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Seismic Analysis and Design Approach Document

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001	Complete Revision. Incorporated ACNO1, the revised ground response spectra, fragility calculation methodology, added Appendix F on seismic requirements for mechanical equipment and other miscellaneous changes.

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ACRONYMS AND ABBREVIATIONS

Acronyms

ACI	American Concrete Institute
AISC	American Institute of Steel construction
ANSI	American National Standards Institute
ASCE	American Society of civil Engineers
BDBGM	Beyond Design Basis Ground Motion
BSC	Bechtel SAIC Company
CDFM	Conservative Deterministic Failure Margin
DBGM	Design Basis Ground Motion
D/C	Demand/Capacity (Ratio)
DIRS	Document Input Reference System
DOE	U.S. Department of Energy
DRS	Design Response Spectra
HCLPF	High Confidence Of Low Probability of Failure
HVAC	Heating, Ventilation, and Air Conditioning
IBC 2000	*International Building Code 2000*
ISRS	In-Structure Response Spectra
ITS	Important To Safety
PDC	*Project Design Criteria Document*
PGA	Peak Ground Acceleration
RSA	Response Spectrum Analysis
SASSI	System for Analysis of Soil-Structure Interaction
SADA	Seismic Analysis and Design Approach
SEI	Structural Engineering Institute
SRSS	Square Root of the Sum of the Squares
SSCs	Structures, Systems, and Components
SSI	Soil-Structure Interaction
YMP	Yucca Mountain Project

Abbreviations

C_c	Computed capacity
D/C	Demand/Capacity ratio
D/E	Point D and Point E on Figure 3-3
ft	foot/feet
kcf	Kips per cubic foot
kip	Kilo (one-thousand) pounds
ksf	Kips per square foot
ksi	Kips per square inch
m	meter
mi	mile
pcf	pounds per cubic foot

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1. INTRODUCTION

The Yucca Mountain Repository site facilities will receive and emplace radioactive and hazardous materials. Consequently, it is necessary to ensure that an adequate level of safety is provided to the facilities, workers, and the public. To achieve this objective, the facilities are required to be designed to withstand the effects of natural phenomena hazards, such as earthquakes, without significant damage or loss of safety function.

This seismic analysis-and-design approach document presents the methods to be used for the preclosure seismic analysis and design of structures, systems, and components (SSCs) at the Yucca Mountain Project (YMP). This document is applicable to both surface and subsurface facilities. This document is complementary to *Project Design Criteria Document* (PDC) (BSC 2007 [DIRS 179641]) and should be used in conjunction with the same. In case of a conflict, the higher level document, PDC (BSC 2007 [DIRS 179641]) shall govern.

2. SCOPE

This document provides guidance for seismic analyses and design using the input data provided in the cited references. The guidelines provided herein are to be used for the preclosure seismic analysis and design of YMP SSCs. Studies regarding site geotechnical conditions and seismicity, and seismic design input based on those studies, have been completed by others. The geotechnical input is given in *Supplemental Soils Report* (BSC 2007 [DIRS 182582]). The seismic design input is provided in the PDC (BSC 2007 [DIRS 179641], Section 6.1.10) and includes design spectra at both surface and subsurface levels for 1,000-year, 2,000-year, and 10,000-year return period earthquakes. Corresponding time-histories have also been developed.

Surface facilities for the repository shall be designed for a preclosure duration period of 50 years. The preclosure duration design period for subsurface facilities is 100 years (BSC 2007 [DIRS 182131], Section 2.2.2.8).

The analysis guidelines include static as well as dynamic analyses. The static analysis procedures cover computation of seismic loads using static force methods. The dynamic analysis procedures cover soil-structure interaction modeling and analysis, and generation of seismic loads and in-structure response spectra (ISRS) for qualification of important to safety (ITS) SSCs.

The guidelines discuss a combination of seismic loads with other loads to be used for structural design, proportioning, and detailing of the structure to ensure ductile behavior, evaluation of foundation stability against sliding and overturning, story drift, building separation, and anchorage. Design and evaluation of slabs and other structural elements for heavy load drop effects, and tornado missile impact effects, are beyond the scope of this document.

These guidelines meet the seismic design requirements of NUREG-0800 (NRC 1987 [DIRS 138431]) for ITS SSCs and of *International Building Code 2000* (IBC 2000) (ICC 2000 [DIRS 173525]) for non-ITS SSCs. In addition, these guidelines also meet the U.S. Department of Energy (DOE) requirements of DOE-STD-1020-02 [DIRS 159258], which addresses the facility safety provisions of DOE O 420.1A [DIRS 159450].

The analysis methodology provides guidance on design of YMP facilities for vibrational ground motion and does not address approaches used for fault displacement from seismic events. Fault displacement hazards are addressed in DOE 2007, YMP/TR-003-NP, REV 5 [DIRS 181572] Section 5.0.

3. FACILITY LOCATION AND DESCRIPTION

3.1 FACILITY LOCATION

The YMP is located in Nye County, State of Nevada, approximately 100 miles northwest of the city of Las Vegas. A site plan is shown in Figure 3-1, including the major surface facilities. Emplacement drifts and other nearby facilities, which are part of the project, are not shown for clarity. Details of some of the major structures in Figure 3-1 are shown in Figure 3-2. A legend for the site plan is provided as Table 3-1. Figure 3-3 (BSC 2007 [DIRS 179641]) shows a hypothetical profile of the repository to illustrate the geometric relations between the surface and subsurface facilities. Points D and E in this Figure 3-3 (BSC 2007 [DIRS 179641]) indicate the location of the major surface facilities. Point B indicates the location of the emplacement drifts, which are about 1,000 ft below the ground surface.

3.2 FACILITY DESCRIPTION

The YMP consists of both surface and subsurface facilities as described in Sections 3.2.1 and 3.2.2.

3.2.1 Surface Facilities

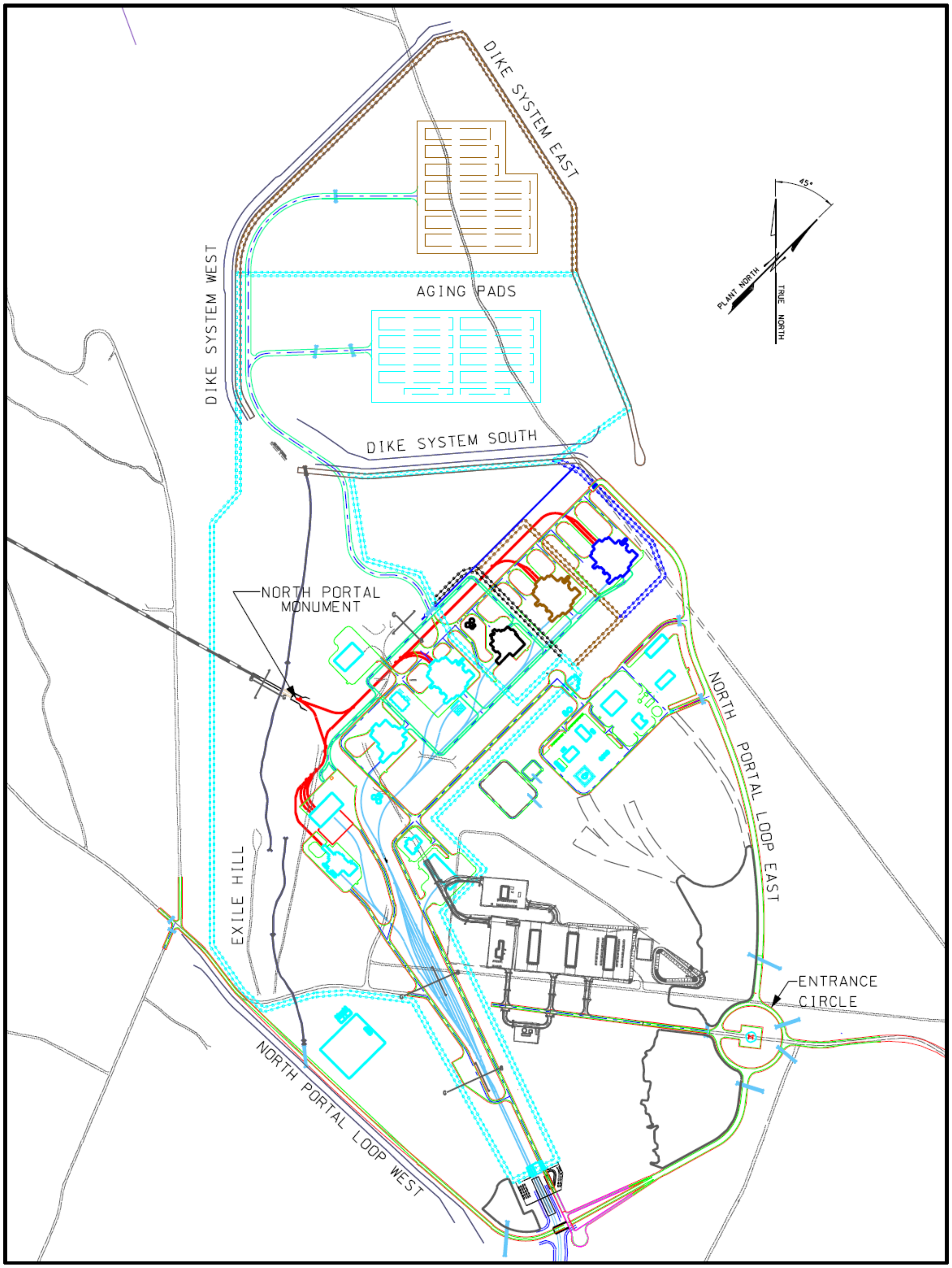
The surface handling facilities receive the high-level radioactive waste and spent nuclear fuel shipped in transportation casks to the site for transfer into waste packages. The waste packages are then prepared for emplacement in the underground drifts. Surface structures are shown on Figures 3-1 and 3-2 for which a legend is provided as Table 3-1. The following systems and components are associated with these structures.

Systems

Control systems
Electrical systems
Heating, ventilation, and air-conditioning (HVAC) system

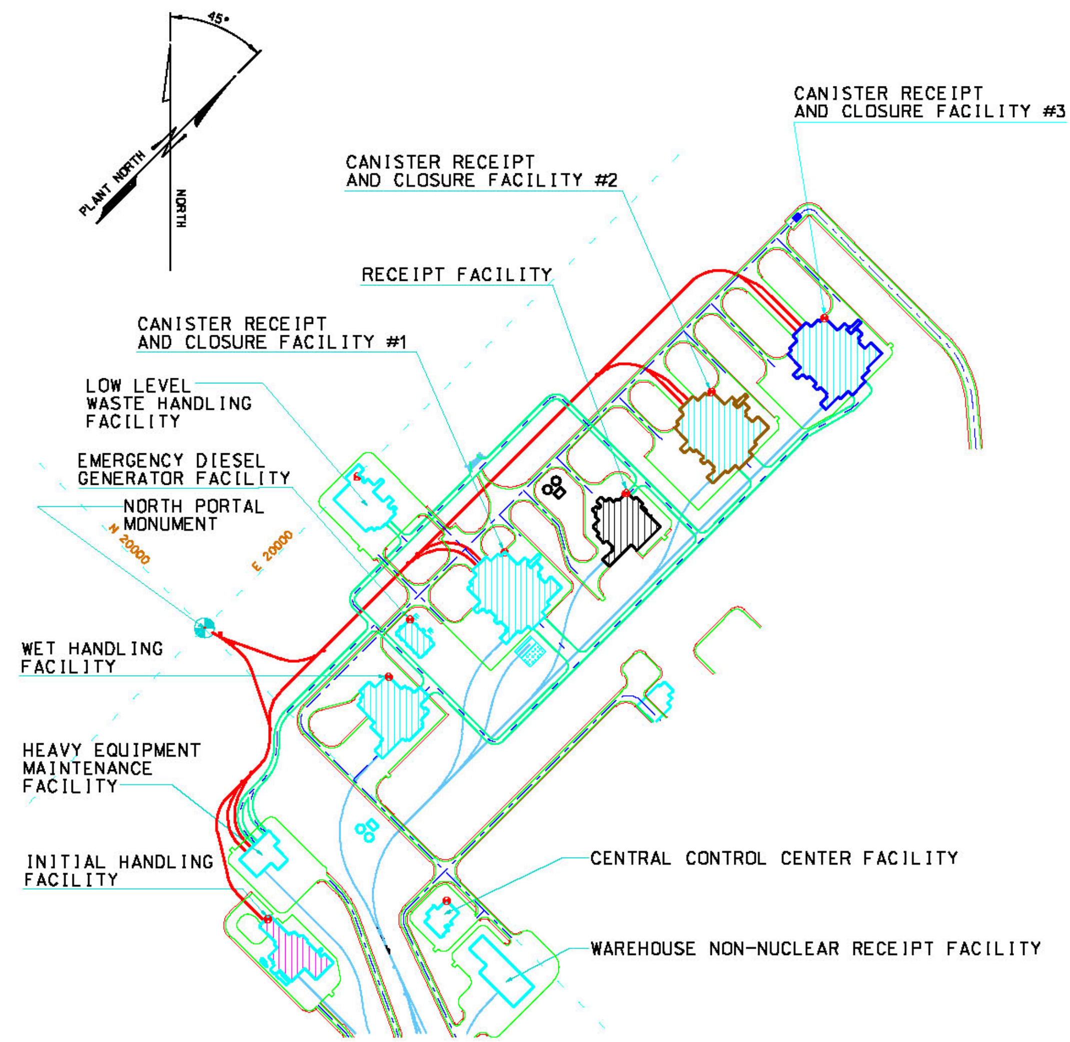
Components

Bridge Cranes
Prime Movers
Rail Cars
Shielding Doors
Transportation Casks
Transporters
Turntables
Waste Packages



Source:100-C00-MGR0-00501-000-00E [DIRS184014] & 170-C00-AP00-00101-000-00B[DIRS 184057]

Figure 3-1. Site Plan



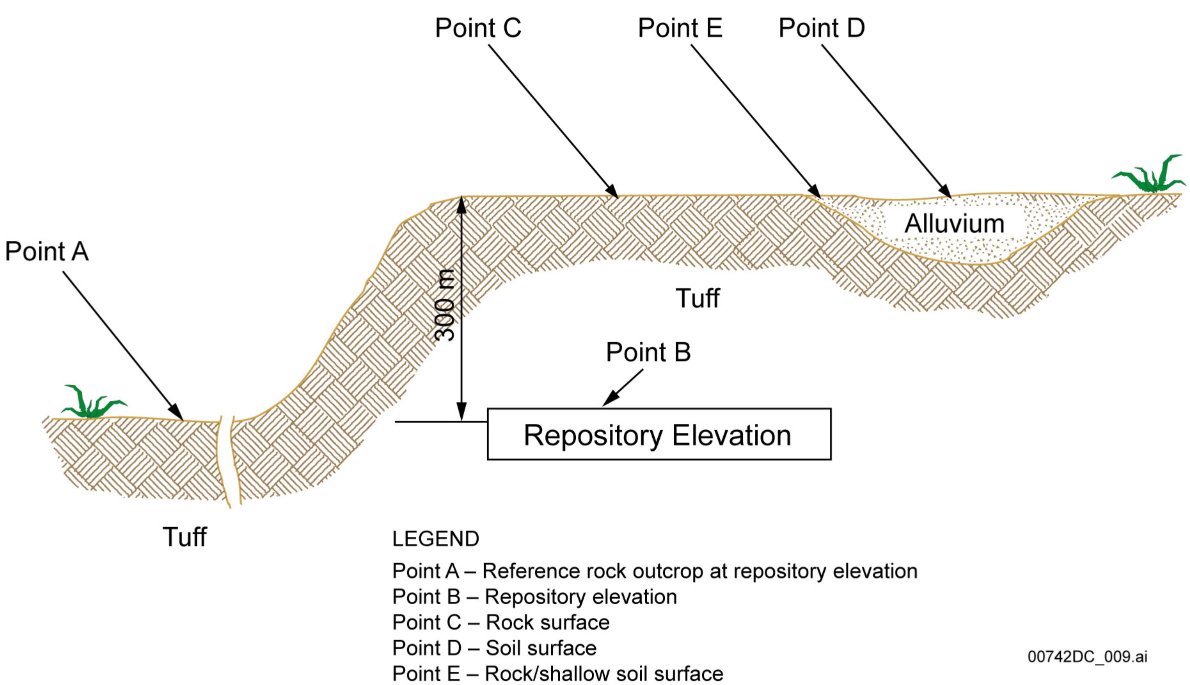
Source: 100-S00-MGR0-00101-000-00C[DIRS 184234]

Figure 3-2. Facilities Building Location Plan

Structures

Table 3-1. Legend for Figures 3-1 & 3-2

Abbreviation	Facility
—	Aging Pads
CRCF-1	Canister Receipt and Closure Facility-1
CRCF-2	Canister Receipt and Closure Facility-2
CRCF-3	Canister Receipt and Closure Facility-3
CCCF	Central Control Center Facility
EDGF	Emergency Diesel Generator Facility
HEMF	Heavy Equipment Maintenance Facility
IHF	Initial Handling Facility
LLWH	Low-level Waste Handling
RF	Receipt Facility
WNNRF	Warehouse Non-nuclear Receipt Facility
WHF	Wet Handling Facility



Source: BSC 2007 [DIRS 179641], Figure 6.1.3-1.

Figure 3-3. Seismic Design Input Locations

3.2.2 Subsurface Facility

The subsurface facility provides space for the emplacement, post-emplacement, and subsurface development activities. The subsurface facility includes the portals, ramps, access mains and rails, turnouts, emplacement drifts (including ground support, invert structures and ballast, waste package emplacement pallet, drip shield, and, if used, backfill), ventilation mains, shafts, shaft access drifts, alcoves, and performance confirmation areas. The facility includes the surface structures at the shafts, and closure seals and plugs. The facility isolates radioactive material from the environment and monitors the underground area (BSC 2007 [DIRS 182131], Section 8.1).

4. ASSUMPTIONS, DIRECT INPUTS, QUALITY ASSURANCE, SOFTWARE USAGE, AND PEER REVIEW

4.1 ASSUMPTIONS

This document presents methods to be used for the preclosure seismic analysis and design of SSCs at YMP. No analyses are performed in this report. As such, there are no assumptions or limitations to the methodologies hereinafter.

4.2 DIRECT INPUTS

4.2.1 Design Response Spectrum for Conventional Surface Facilities, Utilizing Updated Soils Data, Figure 6-8. BSC 2007 [DIRS 184022], Figure 3

4.2.2 Design Response Spectrum for Conventional Subsurface Facilities, Utilizing Updated Soils Data, Figure 6-9. BSC 2007 [DIRS 184192], Figure 3

4.3 QUALITY ASSURANCE

This report was prepared in accordance with PA-PRO-0313, *Technical Reports*. The methodology described in this report will be used for the design of facilities classified as ITS in the *Basis of Design for the Canister Based Design Concept* (BOD) (BSC 2007 [DIRS 182131]). The approved version is designated as QA:QA.

4.4 SOFTWARE USAGE

Excel® 2000 and Word® 2000, which are part of the Microsoft® Office® suite of programs were used in this report. Office® 2000, as used in this report, is classified as Level 2 software usage as defined in IT-PRO-0011, *Software Management*. Office® 2000 is listed on software report SW Tracking Number 607273, and in *Repository Project Management Automation Plan* (ORD 2007 [DIRS 182418]).

The software was executed on a personal computer system running Microsoft® Windows® 2000 operating system. The results can be confirmed by visual inspection and by performing hand calculations.

4.5 PEER REVIEW

An independent peer review panel (PA-PRO-0201, *Peer Review*) should review seismic analysis and design of SSCs designed for Design Basis Ground Motion–2 (DBGM-2). As a minimum, the review should include the following:

- Conformance to the Project Design Criteria
- Conformance to the Seismic Analysis and Design Approach (this document)
- Analysis and design philosophy
- Lateral force resisting systems
- Lateral load path

- Redundancy
- Structural and seismic models of the structures
- Seismic analysis results
- Design of the SSCs
- Constructability

The independent peer review need not provide a detailed check but should be an overview intended to address the following questions:

- Is the overall configuration of SSCs suitable for the goals to be achieved?

- Has the designer considered alternative configurations, alternate analysis, and design methods?

- Are the calculations appropriate for the type of facility being designed?

- Do the calculations address all design issues in adequate depth?

- Are the outputs consistent with the inputs?

- Are appropriate codes, standards, and regulations incorporated?

The questions and responses should be recorded in a peer review report. The peer review should be conducted by individuals with recognized technical expertise concerning the unique features of the analysis and design. The independent peer review panel members must not be involved in the original design.

5. SEISMIC CLASSIFICATION OF SSCs

5.1 CRITERIA FOR SEISMIC CLASSIFICATION OF SSCs

Seismic design basis ground motion (DBGM) hazard levels at the YMP are based on a "risk-informed" approach. According to *Preclosure Seismic Design and Performance Demonstration Methodology for a Geologic Repository at Yucca Mountain* (BSC 2007 [DIRS 181572]), risk-informed design means that facilities and structures with more severe failure consequences should have lower mean annual probabilities of failure. Consistent with this philosophy, YMP SSCs are considered in two groups: (1) Important to Safety (ITS) SSCs, and (2) non-ITS SSCs. Evaluations of the failure consequences will be carried out in the YMP preclosure safety analysis.

Consistent with *Preclosure Seismic Design and Performance Demonstration Methodology for a Geologic Repository at Yucca Mountain* (BSC 2007 [DIRS 181572]), seismic analysis and design are prepared for ITS SSCs, assigned DBGMs based on dose consequences of 10 CFR 63.111 [DIRS 176544], due to postulated Category 1 and Category 2 event sequences. For this purpose, three different levels of seismic ground motion design input events in terms of their return periods are identified in the PDC (BSC 2007 [DIRS 179641], Section 6.1.10), as follows:

- DBGM-1 (Design Basis Ground Motion–1) are events with a mean annual probability of exceedance of 1×10^{-3} (1,000-year return period), designated as Category 1 events.

- DBGM-2 (Design Basis Ground Motion–2) are events with a mean annual probability of exceedance of 5×10^{-4} (2,000-year return period), designated as Category 2 events.

- BDBGM (Beyond Design Basis Ground Motion) are event with a mean annual probability of exceedance of 10^{-4} (10,000-year return period).

Seismic designs of ITS SSCs assigned either DBGM-1 or DBGM-2 shall be prepared to meet the governing code-allowable acceptance criteria. In addition, as shown in Table 5-2, ITS SSCs designed for DBGM-2 will be evaluated at BDBGM to demonstrate the capacity of the ITS SSCs to perform their intended safety functions at BDBGM consistent with the methods outlined in *Preclosure Seismic Design and Performance Demonstration Methodology for a Geologic Repository at Yucca Mountain Topical Report* (BSC 2007 [DIRS 181572]). Appendix B describes the approach to be followed for BDBGM evaluation.

Definitions of the seismic event categories for ITS SSCs are given in Table 5-1. This table also shows the seismic design basis (DBGM and BDBGM) in terms of annual probability of exceedance. Non-ITS SSCs are discussed in Section 5.4.

Table 5-1. Seismic Design Bases for ITS SSCs

SSCs	Seismic Event	Earthquake Annual Exceedance Probability	Earthquake Return Period	Design Consideration
Designed to meet event sequences of Category 1[a]	DBGM-1	10^{-3}	1,000 years	SSCs are qualified to design codes and standards for 1,000-year return period earthquake loads.
Designed to meet event sequences of Category 2[a]	DBGM –2	5×10^{-4}	2,000 years	SSCs are designed to codes and standards for 2,000-year return period earthquake loads.
	BDBGM	10^{-4}	10,000 years	Structures are qualified to remain within acceptable inelastic limits under the 10,000-year return period earthquake.

[a] See 10 CFR 63.2 [DIRS 176544] for a definition of event sequences, and corresponding criteria.

5.2 ITS SSCs

The seismic design basis for ITS SSCs shall be in accordance with *Basis of Design for the TAD Canister-Based Repository Design Concept* (BSC 2007 [DIRS 182131]). Structures listed in Table 5-2 are important to safety (ITS) and are designed to meet Category 1 and Category 2 event sequences. Table 5-2 also identifies their seismic design and evaluation bases. The structures will be designed to meet the requirements of NUREG-0800 (NRC 1987 [DIRS 138431]) and appropriate design codes.

Table 5-2. Seismic Design Basis of ITS Structures

Location	SSCs	Seismic Basis[a] for Analysis/Design	Seismic Basis[a] for Evaluation
Surface	Aging Pads	DBGM-2	BDBGM
	Canister Receipt and Closure Facility	DBGM-2	BDBGM
	Emergency Diesel Generator Facility	DBGM-2[b]	N/A[b]
	Initial Handling Facility	DBGM-2	BDBGM
	Receipt Facility	DBGM-2	BDBGM
	Wet Handling Facility	DBGM-2	BDBGM

[a] Basis of design for the transport, aging, and disposal canister-based repository design concept (BSC 2007 [DIRS 182131]).

[b] Preliminary Preclosure Nuclear Safety Design Bases (BSC 2007 [DIRS 184154) identifies EDGF as an ITS structure but is silent on the seismic requirements. It will be designed for DBGM-2 similar to other ITS structures but not evaluated for BDBGM.

Overall Design Approach for ITS Structures

There are three considerations in the design of the ITS structures consistent with DOE 2007 [DIRS 181572] Sections 3 and 4. These are described below in the sequence to be followed in the design process:

1. Design the ITS structure for the seismic design basis indicated in Table 5-2. The design must be in conformance to project design criteria and the applicable codes.

2. For structures designed to DBGM-2, demonstrate that the HCLPF capacity is greater than the demand corresponding to BDBGM.

3. For structures designed to DBGM-2, develop a fragility curve. This fragility curve will be convolved with the seismic hazard curve to estimate the performance factor (probability of unacceptable behavior of the structure) which should be equal to 2xE-6 or less.

The design, calculation of the HCLPF value and the fragility curve will be performed by the CSA group. The convolution will be carried out by others.

5.3 SEISMIC INTERACTION OF NON-ITS SSCs WITH ITS SSCs

Some of the non-ITS SSCs, if they fail during a seismic event, may affect ITS SSCs. The non-ITS category SSCs in this group are classified as non-safety impacting safety (generally referred to as 2/1 consideration in the nuclear power industry) and are addressed in Section 11.3.

5.4 NON-ITS SSCs

Non-ITS structures will be designed in accordance with IBC 2000 (ICC 2000 [DIRS 173525]). Table 5-3 defines the various non-ITS SSC's and lists their seismic use groups (SUG) and importance factors (I). Table 5-4 lists the various surface and sub-surface non-ITS SSC's along with their SUG's and Seismic Design Catagories. The design spectra for non-ITS SSC's are provided in Section 6.4.

Table 5-3. Seismic Use Group and Importance Factors of SSCs Designed to IBC 2000

Seismic Use Group	Importance Factor, I	SSCs (Non-ITS) Designed to IBC
I	1.0	Non-ITS SSCs for standard occupancy
II	1.25	SSCs that represent substantial hazard to human life (Example: Heavy Equipment Maintenance facility)
III	1.5	SSCs that are essential and hazardous (containing toxic and hazardous materials)

Table 5-4. Classifications of Non-ITS SSCs Designed to IBC 2000

Location	SSCs	Seismic Use Group	Seismic Design Category (a)
Surface	Administration Facility including the EOC	IBC SUG III	D
	Central Control Facility	IBC SUG III	D
	Low level Waste Facility	IBC SUG III	D
	Switchgear Building	IBC SUG III	D
	Waste Package and Non-nuclear Receipt Facility	IBC SUG III	D
	Heavy equipment Maintenance Facility/Warehouse	IBC SUG II	D
	Change House	IBC SUG I	D
	Remaining Balance of Plant Facilities	IBC SUG I	D
	Switchyard	IBC SUG I	D
	Utility Building	IBC SUG I	D
	Warehouse and Non-nuclear Receipt Facility	IBC SUG I	D
Subsurface (b)	Concrete Inverts in Main Drifts	IBC SUG I	C
	Steel Bulkheads	IBC SUG I	C
	Transfer Dock	IBC SUG I	C
	Shaft Collars	IBC SUG I	C
	Muck Handling Facilities	IBC SUG I	C
	Steel Platforms	IBC SUG I	C
	Portal Structures	IBC SUG I	C
	Steel Inverts in Emplacement Drifts	IBC SUG I	C
	Miscellaneous Structures	IBC SUG I	C

NOTES: (a) Seismic Design Categories C and D refer to IBC 2000 (ICC 2000 [DIRS 173525]) Section 1616.3 definitions.

