U.S. Department of Energy
Office of River Protection
P.O. Box 450, MSIN H6-60
Richland, Washington 99352

06-WTP-065

JUN 2 8 2006

The Honorable A. J. Eggenberger
Chairman
Defense Nuclear Facilities Safety Board
625 Indiana Avenue, NW, Suite 700
Washington, DC 20004-2901

Dear Mr. Chairman:

STATUS AND PATH FORWARD FOR THE SEISMIC GROUND MOTION ISSUE AT THE WASTE TREATMENT AND IMMOBILIZATION PLANT (WTP)

This letter provides the status of the seismic ground motion issue, and U.S. Department of Energy's (DOE) plan for the future seismic design at the WTP at Hanford. In February 2005, DOE developed significantly more demanding ground motion criteria based on the implementation of a site-specific seismic site response model for the WTP. These updated ground motion design response spectra are approximately 40% higher than the original 1996 design basis spectra over a range of frequencies from 3 Hz to 8 Hz, and are designated as the revised ground motion (RGM). In 2004, the Defense Nuclear Facilities Safety Board (DNFSB) and DOE Peer Review Team (PRT) identified a number of concerns regarding the mesh densities in the models used for the finite element analysis of the WTP facilities. A six-month lead time was anticipated to complete the dynamic analyses to develop facility loads and the in-structure response spectra. In order to perform design and procurement in the interim period, with minimal risk of rework, bounding interim seismic design criteria was developed in April 2005 for the design of the facilities using the RGM spectra, which was designated as the Interim Seismic Criteria (ISC). The ISC incorporated a 40% increase in loading at all frequencies, as well as bounding mesh amplification factors. Dynamic analyses of the key WTP facilities were completed in September 2005.

In addition, to minimize the impact of the sizeable increase in the spectra to the already constructed, fabricated, and future designs, DOE, in conjunction with the PRT and Bechtel National, Inc. (BNI), developed a list of areas of design and analysis where significant conservatism existed, which could be reduced while providing adequate safety. These changes were evaluated to ensure that they were justified. DOE-PRT, DNFSB staff, and the U.S. Army Corps of Engineers extensively reviewed these changes. Detailed criteria were developed based on parametric studies to ensure that sufficient meshes are incorporated in the finite element modeling of facilities. Finite element mesh densities were significantly increased for the facility structural models, and the original GTStrudl software was changed to more capable SAP2000 software. The Structural Design Criteria (SDC) was revised (Revision 10) and issued in December 2005 that incorporated all of the above changes (Attachment 1). Completion of the dynamic analyses and the approval of the updated SDC allowed the use of ISC to be terminated.

Since then, BNI has been designing the facility structures using the RGM spectra and the revised SDC. Procurement of equipment has incorporated revised in-structure response spectra based on the RGM.

Good geotechnical and geological site profile information was available for the site soils encountered to a depth of several hundred feet for the development of the RGM. However, uncertainty remains as a result of lack of direct shear wave velocity data for the interbed sequence at deeper depths. To compensate for this uncertainty in interbed material properties, the recommended surface design response spectra utilized the 84th percentile of site amplification response to define a conservative representation of the mean surface ground motion (DOE-STD-1020-94, and DOE-STD-1023-95). Due to concerns over the lack of sufficient site data in the development of the RGM, DOE has made the decision to perform deep bore drilling at the site to enhance direct estimates of subsurface dynamic properties. This decision was made largely to confirm that the RGM spectra are a conservative representation of the mean spectra. It is anticipated that with the improved definition of the properties of the site profile from the deep drilling program, the mean spectrum will be less than the current revised design spectra based on the 84th percentile relative amplification function.

In addition to the conservatism that exists in the development of the RGM, DOE considers the ongoing WTP design still maintains other conservatisms. The demand-to-capacity ratios in many of the major walls are significantly less than 1. This allows accommodation of load transfers without exceeding allowable code criteria, which provides added assurance of acceptable structural behavior during an earthquake, even if the future ground motion exceeds the current RGM. The combination of multiple lateral load path capability in the design, together with the use of ductile detailing and the availability of untapped inelastic energy absorption characteristics of the structural elements suggest that the WTP facilities can absorb a significant increase in seismic ground motion without jeopardizing the facility safety.

Also attached is the DOE report *Seismic Ground Motion Issue Report for the Waste Treatment and Immobilization Plant (WTP) Hanford, Washington, Revision 2* (Attachment 2). This report summarizes the evolution of the seismic ground motion issue, the impact of this issue on the project, and the actions taken to mitigate its impact. The document identifies the remaining uncertainty and the path forward for completion of the design and construction of the WTP.

Based on the extensive technical reviews of the RGM and associated SDC described above, DOE considers using these criteria to be justified as the basis for facility design, and re-validation of existing designs. DOE also considers these criteria to be conservative and to provide adequate safety margin. The Board is requested to provide acknowledgement that the issuance of SDC, Revision 10, resolves two of the issues (Seismic Ground Motion and Structural Engineering) outlined in your letter of October 17, 2005, to the Secretary of Energy. The other two issues (Chemical Process Safety and Fire Protection) will be resolved in future correspondence.

If you have any questions, please contact me, or your staff may contact John R. Eschenberg, Project Manager, WTP Project, (509) 376-3681.

Sincerely,

Roy J. Schepens

Roy J. Schepens, Manager
Office of River Protection

WED:WA

Attachments (2)

Attachment 1
06-WTP-065

Structural Design Criteria
24590-WTP-DC-ST-01-001, Revision 10
December 20, 2005

WED:WA
May 16, 2006

R10594710

Document title:	**Structural Design Criteria**
Contract number:	DE-AC27-01RV14136
Department:	Civil, Structural and Architectural
Author(s):	Al Dausman
Principal author signature:	A. V. Dausman
Document number:	24590-WTP-DC-ST-01-001, Rev 10
Checked by:	B. C. McConnel
Checker signature:	
Date of issue:	12/20/05
Issue status:	Approved
Approved by:	Mark Braccia
Approver's position:	CS&A Discipline Production Engineering Manager
Approver signature:	Mark Braccia

ISSUED BY
RPP-WTP PDC

River Protection Project
Waste Treatment Plant
2435 Stevens Center Place
Richland, WA 99354
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History Sheet

Rev	Date	Reason for revision	Revised by
0	01/30/02	Initial Issue	Scott Horn
1	04/28/03	General Update	David Houghton
2	2/5/04	General Update	Al Dausman
3	6/17/04	Deleted Appendix A, revised Sections 5.1, 7.7.2 and other changes as noted	Al Dausman
4	9/28/04	Revised Section 4.3, Added Section 4.18	Al Dausman
5	10/18/04	Revised Table 7 per CCN 100812	Al Dausman
6	1/18/05	Revised Sections 3.0, 4.4.1, 5.7.1 and 6.7.1. Added Sections 2.1.14 and 2.1.15	Al Dausman
7	3/28/05	Revised Sections 2.2, 3.0, 4.6, and 5.2. Revised Table 8 Added Sections 5.4(e) and 5.9. Deleted Table 2	Al Dausman
8	5/2/05	Revised Sections 5.9.2.a) and 5.9.4	Al Dausman
9	9/20/05	Revised Section 6.2 per 24590-WTP-CAR-05-175	Al Dausman
10	12-20-05	Revised Tables: 1, 3, 4, 5, 8. Revised Figures: 1, 2. Revised Sections: 2.1, 3, 4.1, 4.10.5, 4.15.2, 4.16, 5.2, 5.4, 5.7, 6.4, 6.7.1, 6.7.2, 7.6. Added Sections: 2.1.16, 2.1.17, 2.2.8, 2.4.19, 2.4.20, 2.4.21, 2.4.22, 2.4.23, 4.10.6, 4.19, 5.10, 5.11, 5.12 Added Figure 3. Added Appendix C. Deleted Table 7. Deleted Sections: 2.1.3, 2.4.3, 2.4.6, 4.11, 5.3, 5.4.3, 6.3, 6.4.3, 7.3. Several editorial corrections.	Al Dausman

Contents

Appendices

Tables

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Acronyms

The following list contains the acronyms used for the reference documents in this Engineering Criteria.

AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
ASD	Allowable Stress Design
ASTM	American Society for Testing and Materials
CMAA	Crane Manufacturers Association of America
DIN	Deutsches Institut für Normung (German Institute for Standardization)
DBE	Design Basis Earthquake
DOE	Department of Energy
DOE-STD	Department of Energy Standard
E&NS	Environmental and Nuclear Safety
HSS	Hollow Structural Section
ICBO	International Conference of Building Officials
NPH	Natural Phenomena Hazard
NRC	Nuclear Regulatory Commission
PC	Performance Category
PCA	Portland Cement Association
PGA	Peak Ground Acceleration
SC	Seismic Category
SD	Strength Design
SDI	Steel Deck Institute
SIPD	Standards Identification Process Database
SRD	Safety Requirements Document
SSC	Structure, System and Component
UBC	Uniform Building Code
WTP	River Protection Project - Waste Treatment Plant

1 Purpose and Scope

This document provides the minimum structural design criteria for all River Protection Project-Waste Treatment Plant (WTP) facilities.

2 Applicable Codes and Standards

Note: When applying Codes and Standards shown in sections 2.1.1 through 2.1.11 (except 2.1.6 and 2.1.7) and 2.2.1 to ITS equipment, the relevant codes/standards and year of issue referenced in the Safety Requirements Document Volume II (SRD) and listed below shall be used. In addition, any daughter codes/standards, which are referenced in the above parent codes/standards, must also be listed with their year of issue.

2.1 Industry Codes and Standards

2.1.1	ACI 318-99* and ACI 318R-99	*Building Code Requirements for Structural Concrete and Commentary*
2.1.2	ACI 349-01* and ACI 349R-01	*Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary*
2.1.3	*Not Used*	
2.1.4	*Not Used*	
2.1.5	AISC M016-89*	*Manual of Steel Construction – Allowable Stress Design, Ninth Edition*
2.1.6	AISC D807	*Steel Design Guide 7: Industrial Buildings – Roofs to Column Anchorage*
2.1.7	PCA EB080.01	Strength Design of Anchorage to Concrete
2.1.8	ANSI/AISC N690-1994*	*Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities*
2.1.9	ASCE 7-98	*Minimum Design Loads for Buildings and Other Structures*
2.1.10	ASCE 4-98	*Seismic Analysis of Safety-Related Nuclear Structures and Commentary*
2.1.11	1997 UBC*	*Uniform Building Code*
2.1.12	AASHTO HB-16	*Standard Specifications for Highway Bridges, Sixteenth Edition*

2.1.13	SDI Manual No. 30 – April 2001	*Steel Deck Institute Design Manual for Composite Decks, Form Decks and Roof Decks, No. 30*
2.1.14	AWS D1.1-2000	*Structural Welding Code - Steel*
2.1.15	AWS D1.6-1999	*Structural Welding Code - Stainless Steel*
2.1.16	AISC Design Guide 19	*Fire Resistance Of Structural Steel Framing*
2.1.17	ASCE/SEI/SFPE 29-99	*Standard Calculation Method for Structural Fire Protection*

Note: The following two publications supersede the existing publications included in the AISC Manual:

- *Code of Standard Practice for Steel Buildings and Bridges*, March 7, 2000
- *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, June 23, 2000

* This standard is tailored in Appendix C of the SRD (Ref 2.4.2). This Design Criteria document is in accordance with the tailoring in Appendix C.

2.2 DOE Publications

The following DOE orders and standards have been used in part or in their entirety to develop these requirements and criteria. The extent of application of these documents is addressed in *Applicability of DOE Documents to the Design of TWRS-P Facility for Natural Phenomena Hazards* (RPT-W375-RU00003 [Ref. 2.4.4]).

2.2.1	DOE-STD-1020-94* including Change Notice #1 dated Jan 1996	*Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities*
2.2.2	DOE-STD-1022-94 including Change Notice #1	*Natural Phenomena Hazards Characterization Criteria*
2.2.3	DOE-STD-1023-95 including Change Notice #1	*Natural Phenomena Hazards Assessment Criteria*
2.2.4	DOE-STD-1024-92 including Change Notice #1	*Guidelines for Use of Probabilistic Seismic Hazard Curves at Department of Energy Sites for Department of Energy Facilities*
2.2.5	Newsletter dated Jan 22, 1998	Interim Advisory on Straight Winds and Tornadoes
2.2.6	DOE/RL-96-0004	Process for Establishing a Set of Radiological, Nuclear, and Process Safety Standards and Requirements
2.2.7	HNF-PRO-097 Rev. 2	Engineering Design and Evaluation (Natural Phenomena Hazard)
2.2.8	DOE-STD-1066-97	*Fire Protection Design Criteria*

* This standard is tailored in Appendix C of the SRD (Ref 2.4.2). This Design Criteria document is in accordance with the tailoring in Appendix C.

2.3 NRC Publications

The following NRC publications have been used in the development of these Criteria.

2.3.1	Standard Review Plan, NUREG-0800, Section 3.8.4, Rev. 2 (Draft), 4/96	*Other Seismic Category I Structures*
2.3.2	Standard Review Plan, NUREG-0800, Section 3.8.5, Rev. 2 (Draft), 4/96	*Foundations*

2.4 Other Publications

2.4.1	24590-WTP-DB-ENG-01-001	*Basis of Design*
2.4.2	24590-WTP-SRD-ESH-01-001-02	*Safety Requirements Document, Volume II*
2.4.3	*Not Used*	*Superseded by Ref 2.4.19*
2.4.4	RPT-W375-RU00003	*Applicability of DOE Documents to the Design of TWRS-P Facility for Natural Phenomena Hazards*
2.4.5	24590-WTP-DC-ST-04-001	*Seismic Analysis and Design Criteria*
2.4.6	*Not Used*	*Superseded by Ref 2.4.19*
2.4.7	WTSC99-1036-42-17 (H-1616-51)	*RPP-WTP Geotechnical Investigation Report by Shannon & Wilson, Inc., May 2000*
2.4.8	3DG-C01-00004	*Bechtel Design Guide - Seismic Analysis of Structures and Equipment for Nuclear Power Plants*
2.4.9	HNF-SD-GN-ER-501, Rev 1	*Natural Phenomena Hazards, Hanford Site, Washington by NUMATEC Hanford Company* (Sections 4.0, 5.0, 7.0 and 8.0)
2.4.10	24590-WTP-PSAR-ESH-01-002-01	*Preliminary Safety Analysis Report to support Construction Authorization; General Information*
2.4.11	24590-WTP-PSAR-ESH-01-002-02	*Preliminary Safety Analysis Report to support Construction Authorization; PT Facility Specific Information*
2.4.12	24590-WTP-PSAR-ESH-01-002-03	*Preliminary Safety Analysis Report to support Construction Authorization; LAW Facility Specific Information*

2.4.13	24590-WTP-PSAR-ESH-01-002-04	*Preliminary Safety Analysis Report to support Construction Authorization; HLW Facility Specific Information*
2.4.14	24590-WTP-PSAR-ESH-01-002-05	*Preliminary Safety Analysis Report to support Construction Authorization; BOF Specific Information*
2.4.15	24590-WTP-GPP-SREG-002	*Authorization Basis Maintenance*
2.4.16	24590-QL-HC4-HASA-00001-17-00003	*RPP-WTP Supplemental Geotechnical Engineering Studies by Shannon & Wilson, Inc. November 2003*
2.4.17	24590-HLW-RPT-CSA-03-013	*Technical Approach for Boundary Elements in Special Reinforced Concrete Structural Walls*
2.4.18	24590-HLW-RPT-CSA-03-014	*Technical Approach for Boundary Elements in Structural Diaphragms*
2.4.19	CCN 113349	*DOE-ORP letter from R. J. Schepens to J. P. Henschel, BNI, Delivery of Revised Seismic Ground Motion Spectra to be used as the Design Basis for the Design of the Waste Treatment and Immobilization Plant (WTP)*
2.4.20	CCN 116528	*DOE-ORP letter from R. J. Schepens to J. P. Henschel, BNI, Clarification of Fire Protection Requirements for Waste Treatment and Immobilization Plant (WTP), March 18, 2005.*
2.4.21		*ASCE manuals and reports on engineering practice; no. 58., Structural Analysis and Design of Nuclear Plant facilities, ASCE 1980.*
2.4.22		*US Department of the Army, Structures to resist the effects of accidental explosions, Vol. I, Tri-Service manual TM5-1300.*
2.4.23	CCN #133337	*Combination of Thermal and Seismic Loads for the Hanford Waste Treatment Plant Design, dated August 23, 2005.*

2.5 DOE Orders and Standards Applicability

DOE Orders and Standards applicable to the WTP project are contained in the *Safety Requirements Document* (SRD) Volume II, (24590-WTP-SRD-ESH-01-001-02 [Ref. 2.4.2]).

Table 1 provides a summary of the structural design codes and standards and their applicability for Seismic Categories (SC). The application of the particular code or standard is also summarized in the table. Application of these standards is addressed in more detail in the body of this Criterion.

2.6 Authorization Basis Documents

The Authorization Basis is a composite of information provided by the contractor (BNI) in response to the radiological, nuclear, and process safety requirements that is the safety basis on which DOE grants permission to perform regulated activities.

The Authorization Basis describes the safety and administrative control requirements for the design, construction, operation, maintenance and deactivation of the WTP.

A complete listing of the Authorization Basis documents can be found in project procedure 24590-WTP-GPP-SREG-002.

Table 1 Applicability of Design Codes and Standards to Seismic Categories

Title	Applicability	Seismic Categories			
		SC-I	SC-II	SC-III	SC-IV
Natural Phenomena Hazards Design and Evaluation Criteria for DOE Facilities DOE-STD-1020-94 (Ref. 2.2.1)	Design Criteria for Natural Phenomena Hazards	X	X	X	X
DOE Newsletter dated Jan. 22, 1998	Wind and Tornado Loads	X	X	X	X
Seismic Analysis and Design Criteria 24590-WTP-DC-ST-04-001 (Ref. 2.4.5)	Seismic Analysis and Design	X	X	X	X
Seismic Analysis of Safety-Related Nuclear Structures ASCE 4-98 (Ref. 2.1.10)	Dynamic Seismic Design Requirements	X	X		
Uniform Building Code UBC 1997 (Ref. 2.1.11)	Seismic Analysis (Chapter 16)			X	X
Building Code Requirements for Structural Concrete ACI 318-99 (Ref. 2.1.1)	Seismic Detailing of Concrete for High Seismic Risk Regions (Chapter 21)	X	X		
Building Code Requirements for Structural Concrete ACI 318-99 (Ref. 2.1.1)	Seismic Detailing of Concrete for Moderate Seismic Risk Regions (Chapter 21)			X	X
Uniform Building Code UBC 1997 (Ref. 2.1.11)	Seismic Detailing of Structural Steel (Section 2213)	X	X		
Uniform Building Code UBC 1997 (Ref. 2.1.11)	Seismic Detailing of Structural Steel (Section 2214)			X	X
Deleted reference code.					
Code Requirements for Nuclear Safety-Related Concrete Structures ACI 349-01 (Ref. 2.1.2)	Design of Concrete Structures – Strength Design Method	X	X		
Building Code Requirements for Structural Concrete ACI 318-99 (Ref. 2.1.1)	Design of Concrete Structures – Strength Design Method			X	X
Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities ANSI/AISC N690 (Ref. 2.1.8)	Design of Structural Steel – Allowable Stress Design Method (including additional design requirements for stainless steel)	X	X		
Manual of Steel Construction AISC M016, ASD, Ninth Edition (Ref. 2.1.5)*	Design of Structural Steel – Allowable Stress Design Method			X	X
Deleted reference code.					
Minimum Design Load for Buildings and Other Structures ASCE 7-98 (Ref. 2.1.9)	Minimum Live Loads	X	X	X	X
Minimum Design Load for Buildings and Other Structures	Wind Load Design Methodology	X	X	X	X

Title	Applicability	Seismic Categories			
		SC-I	SC-II	SC-III	SC-IV
ASCE 7-98 (Ref. 2.1.9)					
Minimum Design Load for Buildings and Other Structures ASCE 7-98 (Ref. 2.1.9)	Snow Load Design Methodology	X	X	X	X
Steel Deck Institute Design Manual for Composite Decks, Form Decks and Roof Decks No. 30 – April 2001 (Ref. 2.1.13)	Design of Steel Deck	X	X	X	X

* When stainless steel members are used the allowable stresses of N690 may be conservatively used.

3 Categorization of Structures, Systems, and Components

Structures, systems, and components (SSC) are classified into separate safety categories based on the process prescribed by DOE/RL-96-0004 (Ref. 2.2.6). These categories are described in general to define the basis of their respective designs and to indicate expected performance where applicable. The seismic categorization of SSCs follows the guidance of DOE standard DOE-STD-1020-94 (Ref. 2.2.1), except that Performance Category, PC-3 is divided into two parts. One is identified as Seismic Category I, which does not permit credit for inelastic energy absorption as a result of inelastic performance of a structure, except as noted in Section 5 of this document (Criteria for inelastic energy absorption for systems and components are not contained in this document). The other is Seismic Category II (SC-II), which permits the use of inelastic performance of the structure in design of structures detailed to allow for ductile behavior in accordance with the applicable design code or DOE Standards. The limitations for SSCs placed within each of these categories are described in this section.

Seismic Category and Performance Category for SSCs are determined through an Integrated Safety Management Process (ISMP) and documented in the Standards Identification Process Database (SIPD) or technical specification. The engineer shall obtain the SSC category from SIPD or the designated safety representative.

The WTP Facility processes and stores large quantities of radioactive and hazardous materials. Natural phenomena hazards (NPH) such as earthquakes, winds, and floods can result in the uncontrolled release of these materials. Consequently, it is necessary to ensure that facility structures are designed to withstand the effects of those natural phenomena events that are postulated to occur during the life of the facility. Section 4.1 of this Criteria describes the natural phenomena events selected as the bases for the design and evaluation of SSCs important to safety and provides the rationale for their selection.

To ensure that an adequate level of protection is provided for facility workers, co-located workers, and the public from the potential consequences associated with NPH, a graded approach has been employed in the NPH design of the WTP Facility. For seismic events, five seismic categories (SC) are defined as shown in SRD Safety Criterion 4.1-3 (Ref 2.4.2). The seismic categories are derived from the performance categories (PC) defined in DOE-STD-1020-94 (Ref. 2.2.1), which is identified in the SRD as an implementing standard for the WTP seismic design.

Table 2 Deleted

The correlation between the WTP Facility seismic categorization and DOE NPH performance categorization is shown in Table 3.

Table 3 Relationship Between WTP Seismic Categorization and DOE-STD-1020-94 Performance Categorization

WTP Seismic Category	DOE-STD-1020-94 Performance Category
SC-I	PC-3 (see note)
SC-II	PC-3 (see note)
SC-III	PC-2
SC-IV	PC-1
SC-V	PC-0

Note: For the seismic design of SC-I structures, no credit for inelastic energy absorption is taken, except for the evaluation of constructed SC-I structures designed for the original DBE loads. (See Section 5.11.) However, for the seismic design of SC-II structures, credit for inelastic energy absorption is allowed in accordance with Sections 5.4d and 5.12.

4 Design Loads

4.1 Design Load and Capacity Summary

Design loads for SC-I/II, III, and IV structures are provided in Tables 4, 5, and 6.

The Demand/Capacity (D/C) ratios for structural members will be within project-specific and code-allowable limits. The D/C ratio shall be based on load requirements and not code minimums. In addition to the overall D/C ratio the Seismic Demand/Capacity ratio shall also be calculated and documented in the design calculations.

Table 4, provides the NPH design loads for Seismic Category I and Seismic Category II structures as defined in the *Safety Requirements Document* (SRD) Volume II, Safety Criterion 4.1-3 (Ref. 2.4.2).

Table 4 SC-I/II Structures Design Loads

Hazard	Load	Source Document for Load
Seismic	DBE with 0.30 *g* horizontal PGA and 0.21 *g* vertical PGA; See Figures 1 and 2	DOE Letter from ORP (Schepens) to BNI (Henschel) dated 2/11/05, CCN 113349 (Ref 2.4.19)
Straight Wind	111 mi/hr 3-second gust, at 33 ft above ground, Importance Factor, I = 1.00, Exposure Category C	DOE Newsletter (Ref. 2.2.5)
Tornado	Not Applicable	DOE-STD-1020-94 (Ref. 2.2.1)
Wind Missile	2x4 timber plank, 15 lb at 50 mi/hr (horizontal), Max height 30 ft	DOE-STD-1020-94 (Ref. 2.2.1)
Volcanic Ash	12.5 lb/ft^2	HNF-SD-GN-ER-501 (Ref. 2.4.9)
Flooding	Dry site for river flooding Local Precipitation: 4 in. for 6 hours	HNF-SD-GN-ER-501 (Ref. 2.4.9)

Hazard	Load	Source Document for Load
Snow	15.0 lb/ft^2 ground snow load, Importance Factor I=1.2	HNF-SD-GN-ER-501 (Ref. 2.4.9)

Table 5, provides the NPH design loads for Seismic Category III structures as defined in the *Safety Requirements Document* (SRD) Volume II, Safety Criterion 4.1-3 (Ref. 2.4.2).

Table 5 SC-III Structures Design Loads

Hazard	Load	Source Document for Load
Seismic	Uniform Building Code (1997), Static Force Procedure, Importance Factor, I = 1.25 for structures, I_p = 1.50 for systems and components	DOE-STD-1020-94 (Ref. 2.2.1) (See Note 1)
Straight Wind	91 mi/hr 3-second gust, at 33 ft above ground, Importance Factor, I = 1.00, Exposure Category C	DOE Newsletter (Ref. 2.2.5) (See Note 2)
Tornado	Not Applicable	DOE-STD-1020-94 (Ref. 2.2.1)
Wind Missile	Not applicable	DOE-STD-1020-94 (Ref. 2.2.1)
Volcanic Ash	5 lb/ft^2	HNF-SD-GN-ER-501 (Ref. 2.4.9)
Flooding	Dry site for river flooding Local Precipitation: 2.5 in. for 6 hours	HNF-SD-GN-ER-501 (Ref. 2.4.9)
Snow	15.0 lb/ft^2 ground snow load, Importance Factor I=1.0	HNF-SD-GN-ER-501 (Ref. 2.4.9)

Note 1: For the WTP Project, the PGA has been determined from site-specific studies. For SC-III (PC-2) SSCs, the PGA value is approximately 0.2g, corresponding to 1000-year return, 5% damped free-field ground motion spectra. This value is lower than the UBC C_a value of 0.24 that is used for the design of SC-III structures. The shape of the spectrum for SC-III structures shall be the same as the UBC shape.

Note 2: The referenced newsletter establishes the straight wind for SC-III (PC-2) as 85 mi/hr with an Importance Factor of I=1.25 which is equivalent to 91 mi/hr with an Importance Factor of I=1.0.

Table 6 provides the NPH design loads for SC-IV structures.

Table 6 SC-IV Structures Design Loads

Hazard	Load	Source Document for Load
Seismic	Uniform Building Code, Static Force Procedure, Importance Factor, I = 1.0 for structures, I_p = 1.0 for systems and components	DOE-STD-1020-94 (Ref. 2.2.1)
Straight Wind	85 mi/hr 3-second gust, at 33 ft above ground, Importance Factor, I = 1.00, Exposure Category C	DOE Newsletter (Ref. 2.2.5)
Tornado	Not Applicable	DOE-STD-1020-94 (Ref. 2.2.1)
Wind Missile	Not applicable	DOE-STD-1020-94 (Ref. 2.2.1)
Volcanic Ash	3 lb/ft^2	HNF-SD-GN-ER-501 (Ref. 2.4.9)
Flooding	Dry site for river flooding Local Precipitation: 1.8 in. for 6 hours	HNF-SD-GN-ER-501 (Ref. 2.4.9)
Snow	15.0 lb/ft^2 ground snow load, Importance Factor I=1.0	HNF-SD-GN-ER-501 (Ref. 2.4.9)

Figures 1 and 2 provide the horizontal and vertical response spectra, respectively, associated with the Design Basis Earthquake (DBE) (Ref 2.4.19).

