the fluid weight, X, and the frequency of the sloshing mode, , in accordance with the guidance of ASCE 4-98, sections C3.5.4.2 and C3.5.4.3, respectively. The correct equation is used for calculating the Impulsive mode effective fluid weight, W_1, (Eq C3.5-3) for D/H < 1.333, but the Equation C3.5-2 rather than C3.5-4 was incorrectly used for calculating the centroid of the fluid, X1. They calculated 27 in. for this term, when it should have been 30.3 in., based on Eq. C3.5-4. This error appears to be un-conservative, because	b. Agree with the comment and confirmed with the vendor. However since the vessel is supported above the centroid of the fluid and the centroid is actually closer to the support elevation, the overturning reaction is less, and therefore the current version is more conservative. The comment will be incorporated and the analysis resubmitted. c. This comment must be discussed with the vendor prior to final response*. *BNI estimates to have response, including vendor input by 07.20.06.	b. BNI plans are acceptable, but the results should be reviewed. Agree that the current analysis is more conservative because of the support from above the centroid. c. BNI plans are acceptable, but the results should be reviewed.

C.6 Vessels and Nozzles Review Comments				
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		the centroid this load should have been applied at a higher elevation, loading the vessel more heavily. The impact of this error should be minor. c. ASCE 4-98, Section 3.5.4.5.3 Freeboard Requirements, indicates that if the top of the tank is not above the slosh height, it shall be analyzed for the contact pressures and impact force that result from fluid sloshing against it. Section 3.5.3.5.4 Special Provisions for Full Tanks, states that if the distance from the top head to the water surface is less than 50% of the slosh height above the top of the fluid, the tank shall be treated as being full. Section C3.5.4.3.5 Fluid Slosh Height - Fundamental Sloshing Mode, Equation C3.5-12 can be used to estimate this sloshing height, d, which gives a value of 18 in., based on a spectral acceleration, S_{a2} of 0.6 g at sloshing frequency, f_2 of 0.773 Hz, from Figures 31 and 32, presented on pages 61 and 62. The total fluid height, H is 72 in. and the vessel height where it meets the shell wall is 78 in., so that the top head to the water		

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		surface is only 6 in. This top head is less than 50% of the slosh height of 18 in. Therefore, the freeboard requirements are not satisfied, and the contact pressures and impact forces must be analyzed according to Sections 3.5.4.5.3 and 3.5.3.5.4. This was not done in the reviewed report and these effects were not included in the analysis. However, this effect should be minor and the design margin, should more than accommodate their effects.		
VL-3	24590-QL-POA-MVA0-00010-03-01 Rev. 00E, Design Calculations, UFP-VSL-0002A/B. 24590-QL-POA-MVA0-00010-03-00005 Rev. 00E, Nozzle Load Calculations, UFP-VSL-0002A/B. 24590-QL-POA-MVA0-00010-09-00001 Rev. 00A, Seismic Analysis, UFP-VSL-0002A/B. 24590-QL-POA-	Reviewer: P.D. Baughman a. The analysis does not account for seismic fluid pressure load on the internal PJM structure. This could increase the stresses in the PJM support structures and the vessel wall where the PJM supports connect to the vessel. This is probably not serious since the critical locations for seismic are in the skirt and in the upper head. b. The analysis should use a horizontal response spectrum of 1/2% up to 0.5 Hz and 2% above that. This could be done with a single response spectrum that combines the 1/2% up 0.5	BNI/Eaton: a. This is being investigated as part of modeling/CFD work being performed by BNI for applicable WTP vessels globally. Results of this work will be used to respond to this comment. Results of this effort will determine if the sloshing loads on internal components due to seismic inputs is significant*. b. Agree with comment. Analysis will utilize 2% damped horizontal curves for all frequencies above sloshing modes. *BNI estimates to have response, including vendor input by 10.02.06.	Closed

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No.	**Document**	**Comments and Observations**	**BNI Remarks/ Resolutions**	**Follow-up by USACE Review Team**
	MVA0-00010-09-00002 Rev. 00A, Fatigue Analysis, UFP-VSL-0002A/B. 24590-QL-MRC-MVA0-B0002-S0011DC 24590-QL-MRC-MVA0-B0002 Rev. 001	Hz with the 2% starting at the next frequency above that. This would give the correct damping for the sloshing modes and the structural modes. This should significantly decrease the seismic stresses in the vessel (below the top head) and skirt for the full vessel case. Stresses in the top head will not decrease much because they are dominated by the nozzle loads.		
VL-4	24590-QL-POC-MVA0-00001-19-02 Rev. 00C, Seismic Data Report, HLW-VSL-00022. 24590-QL-POA-MVA0-00010-19-08 Rev. 00A, Fatigue Analysis Report, HLW-VSL-00022. 24590-QL-POA-MVA0-00010-03-21 Rev. 00F, Nozzle Loading Calculations, HLW-VSL-00022. 24590-QL-MRB-MVA0-00001-S0017 24590-QL-MRB-	Reviewer: P.D. Baughman a. The analysis does not account for seismic fluid pressure load on the internal PJM/Charge vessel structure. This could increase the stresses in the PJM support structures and the vessel wall where the supports connect to the vessel. Since the PJM maximum stress of 45,453 psi is at a support connection to a PJM, this additional mass could significantly affect the results for the PJM vessels. b. The analysis should use a horizontal response spectrum of 1/2% up to 0.3 Hz and 2% above that. This could be done with a single response spectrum that combines the 1/2% up 0.3	BNI/Eaton: a. This is being investigated as part of modeling/CFD work being performed by BNI for applicable WTP vessels globally. Results of this work will be used to respond to this comment. Results of this effort will determine if the sloshing loads on internal components due to seismic inputs is significant.* b. Agree with comment. Analysis will utilize 2% damped horizontal curves for all frequencies above sloshing modes. *BNI estimates to have response, including vendor input by 10.02.06.	Closed

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	MVA0-00001-S0019	Hz with the 2% starting at the next frequency above that. This would give the correct damping for the sloshing modes and the structural modes. This would significantly decrease the seismic stresses in the skirt. Stresses in the top head will not decrease much because they are dominated by the nozzle loads.		
VL-5	24590-QL-POD-MVA0-00001-09-00024 Rev. 00A, Finite Element Seismic Calculations, PWD-VSL-00044. 24590-QL-POD-MVA0-00001-06-07 Rev. 00D, Nozzle Loading Calculations, PWD-VSL-00044. 24590-QL-POD-MVA0-00001-09-06 Rev. 00C, Vessel Code Calculations, PWD-VSL-00044. 24590-QL-POD-MVA0-00001-09-00023 Rev. 00A, Finite Element Fatigue Calculations,	Reviewer: P.D. Baughman a. The elements representing the fluid should be described in detail, and proper treatment of the fluid response and interaction with the vessel and internal components verified. b. A less conservative, yet valid, result can be obtained by using a composite horizontal response spectrum consisting of the ½% spectrum up a frequency slightly above the sloshing frequency, then the 2% above that frequency c. Reduction in excess conservatism could also be realized by combining stress components on an element by	BNI/Eaton: a. This is being investigated as part of modeling/CFD work being performed by BNI for applicable WTP vessels globally. Results of this work will be used to respond to this comment. Results of this effort will determine if the sloshing loads on internal components due to seismic inputs is significant.* b. Current requirement for the vendor is to use 0.5% damping for the sloshing modes and 2% damping for impulsive modes. Vendor is not required to combine the two modes into one curve but can choose to do this if desired. c. Although conservative, this approach is not incorrect. This calculation will be replaced with BNI analysis to include the increased	Closed

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	PWD-VSL-00044.	element basis rather than just taking maxima from different locations, and by differentiating between local stresses at discontinuities and stresses away from discontinuities. d. Excess conservatism can be removed by running three loading cases (E-W, N-S and Vertical) and then combining the results of the three into a final seismic case by SRSS, to avoid double-counting the vertical excitation results. These results would then be combined with the static load cases (dead load, nozzle loads, etc.) for comparison with the appropriate stress limit. e. The report should describe how static loads are combined with the seismic loads. f. There should be a membrane stress check against the 1.2S limit in addition to the membrane plus bending check against the 1.8S limit, although it is unlikely there would be any overstresses. This might be beyond the capabilities of the computed program used for the	loading from RGM. If the calculation shows overstressed conditions, it will be reviewed to ensure the combined stresses at the overstressed point has conservatisms removed. d. See c. above e. Agree with comment. The vendor calculation is being superceded by BNI analysis. Comment will be included with the BNI analysis. f. Agree with comment. Calculation is being superceded by BNI analysis. Comment will be include with the BNI analysis g. Agree with comment. Calculation is being superceded by latest BNI analysis. Comment will be include with the BNI analysis h. Agree with comment. Analysis of nozzles via FEA seismic calculation includes all loadings from internal	

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		analysis. g. There should be a compressive stress check for the skirt. The allowable compressive stress for the skirt will be lower than the 1.8S stress limit used in the report. The compressive stress check can be done as separate hand calculation using the skirt support reaction forces and moments. h. The nozzle calculations should include reaction loads from the internal pipe, or else the external pipe loads should be included in the vessel seismic analysis. Both sets of loads induce stresses in the vessel.	and external piping/equipment. The nozzle load calculation only includes external loads. The seismic FEA analysis qualifies the nozzles for all load combinations. * * a. BNI estimates to have response, including vendor input by 10.02.06. h. BNI estimates to have response, including vendor input by 08.31.06.	
VL-6	24590-QL-POC-MVAO-00001-03-19-00006 Seismic Report for CXP-VSL-00001 24590-QL-POC-MVAO-00001-03-17 External Nozzle Loads Analysis for CXP-VSL-00001	Reviewer: Robert Campbell a. **Damping:** The table of loads for the skirt refer to N-S and E-W response being computed at 0.5% damping. Only the fluid sloshing mode should have 5% damping. The impulsive mode should have 2% damping consistent with level 1 response. Level 1 response is appropriate since skirt buckling will likely be the governing failure mode. Internal piping should be evaluated using 5% damping as	BNI/Eaton: a. The calculation will be revised with the RGM loading and will provide for 0.5% damping for sloshing modes and 2% damping for impulsive modes. Internal piping will be designed at the 2% damped curves since we are not directed to use ASME Section III.	a. In accordance with Specification 24590-WTP-3PS-SS90-T00001 for Seismic Qualification of Category I/II Equipment and Tanks, up to 4% damping could be used for all other equipment which could be interpreted to include internal piping. The 2% damping clearly applies only to the impulsive mode fluid structure interaction for

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		suggested in ASME Section III, Appendix N and ASCE-43. b. **Earthquake Component Combination:** The analysis of the skirt is based on an SRSS combination of the two overturning moments. This is incorrect as it implies that the two horizontal components are in phase and the vector of the in-phase moments results from SRSS. The end item of interest, namely the stresses, should be combined by SRSS. In this case, the maximum stress from one moment is 90 degrees away from the maximum stress resulting from the orthogonal overturning moment and the governing stress is just that resulting from the maximum overturning moment in one direction. Alternatively, if loads are combined, the 100-40-40	b. This analysis is conservative as discussed in the comment. If this conservatism results in reporting of an overstressed condition then the technique used will be re-evaluated. c. This comment is being discussed with the vendor before final resolution. *	Response level 1 which governs for loads attributed to skirt buckling I don't have the piping specification but assume that either 5% damping or CC N-411 damping is being used for piping external to the tanks. The use of higher damping for internal piping should be reconsidered in lieu of the apparent high stresses created by use of 2% damping. <u>Open to consideration</u> b. Closed c. Open pending final resolution with vendor.

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		rule can be used. In this case, the resulting moment is the vector of 100% of the maximum moment and 40% of the lesser moment. Use of the 100-40-40 rule results in less misinterpretation of resulting stresses. The analysis as conducted is overly conservative. c. **Table of Loads for Support Skirt:** The following table of loads in the support skirt, taken form the skirt analysis report, raises several questions. 1) It is assumed that the difference in the MX and MZ moments is due to different spectral amplifications of the NS and EW earthquake spectra at the tank foundation. Otherwise it would be expected that the dominant frequencies in the two horizontal directions would be the same and for the same input in each direction, the two overturning moments would be equal. 2) There is no vertical load FY for the vertical seismic case. Perhaps the analyst assumed that the dead	d. All nozzle loads are being revised due to the RGM effort and are provided to the vendor along with the revised ISRS curves. Analysis of nozzles via FEA seismic calculation includes all loadings from internal and external piping/equipment. The nozzle load calculation only includes external loads. The seismic FEA analysis qualifies the nozzles for all load combinations. * e. This comment will be discussed with the vendor prior to final comment response to ensure they have properly incorporated the specification requirements. Specification 24590-WTP-3PS-MV00-T0001 Section 3.7 requires the vendor combine weight plus seismic plus operating pressure as primary nozzle loads and to	d. Closed e. The vessel specification appears to be in conflict with the required ASME Code analysis of nozzles. Section VIII Division 2 clearly states that loading on nozzles shall include restraint of thermal expansion loads. The resulting stresses are then classified as primary or secondary. Membrane is primary and bending through

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		weight would cancel out any vertical uplift load. 3) There is no dead weight load. The tank when full weighs over 1 million pounds. 4) The skirt weld analysis should be based on the resulting tensile loading in the skirt. A skirt buckling analysis should be conducted for compressive loads from DW, three directions of seismic loading and external nozzle loading.	combine the primary loads plus thermal loads as secondary nozzle loads. * * BNI estimates to have response, including vendor input by 06.29.06.	the wall is secondary. As stated, the vendor secondary load case includes all of the loads and is valid, but there is no stress check on this load combination for local membrane stress and comparison to primary limits. <u>Open</u>

Design Basis Earthquake Reaction Forces for the CXP Vessel – Harris Thermal #23244								
Seismic Load Case	**FX (lbs)**	**FY (lbs)**	**FZ (lbs)**		**MX (lb-in)**	**MY (lb-in)**		**MZ (lb-i**
EW 0.5% damping	0	2410	257570		55.8E6	138100		191
NS 0.5% damping	560330	2000	0		172.2E6	136300		145
Vertical 2% damping	750	0??	740		16900	42400		21
RSS Seismic Comb.	560340	3130	257600		181.0E6	199000		240
Nozzle Loading	-1430	5030	2600		821000	284000		40
Seismic + Nozzle Loads	558900	8159	260170		181.9E6	482900		241.