(b) Subsurface facilities will be designed in accordance with IBC 2000 (ICC 2000 [DIRS 173525]) as Seismic Design Category C and with the importance factor of 1.0. However, the design spectrum will be based on the site-specific spectrum at depth for 2,000-year return period earthquake as given in Figure 6-7.

IBC = International Building Code; SUG = Seismic Use Group; EOC = Emergency Operations Center.

6. DESIGN MOTION

6.1 GENERAL

Probabilistic Seismic Hazard Analyses for Fault Displacement and Vibratory Ground Motion at Yucca Mountain, Nevada (CRWMS M&O 1998 [DIRS 103731]) is a comprehensive report that was produced as a result of collaboration and review by a multitude of experts. The report provides a probabilistic seismic motion at a hypothetical rock outcrop at the YMP site for various return periods. This report was reviewed and concurred with by a peer review group consisting of experts in the areas of seismology and seismic design. Figure 3-3 shows the relations among the hypothetical point rock outcrop (Point A) and locations of the surface facilities (Points D and E) as well as the location of the repository drifts (Point B). Using the hypothetical rock outcrop motion, the motions at the ground surface level (Points D and E) for surface facilities and at a subsurface depth of 300 m below the surface (Point B) were developed for various return periods using the site-specific soils data (DTN: MO0706DSDR5E4A.001 [DIRS 181422], and DTN: MO0706DSDR1E4A.001 [DIRS 181421]).

Site-specific design spectra were developed by others for the ITS SSCs. In addition, compatible time histories were developed. The design spectra for DBGM-1, DBGM-2, and BDBGM seismic categories, and their compatible time histories, are given in Section 6.3.

Non-ITS SSCs will be designed in accordance with IBC 2000 (ICC 2000 [DIRS 173525]). The design spectrum for these non-ITS SSCs is based on site-specific seismicity considerations and is given in Section 6.4.

6.2 GEOTECHNICAL PARAMETERS

6.2.1 Static and Dynamic Soil properties

The soil bearing strata at the site consist of an alluvium layer with a varying thickness of a few feet to over 100 ft, depending on the location of the structures on North Portal Pad. The alluvium is underlain by a layer of tuff that extends to depths in excess of 1,000 ft. Both layers provide a very competent bearing stratum with adequate bearing capacity and very small compressibility. The soil properties at the site were developed based on the results of field and laboratory investigation, including the results from various field geophysical testing. Geotechnical testing was also used to develop the foundation design parameters. A summary of the geotechnical investigation is presented in *Supplemental Soils Report* (BSC 2007 [DIRS 182582]). According to Section 6.1.4.4 of that report, the water table is below the emplacement drift levels, and thus needs not be considered in design.

Table 6-1 lists the estimated range of soil static and dynamic properties to be used in static analysis for preliminary design purposes for both long- and short-term loads. Table 6-2 lists the friction coefficient and the active, at-rest, and passive pressure coefficients (dynamic incremental pressures are addressed in Section 6.2.2).

Table 6-1. Static and Dynamic Soil Parameters

Material	Case	Elastic Modulus E (ksi)	Coefficient of Subgrade Reaction (kcf)
Alluvium	Static	30 to 75	155 to 520
	Dynamic[a]	100 to 500	310 to 1,040
Engineered Fill	Static	14 to 28	75 to 250
	Dynamic[a]	30 to 170	150 to 500

Source: *Supplemental Soils Report* BSC 2007 [DIRS 182582], Tables 2-1 and 2-2.

[a] Short term or low strain values

Table 6-2. Friction and Lateral Soil Pressure Coefficients

Material	Moist Density (pcf)	Friction Angle (Φ) (degrees)	Friction Coefficie nt $\delta = \tan \Phi$.	Cohesion c	Soil Pressures		
					Active Pressure K_a	At-Rest Pressure K_o	Passive Pressure K_p
Alluvium	114 to 117	39	0.81	0	0.23	0.37	4.4
Engineered Fill	127	42	0.90	0	0.20	0.33	5.0

Source: *Supplemental Soils Report* BSC 2007 [DIRS 182582], Tables 2-1 and 2-2.

For static structural analysis and basemat design, the soil properties are typically characterized by "soil springs" that are determined based on: foundation size and depth, soil properties and layering geometry, and loading conditions with and without temporary loading such as earthquake loading. Therefore, equivalent soil springs used in design shall be determined based on the specific foundation geometry and design loading for the structures(s) of concern (see Appendix C for further discussion on this subject).

The dynamic soil properties for soil structure interaction (SSI) analyses are provided in DTN: MO0706SCSPS5E4.002 [DIRS 181616] for 5×10^{-4} annual exceedance probability, and in DTN: MO0706SCSPS1E4.002 [DIRS 181618] for 10^{-4} annual exceedance probability. These properties include the effect of soil nonlinearity by developing the strain-compatible soil properties obtained from free-field analysis using the design motions. In addition, the strain-compatible damping values (DTNs: MO0706SCSPS5E4.002 [DIRS 181616] and MO0706SCSPS1E4.002 [DIRS 181618]) were developed for use in a system for analysis of soil-structure interaction (SASSI). (See also Appendix C.)

6.2.2 Lateral Dynamic Soil Pressures

When an SSI analysis is performed, the dynamic lateral soil pressures will be calculated in the SSI analysis and will be applied as a static load in the stress analysis of the structure. Dynamic soil pressure will include the effect of structure-to-structure interaction, if warranted.

When SSI analysis is not performed, lateral dynamic soil pressures will be calculated following the procedure given in Section 3.5.3 of the American Society of Civil Engineers (ASCE) code ASCE 4-98 [DIRS 159618], together with the recommendation of the *Supplemental Soils Report* (BSC 2007 [DIRS 182582]).

6.2.3 Foundation Settlement and Bearing Capacity

Due to the relatively dense granular nature of the alluvium at the site, the bearing capacity, particularly for the large foundation mats, is 50 ksf or more *Supplemental Soils Report* (Figure B6-2) (BSC 2007 [DIRS 182582]). This bearing capacity exceeds the anticipated foundation pressure imposed from the structures. Thus, the permissible foundation pressure is controlled by the amount of foundation settlement for which the mat and the structure can be reasonably designed. Since nearly all of the settlement is immediate elastic settlement, using soil springs at the base of the mat best represents the effect of foundation settlement on the mat and structure.

As discussed in Section 6.2.1, the soil springs attached to the base of the mat shall be determined based on the specific foundation geometry and design loading for the structure(s) of concern and will be used in conjunction with the detail stress analysis model of the structures to design the structural members. Foundation springs will be determined for both long-term (i.e., gravity) and short-term (i.e. seismic) loads as discussed in Appendix C.

6.3 DESIGN BASIS GROUND MOTION (DBGM) FOR ITS STRUCTURES

The surface facilities of the YMP are located on the North Portal site. To meet the performance objectives of 10 CFR Part 63 [DIRS 176544], surface facilities that are ITS must be designed for site-specific seismic ground motions. The facility location is schematically identified by Point D on Figure 3-3. As stated in Section 6.1, the seismic motions at Point A, which is a hypothetical rock outcrop, have been defined by a panel of seismic experts. The seismic design inputs at Point D are derived from seismic motions at Point A. Similarly, the input motion for subsurface SSCs is calculated FOR Point B.

The design response spectra (DRS) and the compatible time histories for ITS SSCs are given in Sections 6.3.1 through 6.3.4. The DRS and associated time histories described in these sections are obtained from DTNs: MO0706DSDR5E4A.001 [DIRS 181422], MO0706DSDR1E3A.000 [DIRS 181423], MO0706DSDR1E4A.001 [DIRS 181421], MO0706TH1E3APE.000 [DIRS 182460], MO0706TH5E4APE.001 [DIRS 181961], and MO0706TH1E4APE.001 [DIRS 181960]

Ground motions used during the initial design of the ITS surface facilities are given in DTNs MO0411SDSTMHIS.006 [DIRS 172426], MO0411SDSDE103.003 [DIRS 172425], and MO0411WHBDE104.003 [DIRS 172427].

6.3.1 DRS for Surface Facilities

Figures 6-1 to 6-6 provide the surface DRS for horizontal and vertical directions, for multiple damping, for 1,000-year, 2,000-year, and 10,000-year return period earthquakes. Both spectra shapes and digitized spectra are provided (DTNs: MO0706DSDR5E4A.001 [DIRS 181422], MO0706DSDR1E3A.000 [DIRS 181423], and MO0706DSDR1E4A.001 [DIRS 181421]]). Data for Figures 6-1 through 6-6 are provided in Tables 6-3 through 6-8 respectively.

6.3.2 DRS for Subsurface Facilities

Figure 6-7 provides the subsurface DRS for horizontal and vertical directions at the repository elevation (Point B), with 5% damping value for the 2,000-year return period earthquakes. Both spectral shapes and digitized spectra are provided. (DTN: MO0707DSRB5E4A.000 [DIRS 183130]).

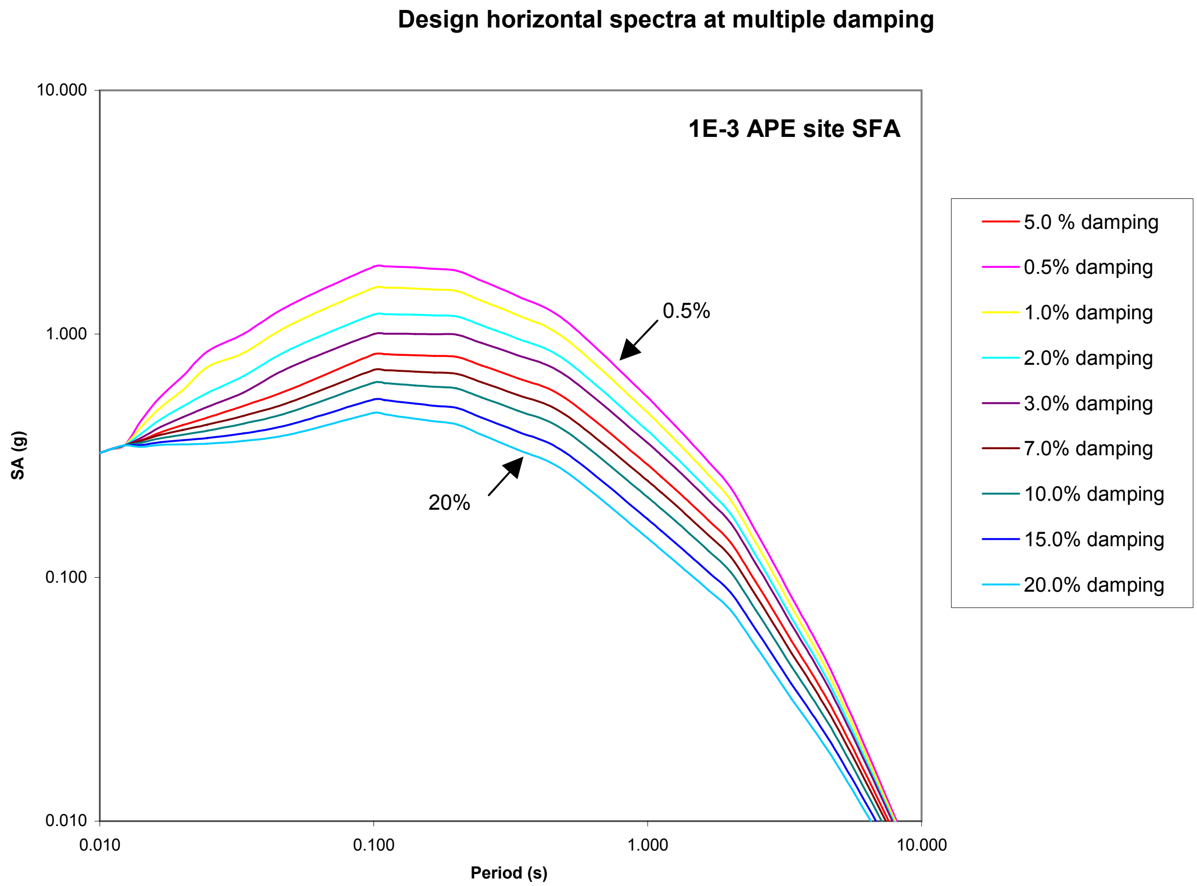
Ground motions used during the initial design of the subsurface facilities are given in DTN MO0407SDARS104.001 [DIRS 170683].

6.3.3 Design Time Histories for Surface Facilities

Time histories are used in soil-structure interaction analysis to determine the seismic responses of the structures in terms of seismic load, in-structure response spectra and dynamic soil pressure. Horizontal and vertical time-history motions compatible with the 1,000-year, 2,000-year, and 10,000-year return period earthquakes, showing the acceleration, velocity, and displacement time history for each earthquake component are available in MO0706TH1E3APE.000 [DIRS 182460], MO0706TH5E4APE.001 [DIRS 181961], and MO0706TH1E4APE.001 [DIRS 181960]

6.3.4 Design Time Histories for Subsurface Facilities

For subsurface SSCs, horizontal and vertical time-history motions compatible with the 2,000-year return period earthquake are given in the PDC (BSC 2007 [DIRS 179641], Table 6.1-1).



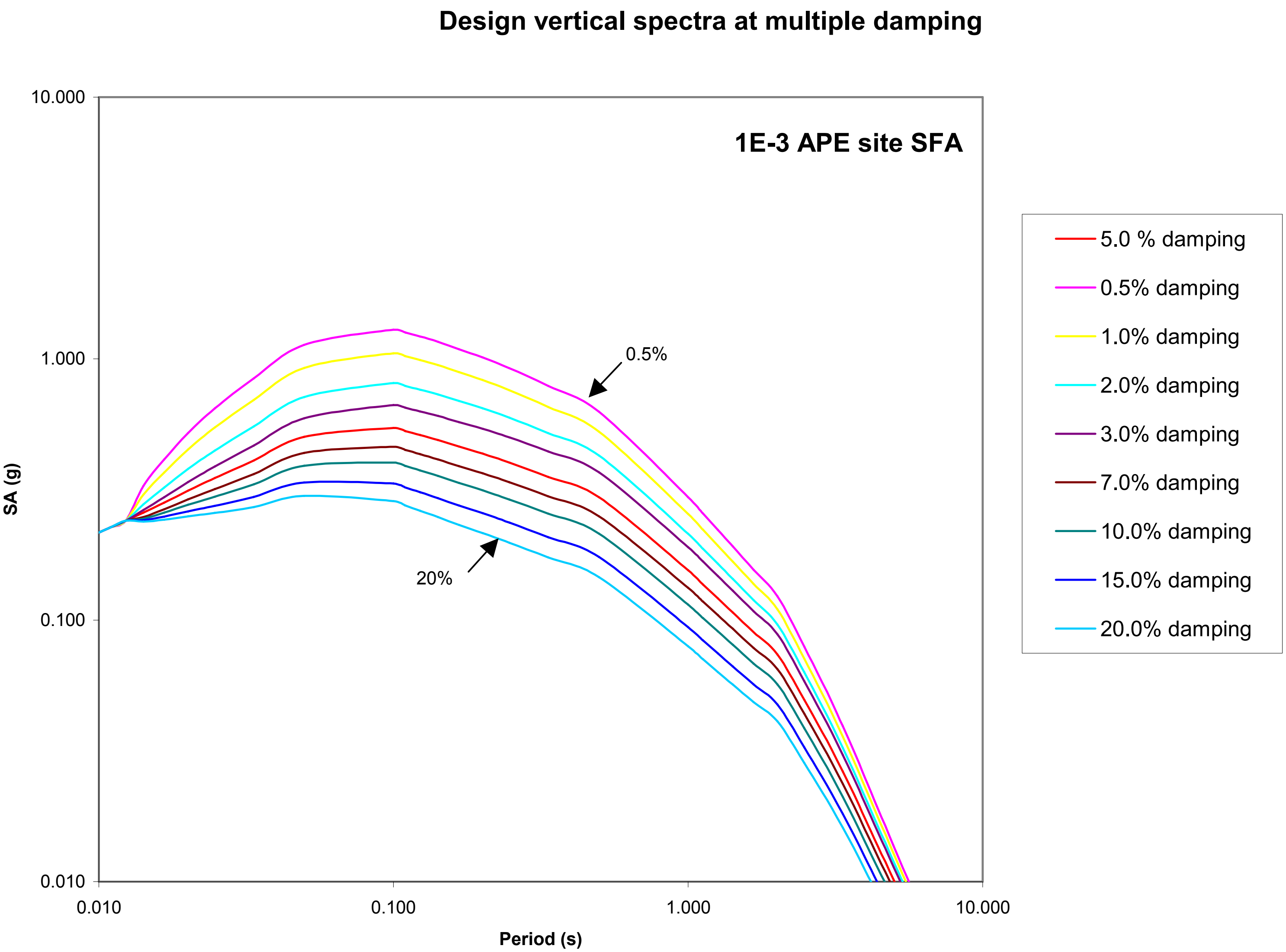
Source: DTN: MO0706DSDR1E3A.000 [DIRS 181423], *SPCDAMP10-3_tdms.xls*, 'Horizontal spectra'.

Figure 6-1. Horizontal Spectra for the Design of Surface Facilities, 10^{-3}, DBGM-1

Table 6-3. Horizontal Spectra (Digitized) for the Design of Surface Facilities, 10^{-3}, DBGM-1

Period (s)	Freq (Hz)	SA (5.0%)	SA (0.5%)	SA (1.0%)	SA (2.0%)	SA (3.0%)	SA (7.0%)	SA (10.0%)	SA (15.0%)	SA (20.0%)
0.010	100.000	0.3250	0.3250	0.3250	0.3250	0.3250	0.3250	0.3250	0.3250	0.3250
0.011	91.116	0.3364	0.3364	0.3364	0.3364	0.3364	0.3364	0.3364	0.3364	0.3364
0.012	81.113	0.3489	0.3489	0.3489	0.3489	0.3489	0.3489	0.3489	0.3489	0.3489
0.014	70.548	0.3659	0.4413	0.4156	0.3899	0.3749	0.3647	0.3577	0.3496	0.3439
0.017	59.948	0.3946	0.5518	0.4986	0.4455	0.4144	0.3851	0.3732	0.3597	0.3501
0.020	49.770	0.4221	0.6709	0.5871	0.5034	0.4544	0.4030	0.3856	0.3658	0.3518
0.025	40.370	0.4541	0.8484	0.7311	0.5728	0.5023	0.4242	0.4005	0.3735	0.3544
0.034	29.837	0.5069	1.0019	0.8370	0.6722	0.5757	0.4614	0.4281	0.3902	0.3633
0.050	20.092	0.5935	1.3190	1.0920	0.8650	0.7323	0.5265	0.4790	0.4251	0.3868
0.100	10.000	0.8246	1.8906	1.5456	1.2006	0.9988	0.7118	0.6309	0.5389	0.4736
0.110	9.112	0.8245	1.8934	1.5486	1.2037	1.0020	0.7102	0.6277	0.5340	0.4675
0.123	8.111	0.8210	1.8861	1.5439	1.2017	1.0015	0.7054	0.6215	0.5260	0.4583
0.142	7.055	0.8169	1.8732	1.5355	1.1979	1.0004	0.7002	0.6146	0.5172	0.4482
0.167	5.995	0.8114	1.8514	1.5211	1.1907	0.9975	0.6939	0.6067	0.5075	0.4372
0.201	4.977	0.8039	1.8178	1.4983	1.1788	0.9919	0.6862	0.5975	0.4967	0.4252
0.248	4.037	0.7414	1.6533	1.3687	1.0840	0.9175	0.6321	0.5482	0.4528	0.3851
0.335	2.984	0.6581	1.4298	1.1925	0.9553	0.8165	0.5608	0.4840	0.3967	0.3348
0.498	2.009	0.5477	1.1398	0.9618	0.7837	0.6796	0.4673	0.4014	0.3266	0.2735
1.000	1.000	0.2904	0.5491	0.4749	0.4007	0.3572	0.2496	0.2141	0.1736	0.1450
1.123	0.890	0.2585	0.4801	0.4170	0.3540	0.3172	0.2227	0.1911	0.1552	0.1297
1.262	0.793	0.2299	0.4190	0.3657	0.3124	0.2812	0.1985	0.1705	0.1387	0.1161
1.417	0.706	0.2041	0.3649	0.3200	0.2750	0.2488	0.1767	0.1520	0.1239	0.1039
1.668	0.600	0.1714	0.2979	0.2630	0.2281	0.2077	0.1490	0.1285	0.1051	0.0886
2.009	0.498	0.1389	0.2334	0.2077	0.1821	0.1670	0.1214	0.1051	0.0865	0.0734
2.477	0.404	0.0960	0.1551	0.1392	0.1233	0.1141	0.0845	0.0736	0.0612	0.0523
3.351	0.298	0.0549	0.0831	0.0758	0.0684	0.0642	0.0490	0.0431	0.0365	0.0318
4.978	0.201	0.0259	0.0356	0.0332	0.0307	0.0293	0.0236	0.0213	0.0185	0.0166
10.000	0.100	0.0053	0.0057	0.0057	0.0056	0.0055	0.0051	0.0049	0.0046	0.0044

Source: DTN: MO0706DSDR1E3A.000 [DIRS 181423].



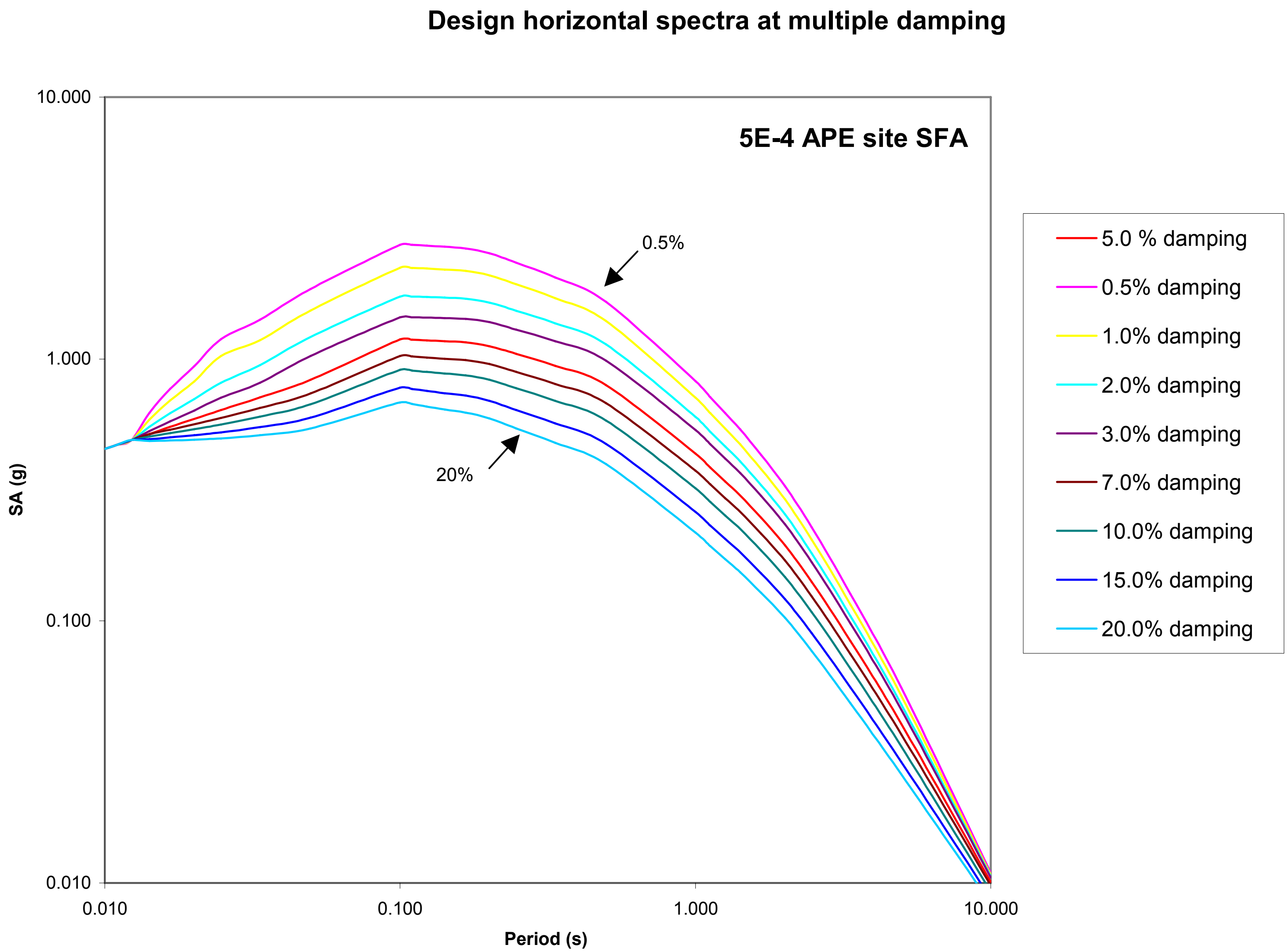
Source: DTN: MO0706DSDR1E3A.000 [DIRS 181423], *SPCDAMP10-3_tdms.xls*, 'Vertical spectra'.

Figure 6-2. Vertical Spectra for the Design of Surface Facilities, 10^{-3}, DBGM-1

Table 6-4. Vertical Spectra (Digitized) for the Design of Surface Facilities, 10^{-3}, DBGM-1

Period (s)	Freq (Hz)	SA (5.0%)	SA (0.5%)	SA (1.0%)	SA (2.0%)	SA (3.0%)	SA (7.0%)	SA (10.0%)	SA (15.0%)	SA (20.0%)
0.010	100.000	0.2164	0.2164	0.2164	0.2164	0.2164	0.2164	0.2164	0.2164	0.2164
0.011	91.116	0.2264	0.2264	0.2264	0.2264	0.2264	0.2264	0.2264	0.2264	0.2264
0.012	81.113	0.2400	0.2400	0.2400	0.2400	0.2400	0.2400	0.2400	0.2400	0.2400
0.014	70.548	0.2576	0.3261	0.3020	0.2778	0.2637	0.2477	0.2445	0.2409	0.2383
0.017	59.948	0.2815	0.4123	0.3672	0.3221	0.2957	0.2658	0.2579	0.2489	0.2425
0.020	49.770	0.3129	0.5201	0.4494	0.3788	0.3375	0.2898	0.2761	0.2605	0.2495
0.025	40.370	0.3494	0.6462	0.5458	0.4454	0.3867	0.3176	0.2970	0.2737	0.2571
0.034	29.837	0.4077	0.8404	0.6954	0.5504	0.4655	0.3622	0.3311	0.2958	0.2707
0.050	20.092	0.5021	1.1287	0.9211	0.7135	0.5920	0.4360	0.3894	0.3365	0.2989
0.100	10.000	0.5431	1.2922	1.0494	0.8065	0.6645	0.4606	0.4010	0.3334	0.2854
0.110	9.112	0.5283	1.2583	1.0224	0.7865	0.6485	0.4472	0.3885	0.3219	0.2746
0.123	8.111	0.5105	1.2153	0.9885	0.7617	0.6290	0.4312	0.3738	0.3084	0.2621
0.142	7.055	0.4888	1.1603	0.9453	0.7303	0.6046	0.4122	0.3564	0.2930	0.2480
0.167	5.995	0.4607	1.0866	0.8875	0.6884	0.5719	0.3879	0.3347	0.2742	0.2313
0.201	4.977	0.4335	1.0115	0.8290	0.6465	0.5398	0.3647	0.3140	0.2564	0.2156
0.248	4.037	0.4005	0.9196	0.7572	0.5948	0.4998	0.3369	0.2897	0.2360	0.1979
0.335	2.984	0.3531	0.7869	0.6531	0.5193	0.4410	0.2974	0.2554	0.2077	0.1738
0.498	2.009	0.2951	0.6262	0.5262	0.4262	0.3677	0.2496	0.2145	0.1745	0.1461
1.000	1.000	0.1550	0.2952	0.2546	0.2140	0.1903	0.1329	0.1148	0.0941	0.0795
1.123	0.890	0.1371	0.2558	0.2217	0.1876	0.1677	0.1180	0.1020	0.0839	0.0710
1.262	0.793	0.1211	0.2213	0.1927	0.1642	0.1475	0.1045	0.0906	0.0747	0.0634
1.417	0.706	0.1070	0.1913	0.1675	0.1437	0.1297	0.0927	0.0805	0.0666	0.0567
1.668	0.600	0.0900	0.1560	0.1375	0.1191	0.1084	0.0784	0.0683	0.0568	0.0486
2.009	0.498	0.0740	0.1235	0.1099	0.0963	0.0883	0.0649	0.0568	0.0476	0.0410
2.477	0.404	0.0496	0.0792	0.0712	0.0632	0.0585	0.0439	0.0387	0.0327	0.0285
3.351	0.298	0.0264	0.0394	0.0360	0.0325	0.0305	0.0237	0.0211	0.0182	0.0161
4.978	0.201	0.0102	0.0138	0.0129	0.0120	0.0114	0.0094	0.0085	0.0076	0.0069
10.000	0.100	0.0020	0.0021	0.0021	0.0021	0.0021	0.0019	0.0019	0.0018	0.0017

Source: DTN: MO0706DSDR1E3A.000 [DIRS 181423].