Figure 1 WTP DBE Horizontal Response Spectra (Original and Revised) (5 % Damping) (Ref 2.4.19)

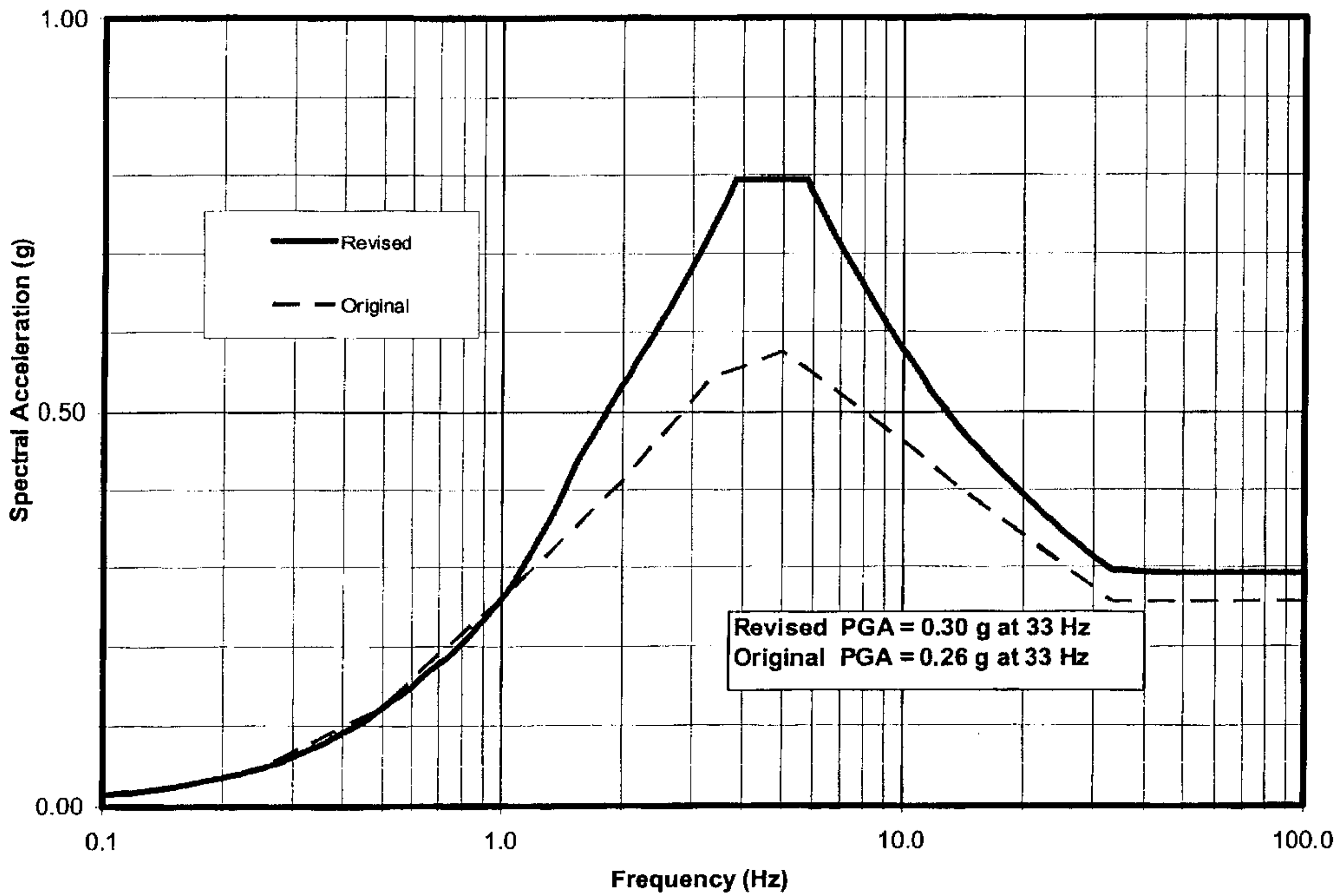


Figure 2 WTP DBE Vertical Response Spectra (Original and Revised) (5 % Damping) (Ref 2.4.19)

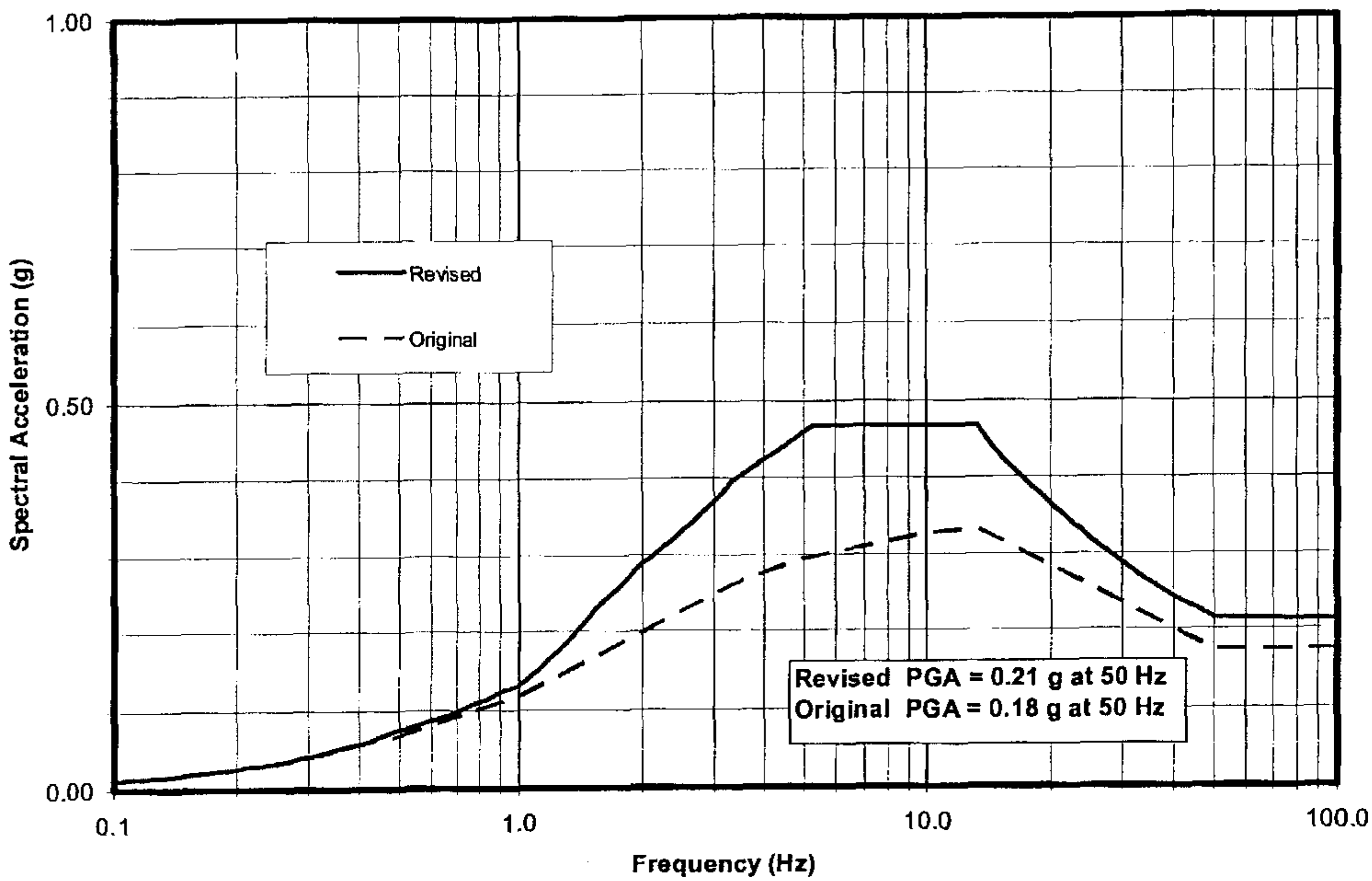


Table 7 Not Used

4.2 Design Load Notation

	Description	Section
D	Dead Load	4.3
L	Live Load (except roof live load)	4.4
L_r	Roof Live Load	4.4.4
S_N	Snow Load	4.5
A	Ashfall Load	4.6
W	Wind Load	4.7
H	Earth Pressure Loads	4.8
E	Earthquake (Seismic) Loads	4.9
T_o & T_a	Thermal Loads	4.10
C	Creep & Shrinkage	4.11
F	Fluid Loads	4.12
R_o	Operating Pipe Reactions	4.13

4.3 Dead Load, D

Dead loads shall include weight of structure, built-in partitions, permanent equipment, piping, raceways, HVAC ductwork, and other permanent static loads.

The following minimum allowance shall be made for dead loads due to piping, raceway, and HVAC ductwork:

Structure	Minimum Uniform Load
• Pretreatment, HLW and LAW Vitrification Buildings	50 psf*
• All other buildings	25 psf

* Where there is sufficient maturity of design for the area below and above the roof, a dead load based on actual weight of commodities plus 20% (i.e. HVAC ductwork, raceway, piping) and equipment supported by the roof may be utilized. The basis for the actual weight shall be documented in the calculation.

The minimum allowance for the weights of partitions shall be as follows:

- For partition weights 150 lb/ft or less, a partition load of 20 psf shall be used.
- Partition weights greater than 150 lb/ft, the actual linear loads shall be used.

The unit weights of materials and construction assemblies for buildings and other structures shall be those given in ASCE 7 (Ref. 2.1.9). Where unit weights are neither established in that standard nor determined by test or analysis, the weights shall be determined from data in manufacturer's drawings or catalogs.

4.4 Live Load, L

Live loads are those loads produced by the use and occupancy of a building or other structure and do not include construction and environmental loads such as wind load, snow load, earthquake load, flood load or dead load. Live loads on a roof are produced by maintenance workers and equipment and within a structure by partitions, people, and office equipment. Live loads can be eliminated for the area of the footprint of large permanently fixed equipment. Live loads for buildings and other structures shall not be less than the minimum uniform load or concentrated load stipulated in ASCE 7 (Ref. 2.1.9).

4.4.1 Floor Live Load

Minimum design floor live loads shall be as follows:

Area Description	Minimum Uniform Load
• Process Areas	100 psf
• Offices	50 psf
• Laboratories	100 psf
• Warehouse and Storage	250 psf
• Locker Rooms	100 psf
• Corridors	100 psf
• Restrooms	60 psf
• Stairs and Exitways	100 psf
• Platforms	100 psf
• Crane and Other Heavy Maintenance Areas	250 psf
• Elevator Machine Room and Laundry Room	150 psf or weight of actual equipment/ stored material, whichever is greater
• Handrails and Guardrails	Per Table 16B of UBC 1997 (Ref. 2.1.11)
• Movable Partitions	20 psf
• Surcharge Outside and Adjacent to Structures	250 psf

The following concentrated loads shall be considered in lieu of uniformly distributed loads, when they produce greater effects on the structure:

• Floor Slabs	2000 lb. applied over an area 2'-6" x 2'-6"
• Roof Trusses and Steel Floor Framing	2000 lb. at any single panel point of truss lower chord or anywhere on the beam
• Stair Treads	300 lb. applied over an area of 4.0 in^2
• Floor Plate	200 lb. applied over an area of 1.0 in^2
• Elevator Machine Room Grating	300 lb. applied over an area of 4.0 in^2

4.4.2 Other Live Loads

The following other live load conditions commonly encountered in design shall be considered where applicable:

- The weight of service equipment that may be removed with change of occupancy of a given area.
- Live loads for truck support structures shall be HS 20-44 loading in accordance with AASHTO HB-16 (Ref 2.1.12).
- The crane live load shall be the rated capacity of the crane. Design loads on the crane runway beams shall include the maximum wheel loads and the vertical impact, and lateral and longitudinal forces induced by the moving crane, per Section 4.10 of ASCE 7 (Ref. 2.1.9) and seismic load. Reduced crane live load for combinations with seismic loads may be used based on operating time determined by the Project Mechanical Handling Group, and shall be stated as a percentage of the crane rated capacity.
- Impact allowances for elevators and machinery shall be in accordance with Section 4.7 of ASCE 7 (Ref. 2.1.9).
- For corridors and maintenance areas, other moving loads shall be evaluated on a case-by-case basis.

4.4.3 Reduction in Live Loads

No credit for the live load reduction factors, as described in ASCE 7 Section 4.8, or UBC Sections 1607.5 and 1607.6, shall be made in the design of WTP facility structures and their supporting foundations.

4.4.4 Roof Live Load, L_r

Minimum design roof live load shall be 20 psf.

4.5 Snow Load, S_N

Snow loads, full or unbalanced, shall not be concurrent with roof live loads and shall be substituted for roof live loads where such loading results in larger members or connections. Snow loads for buildings and other structures shall be in conformance with ASCE 7 (Ref. 2.1.9). A ground snow load, P_g, of 15 psf, shall be used for calculating roof snow load. An importance factor of I=1.2 shall be used to calculate roof snow loads for SC-I and SC-II facilities. For SC-III and SC-IV facilities, use I=1.0 to calculate roof snow loads. Unbalanced snow loads resulting from drifting or sliding shall be considered.

4.6 Ashfall Load, A

The following ashfall loads shall be combined with roof live loads:

Seismic Category (Performance Category)	Design Ashfall Load
SC-I & II (PC-3)	12.5 psf
SC-III (PC-2)	5.0 psf
SC-IV (PC-1)	3.0 psf

Unbalanced ashfall loads resulting from drifting or sliding shall be considered, in accordance with the requirements of HNF-PRO-097 (Ref. 2.2.7).

4.7 Wind Load, W

Wind loads shall be calculated per the provisions of ASCE 7 (Ref. 2.1.9) using the parameters set forth in tables 4, 5, and 6 of section 4.1 of this Criteria.

SC-I and SC-II facilities shall be designed to withstand impacts from wind missiles in accordance with the criteria set forth in Table 4 of section 4.1 of this Criterion.

4.8 Lateral Earth Pressure, H

Every foundation wall or other wall serving as a retaining structure shall be designed to resist (in addition to the vertical loads acting on it) the incident lateral earth pressures and surcharges. Dynamic lateral earth pressures increment due to DBE shall be computed for SC-I and SC-II structures from the soil-structure interaction analysis. At rest lateral earth pressure shall be used in the design of structures. Active lateral earth pressures shall be used in the stability evaluation of structures.

Ground water elevations are located about 275 feet below the ground surface (Ref. 2.4.7) and, therefore, hydrostatic loads are not applicable.

All structural foundations shall extend into the surrounding soil below the frost line. The frost line is 30" below finished grade.

4.9 Earthquake (Seismic) Loads, E

Earthquake loads shall be calculated per the provisions of *Seismic Analysis and Design Criteria* 24590-WTP- DC-ST-04-001 [Ref. 2.4.5].

4.10 Thermal Loads

The design of structures shall include the effects of stresses resulting from variations in temperatures under normal operating conditions and accident conditions. External structural elements exposed to the environment shall consider the maximum seasonal temperature change. The design shall provide for the lags between air temperatures and the interior temperatures of massive concrete members or structures.

4.10.1 Base Temperature

The base temperature for thermal analyses shall be 70°F. This temperature is based on recommendations from ACI 349 (Ref. 2.1.2, Commentary on Appendix A) and shall be used for all seismic category facilities.

4.10.2 Ambient Temperatures

Air temperatures for performance categories PC-1, PC-2, and PC-3 shall be per HNF-SD-GN-ER-501 (Ref. 2.4.9) as shown below.

Seismic Category (Performance Category)	Maximum (°F)	Minimum (°F)
SC-I & II (PC-3)	118	-35
SC-III (PC-2)	117	-30
SC-IV (PC-1)	115	-25

4.10.3 Operating Temperatures, T_o

Internal temperatures at various locations inside the WTP Facility structures during normal operating conditions are obtained from Computational Fluid Dynamic (CFD) analyses, which are described in the facility-specific PSAR's (ref. 2.4.11, 2.4.12, 2.4.13 and 2.4.14). If necessary minimum temperature requirements shall be taken from the Ventilation Basis of Design (Ref. 2.4.1).

4.10.4 Accident Temperatures, T_a

Internal temperatures at various locations inside the WTP Facility structures during accident conditions are obtained from Computational Fluid Dynamic (CFD) analyses, which are described in the facility-specific PSAR's (ref. 2.4.11, 2.4.12, 2.4.13 and 2.4.14). Elevated temperatures due to seismically induced accidents may occur after the seismic event has ended (e.g. melter glass spill).

4.10.5 Temperature Limitations on Structural Elements

The following limitations on structural elements shall be applied:

For structural steel elements, the temperatures may reach up to 150 degree F without reducing the strength (ASCE Manuals and Reports on Engineering Practice No. 78, Structural Fire Protection, Appendix A.1.2.2). Steel properties shall be evaluated for temperatures above 150°F in accordance with the example provided in Appendix C, Strength and Modulus Reduction for Structural Steel Grade A36. Other alloys shall use a similar reduction based on the reference.

- For reinforced concrete structures the temperatures may reach the following limits without reducing the strength (Ref. Appendix A of 2.1.2):

  (a) For normal operation or any other long term period, the temperatures shall not exceed 150°F except for local areas, such as around penetrations, which are allowed to have increased temperatures not to exceed 200°F

  (b) For accidents or any other short-term period, the temperatures shall not exceed 350°F for the surface. However, local areas are allowed to reach 650°F from steam or water jets in the event of a pipe failure.

4.10.6 Thermal Requirements

Because of the Revised Ground Motion in February 2005, the requirement for combining seismic and thermal loads was reexamined. This reevaluation (ENG-DECS-05-066, *Combination of Thermal and Seismic Loads for the Hanford Waste Treatment Plant Design* [Ref 2.4.23]) provided a technical basis to eliminate the thermal component from the seismic and thermal load combinations under certain conditions. Therefore, if those load combinations in Section 5 that include seismic and thermal components result in exceeding code criteria, the guidance in the Reference 2.4.23 may be used, on a case-by-case basis as concurred to by the BNI, CSA Chief Engineer. However, the use of F_μ in this guidance shall be restricted to constructed structures in accordance with Section 5.11.

<u>General Guidance from Ref. 2.4.23 is:</u>

- Develop the full strength of all reinforcing at the face of the joint.
- Ensure that the structure meets ACI 349 Load Combinations #6 and #9.
- Meet the concrete temperature limits of ACI 349 Appendix A.
- Thermal loads which reduce the effect of seismic loads shall be omitted.
- For components which require $F_\mu > 1$, document both the required F_μ and the allowable F_μ.

If the load combination from ACI 349 #4 or #8 results in exceeding the allowable code criteria then additional guidance from Ref 2.4.23 is provided for:

<u>In-Plane Bending and Out-of-Plane Bending</u>

- Omit the terms T_o and T_a in ACI 349 Load Combinations #4 and #8 for calculating bending demand.
- If F_μ>1 is required after omitting the T_o and T_a terms from Load Combinations #4 and #8, reduce the permissible inelastic force reduction factor, F_μ, to account for thermal moments by

$$F_\mu = 2 - \frac{Mt}{My} \leq 1.75$$

where Mt = the thermal bending moment at a section cut, which may include the effects of cracking, and

My = Mn, the nominal in-plane bending capacity calculated per ACI 349.

In-Plane Shear

- Omit the terms T_o and T_a in ACI 349 Load Combinations #4 and #8 for calculating in-plane shear demand.
- If $F_\mu > 1$ is required after omitting the T_o and T_a terms from Load Combinations #4 and #8, reduce the permissible inelastic force reduction factor, F_μ, using Table 1 (Ref 2.4.23).
- Consider the ACI 349 axial load - shear interaction, when required by ACI 349, using axial loads determined in accordance with "Axial Tension."

Out-Of-Plane Shear

- Determine the shear corresponding to a flexural hinge mechanism, and the code shear strength ϕV_n. Scale the seismic loads until a hinge mechanism is achieved as shown in Figure 10 (Ref 2.4.23). If $V_{max} > \phi V_n$, classify the member as shear controlled. If $V_{max} \leq \phi V_n$, classify the member as flexural controlled.
- Shear Controlled Members: Use ACI 349 Load Combinations #4 and #8, including the thermal loads for calculating shear demand. Flexural cracking meeting the requirements of ACI 349 Appendix A may be considered.
- Flexure Controlled Members: Omit the terms T_o and T_a in ACI 349 Load Combinations #4 and #8 for calculating shear demand.
- Consider the ACI 349 axial load - shear interaction, when required by ACI 349, using axial loads determined in accordance with "Axial Tension."

Axial Tension

- Determine the average axial thermal strain on each section cut.
- Determine the average shrinkage strain by laboratory testing or use the default vaue of 0.0006 in/in (Ref 2.4.23, *Recommendations for Axial Tension*, Item #3).
- Combine average axial thermal and shrinkage strains.
  (a) If the average axial thermal plus shrinkage strain is less than the value in Table 2 (Ref 2.4.23), then omit the terms T_o and T_a in ACI 349 Load Combinations #4 and #8 when calculating shear demand.
  (b) If the average axial thermal plus shrinkage strain is greater than the value in Table 2 (Ref 2.4.23), then use ACI 349 Load Combinations #4 and #8 without modification and with $F_\mu = 1.0$, when calculating shear demand.
- Omit the terms T_o and T_a in ACI 349 Load Combinations #4 and #8 when calculating axial tension demand. Use $F_\mu = 1.0$ for axial tension demand in collector elements and axial compression demand. Use $F_\mu = 1.75$ for axial tension demand at locations away from collector elements unless the required reinforcement has been proportioned across the entire width of the diaphragm.

4.11 Deleted

4.12 Fluid Load, F

The design of structures shall include the effects of stresses resulting from fluid loads. Fluid loads include loads due to weight and pressure of fluids. Fluid loads shall include the effects of horizontal sloshing in accordance with Section 3.5.4.3 of ASCE 4 (Ref. 2.1.10).

- US Department of the Army, Structures to resist the effects of accidental explosions, Vol. I, Tri-Service Manual TM5-1300 (Ref. 2.4.22).

4.17 Concrete Wall Out-of-Plane Embedment Load Guidelines

All concrete walls, except for free-standing non-load bearing walls, shall be designed for the additional out-of-plane embedment loads due to equipment/commodity loading. See Appendix B for guidelines.

4.18 Fall Restraints

Fall restraints shall be designed for a 5000 pound load per person.

4.19 Fire Resistant Design of Floor and Roof Systems

Fire resistant floor and roof systems contain Primary steel girders, Secondary steel girders and beams, and concrete floor slabs (see Figure 3). Steel columns and walls are used to support fire-resistant floor systems. The reinforcing steel in concrete walls and slabs is protected by its concrete cover and does not need additional fireproofing. Steel columns and Primary Steel girders are protected by applying fireproofing material (see note), whereas, the secondary steel girders and beams are not protected by fireproofing.

Floor and roof systems shall initially be designed to meet the loads and load combinations of Section 5 and 6. Since some of the beams of the fire-resistant floor systems are not fireproofed, the floor system shall be designed to withstand additional load combinations during a postulated fire, without the support provided by the secondary beams. Secondary beams and girders that are not fire-proofed are assumed to have no load capacity during a fire, and because of this, fire-proofed primary girders are designed to carry the larger tributary areas of floor slab, between fire-proofed members. Therefore, fire-proofed primary girders and floor slabs shall be designed for the following additional loads and load combinations in accordance with ASCE-7 Section 2.5 (Ref. 2.1.9) and the AISC Design Guide 19 (Ref. 2.1.16).

- **Primary Steel Girders** are the steel girders that frame into walls, columns or other fire-protected girders and along with the concrete walls, support the floor slabs during a fire.
  - Additional load combinations, which envelop ASCE-7 load combinations, shall be used:
    For SC-I, SC-II, SC-III and SC-IV structures:
    $$1.33S = D + Equip + L$$
    $$S = D + Equip + 0.5L$$
  Note:
  The load term A_k is not used in the above load combinations because it is taken as zero.
  The load term A_k is taken as zero because there are no transient (i.e., explosive) load cases.

- **Floor Slabs** are concrete floor slabs that span between walls and primary girders during fires and between steel beams for normal loading.
  - Additional load combinations, which envelop ASCE-7 load combinations, shall be used:
    For SC-I, SC-II, SC-III, SC-IV structures:
    $$U = 0.75\ (1.4D + 1.4\ Equip + 1.7L)$$
    $$U = 1.4\ (D + Equip)$$
  Note:
  The load term A_k is not used in the above load combinations because it is taken as zero.
  The load term A_k is taken as zero because there are no transient (i.e., explosive) load cases.

4.13 Operating Pipe Reactions, R_o

Operating pipe reactions include piping reactions during normal, operating, and shutdown conditions.

4.14 Precipitation Levels

Design-basis 6-hour maximum precipitation levels shall be as follows (Ref. 2.4.9):

Seismic Category (Performance Category)	Amount of Rainfall (inches)
SC-I & II (PC-3)	4.0
SC-III (PC-2)	2.5
SC-IV (PC-1)	1.8

4.15 Construction Loads on Steel Deck and Framing Supporting Concrete Slabs

4.15.1 Steel Deck

Steel deck supporting wet concrete shall be designed for the weight of concrete plus 50 psf uniformly distributed load.

4.15.2 Structural Steel Framing

Steel framing supporting steel deck, in addition to other applicable design load combinations, shall be designed for the following load cases:

- The weight of wet concrete plus a 50 psf uniformly distributed load over the tributary area supported by the beam. A note shall be added to the design drawings stating that no rigging shall be permitted from the steel framing during placement of concrete until the concrete has attained its full design strength.
- A 5000 lb concentrated load, without the weight of the concrete, or live load, placed anywhere on the span to maximize moment and shear, prior to placement of the concrete slab. The concentrated load is not cumulative and shall not be carried to columns.

4.15.3 Crane Loads

Crane loading adjacent to below-grade structures shall be considered.

4.16 Drop Load

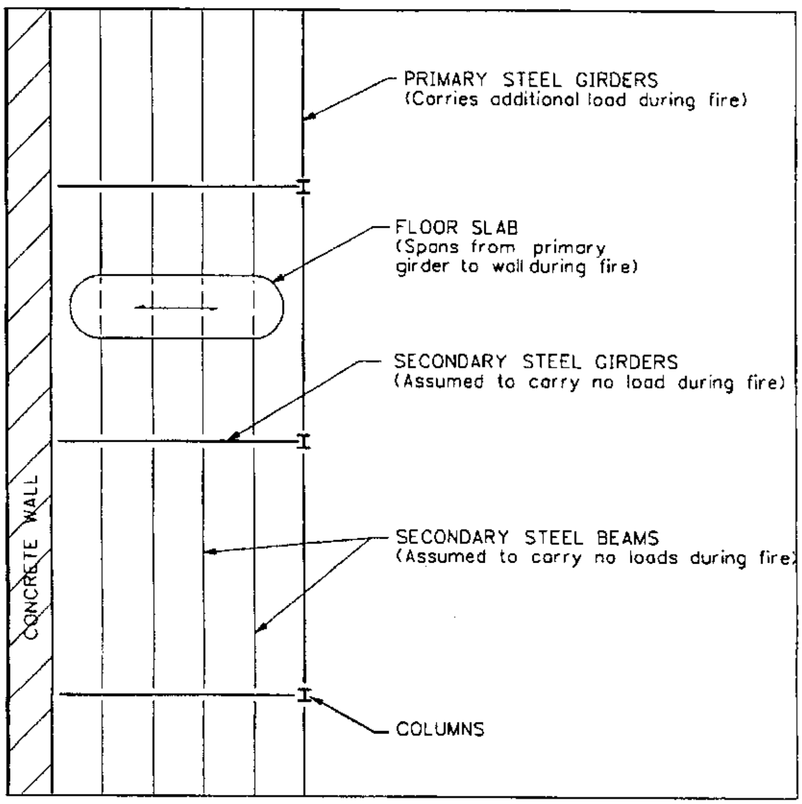
Drop loads shall be treated as live loads with impact. Postulated dropped loads will be evaluated for local damage (for example, penetration, perforation and spalling of a concrete slab) as well as for structural integrity. For the local impact design of structures, credit may be taken for the inelastic absorption (ductilities) of the structural element. The acceptability of damage due to the dropped load will be evaluated by the ISM Process (for example, penetration may be acceptable but perforation may not be acceptable due to loss of confinement). Drop load analysis methods shall be based on:

- ASCE manuals and reports on engineering practice, no. 58., Structural Analysis and Design of Nuclear Plant facilities, ASCE 1980 (Ref. 2.4.21), and

- **Secondary Steel Girders and Beams** are assumed to have no load capacity during a fire, and are not fireproofed. These members span between primary girders and walls, they are relied upon to support floor slabs during construction and normal loads from dead, live, seismic, and equipment but not during a fire.
  - No additional load combinations.

  Note:
  ASCE/SEI/SFPE 29-99 Table X3.1 Construction Classification for Restrained and Unrestrained provides guidance for determining thicknesses of structural steel fire coatings.

Figure 3 Fire Resistant Floor System



5 SC-I and SC-II Facility Design Requirements

5.1 Reinforced Concrete Design

SC-I and SC-II reinforced concrete structures shall be designed in accordance with the following:

- Strength Design Method in accordance with ACI 349 (Ref. 2.1.2).
- Seismic proportioning and detailing shall be per the provisions of ACI 318 (Ref. 2.1.1) Chapter 21 pertaining to structures in "High" seismic risk regions.

- Seismic proportioning and detailing may also include the provisions of Section 21.6.1 of ACI 349 (Ref 2.1.2) (Height / Length criteria). Height is defined as the total height of the wall and length as the length of the wall.
- In addition to the provisions of Sections 21.6.6.3 and 21.7.5.3 of ACI 318 (Ref 2.1.1) boundary elements are not required when the concrete compressive strain, resulting from the worst case loading combination, does not exceed 0.002. (For HLW and PT buildings only.) Note: When evaluating wall cross sections, include the contributing portions of the cross walls as flanges in calculating compressive strains using cracked section properties.
- In lieu of the requirements of Section 21.7.8.1 of ACI 318 (Ref 2.1.1), proportion reinforcement across the entire width of the diaphragm to resist the factored axial forces and moments acting in the plane of the diaphragm.
- For additional information on the technical approach for boundary elements see Ref 2.4.17 and Ref 2.4.18

5.2 Structural Steel Design

SC-I and SC-II steel structures and commodity supports shall be designed in accordance with the following:

- Allowable Stress Design Method using ANSI/AISC N690-1994 (Ref. 2.1.8). Load combinations and allowable stresses shall be per Section 5.4.2.
- Allowable stress for austenitic stainless steel and nickel-based alloy structures shall be determined per ANSI/AISC N690 (Ref. 2.1.8).
- Seismic proportioning and detailing requirements of Section 2213 of UBC (Ref. 2.1.11) shall be met. In applying these requirements, the term "Ω_o times the earthquake load (UBC)" shall be replaced by the term "E" as defined in Section 5.4
- Braced frames of HLW Vitrification Building, Pretreatment Facility, PTF Annex, and ITS Switchgear Building shall be designed as Ordinary Braced Frames per the provisions of Section 2213.8 of UBC (Ref 2.1.11) without use of inelastic energy absorption factor (F_μ = 1.0).

5.3 Masonry Design

- This section has been deleted because masonry is not used in SC I and II facilities.

5.4 Load Factors and Load Combinations

Notations

A	=	Ashfall Load
D	=	Dead Load
L	=	Live Load
L_r	=	Roof Live Load
S_N	=	Snow Load
E	=	Earthquake (Seismic) Load (due to DBE)
F_μ	=	Inelastic Energy Absorption Factor - Table 2.4 of DOE-STD-1020-94 (Ref. 2.2.1)

H = Lateral Earth Pressure Load
T_a = Thermal Loads during Accident Condition
T_o = Thermal Loads during Normal Operating Conditions
F = Fluid Load
R_o = Operating Pipe Reaction Load
S = Allowable Stress per Allowable Stress Design Method
U = Required Strength per Strength Design Method
W = Wind Load

In addition to the load combinations provided in sections 5.4.1, 5.4.2, and 5.4.3, the following load conditions shall be considered:

(a) Where the structural effects of differential settlement, creep, or shrinkage may be significant, they shall be included with the dead load D in all the load combinations. Estimation of these effects shall be based on a realistic assessment of such effects occurring in service.
(b) Where any load reduces the effect of other loads, the corresponding coefficient for that load shall be taken as 0.9 if it can be demonstrated that the load is always present or occurs simultaneously with other loads. Otherwise, the coefficient for that load shall be taken as zero.
(c) All load combinations shall be checked for zero live load condition.
(d) The load combinations for design of SC-II structures shall be identical to those shown below for design of concrete or steel SC-I structures except that "E" in the load combinations shall be replaced by "E/F_μ". SC-II structures using $F_\mu>1.0$ shall be designed to ductile detailing requirements. (If ordinary Braced Frames are used to resist seismic loads, the $F\mu$ shall be equal to 1.0.)
(e) Fire and Seismic conditions (DBE) shall not occur simultaneously.

5.4.1 Reinforced Concrete Design Load Combinations

The following load combinations are based on Section 9.2 of ACI 349 (Ref. 2.1.2).