d. **Nozzle Loads:** The nozzle loads were provided by BNI in an email dated Feb 24, 2003 and are less than the current specified loads in 24590-WTP-3PS-MV00-TP001, Rev. 2. Certain nozzles are also subject to loads from internal piping. The nozzle analyses should

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		include nozzles loaded by external and internal piping and any cases of internal and external nozzle loads on the same nozzle. e. **Nozzle Analysis:** The vendor interprets the nozzle load combination as being two separate load combinations broken down as primary and secondary loads. The primary load case is considered to be dead weight plus seismic. The secondary load case is considered to be dead weight, seismic and restraint of thermal expansion. This is contrary to the ASME code, Section VIII, Division 2 guidance in Appendix 4-138 for nozzle piping transitions. The loading on nozzles is clearly stated to include restraint of thermal expansion, thus there is only one load case. The vendor's analysis for so called primary loads and comparison to primary allowable stresses in not a valid demand to capacity check. However, the vendor's analysis for so called secondary loads includes all external loading and the resulting stresses can be compared to the local		

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		membrane limits and local membrane plus secondary bending stress limits of the code. I was advised by BNI that this issue has been raised with the vendor and the software they use conducts the analysis in this manner and can't be over ridden. Since one of the two load combination cases they evaluate is valid, the computational method is considered acceptable though confusing.		
VL-7	24590-QL-SRA-MTE5-0001-47-00001, FRP-VSL-00002A/B/C/D, Waste Feed Receipt Vessel and Pulse Jet Mixers, Dec. 8, 2005 CCN: 136987 Independent Review of the Waste Feed Receipt Vessel FRP-VSL:-00002 A,B,C,D, March 8, 2006	Reviewer: Robert Campbell a. **Damping used in seismic analysis**: Specification 24590-WTP-3PS-SS90-T0001 specifies damping values that are consistent with recommendations in ASCE4-98. For level 1 response, 2% is to be used for impulsive modes. For Level 2 response, 3% may be used. In each case, 0.5% damping is specified for the sloshing modes. For developing response of the tank for evaluation of attached internal structures, 2% should be used. For structural evaluation of the vessel 3% may be used. However, if instability is the controlling failure mode, then	BNI/Eaton: a. The analysis reviewed utilized a static equivalent approach on the vessel internals. The vendor will be required to provide a coupled dynamic analysis for the vessel and internals. The check run by BNI (coupled dynamic) shows overstressed at the 2" nozzle/PJM interface using a 2% damped curve. BNI has no direction at this time to use 5% damping for the vessel internal piping. If it can be shown that stability is not of concern, the spec allows use of 3% damped curves for Level 2 response.	a. In accordance with Specification 24590-WTP-3PS-SS90-T00001 for Seismic Qualification of Category I/II Equipment and Tanks, up to 4% damping could be used for all other equipment which could be interpreted to include internal piping. The 2% damping clearly applies only to the impulsive mode fluid structure interaction for Response level 1 which governs for loads attributed to skirt buckling I don't have the piping specification but assume that either 5% damping or CC N-411

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		2% should be used. The vendor has used 2% damping for the impulsive mode of the tank response. This is appropriate since the support skirt failure mode is instability and is slightly overloaded. The analysis of the PJM and support structure is based on an equivalent static loading taken as the peak of the 4% damped floor spectra at the tank location. In accordance with the BNI specifications and ASCE 4-98 recommendations, up to 4% damping could be used for the internal structures provided that the failure mode was structural. For instability failure modes 3% would be the correct damping to use. The PJM assembly and internal support structures are a mixed bag for damping. The loads in the PJM head and WFR head at the 14 inch pipe connections could be predicted by 4% damped response whereas loads from the response of the internal assembly applied to some of the internal lateral supports should be based on 3% damping due to their instability failure modes. The stress at the PJM shell and	b. The analysis reviewed utilized a static equivalent approach on the vessel internals. The vendor will be required to provide a coupled dynamic analysis for the vessel and internals.	damping is being used for piping external to the tanks. The use of higher damping for internal piping should be reconsidered in lieu of the apparent high stresses created by use of 2% damping. <u>Open to consideration</u> . b. Closed provided that the coupled model properly accounts for hydrodynamic mass

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		WFR shell from lateral support loads could reasonably be based on 4% damped response of the internal assembly. This would imply that two analyses be conducted for the internal assembly at 3% and 4% damping respectively to get loads compatible with the instability and stress based failure modes. This could make a difference of as much as 15% depending on the input motion spectral shape and frequency of the internal structure. In the case of the coupled BNI 3D model, composite modal damping could be used where, the vessel shell and skirt are assigned 2% damping and the PJM assembly is assigned 3% damping for one case and 4% damping for the second case. This would tend to reduce the loads in the PJM supports, thus the stresses in the PJM head and shell and the WFR head and shell. The response of the 2” pipe from the PJMs top heads inside the 14” support pipes should be based on 5% damping as recommended in ASME Section III, Appendix N and ASCE 43. Use of 5% damping would reduce the	c. Agree with comment. Comment will be incorporated into the revised vendor analysis.	c. Closed

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		response and may alleviate the predicted overstress condition. b. **Hydrodynamic mass:** The CB&I analysis of the PJM assembly and internal support structure is based on an equivalent static loading equal to the peak of the floor response spectra. CB&I stated that the sloshing portion of fluid with the tank full is above the PJMs and with the WJM fluid level low, the sloshing loads on the PJMs is small and is ignored. The effective hydrodynamic mass has been neglected in the analysis. When an assembly is submersed in fluid, a hydrodynamic mass should be added to the immersed elements. A simple approach that is typically taken is to add a mass equal to the mass of the fluid displaced. A more accurate and complex approach is described in ASCE 4, paragraph 3.1.6.2. This would lower the frequency and, depending on the frequency relative to the input motion spectral shape, raise or lower the response. The added hydrodynamic mass would always increase the reactions relative to a given	d. There is no direction to utilize 5% damping based on ASME Section III. Agree that the vendor must include the 2" pipe in the stress model and determine stress levels for those components. This will be provided to the vendor as a comment for inclusion.	d. In accordance with Specification 24590-WTP-3PS-SS90-T00001 for Seismic Qualification of Category I/II Equipment and Tanks, up to 4% damping could be used for all other equipment which could be interpreted to include internal piping. The 2% damping clearly applies only to the impulsive mode fluid structure interaction for Response level 1 which governs for loads attributed to skirt buckling I don't have the piping specification but assume that either 5% damping or CC N-411 damping is being used for piping external to the tanks. The use of higher damping for internal piping should be reconsidered in lieu of the apparent high stresses created by use of 2% damping. <u>Open to consideration</u>

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		spectral acceleration. In this case, the CB&I analysis is very likely un-conservative. The coupled BNI model has all essential components and supports and includes the fluid in the model and it is assumed that the finite element solution incorporates the added hydrodynamic mass effects by means of the connections of the fluid and solid elements. The shell stresses in the WFR and PJM vessels are higher for the coupled model at the point of connection of the lateral supports indicating that the effective added hydrodynamic mass is likely present and increases the loads. c. **Interpretation of secondary stresses**: The allowable combination of primary local membrane (P_L), primary bending (P_b) and secondary (Q) stress is limited to 3S = 60ksi. In cases where shell analysis is used, the calculated through the wall bending stress is linear and is classified as secondary such as around nozzles and in the shell at support attachments. This is typical in the case of analysis of		

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		nozzle loads using WRC-107 equations. However, in cases where axisymmetric or 3 D finite elements are used with several elements through the wall, the maximum surface stress has been taken as the equivalent linear secondary bending stress. In these cases, the interpretation is likely overly conservative as the through the wall stress in highly nonlinear with concentrated stress at discontinuities at the surface. This concentrated stress is used in fatigue analysis but should not be interpreted to be secondary bending. In these cases, the analyst needs to do a linearization of the through the wall stress. This equivalent linearized stress is then interpreted as the secondary stress. In the CB&I analysis of the connections of the 14 inch diameter pipe supports between the PJMs and WFR, the heads of both vessels are analyzed using a solid of revolution FE program and from the plots of stress, the maximum stress used as the secondary stress appears to be very local on the		

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		surface. Linearization of the through the wall stress should result in a lower stress to consider as secondary for comparison to the 60 ksi allowable. The BNI independent review summary does not describe the element mesh or elements used in the model, consequently I can't determine if there should be some linearization of stress through the wall or not. BNI needs to address this with the vendor to assure that the peak stresses are not being used as secondary stress in the evaluation. d. **Missing Hardware**: There is a 2 inch pipe from each PJM to the WFR inside of the 14 inch support pipes. It appears that this was not addressed by CB&I. This pipe is included in the BNI independent review analysis and it appears that the connection of the pipe to the PJM head results in an overstress condition. It is not clear from the BNI summary what this stress is as the reported numbers appear to be the highest of either the horizontal support to shell junction or the 2 inch pipe to PJM head junction stress. The		

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		2 inch pipe to PJM head stress is assumed to be a peak surface stress and not the linearized stress through the wall although it is not clear from the stress plots where exactly the highest stress is. It may be in the cross section of the 2 inch piping itself since the PJM head is thicker than the pipe wall. The 2 inch pipe has a very long unsupported length, thus would be expected to have a high response. As discussed in item 1, damping, the pipe response should be predicted using 5% damping. This would lower the response and possibly alleviate the nozzle overstress condition.		
VL-8	24590-QL-POA-MVAO-00006-08-00002, Rev 00B, Seismic Analysis of 180" Cesium Exchange Treated Law Collection Vessel CXP-VSL-00026 A/B/C 24590-QL-POA-MVAO-00006-04-03 Nozzle Analysis	Reviewer: Robert Campbell a. **Damping:** The vendor used 0.5% damping for the horizontal modes and 2% damping for the vertical modes. The horizontal damping was too low and results in compressive stresses in the shell and support skirt that are near the ASME code limit for instability. For instability failure modes, level 1 response damping should be used. This would be 0.5% for the sloshing mode and 2% for the impulsive mode. The PJMs contribute to	BNI/Eaton: a. The vendor utilized 0.5% damping for both sloshing and impulsive modes since this was conservative and acceptable for the baseline seismic curves. With RGM loading increases the vendor will be required to utilize 0.5% damping for sloshing modes and 2% damping for impulsive modes. Internal piping will be analyzed to the same curves as the vessel since we have no direction at this time to utilize the 5% damped curves. ASME Section III does not apply for these vessels.	a. In accordance with Specification 24590-WTP-3PS-SS90-T00001 for Seismic Qualification of Category I/II Equipment and Tanks, up to 4% damping could be used for all other equipment which could be interpreted to include internal piping. The 2% damping clearly applies only to the impulsive mode fluid structure interaction for Response level 1 which

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		the overturning of the vessel and for the instability failure mode check, they too should have 2% damping. Internal piping should have 5% damping for all conditions in accordance with ASME Section III, Appendix N and as recommended in ASCE 43. The stresses in the PJM appear not to be critical except for the case of the piping in the top head. If other areas become critical on a stress basis, Level 2 damping of 3% for the PJMs would be appropriate. The use of more than one damping ratio requires a composite modal damping analysis as described in ASCE 4-98, paragraphs 3.1.5.2, 3.1.5.3 or 3.1.5.4.	b. Agree with comment. This will be passed along to the vendor as a comment for incorporation and included in the analysis re-submittal. *	governs for loads attributed to skirt buckling I don't have the piping specification but assume that either 5% damping or CC N-411 damping is being used for piping external to the tanks. The use of higher damping for internal piping should be reconsidered in lieu of the apparent high stresses created by use of 2% damping. <u>Open to consideration</u> b. Closed
		b. **Hydrodynamic mass:** The effective hydrodynamic mass for an immersed body appears to have been neglected in the analysis. When an assembly is submersed in fluid, a hydrodynamic mass should be added to the structural elements. A simple approximation is to add the	c. Agree with comment. Specification requires stress intensities versus Von Mises stresses. Vendor will be required to report in stress intensities. More detail will be provided for modeling around the nozzles and contained in the analysis re-submittal. *	c. Closed

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		mass of the fluid displaced. More accurate and complex methods for accounting for this effect are discussed in ASCE 4-98, paragraph 3.1.6.2. This would lower the frequency of the PJMs and, depending on the frequency relative to the input motion spectral shape, raise or lower the response. The added hydrodynamic mass would always increase the reactions relative to a given spectral acceleration. Use of the correct damping for the horizontal modes and addition of hydrodynamic mass tend to offset each other but the analysis must be corrected to determine the overall effect. The internal piping should have hydrodynamic mass added transverse to the piping for the case of the vessel full of fluid. c. **Interpretation of stresses:** The von Mises stress output from NASTRAN has been used to compared to ASME code allowable stress in most cases. The von Mises stress derivation is different than the derivation of stress intensity as required by Section VIII, Division 2. Unless the calculated von Mises stress	d. This comment will be discussed with the vendor prior to final comment response to ensure the vendor has properly incorporated the specification requirements. Specification 24590-WTP-3PS-MV00-T0001 Section 3.7 requires the vendor combine weight plus seismic plus operating pressure as primary nozzle loads and to combine the primary loads plus thermal loads as secondary nozzle loads. * *BNI estimates to have response,	d. The vessel specification appears to be in conflict with the required ASME Code analysis of nozzles. Section VIII Division 2 clearly states that loading on nozzles shall include restraint of thermal expansion loads. The resulting stresses are then classified as primary or secondary. Membrane is primary and bending through the wall is secondary. As stated, the vendor secondary load case includes all of the loads and is valid, but there is no stress check on this load combination for local

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		is near the ASME code allowable stress and the principal stresses are of like sign, the two stress derivations are close enough that the comparison is acceptable. In cases where the von Mises stress in the shell of the vessel and PJM were higher than the ASME code allowable stress for axial buckling, the internal forces were used to derive axial compressive stress. The highest stress area is in the PJMs at the nozzle in the top head. Internal piping loads from the vessel empty condition result in a quoted peak stress of 66,671 psi which is greater than the allowable stress of 60,000 psi. There are two issues here in using surface stress output from finite element models. First, it is not clear if this is a peak stress such as in a fillet or notch or if it is a linearized through wall bending stress. It is stated that the NASTRAN model is constructed of plate elements. If these elements are based on thin plates then the reported peak stress would be considered to be the primary local membrane plus secondary bending stress for comparison to	including vendor input by 06.29.06.	membrane stress and comparison to primary limits. <u>Open</u>