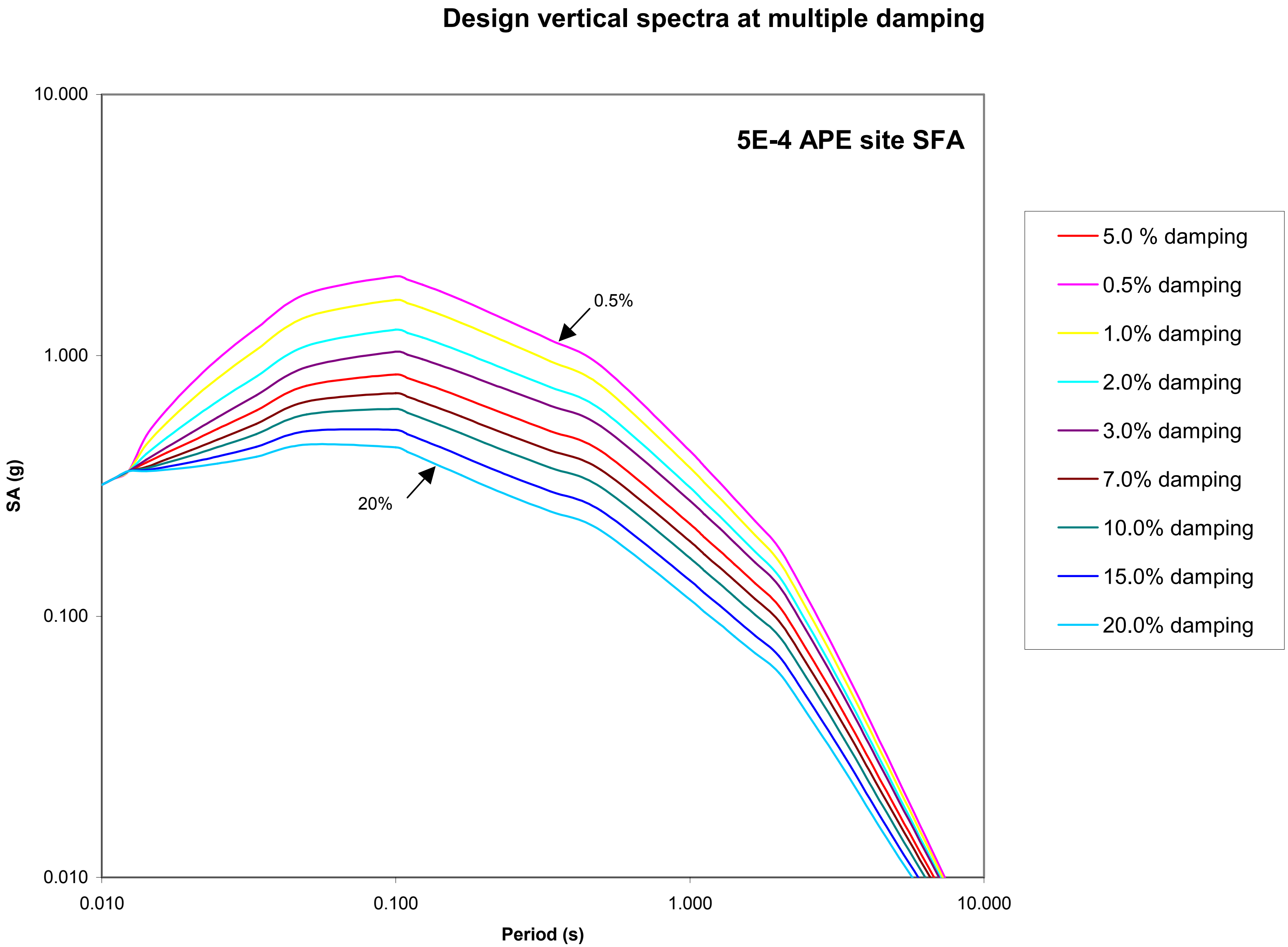
Source: DTN: MO0706DSDR5E4A.001 [DIRS 181422], *spcdamp_5E-4_TDMS.xls*, 'Horizontal spectra'.

Figure 6-3. Horizontal Spectra for the Design of Surface Facilities, 5×10^{-4}, DBGM-2

Table 6-5. Horizontal Spectra (Digitized) for the Design of Surface Facilities, 5×10^{-4}, DBGM-2

Period (s)	**Freq (Hz)**	**SA (5.0%)**	**SA (0.5%)**	**SA (1.0%)**	**SA (2.0%)**	**SA (3.0%)**	**SA (7.0%)**	**SA (10.0%)**	**SA (15.0%)**	**SA (20.0%)**
0.010	100.000	0.4537	0.4537	0.4537	0.4537	0.4537	0.4537	0.4537	0.4537	0.4537
0.011	91.116	0.4700	0.4700	0.4700	0.4700	0.4700	0.4700	0.4700	0.4700	0.4700
0.012	81.113	0.4911	0.4911	0.4911	0.4911	0.4911	0.4911	0.4911	0.4911	0.4911
0.014	70.548	0.5177	0.6243	0.5880	0.5517	0.5304	0.5161	0.5061	0.4947	0.4866
0.017	59.948	0.5506	0.7699	0.6957	0.6216	0.5782	0.5373	0.5207	0.5019	0.4885
0.020	49.770	0.5905	0.9385	0.8214	0.7042	0.6357	0.5638	0.5394	0.5118	0.4921
0.025	40.370	0.6380	1.1920	1.0272	0.8110	0.7112	0.5960	0.5627	0.5248	0.4979
0.034	29.837	0.7141	1.4115	1.1792	0.9469	0.8110	0.6500	0.6031	0.5497	0.5118
0.050	20.092	0.8330	1.8513	1.5327	1.2141	1.0277	0.7390	0.6723	0.5966	0.5428
0.100	10.000	1.1894	2.7270	2.2294	1.7318	1.4407	1.0267	0.9100	0.7773	0.6832
0.110	9.112	1.1863	2.7243	2.2281	1.7319	1.4417	1.0218	0.9032	0.7683	0.6726
0.123	8.111	1.1784	2.7071	2.2160	1.7248	1.4375	1.0125	0.8920	0.7550	0.6578
0.142	7.055	1.1690	2.6805	2.1974	1.7142	1.4316	1.0019	0.8794	0.7402	0.6414
0.167	5.995	1.1581	2.6424	2.1710	1.6995	1.4238	0.9904	0.8659	0.7244	0.6240
0.201	4.977	1.1201	2.5328	2.0876	1.6425	1.3820	0.9562	0.8326	0.6921	0.5924
0.248	4.037	1.0458	2.3322	1.9307	1.5291	1.2943	0.8916	0.7733	0.6387	0.5432
0.335	2.984	0.9418	2.0462	1.7066	1.3671	1.1685	0.8025	0.6926	0.5677	0.4791
0.498	2.009	0.7945	1.6534	1.3951	1.1369	0.9859	0.6778	0.5823	0.4738	0.3968
1.000	1.000	0.4357	0.8239	0.7125	0.6011	0.5360	0.3746	0.3212	0.2605	0.2175
1.123	0.890	0.3854	0.7157	0.6218	0.5278	0.4729	0.3320	0.2849	0.2314	0.1934
1.262	0.793	0.3407	0.6210	0.5420	0.4629	0.4167	0.2942	0.2527	0.2055	0.1720
1.417	0.706	0.3012	0.5385	0.4722	0.4059	0.3671	0.2607	0.2243	0.1828	0.1534
1.668	0.600	0.2477	0.4306	0.3801	0.3297	0.3002	0.2153	0.1856	0.1519	0.1280
2.009	0.498	0.1947	0.3272	0.2912	0.2552	0.2341	0.1701	0.1473	0.1213	0.1028
2.477	0.404	0.1409	0.2276	0.2043	0.1810	0.1674	0.1240	0.1080	0.0898	0.0768
3.351	0.298	0.0837	0.1267	0.1155	0.1044	0.0978	0.0747	0.0658	0.0556	0.0485
4.978	0.201	0.0405	0.0556	0.0518	0.0481	0.0459	0.0370	0.0332	0.0290	0.0260
10.000	0.100	0.0101	0.0109	0.0108	0.0106	0.0105	0.0098	0.0093	0.0087	0.0083

Source: DTN: MO0706DSDR5E4A.001 [DIRS 181422].



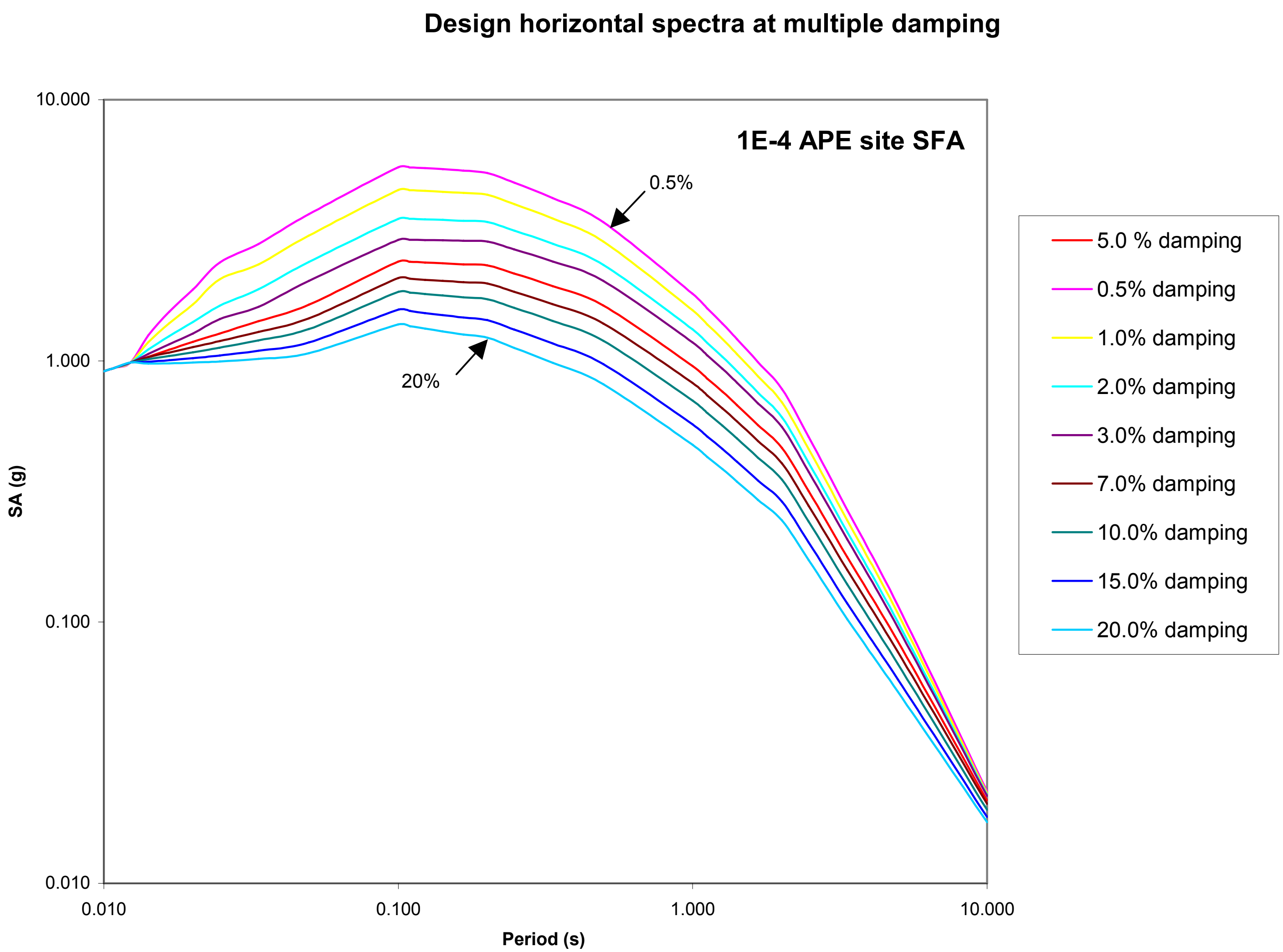
Source: DTN: MO0706DSDR5E4A.001 [DIRS 181422], *spcdamp_5E-4_TDMS.xls*, ‘Vertical spectra’.

Figure 6-4. Vertical Spectra for the Design of Surface Facilities, 5×10^{-4}, DBGM-2

Table 6-6. Vertical Spectra (Digitized) for the Design of Surface Facilities, 5×10^{-4}, DBGM-2

Period (s)	Freq (Hz)	SA (5.0%)	SA (0.5%)	SA (1.0%)	SA (2.0%)	SA (3.0%)	SA (7.0%)	SA (10.0%)	SA (15.0%)	SA (20.0%)
0.010	100.000	0.3194	0.3194	0.3194	0.3194	0.3194	0.3194	0.3194	0.3194	0.3194
0.011	91.116	0.3369	0.3369	0.3369	0.3369	0.3369	0.3369	0.3369	0.3369	0.3369
0.012	81.113	0.3600	0.3600	0.3600	0.3600	0.3600	0.3600	0.3600	0.3600	0.3600
0.014	70.548	0.3892	0.4927	0.4563	0.4198	0.3985	0.3742	0.3694	0.3639	0.3600
0.017	59.948	0.4241	0.6211	0.5532	0.4853	0.4455	0.4004	0.3885	0.3749	0.3653
0.020	49.770	0.4679	0.7777	0.6720	0.5664	0.5046	0.4334	0.4129	0.3896	0.3731
0.025	40.370	0.5235	0.9682	0.8178	0.6674	0.5794	0.4758	0.4450	0.4100	0.3852
0.034	29.837	0.6161	1.2700	1.0509	0.8317	0.7035	0.5473	0.5004	0.4470	0.4091
0.050	20.092	0.7660	1.7219	1.4052	1.0884	0.9032	0.6652	0.5941	0.5133	0.4560
0.100	10.000	0.8454	2.0115	1.6335	1.2555	1.0343	0.7169	0.6243	0.5189	0.4442
0.110	9.112	0.8195	1.9518	1.5859	1.2200	1.0059	0.6937	0.6027	0.4993	0.4259
0.123	8.111	0.7848	1.8683	1.5196	1.1709	0.9669	0.6629	0.5746	0.4742	0.4030
0.142	7.055	0.7425	1.7625	1.4359	1.1094	0.9183	0.6261	0.5414	0.4451	0.3768
0.167	5.995	0.6927	1.6338	1.3344	1.0350	0.8599	0.5833	0.5032	0.4123	0.3477
0.201	4.977	0.6385	1.4899	1.2211	0.9522	0.7950	0.5371	0.4625	0.3777	0.3175
0.248	4.037	0.5830	1.3386	1.1022	0.8658	0.7275	0.4904	0.4217	0.3435	0.2881
0.335	2.984	0.5134	1.1441	0.9496	0.7550	0.6412	0.4323	0.3713	0.3019	0.2527
0.498	2.009	0.4304	0.9133	0.7674	0.6216	0.5362	0.3641	0.3128	0.2545	0.2131
1.000	1.000	0.2261	0.4306	0.3714	0.3122	0.2775	0.1939	0.1674	0.1373	0.1159
1.123	0.890	0.2006	0.3743	0.3244	0.2745	0.2453	0.1726	0.1492	0.1227	0.1039
1.262	0.793	0.1787	0.3265	0.2844	0.2423	0.2177	0.1543	0.1336	0.1102	0.0935
1.417	0.706	0.1583	0.2831	0.2478	0.2125	0.1919	0.1371	0.1190	0.0985	0.0839
1.668	0.600	0.1338	0.2319	0.2045	0.1771	0.1611	0.1166	0.1015	0.0844	0.0723
2.009	0.498	0.1095	0.1828	0.1626	0.1425	0.1307	0.0961	0.0841	0.0704	0.0607
2.477	0.404	0.0755	0.1206	0.1084	0.0962	0.0891	0.0668	0.0589	0.0498	0.0433
3.351	0.298	0.0423	0.0631	0.0576	0.0521	0.0489	0.0380	0.0339	0.0291	0.0258
4.978	0.201	0.0187	0.0252	0.0236	0.0219	0.0210	0.0172	0.0157	0.0139	0.0126
10.000	0.100	0.0045	0.0048	0.0048	0.0047	0.0047	0.0044	0.0042	0.0040	0.0039

Source: DTN: MO0706DSDR5E4A.001 [DIRS 181422].



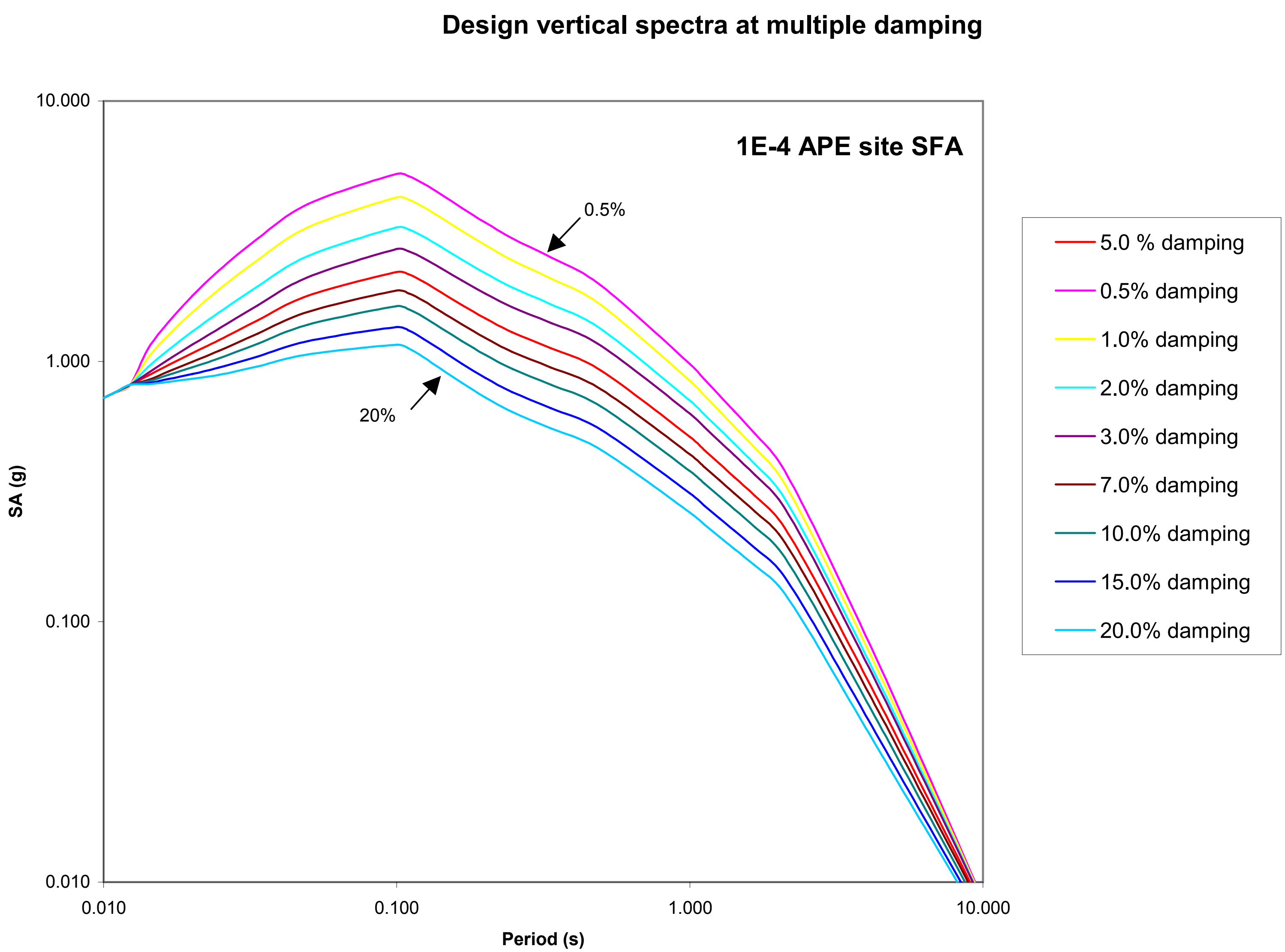
Source: DTN: MO0706DSDR1E4A.001 [DIRS 181421]], *Spcdamp10-4_TDMS.xls*, 'Horizontal spectra'.

Figure 6-5. Horizontal Spectra for the Design of Surface Structures, 10^{-4}, BDBGM

Table 6-7. Horizontal Spectra (Digitized) for the Design of Surface Structures, 10^{-4}, BDBGM

Period (s)	Freq (Hz)	SA (5.0%)	SA (0.5%)	SA (1.0%)	SA (2.0%)	SA (3.0%)	SA (7.0%)	SA (10.0%)	SA (15.0%)	SA (20.0%)
0.010	100.000	0.9138	0.9138	0.9138	0.9138	0.9138	0.9138	0.9138	0.9138	0.9138
0.011	91.116	0.9441	0.9441	0.9441	0.9441	0.9441	0.9441	0.9441	0.9441	0.9441
0.012	81.113	0.9853	0.9853	0.9853	0.9853	0.9853	0.9853	0.9853	0.9853	0.9853
0.014	70.548	1.0378	1.2515	1.1787	1.1059	1.0634	1.0345	1.0144	0.9916	0.9755
0.017	59.948	1.1029	1.5422	1.3936	1.2451	1.1582	1.0763	1.0430	1.0053	0.9784
0.020	49.770	1.1817	1.8782	1.6437	1.4093	1.2722	1.1282	1.0795	1.0241	0.9848
0.025	40.370	1.2742	2.3806	2.0515	1.6255	1.4454	1.1904	1.1238	1.0481	0.9944
0.034	29.837	1.4196	2.8060	2.3442	1.8824	1.6123	1.2922	1.1989	1.0927	1.0174
0.050	20.092	1.6421	3.6495	3.0215	2.3934	2.0260	1.4568	1.3254	1.1760	1.0701
0.100	10.000	2.4037	5.5112	4.5055	3.4998	2.9115	2.0748	1.8390	1.5709	1.3806
0.110	9.112	2.3954	5.5009	4.4990	3.4972	2.9111	2.0633	1.8238	1.5514	1.3582
0.123	8.111	2.3807	5.4692	4.4769	3.4847	2.9042	2.0455	1.8021	1.5253	1.3290
0.142	7.055	2.3632	5.4189	4.4421	3.4653	2.8940	2.0255	1.7778	1.4963	1.2966
0.167	5.995	2.3430	5.3460	4.3922	3.4384	2.8804	2.0037	1.7519	1.4655	1.2624
0.201	4.977	2.3200	5.2461	4.3240	3.4019	2.8625	1.9804	1.7245	1.4335	1.2271
0.248	4.037	2.1576	4.8115	3.9831	3.1548	2.6702	1.8395	1.5953	1.3177	1.1208
0.335	2.984	1.9339	4.2016	3.5044	2.8072	2.3994	1.6479	1.4222	1.1657	0.9838
0.498	2.009	1.6302	3.3925	2.8626	2.3328	2.0229	1.3908	1.1949	0.9722	0.8142
1.000	1.000	0.9568	1.8092	1.5647	1.3201	1.1770	0.8225	0.7053	0.5721	0.4776
1.123	0.890	0.8543	1.5865	1.3782	1.1700	1.0482	0.7360	0.6316	0.5129	0.4287
1.262	0.793	0.7622	1.3892	1.2124	1.0357	0.9323	0.6581	0.5652	0.4597	0.3848
1.417	0.706	0.6772	1.2107	1.0616	0.9126	0.8254	0.5862	0.5042	0.4110	0.3449
1.668	0.600	0.5678	0.9870	0.8713	0.7557	0.6881	0.4935	0.4255	0.3483	0.2935
2.009	0.498	0.4642	0.7801	0.6943	0.6084	0.5582	0.4056	0.3511	0.2892	0.2452
2.477	0.404	0.3171	0.5122	0.4598	0.4074	0.3767	0.2791	0.2430	0.2020	0.1729
3.351	0.298	0.1774	0.2686	0.2449	0.2212	0.2073	0.1582	0.1394	0.1179	0.1027
4.978	0.201	0.0843	0.1158	0.1079	0.1000	0.0954	0.0769	0.0692	0.0604	0.0541
10.000	0.100	0.0208	0.0225	0.0222	0.0218	0.0216	0.0201	0.0191	0.0180	0.0172

Source: DTN: MO0706DSDR1E4A.001 [DIRS 181421]].



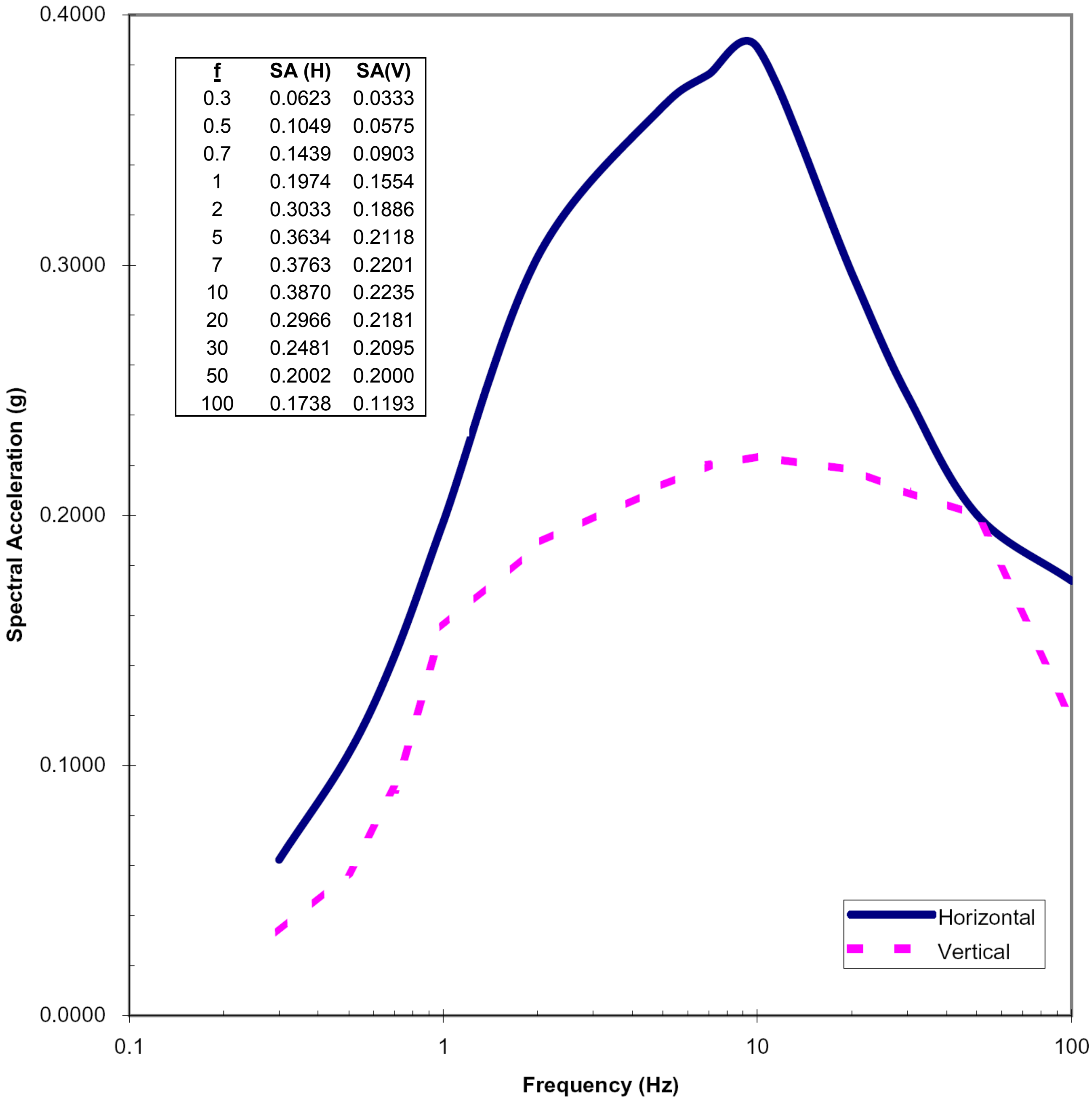
Source: DTN: MO0706DSDR1E4A.001 [DIRS 181421]], *Spcdamp10-4_TDMS.xls*, 'Vertical spectra'.