$U = 1.4D+1.7L+ 1.7L_r+1.7A+1.4F+1.7H+1.7R_o$

$U = 1.4D+1.7L+ 1.7S_N+1.4F+1.7H+1.7R_o$

$U = 1.4D+1.7L+ 1.7L_r+1.4F+1.7H+1.7R_o+1.7W$

$U = 1.4D+1.7L+ 1.7S_N+1.4F+1.7H+1.7R_o+1.7W$

$U = D+L+L_r+F+H+T_o+R_o+E$

$U = D+L+S_N+F+H+T_o+R_o+E$

$U = D+L+L_r+F+H+T_a+R_o$

$U = D+L+S_N+F+H+T_a+R_o$

$U = D+L+L_r+F+H+T_a+R_o+E$

$U = D+L+S_N+F+H+T_a+R_o+E$

$U = 1.05D+1.3L+ 1.3L_r+1.3A+1.05F+1.3H+1.05T_o+1.3R_o$

$U = 1.05D+1.3L+ 1.3S_N+1.05F+1.3H+1.05T_o+1.3R_o$

$U = 1.05D+1.3L+ 1.3L_r+1.05F+1.3H+1.3W+1.05T_o+1.3R_o$

$U = 1.05D+1.3L+ 1.3S_N+1.05F+1.3H+1.3W+1.05T_o+1.3R_o$

5.4.2 Structural Steel Design Load Combinations

The following load combinations are based on Table Q1.5.7.1 of ANSI/AISC N690 (Ref. 2.1.8), as modified by Appendix F of Section 3.8.4 of NUREG-0800 (Ref. 2.3.1).

$S = D+L+L_r+A$

$S = D+L+S_N$

$S = D+L+ L_r+A+R_o+T_o$

$S = D+L+ S_N+R_o+T_o$

$S = D+L+L_r+W$

$S = D+L+S_N+W$

$S = D+L+L_r+W+R_o+T_o$

$S = D+L+S_N+W+R_o+T_o$

The following load combinations shall also apply except for the design of members in compression and shear or for bolted connections.

$1.6\ S = D+L+ L_r+R_o+T_o+E$

$1.6\ S = D+L+ S_N+R_o+T_o+E$

$1.6\ S = D+L+ L_r+T_a+R_o$

$1.6\ S = D+L+ S_N+T_a+R_o$

$1.7\ S = D+L+ L_r+T_a+R_o+E$

$1.7\ S = D+L+ S_N+T_a+R_o+E$

The following load combinations shall apply for the design of members in compression and shear and for bolted connections.

$1.4\ S = D+L+ L_r+R_o+T_o+E$

$1.4\ S = D+L+ S_N+R_o+T_o+E$

$1.4\ S = D+L+ L_r+T_a+R_o$

$1.4\ S = D+L+ S_N+T_a+R_o$

The following load combinations shall be used for the design of members in compression.

$1.6\ S = D+L+ L_r+T_a+R_o+E$

$1.6\ S = D+L+ S_N+T_a+R_o+E$

The following load combinations shall be used for design of members in shear and for design of bolted connections.

$1.4\ S = D+L+ L_r+T_a+R_o+E$

$1.4\ S = D+L+ S_N+T_a+R_o+E$

5.4.3 Masonry Design Load Combinations

This section has been deleted because masonry is not used in SC I and II facilities.

5.5 Stability Requirements for Building Structures

SC-I and SC-II building structures shall meet the following factors of safety:

Load Combination	Sliding	Overturning
D + H + W	1.5	1.5
D + H + E	1.1	1.1

Stability against overturning due to seismic loads shall be evaluated by the "energy approach". i.e., the factor of safety against overturning shall be calculated as the ratio of potential energy required to cause overturning about one edge of the structure, to the maximum kinetic energy in the structure due to the earthquake. The procedure described in Section 4.4.2 of *Seismic Analysis of Structures and Equipment for Nuclear Power Plants* (Ref. 2.4.8) shall be followed for the evaluation.

5.6 Deflection Limits

5.6.1 Reinforced Concrete Members

Deflections in reinforced concrete members shall be computed based on cracked section properties. Control of deflections in reinforced concrete members shall be in accordance with Section 9.5 of ACI 349 (Ref. 2.1.2).

5.6.2 Structural Steel Members

Note: In this section, "F_y" represents the specified minimum yield stress of steel in kips per square inch, and "L" represents the length of span.

(a) The depth of fully stressed floor beams and girders shall not be less than (F_y/800) times the span. If members of less depth are used, the allowable bending stress shall be decreased in the same ratio as the depth is decreased from that recommended above. Also, the deflections under live and combined dead plus live loads shall not exceed L/360 and L/240 respectively.
Note: Construction loads need not be included in these deflection criteria.
Note: These criteria do not apply to platforms, multi-commodity supports and miscellaneous steelwork.

(b) The depth of fully stressed roof purlins shall not be less than (F_y/1000) times the span except in the case of flat roofs.

(c) All roofs shall be designed with sufficient slope or camber to ensure adequate drainage after the long-term deflection from dead loads, or, shall be designed to resist ponding load. Ponding load shall include water accumulation from any source, including snow, due to deflection.

5.6.3 Crane Runway Support Beams and Monorails

The following requirements are from Section 18 of AISC D807 (Ref. 2.1.6):

Maximum vertical deflection (loads without impact) = Span/600 For CMAA Class A, B, C & D cranes
Maximum vertical deflection (loads without impact) = Span/1000 For CMAA Class E & F cranes
Maximum lateral deflection = Span/400 For all cranes

5.6.4 Steel Deck

The live load deflection shall not exceed the lesser of Span/240 or 1".

5.7 Anchorage

5.7.1 Anchor Bolts and Post Installed Concrete Anchors

- Design of anchors shall be in accordance with ACI 349, Appendix B, Steel Embedments (Ref. 2.1.2).
- Anchorage shall be designed utilizing "Cracked" concrete section properties, except that uncracked section properties shall be permitted when it can be demonstrated that the concrete section is not in a region where flexure or thermal stresses induce cracking.
- Anchors subjected to combined tension and shear shall satisfy the following condition:

$$(P_s / P_t)^{5/3} + (V_s / V_t)^{5/3} \leq 1 \qquad \text{where,}$$

P_s = Applied service tension load
P_t = Allowable tension capacity of anchor
V_s = Applied service shear load
V_t = Allowable shear capacity of anchor

5.7.2 Anchorage of SC-I and SC-II Concrete Walls

Concrete walls shall be anchored to floors / roofs that provide out-of-plane lateral support to the walls. The anchorage shall provide a positive direct connection between the wall and the floor / roof.

The anchorage of the walls shall be designed for the lateral force computed as the product of the wall mass and peak acceleration value of the applicable in-structure response spectrum.

5.8 Story Drift

Story drift for SC-I and SC-II structures shall be based on the provisions of DOE-STD-1020-94 (Ref. 2.2.1), as follows:

- Story drift shall be calculated from a dynamic, elastic analysis.
- Calculated drift shall include translational as well as torsional deflections.
- Calculated story drift shall not exceed 0.01 times the story height for structures with contribution to distortion from both shear and flexure. For structures in which shear distortion is the primary contributor to drift, the calculated story drift shall not exceed 0.004 times the story height.

5.9 Nondestructive Examinations

5.9.1 ANSI/AISC N690 (Ref. 2.1.8) Sections Q1.26.2.1 FULL-PENETRATION WELDS and Q1.26.2.2 PARTIAL-PENETRATION WELDS stipulate that full penetration welds shall be 10 percent inspected by ultrasonic examination or radiographic examination and that partial-penetration welds shall be 10 percent inspected by magnetic particle examination or liquid penetration examination. The examination may be 10 percent of each weld or 100 percent of one weld in ten.

5.9.2 To implement the requirements of ANSI/AISC N690 Sections Q1.26.2.1 and Q1.26.2.2 the following steps shall be taken:

a) Any Full or Partial penetration weld which supports, or is a primary load path for an SC-I or SC-II Structure, System, or Component (SSC), shall conform to the requirements of ANSI/AISC N690 Q1.26.2.1, Full Penetration Welds or Q1.26.2.2 Partial Penetration Welds and be annotated as a **Critical Weld (CTL)**. All other Full or Partial penetration welds shall be considered as a **Non-Critical Weld (NCTL).**

b) All Full and Partial Penetration welds shall be characterized as described above. Any Full and Partial penetration weld classified as a Critical weld shall have 100% visual examination and specific additional NDE requirements as noted on the design drawing in accordance with AISC N690 Sections Q1.26.2.1 and Q1.26.2.2. Full and Partial Penetration welds classified as Non-Critical do not require additional NDE per AISC N690 but are subject to 100% visual examination.

5.9.3 ANSI/AISC N690 Section Q1.26.1.5 stipulates that groove and fillet welds subject to impactive, impulsive or fatigue loads shall meet the visual examination criteria of AWS D1.1 Section 6.9 for cyclically loaded connections (Ref 2.1.14). Welds subject to this special NDE requirement shall be specifically called out on the design drawings as **Cyclically Loaded Connections (CLC).**

5.9.4 Ultrasonic Testing (UT) examination requirements are deleted.

5.10 Finite Element Modeling for Structures

The mesh density used for analysis of concrete structures will be defined in the calculation as appropriate for those elements being analyzed. The following guidelines shall be used for the mesh density in the analysis of concrete structures:

1) For in-plane shear, piers need to have at least three (3) elements vertically.
2) For transverse bending in slabs, element size shall be the smaller of the short side divided by 6 or the long side divided by 8.
3) For transverse bending in walls, element size shall be at least four (4) elements across the span.
4) If transverse bending is important in walls (e.g. exterior walls resisting soil pressure) wall shall be meshed as a slab.
5) Additional mesh refinement shall be provided around discontinuities, unusual openings, and high stress areas that may affect the design forces in those areas.

5.11 Usage Of Inelastic Energy Absorption (F_μ) for Constructed SC-I Structures Designed for the Original DBE Loads

5.11.1 Notations

C_C = Code capacity (overall)
C_S = Code capacity (seismic), calculated as C_C minus D_{NS}
D_{NS} = Non-seismic demand
D_S = Seismic demand (elastic)
D_T = Total demand (elastic), calculated as D_{NS} plus D_S
$F_{\mu\ allowable}$ = Inelastic energy absorption factor from Table 2-4 of DOE-STD-1020-94 (Ref. 4.1.1)
$F_{\mu\ required}$ = Required inelastic energy absorption factor, calculated as D_S divided by C_S

5.11.2 Constructed SC-I concrete structures and constructed steel structures, other than ordinary steel braced frames, designed to ductile detailing requirements of DOE Standards shall be evaluated for the revised DBE load in accordance the following:

(a) Calculate D_S and D_T , based on revised DBE loads.
(b) If $C_C \geq D_T$, the original design is acceptable. However, if $C_C < 1\ D_T$, go to step c).
(c) Calculate $F_{\mu\ required}$, and compare it with $F_{\mu\ allowable}$.
(d) If $F_{\mu\ required}$ is less than $F_{\mu\ allowable}$, and applicable requirements stated in step e) are satisfied, then the original design is acceptable.
(e) For structural elements with $F_{\mu\ required} > 1.0$ which are required for a confinement area, the extent of cracking shall be determined and the ability of the ventilation system to maintain negative pressure shall be evaluated based on the cracking.
(f) Elements where F_μ are used for design shall be tracked and the $F_{\mu\ required}$ and $F_{\mu\ allowable}$ shall be tabulated.

5.12 Constructed Steel SC-II Ordinary Braced Frame Shall Not Use $F_\mu > 1.0$.

6 SC-III and SC-IV Facility Design Requirements

6.1 Reinforced Concrete Design

Reinforced concrete structures shall be designed in accordance with the following:

- Strength Design Method in accordance with the ACI 318 (Ref. 2.1.1).
- Seismic Proportioning and detailing shall be per the provisions of ACI 318 (Ref. 2.1.1) Chapter 21 pertaining to structures in "Moderate" seismic risk regions.

6.2 Structural Steel Design

Steel structures shall be designed in accordance with the following:

- *Allowable Stress Design Method Utilizing the Manual of Steel Construction*, ASD, 9th Edition (Ref. 2.1.5).
- Proportioning and Detailing for seismic loads shall meet the requirements of Chapter 22, Division V, Section 2214, *Seismic Provisions for Structural Steel Buildings in Seismic Zones 1 and 2*, of the UBC (Ref. 2.1.11).
- Steel lateral-force resisting systems in the LAW and LAB structures shall be one of the recognized framing systems from UBC (Ref.2.1.11) Table 16-N. In addition, the minimum level of seismic design and detailing for vertical brace members and their connections shall be Special Concentrically Braced Frame criteria from UBC Section 2213.9. The minimum level of seismic design and detailing for columns, column splices, and chevron bracing beams shall be Ordinary Braced Frame criteria from UBC Section 2214.

6.3 Masonry Design

This section has been deleted because masonry is not used in SC III and IV facilities.

6.4 Load Factors and Load Combinations

Notations

A = Ashfall Load

D = Dead Load

L = Live Load

Lr = Roof Live Load

SN = Snow Load

E = Earthquake (Seismic) Load (per UBC)

H = Lateral Earth Pressure Load

To = Thermal Loads during Normal Operating Conditions

F = Fluid Load

Ro = Operating Pipe Reaction Load

S = Allowable Stress per Allowable Stress Design Method

U = Required Strength per Strength Design Method

W = Wind Load

In addition to the load combinations provided in sections 6.4.1, 6.4.2, and 6.4.3, the following load conditions shall be considered:

(a) Where the structural effects of differential settlement, creep, or shrinkage may be significant, they shall be included with the dead load D in all the load combinations. Estimation of these effects shall be based on a realistic assessment of such effects occurring in service.

(b) Where any load reduces the effect of other loads, the corresponding coefficient for that load shall be taken as 0.9 if it can be demonstrated that the load is always present or occurs simultaneously with other loads. Otherwise, the coefficient for that load shall be taken as zero.

(c) All load combinations shall be checked for zero live load condition.

6.4.1 Reinforced Concrete Design Load Combinations

The following load combinations are based on Section 9.2 of ACI 318 (Ref. 2.1.1).

$U = 1.4D+1.7L+ 1.7L_r+1.7A$

$U = 1.4D+1.7L+ 1.7S_N$

$U = 0.75(1.4D+1.7L+ 1.7L_r+1.7W)$

$U = 0.75(1.4D+1.7L+ 1.7S_N+1.7W)$

$U = 0.9D\pm1.3W$

$U = 1.4D+1.7L+ 1.7L_r+1.7A+1.7H$

$U = 1.4D+1.7L+ 1.7S_N+1.7H$

$U = 0.9D+1.7H$

$U = 1.4D+1.7L+ 1.7L_r+1.7A+1.4F$

$U = 1.4D+1.7L+1.7S_N+1.4F$

$U = 0.9D+1.4F$

$U = 0.75(1.4D+1.7L+ 1.7L_r+1.4T_o+1.4R_o)$

$U = 0.75(1.4D+1.7L+ 1.7S_N+1.4T_o+1.4R_o)$

$U = 1.4(D+ T_o)$

In addition, the following load combinations based on UBC Section 1612.2.1 (Ref. 2.1.11) shall apply.

$U = 1.1(1.2D+L+0.2S_N+1.3F+1.6H+1.2T_o+1.2R_o+E)$

$U = 1.1(0.9D\pm E)$

6.4.2 Structural Steel Design Load Combinations

The following load combinations are based on UBC Section 1612.3.2 (Ref. 2.1.11).

$S = D+L+L_r+A$

$S = D+L+S_N$

$S = 0.75(D+L+W)$

$S = 0.75(D+L+S_N/2+W)$

$S = 0.75(D+L+S_N+W/2)$

$S = 0.75(D+L+E/1.4)$

$S = 0.75(0.9D\pm E/1.4)$

Exception: Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.

6.4.3 Masonry Design Load Combinations

This section has been deleted because masonry is not used in SC III and IV facilities.

6.5 Stability Requirements for Building Structures

SC-III, and SC-IV building structures shall meet the following factors of safety.

Load Combination	Sliding	Overturning
D + H + W	1.5	1.5
D + H + E/1.4	1.5	1.5

6.6 Deflection Limits

6.6.1 Reinforced Concrete Members

Control of deflections in reinforced concrete members shall be in accordance with Section 9.5 of ACI 318 (Ref. 2.1.1)

6.6.2 Structural Steel Members

The allowable deflection and depth to span ratio of fully stressed floor beams, girders, and purlins shall be in accordance with the requirements of Section 5.6.2 of this Criteria.

6.6.3 Crane Runway Support Beams and Monorails

The allowable deflection of runway support beams and monorails shall be in accordance with the requirements of Section 5.6.3 of this Criterion.

6.6.4 Steel Deck

The allowable live load deflection of steel deck shall be in accordance with the requirements of Section 5.6.4 of this Criterion.

6.7 Anchorage

6.7.1 Anchor Bolts and Post Installed Concrete Anchors

Anchorage design of QL SC-III SSCs shall meet the following:

- Design of Anchors shall be in accordance with ACI 349, Appendix B, Steel Embedment (Ref 2.1.2).
- Anchorage shall be designed utilizing "Cracked" concrete section properties, except that uncracked section properties shall be permitted when it can be demonstrated that the concrete section is not in a region where flexure or thermal stresses induce cracking.

- Anchors subjected to combined tension and shear shall satisfy the following condition:

$(P_s / P_t)^{5/3} + (V_s / V_t)^{5/3} \leq 1$ where,
P_s = Applied service tension load
P_t = Allowable tension capacity of anchor
V_s = Applied service shear load
V_t = Allowable shear capacity of anchor

Anchorage design of CM SC-III and SC-IV SSCs shall meet the following:

Anchor rods shall be designed per PCA EB080.01 (Ref. 2.1.7). For situations where geometric restrictions limit breakout capacity, reinforcement proportioned to resist the total load, oriented in the direction of the load, within the breakout prism and fully anchored on both sides of the breakout plane, may be provided instead of calculating the breakout capacity per PCA EB080.01. In lieu of the above, anchors may be designed to ACI 349 Appendix B.

Post-installed anchors may be designed utilizing manufacturer's recommendations for allowable design capacities consistent with commercial industry practice.
The manufacturer's product data will be current and will be listed by the International Conference of Building Officials (ICBO), Factory Mutual (FM) or Underwriters Laboratories (UL).

6.7.2 Anchorage of SC-III and SC-IV Concrete Walls

Concrete walls shall be anchored to floors / roofs that provide out-of-plane lateral support to the walls. The anchorage shall provide a positive direct connection between the wall and the floor / roof.

The anchorage of the walls shall be capable of resisting the largest of the horizontal forces specified in Sections 1611.4, 1632 and 1633.2.8 of the UBC (Ref. 2.1.11). Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet.

6.8 Story Drift

Story drift for SC-III and SC-IV structures shall be based on the provisions of Section 1630.10 of the UBC (Ref. 2.1.11). The quoted section and formula numbers are from the UBC.

(a) Story drift shall be computed using the maximum inelastic response displacements. The maximum inelastic response displacement, Δ_M, is given by Formula (30-17) and Table 16-N, i.e.,

$$\Delta_M = 0.7 \, R \, \Delta_S$$

Δ_S is the total story displacement due to the design seismic forces (strength level) and shall be calculated from a static, elastic analysis of the lateral force-resisting system subjected to the design base shear or determined from an elastic dynamic analysis. Calculated displacement shall include translational as well as torsional deflections.

(b) The design lateral forces used to determine the displacement may disregard the limitations of Formula (30-6) and may be based on the period determined from Formula (30-10) neglecting the 30 or 40 percent limitations of Section 1630.2.2, Item 2.

(c) Calculated story drift using Δ_M shall not exceed 0.025 times the story height for structures having a fundamental period of less than 0.7 seconds. For structures having a fundamental period of 0.7 seconds or greater, the calculated story drift using Δ_M shall not exceed 0.02 times the story height.

7 Materials

7.1 Structural Steel

7.1.1 Carbon Steel material shall be per Table 8.

Table 8 Structural Steel Material Designation

Section(s)	ASTM	F_y (ksi)	F_u (ksi)
W-shapes	A992 or A572 Grade 50[1.0]	50 50	65 65
M-shapes	A36 or A529 Grade 50	36 50	58 70
S-shapes	A36 A529 Grade 50 A572 Grade 50 A709 Grade 36 A709 Grade 50	36 50 50 36 50	58 70 65 58 65
HP-shapes	A36 A529 Grade 50 A572 Grade 50 A709 Grade 36 A709 Grade 50	36 50 50 36 50	58 70 65 58 65
Channels	A36 or A529 Grade 50 A572 Grade 50 A709 Grade 36 A709 Grade 50	36 50 50 36 50	58 70 65 58 65
Angles	A36 or A529 Grade 50 A572 Grade 50 A709 Grade 36 A709 Grade 50	36 50 50 36 50	58 70 65 58 65
Structural Plate & Bars	A36 or A529 Grade 50 A572 Grade 50 A709 Grade 36 A709 Grade 50	36 50 50 36 50	58 70 65 58 65
Structural Tees	(per source of split section)		
Steel Pipe	A53	35	60
Round HSS	A500 Grade B	42	58
Square & Rectangular HSS	A500 Grade B [2.0] A500 Grade C	46 50	58 62
Anchor Rods	F1554	36/55/105	58/75/125
Welded Studs	A108[3.0]		
Checkered Plate	Low Carbon Commercial Quality	30	
Steel Deck (Galvanized)	A653	33 to 50	
Rails	A1 A759 DIN 536 Grade S1100 DIN 5901	50 50 80 50	

[1.0] With Special Requirements per AISC Technical Bulletin #3, Dated March 1997

[2.0] Use HSS section properties as published per 1997 HSS Manual or AISC 2001 LRFD manual.

[3.0] Use manufacturer values

7.1.2 Stainless Steel Material:

7.1.2.1 Shapes and Bars: Conform to ASTM A276, Type 304L.

7.1.2.2 Plates: Conform to ASTM A240, Type 304L or 316L

7.1.2.3 Pipes: Conform to ASTM A312, Grade TP 304L.

7.1.2.4 Tubing: Conform to ASTM A554 Grade MT-304L; Tensile strength 70 ksi minimum, yield strength 25 ksi minimum

7.2 Concrete and Reinforcing

- Concrete: Compressive Strength, f'_c = 4000 psi, minimum.
- Reinforcing Steel: ASTM A706, deformed.
- Welded Wire Fabric: ASTM A185.

7.3 Deleted

7.4 Structural Bolting Materials

Structural bolting shall be limited to the following:

7.4.1 For carbon steel members

- ASTM A325, Type 1 with A563 Heavy Hex Nuts, Grade DH
- ASTM A490, Type 1 with A563 Heavy Hex Nuts, Grade DH
- ASTM F436, Type 1 for washers
- ASTM F1852 for Twist Off Type Tension Control Bolt Assemblies

Bolting of members that are not considered to be part of the main building structure, i.e., stair or platform connections, may utilize ASTM A307 bolts Grade A or SAE J429 Grade 5. Nuts shall conform to ASTM A563 Grade A or SAE J995 Grade 5 Hex.

Structural connections shall be Bearing Type connections except where Slip Critical connections are essential. Sizes for structural bolting material should be limited to 7/8" diameter for all A325 bolts or 1-1/8" diameter for A490 bolts.

7.4.2 For stainless steel members

- Bolts: Conform to ASTM A193 Grade B8 or B8M.
- Nuts: Conform to ASTM A194 Grade 8 or 8M, heavy hex.
- Washers: Cut from plate conforming to ASTM A240 Type 304L or machined from bar stock that meets ASTM A193 B8 or B8M, Class 1. Dimensions in accordance with ASTM F436.

7.5 Welding Material

Unless specifically noted otherwise on the design drawings, welding material for carbon steel shall be in accordance with AWS D1.1 with a nominal tensile strength of 70 ksi. Welding material for stainless steel shall be in accordance to Table Q1.17.2.1 of ANSI/AISC N690 and Table 3.3 of AWS D1.6.

7.6 Material Design Properties

The following values are to be used in analysis of steel and concrete structures:

Steel:	Modulus of Elasticity	$E = 29 \times 10^6$ psi
	Poisson's Ratio	$\nu = 0.3$
Concrete (4000 psi):	Modulus of Elasticity	$E = 3.834 \times 10^6$ psi (see note)
	Poisson's Ratio	$\nu = 0.17$
Concrete (5000 psi):	Modulus of Elasticity	$E = 4.287 \times 10^6$ psi (see note)
	Poisson's Ratio	$\nu = 0.17$
Concrete (6000 psi):	Modulus of Elasticity	$E = 4.415 \times 10^6$ psi (see note)
	Poisson's Ratio	$\nu = 0.17$

Note: For purpose of design calculations, the following material density values shall be used:

Concrete	150 pcf
Steel	490 pcf
Ashfall	48 pcf (Ref. 2.4.9)

7.7 Geotechnical Design Parameters and Foundation Design Recommendations

7.7.1 Geotechnical Design Parameters

The geotechnical design parameters provided below and in Table 9 and Table 10 are obtained from the Geotechnical Investigation Report by Shannon and Wilson (Ref. 2.4.7).

Table 9 Summary of Static Modulus Values

Material	Moist Density (pcf)	Mean Friction Angle (degree)	Elastic Modulus (E) (ksi)	Constant of Horizontal Subgrade Reaction (n_h) (pci)	Coefficient of Subgrade Reaction (K_{s1}) (pci)	Vertical Modulus of Subgrade Reaction (K) (pci)
Dune Sand	116 ±5	35 ±2	NR[3]	20 ±5	NR[3]	NR[3]
Structural Fill	130 ±5	41 ±8	37 ±13	30	250	25
Hanford Sand Upper Unit	115 ±5	40 ±3	62 ±16	30 ±5	260 ±15	60 ±12[1] 100 ±14[2]
Hanford Sand Lower Unit	110 ±5	40 ±3	90 ±15	45 ±8	390 ±25	160 ±32

1 Main Building Mat Foundations (Pretreatment, HLW, LAW)

2 Pits within Hanford Upper Sand Unit

3 NR – Not Recommended to Support Vertical Loads

Table 10 Summary of Friction and Lateral Earth Pressure Coefficients

Material	Moist Density (pcf)	Mean Friction Angle (degree) (θ)	Mean Friction Coefficient (δ)	Recommended Design Friction Coefficient[3] (δ)	Mean Active Earth Pressure Coefficient (K_a)	Mean At-Rest Earth Pressure Coefficient (K_o)	Mean Passive Earth Pressure Coefficient (K_p)
Dune Sand	116 ±5	35 ±2	0.70	0.43	0.27	0.43	3.7
Structural Fill	130 ±5	41 ±8	0.87	0.50	0.21	0.34	4.8
Hanford Sand Upper Unit	115 ±5	40 ±3	0.84	0.50	0.22	0.36	4.6
Hanford Sand Lower Unit	110 ±5	40 ±3	0.84	0.50	0.22	0.36	4.6
Hanford Gravel	120 ±5	40 ±3	0.84	0.50	0.22	0.36	4.6

Notes:

1. The mean values presented are based on the mean friction angle. Mean earth pressures are obtained by multiplying density by earth pressure coefficient.
2. $\delta = \tan \theta$ (friction angle)
3. Includes factor of safety.

7.7.2 Foundation Design Recommendations

1. For exterior footings the frost depth is 2'-6". Frost depth does not apply to interior footings of heated structures.
2. No foundation shall bear directly on the dune sand.
3. The foundations shall bear in either structural fill or the Hanford Sand.
4. Foundation for Lab, Pretreatment, HLW Vitrification, LAW Vitrification ,ITS SwitchgearBuildings, ITS Diesel Generators and ITS Fuel Oil Storage Vessels:
   a) Design as a mat on elastic foundation using soil springs.
   b) The dune sand below the foundation shall be completely removed. The excavation shall extend outside the foundation footprint for a distance equal to D/2 or 5 ft., whichever is smaller. "D" is the depth of excavation below the foundation.
5. Isolated Spread Footings:
   a) Bearing failure governs the design.
   b) Footing dimension shall not exceed 20 feet.
   c) Allowable soil bearing capacity shall be per Figure 9-1 of the Geotechnical Report (Ref. 2.4.7). Bearing pressure on the soil shall not exceed 5 ksf for normal working loads. The allowable bearing capacity may be increased by 1/3 for short duration, transient, seismic or wind loads.
   d) The dune sand below the foundation shall be removed in accordance with Figure 11-1 of the Geotechnical Report (Ref. 2.4.7). The excavation shall extend outside the foundation footprint for a minimum distance equal to D/2. "D" is the depth of excavation below the foundation.
6. For BOF buildings as identified in Table 11:

   For non-safety related BOF Buildings/Facilities either Method A or B, as listed in Table 11, shall be used. Method B is based on settlement criteria as listed in Note 1 to Table 11. Method B shall be used only if the settlement criteria for a building/facility meets the settlement limits stated in Note 1 to Table 11. Method A shall be used if the settlement criteria for the building/facility is less than the criteria listed in Note 1 to Table 11

   a) Method A - Follow the guidelines set forth in the original soil report (Ref 2.4.7) by overexcavating up to 5 feet beneath the footing bottom and replacing with structural fill
   b) Method B - Prepare footing foundations by compacting the existing soils to a depth of 2 feet below the bottom footing elevation. This will require overexcavating approximately one foot beneath the bottom footing elevation and compacting the exposed subgrade to 95% of its maximum dry density (as determined by ASTM D1557), followed by replacement and compaction of the overexcavated zone using structural fill.

Table 11 WTP BOF Building/Facility Foundations (Non-Safety Related)

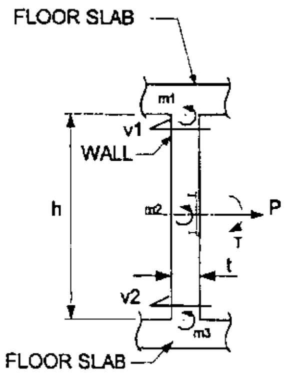
Item	Building / Facility	Estimated Bearing Pressure	Recommended Method of Foundation Preparation (See Note 1 below)
1	**Cooling Tower Basin Foundations**	2 to 3 ksf	A – settlement sensitive structure
2	**Heavy Loaded Pre-Engr Bldg Foundations** (i.e. Steam Plant)	3 to 5 ksf	B
3	**Heavy Loaded Foundations** (i.e. Melter Assembly Slab, Spent Melter Slab)	3 to 5 ksf	B
4	**Medium Loaded Pre-Engr Bldg Foundations** (i.e. Water Treatment, Wet Chemical Bldg)	3 to 4 ksf	B
5	**Light Loaded Pre-Engr Bldg Foundations** (i.e. Switchgear, BOF Switchgear)	2 to 3 ksf	B
6	**Glass Former Silo Foundations**	3 to 4 ksf	B
7	**Tank Ring Wall Foundations**	2 to 3 ksf	B
8	**Standby Diesel Generator Foundations**	3 to 4 ksf	B
9	**Tank Slab Foundations** (Water Tanks)	2 to 3 ksf	B
10	**Pump Houses Slab Foundations**	1 to 2 ksf	B
11	**Light Loaded Yard Equipment Foundations** (i.e. Electrical Panels, Rectifier, HVAC Units)	1 to 2 ksf	B – No frost depth requirement. Compact 2 feet below bottom of footing only.
12	**Heavy Loaded Yard Equipment Foundations** (i.e. Transformers,)	2 to 3 ksf	B
13	**Yard Utility Rack Foundations**	8 to 10 ksf	N/A
14	**Heavy Rotary Equipment Foundation in Chiller Compressor Bldg**	3 to 4 ksf	A – potential settlement from vibrations
15	**Heavy Tank or Equipment Foundations** (i.e. inside Steam Plant, Wet Chemicals)	3 to 4 ksf	B

Note: 1. The estimated bearing pressures were determined assuming an allowable total settlement of 1 inch and a maximum differential settlement of ½ inch between adjacent supports. In the case of ring wall foundations for large storage tanks (Item 7) the differential settlement was determined for ½ - inch over a distance of 40 ft.