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		the 60,000 psi code allowable. It is not clear though what the stress distribution is around the nozzle and if the von Mises derivation is nearly the same as the stress intensity. There is insufficient information in the report to understand the details of the modeling around the nozzle. The vendor needs to examine this case in detail and determine if the stress reported is truly a local membrane plus linear bending stress through the wall and if the von Mises stress is at a local stress concentration or if a stress intensity should be derived from the local membrane and bending loads. d. The vendor interprets the nozzle load combination as being two separate load combinations broken down as primary and secondary loads. The primary load case is considered to be dead weight plus seismic. The secondary load case is considered to be dead weight, seismic and restraint of thermal expansion. This is contrary to the ASME code, Section VIII, Division 2 guidance in Appendix 4-138 for nozzle piping		

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		transitions. The loading on nozzles is clearly stated to include restraint of thermal expansion, thus here is only one load case. The vendor's analysis for so called primary loads and comparison to primary allowable stresses in not a valid demand to capacity check. However, the vendor's analysis for so called secondary loads includes all external loading and the resulting stresses can be compared to the local membrane limits and local membrane plus secondary stress limits of the code. I was advised by BNI that this issue has been raised with the vendor and the software they use conducts the analysis in this manner and can't be over ridden. Since one of the two load combination cases they evaluate is valid, the computational method is considered acceptable though confusing.		
VL-9	24590-QL-POC-MVAO-00001-03-18 HLP-VSL-0027A 24590-QL-POC-MVAO-00001-03-19 HLP-VSL-0027B	Reviewer: Robert Campbell a. The vendor interprets the nozzle load combination as being two separate load combinations broken down as primary and secondary loads. The primary	BNI/Arulampalam: a. This comment will be discussed with the vendor prior to final comment response to ensure they have properly incorporated the specification requirements.	 a. The vessel specification appears to be in conflict with the required ASME Code analysis of nozzles. Section VIII Division 2 clearly states

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	24590-QL-POC-MVAO-00001-03-20 HLP-VSL-0028	load case is considered to be dead weight plus seismic. The secondary load case is considered to be dead weight, seismic and restraint of thermal expansion. This is contrary to the ASME code, Section VIII, Division 2 guidance in Appendix 4-138 for nozzle piping transitions. The loading on nozzles is clearly stated to include restraint of thermal expansion, thus there is only one load case. The vendor's analysis for so called primary loads and comparison to primary allowable stresses in not a valid demand to capacity check. However, the vendor's analysis for so called secondary loads includes all external loading and the resulting stresses can be compared to the local membrane limits and local membrane plus secondary bending stress limits of the code. I was advised by BNI that this issue has been raised with the vendor and the software they use conducts the analysis in this manner and can't be over ridden. Since one of the two load combination cases they evaluate is valid, the computational method is	Specification 24590-WTP-3PS-MV00-T0001 Section 3.7 requires the vendor combine weight plus seismic plus operating pressure as primary nozzle loads and to combine the primary loads plus thermal loads as secondary nozzle loads. * b. This is being discussed with the vendor before final disposition. *	that loading on nozzles shall include restraint of thermal expansion loads. The resulting stresses are then classified as primary or secondary. Membrane is primary and bending through the wall is secondary. As stated, the vendor secondary load case includes all of the loads and is valid, but there is no stress check on this load combination for local membrane stress and comparison to primary limits. <u>Open</u> b. The vessel specification appears to be in conflict with the required ASME Code analysis of nozzles. Section VIII Division 2 clearly states that loading on nozzles shall include restraint of thermal expansion loads. The resulting stresses are then classified as primary or secondary. Membrane is primary and bending through the wall is secondary. As stated, the vendor secondary load case includes all of the

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		considered acceptable though confusing. b. The vendor is inappropriately comparing the wrong stresses to the ASME code limits. The total membrane plus bending stress from the so called primary load case is compared to the ASME code allowable stress for primary membrane plus primary bending allowable. This in not correct. The load case is missing the restraint of thermal expansion loads and the through the wall bending stress is secondary, not primary. This stress check should be ignored. A second stress check is made using the load case with weight, seismic and thermal loads and the membrane plus bending stress is compared to the ASME code allowable stress for local membrane plus secondary bending. This stress check is correct. An additional check is required for this load case to check the membrane stress against the ASME code allowable stress for primary local membrane stress. Examination of the controlling cases shows that this stress check should easily be met. BNI has	c. This is being discussed with the vendor before final disposition. * *BNI estimates to have response, including vendor input by 06.29.06.	loads and is valid, but there is no stress check on this load combination for local membrane stress and comparison to primary limits. <u>Open</u> c. Closed

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		apparently raised this issue with the vendor also and was informed that this is an automatic stress check done by their software that can't be overridden. The nozzles stress check report should explain this so that the reviewer is not confused. Otherwise, it can easily be assumed that the analyst does not understand the code rules and cast doubt on the validity of the analysis. c. The allowable stress for primary plus secondary stress quoted is the lesser of 3S or 2Sy. For SA240 type 316 stainless steel, the 2Sy value governs at the design temperature. The BNI criteria call for use of Section VIII, Division 2, Appendix 4 stress categories and allowable stress values. The 2Sy limit is not present in Appendix 4 of Division 2. The 2Sy limit has recently been added to Section VIII, Division 1 as a limit on discontinuity stresses at transitions in shell geometry and is likely not intended for localized shell bending at nozzles. If the 2Sy limit is used, two nozzles will not pass. The 3S limit is adequately		

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		conservative and is the appropriate limit to use in accordance with the BNI specifications to use Section VIII, Division 2 stress categories and limits. The allowable stress should be based on 3S without consideration of the 2Sy limit for discontinuity stress in Section VIII, Division 1.		

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PS-1	USACE Piping Presentation, April 17, 2006 by John Minichiello	Reviewer: Daniel J. Weinacht, Ph.D., PE Overall Design Process/Approach is complete and comprehensive. Design Codes & Standards, Design Inputs, and Piping and Pipe Support Analysis Techniques are appropriate. Conservative Design results in Prudent Design Margins, given current state of WTF design, engineering, and construction activities. slide #6 - assumption of 5% damping on response spectra input appears reasonable slide #11 - no springs in black cells and no snubbers used thus far on WTF = good design approach	Observations
PS-2	24590-WTP-DC-PS-01-002, Revision 3 - Pipe Support Design Criteria 24590-WTP-GPG-ENG-005, Revision 2 - Engineering Design Guide for Pipe Supports	Reviewer: Daniel J. Weinacht, Ph.D., PE Complete and comprehensive Design Criteria. Design Codes & Standards, Design Inputs, and Pipe Support Analysis Techniques are appropriate.	Observations
PS-3	24590-PTF-PHC-FRP-50001 Engr Support Calc for PTF-FRP-H25087, H25081, H25091 & H25171 24590-PTF-PHC-FRP-50004 Engr Support Calc for PTF-FRP-H25088 &	Reviewer: Daniel J. Weinacht, Ph.D., PE Bechtel was very supportive/forthcoming relative to the needs of the review team. The availability and responsiveness of John Minichiello was especially noteworthy. The calculation packages supplied appeared to be complete. Fifteen different calculations for black cell piping supports were reviewed. Design Codes & Standards, Design Inputs, and Pipe Support Analysis Techniques are appropriate.	Observations

[1] "Observation" is for information only - a response is not required.

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	H25170 24590-PTF-PHC-FRP-50005 Engr Support Calc for PTF-FRP-H25101, H25095, H25098, H25142, H25107, H25115, H25004, H25085,& H25017 24590-PTF-PHC-FRP-50006 Engr Support Calc for PTF-FRP-H25086 24590-PTF-PHC-FRP-50007 Engr Support Calc for PTF-FRP-H25089 24590-PTF-PHC-FRP-50008 Engr Support Calc for PTF-FRP-H35035, H35028, H35030, H35031, H35033, H35037, H35039, H35006, H35021, H35024, H35019, H35017 & H35060 24590-PTF-PHC-FRP-50016 Engr Support Calc for PTF-FRP-H25108 & H25109 24590-PTF-PHC-FRP-50017 Engr Support Calc for PTF-FRP-H35040,	Piping Support analyses/calculations examined and provided margins for: displacements/deflections; stresses; welds; localized effects; and standard hardware components (e.g., U-bolts). All analyses/calculations were linear elastic, considered weight + thermal + seismic, and were for relatively low-temperatures. All analyses reviewed were complete/comprehensive and adhered to good QA practices (e.g., V&V, statement of Codes & Standards employed, checking/signature/approvals, and document control). Appears that prudent design margins were achieved throughout all calculations reviewed. Note - 24590-PTF-PHC-FRP-50070, Engr Support Calc for PTF-FRP-H25111, Sheet 3.1 indicates minimum margin factor is 7.922 (weld at joint 1, sheet 4.12); however, sheet 6.1 indicates that MF = 2.27 for Guide-U Bolt. The following table summarizes the design margins with revised ground motion:	

Calculation #	Subject	OCC	Min. Margin
24590-PTF-PHC-FRP-50001	Engr Support Calc for PTF-FRP-H25087, H25081 & H25091	1.2 w/RGM	2.3
	Engr Support Calc for PTF-FRP-H25171	1.2x1.5 w/o RGM	
24590-PTF-PHC-FRP-50002	Std Support Calc for 24590-PTF-P6C-FRP-50006		
24590-PTF-PHC-FRP-50004	Engr Support Calc for PTF-FRP-H25088	1.2 w/RGM	1.8
	Engr Support Calc for PTF-FRP-H25170	1.2x1.5 w/o RGM	
24590-PTF-PHC-FRP-50005	Engr Support Calc for PTF-FRP-H25101, H25095, H25098, H25142, H25107, H25115, H25004, H25085	1.2 w/RGM	2.7
	Engr Support Calc for PTF-FRP-H25017	1.2x1.5 w/o RGM	
24590-PTF-PHC-FRP-50006	Engr Support Calc for PTF-FRP-H25086	1.2 w/RGM	1.7

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	H35032, H35036, H35038; PTF-PWD-H35008 & H30003, H35016, H35018, H35020, H35023, H35003; PTF-PVP-H35062 & H35059; PTF-PWD-H30022 & H30007 24590-PTF-PHC-FRP-50018 Engr Support Calc for PTF-FRP-H25090 & H25097 24590-PTF-PHC-FRP-50068 Engr Support Calc for PTF-FRP-H35029, H35027, H35034 24590-PTF-PHC-FRP-50069 Engr Support Calc for PTF-FRP-H25110 24590-PTF-PHC-FRP-50070 Engr Support Calc for PTF-FRP-H25111 24590-PTF-PHC-FRP-50071 Engr Support Calc for PTF-FRP-H25112, H25104 24590-PTF-PHC-FRP-50072 Engr Support Calc for PTF-FRP-H25113	(see table below)	

24590-PTF-PHC-FRP-50007	Engr Support Calc for PTF-FRP-H25089	1.2 w/RGM	2
24590-PTF-PHC-FRP-50008	Engr Support Calc for PTF-FRP-H35035, H35028, H35030, H35031, H35033, H35037, H35039, H35006	1.2 w/RGM	1.8
	Engr Support Calc for PTF-FRP-H35021, H35024, H35019, H35017, H35060	1.2x1.5 w/o RGM	
24590-PTF-PHC-FRP-50016	Engr Support Calc for PTF-FRP-H25108 & H25109	1.2 w/RGM	1.2
24590-PTF-PHC-FRP-50017	Engr Support Calc for PTF-FRP-H35040, H35032, H35036, H35038; PTF-PWD-H35008 & H30003	1.2 w/RGM	1.7
	Engr Support Calc for PTF-FRP-H35016, H35018, H35020, H35023, H35003; PTF-PVP-H35062 & H35059; PTF-PWD-H30022 & H30007	1.2x1.5 w/o RGM	
24590-PTF-PHC-FRP-50018	Engr Support Calc for PTF-FRP-H25090 & H25097	1.2 w/RGM	1.6
24590-PTF-PHC-FRP-50068	Engr Support Calc for PTF-FRP-H35029, H35027, H35034	1.2 w/RGM	1.8
24590-PTF-PHC-FRP-50069	Engr Support Calc for PTF-FRP-H25110	1.2 w/RGM	1.7
24590-PTF-PHC-FRP-50070	Engr Support Calc for PTF-FRP-H25111	1.2 w/RGM	2.3 or 7.9?
24590-PTF-PHC-FRP-50071	Engr Support Calc for PTF-FRP-H25112, H25104	1.2 w/RGM	3.8
24590-PTF-PHC-FRP-50072	Engr Support Calc for PTF-FRP-H25113	1.2 w/RGM	2
		Minimum Value	**1.2**

C.7 Piping and Piping Supports Review Comments			
No.	**Document**	**Comments and Observations**	**Action By and Respo**
PS-4	24590-PTF-P6C-FRP-00074, Rev. C, PTF Pipe Stress Analysis – FRP Pipe Systems. 24590-PTF-PHC-FRP-00225, Std Support Calc for 24590-PTF-P6C-FRP-00074 24590-PTF-PHC-FRP-00226, Engr Support Calc for PTF-FRP-H30155&H30156	Reviewer: Paul Baughman Pipe stress calculation: ½" diameter Pneumercator piping. One piping system analyzed is typical of six systems. Piping run in gang hangers with U-bolts, well supported laterally. SC-1 criteria. Response spectrum analysis by ME 101. Reanalyzed with RGM spectra. Maximum stress ratio for occasional loading is 0.66. This is at the terminal anchor point with a SIF of 2.1. No nozzle loads. Current analysis shows margin on seismic loading of about 2, considering that the occasional load stress ratio includes sustained stress in additional to seismic stress. Pipe support calculations: Maximum occasional load interaction ratio for the standard supports is 0.55, for the engineered supports 0.36. Margin on seismic loading is more than a factor of 2.	Observations
PS-5	24590-PTF-P6C-PVP-50004, Pipe Stress Analysis - Forced Purged Air Inlet System 24590-PTF-PHC-PVP-50132 Engr Support Calc for PTF-PVP-H35122 24590-PTF-PHC-PVP-50133 Std Support Calc for 24590-PTF-P6C-PVP-50004 24590-PTF-PHC-PVP-50134 Engr	Reviewer: Paul Baughman Pipe stress calculation: 1-1/2" pipe. SC-1 criteria. Response spectrum analysis by ME 101. Reanalyzed with RGM spectra. Maximum stress ratio for occasional loading is 0.42. Current analysis shows margin on seismic loading of more than 2. Pipe support calculations: There are many engineered guide and three-way supports, as well as many standard supports. One calculation covers the standard supports and the rest cover the engineered supports. The sustained loads are increased by 20%. The interaction ratio for the standard supports (worst one is given) is relative to the support standard capacity. There is additional margin in the standard capacity. The interaction ratios on the engineered supports are based the pipe support stress criteria. The occasional load interaction ratios are:	Observations