Figure 6-6. Vertical Spectra for the Design of Surface Structures, 10^{-4}, BDBGM

Table 6-8. Vertical Spectra (Digitized) for the Design of Surface Structures, 10^{-4}, BDBGM

Period (s)	Freq (Hz)	SA (5.0%)	SA (0.5%)	SA (1.0%)	SA (2.0%)	SA (3.0%)	SA (7.0%)	SA (10.0%)	SA (15.0%)	SA (20.0%)
0.010	100.000	0.7230	0.7230	0.7230	0.7230	0.7230	0.7230	0.7230	0.7230	0.7230
0.011	91.116	0.7603	0.7603	0.7603	0.7603	0.7603	0.7603	0.7603	0.7603	0.7603
0.012	81.113	0.8130	0.8130	0.8130	0.8130	0.8130	0.8130	0.8130	0.8130	0.8130
0.014	70.548	0.8828	1.1176	1.0349	0.9522	0.9038	0.8489	0.8379	0.8255	0.8166
0.017	59.948	0.9694	1.4198	1.2645	1.1092	1.0184	0.9153	0.8880	0.8571	0.8351
0.020	49.770	1.0753	1.7872	1.5445	1.3017	1.1597	0.9960	0.9489	0.8954	0.8574
0.025	40.370	1.2086	2.2352	1.8880	1.5407	1.3376	1.0985	1.0274	0.9466	0.8893
0.034	29.837	1.4388	2.9660	2.4542	1.9423	1.6430	1.2782	1.1685	1.0438	0.9554
0.050	20.092	1.7859	4.0145	3.2761	2.5377	2.1057	1.5508	1.3851	1.1968	1.0632
0.100	10.000	2.2060	5.2488	4.2624	3.2760	2.6990	1.8707	1.6289	1.3541	1.1591
0.110	9.112	2.1577	5.1390	4.1756	3.2121	2.6485	1.8263	1.5868	1.3146	1.1214
0.123	8.111	2.0238	4.8179	3.9187	3.0195	2.4935	1.7094	1.4817	1.2228	1.0391
0.142	7.055	1.8431	4.3750	3.5644	2.7538	2.2796	1.5541	1.3439	1.1049	0.9353
0.167	5.995	1.6461	3.8826	3.1710	2.4595	2.0433	1.3860	1.1959	0.9797	0.8263
0.201	4.977	1.4617	3.4108	2.7954	2.1800	1.8200	1.2296	1.0588	0.8647	0.7269
0.248	4.037	1.2945	2.9723	2.4474	1.9225	1.6154	1.0889	0.9363	0.7627	0.6396
0.335	2.984	1.1253	2.5078	2.0814	1.6549	1.4055	0.9476	0.8139	0.6618	0.5539
0.498	2.009	0.9220	1.9565	1.6440	1.3315	1.1487	0.7800	0.6701	0.5452	0.4566
1.000	1.000	0.5125	0.9760	0.8418	0.7076	0.6291	0.4395	0.3795	0.3112	0.2628
1.123	0.890	0.4563	0.8514	0.7379	0.6244	0.5581	0.3926	0.3395	0.2791	0.2363
1.262	0.793	0.4052	0.7404	0.6449	0.5494	0.4935	0.3498	0.3030	0.2498	0.2121
1.417	0.706	0.3597	0.6432	0.5631	0.4830	0.4361	0.3116	0.2705	0.2238	0.1906
1.668	0.600	0.3041	0.5270	0.4648	0.4025	0.3661	0.2649	0.2308	0.1919	0.1643
2.009	0.498	0.2476	0.4134	0.3677	0.3221	0.2954	0.2173	0.1901	0.1592	0.1373
2.477	0.404	0.1687	0.2694	0.2422	0.2149	0.1990	0.1493	0.1315	0.1112	0.0969
3.351	0.298	0.0895	0.1335	0.1219	0.1103	0.1035	0.0804	0.0716	0.0617	0.0546
4.978	0.201	0.0379	0.0511	0.0478	0.0445	0.0425	0.0349	0.0317	0.0281	0.0256
10.000	0.100	0.0080	0.0086	0.0085	0.0084	0.0083	0.0078	0.0075	0.0071	0.0069

Source: DTN: MO0706DSDR1E4A.001 [DIRS 181421]].



Source: DTN MO0707DSRB5E4A.000 [DIRS 183130]

f = Frequency (Hz); SA(H) = Horizontal Spectral Acceleration (g), SA(V) = Vertical Spectral Acceleration (g).

Figure 6-7. Horizontal and Vertical Spectrum for the Design of Subsurface Facilities, 5×10^{-4}, DBGM-2, Five-Percent Damping

6.4 DESIGN BASIS GROUND MOTION FOR NON-ITS STRUCTURES

Design for non-ITS facilities is based on IBC 2000 (ICC 2000 [DIRS 173525]). To be consistent with the ITS SSCs and to be able to use the database on site-specific geotechnical testing, design of non-ITS facilities will be based on site-specific seismic data as described in Sections 6.4.1 and 6.4.2.

6.4.1 Design Spectra for Non-ITS Surface Facilities

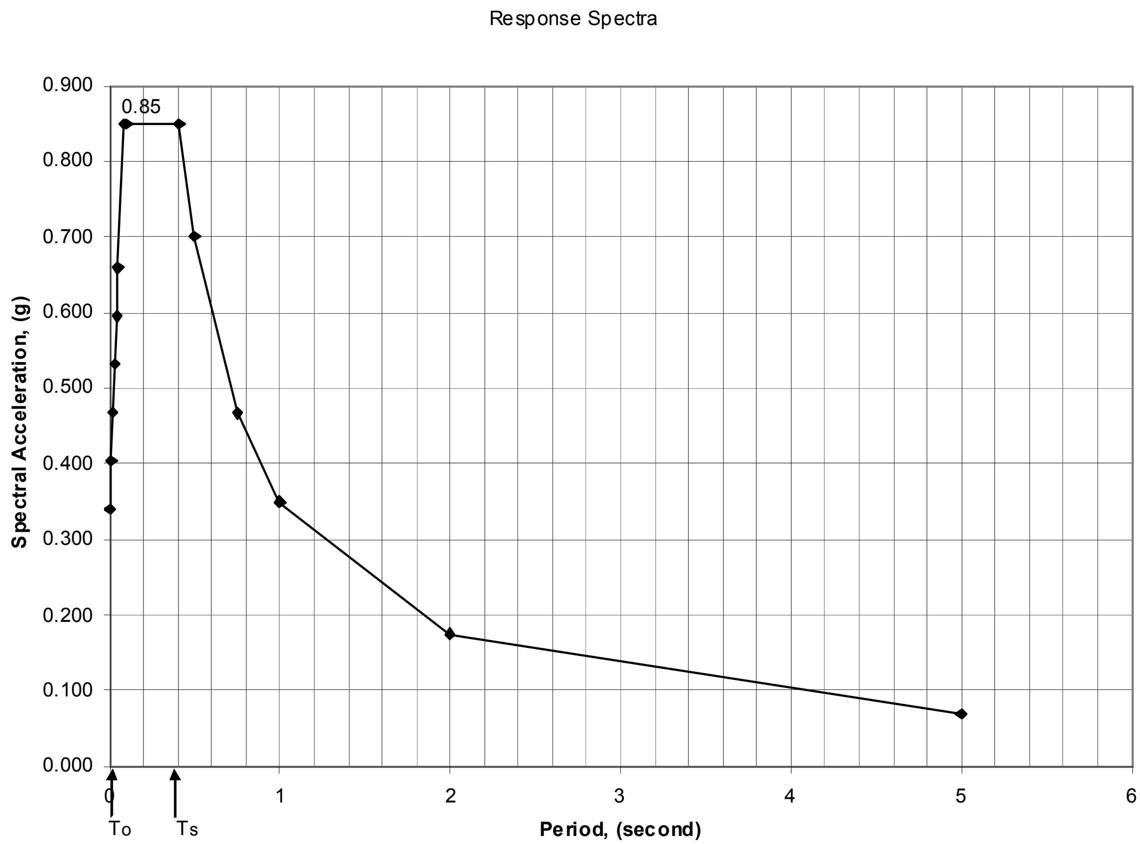
The site-specific design spectra for the non-ITS surface facilities is developed using the approach given in IBC 2000 (ICC 2000 [DIRS 173525]), as follows:

- Determine the site-specific surface spectra for a 2,500-year return period for 5% damping by interpolation between 5×10^{-4} and 10^{-4} probability earthquakes. These spectra will correspond to the "maximum considered earthquake" in IBC.

- Apply the 2/3rds factor to the 2,500-year return period accelerations to obtain the approximate 500-year return period earthquake design parameters.

The resulting design spectrum for non-ITS surface structures is calculated by BSC (2007 [DIRS 184022]) and is shown in Figure 6-8.

Ground motions used during the initial design of the non-ITS surface facilities are given in BSC (2006 [DIRS 177170])



Source: BSC 2007 [DIRS 184022], Figure 3.

Figure 6-8. Horizontal Design Response Spectrum for Non-ITS Surface Facilities, Five-Percent Damping

6.4.2 Design Spectra for Non-ITS Subsurface Facilities

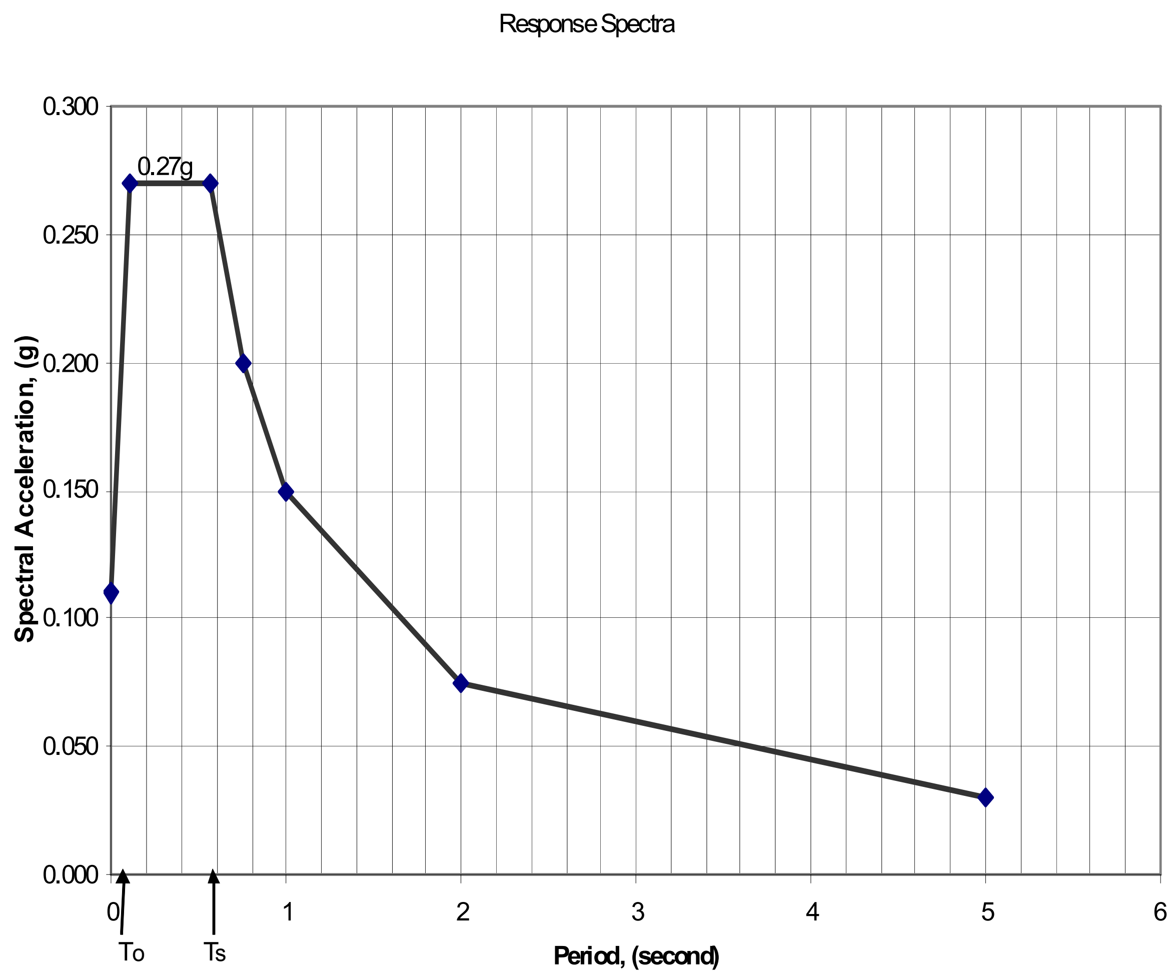
The site-specific design spectra for the non-ITS subsurface facilities is also developed using the approach given in IBC 2000 (ICC 2000 [DIRS 173525]), as follows:

- Determine the site-specific subsurface spectra for a 2,500-year return period for 5% damping by interpolation between 5×10^{-4} and 10^{-4} probability earthquakes. These spectra will correspond to the "maximum considered earthquake" in IBC.

- Apply the 2/3rds factor to the 2,500-year return period accelerations to obtain the approximate 500-year return period earthquake design parameters.

The resulting site-specific design spectrum for the non-ITS subsurface structures is calculated by BSC (2007 [DIRS 184192]) and is shown in Figure 6-9. All non-ITS structures will be designed for 5% damping.

Ground motions used during the initial design of the non-ITS subsurface facilities are given in BSC (2006 [DIRS 178243])



Source: BSC 200 [DIRS 184192], Figure 3.

Figure 6-9. Horizontal Design Response Spectrum for Non-ITS Subsurface Facilities, Five-Percent Damping

7. SEISMIC ANALYSIS OF ITS SSCs

7.1 METHODOLOGY

Various seismic analyses are needed to determine the seismic response of structures as well as to calculate the effects of seismic loads on SSCs. These analyses include: (1) seismic response analysis of structures–to determine seismic responses in terms of the nodal accelerations, and to determine in-structure response spectra, (2) seismic stress analysis of structures–to determine the internal forces and moments in the structures, and (3) seismic analysis of systems and components–to determine the design forces for the supports of systems and components, to qualify these systems and components, and to determine the loads on the supporting structure.

These analyses will be performed during the preliminary design stage (Tier # 1 Analyses) and also during the detail design stage (Tier # 2 Analyses) using different approaches. In general, an approximate method is adequate during the preliminary stages (Tier # 1) and a more refined approach is needed for detail design (Tier # 2). Attributes of Tier # 1 and Tier # 2 analyses are described in the Sections 7.1.1 and 7.1.2.

In addition to seismic, the structures must be analyzed for the non-seismic loads, such as gravity loads, lateral soil pressures, hydrodynamic loads and other applicable loads. The analyses for both seismic and non-seismic loads are discussed in Sections 7.1.1 and 7.1.2.

7.1.1 Tier # 1 Analyses

A response spectrum analysis (RSA) will be carried out using a lumped-mass multi-stick model with the surface design response spectrum. Results of this analysis will include gross overturning moments and sliding forces, floor accelerations, and individual element forces. The overall overturning moments and sliding forces will be used for stability evaluations. The floor accelerations will be used to assess the model and the amplification throughout the structure, and later will be compared with the Tier # 2 results for confirmation of the models. Finally, the element forces will be used for preliminary design of selected critical walls. The Tier # 1 multi-stick model is for a simplified analysis and, therefore, the floors are considered rigid. Furthermore, the primary interest is the in-plane response of the structure and concrete cracking does not significantly affect the in-plane response. Therefore, cracking is not considered in Tier # 1 analysis.

The same multi-stick model will be used to perform a static analysis under the gravity loads. The results of this analysis will be combined with the seismic analysis results to determine the preliminary design forces. A shear wall design procedure has been developed and is provided in Appendix D.

The Tier # 1 analysis will be carried out using SAP2000 (V. 9.1.4. 2005. WINDOWS 2000. STN: 11198-9.1.4-00 [DIRS 178238]).

7.1.2 Tier # 2 Analyses

For Tier # 2 analyses, a detailed finite-element structural model will be developed after the building layout is sufficiently matured. This model will be used to perform the following analyses:

- A time history seismic analysis, including the soil-structure interaction (SSI) effects, is performed using the computer code SASSI2000 (V. 3.1 2007. WINDOWS XP STN: 10825-3.1-00 [DIRS 182945]). Maximum accelerations at each node in the model are calculated for each of the 9 directions (response in X, Y, and Z directions due to input seismic motion in the X, Y, and Z directions). These maximum nodal accelerations are used to develop the static equivalent seismic load. The acceleration time histories at selected nodes may be used to generate In-Structure-Response-Spectra. In addition, SASSI2000 may be used to obtain directly the element seismic design forces.

- The same model will be used to analyze the structure under applicable non-seismic loads using SAP2000 [DIRS 178238]. The analyses results will include element forces and nodal displacements.

7.1.3 Design of Structures

The preliminary structural design will be based on the forces and moments obtained from Tier # 1 analysis. For this purpose the shear wall design spreadsheets previously discussed and described in Appendix D will be utilized for the design. It is expected that the Tier # 1 analyses results will result in a conservative design. Tier # 1 design will be documented with adequate data (i.e. load combinations, section forces, resulting reinforcement, and demand/capacity (D/C) ratios) to permit future comparison with the Tier # 2 design. It is expected that the Tier # 1 design D/C ratios will be significantly smaller than unity, as discussed subsequently in Section 8.4 (Equation 8-4). The resulting design is expected to meet the performance objectives of the limited seismic probabilistic risk assessment.

When the Tier # 2 analyses are completed, the resulting design forces will form the design basis. These forces (or the required member sizes or reinforcement) will be compared with the Tier # 1 analysis results to demonstrate that the Tier # 1 design is adequate. Once the comparison shows the adequacy of the preliminary design, the results will be documented. In areas where further refinement may be necessary or where the Tier # 1 design is not available, additional design calculations will be made and documented to finalize the design. The design acceptance criteria are given in Section 8.4.

In areas where the design reinforcement is deemed to be excessive as a result of the static equivalent seismic analysis and seismic load combination, further analysis and design will be carried out. For this purpose, YMP has modified the design program OPTCON Program Module for SASSI2000 (V. 1.0. 2006. WINDOWS 2000. STN: 11208-UID-1.0-00 [DIRS 177553]), which optimizes the design reinforcement. This program is intended to be used for sections with the highest D/C ratios in conjunction with the Tier # 2 analyses results to (1) confirm the Tier # 1 design and (2) determine a more realistic seismic demand (i.e. more realistic fragility curves) that will help to meet the required performance objectives of the limited seismic probabilistic risk assessment (Appendix B). OPTCON [DIRS 178237] analyses results can also be used to reduce excessive reinforcement where appropriate. In the final design, all D/C ratios will be less than or equal to unity (Section 8.4).

7.1.4 Other Analyses

In addition to the previously described analyses, special analyses will be carried out to determine the structural sliding and overturning responses. Guidance on these issues is provided in ASCE/SEI 43-05 [DIRS 173805], Section 7.0.

Another special analysis is the determination of fragility for structures. In general, the fragilities for structures will be calculated using the "Conservative Deterministic Failure Margin" approach, which may also require special nonlinear analyses. Guidance for performing the Conservative Deterministic Failure Margin (CDFM) analysis is given in Appendix B.

7.2 ANALYSES PARAMETERS

7.2.1 Modeling

Different models will be used for the Tier # 1 (multi-stick model) and the Tier # 2 (finite element model) analyses as described in Sections 7.2.1.1 and 7.2.1.2, respectively, for both seismic response and stress analyses.

In both models the dead load will include the weight of the structure, partitions, permanent equipment, piping, raceways, HVAC ductwork, and other permanent static loads. The seismic mass will consist of full dead load and 25% of design live load.

The other model properties (element types, boundary conditions, soils properties, input motions, coupling criteria, etc.) and parameters to be used in analyses (modal and spatial combinations, damping, etc.) are discussed in Sections 7.2.2 through 7.2.10.

7.2.1.1 Tier # 1 Model

Tier # 1 seismic analysis will be performed using a lumped-mass, multi-stick model in which all walls or segments of walls are modeled as beam elements using gross section properties. The beams span between the floors. Ends of the beams are constrained to a master node at each floor diaphragm level and, thus, the floors are considered to be rigid in all three directions. Soil springs will be calculated in accordance with Appendix C considering the layered media.

7.2.1.2 Tier # 2 Model

The finite element model used for Tier #2 stress analysis will include the entire structure, including the foundation mat, walls, roof and floor slabs, structural steel framing, and major penetrations and openings in the walls and slabs. In general, small openings may be represented by determining an equivalent thickness for the corresponding element. Since the structures are founded on soil, it is important to take into account the effects of foundation flexibility. For seismic stress analysis, this may be accomplished by calculating the soil impedances from the

SSI model or by other appropriate methods and converting them into soil springs (see Appendix C for soil spring calculation methodology). Since the soil impedances are frequency dependent, the values corresponding to the fundamental frequency of the soil-structure system will be used in calculating the soil springs. The distribution of soil springs should be based on the rocking impedance of the foundation.

The concrete slabs and walls will be modeled using an improved shell element (i.e., with out-of-plane shear calculation capability) that has recently been added to SASSI2000 [DIRS 182945]. Similar elements also exist in the SAP2000 [DIRS 178238] library. Concrete columns (if any) and selected steel members will be modeled by beam elements. In slabs, the effect of the supporting steel may be approximated by composite action. The resulting seismic forces will then be used for composite design of the system. Concrete cracking will also be taken into account where it is deemed significant.

The mesh size used in a finite element model should be adequate to obtain accurate design forces and moments. In-plane forces can be accurately determined using a coarse mesh. In general, only two elements would be sufficient for determination of the in-plane forces between two supports (i.e. between floors or walls). On the other hand, out-of-plane forces require a refined mesh; in general a minimum of six elements are needed between supports. Therefore, modeling should consider not only the geometry, but also the relative importance of the in-plane and out-of-plane forces and moments on design. This requires exercise of judgment to obtain sufficient accuracy and to avoid making the model so complicated that meaningful interpretation of the results may be compromised.

The detailed SAP2000 [DIRS 178238] analysis model will include mathematical representation of the soil layers around and beneath the structure. The procedure for calculating soil springs is given in Appendix C.

The embedment effects (if any) will be taken into account by considering the soil backfill. The soil nonlinearity will be considered using an equivalent linear method. The strain-compatible soil properties using the equivalent linear method are provided in DTN: MO0706SCSPS5E4.002 [DIRS 181616] and in DTN: MO0706SCSPS1E4.002 [DIRS 181618]).

7.2.2 Input Motions

7.2.2.1 Tier # 1 Analysis

The DRS are used for the input motion in Tier # 1 analysis. The DRS for the YMP are given in Section 6 for both surface and subsurface facilities.

The spectra for the ITS SSCs are given at different damping levels. These spectra should be applied at the foundation level. In RSA, the appropriate damping curve should be used for determining the modal damping for each mode.

7.2.2.2 Tier # 2 Analysis

In Tier # 2 dynamic analysis, the acceleration time histories defined in Section 6 will be used as input motion in the SSI analyses. The DRS provided in Section 6 include soil amplification effects and, therefore, the control point for the time histories will be set at the ground surface level in the free-field. The wave field will consist of vertically propagating shear and compression waves. Variation of amplitude and frequency content with depth in the free-field motion will be considered in the analysis as recommended in Section 3.3 of ASCE 4-98 [DIRS 159618]. ASCE 4-98 also considers the accidental eccentricity as discussed in Sections 7.2.8 and 7.2.9 to fully account for the possible effects of nonvertically propagating waves.

7.2.3 Dynamic Soil Properties

Poisson's ratio and total density will be obtained from the site-specific geotechnical investigation report (BSC 2007 [DIRS 182582]). Dynamic soil properties in terms of shear and compression wave velocities and low-strain shear wave velocity will be as given in DTN: MO0706SCSPS5E4.002 [DIRS 181616] and DTN: MO0706SCSPS1E4.002 [DIRS 181618]. The strain-compatible soil properties will be used in the SSI analyses.

7.2.4 Damping

7.2.4.1 Soil Damping

Soil damping may be an important factor in the response of the structure in Tier # 1 RSA. Soil springs and associated damping coefficients (dashpots) can be calculated using the half-space approach (ASCE 4-98 [DIRS 159618], Section 3.3). Methods are available to determine the composite modal damping for structures supported on soil springs. The composite modal damping (ASCE 4-98 [DIRS 159618], Section 3.1.5) will be used in the response analysis to ensure more realistic response calculations.

In Tier # 2 SASSI analysis, the soil damping is accounted for by modeling the soil medium, including radiation-damping effects.

Appendix C provides additional information on soil damping.

7.2.4.2 Structural Damping

The structural damping values are given as a function of response level in members and are listed in Table 7-1 (ASCE/SEI 43-05 [DIRS 173805], Section 3.4.3). The response levels relate to the stress levels in terms of demand-capacity ratios; less than 0.5 for Response Level 1, between 0.5 and 1.0 for Response Level 2, and equal to or greater than 1.0 for Response Level 3. Response Level 2 damping values will be used for computing seismic loads. Response Level 1 values will be used for developing in-structure response spectra and input motions for subsystems. Level 3 values will be used in BDBGM evaluations.

Table 7-1. Structural Damping Values for Structures Important to Safety

Structure	Damping (% of critical)		
	Response Level 1	Response Level 2	Response Level 3
Reinforced concrete structures	4	7	10
Bearing-bolted steel structures	4	7	10
Friction-bolted steel structures	2	4	7
Welded steel structures	2	4	7

Source: ASCE/SEI 43-05 [DIRS 173805].

NOTES: Response Level 1 corresponds to ground motion less than DBGM, Level 2 corresponds to DGBM (both DBGM-1 and DBGM-2), and Level 3 corresponds to BDBGM.

Use Response Level 1 damping when elastic buckling controls the design. For all other conditions, use Response Level 2 damping.

In lieu of two seismic analyses of the structure, one with Response Level 1 damping values and the other with Response Level 2 damping values as previously described, a single analysis using only the Response Level 1 damping values may be performed. This will be a conservative approach, as it will increase the seismic design forces.

7.2.4.3 System and Component Damping

The damping values for systems and components will be Level 1 and Level 2 damping values given in ASCE/SEI 43-05 [DIRS 173805]. Response Level 1 corresponds to a ground motion less than DBGM. Therefore, Level 1 damping is used only when the elastic buckling controls the design. For all other cases of DBGM-1and DBGM-2 ground motions, Level 2 damping is used.

7.2.5 Subsystem Dynamic Coupling Criteria

For coupling effects, the recommendation of ASCE 4-98 [DIRS 159618] will be used in modeling the primary system and the secondary systems. Section 3.1.7 of ASCE 4-98 [DIRS 159618] will be used to identify the level of coupling between the subsystems (secondary systems) and the primary system in the structural model. It is anticipated that only the total inertia of the subsystems may need to be included in the primary model. In the event that subsystems need to be modeled as part of the primary model, the recommendations of Section 3.1.7 of ASCE 4-98 will be used to model the subsystems.

7.2.6 Modes and Modal Combinations

In RSA, a sufficient number of modes must be extracted to account for at least 90% of the total mass, in each direction, as required by ASCE 7-98 [DIRS 149921], Section 9.5.4.3. In some structures and systems, this criterion may require extraction of a large number of modes. As for the modal combinations, the methodology given in ASCE 4-98 [DIRS 159618], Section 3.2.7, will be followed. SASSI analysis uses the frequency domain method and, therefore, modal questions do not arise.