Appendix A - Deleted

Appendix B - Concrete Wall Out-of-Plane Embedment Load Guidelines

Appendix B CONCRETE WALL OUT-OF-PLANE EMBEDMENT LOAD GUIDELINES



OUT OF PLANE EQUIPMENT/COMMODITY LOADS

Wall = 12" thk	12" < Wall < 24" thk	Wall > or = 24" thk
P = 10 k or T = 25 ft-k	P = 25 k or T = 70 ft-k	P = 40 k or T = 100 ft-k

1. The above out of plane embed loads are unfactored and shall be classified as DL for Reinforced Concrete Design Load Combinations. Loads shall be applied in each direction.

2. These loads are in addition to all other design loads, applied at mid-height to calculate the internal bending moments m1 through m3. Only one Out-of-Plane load need be applied between floors as shown.

3. Effective width for Major Equipment/ Commodity Embeds shall be taken as the smaller of 4 x concrete thickness, embed spacing or 7 feet maximum.

4. The above loads shall be applied to all walls. Walls with Minor Equipment/ Commodity Embeds shall use the above loads with the effective width equal to the lesser of 4 x concrete thickness or 7 feet.

5. Where a refined structural analysis determines lower internal shear and bending moments, these lower forces may be used.

m1 to m3 ≥ T/(8 x t)
≥ T/(2 x EMBED SPACING)
≥ T/14
≥ Ph/(32 x t)
≥ Ph/(8 x EMBED SPACING)
≥ Ph/56

v1 & v2 ≥ T/(4xhx t))
≥ T/(h x EMBED SPACING)
≥ T/(7xh)
≥ P/(2x4x t)
≥ P/(2 x EMBED SPACING)
≥ P/2(7)

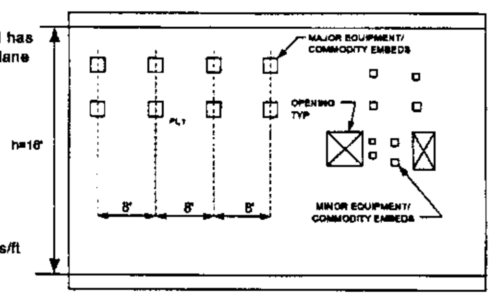
Example Problem: Find the internal wall moments m1 to m3 due to the following wall embed layout using the above out of plane embed loading criteria.
Wall thickness =36"

Solution: By inspection major equipment Plate PL1 has the least tributary width. Find the out of plane wall moments based on plate PL1.

For wall thickness > or = 24"
P=40 kips
T=100 ft-kips

m1 to m3 ≥ T/(8xt) = 100/8x3=4.2 ft-kips/ft
≥ T/2(Embed spacing)=100/16= 6.3 ft-kips/ft
≥ T/(14)=100/14= 7.1 ft-kips/ft
≥ Ph/(32xt)= 40x16/(32x3)= 6.7 ft-kips/ft
≥ Ph/(8xEmbed spacing)= 40x16/(8x8)= 10.0 ft-kips/ft
≥ Ph/(56)= 40x16/(56)= 11.4 ft-kips/ft
Controls

Conclusion: Use (m1 to m3) = 11.4 ft-kips/ft for the out of plane embed moment.



WALL EMBED LAYOUT

v1 & v2 = Internal shear force (kips)
m1 to m3 = Internal wall moment (ft-kips/ft)
P = Horizontal embed force (kips)
T = Embed moment (ft-kips)
h = Clear height of wall (ft)

Appendix C- Strength and Modulus Reduction for Structural Steel Grade A36

The equation below is from ASCE Manuals and Reports on Engineering Practice No. 78, Structural Fire Protection, Appendix A.1.2.2 for 0°C < Temperature < 600°C (32°F < Temperature < 1112 °F)

For A36 Steel: Tensile yield strength, $F_y := 36 \cdot ksi$ Modulus of Elasticity, $E := 29 \cdot 10^3 \cdot ksi$

$$F_{yReduction_i} := \left[1 + \frac{\frac{5}{9}\cdot(Temp_i - 32)}{900 \cdot \ln\left[\frac{\frac{5}{9}\cdot(Temp_i - 32)}{1750}\right]}\right]\cdot F_y \qquad E_{Reduction_i} := \left[1 + \frac{\frac{5}{9}\cdot(Temp_i - 32)}{2000 \cdot \ln\left[\frac{\frac{5}{9}\cdot(Temp_i - 32)}{1100}\right]}\right]\cdot E$$

The tensile strength and modulus of elasticity of steel decrease with increasing temperature as shown below:

Tensile Yield Strength $F_{yReduction}$ (ksi)	Temperature °F (Temp)	Modulus of Elasticity $E_{Reduction}$ (ksi)
35.2	150	$28.7 \cdot 10^3$
34.7	200	$28.5 \cdot 10^3$
34.2	250	$28.2 \cdot 10^3$
33.6	300	$27.9 \cdot 10^3$
32.2	400	$27.2 \cdot 10^3$
30.5	500	$26.4 \cdot 10^3$
28.6	600	$25.3 \cdot 10^3$
27.6	650	$24.7 \cdot 10^3$
26.4	700	$24 \cdot 10^3$
25.2	750	$23.3 \cdot 10^3$
23.9	800	$22.5 \cdot 10^3$
22.5	850	$21.5 \cdot 10^3$
21	900	$20.5 \cdot 10^3$
19.5	950	$19.4 \cdot 10^3$
17.8	1000	$18.1 \cdot 10^3$

Attachment 2
06-WTP-065

Seismic Ground Motion Issue Report for the Waste Treatment and Immobilization Plant (WTP) Hanford, Washington, Revision 2
June 2006

WED:WA
May 16, 2006

SEISMIC GROUND MOTION ISSUE REPORT FOR THE WASTE TREATMENT AND IMMOBILIZATION PLANT (WTP) HANFORD, WASHINGTON

Revision 2

June 2006

Wahed Abdul
John Treadwell

U.S. Department of Energy
Office of River Protection

Executive Summary

The seismic hazard at any site is a function of the location and geometry of potential sources of future earthquakes, the frequency of occurrence of various size earthquakes on these sources, and the characteristics of seismic wave propagation in the region. A seismic hazard model consists of two basic components: a model of the sources of potential future earthquakes and a model of the effects at the site due to the potential of future earthquakes. The model for the Hanford Waste Treatment and Immobilization Plant (WTP) project used a probabilistic framework to address the randomness of the earthquake process and other uncertainties. Due to the lack of historical strong motion data at Hanford, empirical attenuation models also had to be used in evaluating the ground motion hazard. Accordingly, early seismic hazard analysis and resulting ground motion spectra used in the WTP designs had an element of uncertainty that led to several issues over the ensuing years. This report addresses how the WTP has addressed the issues and the basis for continuing design. Figure ES-1 provides the timeline for revised ground motion (RGM) development and implementation.

DOE selected DOE-STD-1020-94[1] as the seismic standard for the WTP facility in 1997. In 1999, after extensive reviews, the U.S. Department of Energy (DOE), Office of River Protection (ORP) approved seismic hazard analysis conducted in 1996 by Geomatrix Consultants, Inc. as the design basis for seismic category (SC)-I and -II.

The Defense Nuclear Facilities Safety Board (DNFSB) staff has questioned the assumptions used in developing the seismic design basis, particularly the adequacy of the site geotechnical surveys and the attenuation relationships. After review and discussion by outside experts, concerns remained. The DNFSB considered the existing site-specific shear wave velocity data insufficient to reliably use California earthquake response data for predicting ground motions at the Hanford Site.

In 2004, ORP initiated a re-evaluation of the seismic ground motion by Pacific Northwest National Laboratory, which included acquiring new site-specific wave velocities and other dynamic soil data by drilling five new boreholes down to approximately 500 feet; re-analysis of the effects of deeper layers of basalt interbedded with sediments (down to a depth of 1,500 feet); and application of new site response models for ground motion estimation. This re-evaluation had uncertainty due to the absence of adequate data for characterizing the deeper, interbedded basalt and sediment layers in and around the WTP site. This uncertainty in site response was accounted for by evaluating the potential effects of several site parameters on the velocity profile to arrive at what is considered a conservative estimate of site response. This re-evaluation was completed in January 2005, and resulted in the RGM spectra, which increased the peak of the existing spectra by approximately 40%.

This large increase posed a major challenge to the WTP project regarding design revalidation, configuration control, and minimizing the risk of rework, because the project was already under construction, with parts of the facility structures constructed and additional systems, structures and components already designed and/or fabricated. DOE directed the project to retard or stop various construction and procurement activities based on the evaluation of risk of rework resulting from redesign, and approved limited construction of concrete structures on a case-by-case basis. It was determined that the new dynamic analysis for the facility structures would take upwards of six months to provide forces for the redesign effort, which in turn was estimated to take a few years to complete. Working with DOE and their consultants, Bechtel National, Inc. (BNI) developed bounding interim seismic criteria to carry on limited designs and construction to avoid significant cost and schedule impact to the project while developing a long-term implementation plan to ensure incorporation of RGM in the design in a controlled manner.

[1] DOE-STD-1020-94, 1996, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities*, Change Notice #1, U.S. Department of Energy, Washington, D.C.

To minimize the impact of the RGM on the already constructed, fabricated, and future design of structures in the redesign effort, BNI developed and ORP accepted several white papers to justify reduction or elimination of embedded additional conservatisms that were deemed "no longer required" due to the maturity of design. This required major changes to the analysis and design criteria, procurement specifications, etc. A change of the primary software for the analysis of facility structures, from GTStrudl to SAP2000 was made. This addressed a concern by the DOE Peer Review Team (PRT) and the DNFSB, regarding the adequacy of the finite element mesh sizes for the facility design. All of these changes have been concurred by the DOE PRT, DNFSB staff, and have been incorporated in the WTP Structural Design Criteria in December 2005.

Dynamic seismic analyses were completed in September 2005. Structural design criteria have been revised to incorporate all pertinent changes. In December 2005, the structural redesign of the facilities, using the results of the dynamic analyses and the revised design criteria, was initiated and the use of interim seismic criteria has been stopped. Initial results from the dynamic seismic analyses indicate that the original facility designs may not require significant modifications. However, piping and vessel designs would require some modifications.

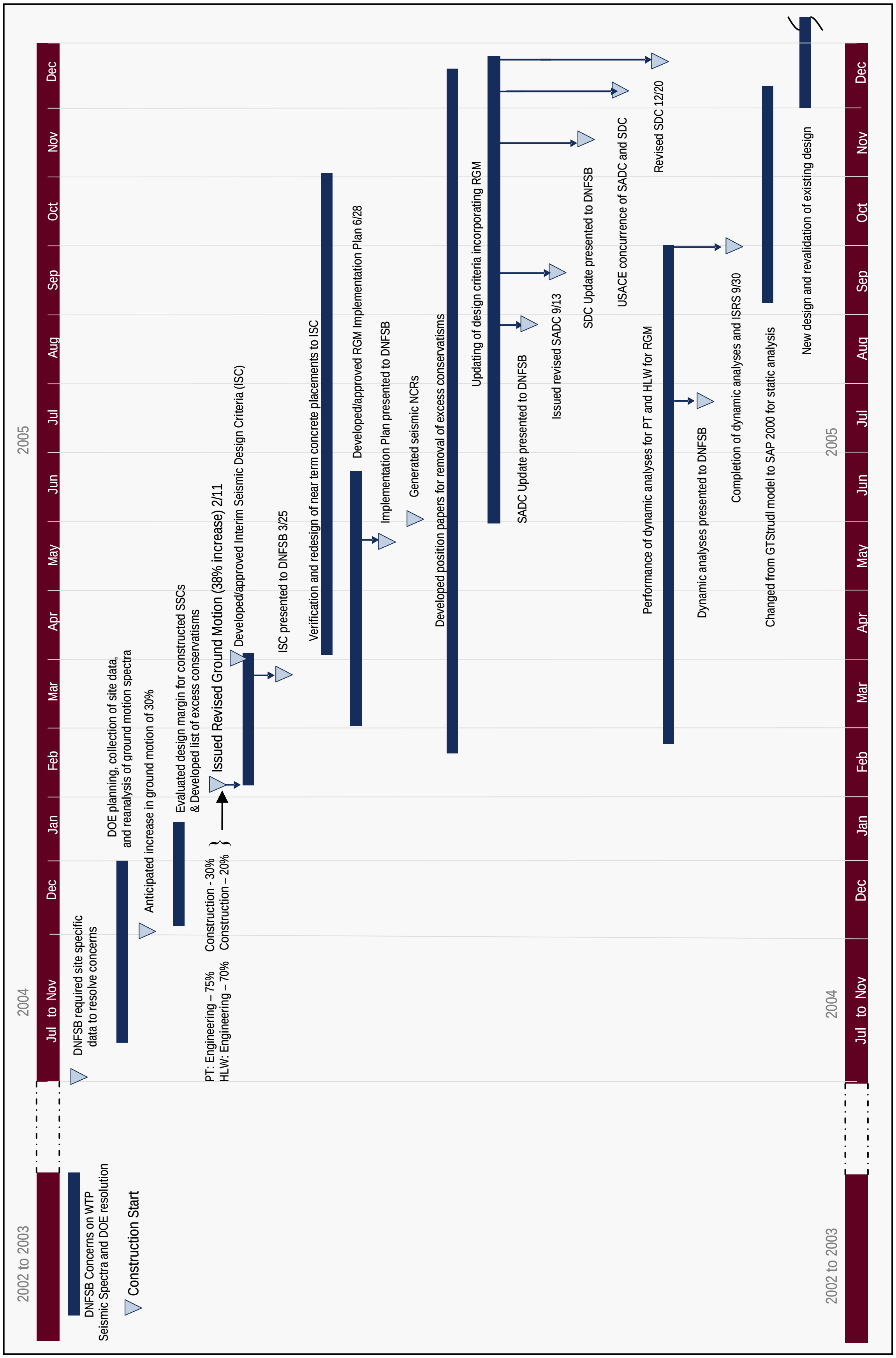
Uncertainty in the RGM was considered by DOE as being already bounded by the method of the development of the RGM, where the 84th percentile relative amplification function (RAF) was used in the response to define a conservative representation of the mean surface ground motion. However, due to further DNFSB concerns over the lack of sufficient site data in the development of the RGM, DOE has made the decision to perform deep bore drilling at the site to enhance direct estimates of subsurface dynamic properties. This decision was made in part to confirm that the RGM spectra are a conservative representation of the mean spectra. It is anticipated that with the improved definition of the properties of the site profile from the deep drilling program, the mean spectrum will be less than the current revised design spectra based on the 84th percentile RAF. However, the extent of reduction cannot be quantified until the deep drilling program is completed. The drilling effort to reach to the depth of 1,500 feet and to perform the associated analyses, completion of the confirmatory analysis for the seismic ground motion is anticipated to be completed in mid-2007. Two of the borehole drilling will be completed in 2006.

In addition to the reviews by DOE PRT, an external, independent review of the design by U.S. Corps of Engineers (USACE) was initiated in August 2005. Reviews by these teams are ongoing and plan to be continued in 2006. The independent USACE review team has concurred with the implementation of the revised Structural Design Criteria (revision 10), and Seismic Analysis and Design Criteria (revision 3), incorporating the RGM and the reduction of conservatisms, in the redesign effort.

DOE considers the currently recommended RGM for the WTP site to be a conservative estimate of the mean seismic hazard. It is noteworthy that the demand to capacity ratios in many of the major walls are significantly less than 1.0. This allows accommodation of transfer of loads without exceeding allowable code criteria, which provides added assurance of acceptable structural behavior during an earthquake, even if the future ground motion exceeds the design ground motion. The combination of multiple lateral load path capability in the design, together with the use of ductile detailing and the availability of untapped inelastic energy absorption characteristics of the structural elements indicate that the WTP facilities can absorb further increase in seismic ground motion, should the guidelines change in future. In addition, the facility structures could be validated for significantly higher loads using coherency concepts, complex fragility or push-over analysis of structures, in the unlikely event of a future ground motion significantly larger than the RGM.

As of February 2006, significant funding reductions from Congress have resulted in stopping construction on both the Pretreatment and High-Level Waste Facilities until late in the fiscal year. Facility design and re-validation of existing designs against the current RGM will be continued during this time to help mature the design, which will provide further assurance that the risk of proceeding with the project does not result in unacceptable risk.

Figure ES-1. Timeline for RGM Development and Implementation



SEISMIC GROUND MOTION ISSUE REPORT FOR THE WASTE TREATMENT PLANT, HANFORD, WASHINGTON

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LIST OF TERMS

ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
BNI	Bechtel National, Inc.
BOF	balance of facilities
CBRG	Columbia River Basalt Group
DBE	design basis earthquake
D/C	demand-to-capacity
DNSFB	Defense Nuclear Facilities Safety Board
DOE	U.S. Department of Energy
DRS	design response spectrum
F_{μ}	In-elastic energy absorption factors
HLW	high-level waste
HVAC	heating, ventilation, and air conditioning
IFC	issued for construction
ISC	interim seismic criteria
ISRS	in-structure response spectra
NCR	nonconformance report
NGA	Next Generation Attenuation
NPH	natural phenomena hazard
NPP	nuclear power plant
NRC	U.S. Nuclear Regulatory Commission
ORP	Office of River Protection
OSR	Office of Safety Regulation
PEER	Pacific Earthquake Engineering Research
PNNL	Pacific Northwest National Laboratory
PRT	Peer Review Team
PSHA	probabilistic seismic hazard analysis
PT	Pretreatment [Facility]
RAF	relative amplification function
RGM	revised ground motion
SADC	seismic analysis and design criteria
SASSI	System for Analysis of Soil-Structure Interaction
SASW	spectral analysis of surface waves
SC	seismic category
SDC	structural design criteria
SER	safety evaluation report
SRD	safety requirements document
SSC	system, structure and component
SSI	soil-structure interaction
SSR	summary structural report
UHS	unbroadened horizontal spectra
USACE	U.S. Corps of Engineers
Vp	compression wave velocity
Vs	shear wave velocity
WHC	Westinghouse Hanford Company
WTP	Waste Treatment and Immobilization Plant
ZPA	zero period ground acceleration

1.0 PURPOSE

The purpose of this report is to provide an understanding of the evolution of the seismic ground motion issue at the Waste Treatment and Immobilization Plant (WTP) project, and to support the current strategy for completing WTP design and construction. The report includes discussion of specific and related concerns raised by reviewers relevant to this issue, and it presents the actions implemented by WTP to resolve those concerns. The report also includes the impact of the ground motion issue to the project, the actions taken to mitigate its impact, identification of remaining uncertainty and the rationale for and discussion of the path forward for completion of the design and construction of the WTP.

2.0 INTRODUCTION

In 1997, DOE selected DOE-STD-1020-94, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities*, as the seismic standard for the WTP facility, using the contractually required standards-based integrated safety management selection process. The U.S. Department of Energy (DOE) Office of Safety Regulation (OSR) approved this selection in 1997.

In 1999, the DOE Office of River Protection (ORP) approved the seismic design basis for the WTP planned for construction in the 200 East Area on the Hanford Site near Richland, Washington. The seismic design basis was based on a 1996 study by Geomatrix Consultants, Inc. (Geomatrix 1996), which refined the seismic hazard model for the region that was begun in 1981 for the Washington Public Power Supply System's reactor sites, and that was subsequently updated to accommodate the latest seismic considerations in 1989 and 1993 – 1996. The Geomatrix study had undergone validation reviews by BNFL, Inc., and their subcontractor Bechtel National, Inc. (BNI), and independent review by seismologists from the U.S. Corps of Engineers (USACE) and Lawrence Livermore National Laboratory prior to ORP acceptance. The study was also consistent with the national probabilistic seismic hazard maps completed by the U.S. Geologic Survey that was used in part to develop recommended provisions for seismic regulations for new buildings and other structures (Federal Emergency Management Agency standards). Subsequently, the same criteria were adopted in 2001 for the new contract.

Based on the Geomatrix probabilistic seismic hazard analysis (Geomatrix 1996), the seismic design basis was developed using the methodology described in DOE-STD-1020-94. A 2,000-year recurrence interval was selected because the highest category of WTP facilities is Performance Category 3, in accordance with DOE-STD-1020-94, having significant radiological hazard (Hazard Category 2), although less than a nuclear reactor. The resulting site-specific seismic spectra adopted bounding zero period ground acceleration (ZPA) of 0.26 g horizontal at 50 Hz, and 0.18 g vertical at 50 Hz.

The Defense Nuclear Facilities Safety Board (DNSFB), an independent federal agency established by Congress in 1988, subsequently initiated a review of the seismic design basis of the WTP. In March 2002, the DNFSB staff questioned the assumptions used in the development of the seismic design basis, particularly the adequacy and applicability of the site geotechnical surveys. These questions were addressed, but in additional meetings and discussions through July 2002, additional questions were raised about the local probability of earthquakes, the adequacy of the site geotechnical surveys, and attenuation relationships. Attenuation relationships describe how ground motion changes as it moves from its hypocenter to the site location. ORP addressed these issues in the position paper ORP/OSR-2002-22 in August 2002. However, as a result of further DNFSB questions regarding the adequacy of seismic modeling, Pacific Northwest National Laboratory (PNNL) was contracted to perform a further re-evaluation of the impact of basalt/sediment interbeds (as well as the thickness and character of the surface sediments directly below WTP) on the ground motion at WTP. In 2004, new site-specific soil data was taken, and revised estimates of local site amplification factors applied. In February 2005, ORP issued the new seismic design basis for the WTP based on a re-evaluation of the interbed shear wave velocity contrasts being greater than assumed in Geomatrix 1996. This resulted in the revised ground motion (RGM) spectra that are from 15% to 40% higher than the previous design basis spectra over frequency ranges important to the structural response. The RGM spectra are now being implemented for design of the WTP seismic category (SC)-I and SC-II structures.

Since the magnitude of the increase was significant, this change had a major impact on the WTP project. The WTP project was already in construction, with parts of the facility structures constructed and additional systems, structures and components (SSC) already designed and/or fabricated. This posed a challenge to the project from design revalidation, configuration control etc., and required significant planning and assessment of the SSCs. This report delineates the activities that led to the revision of the ground motion spectra, and the actions taken towards re-analysis for the incorporation of the RGM while ensuring that the WTP is in a safe configuration and the risk for rework is minimized.

The next section is added to elaborate the DOE process for design and evaluation for the Natural Phenomena hazards, which includes the earthquake hazard, since, it is a complex process. The elaboration only includes the earthquake hazard of the NPH hazards. In addition, it delineates the process and details followed at WTP.

3.0 PROCESS FOR DESIGN AND EVALUATION OF NATURAL PHENOMENA HAZARDS FOR DOE FACILITIES

In accordance with the DOE O 420.1B, *Facility Safety*, and associated guide DOE G 420.1-2, *Guide for the Mitigation of Natural Phenomena Hazards*, an analysis of DOE facilities and operations is required to ensure that SSCs and personnel will be able to perform their intended safety functions effectively under the effects of natural phenomena hazards (NPH).

Four standards provide specific acceptance criteria for various aspects of NPH to implement the NPH mitigation requirements to meet DOE O 420.1B requirements. Figure 1 shows a conceptual NPH design framework, which identifies how the DOE NPH standards are used to assess NPH design requirements.

- The studies of site characteristics are to be performed and the existing data for site characteristics related to NPH to be evaluated in accordance with DOE-STD-1022, *Natural Phenomena Hazards Site Characterization*.

  The site characterization for NPH (seismic) hazard assessment requires investigation of site earthquake ground motion. The extent of the investigation is dependent upon the performance categories of the structures, the geological and seismologic environment of the site region, and the local soil conditions at the site. All seismic sources in the site region that could cause significant ground motion at the site are required be identified and characterized to provide the location, size, and geometry of the seismic sources, maximum earthquake, and frequency of occurrence of earthquakes of various magnitudes (earthquake recurrence).

  Site investigations are required to be conducted for (1) defining site soil properties required for hazard evaluations, and engineering analyses and designs; (2) assessing local soil site effects on ground motions; (3) carrying out soil-structure interaction analyses; and (4) assessing potential of soil failure or deformation induced by ground motion (liquefaction, differential compaction, land sliding, etc.). Site investigations are to include subsurface exploration by borings, soundings, well logs, geophysical survey, etc., and laboratory tests to determine static and dynamic and soil properties.

- Performance of the site-specific NPH hazard assessment is to be performed in accordance with the DOE-STD-1023-95, *Natural Phenomena Hazards Assessment Criteria*, using the site characterization from above to ensure that adequate design basis loads are established. Different sets of NPH loads are generated for facilities with different performance categories and target probabilistic performance goals. These loads are used for NPH design and evaluation of respective facilities in accordance with DOE-STD-1020-94.

  This standard describes methods for conducting a probabilistic seismic hazard analysis (PSHA) to produce a seismic hazard curve to be used in selecting the design basis earthquake (DBE) for PC-3 and PC-4 SSCs. A necessary part of seismic design is the selection of one or more design levels of ground motion. Because of the random nature of earthquakes, a design level of ground motion inherently has a probability of occurrence associated with it. The methodology to conduct a new PSHA has four components: (1) basic hazard model; (2) data used in the hazard modeling; (3) characterization of uncertainty in parameters of the hazard model; and (4) quantification of uncertainty. A new PSHA must incorporate random variability in earthquake location, and size and ground motions associated with future earthquakes. In addition, a component of uncertainty related to lack of knowledge of the models and parameters of seismic hazards must be quantified.

  These uncertainties result in a family of seismic hazard curves representing spectral acceleration versus frequency (or period) for different recurrence intervals, from which the median (50^{th} percentile) or mean seismic hazard may be selected.

- NPH performance categories for the facilities are specified and established based on the significance of the associated hazards in accordance with DOE-STD-1021-93, *Natural Phenomena Hazards Performance Categorization Guidelines for Structures, Systems, and Components*. Performance categories (PC) and performance goals vary for facilities based on the extent of hazardous materials or type of operations.

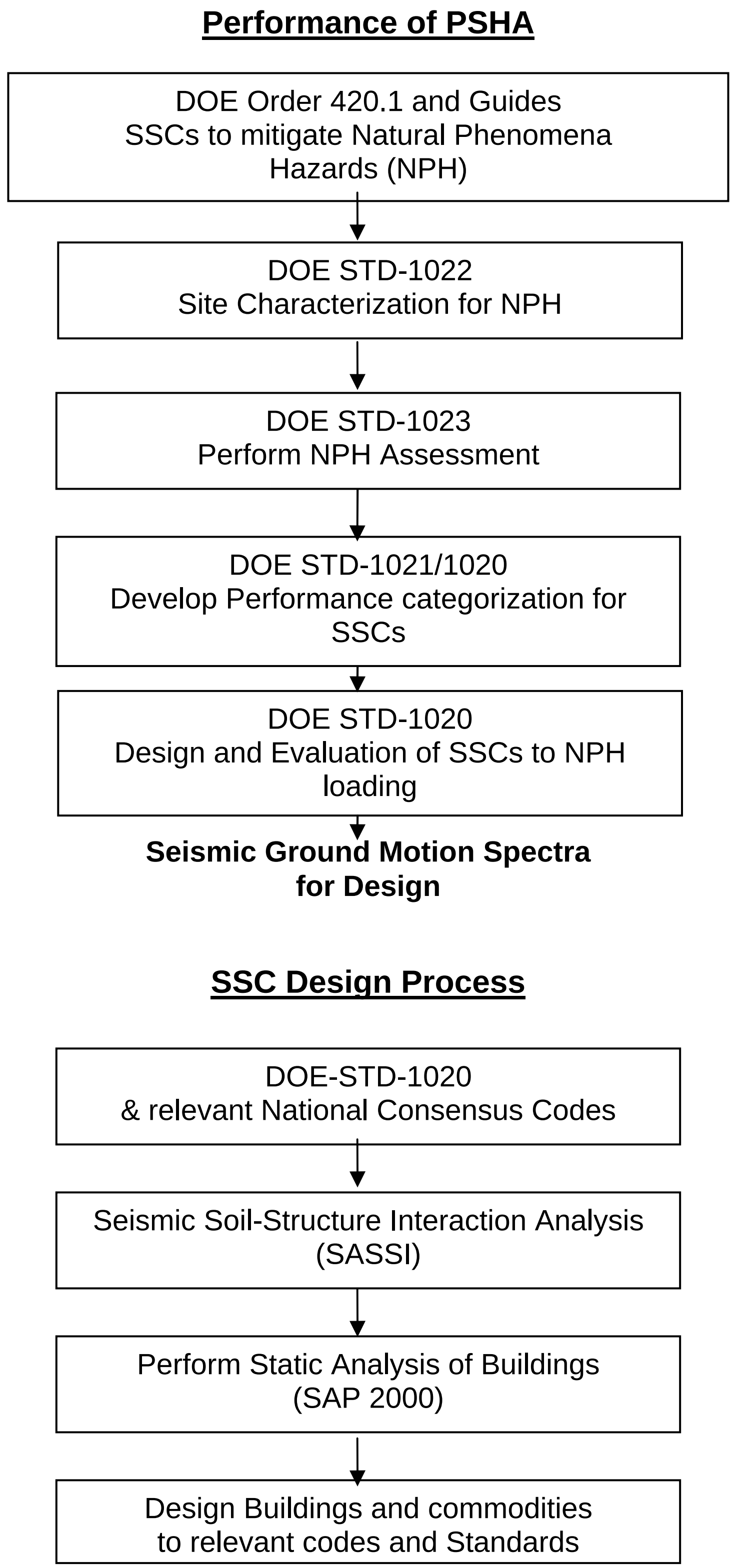
  DOE-STD-1021-93 provides guidelines for selecting SSC performance categories that are consistent with the requirements of 10 CFR 830, "Nuclear Safety Management." Hazard and performance category designation follows a graded approach, and is based on the adverse offsite/onsite consequences from an NPH event.

- Target probabilistic performance goals for the performance categories established by the DOE-STD-1021-93 are derived in accordance with DOE-STD-1020-94.

  To ensure that the level of conservatism introduced in the NPH design/evaluation process is appropriate for facility occupancy and other characteristics such as importance, cost, and hazards to people on and offsite and to the environment, the "performance goal" as a target design/evaluation parameter is used. The performance goal for an SSC is defined as its annual frequency of probable failure to perform or annual probability of exceedance of acceptable behavior limits.