C.7 Piping and Piping Supports Review Comments

No.	Document	Comments and Observations	Action By and Respo
	Support Calc for PTF-PVP-H25165 24590-PTF-PHC-PVP-50135 Engr Support Calc for PTF-PVP-H25327 24590-PTF-PHC-PVP-50136 Engr Support Calc for PTF-PVP-H25164 24590-PTF-PHC-PVP-50137 Engr Support Calc for PTF-PVP-H25162 & H25177 24590-PTF-PHC-PVP-50139 Engr Support Calc for PTF-PVP-H35120 24590-PTF-PHC-PVP-50140 Engr Support Calc for PTF-PVP-H35007 24590-PTF-PHC-PVP-50141 Engr Support Calc for PTF-PVP-H35115 24590-PTF-PHC-PVP-50142 Engr Support Calc for PTF-PVP-H25357 24590-PTF-PHC-PVP-50143 Engr Support Calc for PTF-PVP-H25178 24590-PTF-PHC-	(see table below)	

24590-PTF-PHC-PVP-50132	Engr Support Calc for PTF-PVP-H35122	0.18
24590-PTF-PHC-PVP-50133	Std Support Calc for 24590-PTF-P6C-PVP-50004	0.29
24590-PTF-PHC-PVP-50134	Engr Support Calc for PTF-PVP-H25165	0.07
24590-PTF-PHC-PVP-50135	Engr Support Calc for PTF-PVP-H25327	0.04
24590-PTF-PHC-PVP-50136	Engr Support Calc for PTF-PVP-H25164	0.18
24590-PTF-PHC-PVP-50137	Engr Support Calc for PTF-PVP-H25162 & H25177	0.22
24590-PTF-PHC-PVP-50139	Engr Support Calc for PTF-PVP-H35120	0.25
24590-PTF-PHC-PVP-50140	Engr Support Calc for PTF-PVP-H35007	0.21
24590-PTF-PHC-PVP-50141	Engr Support Calc for PTF-PVP-H35115	0.17
24590-PTF-PHC-PVP-50142	Engr Support Calc for PTF-PVP-H25357	0.24
24590-PTF-PHC-PVP-50143	Engr Support Calc for PTF-PVP-H25178	0.21
24590-PTF-PHC-PVP-50146	Engr Support Calc for PTF-PVP-H25169	0.18
24590-PTF-PHC-PVP-50147	Engr Support Calc for PTF-PVP-H25168	0.44
24590-PTF-PHC-PVP-50148	Engr Support Calc for PTF-PVP-H35132	0.32
24590-PTF-PHC-PVP-50150	Engr Support Calc for PTF-PVP-H25174 & H25175	0.60
24590-PTF-PHC-PVP-50153	Engr Support Calc for PTF-PVP-H25170	0.38
24590-PTF-PHC-PVP-50325	Engr Support Calc for PTF-PVP-H25146, 47, & 48	0.66

The highest interaction ratio is 0.66. As this includes the sustained load stress, the margin on seismic loading is about 2.

C.7 Piping and Piping Supports Review Comments			
No.	**Document**	**Comments and Observations**	**Action By and Respo**
	PVP-50146 Engr Support Calc for PTF-PVP-H25169 24590-PTF-PHC-PVP-50147 Engr Support Calc for PTF-PVP-H25168 24590-PTF-PHC-PVP-50148 Engr Support Calc for PTF-PVP-H35132 24590-PTF-PHC-PVP-50150 Engr Support Calc for PTF-PVP-H25174 & H25175 24590-PTF-PHC-PVP-50153 Engr Support Calc for PTF-PVP-H25170 24590-PTF-PHC-PVP-50325 Engr Support Calc for PTF-PVP-H25146, 47, & 48		
PS-6	24590-HLW-P6C-RLD-00001, Rev. D, HLW Feed Slurry Transfer Piping. 24590-HLW-PHC-RLD-00001, Pipe Support Calculation for 24590-HLW-P6C-RLD-00001	Reviewer: Paul Baughman Pipe stress calculation: 3" and 6" diameter piping. The 3" piping runs inside the 6" piping (double wall). Piping run in gang hangers with U-bolts, well supported laterally. SC-1 criteria. Response spectrum analysis by ME 101. Reanalyzed with RGM spectra. Maximum stress ratio for occasional loading is 0.46. No nozzle loads.	Observations

C.7 Piping and Piping Supports Review Comments

No.	Document	Comments and Observations	Action By and Respon
		Current analysis shows margin on seismic loading of more than 2. Pipe support calculations: This is an analysis of an anchor support. It is a stanchion-type support welded to a collar on the outside pipe. The analysis is conservative, but the occasional load interaction ratio is 1.0. This interaction ratio occurs at the weld in the stanchion between the 6" and 8" pipe sections. However, the stress in the weld is calculated quite conservatively. The sustained load is not multiplied by 1.2 as the design procedure directs. This indicates there is very little margin. However, there is significant conservatism in the design assumptions used.	
PS-7	24590-HLW-P6C-RLD-00012, Rev. C, RLD-FBOX-43 to RLD-VSL-8 Piping thru RLD Ejectors.	Reviewer: Paul Baughman Small bore piping. Seismic stresses low. Stress ratio for occasional stress is 0.36. Displacement stress range ratio is 0.92. High thermal stress at nozzle at top of riser. Due to rigid vertical support on riser. Many guide supports used on system so seismic stresses and support loads are low. One side of horizontal run has expansion loops, other side does not. Large thermal movements at ends of horizontal runs without expansion loops, but apparently does not cause stress problem. SC-1 criteria. Response spectrum analysis by ME 101. Small bore is all butt welded. Uses 3D bends. Uses SIF of 2.1 at anchor points. This is conservative. Rev. B analysis done with interim seismic criteria (OGM with 1.4 increase). Rev. C is reconciliation to RGM - justified on basis that RGM is less than interim criteria so reanalysis is not needed. CC N-411 damping was used in Rev. B, but this statement is crossed out in Rev. C. Reanalysis would 5% constant damping. Therefore, Rev. B analysis damping is conservative for RGM. Nozzle comparison is to pressure vessel specification. This is an open assumption to be checked against final nozzle load criteria. This should not be a problem because seismic loads are low. Current analysis shows margin on seismic loading of more than 2. Reanalysis would use 5% constant modal damping and inelastic energy absorption factor, so further margin could be realized.	Observations

C.7 Piping and Piping Supports Review Comments

No.	Document	Comments and Observations	Action By and Respo
PS-8	24590-PTF-PGC-FEP-00015, Pipe Stress Analysis for FEP System	Reviewer: Larry Nicholson a) This calculation indicates it is revised due to a revision to report writer. Report writer is a program language in ME101 that allows someone to create standard report modules that can be called by the analyst. Since some of these modules do calculation (e.g. Friction forces on support, corroded pipe stresses etc) these modules need to be verified. These modules are not part of the general ME101 verification. No documentation was given to me to indicate this was done. b) The ME101 Deck states that some anchors are friction types (this is correct) and thus an SIF of 1.0 can be used at these anchors. A value of 2.1 was used at these anchors, which is very conservative. c) There are three way supports in this calculation that I thought would have welded attachments and thus require local stress evaluation. I found that they were standard designs that use friction in the axial direction thus no local stress is required. In the process of checking this out I observed that in the Design Calculation 24590-WTP-PHC-P50-0001, to qualify standard supports, that if there is a welded attachment it is assumed that the secondary stress does not exceed 0.4 * Sa and primary stress does not exceed 0.9 * 1.2 *Sh. This limitation is not in the Design Criteria 24590-WTP-DC-01-001. I requested any documentation to indicate this was checked. No response was received. d) Design Criteria 24590-WTP-DC-01-001 states that Residual Rigid Response will be considered. ME101 defaults to using the total mass of the piping analysis time the cutoff acceleration and enveloping this static analysis with the inertia results. This is conservative. The industry practice is to multiply the missing mass time the ZPA and considering it an additional mode. This method is an option in ME101. e) The Hydrotest case WTHY specifies to use a star option “W”. There is no star option “W” in the deck so the default specific gravity of zero is used for this case. It look like a star option “2” should have been used since star option “2” is a not used for anything and it gives a specific gravity of 1. This is not a problem since the normal weight uses a specific gravity of 1.5 and thus controls the design. f) Thermal is run to the design temperature not the maximum temperature. This is conservative since the design temperature is equal to or greater than the maximum temperature. g) Response spectra analysis was run to 50 Hz not 33 Hz, as is the industry standard. Per discussion	a) BNI b) Observation c) BNI d) Observation e) Observation f) Observation g) Observation h) BNI

C.7 Piping and Piping Supports Review Comments			
No.	Document	Comments and Observations	Action By and Respo
		with Bechtel personnel this was done to decrease the cutoff acceleration since the spectra are still decreasing between 33 and 50 Hz. This should have insignificant affect on this analysis since ME101 is using a conservative Residual Rigid response calculation (See Item d). h) This calculation uses SRSS to combine modes. Closely space modes are not considered as required in Design Criteria 24590-WTP-DC-01-001 and is industry practice. This is not conservative and needs to be reviewed for impact.	
PS-9	24590-PTF-PGC-FRP-50006, Pipe Stress Analysis for Plant Wash System 24590-PTF-PGC-PVP-00013, Pipe Stress Analysis for PVP System	Reviewer: Larry Nicholson a) Design Criteria 24590-WTP-DC-01-001 states that Residual Rigid Response will be considered. ME101 defaults to using the total mass of the piping analysis time the cutoff acceleration and enveloping this static analysis with the inertia results. This is conservative. The industry practice is to multiply the missing mass time the ZPA and considering it an additional mode. This method is an option in ME101. b) Thermal is run to the design temperature not the maximum temperature. This is conservative since the design temperature is equal to or greater than the maximum temperature. c) Response spectra analysis was run to 50 Hz not 33 Hz, as is the industry standard. Per discussion with Bechtel personnel this was done to decrease the cutoff acceleration since the spectra are still decreasing between 33 and 50 Hz. This should have insignificant affect on this analysis since ME101 is using a conservative Residual Rigid response calculation. d) These calculations use SRSS to combine modes. Closely space modes are not considered as required in Design Criteria 24590-WTP-DC-01-001 and is industry practice. This is not conservative and needs to be reviewed for impact.	a) Observation b) Observation c) Observation d) BNI
PS-10	24590-PTF-PHC-FEP-00069 Engr Support Calc for PTF-FEP-H20187 & H20189 24590-PTF-PHC-FEP-00077 Engr Support Calc for PTF-FEP-H20246 &	Reviewer: Larry Nicholson Design Codes & Standards, Design Inputs, and Pipe Support Analysis Techniques are appropriate. Piping Support analyses/calculations examined and provided margins for: displacements/deflections; stresses; welds; localized effects; and standard hardware components (e.g., U-bolts). All analyses/calculations were linear elastic, considered weight + thermal + seismic, and were for relatively low-temperatures.	Observations

C.7 Piping and Piping Supports Review Comments

No.	Document	Comments and Observations	Action By and Respo
	H20166 & H20168 24590-PTF-PHC-FEP-00078 Standard Pipe Support for Stress Calc. 24590-PTF-PGC-FEP-00015 24590-PTF-PHC-FEP-00079 Engr Support Calc for PTF-FEP-H20186 & H20272 24590-PTF-PHC-FEP-00080 Engr Support Calc for PTF-FEP-H20188 24590-PTF-PHC-FEP-00081 Engr Support Calc for PTF-FEP-H20200 24590-PTF-PHC-FEP-00082 Engr Support Calc for PTF-FEP-H20185 24590-PTF-PHC-FEP-00086 Engr Support Calc for PTF-FEP-H20190 & H20191 24590-PTF-PHC-FEP-00087 Engr Support Calc for PTF-FEP-H20244 & H10042 24590-PTF-PHC-FEP-00088 Engr Support Calc for PTF-	All analyses reviewed were complete/comprehensive and adhered to good QA practices (e.g., V&V, statement of Codes & Standards employed, checking/signature/approvals, and document control).	