7.2.7 Spatial Combinations

7.2.7.1 Dynamic Analysis

The total response of the structure should be obtained by combining the three co-directional responses by the "Component Factor Method (1/0.4/0.4)," illustrated by equation (3.2-26) of ASCE 4-98 [DIRS 159618]. The forces should be combined in the design stage considering both (+) and (−) signs by permutation (see Appendix A for application). Alternatively, the co-directional responses may be combined using the square-root-of-the-sum-of-the-squares (SRSS) method. The SRSS forces and moments do not have any signs associated with them.

In Tier #2 SSI analysis using SASSI computer code, the calculated maximum accelerations at each node are used to develop the static equivalent seismic load in up to nine directions (3-directional responses due to each of the 3 directional input). Depending on the dynamic behavior of the structure, the co-directional responses may remain separated, combined by (1/0.4/0.4 aka. 100, 40, 40) method or combined by the very conservative absolute sum method. The resulting accelerations will be used as input in the equivalent static stress analysis using a detailed finite element model of the structure.

7.2.7.2 Equivalent Static Analysis

Accelerations obtained from SASSI analyses are applied to the structure statically and the resulting forces are calculated. The resulting section forces can then be combined using the component factor method or the SRSS method. Since the resultant member forces do not carry any sign, variation in sign will be considered in the load combinations. This process will result in a large number of spatial response calculations. The challenge is then to define a simplified approach to reduce the number of combinations. Such a method has been developed and the spatial combinations to be considered in design are reduced to 24. This approximate approach is conservative relative to a more rigorous analysis approach.

Derivation of the method and the application to the seismic analyses results are described in Appendix A.

7.2.8 Torsional Effects

The analytical models used in stress analysis account for the geometrical relationships between the elements of the structure and their masses, and thus the member forces obtained from the computer analyses include the torsional effects from the actual eccentricities.

In addition to the actual torsion, an "accidental" torsion must be incorporated into the design. The effects of accidental torsion will be calculated and then added to the member forces obtained from the static analysis.

The accidental torsional moment is computed at each major floor elevation as the product of the story shear and 5% of building dimension in the direction perpendicular to the direction of the shear force (ASCE 4-98 [DIRS 159618], Section 3.1.1(e)). The resulting torsional moments are applied to each major slab level at the shear center, where the shear center at each level is calculated assuming rigid floor diaphragm and rigidity of the supporting shear walls below.

When RSA is performed, the accidental torsion moment and its effect on individual members should be calculated as previously described. Alternatively, the model can be analyzed for torsional loads by converting the torsion to linear forces at each floor level and applying these forces as lateral loads, with proper directional consideration.

The lateral forces due to torsion should be zero at the center of rigidity and should increase linearly toward the edge of the building.

The accidental torsion effects should be combined with other seismic effects at element level to determine the design forces.

7.2.9 Concrete Cracking

Concrete cracking will reduce the stiffness of a structure (ASCE/SEI 43-05 [DIRS 173805], Table 3-1). The reduction in stiffness will depend on the state of stress and on the dominant response mode (i.e., flexural or shear deformation modes). Considering that the concrete modulus is underestimated using the code minimum design strength formula, and that under the design loads the structure remains essentially elastic, concrete cracking will not be considered for DBGM. For BDBGM, the following criterion will be applied:

- Slabs will be considered cracked and the reduced modulus per ASCE/SEI 43-05 [DIRS 173805], Table 3-1, will be used in the analysis.
- Walls will be uncracked since the out-of-plane bending is generally small with respect to capacity and in-plane stiffness is not significantly affected for Limit States B and C.

Appendix E provides guidance on incorporating the slab cracking into the dynamic analyses.

7.2.10 Equipment-Structure Interaction Analysis

The ISRS discussed in Section 7.3.2.2 is normally used for uncoupled equipment and component analysis and is reasonably conservative for horizontal responses. In the vertical direction, however, due to flexibility of the supporting slabs the results may be too conservative since the mass interaction effects between the mass of the equipment and the mass of the supporting slab are not fully accounted for in SASSI2000 [DIRS 182945]. This inherent conservatism may make it impractical to design the equipment or the component and their supports.

More realistic equipment and component response can be obtained by two different approaches. One approach is to include the contributing masses of the slabs and the equipment in the seismic model as single-degree-of-freedom attachments. This approach will account for most of the mass-interaction effects and yet provide reasonable spectra for subsystem design.

The second approach is to perform an equipment-structure-interaction analysis in which the equipment and supporting mass interaction is analyzed separately, using the wall responses at the slab support points as input. In this analysis the equipment or component is modeled either as finite element or lumped-mass models, together with the impedances at the support points. The system is then analyzed using the time-histories at the support points as input and the subsystem response is calculated.

The guidance given in ASCE 4-98 [DIRS 159618], Section 3.4, will be used in addressing the equipment-structure interaction issues.

7.3 SEISMIC RESPONSE ANALYSIS OF STRUCTURES

The purpose of seismic response analysis is to determine the seismic loads (or nodal accelerations) in the structure and to develop the in-structure response spectra (ISRS). Seismic response analyses in Tier # 1 and Tier # 2 are described in Sections 7.3.1 and 7.3.2.

7.3.1 Tier # 1 Response Spectrum Analysis

Tier # 1 seismic analysis will be performed using a multi-stick model (Section 7.1.1) and the RSA approach (Section 7.4.3). The RSA results will include beam element forces that will be used directly in preliminary design. In addition, floor accelerations will be determined to establish the demand on systems and components at zero-period acceleration. These accelerations may be used to estimate the seismic demand for various systems and components in the structures.

7.3.2 Tier # 2 Time-History Analysis

Structural responses in terms of nodal accelerations (i.e. bubble accelerations) will be obtained from SASSI analyses (Section 7.1.2). These accelerations will be processed to determine conservative floor accelerations over several regions at each floor elevation, which will then be applied to the detailed model of the structure for static stress analysis. The element stresses will be further processed to obtain the section forces and moments for design in accordance with ACI 349-01 [DIRS 181670] requirements.

Structural responses in terms of nodal time-histories will be saved and then processed to obtain the ISRS. The ISRS will be developed at selected wall-slab junctions. In addition, the dynamic analysis model may incorporate single-degree-of-freedom elements (lollipops) to represent individual flexible slabs up to 25.0 Hz. Time-history responses at these nodes will be processed to obtain ISRS considering floor flexibility, including concrete cracking.

7.3.2.1 Soil-Structure Interaction Effects

The seismic response analyses are performed using SASSI2000 [DIRS 182945]. The foundation model and the seismic model of the structure are combined to form the SSI model. SSI analysis of each structure includes:

- The layering effects of the supporting soil layers and the radiation damping associated with soil-foundation interaction
- SSI analysis with best-estimate upper-bound and lower-bound soil property profiles
- Flexibility of the basemat and embedded walls of the structure

- Soil pressures behind the embedded walls of the structure will be tracked in the SSI analysis. Parametric studies will be performed to include the effect of potential soil-wall separation.

- Consideration of structure-to-structure interaction analysis for local effects, such as lateral seismic soil pressures, if warranted, will be given per Section 3.3.1.5 of ASCE 4-98 [DIRS 159618]

The effect of uneven thickness of alluvium on the foundation response will be assessed and incorporated in design loads if warranted.

7.3.2.2 Generation of In-Structure Response Spectra

In-structure acceleration response spectra will be generated for 0.5%, 1%, 2%, 3%, 5%, 7%, and 10% damping at the locations of the subsystems. The SRSS method will be used to combine the spectral amplitudes of co-directional responses. The responses from the best-estimate lower-bound and upper-bound soil properties will be enveloped. ISRS are calculated between 0.2 Hz. and 34.0 Hz. at frequency steps equal to 100 frequencies per decade that are equally spaced in the log scale. A peak broadening of plus or minus 15% will be used in accordance with the recommendations of Section 3.4.2.3 of ASCE 4-98 unless a more rigorous analysis is performed to determine the peak broadening. The enveloping acceleration response spectra will be constructed in accordance with the requirements of Regulatory Guide 1.122 [DIRS 151404].

7.4 SEISMIC STRESS ANALYSIS OF STRUCTURES

Seismic stress analysis of the structures to determine the design forces and moments will be carried out using the Tier # 1 and Tier # 2 models described in Section 7.1.1 and 7.1.2, respectively, and any one of the following approaches:

- **Code Approach**–Using the IBC 2000 (ICC 2000 [DIRS 173525]) approach with the design spectra at the surface as input

- **Static Method**–Using a finite element model of the structure in a static analysis, with the floor accelerations obtained from the seismic response analysis of Section 7.2 as input

- **Response Spectrum Analysis**–Using finite element or lumped-mass models and performing an RSA with the design spectra at the surface as input.

- **Time-History Analysis**–Using time-histories to obtain realistic member design forces.

Application of these methods is discussed in Sections 7.4.1 through 7.4.5.

7.4.1 Code Approach

This method may be used for very simple structures following the equivalent static procedures with the design spectra given in Section 6.3 for ITS structures. This approach is not discussed in detail as the procedures are given in applicable codes (ICC 2000 [DIRS 173525]).

7.4.2 Static Method

Static analysis methods to determine the seismic forces will not be used in Tier # 1.

In Tier # 2, models will be analyzed using the static equivalent methods, using the floor accelerations determined from the time-history response analysis. All floor seismic loads in each direction will be applied simultaneously. The resulting member forces represent the earthquake effects in one direction. The analysis will be repeated for all three earthquakes in three orthogonal directions. The results will be combined as described in Section 7.2.7 to determine the total seismic effects.

All ITS major structures will be analyzed using this approach to determine the design forces. Various aspects of this approach are discussed in Sections 7.4.3 through 7.4.5.

7.4.3 Response Spectrum Method

For Tier # 1, seismic forces and moments for each "stick" will be determined from the RSA of the multi-stick model (Section 7.1.1). These forces and moments may be used in preliminary design directly. If necessary, element forces and moments may be further processed to determine the design forces in parts of the members, such as piers. The foundation soil springs and dashpots need to be computed to accurately represent the SSI effects (see Appendix C and Section 3.3 of ASCE 4-98 [DIRS 159618]).

For Tier # 2, ITS structures may also be analyzed using the RSA method in conjunction with the finite element model (Section 7.1.2). In this case, a fixed-base model of the structure will be subjected to the amplified ISRS at the basemat. The resulting element forces will then be used in detail design with the application of the criterion for spatial combinations.

In either the multi-stick or finite element analysis, the model must have sufficient details to identify the seismic forces associated with each element of the structure.

The RSA method does not provide in-structure response spectra for design and qualification of SSCs. Time history analysis will be performed for development of ISRS.

7.4.4 Time History Analysis

A time history method will provide the most realistic design forces, because simplified conservative approximations, routinely made in the other approaches, is not necessary. The finite element model (Section 7.1.2) of the structure can be analyzed in each direction, tracking the responses. Then, the responses from seismic input in each direction can be combined in the time domain (and using the component factor method) to determine the controlling load combination. Design using these forces will be accurate and realistic and will result in an optimum reinforcement design.

7.4.5 Foundation Evaluation

The foundation will be designed for applicable static and seismic load combinations. The effect of local uplifts will be considered in the design. The foundation soil pressure will be evaluated to assess the potential for any local bearing failure or excessive settlement that may develop in the soil. These evaluations will be carried out for the required load combinations considering the pressure-settlement relationships under both long- and short-term loads.

If the soil pressures, due to the seismic loads, exceed the pressures under the gravity loads, soil-structure separation is indicated. In such cases, iterative analyses may be performed by eliminating the soil springs in tension under the combined effects of seismic and gravity, and by ensuring that the remaining springs are in compression. Alternatively, this could be accomplished by utilizing the non-linear compression only spring elements of SAP2000 or similar.

7.5 SEISMIC ANALYSIS OF SYSTEMS AND COMPONENTS

7.5.1 Analysis Methods

Seismic analysis of systems and components can be carried out by one of the following analysis methods, based on the characteristics and complexities of the system and component:

- Equivalent static analysis
- Dynamic analysis.

Equivalent static analysis may be used for simple systems and components.

The dynamic analysis will generally be carried out using the RSA, using either a lumped-mass or finite element model (Section 7.1.2) of the system or component. Analysis will be carried out following ASCE 4-98 [DIRS 159618], Section 3.4, and BC-Top-4A (Tsai et al. 1974 [DIRS 166056]).

In lieu of an analysis, equipment may be seismically qualified by testing, as described in Section 7.5.5.

7.5.2 Equivalent Static Method for Systems and Components

Equivalent static analysis method may be used in lieu of a dynamic analysis if a simple model can realistically represent the system or component. A static analysis should be performed by application of equivalent static forces at the mass locations in two principal horizontal directions and the vertical direction. The equivalent static force at a mass location should be computed as the product of the mass and the seismic acceleration value applicable to that mass location. The seismic acceleration values should be determined as follows:

- **Single Mode Dominant Response**–The acceleration value from the applicable in-structure response spectrum should be used. In lieu of calculating the natural frequency, the peak value of the in-structure response spectrum acceleration may be used conservatively.

- **Multiple Mode Dominant Response**–1.5 times the peak acceleration value of the applicable in-structure response spectrum should be used. However, this approach may be too conservative and, therefore, a dynamic analysis should be considered.

7.5.3 Dynamic Analysis of Systems and Components

The dynamic analysis of systems and components can be carried out using the response spectrum, or time-history approach (Section 7.1.2). Time-history analysis may be performed using either the direct integration method or the modal superposition method.

7.5.3.1 Modeling

Equipment–Unless a more complex model (e.g., a finite element model) is required, the equipment may be represented by a lumped-mass system consisting of discrete masses connected by weightless springs. The criteria used to lump masses may be summarized as follows:

- The number of masses is chosen so that all significant modes are included. This is accomplished by ensuring that at least 90% of the mass is contained in the modes used (i.e., cumulative modal mass exceeds 90%), as required by ASCE 7-98 [DIRS 149921], Section 9.5.4.3. Alternately, the number of degrees of freedom is taken more than twice the number of modes with frequencies less than 33 Hz.

- Mass is lumped at the following points:

  – Where a significant concentrated weight is located (e.g., the motor in the analysis of pump motor stand, the impeller in the analysis of pump shaft, etc.)

  – Where there is a significant change in either the geometry or stiffness.

Anchors at equipment, such as tanks, pumps, and heat exchangers, should be modeled with calculated stiffness properties.

Distributive Systems–Distributive systems, such as cable trays and the heating, ventilation, and air-conditioning ducts, should be modeled as an assemblage of system elements supported at discrete points. The model should include the mass and stiffness properties of the supports unless it can be demonstrated that exclusion of the support properties will not change the results.

7.5.3.2 Application of Seismic Loads

For dynamic analysis of systems and components, either the RSA approach or the time-history approach (Section 7.1.2) may be used.

If the RSA approach is used, the input spectra will be the ISRS at the support point of the system or component. The ISRS used in dynamic analysis of systems and components already includes the effects of three-directional seismic input motions. Therefore, the ISRS should be applied one direction at a time and the system or component should be designed for the envelope of responses. If there are multiple supports, then the enveloped ISRS at these support points will be used.

If time-history method is used, the first step is to develop a spectrum-compatible time-history at the support point. Time-histories will be developed in all three directions. These time-histories are then applied to a model of the subsystem or component and the responses are tracked.

Spatial and modal combinations are carried out as described in other sections. The resulting seismic forces or stresses are then compared with allowable values to determine the adequacy of the system or component.

Additionally, see Appendix F for seismic requirements for mechanical equipment.

7.5.3.3 Damping

Damping values for systems and components shall be established in accordance with ASCE/SEI 43-05 [DIRS 173805], Table 3-2, Section 3.4.3.

7.5.4 Seismic Analysis of Miscellaneous Systems and Components

7.5.4.1 Multiple-Supported Systems and Components

The inertial response should be calculated using an upper-bound envelope of individual response spectra for the support locations. The relative seismic support displacement (i.e., seismic anchor motion) should be computed per the recommendations of ASCE/SEI 43-05 [DIRS 173805], Section 8.2.1.1. The response from the relative seismic support displacement analysis should be combined with the response from the inertial loads by the SRSS method.

In lieu of an RSA, time histories of the support motions may be used.

7.5.4.2 Recommended Frequencies

In the design of supports for components, the system frequency, taking into account the supports, should be calculated. Whenever practical, the fundamental system frequency of components should preferably be less than one-half or more than twice the dominant frequencies of the support structure, to avoid significant amplification.

7.5.5 Seismic Qualification of Systems and Components by Other Methods

Sections 7.5.1 through 7.5.4 address seismic qualification of systems and components by analysis. Other qualification methods are available and criteria and procedures for these methods are included in applicable standards. These methods include:

- Qualification by testing
- Combined analysis and testing
- Similarity
- Experience database.

Testing procedures presented in IEEE Std 344-2004 [DIRS 176259] should be followed in qualifying the equipment and components by testing. The actual mounting of the equipment should either be simulated or duplicated. All normal loads acting on the equipment during an earthquake should be addressed. The seismic load should be defined by the required response spectrum obtained by enveloping and smoothing (filling in valleys) the ISRS computed at the supports of the equipment by linear elastic analyses, and multiplied by a factor of 1.4, the equipment capacity factor for qualification by test (IEEE Std 344-2004 [DIRS 176259]). The test response spectrum of the shake table should generally envelop the required response spectrum.

When combined analysis and testing method is used, the interface of scope of work for each method must be clearly established. When similarity or experience database methods are used, objective evidence of the applicability of similarity must be documented, including those related to the supports and attachments. For more information, see ASCE/SEI 43-05 [DIRS 173805], Section 8.3.

7.6 SEISMIC EVALUATION OF STRUCTURES FOR BDBGM

Structures designed for DBGM-2 will be evaluated to determine the seismic effects during the 10,000-year return period earthquake, BDBGM in accordance with Table 5-2. These evaluations will include the following:

- Structural analysis to determine the stresses
- Seismic response analysis to determine the ISRS
- Seismic margin assessment to demonstrate that the high confidence of low probability of failure (HCLPF) capacity values are at least 10% higher than the demand imposed by the BDBGM 10,000-year return period earthquake
- Development of the fragility curves for selected structures and components that are credited with preventing/mitigating unacceptable event sequences. These fragility curves will be used to carry out a limited seismic probabilistic risk assessment.

The approach to be used in these evaluations is given in Appendix B.

7.7 SEISMIC ANALYSIS OF UNDERGROUND ITS SSCs

Although most of the subsurface facilities are expected to be non-ITS, this section for ITS subsurface facilities is provided for possible use in the future.

Underground SSCs include main drifts, emplacement drifts, vertical shafts, collars, and all the systems and components supported by these structures. Emplacement drifts are about 1,000 ft below the ground surface and provide for the emplacement of waste packages. Shafts provide access for the ventilation of the repository as well as emergency egress from the repository horizon to the surface. Shaft collars penetrate the top layers of rock strata at a particular location. In addition to linking the shaft tube with the surface, the shaft collar often serves as the foundation for the surface structure (i.e., hoisting facility and the head frame) (BSC 2007 [DIRS 182131], Section 8.1).

Underground structures track the motions of the surrounding soil medium. Consequently, soil-structure interaction analysis is deemed unnecessary. However, variation of the ground motion with depth must be taken into account in the design of SSCs. In addition, the underground SSCs must also be evaluated for the deformations imposed by the surrounding soil medium.

7.7.1 Seismic Analysis of Force-Controlled Underground SSCs

Seismic inertia loads for the underground SSCs will, in general, be computed using the Equivalent Static Load Method (Section 7.4.2) in accordance with the requirements of NUREG-0800 (NRC 1987 [DIRS 138431], Section 3.7.2). To obtain an equivalent static load in the horizontal direction and in the vertical direction, the spectral acceleration at depth may be used (see Figure 6-7 for underground facilities). If the frequency is not calculated, then a factor of 1.5 shall be applied to the respective peak acceleration of the site-specific response spectra, corresponding to a 2,000-year return period. If the SSC frequency is determined using approximate methods, where a single degree of freedom is representative of the SSC response, then a factor of 1.0 is applied. An appropriate structural damping value for the structure or component, expressed in terms of the percent of critical damping, will be used for Response Level 2 as shown in Table 7-1. For components and systems, the damping values given in ASCE/SEI 43-05 [DIRS 173805], Table 3-2, Response Level 2, should be used.

Alternatively, a dynamic analysis, either response spectrum or time history methods, may be used when the use of the equivalent static load method cannot be justified. Where applicable, torsional effects must be included.

Combination of responses from the three orthogonal components of earthquake motions will be carried out using the processes given in Section 7.2.7.

7.7.2 Seismic Analysis of Deformation-Controlled Underground Structures

Invert steel structure and other SSCs connected to the subsurface emplacement and to the main drift walls will additionally undergo structural deformations that are imposed and controlled by the racking of the cross-section of the drift, caused by the seismic ground motion. Such actions are termed deformation-controlled. Seismically-induced racking deformations will be accounted for in the design of the steel invert structure and other structural components connected to the drift walls that may be affected by such racking.

7.7.3 Seismic Analysis of Vertical Shaft Liners and Collars

For the vertical shafts and collars, both acceleration and deformation responses will be considered. The analyses approach will be similar to those utilized for the drift structures. Design spectra to be used in these analyses may be obtained by a linear interpolation of the spectra at the drift and surface levels. Racking analysis of the shafts and collars will consider the maximum strains imposed by the ground motions and considering the motions in three orthogonal directions. The imposed deformations are important in the design process to ensure acceptable behavior of the shafts and collars.

8. SEISMIC DESIGN OF ITS SSCs

8.1 GENERAL

This section details the criteria to be used for the design of ITS SSCs for load combinations that include seismic loads. It lists the acceptable industry codes to be used in design. It identifies the loads that should be considered in conjunction with the seismic loads. It provides the load combination to which design must conform. Finally, it addresses the acceptance for the design of ITS SSCs. This section must be used together with Section 7, which provides the methodology for determination of seismic forces on ITS SSCs. For evaluation of ITS SSCs, for the loads resulting from BDBGM, see Appendix B.

8.2 DESIGN CODES

The design methods and the design codes for the ITS structures are listed as follows:

ACI 349-01 [DIRS 181670]	Reinforced concrete design	Strength Design
ANSI/AISC N690-1994 [DIRS 158835]	Structural steel	Allowable Stress Design

These codes are applicable to both surface and subsurface structures. In addition to the these codes, the design of ITS structures will be based on the following standards:

ANSI/AISC 341-02 [DIRS 171789]	*Seismic Provisions for Structural Steel Buildings*
ASCE 4-98 [DIRS 159618]	*Seismic Analysis of Safety-Related Nuclear Structures and Commentary*
ASCE/SEI 43-05 [DIRS 173805]	*Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities*
IBC 2000 [DIRS 173525]	*International Building Code 2000*

8.3 LOADS AND LOAD COMBINATIONS

This section provides guidance on the load combinations that include earthquake loads. A full set of load combinations is provided in the PDC (BSC 2007 [DIRS 179641], Section 4.2.11.4.4).

8.3.1 Notations and Load Definitions

D = Dead load, including all permanently attached loads as well as crane dead weights, and loads due to weight of fluids

L = Live loads present during an earthquake, including the roof snow load or portion of the roof live load considered to be present during earthquakes. Normally, 25% of the design live load should be considered as existing during an earthquake (where justified, a higher percentage may be used) (IBC 2000 (ICC 2000 [DIRS 173525], Section 1617.5.1))

E = Earthquake load (based on D + 0.25 L as total weight)

H = Lateral earth pressure

T_o = Thermal loads during normal operating conditions. This term includes significant creep, shrinkage, differential settlements and similar self-relieving loads.

T_a = Thermal loads during abnormal conditions

S = Allowable stress per Allowable Stress Design method

U = Required strength per Strength Design method

Special loads, such as ventilation pressure differential and fluid pressure, are added when applicable.

8.3.2 General Notes on Load Combinations

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

- Other loads that could occur simultaneously with the earthquake loads (e.g., differential pressures) should be added to the load combinations.

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

- All load combinations should be checked for zero live load condition.

8.3.3 Load Combinations

The following load combinations, which include seismic loads, are applicable to both ITS surface and subsurface structures. The PDC (BSC 2007 [DIRS 179641], Section 4.2.11.4.4) provides the full set of load combinations; the following load combinations are anticipated to be governing, but this must be verified:

- Reinforced Concrete–Strength Design

  The basic seismic load combination, for concrete design is as follows:

$$U = D + 0.25L + H + (T_o \text{ or } T_a) + E \qquad \text{(Eq. 8-1)}$$

NOTE: In the above equation, 0.25L is used to be consistent with the live load percentage used in calculating earthquake loads. Conservatively, full live load may be used in lieu of 0.25L in this equation.

- Steel–Allowable Stress Design

  The basic seismic load combination for steel structures is as follows:

$$kS = D + 0.25L + (T_o \text{ or } T_a) + E \qquad \text{(Eq. 8-2)}$$

where k = stress increase factor to be applied to the allowable stresses in working stress design to be in line with the load combinations that include seismic loads. The following values of k are appropriate:

k =1.6 All stresses except for compression in members and shear in members and bolted connections

k =1.4 For compression in members and shear in members and bolted connections

In Equations 8-1 and 8-2, the design should be checked first with the real (operating and normal) loads to preclude reduction of design loads due to the thermal effects.

8.4 ACCEPTANCE CRITERIA

For DBGM, the design of ITS SSCs is based on elastic methods. The acceptance criterion for these SSCs is given by:

$$D/C \leq 1.0 \qquad \text{(Eq. 8-3)}$$

where D is the demand, as calculated by the right side of Equations 8-1 and 8-2, C is the capacity of the SSC as determined from applicable codes including the φ-factors (i.e., the left side of Equations 8-1 and 8-2).

Acceptance criteria given by Equation 8-3 may not be adequate to meet the BDBGM requirements as described in Appendix B. To ensure that the design does meet the BDBGM requirements, the following D/C ratio for SSCs designed to DBGM-2 should be satisfied:

$$D/C \leq 0.5 \text{ to } 0.6 \qquad \text{(Eq. 8-4)}$$

In Equation 8-4, the lower bound of 0.5 is derived from the ratio of the BDBGM/DBGM-2 as a reasonable approximation to meet the performance goals stated in 10 CFR 63.111 [DIRS 176544]. The 0.6 factor is similar to the margin required by the U.S. Nuclear Regulatory Commission between the ultimate capacity and the design basis earthquake as described in SECY-93-087 (Taylor 1993 [DIRS 165664]).

Acceptance criteria for BDBGM events are given in Appendix B.

9. SEISMIC ANALYSIS OF NON-ITS SSCs

9.1 GENERAL

Earthquake loads on non-ITS SSCs will be calculated per IBC 2000 (ICC 2000 [DIRS 173525], Section 1616), using the design spectra given in Section 6.4. Either equivalent static or dynamic analysis procedures may be used, depending on the complexity of the structure.

In Sections 9.2 through 9.4, analysis procedures using the equivalent lateral force are summarized. The quoted equations are from IBC 2000 (ICC 2000 [DIRS 173525]). These equations are to be used in conjunction with the design spectra given in Section 6.4.

The IBC 2000 (ICC 2000 [DIRS 173525], Section 1616) equations express the earthquake loads in terms of "strength level." Therefore, the calculated seismic forces are to be used in load combinations based on strength design. If allowable stress design methods are used, the seismic forces should be divided by the factor 1.4 to obtain the appropriate seismic loads. The resulting stresses should be compared with the allowable values without the one-third increase.

If dynamic analysis is used, Section 1618 of IBC 2000 (ICC 2000 [DIRS 173525]) should be followed, in conjunction with the design spectra for non-ITS structures.

9.2 SEISMIC ANALYSIS OF NON-ITS STRUCTURES

- The non-ITS structures at the YMP are in Seismic Design Categories C and D as defined in IBC 2000 (ICC 2000 [DIRS 173525], Section 1616.3) and as shown in Table 5-4.
- The minimum design seismic loads will be determined in accordance with Section 1617.1 of IBC 2000 (ICC 2000 [DIRS 173525]). The minimum load calculation takes into account the "redundancy" factor, which should be determined following Section 1617.2 of IBC 2000.

The base shear will be calculated (ICC 2000 [DIRS 173525], Section 1617.4) as follows:

$$V = C_s W \quad \text{(Eq. 9-1)}$$

$$C_s = (S_{DS}/R) I_E \quad \text{(Eq. 9-2)}$$

with limits on C_s as follows:

$$0.044 S_{DS} I_E \leq C_s \leq (S_{D1}/RT) I_E \quad \text{(Eq. 9-3)}$$

where

C_s = seismic response coefficient
W = total seismic dead load of the structure
I_E = occupancy importance factor
S_{DS} = design spectral acceleration at short period

S_{D1} = design spectral acceleration at one-second period

S_1 = maximum considered earthquake spectral one-second-period response acceleration from IBC 2000 (ICC 2000 [DIRS 173525], Figure 1615(3), Section 1615)

R = response modification factor from IBC 2000 (ICC 2000 [DIRS 173525])

T = the fundamental period of structure.