- Analysis, design, and evaluation of the facilities are to be performed for NPH in accordance with DOE-STD-1020-94. DOE-STD-1020-94 describes requirements for the design or evaluation of all classes (i.e., safety class, safety significant) of SSCs for earthquake ground shaking for each performance category. It deals with how to evaluate earthquake response of SSCs to the DBE loads (developed based on PSHA) on various classes of SSCs; and how to determine the seismic capacity of the SSCs to determine that the response is acceptable, and covers the importance of design details and quality assurance to earthquake safety.

Figure 1. Design and Evaluation Process for NPH Hazards



3.1 IMPLEMENTATION OF THE NPH HAZARD MITIGATION AT WTP

1. Site-specific studies of site characteristics were performed originally prior to 1996, as documented in Geomatrix (1996). This included the source characterization as well as soil and rock characterization.
2. Geomatrix (1996) documents the performance of a comprehensive PSHA for the Hanford Site incorporating the source characterization noted above. The approach included hazard formulation and development of a hazard model incorporating potential earthquake sources with their estimated frequency of occurrence and size, and incorporation of a probabilistic model to address randomness and uncertainty in the modeling process. Due to the lack of recorded strong motion data for earthquakes in the region, the California strong motion data (which was justified by various studies) was judged to be appropriate for the Hanford Site.

   Geomatrix (1996) was reviewed by multiple organizations and seismic expert panels and was adopted for application to Hanford facilities. In 2004, additional soil and rock characterization was performed based on new data from core drilling reanalysis of existing data to determine the sensitivity of the previously (1996) predicted ground motion to the uncertainty in the soil and rock characterization. Subsequently, the Geomatrix report was updated in 2005 using the new geophysical data, generating RGM, as documented in *Site-Specific Seismic Site Response Model for the Waste Treatment Plant, Hanford Washington* (Rohay and Reidel 2005). This update significantly increased the design ground motion to account for uncertainty in the effects of interbeds located in the soil and rock underneath the site.
3. Performance categories for the WTP facilities have been established and the detailed safety classification of SSCs has been performed and documented in a preliminary safety analysis report. This classification determined the requirements for specified annual probabilities of exceedance for earthquake recurrences to establish loads following the requirements in DOE-STD-1020-94.
4. The detailed facility analysis and designs using the design ground motion have been performed in accordance with DOE-STD-1020-94, as noted below.

3.2 IMPLEMENTATION OF ANALYSIS AND DESIGN FOR FACILITY SSCS AT WTP

Seismic ground motion spectral accelerations developed from the PSHA and performed in accordance with the DOE-STD-1023-95 are enveloped by the DBE ground motion response spectra.

Acceleration time histories that have response spectra that match the DBE are developed in accordance with the requirements of American Society of Civil Engineers (ASCE) 43-05, *Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities,* and NUREG 0800, *Standard Review Plan for the Review of Safety Analysis Reports Nuclear Power Plants*, which define the DBE control motion in the free-field. Site/project-specific design criteria are developed in accordance with DOE-STD-1020-94, which provides the methodology for the dynamic analysis and static analysis of the PC-3 (SC-I and -II) facility SSCs in accordance with the DBE spectra.

3.2.1 Dynamic Analysis

- A dynamic GTStrudl finite element model of the building was generated mainly for the development of the dynamic fixed base modal properties of the building. In this analysis, the modal frequencies, mode shapes, and mass participation factors of all major modes of the structure are identified.

- Seismic soil-structure interaction (SSI) analysis, based on the GTStrudl model, was performed using the computer program SASSI (System for Analysis of Soil-Structure Interaction), as recommended for SSI analysis by DOE-STD-1020-94.
- Three SASSI analyses were performed using lower bound, mean, and upper bound soil profiles with the WTP DBE input acceleration time histories applied at the ground surface.
- The SASSI results were post-processed to generate (1) ZPA at all floor locations to generate static equivalent design forces; (2) in-structure response spectra (ISRS) for design of building floors, piping, and qualification of equipment; and (3) the dynamic soil pressure distribution for design of underground walls.

3.2.2 Static Analysis

Detailed finite element models with finer meshes were developed using SAP2000 software to perform the static analyses for each of the SC-1 buildings to obtain final loadings at design elements (e.g., floors, walls etc.) for various load combinations with seismic and other dead loads, live loads etc., as required by relevant codes.

- Reinforced concrete structures are designed in accordance with the American Concrete Institute (ACI) codes.
- Steel structures are designed in accordance with American National Standards Institute (ANSI)/ American Institute of Steel Construction (AISC) codes.
- Commodities are analyzed using the location specific ISRS generated by the SASSI analysis and designed in accordance with the respective codes and standards.

3.3 SUMMARY

The process for design and evaluation of NPH (seismic) at WTP followed the DOE process detailed in the standards, reports, and guides described above, and had been accepted through significant reviews. The 2005 revision of the ground motion provides a conservative evaluation of the motion despite lack of more recent data on the interbed layers in basalt. The lack of sufficient data for the interbed layers was accounted for by the use of a conservative 84th percentile amplification functions instead of the required mean amplification functions. However, the lack of soil and rock characterization for interbed layers causes uncertainty in the developed ground motion. A drilling program is planned to collect data for the deep interbed layers in 2006, to confirm the RGM spectra. The current seismic ground motion and the design approach of SSCs incorporate design conservatisms to envelope the potential effects of uncertainty in the characterization of soil and rock underneath WTP.

4.0 DNFSB CONCERNS WITH THE ORIGINAL DESIGN BASIS SEISMIC SPECTRA

DNFSB raised their concerns regarding the original ground motion spectra in a letter dated July 30, 2002 (DNFSB 2002), which identified the following key concerns:

1. The probability of tectonic activity of the anticlines and associated faults for the Yakima folds
2. The spectral amplification associated with the attenuation relationship
3. The amplified floor and equipment response of the superstructure.

The letter also recognized that the foundation design for the High-Level Waste (HLW) Facility included sufficient margin to safely accommodate increases in predicted seismic loading that could result from these issues.

ORP responded in September 2002 with a comprehensive review of the probability of earthquakes and the adequacy of the attenuation relationships in a position paper (ORP/OSR-2002-22). DOE ORP's best estimate of the probability of tectonic activity at the WTP site, and of the spectral amplification, remained as developed in Geomatrix 1996. New issues were raised in January 2003 in a DNFSB letter (DNFSB 2003), which noted that all issues were closed except one, namely that "the Hanford ground motion criteria do not appear to be appropriately conservative" because of uncertainty in the extrapolation of soil response data from California to the Hanford Site. However, DNFSB 2003 again noted that the WTP Contractor's conservative compensatory design measures limiting demand-to-capacity (D/C) ratios have been acceptably implemented to account for this uncertainty, and recommended that ORP maintain these measures for all future design work.

DNFSB also noted that DOE O 420.1B recommends that the natural phenomena hazard assessment be reviewed and updated, as necessary, at least every 10 years for existing sites. DNFSB also recommended the following issues that needed to be addressed for a proper assessment of the site-specific hazard assessment:

- New borings shall be done to detail the shear wave velocity profile, modulus degradation and damping characteristics of the upper 1,000 ft of the Columbia River Basalt Group (CBRG)
- Randomized profile should be developed similar to seismic hazard analysis at Savannah River Site
- Earthquake sources and associated source parameters would be defined, with proper consideration of the range of uncertainty
- Source-to-site attenuation relationship to be developed with adequate consideration to Hanford Site response and rock conditions.

Through late 2003 and the first half of 2004, ORP developed a program to undertake the collection of additional subsurface data, specifically shear wave velocity data. The DNFSB subsequently issued an additional letter in July 2004 (DNFSB 2004) regarding design basis earthquake ground motion criteria, and identified seven site response modeling technical issues that needed to be addressed. DNFSB 2004 also requested that a formal program plan be developed; this plan was submitted to DNFSB by letter dated September 3, 2004 (04-WTP-202). The program plan described the acquisition of additional site data and analysis to address the seven DNFSB issues.

5.0 DEVELOPMENT OF THE REVISED GROUND MOTION (RGM)

As noted above, ORP prepared a detailed plan to address these remaining concerns and submitted it to DNFSB in September 2004. The key features of this plan required PNNL to acquire new soil data down to about 500 ft, as well as to re-analyze the effects of deeper layers of sediments interbedded with basalt (down to about 1,500 ft). At the request of DOE, BNI provided expertise support to this effort. Based on this new data, a reassessment of the predicted Hanford seismic ground motion was completed.

As a result of executing this program plan, PNNL, in conjunction with Geomatrix, developed the report, *Site-Specific Seismic Site Response Model for the Waste Treatment Plant, Hanford, Washington* (Rohay and Reidel 2005). This report documents the collection of new and existing site-specific geologic and geophysical characteristics of the WTP site, and the modeling of the WTP site-specific ground motion response. Some limited new geophysical data acquired, analyzed, and interpreted with respect to existing geologic information gathered from other Hanford Site-related projects in the WTP area were included.

The density of the soil and rock layers present beneath the WTP site was obtained from existing borehole gravity data taken in the late 1970s and 1980s at Hanford. Shear wave velocity (Vs) data were obtained directly beneath the planned location of four major WTP facilities (Shannon and Wilson 2000). These data provide a detailed characterization of the upper 270 ft of soils. New data were obtained in 2004 including downhole shear wave logging at five additional locations, suspension logging in one of these boreholes, and the surface geophysical method known as spectral analysis of surface waves (SASW). The new data from four of the boreholes extended to depths of 180 ft to 260 ft, and data from the fifth borehole extended through additional soil layers to 530 ft, the depth of the top surface of the uppermost basalt rock. The SASW data were taken at the surface near the same five boreholes and at four additional locations near the WTP site. A tenth SASW measurement was made at a nearby location where the basalt rock is exposed at the surface. Existing data from previous geological and geophysical borehole characterizations of the basalts and interbedded sedimentary layers were assembled and evaluated. Compression wave (Vp) sonic logs and check shot surveys taken in the late 1970s and 1980s at Hanford were assembled and analyzed to obtain velocity data for the basalts and interbedded sedimentary layers. Limited available shear wave data for interbed from cross-borehole data at Hanford and suspension logging outside Hanford was used to determine the ratio Vp/Vs. This ratio was used to convert the Vp profiles into Vs profiles in the basalts. The new downhole and suspension logs in the 530 ft borehole near the WTP site were used to determine Vp/Vs in the lower part of the borehole as an analogue to estimate Vs in the similar sediments in the interbeds between the top four basalt units.

The earthquake ground motion response was modeled, and sensitivity studies were conducted to address areas in which the geologic and geophysical information have significant remaining uncertainties. Rohay and Reidel (2005) describe the geologic history of the Hanford Site, and assembled new and existing data on physical properties and measured the statistical variability. These data led to construction of a base case model and an extensive series of perturbations using a logic tree with alternate parameter possibilities, which were then used to simulate the seismic ground motion response at the WTP site.

The process used arrived at a frequency-dependent RAF of the WTP site with respect to the empirical California deep soil profile. This RAF was then applied to the existing design response spectrum (DRS) to arrive at a DRS that can be used to continue the WTP design process. This DRS is an approximation expected to be conservative for application to the facility design.

Based on discussions and workshops with DNFSB staff between late 2004 and early 2005, the 84th percentile results for the RAF from the full logic tree were used to:

1. Guide the final selection of the RAF
2. Envelop the mean responses from individual subsets of the logic tree that were found to lead to higher estimates of the RAF
3. Provide conservatism in the final design recommendation by enveloping the uncertainties in the response spectra.

DOE chose the 84th percentile RAF from the logic tree to develop a conservative estimate of the mean surface response spectrum to be used for design.

The original 1996, 5% damped horizontal DRS was scaled by the 84th percentile frequency-dependent RAF from the full logic tree result to obtain a conservative estimate of the horizontal response spectrum appropriate for the WTP site. This spectrum was then broadened at the peak to arrive at the recommended horizontal DRS for the WTP site; that conservatively accounts for the differences between the WTP site and the California deep soil profile associated with the attenuation models used in the original unbroadened horizontal spectra (UHS) development. Subsequent activities discussed in this report continue to support the conservatism assumed by using 84th percentile RAF used for design.

The peak of the recommended spectrum is at 5 Hz. The spectral broadening process was applied to the spectrum by extending the peak on the low-frequency side about 30% to about 3.85 Hz and about 15% on the high-frequency side to about 5.75 Hz. For higher frequencies, the spectrum was then extended linearly (in log-log space) to 12 Hz. The conservatism in the higher frequencies above 12 Hz was found to be significant because the logic tree results indicated that the higher-mode responses of the subsets of the logic tree yielded a dip in the spectra at these frequencies.

Using this information in February 2005, the seismic ground motion response spectra were updated from the 1996 baseline spectra, and BNI was directed to implement this as the new design basis. The new horizontal ground response spectrum increased the ZPA from 0.26 g to 0.29 g (Figure 2) and the vertical ZPA increased from 0.18 g to 0.21 g (Figure 3). The peak of the horizontal spectrum increased from 0.57g to 0.79 g (~ 40%) and the peak of the vertical spectrum increased from 0.33 g to 0.47 g (~ 42%). The increased ground motion is attributed to:

1. Thickness of soil and gravel directly below the WTP is less than previously assumed (365 ft rather than 500 ft)
2. Lower effective damping from the four deeper soil interbed/basalt layers.

Figure 2. WTP Design Basis Event Ground Motion Spectra (Original and Revised) <u>Horizontal</u>

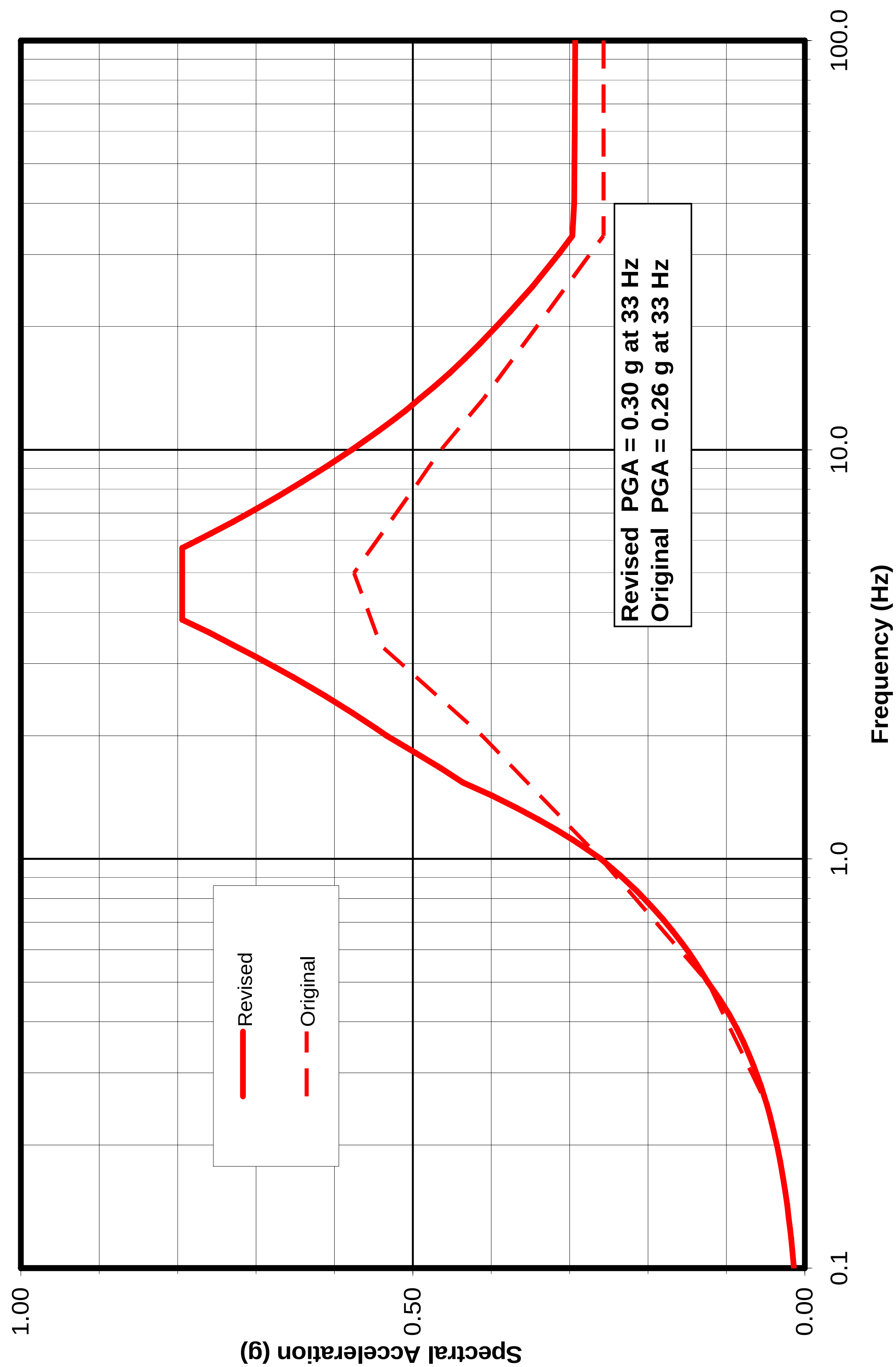
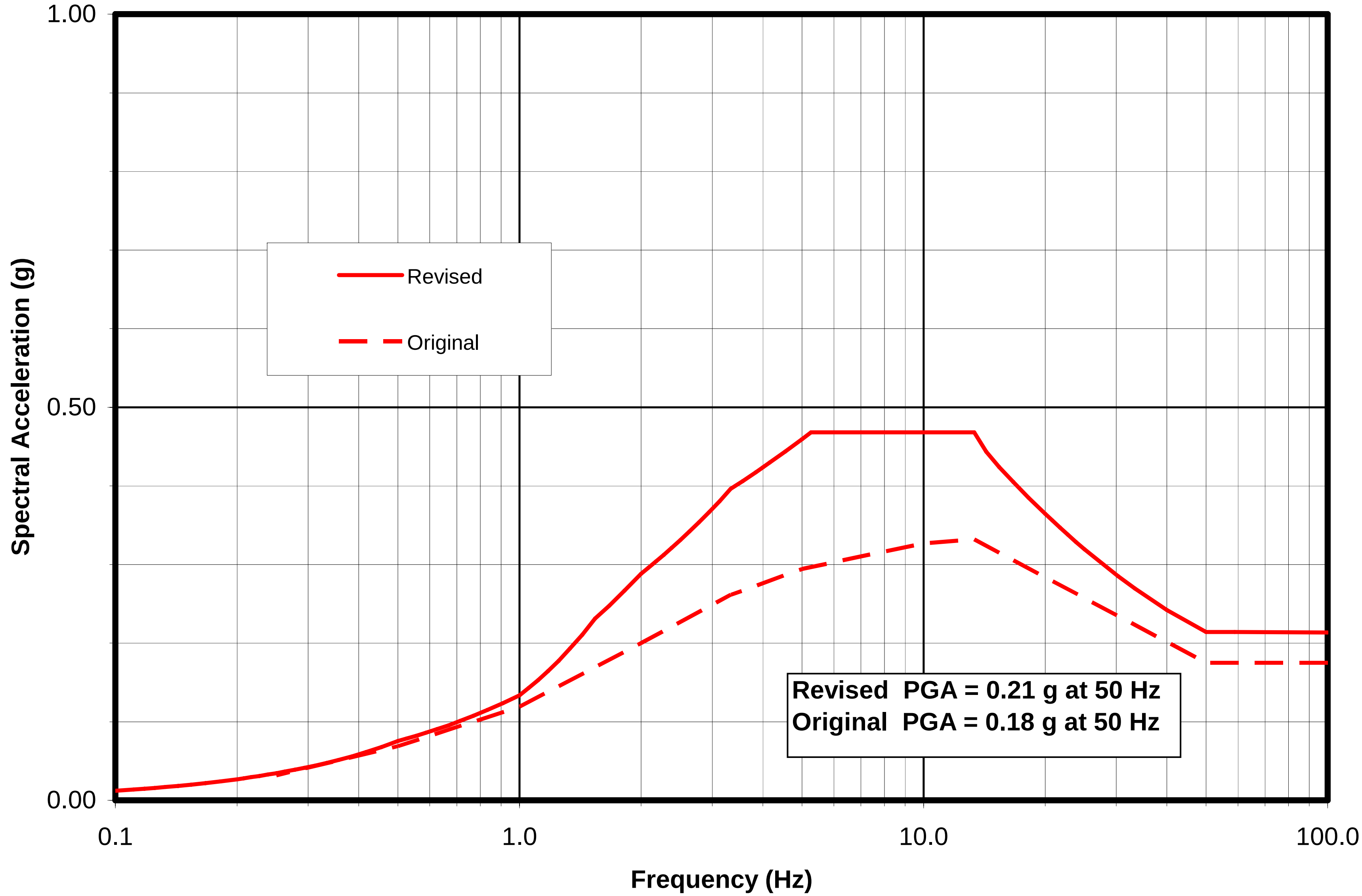


Figure 3. WTP Design Basis Event Ground Motion Spectra (Original and Revised) <u>Vertical</u>



6.0 IMMEDIATE ACTIONS TO MINIMIZE THE RISK

The development of RGM required DOE to direct BNI to use the RGM response spectra as the design basis for the WTP project SC-I and SC-II facilities. The RGM free field response spectrum had higher seismic accelerations than those in the current safety requirements document (SRD), and impacted the Pretreatment (PT) Facility, the HLW Facility, and a few important-to-safety balance of facilities (BOF) structures.

ORP took the lead in developing options for addressing the expected increase in seismic ground motion as early as December 2004. ORP and BNI presented these options to DNFSB staff in January 2005, a month before the RGM was finalized and formally transmitted to BNI. BNI also presented to DNFSB staff a comprehensive evaluation of the effects of the pending increase on the constructed and yet to be constructed SSCs in the PC3 facilities. The DOE and DOE Peer Review Team (PRT) also held several meetings with BNI to plan a course of action for the anticipated increase in the seismic ground motion, and to brainstorm options to minimize the impact to the WTP project. The reviewers considered the evaluation of the risk to rework the already constructed and fabricated SSCs; the development of processes to maintain configuration control for existing and ongoing design; the evaluation of existing design and design criteria to understand the extent of conservatisms and identify which of those could be reduced or removed without affecting the safety of the WTP SSCs; and the development of revised design criteria for ongoing and future designs to minimize the risk of rework and modifications. On February 11, 2005, DOE formally issued the RGM spectra to BNI for incorporation in the design basis and implementation into the design. The following provides summary of the actions taken:

<u>December 2004</u>

- DOE notified BNI of the forthcoming increases in seismic ground motion, and asked BNI to evaluate the pace of design, construction, and procurement to minimize the risk of significant rework or plant modification in consideration of all options including acceleration of activities in the low risk areas and retarding or ceasing activities in the areas of high risk.
- BNI (CCN 10132) responded to DOE with a preliminary evaluation of the PT and HLW Facilities using a 30% increase in seismic design basis across all frequencies.
- DOE and BNI began to develop options for addressing the modifications to the design basis; evaluation of embedded conservatisms; and mitigating the impact on equipment. Using an expected increase of at least 30%, DOE, PRT, and BNI developed a list of conservatisms to be evaluated by BNI for reduction or elimination (Attachment A).
- To ensure the risk of rework was minimized on facilities affected by the RGM, DOE directed BNI to stop placement of concrete walls and slabs for the PT and HLW facilities, except on a case-by-case approval by DOE. The approval would be based on re-verification of each element against the potential RGM increase.

<u>January 2005</u>

- ORP and BNI's identification of design conservatisms was presented to DNFSB staff, and categorized as "A," "B," or "C," based on the benefit and the difficulty of providing (meeting with DNFSB staff at Hanford [D-05-STRUCTURAL DESIGN-25]).
- ORP reiterated to BNI the importance of the resolution of the finite element model mesh refinement issue for the design of the facility concurrent with the re-evaluation of the facilities for the RGM, while noting that BNI was able to demonstrate that on a global basis, FEM models using GTStrudl software provided sufficiently conservative design loads (05-WED-002).

- BNI completed their evaluation of the SSCs to provide a qualitative understanding of the risk of modifications associated with various SSCs and presented it to DNFSB staff.
- Discussions with BNI identified that it would take 4 to 6 months to complete the detailed SASSI before the WTP could obtain in-structure response spectra for the redesign of SSCs. Therefore, ORP and BNI started to develop the interim design criteria and implementation plan for its use for the continuation of design.
- DOE initiated review and approval of the concrete wall and slab placements on a case-by-case basis.
- Even before issuing the RGM spectra, ORP formally directed BNI to
  - Start updating the SASSI models
  - Develop the rationale for eliminating or reducing previously identified "conservatisms"
  - Develop the interim seismic criteria (ISC) to continue design (05-WTP-016).

<u>February 2005</u>

- DOE issued the RGM spectra to BNI showing a 40% increase at the peak of the response, directing BNI to incorporate the RGM as the design basis. DOE further directs BNI to develop bounding interim design criteria to minimize the impact to the WTP project. (05-WTP-036).

7.0 STRATEGY FOR IMPLEMENTING RGM

Implementation of the RGM into the WTP project is a major effort, and was anticipated to require significant resources for a number of years and to affect many aspects of the project. Because of the magnitude and duration of the efforts, it required that multiple activities be performed simultaneously to ensure proper and effective implementation. The following four key activities identified by ORP and BNI were initiated simultaneously as the first step towards the full implementation of the RGM:

1. **Development of Bounding Interim Seismic Criteria (ISC)**. This would allow continuation of design and limited procurement and construction instead of waiting for the actual revised seismic loads (which would require 4 – 6 months to develop).
2. **Development of a Detailed RGM Implementation Plan**. This plan would provide the detailed road map for the implementation of the RGM, including maintaining configuration control of all the constructed and issued for construction (IFC) structures and components.
3. **Performance of the Dynamic Analysis for the SSI**. Initiate the SSI analysis for each affected facility to provide the ISRS and structural loads that are needed in the more detailed design of equipment and structures. Once the results from the SSI dynamic analyses are available, follow-on facility-specific analyses would be required to obtain the member forces for the design of walls and slabs.
4. **Evaluation of the Design and Analysis Conservatisms**. To minimize the impact of the large increase in the RGM on the already constructed, fabricated, and future designs, the ORP Structural PRT together with BNI developed a list of areas of design and analysis where conservatism exists, and which could be reduced or eliminated without affecting the safety of the SSCs due to the current maturity in design. DOE directed BNI to evaluate those areas and develop rationale in white papers justifying their elimination in the final design criteria. These white papers would be reviewed by PRT and DNFSB staff and approved by DOE before implementing in the design criteria.

Discussion of the each of these key implementation activities follows.

7.1 DEVELOPMENT OF THE BOUNDING INTERIM SEISMIC CRITERIA (ISC)

ORP directed BNI to develop the ISC to facilitate ongoing design activities while waiting for the revised SSI analysis. The ISC applied to all SC-I and SC-II SSCs being evaluated or designed. The ISC was planned to be used until such time that new seismic loads and ISRS, based on the new SSI analysis, would be available. It was decided that all evaluations performed under the ISC would be re-evaluated when the new seismic loads (based on the new SSI) become available. The ISC was structured such that evaluations performed under them are conservative and SSCs installed under them would most likely support the final revised loads. DOE recognized that there was a possibility that in some isolated cases, the evaluations performed to the ISC may require rework when evaluated using the final RGM loads. However, the cost of delaying design far outweighed this potential risk. In addition, the ISC needed to provide measures to account for the ongoing DNFSB staff and PRT concern that the static finite element design models (GTStrudl) for the PT and HLW facilities had inadequate mesh density to provide the needed precision in results to assure adequacy.

The BNI-prepared ISC was approved by DOE on April 1, 2005 (05-WTP-054), and would be effective until a final design criteria was developed that implemented the new SSI results. The ISC devised a methodology using various multiplication factors to conservatively increase the existing load results to address the mesh density concerns in the interim. The key elements of the ISC were:

- Seismic loads were increased by 40% across the board (independent of the actual frequency) for building and equipment designs to account for the RGM
- Bounding load amplification factors were developed to account for the mesh refinement concerns in local areas.
- Target D/Cs were developed based on engineering judgment considering the maturity and the remaining uncertainty in the design of the specific commodity.

DOE approval of the ISC was discussed with the DNFSB staff at great length, explaining the intended conservatism of the approach. DNFSB accepted the validity of the approach based on the "interim" and "short-term" use of the ISC. DOE approval of the ISC was based on the following rationale, as documented in Safety Evaluation report (05-WTP-054):

1. **Rationale for the 40 Percent Increase in the Existing Seismic Loads:** The RGM acceleration was increased by 40% in the 4-7 Hz range, and to a smaller magnitude outside this range. Assessment of the seismic analysts was that the magnitude of increase in loading for the SSCs for most of the cases would be at least 5% -10% less than the assumed 40% increase in the peak seismic accelerations. This is due to the fact that: 1) the 40% increase in seismic acceleration was limited to a narrow frequency band, and the increase was much less outside of the limited frequency band; and 2) the current loading associated with accidental torsion was considered by increasing the net story shear for every floor. A more refined application of accidental torsion would maintain the load at external walls, with linearly decreasing loads as the distance to the shear center is reduced, instead of the current analysis using the same maximum load of the external wall at all internal walls. Thus, the approach used was conservative, particularly with respect to the internal walls and walls close to the shear center. These considerations were anticipated to result in lesser loading in the final analysis than considered in the Interim Design Criteria.
2. **Rationale for Acceptance of the Amplification Factors to be used in the Interim Design Criteria to Account for Mesh Density Concerns**: Concerns regarding the coarseness of the mesh densities were recognized, and to remedy that, a number of sample sections representing bounding conditions were studied. The sample sections were re-analyzed using a SAP software model with finer mesh densities (3X3) compared to the coarse mesh densities (1X1, or 2X2) used in the calculation of record, GTStrudl model. The resulting stresses from these analyses were compared to develop the bounding multiplication factors (>1.0) planned to be applied to the results of the GTStrudl model for the interim design. These bounding factors were provided by BNI. The ORP PRT reviewed the basis of this approach and the resulting bounding amplification factors, and found them acceptable and conservative for use during interim design. The ISC required the results from the GTStrudl model analysis to be amplified by the bounding amplification factors, which is assumed to remove the uncertainty of the mesh density considerations.
3. **Rationale for Changing the D/C limits for use in the Interim Design Criteria**: BNI design criteria, SDC, revision 1 (April 2003), required the D/C for the design of concrete walls and slabs at grade or below to be maintained at 0.85, and above grade to 0.9. This 10 to 15% design margin was maintained due to the early stage of design, and to account for uncertainties in the equipment loadings, change of configurations, etc. Later, in September 2004, DOE directed BNI to increase and maintain the design margin for all concrete design to 0.85, due to concerns over the coarse mesh densities in the facility GTStrudl model and the DNFSB seismic concerns. The refined mesh density had the potential to generate higher stresses in various elements, which may need further validation. In addition, concerns regarding existing seismic ground motion were raised at the same time. In light of these concerns, DOE requested BNI to maintain an additional margin

of 5% above the BNI design criteria to account for the uncertainty of the analysis and to provide a reasonable assurance that the designed and constructed walls and slabs will be adequate when the mesh density issues and seismic issues were resolved. The ISC proposed limiting the allowable D/C ratio to 0.95. The revised margin had been accepted based on engineering judgment, considering the conservative nature of developed RGM, the conservative bounding load amplification factors, and the stage of design. The proposed Interim D/C ratio was limited to 95%; i.e., 5% below the code allowable in HLW and PT Facility to retain some margin for unknown uncertainties, including potential future modifications.