C.7 Piping and Piping Supports Review Comments			
No.	**Document**	**Comments and Observations**	**Action By and Respo**
	FEP-H20235		
PS-11	24590-PTF-PHC-PVP-00044 Engr Support Calc for PTF-PVP-H30041 24590-PTF-PHC-PVP-00045 Engr Support Calc for PTF-PVP-H20042 & H30042 24590-PTF-PHC-PVP-00046 Engr Support Calc for PTF-PVP-H20043 24590-PTF-PHC-PVP-00047 Engr Support Calc for PTF-PVP-H20079 24590-PTF-PHC-PVP-00048 Engr Support Calc for PTF-PVP-H20044 24590-PTF-PHC-PVP-00049 Engr Support Calc for PTF-PVP-H20045 & H20046 24590-PTF-PHC-PVP-00050 Engr Support Calc for PTF-PVP-H20048 24590-PTF-PHC-FEP-00055 Engr Support Calc for PTF-FEP-H20099 & H20077	Reviewer: Larry Nicholson Design Codes & Standards, Design Inputs, and Pipe Support Analysis Techniques are appropriate. Piping Support analyses/calculations examined and provided margins for: displacements/deflections; stresses; welds; localized effects; and standard hardware components (e.g., U-bolts). All analyses/calculations were linear elastic, considered weight + thermal + seismic, and were for relatively low-temperatures. All analyses reviewed were complete/comprehensive and adhered to good QA practices (e.g., V&V, statement of Codes & Standards employed, checking/signature/approvals, and document control).	Observations

C.7 Piping and Piping Supports Review Comments			
No.	**Document**	**Comments and Observations**	**Action By and Respo**
	24590-PTF-PHC-PVP-00061 Engr Support Calc for PTF-PVP-H20050 24590-PTF-PHC-PVP-00062 Engr Support Calc for PTF-PVP-H20051		
PS-12	Pipe Stress Analysis	Reviewer: Larry Nicholson 24590-PTF-PGC-FEP-00015, Pipe Stress Analysis for FEP System uses SRSS to combine modes. Closely space modes are not considered as required in Design Criteria 24590-WTP-DC-01-001 and is industry practice. This is not conservative and needs to be reviewed for impact. An E-mail from John Minichiello to Mark Summers on 4/19/2006 says the 10% grouping method is the default now for ME101. The N8 manual states that SRSS is the default and since no card is included in the deck to request the 10% grouping method the default would be used. I was provided a copy of the ME101 output for this calculation and in the preprocessor for load case SEISDB it is stated that the COEFFICIENT is SRSS this should say CS4 for the 10% grouping method. In addition if the grouping method is used the group number should be listed in the MODAL ACCELERATIONS. They are not. Thus there is nothing in the output to indicate that anything other than SRSS was used. Recommend reviewing all calculations for impact of using SRSS modal combinations This may be a generic problem.	BNI

C.8 Equipment Review Comments			
No.	**Document**	**Comments and Observations**	**Action By and Response[1]**
EQ-1	Julyk, John L., USACE Equipment Design and Seismic Qualification Requirements Review Presentation, May 8, 2006 24590-WTP-3PS-SS90-T0001, Rev. 1, Engineering Specification for Seismic Qualification of SC I/II Equipment & Tanks 24590-WTP-3PS-JQ06-T0003, Rev. 4, Engineering Specification for Seismic Qualification of SC I Control and Electrical Systems and Components 24590-WTP-LAW-3PS-M000-T0002, Rev. 0, Engineering Specification for Master Slave Manipulators for PTF, HLW, LAW & LAB 24590-QL-POA-MJW0-00003-09-00003, Rev. G, Model RE-T Telemanipulator, Seismic Analysis 24590-WTP-3PS-ADDC-T0002, Rev. 1, Engineering Specification for HLW/PT System Transfer Hatches, Hatch Drives, Hatch Pushrod Assemblies, and Floor Penetration Liner 24590-QL-POA-ADDH-00003-08, Rev. B, Seismic Qualification Report, Hatch	Reviewer: Steve Short General Approach and Mechanical Handling Systems Equipment seismic response differs from building response in several areas: (1) input motion is that of the attachment point in the building or on another equipment item rather than the ground motion; (2) equipment may be supported at multiple points, each with different seismic input. As a result, this equipment is subject to both seismic inertial forces and relative displacements between support points; and (3) equipment has less redundancy and inelastic energy absorption capacity than buildings such that F_μ values for equipment are lower than that for buildings. All of these factors must be incorporated into seismic design of equipment. The approach for equipment seismic qualification is for the mechanical or electrical responsible engineer to request the BNI civil-structural group to provide in-structure response spectra (ISRS) at equipment attachment points to account for building input to the equipment. The responsible engineer then combines these ISRS with functional and safety requirements, including seismic criteria into material requisition documents for vendor/suppliers. Seismic qualification calculations or test plans are prepared by vendors and reviewed by the BNI civil structural group. The samples of mechanical handling equipment reviewed appear to have adequate margin for the revised ground motion. Inconsistencies in seismic criteria and implementation of the criteria by vendors have been observed.	Observation

[1] "Observation" is for information only - a response is not required.

C.8 Equipment Review Comments			
No.	**Document**	**Comments and Observations**	**Action By and Response[1]**
	24590-HLW-3PS-MX00-T0001, Rev. 2 Engineering Specification for QL Shielded Personnel Access Doors 24590-QL-POA-ADDB-00001-09-33, Rev. F, Design File, Shielded Personnel Access Doors, Bechtel Hanford, River Protection Project-Waste Treatment Plant		
EQ-2	24590-WTP-3PS-SS90-T0001, Rev. 1, Engineering Specification for Seismic Qualification of SC I/II Equipment & Tanks	Reviewer: Steve Short The engineering specification for tanks gives F_μ values in the 1.25 to 1.5 range that are consistent with DOE-STD-1020. It is noted that the F_μ value for tank seismic response will be 1.15 for Limit State C (compatible with PC-3 or SC-II) in ASCE 43-05. It is anticipated that ASCE 43 will replace DOE-STD-1020 in the near future. This comment is for information only and has no impact on the current design process.	Observation
EQ-3	24590-WTP-3PS-SS90-T0001, Rev. 1, Engineering Specification for Seismic Qualification of SC I/II Equipment & Tanks 4590-WTP-3PS-JQ06-T0003, Rev. 4, Engineering Specification for Seismic Qualification of SC I Control and Electrical Systems and Components	Reviewer: Steve Short The requirements for seismic qualification of equipment with regard to input from supported structure response, effects of multiple supports, and reduced inelastic energy absorption capacity are recognized in seismic criteria for DOE-STD-1020 and from the BNI Seismic Analysis and Design Criteria for the WTP project. However, all of these requirements have not been carried down into the seismic qualification engineering specifications for equipment and tanks and for control and electrical systems. These general specifications do not include any considerations for seismic anchor motion (i.e., relative displacements between multiple support points) as required in Section 2.4.1 of DOE-STD-1020 and in the BNI SADC.	BNI
EQ-4	Engineering Specifications for Seismic Qualification	Reviewer: Steve Short The engineering specifications permit four seismic qualification methods: (1) analysis; (2) test; (3) combined analysis and test; and (4) past	BNI

C.8 Equipment Review Comments			
No.	**Document**	**Comments and Observations**	**Action By and Response[1]**
		qualification in a nuclear installation. The specifications provide detailed requirements for past qualification by test. Similar detailed requirements are needed for past qualification by analysis.	
EQ-5	24590-WTP-LAW-3PS-M000-T0002, Rev. 0, Engineering Specification for Master Slave Manipulators	Reviewer: Steve Short The master slave manipulator seismic qualification calculation (conducted by an outside vendor) appears to be reasonable and complete. However, implementation of the seismic criteria is confusing. The criteria in the beginning of the calculation states load combinations and capacity levels in accordance with AISC N690 the allowable stress criteria for nuclear structures. However, the capacity used for all stress conditions is conservatively taken as the lower value applicable for shear (1.4 times allowable stress). For other stress conditions, the capacity can be 15% greater (1.6 times allowable stress). The load combination in the calculation states that the sum of seismic and non-seismic loads all divided by 1.4 shall be equal to the allowable stress capacity. However, these load combinations and capacity levels are never actually used in the calculation. Instead seismic and non-seismic loads are added together unreduced and the resulting stress is compared to strength design capacities, in most cases. For anchor bolts, allowable stress capacities are used and this is conservative. It may be noted that there is a version of AISC N690 that follows LRFD methodology now. The results of this calculation seem to indicate that the manipulators have adequate seismic capacity but the seismic criteria of DOE-STD-1020, BNI SADC, the general project equipment specifications for seismic qualification, and the early part of the manipulator calculation were not followed. Evaluation of the manipulators for the revised ground motion was not in the available documents for review.	BNI
EQ-6	24590-QL-POA-ADDB-00001-09-33, Rev. F, Design File, Shielded Personnel Access Doors, Bechtel Hanford, River Protection Project-Waste Treatment Plant	Reviewer: Steve Short The seismic qualification of shielded process doors (conducted by an outside vendor) used an equivalent static seismic load of 1.5g and compared the resulting response to allowable stress limits for the steel seismic response. The criteria states at one location that a 1.33 increase	Observation

C.8 Equipment Review Comments			
No.	**Document**	**Comments and Observations**	**Action By and Response[1]**
		in capacity for load combinations including seismic is to be used, but I did not observe that this increase was used. As an SC-I equipment item, the doors can be designed in accordance with AISC N690 such that for steel seismic response the allowable capacities may be increased by 1.6 (1.4 for shear). Hence, the steel seismic evaluation is conservative. Anchorage to concrete is in accordance with ACI 349-01 that is appropriate for SC-I. BNI used the vendor calculations to re-evaluate the doors for the revised ground motion based on recently computed ISRS. The actual seismic input from the RGM is much lower than the 1.5g input used in the seismic qualification calculation. However, the calculation considered ground motion only in one direction and not the simultaneous occurrence of three components of ground motion. BNI adjusted the calculations for three components of input and concluded that the doors have adequate seismic capacity. This conclusion was based on the use of allowable stress capacities of the steel members and, as discussed above, this is conservative.	
EQ-7	24590-QL-POA-ADDH-00003-08, Rev. B, Seismic Qualification Report, Hatch	Reviewer: Steve Short The hatch seismic qualification calculation (conducted by an outside vendor) appears to be conservative and complete. However, it should be noted that this evaluation was performed in an extremely conservative manner. A dynamic analysis of the hatch was performed using a detailed ALGOR finite element model. From this analysis, it was demonstrated that the hatch is rigid with a fundamental frequency of about 39 Hz. As a result, the calculation concludes correctly that a response spectrum analysis is not needed and seismic qualification can be accomplished by static analysis. However, the resulting static analysis used the peak of the ISRS rather than the zero period acceleration (ZPA) values from the ISRS. Seismic inertial loads were based on 0.85g horizontally and 0.66g vertically. The appropriate ZPA levels for seismic inertial loads are about 0.25g horizontally and 0.20g vertically. As a result, conservative margin on the order of a factor of 3.3 is introduced by use of the peak spectral acceleration. There is additional conservatism in that allowable stresses are used for capacity rather than the increased allowables permitted by AISC N690. Additional conservative margin on the order of a factor of	Observation

C.8 Equipment Review Comments			
No.	**Document**	**Comments and Observations**	**Action By and Response[1]**
		1.6 is introduced by use of allowable stresses for capacity of these SC-I equipment items. Further conservatism is introduced in finite element mesh size compensation. A mesh size of 0.75 inches is used where it is determined that a mesh size of 0.35 inches is needed to capture peak stresses. The estimated difference is stress is that a factor of 1.8 is needed to compensate for the more coarse mesh size. A factor of 2.5 is used in the calculations as mesh size compensation introducing additional conservative margin on the order of 2.5/1.8 = 1.4. Overall, the conservative margin accumulates to 3.3*1.6*1.4 or a factor of about 7.4. The design is further conservative in that peak stresses are used for the seismic demand. These peak stresses are easily reduced by local yielding that would have no adverse impact on hatch function.	
EQ-8	24590-WTP-3PS-SS90-T0001, Engineering Specification for Seismic Qualification of Seismic Category I/II Equipment and Tanks 24590-WTP-3PS-JQ06-T0003: Seismic Qualification of Seismic Category I Control and Electrical Systems and Components 24590-QL-MRA-EK00-00001-S0004, Material Requisition Supplement for 480 V Secondary Unit Substation Load Centers (ITS) 24590-WTP-3PS-EKL2-T0002; Engineering Specification for 480 V Secondary Unit Substation Load Centers (ITS) 24590-WTP-3PS-ESM2-T0001; Engineering Specification for 4.16 kV Switchgear (ITS)	Reviewer: Jim Wilcoski The required Response Spectrum, RRS is obtained by enveloping and smoothing the ISRS ..... and multiplying the spectral acceleration value by 1.4. The 1.4 value appears to be an equipment capacity factor for qualification by test or TES that provides the margin to obtain the required confidence level of performance (see ASCE/SEI 43-05, Section 8.3.2). A 1.4 multiplier is also present in 24590-WTP-DC-ST-04-001, Rev 3, Seismic Analysis and Design Criteria, Section 7.2.2.4. The definition or purpose for this factor should be defined wherever it is used (design criteria, design specifications and material requisition) so that it is not confused with the interim increase in ground.	BNI

C.8 Equipment Review Comments			
No.	**Document**	**Comments and Observations**	**Action By and Response[1]**
EQ-9	24590-WTP-3PS-SS90-T0001, Engineering Specification for Seismic Qualification of Seismic Category I/II Equipment and Tanks	Reviewer: Jim Wilcoski Page 10, Sections 7.2, 7.3 and 7.4. How are support reactions; acceleration at nozzle location used to develop response spectra; and displacements at nozzle locations to be measured in testing and reported. Support reactions at welded connections may be difficult to measure, while reactions at bolted connections could be measured with strain bolts. Accelerations should be measured at nozzles in order to develop a basis for nozzle response spectra. Absolute seismic displacement at nozzles could be measured directly in tests relative to a fixed location off the shake table, or be based on integrated accelerations. If absolute displacements are needed this should be stated in the document as such. If on the other hand displacements relative to the equipment supports are needed (i.e., equipment deformations at nozzles), this should also be stated clearly so that proper instrumentation can be installed for the test. In this case, displacements should be measured directly relative to a rigid test fixture anchored to the shake table.	BNI
EQ-10	24590-WTP-3PS-JQ06-T0003: Seismic Qualification of Seismic Category I Control and Electrical Systems and Components	Reviewer: Jim Wilcoski Page 1, Section 1.2, 1st para and Page 2, 2nd para. The operating basis earthquake (OBE) requirement from IEEE Std 344 does not require to be considered for the WTP, per Appendix C: Implementing Standards for IEEE 344 (pages C-22-1 and C-22-2). Under these requirements, a relatively small earthquake during the operational time frame of the WTP could damage components without a seismic safety function (SC-I or SC-II) that could prevent further operation of the WTP. If these components were located in areas inaccessible to repair (e.g., black cells), operations of the WTP could be permanently stopped. Could all components within the black cells currently classified as SC-III and SC-IV be changed to SC-II, so that a high level of confidence of continued operation could be achieved. Alternatively, could a lower level of seismic motions be defined, to which all SC-III and SC-IV components within the black cells	BNI