In Equations 9-1 to 9-3, S_{DS} and S_{D1} account for the seismic hazard, which is discussed in Section 6.4. The importance factor corresponds to the seismic use groups and reflects the higher seismic design forces for the more important structures (Section 5). The C_s factor accounts for the amplification of the ground motion through the structure as a function of the fundamental period of the structure (T) and including the soil effects. Finally, the R factor reflects the energy dissipation characteristics of the structure; higher values being allowed for structures with greater demonstrated energy dissipation capacity while remaining within the acceptable limits of deformation.

9.2.1 Site-Specific Design Parameters

The design shall be based on the site location being at the North Portal, Latitude N 36.85°, Longitude W 116.43 (BSC 2007 [DIRS 179641], Section 6.1.10.2.1), and on site-specific design parameters. The site-specific design ground motion has been derived in accordance with IBC 2000 (ICC 2000 [DIRS 173525], Section 1615) from site investigations. The value of S_1 is from IBC 2000 and the values of S_{DS} and S_{D1} are from Figure 6-8 of this document:

- S_{DS} = 0.85
- S_{D1} = 0.35
- S_1 = 0.22.

9.2.2 Earthquake Loads Criteria Selection

The criteria selection for earthquake loads shall be based on IBC 2000 (ICC 2000 [DIRS 173525], Section 1616.1). The seismic design category shall be based on IBC 2000, Tables 1616.3 (1) and 1616.3 (2) for short period response acceleration, S_{DS}, and one-second-period response acceleration, S_{D1}, respectively, using the values of S_{DS} and S_{D1} defined in Section 9.2.1. The seismic use group shall be the category for the nature of occupancy based on IBC 2000, Table 1604.5. The importance factor, I_E, shall correspond to the nature of occupancy shown in IBC 2000, Table 1604.5. Non-ITS SSCs currently identified are listed in Table 5-4.

9.3 SEISMIC ANALYSIS OF NON-ITS, NON-BUILDING STRUCTURES

Non-building structures are structures that generally do not have the features of buildings, but carry gravity loads to the ground and resist earthquake loads. In accordance with IBC 2000 (ICC 2000 [DIRS 173525], Section 1622), the following considerations apply to these structures:

- Non-building structures may be analyzed using the equivalent lateral force procedure or dynamic analysis.

- The base shear for non-building structures will be determined as in Section 9.2, with the exception that the minimum seismic response coefficient must be at least equal to:

$$C_s = 0.14 S_{DS} I \qquad \text{(Eq. 9-4)}$$

where I is the importance factor for the non-building structure as given in Table 1622.2.5(2) of IBC 2000 (ICC 2000 [DIRS 173525]).

When dynamic analysis methods are used, lumped-mass models or finite element models will be utilized in conjunction with the design spectra for non-ITS structures.

9.4 SEISMIC ANALYSIS OF NON-ITS SYSTEMS AND COMPONENTS

Mechanical and electrical equipment not important to safety (non-ITS) shall be designed using the following criteria for seismic loads:

9.4.1 Mechanical and Electrical Components Supported by Non-ITS Buildings

Seismic loads for the mechanical and electrical equipment supported by the Non-ITS buildings shall be determined using IBC 2000 (ICC 2000 [DIRS 173525], Section 1621). The S_{DS} value in IBC 2000 equations 16-67, 16-68, and 16-69 shall be taken as 0.91.

9.4.2 Mechanical and Electrical Components Supported at Grade

Seismic loads for non-ITS components supported directly on the ground shall be determined using IBC 2000 (ICC 2000 [DIRS 173525], Sections 1622 and 1517.4). The S_{DS}, S_{D1}, and S_1 values in the equations shown in these sections of IBC 2000 are as follows:

- S_{DS} = 0.85
- S_{D1} = 0.35
- S1 = 0.22.

9.4.3 Mechanical and Electrical Components Supported Either by Non-ITS Buildings or at Grade

The following criteria shall be applied to both categories of components covered under Sections 9.4.1 and 9.4.2 above:

- Seismic loads obtained from the referenced equations are intended for use with the strength design methods. If the allowable stress design methods are being used, then the seismic forces determined from these equations shall be divided by the factor 1.4.

- The calculated lateral force FP shall be distributed in proportion to the mass distribution of the equipment.

- The anchorage for the component shall be designed for the total lateral loads, including the overturning effects.

- Where approved national standard or approved physical test data are available, such data would be acceptable if they comply with IBC 2000 (ICC 2000 [DIRS 173525]) requirements.

10. SEISMIC DESIGN OF NON-ITS SSCs

10.1 GENERAL

This section details the methodology to be used for the design of non-ITS SSCs for load combinations that include seismic loads. It lists the acceptable industry codes to be used in the design. It identifies the loads that should be considered in conjunction with the seismic loads. It provides the load combinations to which design must conform. Finally, it addresses the acceptance criteria for design of such SSCs.

This section must be used together with Section 9, which provides the methodology for determination of earthquake loads on non-ITS SSCs.

10.2 DESIGN CODES

The design codes and design methods to be used for the non-ITS structures are as follows:

ACI 318-02/318R-02 [DIRS 158832]	Reinforced concrete design	Strength Design
AISC 1997 [DIRS 107063] AISC 1995 [DIRS 146097]	Structural steel	Allowable Stress Design Load and Resistance Factor Design
ACI 530-02 [DIRS 158925]	Masonry design	Allowable Stress Design
IBC 2000 (ICC 2000 [DIRS 182945])	Design of non-ITS Structures	

These codes and design methods are applicable to both surface and subsurface non-ITS structures.

10.3 LOADS AND LOAD COMBINATIONS

10.3.1 Notations and Load Definitions

The load definitions are similar to those for ITS SSCs except that the non-ITS structures are not normally designed for thermal effects and pipe reactions. However, thermal and other loads that may have a significant effect on the behavior of the non-ITS structures should be included in the load combinations.

D = Dead load, including all permanently attached loads as well as crane dead weights, and loads due to weight of fluids.

L = Live loads present during an earthquake, including the roof snow load or portion of the roof live load is considered as present during earthquakes. Normally, 25% of the design live load should be considered as existing during an earthquake (where necessary, a higher percentage may need to be considered) (IBC 2000 [DIRS 173525], Section 1617.5.1)).

E = Earthquake load reduced by the appropriate response reduction factor (R)

H = Lateral earth pressure

S = Allowable stress per Allowable Stress Design method

U = Required strength per Strength Design method

10.3.2 Load Combinations for Non-ITS Structures

The load combinations involving seismic loads that will be used in the design of non-ITS structures are:

- Reinforced Concrete-Strength Design:

$$U = 1.2D + 0.25L + 1.6H + E \quad \text{(Eq. 10-1)}$$

$$U = 0.9D + E \quad \text{(Eq. 10-2)}$$

- Steel and Masonry-Allowable Stress Design:

$$S = D + 0.25L + E/1.4 \quad \text{(Eq. 10-3)}$$

$$S = 0.9D + E/1.4 \quad \text{(Eq. 10-4)}$$

These load combinations are anticipated to be governing, but this must be verified; the full set of load combinations is provided in the PDC (BSC 2007 [DIRS 179641], Section 4.2.11.5).

Equations 10-3 and 10-4 are to used in the working stress design methods for the design of steel and masonry structures.

Alternatively, steel structures may be designed using the Load and Resistance Factor Design method and the masonry structure may be designed using the strength method. In such cases, Equations 10-1 and 10-2 must be used instead of Equations 10-3 and 10-4.

10.4 ACCEPTANCE CRITERIA

The analysis and design of non-ITS SSCs are based on elastic methods. However, the response modification factors are greater than unity (as given in IBC 2000 (ICC 2000 [DIRS 173525])) to account for inelastic response under the design ground motions. Nonetheless, the acceptance criterion for these SSCs is given by:

$$D/C \leq 1.0 \quad \text{(Eq. 10-5)}$$

where D is the demand, as calculated by the right side of Equations 10-1 to 10-4, C is the capacity of the SSC as determined using the applicable codes.

The design intent for non-ITS SSCs is to ensure life safety. Therefore, inelastic behavior is expected under design basis seismic loads. These SSCs are expected not to collapse.

11. MISCELLANEOUS SEISMIC ANALYSIS AND DESIGN ISSUES

This section addresses miscellaneous analysis and design issues that will be encountered during the execution of the project. It covers evaluation of stability of structures against sliding and overturning, story drift, building separation, criteria for the design of anchorage, and detailing of structures to ensure ductile behavior.

11.1 STABILITY REQUIREMENTS

11.1.1 Surface Structures and Components

The following minimum factors of safety shall be provided for sliding and overturning:

Load Combination	Sliding	Overturning	Reference
D + H + E	1.1	1.1	NRC 1987 [DIRS 138431], Section 3.8.5

Sliding–The first step in sliding evaluations is to calculate a static factor of safety against sliding. If the static sliding factor of safety is not achieved, the following approaches should be used to assess the potential for relative displacements and, if necessary, to alleviate the consequences of these relative displacements and, thus, to ensure the integrity of the SSCs contained therein:

- Calculate the predicted magnitude of motion of a structure (i.e., sliding and rocking) relative to the ground, using approximate methods (see ASCE/SEI 43-05 [DIRS 173805], Section 7.1).
- Design the connections of systems that enter and exit the structure such that the predicted relative motions will not adversely affect these connections, using a factor of safety of 2.
- If deemed necessary, an SSC may be analyzed using nonlinear time-history methods to confirm the results of the simplified approaches or to calculate more realistic displacements. These nonlinear analyses will be carried out using five time-histories with different phasing, all matching the acceleration DRS, and the average displacement times a factor of safety of 2 will be taken as the design relative motion. Vertical ground motions will be accounted for in these calculations.
- If the resulting displacements exceed the acceptance criteria, revise the layout such that there is no interaction between adjacent structures.

Overturning–In the case of overturning, the minimum factor of safety listed above should be provided for both ITS structures and non-ITS structures. Overturning stability may be demonstrated by static calculations or using the reserve energy approach given in ASCE/SEI 43-05 [DIRS 173805], Section 7.2.

11.1.2 Subsurface Structures and Components

Some underground SSCs are anchored to the main and emplacement drift walls. Therefore, the stability (sliding and overturning) is not a consideration in the design. However, in other cases an analysis may be necessary for calculating the restoring forces required for stability. Evaluation of both static sliding and overturning may be performed using the non-ITS methods. When sliding and overturning evaluations are performed, the following factors of safety should be used with the DBGM.

Load Combination	Sliding	Overturning	Reference
D + H + E	1.1	1.1	NRC 1987 [DIRS 138431], Section 3.8.5

11.2 STORY DRIFTS

Story drifts should be calculated using the deflections from elastic analysis of the structure. Story drift is the difference between the lateral displacements at the top and bottom of the story under consideration. In the calculation for story drifts, both translational and torsional deflections should be considered.

Story drifts for ITS structures and for non-ITS structures should not exceed the limits given in ASCE/SEI 43-05 [DIRS 173805], Section 5.2.3, and IBC 2000 (ICC 2000 [DIRS 173525]), respectively.

In calculating the story drifts for the non-ITS structures, the deflection under lateral loads should be determined without dividing the earthquake forces by the factor 1.4 (i.e., at the strength design level). Furthermore, the anticipated inelastic response should be taken into consideration through the application of the deflection amplification factor given in Section 1617.4.6 of IBC 2000 (ICC 2000 [DIRS 173525]).

11.3 INTERACTION OF NON-ITS WITH ITS SSCs (Seismic 2/1 Issue)

Based on the provisions of Section 3.7.2 of NUREG-0800 (NRC 1987 [DIRS 138431]) the design of a non-ITS structure adjacent to an ITS structure must meet one of the following requirements:

- The collapse of the non-ITS SSCs will not cause it to strike an ITS structure or component.

- The collapse of the non-ITS SSCs will not impair the integrity of an ITS structure or component.

- The non-ITS SSCs will be analyzed and designed to prevent its failure under the DBGM conditions.

11.4 BUILDING SEPARATION

Structures should be separated from each other to preclude seismic interaction. The minimum separation between adjacent structures should be as follows:

- For ITS structures: $\Delta = 2\sqrt{(\Delta_1)^2 + (\Delta_2)^2}$

- For non-ITS structures: $\Delta = \sqrt{(\Delta_1)^2 + (\Delta_2)^2}$

where Δ_1 and Δ_2 are the maximum computed elastic displacements along the same axis for the adjacent structures.

11.5 ANCHORAGE

Anchor rods (bolts) and concrete expansion anchors in all buildings must comply with the provisions of one of two codes, depending on the categorization of the SSC:

ITS structures: Appendix B of ACI 349-01 [DIRS 181670]
Non-ITS structures: Appendix D of ACI 318-02/318R-02 [DIRS 158832].

The following general requirements are applicable to all anchorage:

- In the design of anchors, all possible failure modes should be considered.

- Only ductile failure modes are permitted in ITS structures (i.e., failure mode should be by yielding of the steel element and not concrete failure or slip failure). This requirement can be readily satisfied by using cast-in-place anchors whenever possible. When expansion anchors must be used, only those types that exhibit ductile behavior should be specified.

- The allowable design capacities of concrete expansion anchors should be based on the manufacturers' recommendations, and should include a minimum factor of safety of 4 applied to the mean ultimate capacity. The manufacturers' test data should be current and approved by an independent approval authority.

12. USE OF COMPUTER PROGRAMS

Several computer programs will be used in the course of the analysis and design of SSCs for the YMP. These computer programs have been verified and validated by the project. These programs include:

- GT-STRUDL (V.26. 2003. Windows® 2000. STN: 10829-26-00 [DIRS 166081]) (Georgia Tech Structural Design Language)—a computer aided structural engineering software system for assisting an engineer in the structural analysis and design process. It is a state-of-the-art, efficient, reliable, and fully integrated general-purpose structural information system capable of supplying an engineer with accurate and complete technical data for design decision-making. However, GT-STRUDL is currently not used on the YMP.

- SASSI2000 (V. 3.1. 2007. WINDOWS® XP/X64. STN: 10825-3.1-00 [DIRS 182945]) (A System for Analysis of Soil-Structure Interaction)—a linear elastic finite element substructuring program that can solve two- and three-dimensional SSI problems with embedded flexible foundations. The program is formulated in the frequency domain using the complex response method. Conversions between the time domain and the frequency domain are performed by the Fast Fourier Transform technique. SASSI2000 is a state-of-the-art industry program for dynamic SSI analysis of critical structures.

- SAP2000 (V. 9.1.4. 2005. WINDOWS® 2000. STN: 11198-9.1.4-00 [DIRS 178238]) and WINDOWS XP. STN: 11198-9.1.4-01—a structural engineering, finite element software program that allows model creation and modification, execution of static, dynamic, linear and non-linear analyses, design optimization, and results review. The program includes graphical three-dimensional model generation using plan, elevation, and developed views. Steel member design is done based on American Institute of Steel Construction design code. Animation of deformed shapes, mode shapes, stress contours, and time history results can be displayed.

- YMP OPTCON (STN:11244-1.0-00). OPTCON Program Module (V. 1.0. 2006. WINDOWS 2000. STN: 11208-1.0-00 [DIRS 178237]) was originally installed on the project to have interface with SASSI [DIRS 182945] and SAP2000 [DIRS 178238]. Currently YMP OPTCON has been developed to interface with SAP2000 [DIRS 178238] only. The purpose of the OPTCON program is to design reinforced concrete structures per ACI 349-01 [DIRS 181670] requirements. On the YMP, it is used to design shear wall structures with the following features:

  - The program imports SAP2000 [DIRS 178238] finite element analysis results under non-seismic loads for design.

- It imports SAP2000 [DIRS 178238] seismic forces obtained from a static equivalent analysis (i.e., using “bubble accelerations”) or from a response spectrum analysis (RSA)

- It combines the seismic and non-seismic loads in accordance with the project criteria and taking into account the spatial component of the seismic loads.

- It designs the shear walls and diaphragms using the forces and moments based on “section cuts” or “element stresses.”

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13.4 SOFTWARE CODES

166081 GT STRUDL V.26. 2003. Windows 2000. STN: 10829-26-00.

178237 OPTCON Program Module for SASSI2000 V. 1.0.2006. WINDOWS 2000. STN: 11208-1.0-00

178238 SAP2000 V. 9.1.4. 2005. WINDOWS XP. STN: 11198-9.1.4-00.

182945 SASSI2000 V. 3.1. 2007. WINDOWS XP. STN: 10825-3.1-00.

APPENDIX A
COMPUTATION OF COMBINED EARTHQUAKE-INDUCED STRUCTURAL RESPONSE

A1 SCOPE

This appendix describes a step-by-step procedure for computation of combined earthquake-induced structural response (R). The R is combined with response due to other loads to obtain the design forces in the elements of the structure.

A2 NOTATIONS

X = North–South direction
Y = Vertical, Positive Up
Z = East–West direction

X_x = Accelerations in X-direction due to earthquake in +X-direction
X_y = Accelerations in X-direction due to earthquake in +Y-direction
X_z = Accelerations in X-direction due to earthquake in +Z-direction

Y_x = Accelerations in Y-direction due to earthquake in +X-direction
Y_y = Accelerations in Y-direction due to earthquake in +Y-direction
Y_z = Accelerations in Y-direction due to earthquake in +Z-direction

Z_x = Accelerations in Z-direction due to earthquake in +X-direction
Z_y = Accelerations in Z-direction due to earthquake in +Y-direction
Z_z = Accelerations in Z-direction due to earthquake in +Z-direction

H_x^+ = Dynamic lateral soil pressure applied in −X direction due to excitation in +X-direction
H_x^- = Dynamic lateral soil pressure applied in +X direction due to excitation in −X-direction
H_z^+ = Dynamic lateral soil pressure applied in −Z direction due to excitation in +Z-direction
H_z^- = Dynamic lateral soil pressure applied in +Z direction due to excitation in −Z-direction

A3 PROCEDURE

A3.1 BASIC LOADS

X-Excitation: X_x, Y_x, Z_x (North–South, Positive North)
Y-Excitation: X_y, Y_y, Z_y (Vertical, Positive Up)
Z-Excitation: X_z, Y_z, Z_z (East–West, Positive East)

NOTE: For typical shear wall buildings, $X_z = Z_y = 0$.

A3.2 COMPLETE SEISMIC LOAD COMBINATIONS WITHOUT CONSIDERING DYNAMIC SOIL PRESSURE

$R_x = (\pm X_x \pm Y_x \pm Z_x) + 0.4(\pm X_y \pm Y_y \pm Z_y) + 0.4(\pm X_z \pm Y_z \pm Z_z)$ (Eq. A-1)
(512 combinations)

$R_y = 0.4(\pm X_x \pm Y_x \pm Z_x) + (\pm X_y \pm Y_y \pm Z_y) + 0.4(\pm X_z \pm Y_z \pm Z_z)$ (Eq. A-2)
(512 combinations)

$R_z = 0.4(\pm X_x \pm Y_x \pm Z_x) + 0.4(\pm X_y \pm Y_y \pm Z_y) + (\pm X_z \pm Y_z \pm Z_z)$ (Eq. A-3)
(512 combinations)

or

$R_x = (\pm X_x \pm 0.4X_y \pm 0.4X_z) + (\pm Y_x \pm 0.4Y_y \pm 0.4Y_z) + (\pm Z_x \pm 0.4Z_y \pm 0.4Z_z)$ (Eq. A-4)

$R_y = (\pm 0.4X_x \pm X_y \pm 0.4X_z) + (\pm 0.4Y_x \pm Y_y \pm 0.4Y_z) + (\pm 0.4Z_x \pm Z_y \pm 0.4Z_z)$ (Eq. A-5)

$R_z = (\pm 0.4X_x \pm 0.4X_y \pm X_z) + (\pm 0.4Y_x \pm 0.4Y_y \pm Y_z) + (\pm 0.4Z_x \pm 0.4Z_y \pm Z_z)$ (Eq. A-6)

NOTE: The total number of complete permutations is 1,536.

A3.3 THE CRITICAL SEISMIC LOAD COMBINATIONS FOR STRUCTURAL DESIGN

$R_x = \pm (X_x + 0.4X_y + 0.4X_z) \pm (Y_x + 0.4Y_y + 0.4Y_z) \pm (Z_x + 0.4Z_y + 0.4Z_z)$ (Eq. A-7)
(eight combinations)

$R_y = \pm (0.4X_x + X_y + 0.4X_z) \pm (0.4Y_x + Y_y + 0.4Y_z) \pm (0.4Z_x + Z_y + 0.4Z_z)$ (Eq. A-8)
(eight combinations)

$R_z = \pm (0.4X_x + 0.4X_y + X_z) \pm (0.4Y_x + 0.4Y_y + Y_z) \pm (0.4Z_x + 0.4Z_y + Z_z)$ (Eq. A-9)
(eight combinations)

These 24 load combinations are critical because nodal accelerations are maximum using the factor method and the accelerations are considered additive (e.g., $X_x + 0.4X_y + 0.4X_z$). This critical set of seismic load combinations can be used to combine with static loads for structural design.

For typical shear wall structures, the horizontal accelerations due to vertical seismic input are zero and, therefore, Equations A-7 to A-9 are reduced to:

$R_x = \pm (X_x + 0.4X_z) \pm (Y_x + 0.4Y_y + 0.4Y_z) \pm (Z_x + 0.4Z_z)$ (Eq. A-10)
(eight combinations)

$R_y = \pm (0.4X_x + 0.4X_z) \pm (0.4Y_x + Y_y + 0.4Y_z) \pm (0.4Z_x + 0.4Z_z)$ (Eq. A-11)
(eight combinations)

$R_z = \pm (0.4X_x + X_z) \pm (0.4Y_x + 0.4Y_y + Y_z) \pm (0.4Z_x + Z_z)$ (Eq. A-12)
(eight combinations).

A3.4 THE CRITICAL SEISMIC COMBINATIONS CONSIDERING DYNAMIC SOIL PRESSURE

$+ R_x = (+ X_x + H_x^+ + 0.4X_y + 0.4X_z) \pm (Y_x + 0.4Y_y + 0.4Y_z) \pm (Z_x + 0.4Z_y + 0.4Z_z)$ (Eq. A-13)
(four combinations)

$- R_x = (- X_x + H_x^- - 0.4X_y - 0.4X_z) \pm (Y_x + 0.4Y_y + 0.4Y_z) \pm (Z_x + 0.4Z_y + 0.4Z_z)$ (Eq. A-14)
(four combinations)

$+ R_y = \pm (0.4X_x + X_y + 0.4 X_z) + (0.4Y_x + Y_y + 0.4Y_z) \pm (0.4Z_x + Z_y + 0.4Z_z)$ (Eq. A-15)
(four combinations)

$- R_y = \pm (0.4X_x + X_y + 0.4 X_z) + (- 0.4Y_x - Y_y - 0.4Y_z) \pm (0.4Z_x + Z_y + 0.4Z_z)$ (Eq. A-16)
(four combinations)

$+ R_z = \pm (0.4X_x + 0.4X_y + X_z) \pm (0.4Y_x + 0.4Y_y + Y_z) + (0.4Z_x + 0.4Z_y + Z_z + H_z^+)$ (Eq. A-17)
(four combinations)

$- R_z = \pm (0.4X_x + 0.4X_y + X_z) \pm (0.4Y_x + 0.4Y_y + Y_z) + (-0.4Z_x - 0.4Z_y - Z_z + H_z^-)$ (Eq. A-18)
(four combinations)

NOTE: The total number of critical seismic load combinations is 24.

For typical shear wall buildings, the horizontal accelerations due to vertical seismic input are zero and, therefore, Equations A-13 to A-18 are reduced to:

$+ R_x = (+ X_x + H_x^+ + 0.4X_z) \pm (Y_x + 0.4Y_y + 0.4Y_z) \pm (Z_x + 0.4Z_z)$ (Eq. A-19)
(four combinations)

$- R_x = (- X_x + H_x^- - 0.4X_z) \pm (Y_x + 0.4Y_y + 0.4Y_z) \pm (Z_x + 0.4Z_z)$ (Eq. A-20)
(four combinations)

$+ R_y = \pm (0.4X_x + 0.4X_z) + (0.4Y_x + Y_y + 0.4 Y_z) \pm (0.4Z_x + 0.4Z_z)$ (Eq. A-21)
(four combinations)

$- R_y = \pm (0.4X_x + 0.4X_z) + (-0.4Y_x - Y_y - 0.4 Y_z) \pm (0.4Z_x + 0.4Z_z)$ (Eq. A-22)
(four combinations)

$+ R_z = \pm (0.4X_x + X_z) \pm (0.4Y_x + 0.4Y_y + Y_z) + (0.4Z_x + Z_z + H_z^+)$ (Eq. A-23)
(four combinations)

$- R_z = \pm (0.4X_x + X_z) \pm (0.4Y_x + 0.4Y_y + Y_z) + (-0.4Z_x - Z_z + H_z^-)$ (Eq. A-24)
(four combinations).

A4 COMBINED STATIC LOAD WITH SEISMIC LOADS

Static Load ± R_x (8 combinations for each static load with X-excitation)
Static Load + R_y (8 combinations for each static load with Y-excitation)
Static Load ± R_z (8 combinations for each static load with Z-excitation)

APPENDIX B
EVALUATIONS FOR BDBGM

The ITS structures analyzed and designed in accordance with Sections 7 and 8 need to be evaluated for BDBGM. The goals of these evaluations are: (1) to demonstrate that there is margin beyond the rare earthquakes with a 10,000-year return period, and (2) to establish approximate fragility curves for the structuresfor a limited probabilistic risk assessment using Conservative Deterministic Failure Margin (CDFM) methodology described in Section B3.1 below. However, fragility for equipment and components will be based on the median-centered fragility analysis method as noted in Section B3.4.

The evaluations for BDBGM will be carried out using the mean-centered seismic hazard curves. A seismic hazard curve shows the annual probability of exceedance versus a defined ground motion parameter. A typical curve is shown in Figure B-1. The ground motion parameter in this figure is the peak ground acceleration. Seismic hazard curves at 5 and 10 Hz (or the average of 5 and 10 Hz) will result in more realistic evaluations and may be used on this project. The seismic analyses and the evaluations performed for BDBGMs are described in Sections B1 through B3.4.