7.2 DEVELOPMENT OF THE RGM IMPLEMENTATION PLAN

The BNI-prepared implementation plan describes the phased approach to incorporate revised seismic criteria into the design of SSCs designated as SC-I and SC-II) for WTP. The seismic analyses and tests that were completed for various SSCs using spectra based on the original ground motion were required to be re-analyzed or re-performed using the RGM. Implementation included documentation of analyses (by BNI Engineering and supplier calculations) or supplier test reports confirming the as-built SSCs would perform their safety functions when subjected to RGM. The revised criteria was required to be incorporated into the top-level requirements documents and would systematically flow down into the affected calculations, drawings, and other design deliverables in accordance with existing project procedures. Any necessary design changes needed would be incorporated in the fabricated or as-built SSCs. The revised seismic criteria would then be implemented in a manner that reduced the impact to ongoing project work and minimized risk of rework, and at the same time provided a configuration control over the already designed and/or procured or installed SSCs. The RGM implementation plan was developed by BNI in discussion with DOE on May 3, 2005 (CCN: 116994).

The following activities were required to be performed to incorporate the RGM into the design and drawings:

- Update top-level project requirements documents to incorporate the RGM. This included revising the Authorization Basis documents and seismic analysis and design criteria (SADC), and incorporating the RGM and other changes made to facilitate incorporation of the RGM. This allowed the performance of the SSI analyses and other front-end calculations using the revised RGM spectra to determine facility-specific seismic loads and ISRS.
- Update the next level of requirement documents, e.g., SDC, various commodity design criteria, and specifications for the equipment designed by the supplier vendor. This included incorporation of ISRS from the SSI analysis, and other relevant changes.
- Revise the facility-specific static analyses and other structural calculations using the results of the revised SSI calculations.
- Revise structural drawings and requisitions for structural steel fabrication as necessary to conform to the results of updated structural analyses.
- Obtain new or revised seismic qualification data from supplier vendors, and review and disposition the proposed design changes resulting from the revised seismic criteria.
- Perform new or revised pipe stress and support calculations using the revised seismic loads.
- Complete new or revised pipe isometrics, pipe support, and other layout drawings as necessary.
- Revise calculations for heating, ventilation, and air conditioning (HVAC) ducting, electrical raceways, and associated supports using new seismic loads. Complete new or revised HVAC ducting and electrical raceway layout and support drawings as necessary.

- Tracking of SSCs During the Implementation of RGM

The following table illustrates the hierarchical flow down of seismic criteria and identifies the methods to be used to identify and track conditions where compliance with the revised criteria needed to be confirmed.

Hierarchical Flow down of Revised Seismic Criteria	Method of Identifying and Tracking Indeterminate or Nonconforming Conditions
Top-level requirements	N/A (top-level documents will be updated)
Engineering calculations	Committed (or interim) vs. confirmed status
Design deliverables	Placement of holds (e.g., indicated by “clouds” on drawings)
Fabricated or constructed SSC	Nonconformance report (NCR)

Affected engineering calculations were to be designated as “committed” (or “interim” for piping calculations governed by BNI procedure) until they incorporated the top-level revised seismic requirements. The designation of “confirmed” status indicated that the design input complies with the top-level requirements (including the revised seismic criteria) and the assumptions had been verified as adequate. Drawings and other design deliverables needed to be revised, if necessary, to be consistent with the updated calculations. If the SSCs affected by these design changes were not constructed, then “holds” were to be placed on the associated portions of drawings until the design was revised accordingly.

If changes were identified to the design of SC-I/II SSCs that had been delivered or constructed, the affected SSCs needed to be identified on nonconformance reports (NCR) and tracked for resolution. This included procured SC-I/II equipment that had been delivered before the revised seismic criteria were incorporated in the requisition. NCRs were to be issued if the equipment did not conform to the revised seismic criteria. This approach used standard work processes in existing project procedures for implementing design requirements (and associated changes).

A log was prepared of the BNI Engineering deliverables: 1) Engineering calculations addressing seismic loads and 2) structural drawings (concrete, structural steel, pipe isometrics, and raceway and ducting support drawings) that had been IFC; requisitions of equipment that had been issued to procurement. This list would identify the IFC design deliverables that had not been verified as compliant with the RGM. This list formed the basis for general NCRs (called “Seismic NCRs”) that identified facilities with indeterminate and/or potentially changing design due to the revised seismic criteria. Four “Seismic NCRs” (one each for PT Facility, HLW Facility, and BOF, and one for vendor procurements) were generated; these are subdivided into three subcategories (concrete, steel, and piping) to track the implementation of RGM in BNI’s design deliverables, and to provide sound monitoring and control of the progress of the re-evaluations. These NCRs identified the affected engineering deliverables and tracked the status of incorporating the revised seismic criteria. The interim disposition of these NCRs allowed work to proceed on the affected SSCs in accordance with this plan. This log is updated as the revised seismic criteria are incorporated into the associated design deliverables.

7.2.1 Release for Fabrication/Construction during the RGM Implementation

The following actions were planned to minimize the risk of rework while reducing the impact of the RGM change on the progress of fabrication/construction work:

- Pending completion of SSI analyses and the development of revised seismic loads (e.g., accelerations and ISRS), the design of SC-I/II SSCs would be based on ISC to support the near-term fabrication and installation work.
- With the exception of concrete placement, current and near-term fabrication and installation work for SC-I/II structures (steel and other structural features such as rebar and formwork) would continue based on the existing design unless ongoing re-analyses indicated that the design did not comply with the interim criteria.
- The consequences of rework for concrete were deemed much greater than for steel and other SSCs. Therefore, holds had been placed on future concrete placements. Before these holds were released and concrete placed, the seismic analyses for concrete structures were required to be completed and documented in accordance with established project procedures to confirm the associated design complied with the revised seismic criteria (either interim or final seismic loads, depending on whether the final loads were available).

Until the interim seismic criteria were incorporated into engineering calculations and the design revised accordingly, some of the physical work was allowed to proceed (at risk) to avoid substantial impacts of stopping work. The following table identifies the prerequisite conditions for proceeding with physical work using the existing design.

Location of Work	Type of Work	Prerequisite for Proceeding with Fabrication or Construction Work Using Current Design
Construction site (field)	Concrete placement	Re-analyses documented in engineering calculation change notice (ECCN) or calculation revision showing the current design meets the interim seismic criteria or the final seismic criteria
	Other construction or installation work (e.g., installation of structural steel, equipment, piping, or other commodities)	Release form signed by project management that includes qualitative assessment of risk/benefit
Supplier shops	All fabrication and assembly work (including structural steel, equipment, piping, and other commodities)	None (proceed while re-analyses are being conducted)

Judgment was used in assessing the costs and benefits of continuing versus stopping construction. A construction release form was developed to allow construction work to proceed on non-concrete SSCs, and used to evaluate the impacts and risks and to determine whether to proceed with physical work using the existing design. Management approval of the release of construction activities was required prior to the completion of seismic redesign. The approval needed to weigh the cost and schedule impact of delaying work pending completion of analyses, against the cost and schedule impact of rework that could result from continuing the construction or fabrication.

7.3 DYNAMIC RE-ANALYSIS FOR THE SOIL-STRUCTURE INTERACTION (SSI)

Since the WTP Facility structures are located on a deep soil site, seismic SSI analysis was required to be performed to obtain seismic responses for design of SSCs in accordance with DOE-STD-1020-94. This analysis was implemented for the original design basis ground motion and would have to be repeated for the RGM. Re-run of the SASSI included the following:

- Update of the dynamic three-dimensional (3D) GTStrudl finite element model of the affected facility structures with current facility configuration
- Use of actual building loads to replace conservative assumptions used in previous analyses
- Conversion the GTStrudl structural model for use as input in the SASSI model
- Update of the SHAKE model with revised soil-column properties to obtain strain-compatible soil stiffness and damping values
- Development of statistically independent time histories for input in the SASSI model based on the revised spectra
- Running the SASSI model
- SASSI results provided:
- ZPA at floor locations to generate seismic design loads
  - ISRS at various facility locations for design of floors, piping, and equipment
  - Dynamic soil pressure distribution.

SSI analyses of the PT and HLW buildings based on the RGM were completed in July 2005. ISRS based on the revised SSI analyses for the design of equipment, piping, and other commodities were issued in September 2005.

7.3.1 Results of the SSI Analyses and Assessment of the Impact on the Facility Design

New seismic design loads for both the PT Facility and the HLW structures from the revised SSI analyses generally indicated:

- The increase in lateral seismic loads compared to the pre-ISC loads in PT Facility is less than 35%.
- The increase in lateral seismic loads compared to the pre-ISC loads in HLW is less than 10%.
- The increase in vertical seismic loads for PT Facility and HLW concrete slabs are in general less than 40%, but exceed 40% in a few locations.
- The increase in seismic soil pressure on the below ground walls for both PT and HLW is less than 20%.
- The increase in horizontal ISRS, required for design and qualification of equipment and distribution systems, is less than 40% except, for a limited frequency band width in the HLW and for limited areas in PT Facility.
- The increase in vertical ISRS is more than 40% over a limited frequency band in HLW.
- The increase in vertical in-structure response spectra is generally more than 40% in PT Facility.

Since the increase in seismic horizontal building load is generally less than 40%, the adequacy of assumptions in ISC for building design has been confirmed and building structural design released in the interim period would be acceptable for the RGM. Equipment designed for in-structure spectra that exceeded the interim criteria will be handled on a case-by-case basis. Often the use of location specific spectra as an alternative to using the more conservative enveloping spectra will provide an adequate load reduction to qualify equipment. It is also possible to consider actual nozzle loads rather than envelope nozzle loads when evaluating tanks, valves, and other equipment that attaches to piping.

7.4 EVALUATION OF DESIGN AND ANALYSIS CONSERVATISMS

In January 2005, the ORP Structural PRT together with BNI evaluated areas of conservatisms embedded in the design and analysis that, if removed, would minimize the impact of the increase in the RGM on the already fabricated/constructed and future designs. Attachment A provides tables that list the embedded conservatism; each conservatism is given a ranking of A, B or C. Items ranked A were well understood and were implemented without additional justifications. Items ranked B required further evaluation to determine if implementation was warranted. Items ranked C were deemed very unlikely for implementation. An assessment of the difficulty associated with development of the rationale for elimination of those conservatisms was performed jointly with the PRT and DNFSB staff and noted on the table.

The PRT team performed an assessment of the additional design margins available due to these existing conservatisms. They concluded that as much as 50% of additional design margin could be obtained, depending on the location and type of structures, by reducing the listed conservatisms. These reductions reflect the conservatism in the seismic design loads used for designs under the original design and ISC. *This assessment provided assurance that significant majority of the existing designs when evaluated to the RGM would not likely to require modification.*

In addition, the ISC applied very conservative factors to account for mesh refinement effects so it is not expected that the final design forces will exceed any designs conforming to the ISC. Designs originally performed to the criteria and tools, which were evaluated during the interim period using the conservative mesh applications factors, were found to be adequate without modification.

The major areas of conservatisms that were evaluated for reduction or elimination were: allowable design margins; use of ductility considerations; thermal loading in the design; coherency effect; calculation of accidental torsion; assumptions in SSI; and assumptions in the structural design. Each of these is discussed below.

7.4.1 Existing Design Margin

One essential measure for the acceptability of a building structure with respect to building code criteria is the structural design margin, called the “demand to capacity” ratio (D/C). The “demand” is the force a structural element is required to carry and the “capacity” is the code allowable force for the structural element. This ratio is calculated for all major load-carrying elements in the structure; i.e., every wall, floor slab, column and beam will have a unique demand to capacity ratio. If the D/C is equal to or less than 1.0, then that structural element is acceptable with respect to the building code. It is important to recognize that there is a significant margin between meeting code criteria and structural failure. The code imposes load factors on the demand; strength reduction factors on the capacity and minimum material strengths. In general, actual structural failure will be at a load much higher than the calculated capacity.

With respect to the HLW and PT buildings, a margin management program that imposed additional conservatism for the D/C ratios was implemented for the project. For the lower levels of the structure, the D/C was limited to 0.85 and increased to 1.0 for the upper elevations of the buildings. In general, the HLW and PT concrete designed to the existing design criteria have been limited to a D/C of 0.85. This conservatism was imposed since the building was designed and constructed to a closed-couple design approach. This means that the designs of the lower portions of the structure were released for construction before the designs of the upper portions were completed. Other conservative judgments were invoked during the early design phase; i.e., 1.5 times the peak of the ground spectra and heavy load estimates for equipment. Designs were released and constructed based on these conservative judgments.

The status of the project is now such that the building design is nearing completion, the general arrangement is finalized and all major equipment weights and locations are defined. These changes are being incorporated into the design models so that the structural design loads reflect the final design configuration. The seismic ground motion has been conservatively modified (increased) to account for in-situ site conditions. There have been improvements, because of mesh refinement, in the finite element models. All of these factors result in the calculation of more realistic demands in the structural elements than for the earlier design. This higher level of accuracy reduces the risk that was accounted for in the earlier design phases by implementing conservative assumptions. *In particular, it is now reasonable and prudent to allow demand to capacity ratios in structural elements to be up to 1.0, which is in conformance with building code criteria.*

By allowing the D/C ratio to reach 1.0 does not imply that all structural elements that make up the building structure will reach 1.0 simultaneously. For example, the HLW and PT reinforced concrete structural elements are often sized for radiation protection, providing in many cases designs that are more robust than if designed for structural loads alone. Such elements typically have low D/C ratios. Additionally, the reinforcing steel that is embedded in the concrete has uniform spacing patterns and sized for ease of construction and generally envelopes the calculated amount of required steel. In cases where the thickness is determined by radiation protection requirements, the minimum required steel by code may be greater than needed to carry the building loads. These factors lead to D/C ratios that are less than 1.0 in the majority of the structural elements.

Based on the above rationale, and the review of the results of the revalidation of facility structures based on the RGM completed so far, it is understood that the majority of the concrete already placed will have design margins compared to the code allowables.

7.4.2 Ductility

For SC-I SSCs, the use of ductile behavior, by including inelastic energy absorption factors (Fμ), in the applicable load cases, is in accordance with the requirements of DOE-STD-1020-94 and ASCE 43-05. The WTP seismic design criteria limited Fμ to unity, which equates to elastic behavior. This was initially imposed to:

- Be consistent with the U.S. Nuclear Regulatory Commission (NRC) requirements for nuclear power plant (NPP) SC-I SSCs, in the event the WTP was required to be licensed by the NRC
- Compensate for the uncertainties associated with the 1996 design basis seismic ground motion.

For the following reasons, higher F_{μ} factors in accordance with DOE and current ASCE criteria were deemed acceptable:

- At the time of the development of the design criteria for WTP, the more restrictive NRC criteria were adopted as it was not yet determined if the project was to be under NRC or DOE cognizance. The NRC requires SC-I SSCs in nuclear power plants to remain elastic due to the severity of failure consequences at the Nuclear power plant. The same criterion was adopted for WTP SSC, even though the severity of WTP SSC failure consequences is significantly less than for nuclear power plants. In 2001, the contracting method for the WTP was changed from “privatized” to a “DOE managed” facility, which required the design to meet DOE requirements. DOE criteria do not impose the requirement to remain elastic for PC-3 facilities such as HLW and PT.
- The recent increase in the design basis seismic ground motion accounted for the principal uncertainties identified by way of using additional site soil data and the latest site response determination methodology. Furthermore, since the revised design basis ground motion spectra were based on 84th percentile estimation of the mean ground response, the estimate of the RGM is considered adequately conservative.
- The uses of Fμ factors greater than unity are <u>limited to only those portions of the building structure that are already constructed that may have D/C ratios greater than 1.0 under the RGM</u>.

The use of these factors is to be tracked, compared with the code allowed values to indicate the margin to the allowable value, and documented. Even within the limited set of design, where the use of the Fμ factors greater than unity is allowed, only a small number of sections are anticipated to actually use this factor.

By allowing a limited applicability of inelastic energy absorption on a case by case basis for concrete already placed, the facility design maintains additional margins compared to code allowables.

7.4.3 Thermal

During the design process, the effects of thermal loading have been incorporated into load combinations in accordance with ACI 349-01. The ACI 349-01 subcommittee on the effects of thermal loads provided a draft change to the code that eliminated, for certain modes of deformation, the thermal loads from the seismic load cases, but still required the consideration of the effects of cracking on the remaining mechanical loads. An ad hoc committee of ORP PRT consultants and BNI had reviewed the ACI subcommittee recommendations for application to HTP and PT wall and floor slab designs. The committee issued a white paper, *Combination of Thermal and Seismic Loads for the Hanford WTP* (Mertz 2005), which recommended changes to the WTP design criteria to reduce excessive conservatism in the treatment of thermal loads in HLW and PT. These recommendations, while more stringent than the ACI draft change, were to provide relief for cases of combined thermal and seismic loads. This provision is to be used only in the limited cases where the stresses based on the existing ACI method exceeded the ACI allowable limits. This change was briefed to DNFSB staff in November 2005. DNFSB staff accepted the general approach, however, raised some concerns regarding the fatigue failure of the reinforcing steel for potential large thermal cycles. Based on the path forward agreed upon, the ad hoc committee has evaluated the thermal stress and cycles in the critical areas to determine the severity of the thermal cycles. The draft report, *Effects of Thermal Loading in the WTP HLW and PT Buildings* (Mertz 2006) was completed in February 2006, which validated the previous assumption that the thermal cycle is not significant, and hence, will not cause fatigue due to cyclic thermal loads for these facilities. The DNFSB staff has been briefed on the result.

This criteria change will result in reduced design forces in structural elements that are currently controlled by design load cases that include both seismic and thermal components, further ensuring the acceptability of portions of the structure that were previously controlled by the combined thermal and seismic load combination.

7.4.4 Coherency

Coherency is a phenomena associated with seismic events that occurs at the location of the facility. It is a measure of similarity of the ground motion at different locations within the footprint of the building foundation caused by a seismic event. For a facility with a large footprint, the seismic motion as measured at one point at the ground elevation can be different from another point that is some distance away form the first point. Recorded data from dense arrays confirm differences in the ground motion.

Traditionally, in building seismic analysis, an assumption is made that the seismic motion is the same at all points over the footprint of the building. This traditional approach was used for the seismic analysis of the HLW and PT buildings for both the initial and final soil-structure interactions analyses. This assumption leads to an over prediction of the seismic design loads for the building and equipment, particularly in frequency ranges greater than 8 Hz.

A preliminary SSI model that included the coherency effects with a simplified HLW building model was analyzed. Results showed that in the frequency range of 8 to 15 Hz, a reduction of 10% to 20% in horizontal responses is expected. In the vertical direction, the reduction was larger, and at higher building elevations reached 40%. Thus, consideration of coherency is capable of alleviating the impact of the increased design motion on equipment, distributed commodities, and floor slabs.

BNI presented the model and its basis to DNFSB staff and its consultants in a July 2005 WTP Seismic Review Meeting in San Francisco. Upon review of the supporting documents, DNFSB staff and its consultants agreed that the phenomenon is real. However, the DNFSB staff was concerned that its application on a DOE nuclear facility would be precedence-setting. The DNFSB staff and PRT asked that a detailed implementation plan be developed and presented for further understanding of how the coherency model would be applied to any WTP structures. The plan was issued on September 30, 2005. If it is deemed necessary to reduce seismic loads for systems and components, then the SSI would be run in accordance with the detailed implementation plan subject to a full technical justification to the DOE and DNFSB staff. *However, due to the schedule conflict of the implementation, and perceived difficulty of convincing DNFSB for the precedent-setting adoption of this approach, this concept has been put on hold. Only if the facility design faces significant rework from the RGM (which is very unlikely), the coherency approach would be reconsidered.*

7.4.5 Accidental Torsion

In order to simplify the design process, a conservative treatment of code required “accidental” torsional load was adopted into the original structural criteria. This conservatism was removed and replaced by standard accepted methodology that resulted in lowering loads in most of the interior walls. Under the new criteria, this load was applied as a function of the distance from the shear center of the wall system, which reduced the total seismic shear loading compared to the original criteria.

7.4.6 Equipment Structure Interaction

It was anticipated that the seismic load for equipment could be reduced by performing dynamic analysis considering the building and the equipment structure interaction. It was also noted that due to the complexity of the dynamic analysis for the complex interaction, it would be utilized only for extremely large equipment that may not meet acceptance criteria using the revised ISRS.

7.4.7 Conservative Assumptions in Soil-Structure Interaction

Two assumptions made in the development of the original seismic loads that were not included in the revised SSI analyses of the HLW and PT because of the design maturity of these buildings. These changes from the earlier design practice were considered well understood by BNI, ORP, PRT, and the DNFSB staff, and did not require the development of a white paper.

- Reduction of the “bump” factors used, which made the equivalent static seismic accelerations used in the structural design higher (and conservative), and
- Applied code acceptable reductions to peaks in the ISRS.

7.4.8 Conservative Assumptions in the Structural Design

The following improvements to the design process were implemented for the redesign effort. These improvements removed conservatism used in the initial design, as well as provided a better measure of the structural margin. These changes from the earlier design practice were considered well understood by BNI, ORP, PRT and the DNFSB staff, and did not require the development of a white paper.

Structural members were designed to control load cases rather than the practice of envelope load cases as used in the initial design.

- Use the moments and shears forces at the face of walls, as allowed by code rather than the centerline values used in the initial design.

- Use the soil-structure interaction seismic soil wall pressures for below grade wall design.

8.0 STRUCTURAL MODELING ISSUES AND RESOLUTION

The original analysis of the building structures used the GTStrudl finite element analysis software. This approach required the building structure be subdivided into a number of elements that form a mathematical basis for determining the force distribution throughout the structures. In the HLW and PT buildings, the concrete walls and slabs were modeled with finite element shell elements to form a continuous structure. The number of elements that are used in a wall or slab is referred to as mesh density; the more elements, the finer the density and the more precise the resulting forces and moments will be that are used to design the reinforced concrete.

In 2003, the question of adequate mesh refinement was raised by DNFSB staff and the ORP PRT, and it was limited to local areas of the building model of HLW where it did not appear that sufficient element refinement practices were used. Based on this perspective, it was anticipated that upper bound generic "force amplification factors" would be developed once trending due to primary controlling loads or the existing mesh density was determined to be reasonable. After these factors were developed, interaction effects due to close proximity of some of these localized areas would similarly be addressed by developing additional correction factors.

BNI performed a number of studies for shear wall openings, and "typical" floor areas of the HLW building to address local and interaction effects. As a result of that study, it was concluded by BNI that a nominal load transfer of approximately 10% for one wall configuration occurred between the smaller to larger piers as the mesh is refined. BNI also concluded that the changes were within the normal variability that might be encountered in more refined versus approximated design analysis, and that no adjustment factors needed to be developed due to the design margins available at that time. The DNFSB staff did not agree with this conclusion and therefore these issues remained unresolved. After several discussions, a closure plan was agreed upon in November, 2004. This provided a path forward on resolving the mesh density issue. During a January 11 - 13, 2005, meeting, BNI presented the results of the study of global as well as local effects. The methodology involved comparing the displacement and force resultants based on the project design model for HLW at the same location to results of successively more refined models using SAP2000 software. The initial SAP2000 model had the same number of nodes and elements as the production model of GTStrudl model. The next SAP2000 run used four times the number of elements (a 2 x 2 replacement) and the last SAP2000 analysis used nine times the number of elements (a 3 x 3 replacement). The number of nodes increased proportionally. GTStrudl in-plane shear results were within 3% of the converged SAP2000 results, and in-plane moments were generally within 10% of the converged SAP2000 results. It was concluded that on a global basis, finite element models using GTStrudl software provided sufficiently conservative design loads. However, considerable uncertainty remained in local areas of HLW where discontinuities had strong influence on results.

Because of the result of the above studies, and the upcoming RGM issue around the same time, ORP directed BNI on January 24, 2005 (05-WED-002), to develop an acceptable strategy for closing the mesh density issue in conjunction with the ground motion re-analysis; and incorporate a mesh refinement to a 3x3 mesh density as measured against the existing GTStrudl mesh size using equivalent methods to SAP2000 software. In addition, areas of unusually complex geometry need to be reviewed and where needed, a further local refinement of the mesh will be made. DNFSB reiterated the issue as open in their letter dated April 19, 2005. Based on the above, BNI decided to use SAP2000 software (since it is capable of handling much larger numbers of elements in the finite element model), and perform parametric analysis to establish the number of elements required to provide convergence of the analysis for areas of irregularities, openings, and offsets. In November 2005, BNI completed the facility models with the SAP2000 software and successfully briefed the DOE PRT and DNFSB staff demonstrating adequacy of the revised models.

This change in the analysis methodology closed one of the major DNFSB issues, and provides a high level of confidence to the ORP that the resulting finite element forces are sufficiently accurate for structural design.

9.0 REVISION 10 OF THE STRUCTURAL DESIGN CRITERIA (SDC)

BNI issued Revision 10 of the SDC (24590-WTP-DC-ST-01-001) in December 2005 to incorporate the RGM spectra. The revision also incorporated changes from Tables 1 and 2, provided guidance on finite element mesh refinement, and amended the allowable D/C ratios. These changes have been reviewed and concurred by DOE, the PRT and the USACE independent review team. However, it should be recognized that a number of conservatisms still exist in the design that were not addressed in the revised SDC. SDC still limits the use of the inelastic energy absorption factors for building structures, to those portions of the building that have been constructed to the existing design criteria and to equipment already placed or procured. Inelastic energy absorption factors will not be used for new construction which is allowed by DOE 1020-94. The coherency concept, as evaluated, demonstrated that it reduces the seismic loadings on facility structures and components. This concept has not been incorporated in the SDC or the design at this time.

10.0 CURRENT DESIGN STATUS

New seismic design loads for both the PT Facility and the HLW structures have been calculated for the RGM by incorporating the updated soil profile and the most up-to-date structural models of the buildings in September 2005. Evaluation of the results indicate that the increase in seismic building load is generally less than 40%, confirming the adequacy of interim criteria for building design.

GTStrudl models for facility design were updated to align the model with the current facility layout and loading configurations. The models were then converted to SAP2000 models, and the mesh was significantly refined to resolve the concerns regarding the coarseness of the finite element mesh. The SAP2000 models were also refined to align more closely with the building geometry. The model conversions and the static analysis for the design of the facilities have been completed.

The design of the facility structures, systems and components are currently being re-evaluated in accordance with the SDC, Rev. 10, based on the revised loads from the SAP2000 analysis, and the revised ISRS, respectively.

11.0 ORP STRUCTURAL DESIGN OVERSIGHT

DOE instituted an independent Peer Review Team (PRT) to review the design of WTP facilities in May 2003 due to the importance of the project and complexity of the geometry and the analysis of the facilities. The objective of the PRT was to broadly review the design and construction processes to determine if the design and construction are code-compliant. Initial PRT review was focused towards the BNI structural design processes, including flow down of requirements, modeling methods, design methods, and appropriateness of design margins. After that, the PRT continued review of the structural design criteria and the design deliverables addressing unique features of the WTP design (e.g., wall/floor offsets, load transfers, modeling, construction approaches, existing design margins, etc.) at risk due to closed-couple design schedule, in reference to the key assumptions that could result in unanticipated design changes.

The initial PRT review took place in June 2003, and the reviews have continued on a fairly routinely, as often as monthly. PRT review of the BNI design has been thorough and detailed. Also, the PRT interacted with the DNFSB staff on numerous occasions to discuss the issues they had raised, and also provided their conclusions to the DNFSB staff. The PRT review efforts were led by John Treadwell of ORP (Civil/Structural) and by Wahed Abdul since early in 2005. The PRT review team consisted of high qualified, industry recognized members: Fred Loceff (Team Lead, Structural and soil-structure interaction); Loring Wyllie (Structural design and construction); Dr. Greg Mertz (Structural and soil-structural interaction); Dr. Terrence Holland (Concrete technology and concrete construction). Dr. Holland is no longer participating in the PRT review as the concrete mix design issues have been resolved. Summary resumes for the PRT team members are included in attachment C.

In January 2005, DOE initiated a new PRT team to review key process equipment and piping to ensure that the equipment are adequately qualified for seismic loading and adequate for supporting an anticipated 40-year design life. The PRT was also chartered to provide recommendations for improving design, and to identify any concerns with the design, fabrication, and installation of key process equipment and piping. The PRT consisted of industry-recognized experts: Timothy Adams, George Antaki, Fred Loceff, and Quazi Hossain. This effort was also led by Wahed Abdul of ORP.

The following list provides a brief summary of the reviews performed by the PRT:

- June 2003: The PRT review included the review of design and construction processes, organization, structural design criteria, procedures, design calculations, and finite element modeling for PT Facility, etc. The PRT concluded that the design process was sound and should provide code-compliant design. The construction process was also judged to be excellent. However, they noted some issues with the finite-element modeling, the design of the belowgrade concrete tunnels, and the "coupling" of flexible steel structures and stiffer concrete structures in the analytical model.
- September and November 2003: The PRT reviewed the adequacy of the design calculations for the HLW Facility to support the release for construction of the -21'-0" base slab and the -21'-0" slab to grade walls. The review agreed that the design was sufficiently conservative for construction for below-grade structures. The PRT also made some comments that needed to be resolved before the completion of the above-grade designs. The PRT also discussed the conclusions with the DNFSB staff. The PRT reviewed the BNI SSI analyses of the PT and HLW buildings.
- November 2003 – February 2004: DOE and the PRT developed the guideline for BNI to develop their summary structural report (SSR) for the HLW Facility. The guideline stressed that the report shall detail 1) the "load path evaluation" to provide a sanity check for the modeling assumptions, and 2) the structural analysis process. The PRT also developed a plan for the review of the SSR.
- December 2003 – March 2004: DOE and PRT developed the review plan for the LAW Facility to determine the adequacy of the concrete and steel design and the design process for this facility.
- January 2004: The PRT prepared a report on past reviews performed on the design of PT Facility and participated in the presentation and discussions with the DNFSB staff. The PRT also initiated the review of LAW Facility design
- March 2004: The PRT review consisted of the review of the BNI development of the SSR, resolution of the previous PRT comments on the HLW design, adequacy of HLW slab 0'-0", and completion of the LAW design. The key observations made on the LAW design were the lack of "collector" steel across the critical wall discontinuities, and the mixing of welded and bolted connections for gusset plate designs. The PRT also noted that the design calculations showed significant improvements in the documentation area, and BNI had resolved the majority of the HLW comments.
- June 2004: The PRT team participated in the review and discussion with BNI for the resolution of LAW and HLW comments. They noted additional concerns with BNI's interpretation of the uniform building code (UBC) as it relates to "collector" elements, and in the use of "omega" factors.