C.8 Equipment Review Comments			
No.	**Document**	**Comments and Observations**	**Action By and Response[1]**
		must be qualified? Even with this approach, the WTP may need to be shut down for a few years while repairs to SC-III and SC-IV components outside the black cells are inspected and repaired.	
EQ-11	24590-WTP-3PS-JQ06-T0003: Seismic Qualification of Seismic Category I Control and Electrical Systems and Components	Reviewer: Jim Wilcoski Page 9, section 7.2, does this section require that all support reactions be measure during testing. If so, the methods used to measure reactions should be developed by the supplier and provided to the buyer for review and approval prior to testing. Some description of what these reactions would be useful, as they may guide the proper representation of support conditions in tests. For example, it should be relatively simple to use strain bolts for bolted connections at supports, but measuring reactions at welded connections could be more difficult (e.g., may require extensive strain gauges). However, strain gauges may not properly measure reactions, when strains at the surface do not represent strains through the depth of the material. In some cases, strain gauges could be installed some distance from the supports where strains may be more uniform. In other cases, reactions could be more accurately determined by appropriate instrumentation and analytical modeling.	BNI
EQ-12	24590-QL-POA-MEE0-00001-05-00001; Seismic Design Report for the Off-Gas HEPA Pre-Heaters	Reviewer: Jim Wilcoski Page 4, 3rd and 4th paragraph, references 24590-WTP-3PS-MV00-T0002. This reference is titled “Seismic Qualification Criteria for Pressure Vessels,” and pre-heaters are outside the Scope (Section 1) of 24590-WTP-3PS-MV00-T0002.	BNI
EQ-13	24590-QL-POA-MEE0-00001-05-00001; Seismic Design Report for the Off-Gas HEPA Pre-Heaters	Reviewer: Jim Wilcoski Page 8, Section 3.1.6 shows seismic acceleration calculations based on the zero period spectral acceleration values from ISRS in Appendix M. A review of the revised ground motion ISRS shows that these zero period accelerations have increased to approximately 0.3 g, 0.28 g and 0.25 g for the E-W, N-S and Vertical directions at the Figures 46, 47 and 48 ISRS location respectively, and even further to 0.4 g and 0.3 g for the E-W and N-S directions at the Figures 73 and 74 location. The values from	BNI

C.8 Equipment Review Comments			
No.	**Document**	**Comments and Observations**	**Action By and Response[1]**
		Revision 0C of these Figures were used, but the values increased in Revision 0D for the revised ground motions. The calculations and their use in later seismic contributions should be revised based on the current ISRS values. (The peak-combined stress that includes seismic is 25,351 psi (see page 7, 1st paragraph). This stress will increase when revised ground motions are accounted for, resulting in a reduction or perhaps elimination of the margin show at the center of page 10. The increased the loads and stresses in the brackets and supporting bolts reported in Appendix L should also be confirmed.)	
EQ-14	24590-QL-POA-MEE0-00001-05-00001; Seismic Design Report for the Off-Gas HEPA Pre-Heaters	Reviewer: Jim Wilcoski Page 10, presents nozzle forces and moments for the support reactions at the liner, taken from Appendix F, page F5. The sign convention has apparently changed for these between page 10 and F5 - confirm that the forces and moments are being used properly.	BNI
EQ-15	24590-QL-POA-MEE0-00001-05-00001; Seismic Design Report for the Off-Gas HEPA Pre-Heaters	Reviewer: Jim Wilcoski Page 11, Section 3.1.10 presents the stresses due to normal operating loads. It appears that the allowable stress in the stainless steel plate under these loads should not have exceeded 16.7 ksi, while this section shows a 1.5 factor for combining with seismic accelerations. Why is the allowable load defined based on a seismic related increase (i.e., 1.5 factor), when this section applies only to normal operating loads with no seismic accelerations. Should the allowable margin rather be 16.7 ksi / 15.739 ksi = 1.06?	BNI
EQ-16	John L. Julyk et al., "USACE Equipment Design and Seismic Qualification Requirements Review," Presentation, May 8, 2006.	Reviewer: James Johnson There were a very limited number of calculations to review due to the hold on work. In addition, in some cases, the work presented was intended to be representative of the process rather than a focus on the specifics. BNI presented a cross-section of mechanical, electrical, C & I, HVAC, and maintenance activities on-going. In most cases, there is significant interaction between disciplines required	Observation

C.8 Equipment Review Comments			
No.	**Document**	**Comments and Observations**	**Action By and Response[1]**
		to achieve seismic design and qualification of SSCs. Systems design defines performance requirements of systems and their components. Mechanical, electrical, C & I, HVAC, fire protection, and others design the functional systems and their support systems. IMS apparently coordinates the various activities. With respect to seismic design and qualification, CS & A provides: in-structure response spectra for analysis and testing of systems and components; reviews Material Requisition (with regard to seismic requirements); reviews Vendor submittals (proposal and final design package) for acceptable seismic analysis and testing and compliance with SADC and SRD; receives seismic anchorage loads for inventorying and verification that embeds or other anchorage systems are adequate; and receives other loading conditions to be included in the structural element design. The number of interactions and the stages of design at which interaction is required amongst all of the above-mentioned disciplines requires close coordination for project success. CS & A has extensive responsibilities in the seismic design and qualification process. An effective team approach to implement the review and approval of the various steps will be needed as procurement is re-started. A seminar or workshop by knowledgeable Bechtel engineers or consultants from the CS & A organization describing the seismic analysis, design, and qualification procedures for the WTP would be helpful. The audience would Responsible Engineers for structures, systems, and components (SSCs). The workshop could include items, such as qualification procedures, dynamic analysis methods, equivalent static analysis methods, conservatisms and unconservatisms in various approaches, etc. such a workshop would provide a common base from which Responsible Engineers could develop Material Requisitions, evaluate proposals, evaluate vendor submittals, etc. the workshop would be one step towards assuring smooth operations and interactions with the CS & A as discussed above.	
EQ-17		Reviewer: James Johnson	BNI

C.8 Equipment Review Comments			
No.	Document	Comments and Observations	Action By and Response[1]
		Large in-wall and in-slab components, such as doors, hatches, other large penetrations, should be designed and qualified to a RGM seismic environment that includes ISRS at the support locations and relative motions of the wall or slab. For inclusions that are stiff relative to the supporting wall or slab, a combined analysis including the wall or slab and the relatively rigid inclusion may be required to adequately account for the interaction between the two. For inclusions that are relatively flexible relative to the supporting wall or slab, the displacements at the cutout or penetration should be superimposed on the inclusion to account for relative motions. For small inclusions, relative motions of wall or slab are likely not of concern.	
EQ-18		Reviewer: James Johnson For all systems, components, and commodities, the stiffness of the supporting sub-structure should be taken into account. Examples include walls, slabs, structural steel frames, vessels, equipment in which devices are located, equipment racks, piping or HVAC for in-line components, etc. Accounting for supporting sub-structure is required for all items designed or qualified by analysis, test, or a combination of the two. The design/qualification can be performed by considering a combined model of the supporting system and the component; or separate dynamic analyses can be performed for the supporting system (including interaction effects of all supported items) and in-system response spectra generated for qualification of the items.	BNI
EQ-19		Reviewer: James Johnson Philosophically, items in the Black Cells should be categorized SC-1 or SC-2 given that entry and inspection of any items is not possible after an earthquake of any size occurs. The occurrence of an earthquake of size less than the RGM will require some verification of the lack of damage to all systems – process and safety. Inaccessible components will need to be considered as undamaged in this case – one reasonable way of providing high confidence of no damage is by designing these	BNI

C.8 Equipment Review Comments			
No.	**Document**	**Comments and Observations**	**Action By and Response**[1]
		components to SC-2 requirements.	
EQ-20	24590-WTP-SRD-ESH-01-001-02, Rev. 3h, "River Protection Project – Waste Treatment Plant Safety Requirements Document," Vol. II, Appendix C, Rev 3m.	Reviewer: James Johnson SRD specifies Tailoring of applicable standards. For IEEE 344, SRD states "The Scope, section 1.0, of IEEE 344 applies to equipment that needs to function during and after an SSE for a Nuclear Power Generating Station. For RPP-WTP the equipment that needs to function during and after a design basis earthquake is SDC/SDS/SC/SS equipment which must be qualified to SC-1." This requirement needs to be specified in Material Requisitions and accompanying documents. It seems that the requirement of function during the earthquake may be too restrictive in some cases and exceptions due to timing required to switch from normal power to emergency power may be taken into consideration. Also, this provision of operating during the shaking may pertain to after shocks that could be large and subject the then operating systems to additional loading conditions. The requirement of the tailored IEEE 344 to require all SDC/SDS/SC/SS equipment to be tested to remain functional during and after the earthquake should be verified as necessary. If so verified, all Material Requisitions should highlight this requirement to the bidders.	BNI
EQ-21	1. John L. Julyk et al., "USACE Equipment Design and Seismic Qualification Requirements Review," Presentation, May 8, 2006. 2. 24590-QC-HCH-W000-00011-00269, Rev. 00B, "River Protection Project, Waste Treatment Plant, Calculation for HLW ADS Pump Stress Analysis," 11/08/04.	Reviewer: James Johnson The approach for the Air Displacement Slurry (ADS) Pumps was reviewed based on References 1, 2, and other references on the same topic in conjunction with a meeting with the Responsible Engineer (Michael Seed). M. Seed conveyed that the design process was being re-evaluated and modified to better address a number of issues – seismic being one. In particular, the seismic analysis, design, and qualification process was being re-considered. The ADS Pumps are supported from the HFP tank and the decision is to be made as to the best method of generating the seismic input for the ADS Pump design. The ADS Pump is SC-II due to it's potential affect on SC-I components. It need not function during or after the earthquake, but it must not cause failure of the	Observation

C.8 Equipment Review Comments			
No.	**Document**	**Comments and Observations**	**Action By and Response[1]**
		SC-I components. Issues to be considered are: development of and integrated model of the tank and ADS Pump; if not an integrated model, develop in-structure response spectra at ADS Pump support locations and relative displacements if necessary for ADS Pump stress analysis; in this latter case, specification of supporting structure (tank) conditions for inclusion in the Vendor's dynamic analysis of the Pump; etc. Reference 2 contained the ground motion response spectra as input for the Vendor rather than ISRS at the appropriate Pump support location. The Vendor used these spectra, but incorrectly for the stress analysis. It is concluded that the Responsible Engineer is approaching this re-evaluation in the correct manner. Comment EQ-18 above is being followed in the re-evaluation.	
EQ-22	1. John L. Julyk et al., "USACE Equipment Design and Seismic Qualification Requirements Review," Presentation, May 8, 2006. 2. "Design Evaluation for the Bulges Process Shielded Containment and Operating Platform Structure," 24590-QL-POA-PY33-00002-13-00001, Rev. 00A, with handwritten comments 3/20/06.	Reviewer: James Johnson A limited review of the PTF bulge denoted CXP-BULGE-00004 was performed based on Refs. 1 and 2 and other related documents available on the ftp site. Reference 2 is the structural design evaluation submitted by the Vendor and included handwritten comments by the BNI reviewer, presumably the Responsible Engineer with input on seismic from CS&A. The handwritten comments were generally very appropriate and needed to better understand the structural analysis. The containment structure and the operating platform structure are treated as independent structures for the seismic analysis and design. The BNI reviewer requests the Vendor to justify this assumption. The Vendor performed equivalent static analysis to develop the stresses for design of the containment tub. For reasons of expediency and convenience, the Vendor limited the number of load combinations by making conservative assumptions with respect to spatial combinations, i.e., instead of the 100-40-40 rule or SRSS of directional effects, the Vendor applied absolute summation of directional effects. This could be unnecessarily conservative if it dictates the design. The Vendor performed dynamic response spectrum analysis to develop	Observation

C.8 Equipment Review Comments

No.	Document	Comments and Observations	Action By and Response[1]
		the stresses for design of the operating platform structure. It appears that SRSS of modal responses was used for modal combination rules. The BNI reviewer requested appropriate additional information and clarification.	
EQ-23	24590-PTF-SSC-S15T-00051, Rev D - Standard Seismic Category I & II HVAC Ducts and Duct Supports for PT Bldg 24590-PTF-SSC-S15T-00014, Rev A - Design of Pipe Rack Framing Below El. 28'-0" 24590-QL-POA-HCTH-00002- 03-00013-00D, submittal "Seismic Analysis of the HLW HEH and RWH Cask Lidding Machines" 24590-WTP-3PS-MJW0-T0001, Rev 2, Engineering Specification for HLW And PTF Cast Lidding Machines 24590-QL-POA-HDYR-00001- 09-00008-00A, submittal, "Through Wall Drive Load Calculation" 24590- WTP-M0C-M10T-00005, Rev C - HLW, LAW, PT, and LAB Encast Liner Weight, Stresses and Thermal Expansion	Reviewer: Mark Summers HVAC and Mechanical Handling Equipment a. The available HVAC documents were based on the original ground motion. The calculations were an in-house product. For the original ground motion the ducts and duct supports calculation is conservative using an enveloped ISRS from the base up to El. 98 and below, used 3% damping verse 5%, and D/C ratios were less than 0.90. The pipe rack framing calculation had demand/capacity ratios less than 0.75, based on peak acceleration with and 4% damping verse 7%. b. The cask lidding submittal ground motion is based on the ISC. Memorandum CCN: 134568 dated January 10, 2006 cites the ISC bounds the RGM for the cask lidding equipment. The submittal has been returned for correction. BNI's review identified areas of design that require additional effort by the vender. Based on BNI's comments, BNI performed a reasonable review of the vendors submittal. c. The swabbing equipment through wall drive submittal ground motion is based on the ISC. Memorandum CCN: 134569 dated January 11, 2006 cites the ISC bounds the RGM for the Swab and Monitoring System Equipment Purchase Order. The submittal has been returned for correction. BNI's review identified areas of design that require additional effort by the vender. Based on BNI's comments, BNI performed a reasonable review of the vendors submittal. d. The liner calculation is an in-house product based on revised ground motion. The calculation does not include the increased allowable	Observation

C.8 Equipment Review Comments			
No.	**Document**	**Comments and Observations**	**Action By and Response[1]**
		stresses outlined in the SADC for seismic loadings resulting in additional capacity.	