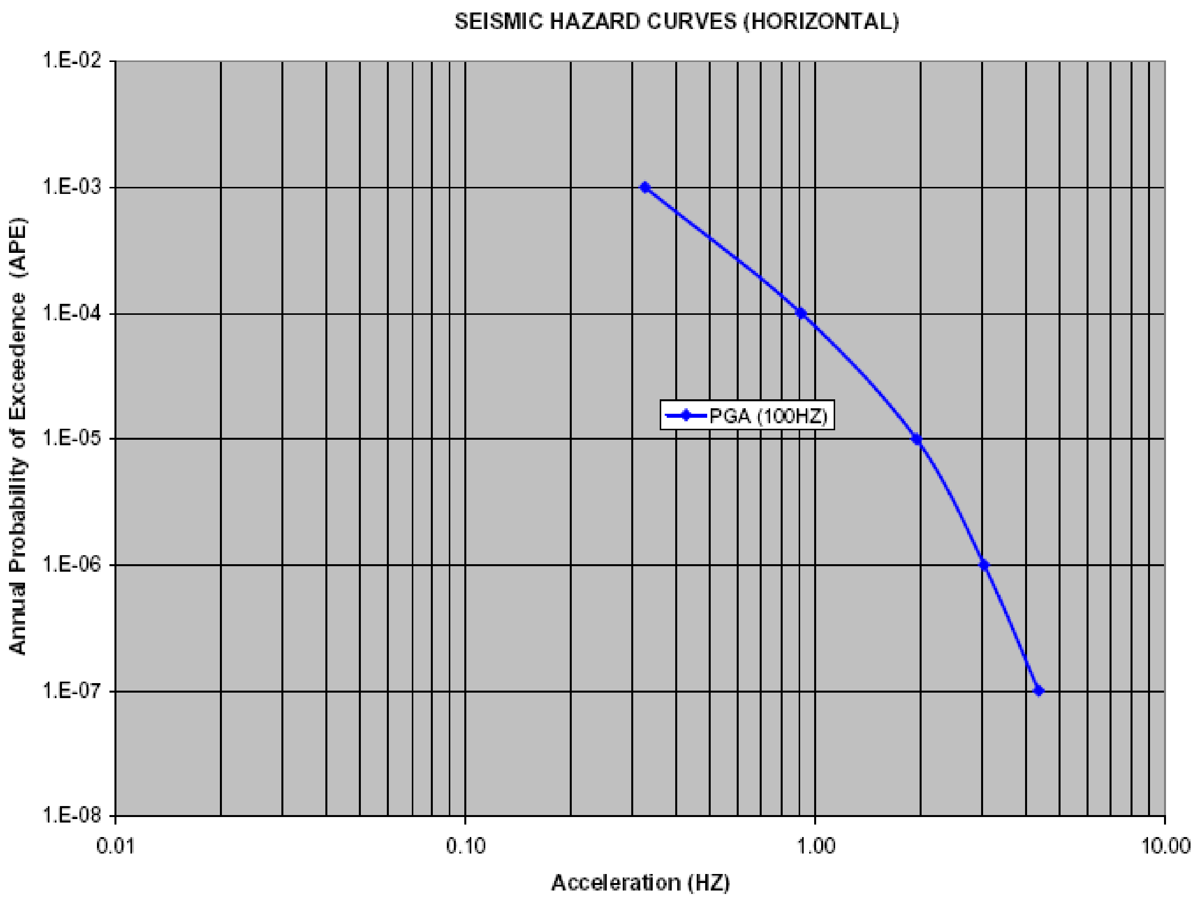


Figure B-1. Example Seismic Hazard Curve for YMP

B1 SEISMIC ANALYSIS FOR BDBGM

The first step in performing evaluations for BDBGM is to analyze structures for a reference ground motion. On the YMP, the reference ground motion is defined as the mean-centered, 10,000-year return period earthquake, also termed BDBGM. All ITS structures designed for DBGM-2 will be analyzed using the BDBGM spectra and the compatible time-histories. The basic principles in these analyses are as follows:

- The BDBGM input spectra will be the mean-centered spectra (based on seismic hazard curve and considering site-specific soil properties) at the site for 10^{-4} exceedance probability. Time-histories will be compatible with the mean-centered spectra.

- A linear elastic seismic analysis will be performed using finite element models and SASSI2000 [DIRS 182945] computer code.

- The structural damping will be as shown in Table 7-1.

The computed seismic demands will be used to calculate the D/C ratios (D_{BDBGM}/C_C) using the code capacities (including code-specified strength reduction factors ϕ) and the F_μ defined in ASCE/SEI 43-05 [DIRS 173805]. In equation form,

$$(D_{BDBGM} / C_C) \leq F_\mu, \qquad \text{(Eq. B-1)}$$

where

D_{BDBGM} = seismic demand computed for the BDBGM input in accordance with the requirements of ASCE 4-98 [DIRS 159618], Section 3.1.1.2

C_C = capacity computed using code capacity acceptance criteria (including code specified strength reduction factors ϕ

F_μ, = inelastic energy dissipation factor (termed energy absorption factor in ASCE/SEI 43-05 [DIRS 173805].

In the application of Equation B-1, F_μ values for Limit State A (large permanent distortion, short of collapse) or Limit State C (limited permanent distortion, minimal damage) will be used, whichever is applicable. The limit states are defined per ASCE/SEI 43-05 [DIRS 173805], Table 1-4. (Note that, in general, Limit State A is used; Limit State C is applicable where confinement is necessary to maintain a negative pressure.)

The purpose of the D/C ratio calculation is to obtain a sense of seismic resistance capability of the structure. If the calculated D/C ratios are greater than the corresponding F_μ, additional analyses may be performed to determine a more realistic capacity. These analyses may include one or more of the following refinements:

- **Capacities Based on Test Data**. Evaluations may be carried out using more realistic (yet still conservative) strength capacities. The realistic capacities will be established if sufficient test data exist. Any strength increase beyond Code capacities will be established at about the 98% exceedance probability capacity (2% non-exceedance probability capacity, or approximately 2 standard deviations below the mean capacity) in order to have high confidence in these strength capacities. Additional details on criteria to be used in establishing high confidence strength capacities are provided in: *A Methodology for Assessment of Nuclear Power Plant Seismic Margin (Revision 1)* (EPRI 1991 [DIRS 161330], Section 2 and Appendix L); *Basis for Seismic Provisions of DOE-STD-1020* (Kennedy and Short 1994 [DIRS 161326], Section 4.2); DOE-STD-1020-94 [DIRS 161324], Section C.5; and ASCE /SEI 43-05 [DIRS 173805].

- **Pushover Analysis**. A pushover analysis is a nonlinear analysis where incremental lateral loads are applied, the state of stress in the members is monitored and the stiffness matrix is adjusted prior to application of the next incremental load. The purpose of this analysis is to establish an approximate load-displacement diagram for the structures well into the nonlinear range. In this nonlinear analysis, well-established constitutive relations must be used in each load increment. The result is expressed as a load-displacement diagram. Once this diagram is established, a more realistic F_{μ} (ductility factor) can be established, which will reflect the overall nonlinear behavior of the structure or the component. If the calculated ductility factor is greater than the ratio obtained from Equation B-1, the structure is adequate to resist the BDBGM loads. Otherwise, changes in the layout may be necessary. The load-displacement diagram thus calculated can be used to obtain the fragility curve that is necessary for the probabilistic seismic risk assessment.

- **Nonlinear Dynamic Analysis**. In this alternative, nonlinear dynamic analyses may be performed using a simplified computer model of the structure or component. As in the pushover analysis, the previously defined high-confidence strength capacities will be used in the non-linear evaluations. These analyses will be carried out using five time-histories with different phasing, all matching the acceleration response spectrum. Non-linear distortions (defined in terms of total story drifts per story for concrete shear wall and steel braced frame structures and in terms of inelastic hinge rotations and total story drifts for moment frame structures, and similar definitions for components) will be evaluated. In addition, the system ductility will be established based on the load-displacement curves and compared with the required ductility calculated in Equation B-1. If the system ductility is greater than the required ductility, the structure is adequate as designed. Otherwise, changes in the layout may be necessary. (ASCE/SEI 4-98 [DIRS 159618], Sections 3.2.2.3 and C3.2.3.2)

In summary, the analyses using the BDBGM input and evaluation of the results will be indicative of the robustness of the design.

B2 SEISMIC MARGIN ASSESSMENT

Subsequent to the analyses for BDBGM, the DOE intends to demonstrate seismic margins of the ITS structures designed to DBGM-2 against the BDBGM. The approach used is similar to a seismic margin assessment, which has been used for nuclear power plants. Specifically, the assessment will demonstrate that ITS structures have High Confidence Low Probability of

Failure (HCLPF) capacity that exceeds the designated review level earthquake, termed BDBGM event for Yucca Mountain facilities (BSC 2007 [DIRS 181572])

Seismic margins assessment is based on a comparison of a conservative estimate of the *capacity* of the facility to maintain safety functions with the *demand* imposed by "review level earthquake" ground motions. HCLPF capacities (see Section B3) will be estimated for ITS structures using the Conservative Deterministic Failure Margin (CDFM) approach, following the guidance given in *A Methodology for Assessment of Nuclear Power Plant Seismic Margin (Revision 1)* (EPRI 1991 [DIRS 161330]). The HCLPF capacity is defined as the ground motion level at which there is a mean conditional probability of unacceptable performance of 0.01 or less. ASCE 4-98 [DIRS 159618], Appendix A, provides a discussion of the applicability of the seismic margin analysis approach to demonstrate seismic safety of plants designed using NUREG-0800 (NRC 1987 [DIRS 138431]) codes and standards. As discussed in Section B3, the HCLPF capacity estimates will also be used to develop fragility curves for the probabilistic seismic analyses for the compliance demonstration.

The seismic margins evaluation will ensure that the HCLPF capacity of individual ITS structures is greater than the demand imposed on the structures by the BDBGM event. It is recommended that approximately 10% margin be provided as per Equation B-2 below:

$$(C_{HCLPF}/D_{BDBGM}) \geq 1.10 \qquad \text{(Eq. B-2)}$$

Satisfaction of the above equation will ensure that adequate seismic design margin will exist for ITS structures, such that they will maintain their defined functions credited in the preclosure safety analysis to prevent or mitigate dose consequences. Although the seismic margins assessment is not a demonstration of compliance with the preclosure performance objectives in 10 CFR 63.111 [DIRS 176544], its widespread precedent in seismic safety evaluation for nuclear power facilities will ensure the adequacy of the seismic design bases and the design codes and design procedures.

B3 FRAGILITIES FOR LIMITED PROBABILISTIC SEISMIC ANALYSES

For compliance with the requirements of 10 CFR 63.111(b)(2) [DIRS 176544], limited probabilistic seismic analyses will be performed, considering the seismic hazard and the behavior of ITS structures under seismic loads. This information is currently being prepared in a preclosure seismic design document that will be issued in the future. These evaluations require definition of fragility curves for the individual components or event sequences.

This subsection defines the approach to be used in fragility calculations. Prior to the calculations, the permissible limit states will be defined per ASCE/SEI 43-05 [DIRS 173805], Table 1-4. A fragility curve shows the probability of unacceptable seismic performance as a function of a ground motion parameter such as peak ground acceleration or dominant spectral acceleration. Seismic fragilities will be developed as a function of the limit states and ground motions using the methods described in Sections B3.1 to B3.5.

B3.1 DEVELOPMENT OF FRAGILITY CURVES

Seismic fragility curves will be developed using the 1% probability of unacceptable performance, $C_{1\%}$, and the composite logarithmic standard deviation, β (EPRI 1994 [DIRS 161329], Section 4; Kennedy 2001 [DIRS 155940], Sections 5 and 6). Other methods, such as the Fragility Analysis methods outlined in *Methodology for Developing Seismic Fragilities* (EPRI 1994 [DIRS 161329], Section 4) may be used on a case-by-case basis.

The 1% probability of unacceptable performance seismic capacity will be approximated by the deterministically computed CDFM methodology (EPRI 1991 [DIRS 161330], pp. 2-45 to 2-56; ASCE/SEI 43-05 [DIRS 173805], Section C1.3 for 1% conditional probability of failure [unacceptable behavior]). The capacity obtained from the CDFM method is called C_{CDFM}. Alternatively, the capacity evaluation methodology (DOE-STD-1020-94 [DIRS 161324], Section C.5; DOE-STD-1020-2002 [DIRS 159258]; Kennedy and Short 1994 [DIRS 161326], Section 4.2) can be used to determine the C_{CDFM}. Kennedy (2001 [DIRS 155940], Sections 3 and 5) showed that the HCLPF capacity computed by the CDFM method closely approximates the 1% probability of unacceptable performance seismic capacity, $C_{1\%}$, point on the mean seismic fragility curve:

$$C_{HCLPF} \approx C_{CDFM} \approx C_{1\%} \qquad \text{(Eq. B-3)}$$

Thus, these capacity definitions may be used interchangeably.

The mean fragility curve will be defined as lognormally distributed with a $C_{1\%}$ capacity and logarithmic standard deviation, β. Utilizing $C_{1\%}$ from the deterministic computations, the median capacity is given by:

$$C_{50\%} = C_{1\%}e^{2.326\beta} \qquad \text{(Eq. B-4)}$$

where 2.326 is the number of standard normal variants that the 1% point lies below the 50% point (Kennedy 2001 [DIRS 155940], Section 2.1.2 and Table 3.). For any other probability level x, the capacity is given by

$$C_{x\%} = C_{50\%}e^{z\beta} \qquad \text{(Eq. B-5)}$$

where z is the number of standard normal variants from the mean to the x-level of performance. Figure B-2 shows the family of curves with the same HCLPF value for β values ranging from 0.3 to 0.6. This figure shows that the lower β value will result in higher probability of exceedance for a given ground motion parameter.

The fragility curve is normally plotted as probability of unacceptable performance (ordinate) versus a ground motion parameter. The ground motion parameters used in the past include peak ground acceleration or 5/10 Hz spectral frequencies. Considering the high seismic demand at YMP, the expected soil-structure system frequency for the surface facilities will be about 5 Hz during low probability ground motions. Therefore, fragility calculations may be based on either PGA or spectral acceleration at the 5 Hz frequency seismic hazard curve with the damping corresponding to the response level shown in Table 7-1.

The fragility logarithmic standard deviation, β, will be estimated following guidance in ASCE/SEI 43-05 [DIRS 173805], Section C2.2.1.2. For structures and major passive mechanical components mounted on the ground or at low elevations within structures, β typically ranges from 0.3 to 0.5. For active components mounted at high elevations in structures, β typically ranges from 0.4 to 0.6.

The annual probability of unacceptable performance (seismic risk), P_F, for any structure is relatively insensitive to β. This point is illustrated by Kennedy (2001 [DIRS 155940], Section 5.3 and Table 4) and by EPRI (1994 [DIRS 161329], Section 5). Over the range of β from 0.3 to 0.6, the computed seismic risk differs by a factor of approximately 2.6. The computed seismic risk at β = 0.3 is approximately 1.5 times that at β = 0.4, while at β = 0.6 the estimated seismic risk is approximately 60% of that at β = 0.4. An estimate of β is sufficient to determine the seismic risk, P_F, within a factor of 1.6. Therefore the annual probability of unacceptable performance (seismic risk) can be computed with adequate precision using $C_{1\%}$ and an estimate of β.

In summary, the complete mean fragility curve is defined by the $C_{1\%}$ capacity deterministically computed using the CDFM methodology and a judgment-based estimate of β.

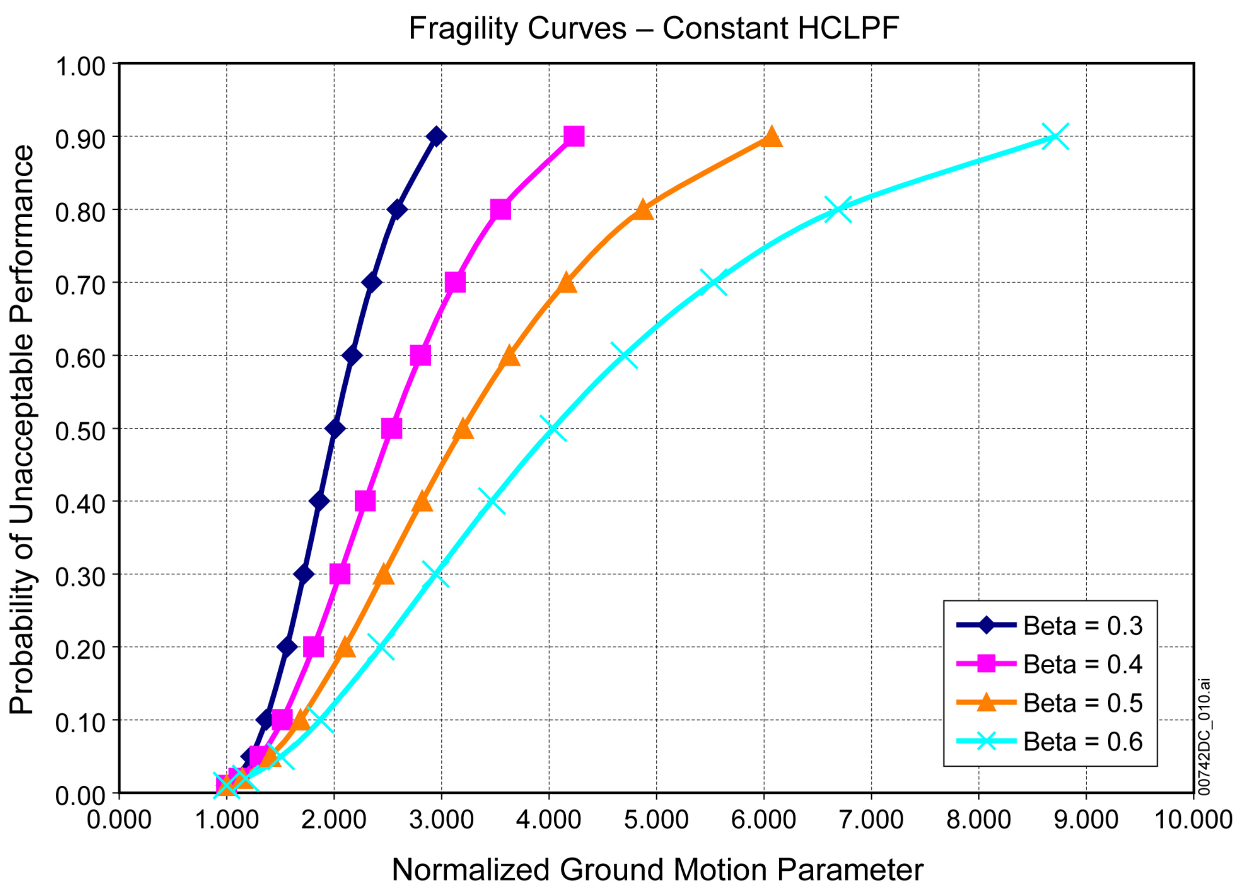


Figure B-2. Fragility Curves for Different Logarithmic Standard Deviation with Constant HCLPF Capacity

B3.2 DETERMINATION OF CDFM CAPACITY

By definition, the CDFM capacity of any structure can be estimated from:

$$C_{CDFM} = F_S * F_\mu * BDBGM \qquad \text{(Eq. B-6)}$$

where

BDBGM = Spectral acceleration at 5 Hz for the beyond design basis ground motion for which the structure has been evaluated

F_S = computed strength margin factor

F_μ = inelastic energy dissipation factor

B3.2.1 Strength Margin Factor

The strength margin factor is given by:

$$F_S = \frac{F_C C_C - D_{NS}}{D_{BDBGM}} \qquad \text{(Eq. B-7)}$$

where

C_C = capacity computed using code capacity acceptance criteria (including code specified strength reduction factors ϕ)

D_{NS} = expected concurrent non-seismic demand

D_{BDBGM} = seismic demand computed for the BDBGM input in accordance with the requirements of ASCE 4-98 [DIRS 159618], Section 3.1.1.2

F_C = capacity increase factor based on information from EPRI (1991 [DIRS 161330], Equation 2-6) and from Kennedy (2001 [DIRS 155940], Appendix A)

$$F_C = \frac{C_{98\%}}{C_C} \qquad \text{(Eq. B-8)}$$

where $C_{98\%}$ is the estimated 98% exceedance probability capacity.

The estimate of $C_{98\%}$ capacity for the shear strength of low-rise concrete shear walls will be based on ASCE/SEI 43-05 [DIRS 173805], Section 4.2.3. A number of examples for estimating $C_{98\%}$ for other structures are given by EPRI (1991 [DIRS 161330], e.g., Appendices L and M), and this guidance will be followed. When data are inadequate to estimate $C_{98\%}$ or for the sake of simplicity, F_C can be taken as 1.0.

B3.2.2 Non-Linear Margin Factor

In the CDFM method (Kennedy 2001 [DIRS 155940], Section A.2.4; EPRI 1991 [DIRS 161330], Table 2-5), the inelastic energy dissipation factor, F_μ, is estimated at the 95% exceedance probability. Generic estimates of the 95% exceedance probability F_μ for structures are given in ASCE/SEI 43-05 [DIRS 173805], Tables 5-1 and 8-1 for Limit States A, B, and C (F_μ values are unity for Limit State D). The corresponding drift and rotation limits are given in Tables 5-2 and 5-3, respectively, of ASCE/SEI 43-05 [DIRS 173805]. The basis for the low-rise concrete shear wall drift limits is presented in ASCE/SEI 43-05 [DIRS 173805].

As an example, the lateral drift per story of a low-rise concrete shear wall (height to length ratio less than 2.0) is limited to less than 0.4% of the story height for Limit State C per Table 5-2 of ASCE/SEI 43-05 [DIRS 173805]. Thus, for a 10 ft story height, the lateral drift is limited to 0.48 in. This limit provides high confidence that shear cracks in the wall will be small and that the ultimate strength of the wall will not be reduced by a few cycles of plus and minus distortion carried to this drift limit. The wall retains its full strength and serviceability. This 0.4% of the story height drift limit is identical to the drift limit specified in DOE-STD-1020-94 [DIRS 161324], Section 2.3, for low-rise concrete shear walls.

B3.3 REFINEMENT OF FRAGILITY ESTIMATES

For some unique systems and components, estimates of the CDFM capacity and β values may be difficult due to lack of data in the literature. In such cases, confirmatory nonlinear analyses may be carried out to establish the fragility curve for the system or component. Although not anticipated, a similar approach may also be used for structures.

Such analyses will be performed for a BDBGM established at the 10^{-4} per year exceedance frequency. After completion of the BDBGM non-linear evaluation, the same non-linear model of the system or component can be used to refine the CDFM capacity estimate. The refined CDFM capacity estimate is then incorporated into the probabilistic calculations.

B3.4 FRAGILITY ESTIMATES FOR COMPONENTS

Development of the fragility curve for structural type components will follow similar analytical methodology as for the structures described in the preceding sections. However, for equipment and components the development of fragility curves will be based on median-centered fragility analysis method described in Section 4.4.4 0f DOE 2007 [DIRS 181572].

In cases where the fragility curve is difficult to establish by analytical methods or if no information is available from the literature, testing may be performed to determine the fragility of a component. By performing testing with increasingly higher input motion, a fragility curve can be established (EPRI 1991 [DIRS 161330], Appendix Q). Testing of components will be initiated only when there is no other alternative to estimating a realistic fragility curve.

B3.5 SUMMARY OF METHODS FOR FRAGILITY ESTIMATES

In summary, the following methods will be used for fragility estimates:

- Use elastic seismic analysis results under BDBGM input for determining the CDFM capacity to define $C_{1\%}$, the 1% probability of unacceptable seismic performance for structures. The composite logarithmic standard deviation, β, is estimated from values available in the literature for similar structures. Using the $C_{1\%}$ and β value, the complete fragility curve can be developed.

- For equipment, use the median centered fragility analysis method to estimate seismic performance.

B4 SIMPLIFIED FRAGILITY CALCULATION METHOD

B4.1 PURPOSE

This document presents a simplified procedure for determination of mean seismic fragility curves for structures important to safety. The mean fragility curve describes the conditional probability of unacceptable performance versus the ground motion level (ASCE/SEI 43-05 [DIRS 173805]). In this document the mean fragility curve is based on high confidence of low probability of failure (HCLPF) capacity and composite logarithmic standard deviation. Both concrete and steel structures are addressed

B4.2 METHODOLOGY

At YMP, structures important to safety (ITS) are designed for Design Basis Ground Motion-2. These structures are also evaluated for Beyond Design Basis Ground Motion (BDBGM) to demonstrate their acceptable behavior. These two ground motions are defined as follows (BSC 2007 [DIRS 181572]):

- DBGM-2: Mean annual probability of exceedance of 5×10^{-4} (2,000-year return period)
- BDBGM: Mean annual probability of exceedance of 10^{-4} (10,000-year return period)

The preliminary design of the structure for DBGM-2 must be completed prior to the fragility calculations. For concrete walls and slabs, thicknesses, vertical and horizontal reinforcement must be determined. For steel structures, preliminary sizes of beams, girders (if any), columns and braces must be established. These calculations are carried out using the design basis ground motion (DBGM-2); with sufficient margin in demand/capacity ratios as defined in Section 8.4 of this document.

Once the preliminary design of the structure for DBGM-2 is completed, the fragility calculations are performed using the following steps:

Step 1. Statically analyze the structure under both the non-seismic and seismic loads using an appropriate model. For the seismic loads, use the beyond design basis ground motion (BDBGM) as input.

Step 2. Determine the seismic demand under BDBGM for sections (concrete) or members (steel).

Step 3. Determine the capacities based on strength (concrete) or allowable stresses (steel).

Step 4. Determine the available strength margin (F_S) for both concrete and steel.

Step 5. Determine the allowable energy dissipation factor (F_μ).

Step 6. Calculate the HCLPF capacity using the Conservative Deterministic Failure Margin (CDFM) method.

Step 7. Determine the composite logarithmic standard deviation, β.

Step 8. Develop the mean seismic fragility curve in terms of conditional probability of failure as a function of seismic ground motion parameter such as peak ground acceleration (PGA) or 5% damped spectral acceleration at a specified natural frequency. Considering the soil-structure interaction frequency of structures ITS at high ground motion levels, spectral accelerations at 5 Hz should be used.

Once the mean seismic fragility curve is known, the probability of unacceptable behavior of the structure can be estimated by convolving the seismic hazard curves with the fragility curves.

B4.3 HCLPF CALCULATIONS FOR CONCRETE STRUCTURES

By definition, HCLPF is the ground motion level at which there is less than 1% probability of unacceptable behavior under seismic loads. Expressed differently, it is the ground motion level corresponding to 99% non-exceedance probability. HCLPF may be expressed in terms of a ground motion parameter such as PGA or 5% spectral acceleration at a specified natural frequency.

HCLPF calculations must consider the mode of failure that will control the behavior of the structure, i.e., the "weakest link." Based on experience with seismic probabilistic risk assessments for nuclear plants, the HCLPF capacity is based on in-plane shear demand-capacity calculations for walls and out-of-plane bending requirements for the slabs. However, other modes of failure will be considered to establish confidence in the process as discussed in Section 5. Furthermore, the HCLPF capacity for the entire structure will be conservatively set equal to the lowest HCLPF capacity of a major wall or a slab.

Details of the steps for determination of fragilities for concrete structures are as follows:

Step 1 Structural Analysis

Concrete structures for surface facilities at YMP consist of shear walls and slabs with a few columns in some buildings. Seismic analyses of the concrete structures are carried out using multi-beam stick models (for Tier # 1) and finite element models (Tier # 2). All seismic analyses are conducted in accordance with this document. The design forces are then determined with the following parameters:

1. Gravity loads:

    a. 100 % of dead load and self weight.
    b. 25% of live load.
    c. Other gravity loads as applicable.

2. Seismic loads:

    a. Tier # 1 or Tier # 2 analyses under BDBGM
    b. Responses to the three-components of earthquake combined using either square-root-of-the-sum-of-squares or the component factor method.

Step 2 Demand Calculations

For Tier #1, both the non-seismic and seismic demands will be obtained directly from the multi-beam-stick model computer analyses. Tier # 1 HCLPF calculations will be based on wall in-plane shear only. Thus:

V_D = in-plane shear demand

The floors are not modeled in the Tier # 1 analysis. Hence, the HCLPF capacity for the slabs will be calculated in Tier # 1 using a simplified approach.

In Tier # 2, a finite element model will be used in the analyses. Thus, both the gravity and seismic load analyses results will be obtained as element forces. For fragility calculations, horizontal cut sections will be defined at the wall basemat junctions where the highest in-plane shears are expected. Similarly, horizontal cut sections will be defined for the slabs at the wall-slab junctions (assuming that the out-of-plane bending will be the weakest link for the slabs). Forces and moments will be obtained at the cut sections for use in the following steps. Thus, Tier # 2 fragility calculations will be carried out for both wall and slab segments:

V_D = in-plane shear demand for walls

M_D = out-of-plane moment demand for slabs

It should be noted that the in-plane shear forces under non-seismic loads are generally negligible whereas the out-of-plane moments due to gravity loads are significant in the slab evaluations.

Although the fragility calculations will be carried out for in-plane shear (walls) and out-of-plane moment (slabs), the other section forces will also be determined to demonstrate that any other failure mode will have a higher capacity than the HCLPF capacity obtained in the evaluations (See Section 5). The additional evaluations will be performed at any section with the highest demand/capacity ratios.

Step 3 Capacity Calculations

At this stage it is necessary to determine a preliminary reinforcement pattern (sizes and spacing) for the walls and slabs. The capacity calculations will be carried out using the preliminary design. As noted above, the fragility calculations for walls will be based on the in-plane shear demand and for slabs, out-of-plane bending moment demand.

Capacities for structures and components should be defined at about 98% exceedance probability. In the capacity calculations for concrete, the minimum specified concrete design strength and appropriate capacity reduction factor will be used to approximate the 98% probability of exceedance of shear wall or slab capacity.

For shear, the in-plane shear capacity will be calculated using the following equation (ASCE/SEI 43-05 [DIRS 173805]):

$$v_u = \phi\left[8.3\sqrt{f_c'} - 3.4\sqrt{f_c'}\left(\frac{h_w}{l_w} - 0.5\right) + \frac{N_A}{4l_w t_w} + \rho_{se} f_y\right] \qquad \text{(Eq. B-9)}$$

where

ϕ = capacity reduction factor,

f_c'= concrete compressive strength,

h_w = wall height,

l_w = wall length,

N_A = axial force (compression is positive),

t_w = thickness of the wall,

ρ_{se} = horizontal reinforcement ratio,

f_y = yield strength of reinforcing steel.