- June 2004: The PRT team reviewed the progress of the BNI studies towards the development of the SSR. The team accepted the study progress in the areas of the load distribution mechanisms; however, the PRT noted that additional studies were required for the redistribution of the lateral load due to out-of-plane wall cracking. The PRT also participated in the review of additional PT and HLW design and closure of earlier comments on the PT design. The PRT noted that full-length section cuts are needed for the out-of-plane shear calculation; raised questions on the adequacy of the roof bracing design; noted that the crane rail bracket support needs to be checked for lamellar tearing due to high loads; and made miscellaneous other comments.
- August 2004: The PRT participated in review and discussion with BNI and the DNFSB staff to evaluate the closure of past issues and the progress of the four parametric studies being performed by BNI for the validity of the BNI load redistribution evaluation and the mesh density concerns on HLW.
- October 2004: The PRT evaluated the status, the results, and the conclusion of the BNI parametric studies for HLW, and concluded that additional work needed to be performed by BNI to make sound conclusions. The PRT also came to an agreement with BNI on the approach to using SAP2000 software with 2 x 2 and 3 x 3 times the original mesh density, and to compare with the original GTStrudl results towards the resolution of the mesh density concerns raised earlier.
- November 2004: The PRT and DNFSB staff jointly reviewed the BNI closure of issues with the PT design, and agreed to follow-on with the results of the HLW parametric studies performed by BNI.
- December 2004: DOE brought industry experts in the area of seismic qualification of the equipment to brainstorm the impact of potential increase of 30%/40% in the seismic ground motion on the already constructed facility walls and slabs, already fabricated equipment, and other commodities. The review team came up with the path forward of ranking a number of embedded conservatisms in the design criteria and the potential for offsetting some of the RGM impacts.
- December 2004: The PRT review of the concrete detailing for PT resulted in some comments regarding the use of the lap splicing
- January 2005: DOE initiated a new PRT to perform a review of the key process equipment and piping to ensure that the equipment was adequately qualified for seismic loading and adequate for supporting an anticipated 40-year design life.
- January 2005: DOE, PRT, and DNFSB staff participated with PNNL in the review of the development of the RGM spectra. The review also included review of the extent of existing design margins in the PT and HLW. The team also reviewed the BNI studies regarding the mesh density concerns and concluded that there was little effect on the global structure, thus a re-analysis of the entire structure would not be necessary. However, it could not be concluded that the results of the original GTStrudl model are conservative for the design of some local individual elements.
- February 2005: The PRT reviewed the BNI design of HLW concrete and steel design for elevation 0' to 14'.
- March 2005: The PRT reviewed the steel design for higher elevations and the BNI development of the mesh density correction factor based on the comparative study done with the SAP2000 modeling. The PRT also reviewed and recommended some modifications for the acceptance of the ISC developed by BNI incorporating the RGM. The DNFSB staff also reviewed the ISC and

the development of the subject white papers that addressed some of the reductions planned for prior conservatisms.

- March – November 2005: The PRT had extensive participation in the review of the ISC, the path forward towards implementation of the RGM, the conclusion of the white papers, and the implementation of the ISC into the design.
- May – June 2005: The PRT reviewed the revision of the BNI criteria for seismic analysis (SADC) to ensure the incorporation of the RGM and BNI's development of white papers to justify removal of excess conservatisms from the design criteria. The DNFSB staff performed a separate review of the same issues.
- July 2005: The PRT and DNFSB staff participated in the review of the dynamic analysis of the PT and HLW facilities and the progress and incorporation of the results from BNI white papers. The PRT was tasked to develop the thermal white paper.
- August 2005: The PRT reviewed the implementation of the RGM in the BNI SADC; reviewed BNI's position on the mesh density; and reviewed and recommended for DOE approval BNI's interim designs using the ISC for the concrete placements.
- September – November 2005: The PRT continued review and acceptance of BNI design calculations for concrete placements using ISC.
- October 2005: The Equipment PRT performed initial review of the equipment and commodity designs. The review included the various design criteria and specifications for vendor design and fabrications, verification of vendor design documents, etc. The initial review resulted in a number of recommendations for BNI to implement to improve the design. DNFSB staff participated in the same review.
- November 2005: The Structural PRT reviewed the BNI implementation of the finite element models to the SAP2000 software, and the supporting parametric studies performed by BNI for its validation. The PRT also reviewed the BNI revision 10 to the SDC. The DNFSB staff also performed an independent review of the revised SDC criteria.
- December 2005: The PRT, in conjunction with BNI experts, continued with additional fatigue evaluations of the concrete due to thermal cycles in order to resolve DNFSB staff concerns. The Equipment PRT reviewed the BNI proposed changes to the piping design criteria made to incorporate inelastic energy absorption factors, and ASME B31.3 design allowables.
- January 2006: The Structural PRT began reviewing the redesign of the structural steel framing for the Analytical Laboratory.

DOE plans to continue review of BNI structural and equipment design using the PRTs throughout 2006.

12.0 CONTINUING ANALYTICAL AND DESIGN CONFIRMATION

The facility design has progressed substantially since its initiation, and the facility layout has matured. Equipment sizes and weights, distribution systems, and structural layouts are well-established, and only a minimal number of changes are anticipated in these areas. The seismic ground motion response spectra have been updated to the new site-specific soil data. Finite element models for the design of the facilities have been refined to a mesh density of a minimum of nine times the original mesh to provide an analysis that is more accurate by taking into account the openings, offsets, and irregularities in the structures at different areas. Models also incorporated the current facility configurations. Detailed design revalidations of the SSCs are ongoing, with the priority given to the facility concrete and steel design, piping design, and vessel design.

The only uncertainty in the structural loading and configuration considered to remain is a potential additional change in seismic ground motion. This concern stems from the remaining DNFSB issue that the soil characterization lacks sufficient field verified data for the deep basalt layers under the WTP site. The issue and the actions being taken regarding the uncertainty in the RGM spectra and its resolution are described below.

12.1 CONFIRMATION OF REVISED GROUND MOTION ADEQUACY

The standard procedure used to define the surface ground motions involves performing probabilistic site response evaluations and obtaining the mean surface response spectrum. This procedure assumes that the basic site properties defined in terms of a base case shear wave velocity profile and its potential variability across the site footprint, together with the material strain-related stiffness and damping properties, are known from site geotechnical investigations. The surface spectrum is defined as the mean of the surface spectra generated from the many individual convolutions. The selection of the mean spectrum follows the general recommendations contained in NRC Regulatory Guide 1.165, NUREG/CR-6728, the recent ASCE 4-98, as well as DOE-STD-1022-94 and -1023-95.

For the WTP site, good geotechnical and geological site profile information is generally available for the site soils encountered to a depth of several hundred feet. However, information on the interbed sequence at deeper depths was only able to be estimated from a sparse data set. The properties of this interbed sequence were found to play a dominant role in determining appropriate site amplification factors. To compensate for this uncertainty in interbed material properties, the recommended surface DRS was developed using the 84th percentile site amplification results generated from the probabilistic data set of site amplification, instead of the mean (per DOE-STD-1020-94). This additional conservatism applied to the recommended site amplification factors was added to accommodate the uncertainty in material properties of the interbeds that exist below the WTP site. There are still some concerns whether the actual site data is bounded by the conservative approach taken to develop the revised ground motion; likewise, there are some that believed the mean would have adequately represented the site risk.

To confirm that the current conservatively revised ground motion bounds these uncertainties, including the use of soil attenuation model compared to the rock model currently considered appropriate for the site, DOE has planned for additional evaluations of the soil characterization and the attenuation models. A drilling plan and statement of work have been developed and subcontracts are being awarded to drill multiple deep bore holes at the site to a depth of 1,500 ft to obtain shearwave velocity and other soil characterization data for basalt interbed layers. Once the data is available, the site response analysis will be reperformed using the appropriate attenuation models for the WTP site. This effort is scheduled to be completed in 2007.

It is anticipated that with the improved definition of the properties of the site profile from the deep drilling program, the mean spectrum will be less than the current revised design spectra based on the

84th percentile RAF. Above-mentioned changes will be incorporated in the future ground motion spectra when the deep bore soil characterization is made, to confirm the adequacy of the current RGM.

12.2 EXTERNAL INDEPENDENT REVIEW OF DESIGN

Because of the above uncertainties existing in the RGM and the subsequent impact on the facility design and path forward, the DOE, Office of Environmental Management requested the USACE in July 2005 to conduct independent reviews of the WTP project for the following: 1) development and implementation of the revised seismic design criteria, and 2) activities to gather additional geophysical data to confirm the revised seismic design criteria.

In accordance with the DOE request, USACE set up two independent teams that included industry recognized consultants to perform reviews of the RGM and the implementation of RGM to the design. One of the teams was formed to perform independent analysis to determine the basis for the revision to the seismic design criteria, the application of the design criteria to the facility design and safety analysis, and the assignment of design safety margin, as well as the oversight review of the structural analysis and design. This includes performing "over-the-shoulder" design reviews of ongoing design activities against SDC Rev. 10 to ensure code compliance and safety of WTP SSCs are being addressed, while cost and schedule impacts are being minimized. The second team was chartered to review the plan for gathering additional geophysical data to confirm the revised seismic ground motion; prepare detailed specification for the drilling subcontract(s); assist in the field oversight/inspection of the PNNL drilling, provide a review of subcontractor(s) to collect added seismic data; and independently evaluate the recommendations stemming from the collection of this added data.

The USACE has started independent confirmation of WTP design adequacy by conducting the following reviews:

- August 2005: The USACE reviewed the calculations for individual concrete placements in conjunction with the PRT.
- October 2005: USACE performed initial review of the seismic qualification of equipment and the commodities in conjunction with the Equipment PRT.
- November/December 2005: USACE reviewed the revised SAP \2000 models, the white papers, the SADC, and the SDC. Based on the resolution of their comments, the USACE concurred with the above documents.
- January 2006: The USACE team reviewed the dynamic analysis of the facilities. Comment resolutions are ongoing.
- February 2006: The USACE review team has completed the review of the DOE plan for the drilling of deep bore for collection of confirmatory soil characterization data.

The USACE has provided review plans to perform detailed reviews of the BNI design through the summer of 2006. Based on the confidence level from the PRT review process, it is not expected that it would result in changes to the design.

13.0 CONCLUSION

The seismic ground motion and the structural design of the WTP SSCs have been reviewed extensively by DOE, the PRT, and the DNFSB staff since 2002. Based on the reviews, a number of concerns were raised in the area of the seismic ground motion and the structural design. These have been resolved by BNI through additional evaluations with the participation of DOE, PRT, and DNFSB staff. Revised design criteria, the dynamic analyses, and the subsequent facility analysis have been reviewed satisfactorily by the PRT and DNFSB staff. ORP considers that the actions taken to address the RGM, reducing additional conservatism from structural and seismic analysis, and adoption of SAP2000 software are prudent and provide the necessary assurance that continuing design and construction do not expose the project to excessive risk.

Uncertainty in the RGM was considered by DOE as being already bounded by the method of the development of the RGM, where the 84th percentile was used in response to define a conservative representation of the mean surface ground motion. However, due to the continued DNFSB concerns over lack of sufficient site data in the development of the RGM, DOE has made the decision to perform deep bore drilling at the site to enhance direct estimates of subsurface dynamic properties. This decision was made in part to confirm that the RGM spectra are a conservative representation of the mean spectra. It is anticipated that with improved definition of the properties of the site profile from the deep drilling program, the recommended mean spectrum will be somewhat reduced as compared to the current design spectrum. However, the amount of reduction cannot as yet be determined until the deep drilling program will be completed. Due to the time needed for the drilling effort to reach to the depth of 1,500 feet and to perform the associated analyses, completion of the confirmatory analysis for the seismic ground motion is anticipated to be completed late in 2007.

In addition to the reviews by DOE PRT, an external, independent review of the designs and design criteria by U.S. Corps of Engineers (USACE) was initiated in August 2005. Reviews by these teams are ongoing and plan to be continued in 2006. The USACE review team has concurred with the implementation of the revised SDC and SADC, incorporating the RGM and the reduction of conservatisms, in the redesign effort.

DOE considers the currently recommended RGM for the WTP site to be a conservative estimate of the mean seismic hazard. It is noteworthy that the demand to capacity ratios in many of the major walls are significantly less than 1.0. This allows accommodation of transfer of loads without exceeding allowable code criteria, which provides added assurance of acceptable structural behavior during an earthquake, even if the future ground motion exceeds the design ground motion. The combination of multiple lateral load path capability in the design, together with the use of ductile detailing and the availability of untapped inelastic energy absorption characteristics of the structural elements indicate that the WTP facilities can absorb certain increase in seismic ground motion. In addition, the facility structures could be validated for significantly higher loads using coherency concepts, complex fragility or push-over analysis of structures, in the unlikely event of a future ground motion significantly larger than the RGM.

As of February 2006, significant funding reductions from Congress have resulted in stopping construction on both the PT and HLW facilities until late in the fiscal year. Facility design and re-validation of existing designs against the current RGM will be continued during this time to help mature the design, which will provide further assurance that the risk of proceeding with the project does not result in unacceptable risk.

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ATTACHMENT A. ANALYSIS AND DESIGN CONSERVATISM

Number	Item	Remarks	Value
Table 1 - Soil Structure Interaction Analysis			
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, a revised SHAKE analysis that incorporates the new motion will be made. This will result in new strained soil properties to be used in the SASSI model. The SHAKE 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-STD-1020-94 damping for Response Level 2 in the SSI analysis.	DOE-STD-1020-94 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 as 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 ISRS when broadening as permitted by ASCE 4-98.	When smoothing and broadening the envelope in-structure floor response spectra, ASCE 4-98 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

ATTACHMENT A. ANALYSIS AND DESIGN CONSERVATISM

Number	Item	Remarks	Value
Table 2 - Building Analysis and Design			
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
2.1	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 out-of-plane 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 center of gravity of the wall system.	B
2.6	Allow for F_μ greater than 1.0 for in-plane shear, bending and out-of-plane bending in accordance with DOE-STD- 1020-94.	In accordance with DOE-STD-1020-94, it is suggested that the project allow the use of F_μ factors. It is necessary to show that the building retains confinement during and after a seismic event if F_μ 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

ATTACHMENT A. ANALYSIS AND DESIGN CONSERVATISM

Number	Item	Remarks	Value
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 calculated and used. This process shall only be applied to constructed structures.	C
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.	B
2.9	Reduce conservatism in belowgrade wall design by using SASSI wall pressures.	Use the SASSI wall pressures for design of belowgrade 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

ATTACHMENT A. ANALYSIS AND DESIGN CONSERVATISM

Number	Item	Remarks	Value
Table 3 - In-Structure Response Spectra			
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 4-98.	This is an action that can be used for the interim review of equipment qualification.	A

ATTACHMENT A. ANALYSIS AND DESIGN CONSERVATISM

Number	Item	Remarks	Value
Table 4 - Equipment Qualification			
Number	**Item**	**Remarks**	**Value**
4.1	Reduce seismic demand on equipment by F_μ as permitted in DOE-STD-1020-94 and ASCE 4-98.	DOE-STD-1020-94 permits a seismic demand reduction for ductile behavior F_μ but does not specify the value of the ductility reduction factor F_μ. ASCE 4-98 specifies F_μ values for use in equipment seismic qualification. Passive, ductile, not pressure boundary = limit state A $\rightarrow F_\mu = 1.50$ to 2.00 Passive, ductile = limit state B $\rightarrow F_\mu = 1.25$ to 1.50 Active post-design basis event (DBE) with operator set = Limit state B $\rightarrow F_\mu = 1.00$ to 1.25 Active = Limit state D $\rightarrow F_\mu = 1.00$	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 4-98. 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 requalification 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. 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 4-98. Because much of the piping response is governed by flexible modes, a drop in high-frequency damping will have limited benefit.	A

ATTACHMENT A. ANALYSIS AND DESIGN CONSERVATISM

Number	Item	Remarks	Value
4.5	Pipe stress limit of min (3S; $2S_Y$).	The current allowable stress limit of 1.3S for SC-I and SC-II 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; 2SY}. 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 $F_\mu > 1.00$ as it would account twice for ductility effects. The adoption of $F_\mu > 1$ (3.1 above) is preferable, as it reduces demand, consistent with DOE-STD-1020-94 and ASCE 4-98.	B

ATTACHMENT B. CHRONOLOGY OF HANFORD SITE GROUND MOTION ISSUES

1. November 15, 1993: A presentation was provided to DNFSB staff on Hanford Site geology and seismic hazard characterization studies. The presenters included Robert Youngs of Geomatrix Consultants, Inc., Ann Tallman of Westinghouse Hanford Company (WHC), Alan Rohay of PNNL, and William Keil of the Washington Public Power Supply System.

2. December 16, 1995: A meeting was held between Hanford employees and DNFSB staff to discuss the Hanford Site ground motion issue. Participants included Asa Hadgian and Paul Rizzo (consultants of DNFSB staff), Ann Tallman of WHC, and Robert Youngs of Geomatrix.

3. Geomatrix Consultants, Inc. 1996: WHC-SD-TI-002, *Probabilistic Seismic Hazard Analysis*, DOE Hanford Site, Washington.

4. March 17, 1999: The WTP privatization project contractor, BNFL, decided to use the 1996 Geomatrix report. BNFL issued the validation result of the 1996 Geomatrix report as the basis for the WTP site seismic design.

5. June 30, 1999: 99-RU-0394, Regulatory Unit (RU) of DOE letter to M.J. Lawrence, BNFL, “Acceptance of peak ground acceleration (PGA) for the RPP-P facility design basis earthquake.” Accepting WTP Project to use the 1996 Geomatrix design basis acceleration and spectra (0.26g as the peak horizontal ground acceleration), which actually was based on 200 West Area soil data and was more conservative than that of 200 East Area (the location of WTP).

6. July 1, 2002: CCN: 035841, BNI letter to DOE, “Defense Nuclear Facilities Safety Board Issues.” Responding to DNFSB issues: requesting a specific report presenting the HLW and LAW facility load path behavior; recommending special detailing for the confinement of concrete in HLW; and concerns on the Geomatrix ground motion (need for additional parametric run; additional soil-column analysis; and need for energy plots). Based on the collective opinions of the Geomatrix, BNI, and independent experts, BNI recommends that the design continue with the current design motion.

7. July 30, 2002: DNFSB letter to DOE (from John Conway of DNFSB to Jessie Roberson of DOE), “The seismic design of the Pretreatment, Low-Activity Waste, and High-Level Waste (HLW) Facilities of the Hanford Waste Treatment Plant.” DNFSB expressed concerns on WTP seismic design. Specifically, the letter requested a DOE response on the following three areas: 1) the probability of tectonic activity of the anticlines and associated faults for Yakima Folds, 2) the spectral amplification associated with attenuation relationship, and 3) the amplified floor and equipment response of the superstructure. The letter did conclude “...current foundation design for HLW Facility includes sufficient margin to safely accommodate increase in predicted seismic loading that could result from these issues.”

8. August 14, 2002: ORP memorandum to Jessie Roberson (DOE), “Status of ORP actions to address DNFSB seismic design issues.”

9. September 18, 2002: DOE response letter from Jessie Roberson to DNFSB to the concerns noted in the DNFSB letter (Ref. 7). DOE submitted the position paper (ORP/OSR 2002-22, “Office of River Protection Position Concerning Assumed Probability of Tectonic Activity and Adequacy of Ground Motion Attenuation Model Used in the Design of the Waste Treatment Plant”) concluding that the probability of tectonic activity previously assumed and the use of California soil attenuation models in the 1996 Geomatrix work remains appropriate.

10. December 16, 2002: DNFSB letter to DOE (from John Conway of DNFSB to Spencer Abraham, Secretary of DOE), expressing concerns that the D/C ratio limit of 0.85 applied to the structural

design will be maintained only for the base mat and walls to grade, and not for other portions of the high-level waste structures.

11. January 21, 2003: DNFSB letter to DOE (from John Conway of DNFSB to Jessie Roberson of DOE), indicating that most of the technical issues involved in the WTP ground motion design criteria has been resolved. The only issue remaining was the approach used to develop attenuation relationship for deep geological formations to characterize the Hanford Site seismic hazard. DNFSB analysis of the existing data showed that the Hanford Site response in the frequency range of 4 to 10 Hz is about 15% greater than that of California sites.
12. February 5, 2003: 03-AMWTP-007, ORP memorandum to Jessie Roberson (DOE), "DOE response to the DNFSB letter of December 16, 2002(reference 10) regarding concerns about the WTP." The letter indicates that although the D/C ratio limit of 0.85 is not expected to be extended to abovegrade structures because of the more advanced definition and more advanced review of the structure design, significant structural design margin exists to account for uncertainties remaining in the development of design details.
13. October 28, 2003: ORP e-mail to DNFSB staff (Lewis Miller to Joel Blackman) providing the detailed plan for seismic borehole study at WTP to reduce the seismic attenuation uncertainty.
14. June 15, 2004: CCN 089932, BNI letter to ORP, "Hanford WTP Summary Structural reports (SSR) for primary process facilities."
15. July 4, 2004: letter from BNI, to DOE-ORP, "Design /Capacity Margins." The letter notes that facilities have added conservatism to the guidelines where they believed additional design/capacity margin was warranted at this stage of the design process. In general, the D/C ratio recommendations in the structural design criteria developed by BNI range from 0.85 to 1.0 for different elements in the HLW and PT structures, while the target values currently agreed to by BNI design supervisors and engineering managers were uniformly 0.85.
16. July 09, 2004: 04-WED-037, "SSR for the WTP." This letter documented the agreements reached with BNI on the contents of the SSRs for HLW, PT, and LAW facilities.
17. July 29, 2004: DNFSB letter to DOE (from John Conway of DNFSB to Paul Golan of DOE), "Design Basis Earthquake Ground Motion Criteria for the Hanford Waste and Waste Treatment Plant." Letter noted that the Hanford ground motion criteria was not conservative, and the DNFSB understood, that to compensate for this issue, the WTP contractor was implementing acceptably conservative design features and observed that this conservatism should be maintained for the future design unless site-specific attenuation relationships were developed. The DNFSB also requested a program plan specifying how ground motion issues will be addressed within 30 days of receipt of the letter. Specific issues to be addressed included: soil thickness and Vs for all layers; evaluation of how the laboratory data were corrected; justification of damping and modulus degradation curves; and how rock ground motion attenuation for Hanford can be developed.
18. September 1, 2004: 04-WTP-189, "Demand/Capacity (D/C) Ratios for HLW and PT Facilities." ORP directed BNI to maintain a D/C ratio of 0.85 on both HLW and PT reinforced concrete design until the resolution of finite element model mesh density on governing design loads for structural walls and slabs.
19. September 3, 2004: 04-WTP-202, ORP memorandum to Paul Golan (DOE), "Response to the DNFSB letter of July 29, 2004 request for WTP Program plan" (reference 14). The memo provides an ORP program plan for future reassessment of the seismic ground motion, particularly related to resolving uncertainty in shear wave velocity at different depths under the WTP site.
20. September 23, 2004: Lew Miller of ORP provides a presentation to DNFSB staff on Hanford Site ground motion issues, current status, and plans.

21. December 1, 2004: 04-WTP-273, "Request for participation in team review of impact of changes in estimates of predicted ground motion at WTP." The letter requests BNI to participate in the ORP expert seismic steering team review of the PNNL development of the RGM.
22. December 7-8, 2004: DOE and PRT met to identify conservatisms assuming at least a 30% increase in RGM. It was sent to DNFSB staff on December 11, 2004.
23. December 9, 2004: John Eschenberg, DOE Project Manager for WTP, briefs DOE Headquarters (DOE-HQ) on the upcoming increase in the RGM, and proposed path forward.
24. December 16, 2004: 04-WTP-279, DOE letter to BNI, "Management of emerging project items." Based on the new uncertainties seismic design basis and other technical issues, DOE directed BNI to evaluate the pace of design, construction, and procurements to minimize the risk of rework, and based on the level of risk, propose to DOE which areas are prudent to be retarded or ceased, and which areas are to be accelerated.
25. December 16, 2004: CCN 10841, BNI letter to DOE acknowledges stop work order on PT concrete.
26. December 29, 2004: CCN 101132, "Partial response to ORP letter 04-WTP-279 - Management of emerging project items." It provides the BNI preliminary risk evaluation recommendations to suspend various activities and slow down others. Specifically, the concrete placement was recommended to be put on hold until re-evaluation of the design is performed based on 30% increase in ground motion.
27. January 7, 2005: 05-WTP-008, "Release of PT wall sections 3-32and 3-37 for concrete placement." The first letter by DOE on the case-by-case approval of the BNI evaluation of the design for 40% increase in seismic ground motion. In addition, requested BNI to provide a list and engineering evaluation of the required near-term placements.
28. January 18, 2005: John Eschenberg, DOE Project Manager for WTP, briefs DOE-HQ on RGM.
29. January 24, 2005: 05-WED-002, DOE letter to BNI "HLW SSR Revision B." The letter identifies that the mesh density issue is still open and requested BNI to develop an acceptable strategy for closing this issue using finer (3 x 3) mesh refinement, other detailed calculations etc., in conjunction with the re-analysis effort to address the increase in the seismic ground motion.
30. February 01, 2005: 05-WTP-016, DOE letter to BNI, "Response to BNI recommendations on the action towards uncertainty of Seismic Design Basis." ORP letter acknowledges BNI recommendations (reference 26), and requests BNI to update models for SASSI runs; evaluate rationale for "conservatisms" that may be reduced; and to develop interim design criteria for the design of facility and components for construction release in advance of the completion of detailed model runs.
31. February 3, 2005: CCN 110738, BNI letter to DOE, "PT facility wall and slab placements." The letter provided the list of near-term placements of PT walls and slabs.
32. February 11, 2005: 05-WTP-036, OPR letter to BNI, "Delivery of revised Seismic Ground Motion Spectra to be used as the design basis for the design of the WTP." The letter delivered the RGM spectra showing a 38% increase in peak response and directed BNI to incorporate this into the design while minimizing the impact to the project. DOE also reiterated the need for developing bounding interim design criteria, and the evaluation of excess conservatisms.
33. February 18, 2005: CCN 111861, BNI letter to DOE, "Impacts of the revised seismic ground motion spectra." The first of the weekly impact lists provided by BNI to DOE. It also provided BNI decision that all existing calculations related to seismic must be re-evaluated, and noted that

the continued construction will have some risk of rework, even though significant efforts to minimize that risk is being made.

34. March 2, 2005: ORP memorandum to Principal Deputy Assistant Secretary for Environmental Management, "Transmittal of Revised Site-Specific Response Model for the WTP." The memo provided a copy of the *Site-Specific Seismic Site Response Model for the Waste Treatment Plant, Hanford, Washington* (PNNL-15089).

35. March 8, 2005: CCN: 089108, BNI letter to DOE, "Interim Seismic Criteria." The letter submitted the ISC applicable to all SC-I and -II SSCs for DOE approval.

36. March 10, 2005: 05-WTP-045, letter from DOE to BNI, "Clarification on the Identification of Impacts due to revised Seismic Ground Motion Spectra for the WTP." The letter clarified the guidance for the weekly impacts list provided by BNI, and reiterated the update of SASSI modelsand evaluation of the "conservatisms."

37. March 25, 2005: DOE briefing to DNFSB staff in Washington D.C. John Treadwell from ORP and Fred Loceff representing the PRT presented the ISC to the DNFSB staff. The meeting objective was to inform the DNFSB staff on ORP's plan to approve credible interim design criteria; continue design and construction in a cost effective manner; minimize the risk of rework; develop a final design criteria addressing RGM and mesh density; and maintain oversight using the ORP PRT. The technical basis for adopting the proposed ISC was discussed at length including demand to capacity ratios; utilization of across-the-board 40% increase in seismic response spectra; closure of the mesh density issue; and a near term schedule.

38. March 28, 2005: Received PRT review comments on the review of ISC via e-mail with recommended changes/additions to the ISC for acceptance.

39. April 1, 2005: 05-WTP-054, DOE letter to BNI, "Approval of the Interim Seismic Criteria for the WTP with comments." The letter provided the DOE and PRT approved modified ISC to BNI for implementation and use until September 16, 2005, and asked for the implementation schedule for all SSC re-evaluation.

40. April 27, 2005: Formal PRT correspondence to ORP approving the draft proposed approach by BNI (reference 41) for accommodating the mesh density issues on PT and HLW.

41. April 29, 2005: CCN 117383, BNI letter to DOE, "Mesh Density Strategy." This letter provided BNI strategy to purchase and validation of SAP2000 software to allow 3 x 3 mesh refinement of the facility models for checking the validity of GT Strudl models.

42. May 03, 2005: CCN 116994, BNI letter to DOE, "Revised Ground Motion Implementation Plan." BNI submittal of the implementation plan providing detailed plan for BNI's basis for decisions to proceed or suspend physical work; sequence and approach for completing verification of all SSCs; and method of documenting and tracking the incorporation of RGM for the existing SSCs.

43. May 25, 2005: DOE meeting with DNFSB staff at Washington D.C. A second meeting with DNFSB staff was held with John Treadwell and Wahed Abdul representing ORP and Fred Loceff representing the PRT. This meeting focused on BNI's rationale (white papers) for the reduction of previous conservatisms used in analysis and design, which could offset significant portions of the RGM spectra. In addition, more detail was provided on the path forward and schedule.

44. July 7, 2005: BNI submits via e-mail drafts of the revised SADC document, incorporating the RGM, and the white papers on the items of "conservatisms" for ORP and PRT review.

45. July 13, 2005: Memorandum from C.E. Anderson (Principal Deputy Assistant Secretary for Environmental management, DOE) to ORP, "WTP Program direction." The memo directed ORP that the approval of any work affected by RGM in PT and HLW will be approved by Mr. Anderson.

In addition, it directed ORP to develop a plan for an orderly cessation of construction work impacted by the RGM, with rationale for determining which activities are impacted.