PROGRESS REPORT

Independent Review of Implementation of Revised Seismic Design Criteria
Hanford Waste Treatment and Immobilization Plant (WTP)

Revision D

Appendix D

List of Conservatisms and Actions

PRELIMINARY EVALUATION OF POTENTIAL CONSERVATIVISM IN ANALYSIS AND/OR DESIGN

Analysis and Design Conservatism

108127

Table 1. – Soil Structure Interaction Analysis

Number	Item	Remarks	Value
1.1	Perform new SSI analysis for new ground motion and latest building configuration	Because the new response spectra are increased and the PT and HLW buildings are near design completion, new seismic loads and equipment response spectra will be needed. This will be done by incorporating the latest designs into the SASSI model.	A
1.2	Determine SSI parameters consistent with new ground motion. This may increase the soil dampening with resulting load reduction.	Since the seismic ground motion has increased, it will result in new strained soil properties to be used in the SASSI model. The analysis should include any recent changes in soil properties. The higher seismic ground motion should result in increased soil damping and potentially lower design loads.	A
1.3	Use DOE 1020 damping for Response Level 2 in the SSI analysis.	DOE 1020 allows the use of material damping higher than used in the current design calculations when developing building loads. It is suggested that the project use the higher damping values for the SSI analysis.	A
1.4	Remove conservatism associated with the development of enveloping static seismic loads from the bubble sheets.	In previous project calculations summarizing the results of the SSI analysis conservative factors have been applied to the floor area loading developed from the SSI "Bubble Sheets". It is suggested that this conservatism is no longer necessary since the building design is now well delineated, with few changes anticipated.	A
1.5	Include ground motion incoherence in the existing SSI analysis.	Ground motion incoherence as it affects large mat foundations, such as at the PT building if accounted for in the SSI analysis will result in lower SSI loads and should be included. If SASSI is used for this evaluation it will have to be verified.	B
1.6	Allow reduction of peak of the in-structure response spectra when broadening as permitted by ASCE 4.	When smoothing and broadening the envelope in-structure floor response spectra ASCE 4 allows a reduction of the peak of the spectra by 15%. Since equipment qualification will be a critical issue this allowable reduction should be implemented.	A

Value A: Non-controversial changes
Value B: Additional justification necessary for acceptability
Value C: Very controversial changes

PRELIMINARY EVALUATION OF POTENTIAL CONSERVATIVISM IN ANALYSIS AND/OR DESIGN

Analysis and Design Conservatism

108127

Table 2. - Building Analysis and Design

Number	Item	Remarks	Value
2.1	Perform a static analysis of the building using the new seismic loads from the SSI analysis.	The new SSI will provide new seismic loads for the building structures. These new SSI loads will be stripped of unnecessary conservatism as discussed above. Using the latest building configuration develop new element seismic loads that will be used in load combination with other design loads.	A
2.2	Member design using controlling load case	The results of the static analysis will be included in the design basis load combinations and design loads developed. It is suggested that the controlling design load combination be used for design rather the envelope load components as was done in the current design calculation in order to minimize design loads.	A
2.3	Reduce OOP moments and shears to face of walls or d/2 from wall respectively as allowed by code.	It is acceptable to reduce design moments to the face of the supporting walls and for in-plane shears to a distance of d/2 from the face of the supporting wall. Including this feature in developing the design loads will result in reduced member end loads. Note, when evaluating already designed structural elements it is only necessary to compare the previous design loads and D/C ratios to determine if the new load would be within code acceptable criteria if this would result in more efficient way to show qualification.	A
2.4	Realistic treatment of thermal gradient loads by reduction associated with cracking.	When thermal gradients result in bending moments that crack the concrete, it is acceptable to redistribute the moment considering a reduced member stiffness.	B
2.5	Minimize conservatism associated with accidental torsional loading applied to shear walls.	In the existing design calculations the effect of torsions load on building walls was uniformly applied to all walls. It is suggested that the load be applied as a function of the distance from the cg of the wall system. Rather difficult to apply.	B
2.6	Allow for Fμ greater than 1.0 for in-plane shear, bending and out-of-plane bending in accordance with DOE 1020	In accordance with DOE 1020 it is suggested that the project allow the use of Fμ factors. It is necessary to show that the building retains confinement during and after a seismic event if Fμ factors are used. Therefore, it is necessary to demonstrate that the safety class HVAC system used to maintain negative pressure be qualified to the new seismic ground motion and have sufficient capacity for the calculated concreted cracking.	C
2.7	Use f'c based on verified concrete test properties from site test cylinders	In the event that a specific area is not able to be qualified using the specified design concrete strength, then a revised design strength based on ACI methodology can be	C

Value A: Non-controversial changes
Value B: Additional justification necessary for acceptability
Value C: Very controversial changes

PRELIMINARY EVALUATION OF POTENTIAL CONSERVATIVISM IN ANALYSIS AND/OR DESIGN
Analysis and Design Conservatism

108127

Table 2. - Building Analysis and Design

Number	Item	Remarks	Value
		calculated and used. This process shall only be applied to constructed structures.	
2.8	Consider response spectra analysis for seismic design	As an alternative to the static seismic analysis of the building, the project may consider a response spectra methodology. This can be considered if the new seismic loads do not result in a design that meets code acceptance criteria, but are within approximately 10% of the acceptance criteria. Very difficult to apply in complex structures.	B
2.9	Reduce conservatism in below grade wall design by using SASSI wall pressures.	Use the SASSI wall pressures for design of below grade walls for lateral seismic soil pressure.	A
2.10	Review and revise existing assumed design commodity floor loads.	With design nearing completion review and revise the assumed building loads, for example the overpack has been removed from the building resulting in a significant change in dead load.	A

Table 3. – In-Structure Response Spectra

Number	Item	Remarks	Value
3.1	Perform new SSI analysis for new ground motion and latest building configuration and *generate new in-structure response spectra.*	This is the same as item 1.1 in Building qualification and includes the removal on conservatism as specified in Items 1.2 through 1.5.	A
3.2	Remove conservatism from existing spectra, both PT and HLW have vertical amplification factors, and one has a horizontal amplification factor	This is an action that can be used for the interim review of equipment qualification.	A
3.3	Reduce peaks of existing response spectra in accordance with ASCE 4.	This is an action that can be used for the interim review of equipment qualification.	A

Value A: Non-controversial changes
Value B: Additional justification necessary for acceptability
Value C: Very controversial changes

PRELIMINARY EVALUATION OF POTENTIAL CONSERVATIVISM IN ANALYSIS AND/OR DESIGN

Analysis and Design Conservatism

108127

Table 4. - Equipment Qualification			
Number	**Item**	**Remarks**	**Value**
4.1	Reduce seismic demand on equipment by F_μ as permitted in DOE-STD-1020 and ASCE-43	DOE-STD-1020 permits a seismic demand reduction for ductile behavior F_μ but does not specify the value of the ductility reduction factor F_μ. ASCE-43 specifies F_μ values for use in equipment seismic qualification. Passive, ductile, not pressure boundary = limit state A → F_μ = 1.50 to 2.00 Passive, ductile = limit state B → F_μ = 1.25 to 1.50 Active post-DBE with operator set = Limit state B → F_μ = 1.00 to 1.25 Active = Limit state D → F_μ = 1.00	B
4.2	Scope of SC I and SC II	Review basis for seismic classifications SC I and SC II for possible downgrades to SC III within safety basis	A
4.3	Experience data DOE-EH-0545	Evaluate installed or procured equipment based on earthquake experience data for the higher floor spectra. The technique is permitted for new equipment in ASCE-43. This recommendation does not apply to piping systems, electrical or electronic equipment. It is also limited to building locations where the spectrum is below the DOE-EH-0545 applicability spectrum (1.2g peak). This option would prevent re-qualification of some mechanical equipment (valves, pumps, fans) and conduit and cable trays.	C
4.4	Pipe damping of 5%	Use 5% damped spectra in piping analysis rather than CC N-411 which has a lower damping at high frequency. When the Code Case was incorporated into the current ASME III it was replaced by a constant 5% damping. The 5% damping value for piping is also in ASCE-43. Because much of the piping response is governed by flexible modes, a drop in high frequency damping will have limited benefit.	A

Value A: Non-controversial changes
Value B: Additional justification necessary for acceptability
Value C: Very controversial changes

PRELIMINARY EVALUATION OF POTENTIAL CONSERVATIVISM IN ANALYSIS AND/OR DESIGN
Analysis and Design Conservatism

108127

Table 4. - Equipment Qualification			
Number	**Item**	**Remarks**	**Value**
4.5	Pipe stress limit of min(3S;2S_Y)	The current allowable stress limit of 1.3S for SCI and SCII is based on ASME B31.3. This value is decades old, and does not reflect the updates in seismic design rules introduced in ASME III. Currently, a standard ASME B31E is being developed by ASME B31 Mechanical Design Committee based on an allowable stress of min{3S;2S_Y}. This is consistent with ASME III Class 2 and 3 allowable stresses and Markl's fatigue failure rule. However, this higher allowable should not be used in conjunction with $F_\mu > 1.00$ since it would account twice for ductility effects. The adoption of $F_\mu > 1$ (3.1 above) is preferable, since it reduces demand, consistent with DOE-STD-1020 and ASCE-43.	B

Value A: Non-controversial changes
Value B: Additional justification necessary for acceptability
Value C: Very controversial changes

108127

WALL PIERS

Fig	Type	Elev.	Cut Length	1x1 GTS Conservative					Convergence Achieved (5%)					Design Load Within 10% of Converged Result Result?				
				Pn	Vn	Mtr	Vtr	Mn	Pn	Vn	Mtr	Vtr	Mn	Pn	Vn	Mtr	Vtr	Mn
22	NS Wall 17.7, Small Pier Two Wide	0	13	N	N			Y	Y	Y			N	Y	Y			na
23	EW Wall 7, Small Pier	0	29	N	Y			Y	Y	Y			6%	Y	na			na
24	EW Wall 7, Small Pier	0	17	Y	Y	Y		Y	Y	Y	Y		N	na	na	na		na
25	EW Wall J4, One Element Pier	3	10	N	N	N		Y	Y	Y	Y		N	N	Y	N		na
26	EW Wall J4, Small Pier	3	26	N	Y	N	N	Y	Y	Y	Y	N	Y	Y	na	N	N	na
27	NS Wall 8.6, Small Pier	3	6	Y	Y	N	N	Y	Y	Y	Y	Y	Y	na	na	Y	Y	na
28	NS Wall 8.6, Small Pier	3	7	Y	Y	N	N	Y	Y	Y	Y	N	Y	na	na	N	N	na
29	NS Wall 8.6, Large Pier	3	50	N	Y	N	N	N	Y	Y	Y	N	Y	Y	na	N	N	N
				Y-50%	Y-67%	Y-17%	Y-0%	Y-88%	Y-100%	Y-100%	Y-100%	Y-25%	Y-63%	Y, na-100%	Y, na-100%	Y, na-33%	Y-25%	Y, na-88%

Loads under 10 kip/ft or 10 kip-ft/ft ignored

na = Design load greater than converged load

Conclusion: Traverse shear and Bending not conservative and don't close well, In-plane moment generally conservative
For existing design all walls reviewed are within acceptance demand/capacity criteria.

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FLOOR SLABS ADJACENT TO OPENINGS

Fig	Type	Elev.	Cut Length	1x1 GTS Conservative					Convergence Achieved (5%)					Design Load Within 10% of Converged Result?				
				Pn	Vn	Mtr	Vtr	Mn	Pn	Vn	Mtr	Vtr	Mn	Pn	Vn	Mtr	Vtr	Mn
30	Adjacent to floor opening	14	46	N		N	N		Y		Y	Y		Y		Y	Y	
31	Adjacent to floor opening	14	16	Y		N	Y		N		Y	Y		na		Y	11%	
32	Floor Slab, Openings both sides	14	12	N		N	N		Y		N	Y		N		N	N	
33	Exterior wall and adjacent opening	0	38	Y		Y	Y		N		Y	Y		na		na	Y	
34	Exterior wall and adjacent opening	0	12	Y		Y	Y		Y		Y	Y		na		na	na	
35	Corner and adjacent opening	0	17	N		N	N		Y		Y	N		N		N	N	
36	Mid slab, Center Cut	0	9	N		N	N		Y		Y	Y		N		Y	Y	
37	Center and adjacent opening	37	14	Y		Y	Y		Y		N	N		na		Y	na	
38	Openings both sides	14	7	N		N	N		Y		N	N		N		N	N	
39	Openings both sides	14	15	Y		N	N		Y		Y	N		na		N	N	
41	Corner adjacent to openings	0	9	Y		N	N		Y		N	N		na		N	N	
43	Two element deep, center cut	3	8	N		Y	Y		N		Y	Y		N		na	na	
44	Adjacent floor openings	14	12	Y		Y	N		Y		Y	N		na		na	*	
45	Mid Slab, Single Element	14	7	N		N	Y		Y		Y	Y		11%		Y	na	
47	Slab pier between openings	14	15	Y		N	Y		N		Y	Y		na		N	na	
48	Exterior wall and adjacent opening	37	15	Y		Y	Y		6%		Y	Y		Y		Y	na	
49	Three element pier, center cut	37	7	N		Y	N		Y		Y	7%		N		na	N	
50	Three element pier, center cut	37	7	N		Y	Y		Y		Y	Y		N		na	na	
51	Two element pier, center cut	37	11	N		N	N		Y		N	N		N		N	N	
52	Single element pier	37	16	N		Y	Y		N		Y	Y		N		na	na	
53	Not shown	0	42	N		N	N		Y		Y	N		Y		Y	N	
54	Not shown	3	15	Y		N	N		Y		Y	N		na		N	N	
55	Two element deep, full cut	3	21	N		N	N		Y		N	N		N		N	N	
56	Single element slab	14	12	N		N	N		N		Y	N		N		N	N	
57	Corner with openings each side	14	30	Y		Y	N		Y		Y	Y		Y		Y	Y	
59	Two element wide at wall	14	25	N		N	N		Y		Y	N		Y		N	N	
60	Two element wide	37	~20	Y		N	N		N		Y	Y		na		Y	Y	
61	Three element wide at wall	37	21	Y		N	N		Y		Y	Y		na		N	N	
62	Three element wide	37	20	N		N	N		N		Y	N		N		N	N	
				Y-45%		Y-34%	Y-34%		Y-72%		Y-79%	Y-55%		Y, na-58%		Y, na-55%	Y, na-50%	

24 Additional slab sections not tabulated

Vn and Mn not usually important for slab design and therfore not shown.

na = Design load is greater than converged load

* indeterminate because of changes in cut length

Major force components for design of slabs are generally Mtr and Vtr

Conclusion: GTS is conservative only 34% to 45% of the time, Finer mesh generally closes
For existing design, all slabs reviewed are within demand/capacity acceptance criteria.

MAJOR SHEAR WALLS

Fig	Type	Elev.	Cut Length	1x1 GTS Conservative					Convergence Achieved (5%)					Design Load Within 10% of Converged Result?				
				Pn	Vn	Mtr	Vtr	Mn	Pn	Vn	Mtr	Vtr	Mn	Pn	Vn	Mtr	Vtr	Mn
1	NS Wall 5, Major Shear Wall	0	120	Y	N			Y	Y	Y			Y	Y	Y			Y
2	NS Wall 5, Major Shear Wall	0	82	Y	N			Y	Y	Y			N	Y	Y			Y
3	NS Wall 7.5, Major Shear Wall	0	134	Y	N			Y	Y	Y			Y	Y	Y			Y
4	NW Wall 12.1, Major Shear Wall	14	134	N	Y			N	Y	N			Y	N	Y			Y
5	NS Wall 17.7, Major Shear Wall	14	71	N	N			Y	Y	Y			Y	Y	Y			Y
6	NS Wall 17.7, Major Shear Wall	14	51	Y	N			Y	Y	Y			Y	Y	Y			Y
7	EW Wall B, Major Shear Wall	0	91	N	N			N	Y	Y			Y	Y	Y			Y
8	EW Wall C, Major Shear Wall	0	24	N	Y			Y	Y	N			Y	Y	Y			na
9	EW Wall C, Major Shear Wall	0	48	N	Y			Y	Y	Y			Y	N	Y			Y
10	EW Wall M, Major Shear Wall	0	40	N	Y			Y	Y	Y			N	Y	na			na
11	EW Wall P, Major Shear Wall	0	90	Y	N			Y	Y	N			N	na	Y			na
12	EW Wall R, Major Shear Wall	0	118	N	N			Y	Y	Y			Y	Y	Y			na
13	EW Wall R, Major Shear Wall	0	92	Y	N			Y	Y	Y			N	na	Y			na
14	EW Wall T, Major Shear Wall	0	92	N	N			Y	Y	Y			Y	N	Y			na
15	EW Wall T, Major Shear Wall	0	62	N	N			Y	Y	Y			Y	N	Y			na
16	NS Wall 7.4, Major Shear Wall	0	29	Y		N	N		Y		Y	Y		Y		N	N	
17	NS Wall 7.4, Major Shear Wall	14	29	N		N	Y		Y		Y	Y		Y		Y	Y	
18	NS Wall 11, Major Shear Wall	0	28	N		N	Y		Y		Y	Y		Y		N	Y	
19	NS Wall 11, Major Shear Wall	14	28	Y		N	N		Y		Y	Y		na		Y	N	
20	NS Wall 11, Major Shear Wall	14	28	N		Y	Y		Y		Y	Y		Y		na	na	
21	NS Wall 11, Major Shear Wall	37	28	Y		N	Y		Y		Y	Y		na		N	na	
				Y-43%	Y-26%	Y-16%	Y-67%	Y-87%	Y-100%	Y-80%	Y-100%	Y-100%	Y-73%	Y,na - 86%	Y,na-100%	Y,na-50%	Y,na-67%	Y,na-100%

Additional Information

In-plane shear is conservative approximately 27% of the time but usually within 3% of the final load

In-plane moment is conservative approximately 87% of the time and within 10% of the final load

Axial Forces are conservative 43% of the time or generally within 10% of final load

na = Design load greater than converged load

Conclusion 1: Major shear wall loads from GTS 1x1 are generally conservative or within 10% of a converged load.

Conclusion 2: Out-of-plane bending loads are generally not conservitive and in some cases are not within 10% of the design loads.

For existing design all walls reviewed are within demand/capacity acceptance criteria.

PROGRESS REPORT

Independent Review of Implementation of Revised Seismic Design Criteria Hanford Waste Treatment and Immobilization Plant (WTP)

Revision D

Appendix E

Discussion of Probabilistic Risk Assessment and Seismic Margin Assessment

Alternative Methodologies to Address Issues Related to Changes in Seismic Design Parameters for the WTP Site

James J. Johnson

E-1. Summary

A seismic margin assessment (SMA), seismic probabilistic risk assessment (PRA), or the hybrid approach should be used to address future seismic design issues, including redefinition of the seismic hazard for the Hanford Waste Treatment and Immobilization Plant (WTP). The SMA and seismic PRA approaches are well established. Most recently, the hybrid approach merges attributes from the SMA capacity calculations with generic fragility function data to derive fragility functions for complete or limited PRA evaluations. This hybrid is particularly attractive since it permits calculation of risk metrics if required for decisionmaking. This strategic approach for decisionmaking represents the current state-of-practice domestically and internationally for critical facilities. It should be adopted by the U.S. Department of Energy (DOE) and Bechtel National Incorporated as one method to address future seismic issues.

E-2. Perspective

The DOE, the international nuclear industry, and government regulatory agencies are frequently faced with decisions related to seismic design of critical facilities during the design process, construction, commissioning, and operation. The following are typical issues that may need to be addressed:

- Evidence of a seismic hazard at the site greater than the earthquake design basis due to new or additional data, e.g., newly discovered faults, and/or due to new methods of seismic hazard assessment.
- Ongoing or new regulatory requirements, such as periodic safety reviews, which take into account the “state of knowledge” and may require evaluation.
- New technical findings, such as vulnerability of selected structure components, e.g., masonry walls, systems or components (relays), etc. New knowledge from earthquakes – more extensive and complete recorded ground motion data and observed performance of structures, systems, and components (SSCs).
- The need to address the question and provide confidence that a “cliff edge” effect does not exist, i.e., if an earthquake occurs greater than the design basis earthquake, demonstrate that significant failures in the facility will not occur and that margin exists.
- Facility performance for “beyond design basis earthquake ground motions.”
- Risk Informed Safety Assessments.
- Lack or inadequate seismic design generally due to the vintage of older facilities.

- Changes to the plant design (new systems/components, new potential systems interactions, and inclusion of non-safety plant equipment in the seismic risk model).

Addressing these and other, yet to be defined, issues can be accomplished through a number of different options. The least cost effective option is reconstitution of the seismic design basis and redesign and requalification of SSCs. In applications to date, for operating facilities and new facilities in the advanced design phase, typically this option has been judged unnecessary to address these issues. It is universally recognized that well designed industrial facilities, especially nuclear facilities, have an inherent capability to resist earthquakes larger than the earthquake used in their original design. This inherent capability is a direct consequence of the conservatism that exists in seismic design procedures and usually described in terms of "seismic design margin." This existing seismic margin is one basis for the development and implementation of the SMA and PRA methodologies to successfully address many seismic issues that arise. The difference in cost between a design basis reconstitution and the effective implementation of a SMA or PRA approach to address these issues can range from one to several orders of magnitude depending on the state of the facility, i.e., in design, construction, or operation.

E-3. Background and Methodology Development

Two methodologies have evolved over the last 30 years to address seismic issues outside of the seismic design process: SMA and PRA (3, 7, and 9).

SMA approaches evolved from an initial study, documented by Budnitz et al. (4), to the Electric Power Research Institute (EPRI) approach (5). These two approaches developed along different paths: the success path methodology denoted EPRI-SMA (5) and the event tree/fault tree methodology denoted U.S. Nuclear Regulatory Commission (NRC) SMA (4). The differences lie in the systems modeling approach and in the capacity evaluation. Systems modeling in the former are by success paths; the latter by event trees/fault trees. Capacity evaluations of SSCs are deterministically calculated as High Confidence of Low Probability of Failure (HCLPF) values in the former; the latter by probabilistically defined fragility functions. One result in each case is the HCLPF of the plant expressed in terms of a ground motion parameter, e.g., peak ground acceleration. Many other products are developed with applications in decisionmaking, including importance ranking of SSCs. The EPRI SMA approach has been the most dominant SMA approach for domestic and worldwide implementations.

HCLPF values are defined probabilistically as about the 95 percent confidence of a 5 percent probability of failure or less. This probabilistic definition assumes that fragility functions are developed with a separation of the two common sources of uncertainty, i.e., aleatory and epistemic uncertainties. Aleatory uncertainty is the inherent randomness of the phenomena. In principle, it cannot be reduced with the accumulation of additional data. It is also referred to as randomness. Epistemic uncertainty is due to incomplete knowledge concerning the modeling of the phenomena. In principle, epistemic uncertainty can be reduced with additional data and research. If capacity and fragility functions are developed and characterized by combined values of aleatory and epistemic uncertainties, termed composite uncertainty, then the HCLPF value is defined as about a 1 percent probability of failure or less.

The seismic PRA methodology has evolved over the past 30 years along with the development of PRA methodologies for internal events. Budnitz (3), Kennedy (7), Ravindra (9), and Kennedy and Ravindra (8) document various stages of its development. The key elements of the PRA are a Probabilistic Seismic Hazard Assessment (PSHA); seismic fragility evaluation; systems/accident sequence modeling and analysis; and risk quantification. Important considerations for the implementation of the PRA approach for a specific facility are: (1) The existence of a PSHA or its performance simultaneously with the PRA; and (2) The existence of an internal events PRA. Without these two elements, a complete PRA is more difficult and resource intensive to perform.

Table 1 compares elements of the SMA and PRA methods. In addition, several steps are common to the two approaches: seismic capacity screening, plant walk downs (10), relay reviews, and seismic-induced fire and flood hazard reviews.

Within the U.S. commercial nuclear power industry, the U.S. NRC required seismic evaluations be performed using SMA or PRA methodologies or modifications. Notes 12 and 14 itemize the requirements and the guidance to be used in performing these seismic evaluations. A total of 103 U.S. nuclear power units were evaluated with approximately one-half being conducted with the SMA methodology and the other half being conducted with the PRA. Notes 6 and 13 summarize the approaches taken, the quantitative and qualitative results, and lessons learned. In addition, numerous implementations of SMA and PRA methodologies have been performed internationally.

A hybrid approach has evolved over the last few years to take advantage of the positive attributes of the SMA and PRA. In this hybrid, the seismic capacities of SSCs are calculated as HCLPF values using the deterministic approach. These HCLPF values define the 95 percent confidence and the 5 percent probability of failure for the SSC of interest and are coupled with generic uncertainty values to define the fragility function. These generic uncertainty values are based on the results of the numerous PRA fragility assessments performed to date. This allows the HCLPF values to be developed by engineers without special training in probability theory, and the resulting fragility functions are reasonable estimates. Event tree/fault tree systems models can then be used to calculate:

- Overall plant risk (if complete systems models are available).
- Changes in risk due to proposed changes in the facility, including systems modifications (hardware, procedures, classification, etc.) to judge acceptability of the modification.
- Changes in the reliability of individual systems due to modifications (limited systems models required for this application).

A representation of the Probabilistic Seismic Hazard is needed for risk calculations. Site-specific seismic hazard is desired when calculating overall risk. In some cases, generic seismic hazards (industry-wide studies of multiple sites) may be used for comparative purposes for alternative designs within a facility or for comparison with other facilities.

E-4. Conclusions

These SMA and PRA methodologies have developed and matured over the last 30 years to the status of a standard. The American Nuclear Society Standard *External Events PRA Methodology* (1), published in 2003, specifies the elements and requirements of the PRA and SMA as they should be applied to the evaluation of critical facilities, with an emphasis on nuclear powerplants. It also includes complete descriptions of the methodologies.

Application of SMA or PRA methodologies to address future seismic design issues, including redefinition of the seismic hazard for the Hanford WTP, represents the current state-of-practice.

E-5. Applications

Two standards for the seismic design of SSCs in the DOE community are DOE-1020 (11) and American Society of Civil Engineers (ASCE) 43-05 (2). Both standards employ seismic design philosophy that lead to significant seismic design margin even for cases where the ratio of seismic demand to seismic design capacity is equal to one.

The WTP seismic design for PC-3 facilities follows the principles of DOE-1020. Consequently, substantial seismic design margin exists. This seismic design margin should be quantified through performance of an SMA, PRA, or the hybrid approach described herein.

A representative list of previous applications of SMA and/or PRA at DOE facilities includes:

- Seismic PRA – N-reactor, Hanford, WA.
- Seismic PRA – High Flux Isotope Reactor, Oak Ridge National Laboratory, TN.
- Seismic PRA or fragility assessment – K, L, and P reactors or portions thereof, Savannah River Site, SC.
- Fragility assessment – Receiving Basin for Offsite Fuels, Savannah River Site, SC.
- Seismic PRA - Advanced Test Reactor, Idaho National Engineering Laboratory, ID.
- SMA – Plutonium Facility PF-4, Los Alamos National Laboratory, NM.

Table 1. Comparison of Elements of SMA and PRA

Steps in SMA/PRA Implementation	SMA	PRA
Seismic Input	Review Level Earthquake (RLE)	PSHA
Plant/Systems Models	Success Path(s)	Event Trees/Fault Trees
Seismic Response Analysis	Deterministic or Probabilistic Best Estimate – for RLE	Deterministic or Probabilistic Best Estimate – for range of earthquakes or as benchmark for extrapolation
Capacity or Fragility Assessment	HCLPF	Fragility Functions – Probability of Failure as a Function of Earthquake Level
Quantification	Deterministic Calculation of Plant HCLPF	Probabilistic Calculation of Plant End State [for nuclear powerplants - Core Damage Frequency (CDF) and Large Early Release Frequency (LERF)] – Point Estimates and Confidence Intervals
End Metrics	SSC and Plant HCLPF Values	CDF, LERF, and Risk Related Ranking of SSCs

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