46. July 20-22, 2005: DOE-PRT review of BNI SASSI analysis, and briefing to DNFSB staff at BNI San Francisco office. DOE and PRT met with the BNI dynamic analysis group to review the results of the just completed SASSI model runs using the RGM. Both the PRT and DNFSB staff were satisfied that the ISC was sufficiently conservative for building structure design. The design of equipment and piping, based on the ISC vertical in-structure response spectra in the PT building, poses some risk because of increases resulting from rocking motion. It is expected that these will be resolved on a case by case basis. In addition, BNI, ORP, and DNFSB staff reviewed and discussed the resolution of comments on the SADC submittal (Reference 44). A path forward was developed and agreed upon.
47. August 17, 2005: James M. Owendoff (Team Lead OEM, WTP Management Team, DOE) to James A. Rispoli (Assistant Secretary for EM, DOE), "Decisions concerning the design of the Hanford WTP."
48. August 19, 2005: 05-WTP-183, DOE letter to BNI, "Direction to develop a plan for the orderly cessation of seismically impacted construction work activities in the PT and HLW facilities."
49. August 22, 2005: CCN 124242, BNI letter to DOE, "Response to direction to develop a plan for the orderly cessation of seismically impacted construction work activities in the PT and HLW facilities," (reference 48).
50. August 31, 2005: CCN 127026, BNI letter to DOE, "Response to direction to develop a plan for the orderly cessation of seismically impacted construction work activities in the PT and HLW facilities," (reference 48).
51. September 13, 2005: 05-WTP-206, DOE letter to BNI, "Rejection of response to direction to develop a plan for the orderly cessation of seismically impacted construction work activities in the PT and HLW facilities," (reference 50).
52. September 16, 2005: CCN 127939, BNI letter to DOE, "Summary milestones for Implementation of the RGM."
53. September 16, 2005: 05-WTP-192, DOE letter to BNI, "Approval of ABAR 24590-WTP-SE-ENS-05-0017, Revision 1, "Implementation of the RGM spectra into the SRD."
54. October 18, 2005: CCN 128930, BNI letter to DOE, "Additional milestones for Implementation of the RGM."
55. October 06, 2005: 05-WTP-208, DOE letter to BNI, "DOE PRT review of BNI implementation of the ISC for WTP."
56. October 17, 2005: DNFSB letter to DOE (Samuel Bodman, Secretary of Energy), "Review of the design and construction of the Waste Treatment Plant at the Hanford Site."
57. October 13, 2005: PRT review of equipment.
58. October 24, 2005: CCN 130182, "Response to DOE PRT review of BNI implementation of the ISC into structural calculations."
59. November 10-14, 2005: DOE, PRT, and BNI briefing to DNFSB staff at Hanford.
60. November 2005: BNI submittal of the revision 10 of the SDC.
61. December 7, 2005: DOE letter of concurrence on the revised SDC.
62. December 20, 2005: BNI incorporates DOE, PRT, USACE, and DNFSB staff comments and submits the final version of SDC, Rev. 10.

ATTACHMENT C. RESUMES FOR THE PRT TEAM MEMBERS

Loring Wyllie
Structural Engineer and Senior Principal
Degenkolb Engineers
San Francisco, California 94104-4207
415.392.6952

Summary

Loring A. Wyllie, Jr. has 40 years professional experience. His work has included seismic evaluations, analysis, and design of strengthening measures for improved seismic performance. A number of these buildings are of historical significance. He is a past Chairman of the State Historical Building Safety Board, whose mandate is to evaluate and analyze methods for strengthening buildings that reserve their historic character. Mr. Wyllie is past-President of the Earthquake Engineering Research Institute (EERI). His contributions to the profession of structural engineering were recognized by his election to the National Academy of Engineering in 1990. He was also made an Honorary Member of the Structural Engineers Association of Northern California. In recognition of Mr. Wyllie's expertise in concrete design and performance, and his long service on the Building Code Committee, the American Concrete Institute named him an Honorary Member in 2000. Mr. Wyllie was elected an Honorary Member of the American Society of Civil Engineers in 2001.

Education

M.S. University of California, Berkeley, 1962
B.S. with Highest Honors, University of California, Berkeley, 1960

Registration

P.E. Civil/Structural from California, Oregon, Utah, Nebraska, Texas, and Wyoming

Professional Affiliations

International Association for Bridge and Structural Engineering: Vice President, 1997 – 2005
Chairman, USA Group, 1987 to present; Chairman
Organizing Committee, Annual Meeting, 1995
Member, Working Commission III, Reinforced Concrete, 1985 – 1993
Earthquake Engineering Research Institute: President, 1995 – 1997
Member, Steering Committee, Eighth World Conference on Earthquake Engineering, 1984
State Historic Building Safety Board: State of California, 1976 to present: Chairman
American Society of Civil Engineers: President, San Francisco Section, 1980 – 1981
Chairman, Committee on Concrete and Masonry Structures, 1981 – 1984
Chairman, Joint ASCE-ACI Committee on Reinforced Concrete Columns
American Concrete Institute: Director, 1985 – 1988
Member, Committee 318, Standard Building Code, 1972 to present
Structural Engineers Association of Northern California: President, 1985 – 1986
Chairman, Building Codes Committee, 1971 – 1972
Chairman, Seismology Committee, 1975 – 1976, Honorary Member, 1998.
International Association for Earthquake Engineering: Director, 2000-2008.

Loring Wyllie (cont'd)

Experience

Mr. Wyllie has served on the U.S. Department of Energy, Office of River Protection peer review team for the design of three major structures designed to PC-3 and PC-2 DOE standards. Included extensive review of criteria, calculations, drawings and project meetings and participation in DNFSB project review meetings.

Pantex Plant Amarillo, Texas. Mr. Wyllie has consulted and provided peer review for the seismic analysis of Buildings 12-64 at the Pantex Plant. Provided structural analysis to seismically qualify new equipment or PC-3 seismic exposure.

Stanford linear Accelerator Buildings 005 and 272 Stanford University Stanford, California. Mr. Wyllie performed detailed evaluations and developed strengthening concepts for two buildings at the Stanford Linear Accelerator.

Los Alamos National Laboratory Los Alamos, New Mexico. Mr. Wyllie performed peer review for various projects, including an incinerator facility seismic upgrade, and evaluated the seismic capacity of the general laboratory and administration building.

Chemical and Metallurgy Research Building Los Alamos National Laboratory Los Alamos, New Mexico. Mr. Wyllie served on the peer review panel to review the structural aspects of the Conceptual Design Report and Title Design for this building.

Plutonium Building 332 Lawrence Livermore National Laboratory Livermore, California. Mr. Wyllie performed a dynamic analysis of the facility in accordance with U.S. Department of Energy seismic criteria and evaluated the capability of the existing structure to comply with it. Discussed members and connections that were over stressed and recommended structural retrofit solutions. Walked down several selected piping and duct systems, determined worst case conditions by judgment, and analyzed these conditions for DOE standards. Developed retrofit solutions that carefully considered all interferences and adjacent systems.

HEUMF Peer Review Oakridge, Tennessee. Mr. Wyllie was a member of peer review team for the conceptual design of the Highly Enriched Uranium Materials Facility at the Oakridge National Lab for the U.S. Department of Energy.

Impact Tester Building Los Alamos National Laboratory Los Alamos, New Mexico. This small building, an addition to the Plutonium facility, was designed by Merrick & Co. Several years after construction, Mr. Wyllie was asked to peer review the design to resolve review comments. Degenkolb performed dynamic analysis using soil springs and restraint springs for filler between adjacent buildings to determine compliance with DOE standards.

Senior Seismic Review and Advisory Panel (SSRAP). Mr. Wyllie provided consultation and review as part of this panel. Assembled by the Nuclear Regulatory Commission (NRC) and the Seismic Qualification Utility Group to advise them on the seismic evaluation issues associated with equipment in older nuclear power plants, this advisory group included engineers from various disciplines experienced in seismic design. The panel members reviewed data from various sources and developed generic criteria.

Loring Wyllie (cont'd)

Other Review Panels

Peer Review Group, Seismic Margins Evaluation of Maine Yankee Nuclear Power Plant, for Lawrence Livermore National Laboratory (LLNL) for NRC, 1986 to 1987.
Peer Review Panel, Seismic Margins Evaluation of Hatch Nuclear Power Plant, for Sandia National Laboratory for NRC, 1988-1989.
Panel Member to Review EPRI 5930 on OBE Exceedance Criterion, 1989.
Review of Selected Systems for Seismic Ruggedness, Savannah River Plant, for Du Pont, 1988.
Panel Member for the Seismic Isolation Study for a New Production Reactor, for Argonne National Laboratory and Department of Energy.
Senior External Events Review Group, for New Production Reactor, for Lawrence Livermore National Laboratory for the Department of Energy, 1991-1993.
Structural Advisory Committee for Westinghouse Savannah River Company at the Savannah River site, 1992-1994.

Greg Mertz
Los Alamos National Laboratory
1921 Camino Durasnilla
Los Alamos, New Mexico, 87544
Work (505) 606-0264

Summary

Results oriented Ph.D. Structural Engineer who enjoys challenging technical assignments. Significant experience in successfully applying advanced structural analysis methods including: nonlinear pushover analyses; nonlinear dynamic analyses; impact (aircraft, drop, tornado missile) analyses; fracture mechanics (LEFM, EPFM, Leak-Before-Break); fragility analyses; and soil-structure interaction analyses. Experienced structural designer for both nuclear and non-nuclear buildings. Experienced in the structural qualification of existing DOE structures. Performs failure investigations of both building structures and mechanical components. Independent structural peer reviewer for nuclear facilities throughout the DOE complex. Significant experience resolving technical issues with customers and regulatory bodies. Successfully leads engineering teams through technically demanding projects.

Education

University of Missouri - Rolla
Ph.D., Civil Engineering, 1989
M.S. Civil Engineering, 1986
B.S. Civil Engineering, 1979
Continuing education in probabilistic analysis methods, nonlinear analysis and fracture mechanics

Registered Professional Engineer

Registered Professional Engineer in Georgia and Missouri

Professional Experience

Los Alamos National Laboratory (2005 – Present)
Technical Staff Member, Probabilistic Structural Mechanics Team, D5 – Nuclear Design and Risk Analysis Group. As a Technical Staff Member, performs structural evaluations of both new and existing safety related structures, develop retrofit strategies, resolve structural safety issues to support continuing facility operations and peer review engineering submittals. Performed accidental impact analyses for nuclear facilities. Developed a seismic retrofit strategy for the Radioassy and nondestructive test facility.

Waste Treatment Plant, DOE Office of River Protection (2003 – Present)
Member of a Peer Review Team advising on the adequacy of the BNI Waste Treatment Plant structural design effort. This team is actively reviewing the structural design of the High Level Waste, Pretreatment and Low Activity Waste buildings.

Westinghouse Savannah River Company
1999 – 2004. Manager, Structural Engineering Group, Structural Mechanics Section. Managed a group of 10 structural engineers who design new facilities, evaluate existing facilities to support new missions and resolve safety issues to support continuing facility operations.

Greg Mertz (cont'd)

1993 – 1999. Engineer, Structural Engineering Group, Structural Mechanics Section. In the Structural Engineering Group, progressed from Senior Engineer to Fellow Engineer. This effort included nonlinear pushover analyses, numerous nonlinear dynamic analyses and development of a probabilistic damage model for reinforced concrete joints. Presented and defended these analyses to a DOE Independent Review Team and the Defense Nuclear Facilities Safety Board.
Performed nonlinear pushover analyses of the Rocky Flats 371 Building. Presented and defended this analysis to the Defense Nuclear Facilities Safety Board.
Identified an appropriate analysis methodology, developed analytical tools and performed preliminary analyses to determine the stability of cracked steel High Level Waste Tanks.
Performed probabilistic fracture mechanics analyses to determine appropriate safety factors.
Performed a soil-structure-interaction analysis of the Idaho National Engineering Laboratory Irradiated Fuel Storage Facility. Presented this analysis to the Defense Nuclear Facilities Safety Board staff.
Developed a computer program to efficiently evaluate yield-line mechanisms, which quantify the collapse load of reinforced concrete slabs.
Performed both analytical and experimental investigation to determine the probable loading of a rappelling rope involved in a fatal accident and the rope's strength in the field condition.

1989 – 1993. Engineer, Materials Technology Section, Savannah River Technical Center. As Senior Engineer, performed structural integrity evaluations of defense reactor vessels, primary and secondary cooling systems, engineered safety systems and components.
Led a team of seven engineers in a structural integrity evaluation of four reactor engineered safety systems to address issues raised by a DOE Safety Evaluation Report.
Performed fracture analyses of a reactor vessel, primary piping and components for a Leak-Before-Break argument to support reactor power limits.
Developed acceptance criteria for wall thinning in low-pressure carbon steel piping and used these criteria to disposition In-Service-Inspection results.
Performed failure investigations of a fallen monorail crane trolley, leaking heat exchanger components, and a leaking rupture disk.
Performed reactor severe accident analyses considering high temperature material behavior, nonlinear structural response and the behavior of postulated flaws.

University of Missouri - Rolla (1983 – 1989)

As a graduate student, developed pushover and hysteresis models for the nonlinear dynamic analysis of low-rise reinforced concrete shear wall structures. To support this effort, wrote numerous computer programs to process dynamic test data and analyze shear wall structures. Also taught an undergraduate course in structural steel design.

Personal Accomplishments

Developed Seismic Inelastic Force Reduction Factors, Fu, for use in ASCE 43-05.
Performed nonlinear dynamic analyses to determine Seismic Inelastic Force Reduction Factors for shear wall structures including the effects of Soil Structure Interaction.

Frederick Loceff, P.E.
472 Timberwolf Trail
Martinez, GA 30907
706.863.5255

Education

M.S. Structural Engineering – Michigan State University (1965)
B.S. Architecture – University of Michigan (1963)

Professional Engineering Registration

Michigan, Pennsylvania, and Illinois

Technical Publication

Author of several technical publications and professional society presentations

Professional Activities

Member, University of South Carolina, Department of Civil and Environmental Engineering Industrial Advisory Board
Past Chairman, DOE Contractors Committee The Use Seismic Experience Data for Evaluate Existing Equipment
Past Chairman, DOE Contractors Task Force on Development of Requirements for the Design, Operation and Maintenance of DOE Facilities

Professional Experience

Worked in the Commercial and Government Nuclear Energy Industry since starting with the Westinghouse Electric Corp. in January 1971. Have held positions of increasing responsibility in Westinghouse from Senior Engineer to Department Manager. Prior to joining Westinghouse, worked as a structural engineer and analyst in aerospace and the commercial building industry.
Current position is that of Manager, Structural Mechanics for Westinghouse Savannah River Company (WSRC). In this position, directs the activities of 30 engineers who apply state-of-the-art technical capability to prepare studies and designs, and resolve issues relating to the structural capability of DOE safety class buildings, equipment, piping, and systems that provide confinement or containment of nuclear materials. Called on to present the results of the analyses and studies that include a high level of technical content, to the Defense Nuclear Safety Board and Industry Peer reviewers. Advanced techniques, such as push-over analyses and non-linear analyses, are used to qualify existing and new designs. During this period, served as a peer reviewer for several major DOE facilities.
Prior to managing Structural Mechanics, managed the 90-2 Program for WSRC. As manager of this program, directed the development of the standards and requirements documents for SRS facilities and for the general site and for the DWPF building in particular. The documents contain contractual requirements to maintain safety to the public, site workers, and the environment. They cover 20 functional areas important to the safety of DOE nuclear facilities including, Engineering, Fire Protection, Operations, Environmental Protection, Construction, Training, and Radiation Protection.
Served as Department Manager for Westinghouse Generation Technology Systems Division in Pittsburgh PA. In this position, directed the day-to-day operations of a department of over 500 engineers responsible for the design and evaluation of commercial nuclear and non-nuclear facilities. Participation in the commercial nuclear activities involved the implementation of design and analyses to satisfy NRC regulatory requirements as delineated in NRC regulatory guides and the standard review plan as applicable to buildings and equipment. The design and qualification of buildings and equipment for seismic loads was a routine activity.

Quazi Hossain
Lawrence Livermore National Laboratory, Livermore, CA
hossain1@llnl.gov
Office: 423-2289

Areas of Expertise

Structural/seismic design and safety evaluation of nuclear and hazardous facilities, structures, system,s and components. Safety analysis review, system safety classification, and development of QA plans. Development of design criteria, codes and standards for natural phenomena hazard (NPH) evaluation. Preparation and review of documents related to NRC license. Management of multi-disciplinary projects.

Education

Ph.D. University of California Davis, Structural Engineering, 1974
M.S. Texas A & M University, Structural Engineering, 1967
B.S. Bangladesh University of Engineering & Technology, Civil Engineering, 1963

Certificates/Licenses

CE Professional Engineer, California (since 1974)

Professional Experience

1992 – Present Lawrence Livermore National Laboratory, Livermore, California
- Principal Investigator, NRC's Advanced Light Water Reactor Design Review
- Project/Task Leader, DOE's Defense Program's seismic and tornado hazard projects

1974 – 1991 Quadrex Corporation, Campbell, California
- Manager, Structural & Seismic Engineering
- Manager of Engineering

1973 – 1974 Woodward-Clyde & Associates, Oakland, California
Specialist Engineer, Soil-Structure Interaction Analysis

1964 - 1970 Mymensingh University (World Bank Project), Bangladesh:
Asst. Professor, Executive Engineer

1963 - 1964 Berger Engineers (An affiliate of Louis Berger, Inc. of N.J.), Dhaka, Bangladesh:
Structural Design Engineer

- Assisted the US Enrichment Corporation in preparing NRC license documents.
- Participated in the development of DOE's standard for Aircraft Crash Risk Analysis.
- Participated in the development of seismic topical reports for the Yucca Mountain Nuclear Waste Repository Project for NRC review.
- Assisted the NRC in performing safety evaluation of Advanced Light Water Reactors (AP600, CE-80 plus)
- Developed the DOE Standard for Performance Categorization of Structures, Systems, and Components from failure consequence and risk considerations.
- Managed multi-disciplinary structural and seismic study projects and interfaced with regulatory authorities.
- Supervised seismic soil-structure interaction projects for four nuclear plants.
- Managed multi-disciplinary system safety studies for safety classification of equipment and components for two nuclear power plants, and participated in the reconstitution of design basis documentation.
- Supervised spent fuel densification projects for more than ten plants including structural and seismic design, fabrication, and installation of spent fuel racks.

Quazi Hossain (cont'd)

- Supervised piping design and non linear dynamic analysis of several nuclear plants.
- Supervised dynamic response and stress analysis of many types of mechanical equipment and electrical cabinets, racks, etc.
- Performed quality assurance and technical audits of major nuclear projects.
- Performed building design and evaluation using AISC, UBC, ACI Codes, and NRC regulations.

Publications

More than 50 technical publications in journals and proceedings of ANS, ASCE, and ASME conferences and symposiums on seismic design, analysis, and evaluation of structures and equipment

Societies and Committees

Member, Committee on Dynamic Analysis of Nuclear Structures, Committee, American Society of Civil Engineers (ASCE)
Past Chairman, ASCE Subcommittee on Seismic & Dynamic Analysis and Design of High Level Nuclear Waste Repositories
Past Member, Seismological Committee, Structural Engineers Association of Northern California
Member, Tau Beta Pi Honor Society
Member, ASCE working group on Standard 4-86
Chairman, ASCE Subcommittee on Analysis and Design for Seismic Fault Movement
Member, ANS Subcommittee on Safety Classification of Nuclear Facility Structures, Systems and Components

George A. Antaki, P.E., Fellow ASME
Westinghouse Savannah River Company Savannah River
Aiken, South Carolina
george.antaki@srs.gov
Tel. 803-952-9728

Education

M.S. Engineering, University of Liege, Belgium, 1975
M.S. Mechanical Engineering, Carnegie Mellon University, Pittsburgh, Pennsylvania, 1985

Publications

Over 30 papers published in the field of design and integrity of tanks, vessels and piping; seismic and extreme loads analysis and qualification, structural integrity, inspection, and fitness-for-service.
Author, "Piping and Pipelines Engineering," published by Marcel Dekker, New York.
Author, "Fitness-for-Service and Integrity of Piping, Vessels and Tanks," published by McGraw Hill, New York.

Professional Experience

Savannah River Site, Aiken, South Carolina, 1989 – Present.

Manager, Systems Structural Analysis, Structural Mechanics section. Manage a group of engineers, with responsibility for stress analysis, mechanical design, qualification and field support for equipment, components, piping and distribution systems. Expertise includes structural integrity and fitness-for-service analysis, ASME code (design, qualification, fabrication, and testing), stress analysis, thermohydraulics, seismic qualification, explosion and extreme loads analysis and qualification.

Chairman, Savannah River Site Pressure Equipment Protection Committee; Committee responsible for the initial and continued safe operation of over 2,500 pressure vessels and relief devices.

Chairman, Savannah River Site Piping and Valves Technical Committee; Committee responsible for site technical standards, piping and valves, ASME code compliance of materials, design, fabrication, NDE and pressure testing.

Engineering consulting and support to DOE sites: Los Alamos National Laboratory, Hanford, Pantex, etc.

Westinghouse Energy Systems, Pittsburgh, Pennsylvania, 1978 – 1989

Mechanical design, qualification and testing of nuclear and fossil power plant components and piping systems for normal operating loads (pressure, temperature, vibration) and extreme loads (pressure transients, postulated break, seismic). Construction/engineering interface for power plant completion. Startup testing of nuclear power plants (hot functional testing, vibration testing) and licensing reviews and utility client support with the U.S. Nuclear Regulatory Commission.

Westinghouse Nuclear Europe, Brussels, Belgium, 1975 – 1978

Mechanical design of nuclear reactor internals and fuel assemblies, manufacturing follow, shop fabrication oversight and quality control of mechanical components for nuclear power plants, site delivery and outage inspections and fitness-for-service of reactor vessel equipment, internals and fuel assemblies.

George A. Antaki (cont'd)

Code Committees

Fellow, American Society of Mechanical Engineers (ASME)
Chairman, Pressure Vessel Research Council, Subcommittee Dynamic Stress Criteria
Member, ASME Post-Construction Subcommittee Repairs and Testing
Member, Joint ASME-API Committee Fitness-for-Service tanks, Vessels, Piping
Member, ASME Committee Vessel Design for Explosive Loads
Member, ASME III Main Committee Qualification of Mechanical Equipment (QME)
Member, ASME III Working Group Piping Design
Member, ASME B31 Mechanical Design Committee
Member, ASME VIII Div.3 Task Group Impulsively Loaded Vessels

Teaching

Instructor ASME Course PD-394 Seismic Design and Retrofit of Equipment and Piping
Instructor ASME Course PD-398 Operation, maintenance and Repair of Plant Piping Systems
Instructor ASME Course PD-077 Failure, Failure Prevention and Repair of Pressure Vessels, Piping, Boilers and Rotating Machinery
Instructor DOE EH-0545, Seismic Retrofit (SQUG)

Timothy M. Adams
Chief Mechanical Engineer and the General Manger
Stevenson & Associates, Cleveland, Ohio

Summary

Mr. Adams is the Corporate Chief Mechanical Engineer and the General Manger of the Cleveland Office of Stevenson & Associates. He has over 24 years experience in the design of Pressure retaining components to Section III and Section VIII of the ASME Boiler and Pressure Vessel Code and the B31 series codes. In addition to his general management responsibilities, Mr. Adams is responsible for: project management; provision of technical consulting and actual design work in the areas of design/analysis of piping systems; pressure vessels/tanks; mechanical equipment; structures; and application of Industry consensus codes and standards for the electric power generation; petrochemical; and, process industries and DOE nuclear facilities.

Education

M.S. Mechanical Engineering, University of Pittsburgh, 1985
B.S. Mechanical Engineering, Summa Cum Laude, University of Pittsburgh, 1977

Registration/Certification

Certified Pennsylvania Engineer in Training (EIT)
Previous DOE Q-Clearance

Professional History/Experience

Stevenson & Associates, Cleveland, Ohio, 1991 - present

Chief Mechanical Engineer and General Manager. Mr. Adams is an expert in the application of experience based and traditional qualification techniques to the seismic evaluation of piping systems (aboveground, buried, etc.) valves, component equipment and supports for A-46, IPEEE, JCO evaluations and traditional design basis evaluations. Mr. Adams is also a SQUG trained Seismic Capability Engineer.

Significant accomplishments in this position include: Mr. Adams has been instrumental in the development of walkdown screening approaches for the seismic evaluation of piping systems. He has developed both analytical based and earthquake experience based walkdown methodologies for the seismic qualification of piping. He was instrumental in the development of the EPRI and DOE's JCO Seismic Walkdown Criteria and has successfully applied the experienced-based seismic qualification methodology put forth by the BWROG's for the MSIV leakage update program. He also developed and implemented an extensive program to use a walkdown approach to verify the seismic design basis for all safety related small bore piping in a major client's plant.

Project Manager, US DNFSB project. Responsible for budget and administrative management, and provision of technical consulting to the DNFSB in their oversight of DOE Defense Nuclear Facilities. Technical consulting provided in hazard evaluation, piping, pressure vessels/tanks, mechanical equipment design, application of Industry codes and standards and project management. This effort included the extensive review of waste management, processing, and storage facilities.

Timothy M. Adams (cont'd)

Project Manager, for two major USNRC Projects (> $350K ea.). The first project evaluated the changes required for application of Industry codes and standards to the design of Advanced Light Water Reactors. The second project involved reevaluation of the regulatory guidance in Regulatory Guides 1.142 and 1.143. Responsible for project planning, budget and cost reporting, technical direction of six associates, and provision of technical reviews of the work conducted by these associates.

Provision of extensive technical support to the Advanced Reactor Corporation in the development of the New ASME BPVC, Section III Piping Seismic Design Criteria for the Advanced Light Water Reactor. Mr. Adams, in conjunction with other S & A staff, has developed methodologies and techniques for the evaluation and resolution of low-cycle and high-cycle vibration fatigue failures in nuclear power plants. This includes extensive review of SIF test data and piping cyclic fatigue data.

Peak Technical Services Inc., Pittsburgh, Pennsylvania, 1989 - 1991

Director - ENSYS Division. Mr. Adams was a Director with the ENSYS Division of Peak Technical Services responsible for provision of technical consulting services, new business development, and the hiring, training, administration, and direction of 3 managers and 19 technical, sales, and clerical personnel.

Significant accomplishments in this position included: Provided design review and technical consulting services to clients on the ASME Boiler and Pressure Vessel and B31.1 Codes. This included reviews of Class 1 piping system analysis (including fatigue analysis) for PWR and BWR plants and the provision of general piping design services. Provided engineering and project management consulting services for a major Vendor of Combined Cycle Combustion Turbine Plants. Reported to the project manager and provided technical support and direction to the Piping, Mechanical, HVAC, and I & C sections to expedite completion of critical tasks associated with major project milestones and provided technical consulting services for steam hammer evaluation of the Primary Steam Supply System.

Westinghouse Electric Corporation, Pittsburgh, Pennsylvania, 1977 - 1989

Project Manager, Vogtle Unit #1 Piping Completion Program (2 years). Responsible for administration and direction of 7 supervisors and 70 engineering, clerical, and secretarial personnel. Implemented and managed: scheduling and planning, manpower allocation, budgets, and cost control procedures. Given this assignment with the charter to significantly improve project schedule and budgetary performance. Developed and implemented a significant project restructuring. This transformed the project from 2 months behind schedule and 40% over budget to completed 2 weeks ahead of schedule and reduced the cost overrun to 10%.

Manager, Applications Software Group (3 years). Responsible for the management of 15 software developers charged with development, upgrade, and error fixes for 55 engineering computer software packages used by 5 engineering departments in the Plant Engineering Division.

Senior Engineer, Engineer, Associate Engineer (8 years). 8 years experience in the Design and Qualification of nuclear (PWR, BWR) and non-nuclear Steam and Gas Electric Generation Plants. Demonstrated leadership capability and strong technical foundation resulted in assignment to 2 Divisional Task Teams for Design Standards development and 5 Divisional Task Teams for resolution of complex component failure and operational issues. Served as Lead Piping Engineer for 6 major projects providing all technical and non-technical direction. The quality and scheduler and cost performance on these projects directly resulted in the receipt $10 million in additional contracts.

Timothy M. Adams (cont'd)

Publications

Authored and co-authored over 40 technical publications in the Computer and Mechanical Engineering fields.

Professional Activities

Member, American Society of Mechanical Engineers (ASME)
Member, American Welding Society (AWS)
Member, ASME BPVC Section III, Division 1, Subgroup on Design
Member and past secretary, ASME BPVC Section III, Division 1, Working Group on Piping Design
Member, ASME Committee on the Qualification of Mechanical Equipment in Nuclear Power Plants (ASME-QME)
Member, ASME QME Subcommittee on General Requirements (QME-SCGR)
Chairman, ASME QME Subgroup on Dynamic Qualification (QME-SDQ)
Member Joint ASME/ASCE Special Task Group Buried Piping Design
Member, ASME BPVC Section III, Division 1, Special Working Group on Seismic Rules
Member, ASME/IEEE Special Working Group on Standardization of Experience Based Seismic Equipment Qualification (ASME/IEEE – SWG – SEBSEQ)
ASME Alternate Representative to Building Seismic Safety Council
Member, Pressure Vessel Research Council (PVRC), Subcommittee on Stress Indices and Flexibility Factors of the Committee on Piping and Nozzles

Honors/Awards

Recipient of numerous Westinghouse and Industry awards

Publications

2003, PVP Conference, Cleveland, Ohio, "Implementation of Experienced Based Seismic Equipment Qualification of the ASME-QME Standard – *A Status Report*," co-author.

2002, PVP Conference, Vancouver, Canada, "Background to Recent Revision of the Section III Piping Rules," co-author with J. Minichiello, R. Barnes, E. Branch, and Y. Asada.

2000, PVP Conference, Seattle Washington, July 24-28, 2000, "Development of a Walkdown Screening Criteria for Application in the Verification of Small Bore Piping Systems," co-author with Dr. Rolfe B. Jenkins.

2000, PVP Conference, Seattle Washington, July 24-28, 2000, "Application of Earthquake Experience Data to the Seismic Verification of Main Steam Isolation Valve Leakage Piping," co-author with John O'Sullivan and Dennis Zercher.

2000, PVP Conference, Seattle Washington, July 24-28, 2000, "Implementation of Experienced Based Seismic Equipment Qualification of the ASME-QME Standard," co-author with G. Antaki, et al.

1999, PVP Conference, Boston, Massachusetts, "Evaluation of the Response of Low Frequency Piping Systems to Strong Motion Earthquakes."

1999, NUREG/CR-5733, "Re-evaluation of Regulatory Guidance Provided in Regulatory Guides 1.142 and 1.143," co-author with J.D. Stevenson and G.G. Thomas.

1999, WRC Bulletin 441, "Development of a Comprehensive Static Seismic Analysis Method for Piping Systems," co-author with J.D. Stevenson.

Timothy M. Adams (cont'd)

1998, WRC Bulletin 437, "Assessment, Sample Problems, and Commentary on Design of Section III, Division 3, (NUPACK) of the ASME Boiler and Pressure Vessel Code."

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