In this evaluation, use a ϕ-factor of 0.8, consistent with ASCE/SEI 43-05 [DIRS 173805]. Also, use the minimum specified concrete design compressive strength at 28 days. Combination of the ϕ-factor of 0.8 and the specified design strength at 28 days results in approximately 98% exceedance level. Equation B-9 implies that the in-plane bending moment does not significantly affect the in-plane shear capacity of a wall.

The total in-plane shear capacity, V_U, is then calculated from:

$$V_U = v_u d t_w \qquad \text{(Eq. B-10)}$$

Where d = the distance (in-plane) from extreme compression to the centroid of the tension reinforcement which can be determined from a strain-compatible section analysis. If such an analysis is not available, conservatively use $d = 0.6l_w$.

For out-of-plane bending, determine the capacity using the classical equation:

$$M_U = \phi(A_s f_y(d-a/2)) \qquad \text{(Eq. B-11)}$$

where

> ϕ = 0.9, capacity reduction factor in bending,
>
> A_s = the area of total reinforcement on one face of the slab,
>
> f_y = the specified minimum yield strength of the reinforcement,
>
> d = distance from the compression fiber to the centroid of the bending reinforcement, (note that d = length of the wall in shear calculations but it is the distance to the centroid of tension reinforcement perpendicular to the plane of the slab in bending calculation)
>
> a = depth of the equivalent stress block.

In the case of flexure, combination of the use of a ϕ-factor of 0.9 and the minimum specified yield strength of reinforcing steel will result in approximately 98% exceedance level.

For slabs supported by beams and girders, a more detailed capacity calculation that will consider the contributions of the beams and girders, and the effect of composite action (if applicable) may be carried out.

Step 4 Strength Margin Factor, F_s

F_s is defined as the strength margin factor i.e. a factor by which the calculated seismic demand at any cut section can be increased to reach a total demand / capacity ratio of unity. In equation form:

$$F_s = \frac{C_{98\%} - D_{NS}}{D_{BDBGM}} \qquad \text{(Eq. B-12)}$$

where $C_{98\%}$ is the section capacity at 98% probability of exceedance, D_{NS} is the non-seismic demand and D_{BDBGM} is the seismic demand under BDBGM.

Assuming that there exists non-seismic demand in shear, V_{NS}, and in bending, M_{NS}, and applying this equation at a cut section, the strength margin factors for in-plane shear and out-of-plane bending will be:

$$F_{SV} = (V_U - V_{NS}) / V_D\text{, and} \qquad \text{(Eq. B-13)}$$

$$F_{SM} = (M_U - M_{NS}) / M_D \qquad \text{(Eq. B-14)}$$

These strength margin factors will then be used in calculating the HCLPF capacities.

Step 5 Energy Dissipation Factor, F_μ

The energy dissipation (termed energy absorption in ASCE/SEI 43-05 [DIRS 173805]) factors are given in Table 5-1 of ASCE/SEI 43-05 [DIRS 173805]. These factors depend on the allowable limit state. Where confinement is not required, Limit State A is appropriate for these facilities. Where HVAC-controlled confinement is necessary (areas where canisters are opened, except underwater), Limit State C is more appropriate (BSC 2007 [DIRS 181572], Appendix A, Section A1.2). Based on ASCE/SEI 43-05 [DIRS 173805], the energy dissipation factor for concrete slabs and walls range from 1.5 to 2.0 for Limit State C. For Limit State A, the range is from 2.0 to 2.5. Thus, conservatively, the following energy dissipation factors will be used for concrete structures:

Limit State C (in confinement zone): $F_\mu = 1.50$

Limit State A (outside confinement zone): $F_\mu = 2.0$

However, for conservatism for confinement controlled designs, Limit State D (having $F_\mu = 1.0$) may be used.

Step 6 HCLPF Capacity

By combining Equations B-3 and B-6, the HCLPF capacity may be approximated by:

$$C_{HCLPF} \approx C_{CDFM} \approx C_{1\%} \approx (F_S).(F\mu).(BDBGM) \qquad \text{(Eq. B-15)}$$

Where BDBGM is the ground motion parameter such as PGA or 5% damped spectral acceleration at a specified natural frequency.

Thus, incorporating the strength margin factor and the energy dissipation factor described in steps 5 & 6, the HCLPF capacity for concrete structures will be obtained using the following equation:

$$\text{Limit State A:} \quad C_{HCLPF} \approx C_{1\%} \approx (F_S).(2.0).(BDBGM) \qquad \text{(Eq. B-15a)}$$

$$\text{Limit State C:} \quad C_{HCLPF} \approx C_{1\%} \approx (F_S).(1.5).(BDBGM) \qquad \text{(Eq. B-15b)}$$

Step 7 Estimation of Composite Variability, β

In general, fragility curves are defined by the mediun capacity and two lognormally distributed random variables β_R and β_U which define the uncertainty and randomness (Kennedy 2001 [DIRS 155940]). It is sufficient to define the fragility curve by a single mean (composite) fragility curve defined by a median capacity and composite logarithmic standard deviation β_c given by:

$$\beta_c = (\beta_R^2 + \beta_U^2)^{1/2} \qquad \text{(Eq. B-16)}$$

Studies show that the composite logarithmic standard deviation value ranges between 0.3 and 0.5 for structures (Kennedy 2001 [DIRS 155940], ASCE 2005 [DIRS 173805]). The lower value of

β_c will result in higher probability of unacceptable behavior (see Figure B-2). Therefore, only β_c = 0.3 need to be used in the calculations and explicit calculation of the β_c value is not necessary. Higher values of β_c may be used to perform additional calculations to develop insight on the resulting probabilities.

Step 8 Mean Fragility Curves

With the HCLPF capacity known, the median capacity is determined from the following equation

$$C_{50\%} = C_{1\%}\ e^{2.326\beta} \qquad \text{(Eq. B-17)}$$

where β is the logarithmic standard deviation. .

Once the median capacity is established, the mean fragility curve is given by the following equation:

$$C_{X\%} = C_{50\%}.e^{z\beta} \qquad \text{(Eq. B-18}$$

where

C_x = Capacity at 'x' exceedance probability (non-exceedance)

z = Value of normal variant (standard deviation) corresponding to "x"

The capacities obtained from Equation 3-18 for each probability of exceedance are convolved with the seismic hazard curves to estimate the probability of unacceptable behavior.

B4.4 HCLPF CALCULATIONS FOR STEEL STRUCTURES

Steel Frames at YMP

At YMP, steel structures will be mostly braced frames. In general, concentric bracing (between two beam-column joints) but occasionally chevron bracing (inverted v connected to a beam, with the other ends connected to beam-column joints) will be used to accommodate access requirements. In rare cases there may be a need to use special moment frames. In the braced frames, only the braces and the beams that are connected to chevron braces are expected to yield. In the special moment frames, only the beams will be permitted to yield (i.e., strong column-weak beam design). Thus, there are only three types of members which need to be evaluated for determination of the HCLPF capacity of a steel structure:

1. Braced Frames: braces
2. Braced Frames: beams connected to chevron braces
3. Moment Frames: beams

The above approach implies that the other members of the frames will not yield even under the BDBGM. Therefore, in preliminary design, it is of utmost importance to size the members such that, under the DBGM loads, the demand-capacity ratios for the yielding members are greater than the demand-capacity ratios for the non-yielding members in the same frame. To assure the structural behavior will be as intended, the D/C ratios of yielding members should be at least 20% greater than the D/C ratios of non-yielding members. Assuming that the demand-capacity ratios for the yielding members will be about 0.6 under DBGM-2, the D/C ratios of the non-yielding members should not exceed 0.50. Since the behavior of the structure is elastic until yielding initiates, the stresses in the members are proportional to the earthquake input. When the members intended to dissipate the input energy inelastically start yielding, the forces in the non-yielding members will no longer increase and remain elastic during the nonlinear response of the overall structure.

The lower demand/capacity ratio for non-yielding members is essential to assure that the inelastic deformations will take place only in the designated yielding members. In order to demonstrate compliance with this requirement, tables of demand-capacity ratios under gravity plus DBGM loads will be prepared for each frame. Furthermore, the demand-capacity ratios for yielding members in each lateral load resisting vertical plane should be approximately the same.

HCLPF Calculation Approach

HCLPF calculations for the steel structures will be based on individual element fragility evaluations. The HCLPF capacity for the entire structure will be conservatively set equal to the lowest HCLPF of any main member. This approach assumes that connections are at least as strong as the yielding members.

Prior to the fragility calculations, the structure is analyzed using approximate methods. Seismic analysis is performed using the DBGM-2 loads at 7% damping which is appropriate for bolted construction. The initial member sizes are determined using the project loading combinations. All members must comply with the code requirements and are required to meet the AISC equations for axial loads (bracing) and combined axial load and biaxial bending (beams and beam-columns). Applicable design equations are satisfied with a stress ratio < 1.0. The slenderness ratio of the bracing members must be less than 120 to preclude elastic buckling.

The steps to be followed for steel structures are as follows:

Step 1 Structural Analysis

A finite element model of the steel building consisting of beams, columns and braces is analyzed utilizing SAP2000 program (or other computer codes with similar capabilities) with the following parameters:

1. Modeling:

    a. Model all major beams, columns, and braces.
    b. All connections are modeled as pinned connections, with release of moments. If there is a moment connection by design, it should be modeled as such.

2. Analysis under gravity loads:

   a. 100 % of dead load and self- weight.
   b. 25% of live load.
   c. Other gravity loads as applicable
   d. Perform analysis to obtain non-seismic forces and moments.

3. Analysis under seismic loads:

   1. Input horizontal and vertical free field response spectra of beyond design basis ground motion (BDBGM) with 10% structural damping. Higher damping of 10% is permitted since the structure will respond in the inelastic range, even though the inelasticity will take place in the braces (braced frames) or beams (moment frames).

   2. Perform analysis to obtain seismic forces and moments.

   3. Use SRSS to combine the responses from earthquake input in three orthogonal directions.

Step 2 Demand Calculations

Using the SAP2000 analyses results for gravity loads and the BDBGM seismic loads, define the following:

1. Non-seismic forces and moments (i.e. self weight, dead and live loads):

   a. P_{ns} = Non seismic axial force
   b. M_{1ns} and M_{2ns} = Non-seismic bending moments about two bending axes

2. Seismic forces:

   a. P_s = Seismic Axial force
   b. M_{1s} and M_{2s} = Seismic bending moments

where M_{1s} and M_{2s} refer to the bending moments about the two bending axes of the beams.

Step 3 Capacity Calculations

For fragility calculations, the capacities of the members must be established at "strength design" level, including the capacity reduction factors. For this purpose, the allowable stresses will be increased by a factor of 1.5 based on supplement 1 to AISC/ANSI N690 [DIRS 158835] note 'k' in Table Q.1.5.7.1 and ASCE/SEI 43-05 [DIRS 173805] Table 4-2. This factor (along with the minimum specified yield strength) will be used to approximate the 98% exceedance level capacity.

Thus, the following capacities will be determined:

1. Bracing member

    a. P_a = Maximum permissible axial force

2. Beams connected to chevron braces and beams in moment frames

    a. F_a = Allowable Axial stress
    b. F_b = Allowable Bending Stress

Step 4 Demand-Capacity Calculations

The demand-capacity ratios will be determined using the information obtained in Steps 2 and 3. In these calculations demand-capacity ratios for the non-seismic and seismic loads will be calculated separately and then added together to obtain the total ratio. This is acceptable since the calculations are carried out elastically. For braces, only axial loads are considered. In the case of the beams, axial load and bending about both axes are considered. Since the allowables are different for axial and bending stresses, the ratios must be calculated using the stresses due to axial load and bending separately, with the corresponding allowable stress.

1. Brace members

$$\text{Demand / Capacity} = \alpha_{\text{axial}} = \left\{ \frac{\text{Calculated Axial Stress}}{\text{Allowable Axial Stress}} \right\}$$

$$\alpha_{\text{axial}} = \left\{ \frac{f_{\text{ans}}}{F_a} + \frac{f_s}{F_a} \right\} \qquad \text{(Eq. B-19)}$$

$$\alpha_{\text{axial}} = \alpha_{\text{axial}_{\text{NS}}} + \alpha_{\text{axial}_{\text{S}}} \text{, or}$$

$$\text{Demand / Capacity} = \alpha_T = \alpha_{NS} + \alpha_S$$

where

f_{ans} = Axial Stress due to non-seismic loading

f_{as} = Axial Stress due to seismic loading

F_a = Allowable axial stress

$\alpha_{\text{axial}_{\text{NS}}}$ = α_{NS} = Demand Capacity Ratio of axial forces (i.e. calculated versus maximum permissible) due to non-seismic loading

$\alpha_{\text{axial}_{\text{S}}}$ = α_S = Demand Capacity Ratio of axial forces (i.e. calculated versus maximum permissible) due to seismic loading

α_T = Total demand-capacity ratio

2. Beams connected to chevron braces and beams in moment frames

In a similar manner, the demand-capacity ratio for the beams connected to chevron braces and beams in moment frames will be calculated using the calculated and allowable stresses for the non-seismic and seismic loads separately. Then the total demand-capacity ratio will be obtained by simple addition of the ratios due to non-seismic and seismic loads since this is elastic analysis and thus the principle of superposition is valid.

a. Non Seismic Loads

α_{NS} = Demand /Capacity ratio due to non-seismic loading

$$\alpha_{NS} = \frac{f_{ans}}{F_a} + \frac{f_{blns}}{F_{bl}} + \frac{f_{b2ns}}{F_{b2}} \quad \text{(Eq. B-20)}$$

b. Seismic Loads

α_S = Demand /Capacity ratio due to seismic loading

$$\alpha_s = \frac{f_{as}}{F_a} + \frac{f_{bls}}{F_{bl}} + \frac{f_{b2s}}{F_{b2}} \quad \text{(Eq. B-21)}$$

$$\text{Total Demand / Capacity} = \alpha_T = \alpha_{NS} + \alpha_S \quad \text{(Eq. B-22)}$$

where

f_{ans} = Axial Stress due to non-seismic loading

f_{b1ns} & f_{b2ns} are bending stresses due to non-seismic loading

f_{as} = Axial Stress due to seismic loading

f_{b1s} & f_{b2s} are bending stresses due to seismic loading.

F_a, F_{b1} and F_{b2} are corresponding allowable stresses. It is assumed throughout that the allowable stresses at strength level are the same for both non-seismic and seismic loads.

Note that Equations B-20 and B-21 calculate the maximum stresses in one corner of the cross section. This approach is conservative since it is unlikely that all three components of the stress will be additive at any corner at any given time.

Step 5 Strength Margin Factor: (F_s)

F_s is defined as the strength margin factor i.e. a factor by which seismic demand for any given member can be increased to reach a total demand-capacity ratio, "α," is equal to unity. In equation form, this can be expressed as the net capacity (total capacity less the required capacity for the non-seismic loads) divided by the seismic demand, as given in this Appendix:

$$F_S = \frac{C_{98\%} - D_{NS}}{D_S} \quad \text{(Eq. B-23)}$$

where

$C_{98\%}$ = Member capacity at 98% exceedance probability

D_{ns} = Non-seismic Demand

D_s = Seismic Demand

Dividing both side by $C_{98\%}$

$$F_S = \frac{1-\left(D_{ns}/C_{98\%}\right)}{D_s/C_{98\%}} \quad \text{(Eq. B-24)}$$

With the assumption that code-based allowable values, including capacity reduction factor (ϕ factor) are at 98% exceedance probability level.

$$D_{ns}/C_{98\%} \approx \Sigma\, f_{ns}/F = \alpha_{NS} \quad \text{(Eq. B-25)}$$

Similarly,

$$D_s/C_{98\%} \approx \Sigma\, f_s/F = \alpha_S \quad \text{(Eq. B-26)}$$

where

f_{ns} = Calculated axial and bending stress due to non-seismic loading

f_s = Calculated axial and bending stress due to seismic loading

F = The appropriate strength level allowable stress corresponding to the 98% exceedance probability level.

α_{NS} = As defined previously

α_S = As defined previously

then, FS can be approximated as

$$F_S = \frac{1-\alpha_{NS}}{\alpha_S} \quad \text{(Eq. B-27)}$$

Step 6 Allowable Energy Dissipation Factors

Allowable energy dissipation factors (i.e., ductility of members) are given in ASCE/SEI 43-05 [DIRS 173805]. The approach described above is consistent with ASCE/SEI 43-05 [DIRS 173805] which provides the following criteria for braced frames (Section 6.1.2):

1. Energy Dissipation is permitted for bracing members only.
2. All other members are required to remain elastic under the design loads. (see ASCE/SEI 43-05 [DIRS 173805] 6.1.2).

For fragility calculation, the following energy dissipation factors will be utilized:

Table B-1. Allowable Energy Dissipation Factors for Steel Structures

Item	Member Type	F_μ
a	Braced frames: braces	2.50
b	Braced frames: beams connected to chevron braces	2.50
c	Moment frames: beams	2.50

Item a is consistent with ASCE/SEI 43-05 [DIRS 173805] Table 5.1 for Limit State "A". Item "b" is not explicitly addressed in ASCE/SEI 43-05 [DIRS 173805]. However, energy dissipation factor of greater than unity is reasonable for beams connected to chevron braces since most of the seismic load will be applied to the beam as bending moment with small axial load relative to the capacity of the member. Therefore, similar to columns with small axial load in moment frames, it is reasonable to assume "F_μ" > 1.0. The value of $F_\mu = 2.5$ is consistent with the value for the bracing member and is judged to be appropriate for beams connected to chevron bracing for Limit State "A".

For the moment frames, the allowable energy dissipation factor of 2.50 corresponds to Limit State C. This value was chosen for consistency and is conservative.

Step 7 Estimation of Composite Variability, β_c

Most of the studies regarding the composite logarithmic standard deviation are based on reinforced concrete structures (Kennedy 2001 [DIRS 155940], ASCE 2005 [DIRS 173805]). As defined by Eq. B-16, the composite variability consists of two parts: variability related to uncertainty and variability associated with randomness. Considering these factors, it is reasonable to assume that the range of composite variability will be from 0.3 to 0.5 for steel braced frames also.

Therefore, similar to reinforced concrete structures, a lower bound value of $\beta_c = 0.3$ will be used to define the fragility curve for steel structures. Explicit calculations for the composite variability are not deemed essential for the probabilistic estimates. However, as before, higher values of β_c may be used to perform additional calculations to develop insight on the resulting probabilities.

Step 8 HCLPF Capacity

As stated before in Equation B-15, the HCLPF capacity is approximated by the following equation:

$$C_{HCLPF} \approx C_{CDFM} \approx C_{1\%} \approx (F_s)(F\mu)(BDBGM) \qquad \text{(Eq. B-28)}$$

Combining Steps 5 and 6, the HCLPF capacity for the yielding steel members can be expressed by:

$$C_{HCLPF} \approx C_{1\%} \approx (F_s)(2.5)(BDBGM) \qquad \text{(Eq. B-29)}$$

Step 9 Mean Fragility Curve

With the HCLPF capacity for the member known, the median capacity is calculated from

$$C_{50\%} = C_{1\%} \; .e^{2.326\beta} \qquad \text{(Eq. B-30)}$$

Once the median capacity is established, the mean fragility curve is given by the following equation:

$$C_{x\%} = C_{50\%} \; .e^{-z\beta} \qquad \text{(Eq. B-31)}$$

where

C_x = Capacity at ‘x’ exceedance probability (non-exceedance)

z = Value of normal variant (standard deviation) corresponding to “x”

B4.5 ENERGY DISSIPATION FACTOR AND CONFINEMENT

In Sections B4.3 and B4.4 of this appendix, the reference limit states used were Limit State A (large permanent distortion, no confinement) and Limit State C (limited permanent distortion, HVAC-controlled confinement). The energy dissipation factors given in those sections are consistent with the limit states.

In some cases Limit State D (elastic response, no damage) may need to be specified, with an energy dissipation factor F_μ of 1.0. For this limit state, the Preliminary Preclosure Nuclear Safety Design Bases (BSC 2007 [DIRS 184154]) referenced in BOD document (BSC 2007 [DIRS 182131] specifies the following requirements:

> Performance Goal for Building Collapse $\leq 2 \times 10^{-6}$ [mean frequency of 10^{-4} over a preclosure period of 50 years].

The HCLPF capacities are computed for establishing the fragility curves that will be convolved with the seismic hazard curves for determination of the performance factors to meet the performance goals. The HCLPF capacity is computed by combining Equations B-3 and B-6, as follows.

$$C_{HCLPF} = (F_s)\,(F_\mu)\,(PGA) \qquad \text{(Eq. B-34)}$$

The performance goal is accomplished as follows:

1. By having an appropriate HCLPF capacity determined by Equation B-34 with the F_μ factor associated with Limit State A defined in ASCE/SEI 43-05 [DIRS 173805]

2. By having the HCLPF capacity determined by the Equation B-34 with the energy dissipation factor F_μ associated with limit state D defined in ASCE/SEI 43-05 [DIRS 173805]

B5 SPECIAL EVALUATIONS

The HCLPF calculations discussed in the previous sections imply that the behavior of the structure is controlled by the section (concrete) or member (steel) with the lowest HCLPF value. It also implies that, any of the other failure modes will not occur in the lateral load path prematurely in any other member. In order to demonstrate the validity of this implicit assumption, additional evaluations must be carried out. This section provides the minimum additional evaluations for concrete and steel buildings as part of estimation of probability of unacceptable behavior.

Special Provisions for Concrete Structures

For concrete structures, the first onset of significant inelastic deformation (FOSID) value was based on in-plane shear for walls and out-of-plane bending for slabs. In order to demonstrate the adequacy of the entire structure, the additional evaluations shown in the following table must be carried out:

Table B-2. Special Evaluations for Concrete

Cut Section	Basic Evaluation	Additional Evaluations
Wall at basemat junction	In-plane shear	In-plane moment and axial load, Out-of-plane bending, Out-of-plane shear
Slab at wall junction	Out-of-plane bending	In-plane shear, In-plane bending and axial load Out-of-plane shear

The evaluations shown in the last column will be carried out selectively at sections with the highest demand/capacity ratios.

In these evaluations, the seismic forces must correspond to the HCLPF level, modified by the applicable energy dissipation factor. In other words, the seismic demand must be calculated without any energy dissipation factor. Methods to be used in these evaluations are as follows:

1. **Combined in-plane bending and axial load**. For the combined effects of in-plane bending and the axial load, a strain-compatible analysis must be performed to obtain a partial interaction diagram. The EXCEL spreadsheet for the design of reinforced concrete shear walls and slabs contains a subroutine to perform strain-compatible section analysis and is given in Appendix D of this document. The strain and stress diagrams assumed in this analysis are shown in Figure B-4. Figure B-5 shows an example of an interaction diagram that has been calculated using the concrete EXCEL algorithm. A point on this diagram represents the axial load capacity, P_C, and the corresponding moment capacity, $M_{C,}$ for a given eccentricity. The next step is to calculate the in-plane bending moment and the axial load corresponding to the HCLPF value for this element. These moments and axial loads will be:

   $$P_T = P_{NS} + P_{HCLPF}/\ F_{\mu,} \text{ and,} \qquad \text{(Eq. B-32)}$$

   $$M_T = M_{NS} + M_{HCLPF}/\ F_{\mu,} \qquad \text{(Eq. B-33)}$$

   where, P_T = total axial demand, P_{NS} = non-seismic axial load, P_{HCLPF} = seismic axial load at HCLPF level, M_T = total in-plane bending demand, M_{NS} = non-seismic in-plane bending, M_{HCLPF} = in-plane bending demand at HCLPF level, and F_μ = energy dissipation factor for concrete.

   The total axial load , P_T, and in-plane bending moment , M_T, thus calculated will then be plotted on the interaction diagram. If the point falls within the diagram, the section is adequate. If not, the vertical reinforcement is increased and the evaluation is repeated.

2. **In-plane shear**. The EXCEL spreadsheet for concrete shear walls and slabs for each cut section includes the demand capacity ratios for in-plane shear. A simple check using the in-plane shear corresponding to the HCLPF capacity can be made. If the D/C ratio is less than 1.0, the section is adequate; if not further iteration is necessary.

3. **Out-of-plane bending**. For this evaluation, the seismic demands for out-of-plane bending and shear are calculated at the HCLPF level. Using the moments M_{HCLPF}/ F_{μ}, the out-of-plane reinforcement required is determined. Again, comparing this reinforcement which the capacity shown on the EXCEL spreadsheet for concrete shear walls and slabs for the related cut section includes will show the adequacy of the existing design. If the corresponding D/C ratio is less than 1.0, the section is adequate; if not further iteration is necessary.

4. **Out-of-plane shear**. Similarly, out-of-plane shear corresponding to the HCLPF level will be calculated and compared with the capacity indicated on the concrete design spreadsheet. If the out-of-plane capacity without reinforcement is adequate, no further action is necessary. If not, further evaluation is required.

Special Provisions for Steel Structures

In the case of the steel structures, the requirement is that non-controlling members must not yield at HCLPF level loads. The additional evaluations include:

1. Connections of yielding members must not fail at member yield levels. Thus:

   $U_{con} > 1.5$ Sy

   where

   U_{con} = Ultimate strength of connection

   S_y = Yield strength of yielding members (i.e. braces and beams connected to chevron bracing).

2. **Other connections**: The other connections in the load path must be designed for the maximum loads that can be transmitted to these connections.

B6 REFINEMENTS FOR THE FRAGILITY CALCULATIONS

The simplified fragility calculation method described above will result in conservative fragility curves. Improvements to the method can be made without invalidating the fundamental principles of fragility calculations. The main areas where such improvements can be made are discussed below.

Realistic Strength Margin Calculation - Load Redistribution

In sections 3 (concrete shear wall structures) and 4 (steel braced frames), the overall structure fragility was assumed to be equal to the fragility of its weakest member. Obviously this is a conservative assumption as there will be re-distribution of seismic forces once the capacity of the controlling member is reached. This re-distribution will increase the capacity of the structure. In order to account for the additional capacity, the following approaches are recommended:

1. For shear wall structures, the shear capacity of each major wall in one direction is calculated using Equations B-9 and B-10 and added together to calculate the total capacity. The demands on these walls are also added to determine the total demand. Then the overall structure demand-capacity ratio is established and the strength margin for the overall structure is calculated using Equation B-12. The value of F_S thus calculated (which should be higher than the value for the controlling wall) is used in the determination of the CDFM for the entire structure. In these calculations, the effect of torsion on the extreme walls should be taken into account as it will reduce the capacity of the structure.

2. For braced frames, a similar approach may be used also. In this case the strength margin must be calculated in two stages. First, the lateral load capacity of each braced frame is calculated as the lateral load level at which braces in tension have yielded and braces in compression have reached their capacity. In the second stage, the capacities of all frames are added together to determine the overall structural capacity. The lateral loads on each frame must also be calculated taking into account the torsional effects (the torsional effects should be added to one side of the structure, without subtracting from the other side). Finally, the strength margin is determined using Equation B-24.

As described above, redistribution of lateral loads will increase only the strength margin factor, F_S, thus increasing the HCLPF capacity of the structure.

Analysis of the Logarithmic Standard Deviation – β_c -Factor

The simplified procedure given above is based on the assumption that the β value is 0.3. This value of β_c is considered the lower bound for complex structures and results in a conservative estimate of the fragility curve. The composite variability β_c is defined as $\beta_c = (\beta_R^2 + \beta_U^2)^{1/2}$ where β_R is the variability due to randomness and β_U is the variability due to uncertainty. Both variabilities depend on many factors including seismic input and site response, seismic analysis, modeling, soil-structure interaction, and ductility.

Both randomness and uncertainty variabilities can be calculated using the project-specific data by the “separation of variables” method to obtain a more realistic composite logarithmic standard deviation. It is expected that the actual β_c value will be greater than 0.4 changing the shape of the fragility curve. This change will ultimately result in a smaller performance factor which may be needed to demonstrate compliance with 10 CFR 63 [DIRS 176544].

Other Refinements

The ductility factors used in sections B3 and B4 are taken from ASCE/SEI 43-05 [DIRS 173805]. These factors are generally conservative. More realistic ductility factors may be calculated considering the actual deformations of the structure and/or specific test data applicable to structures and systems of the YMP.

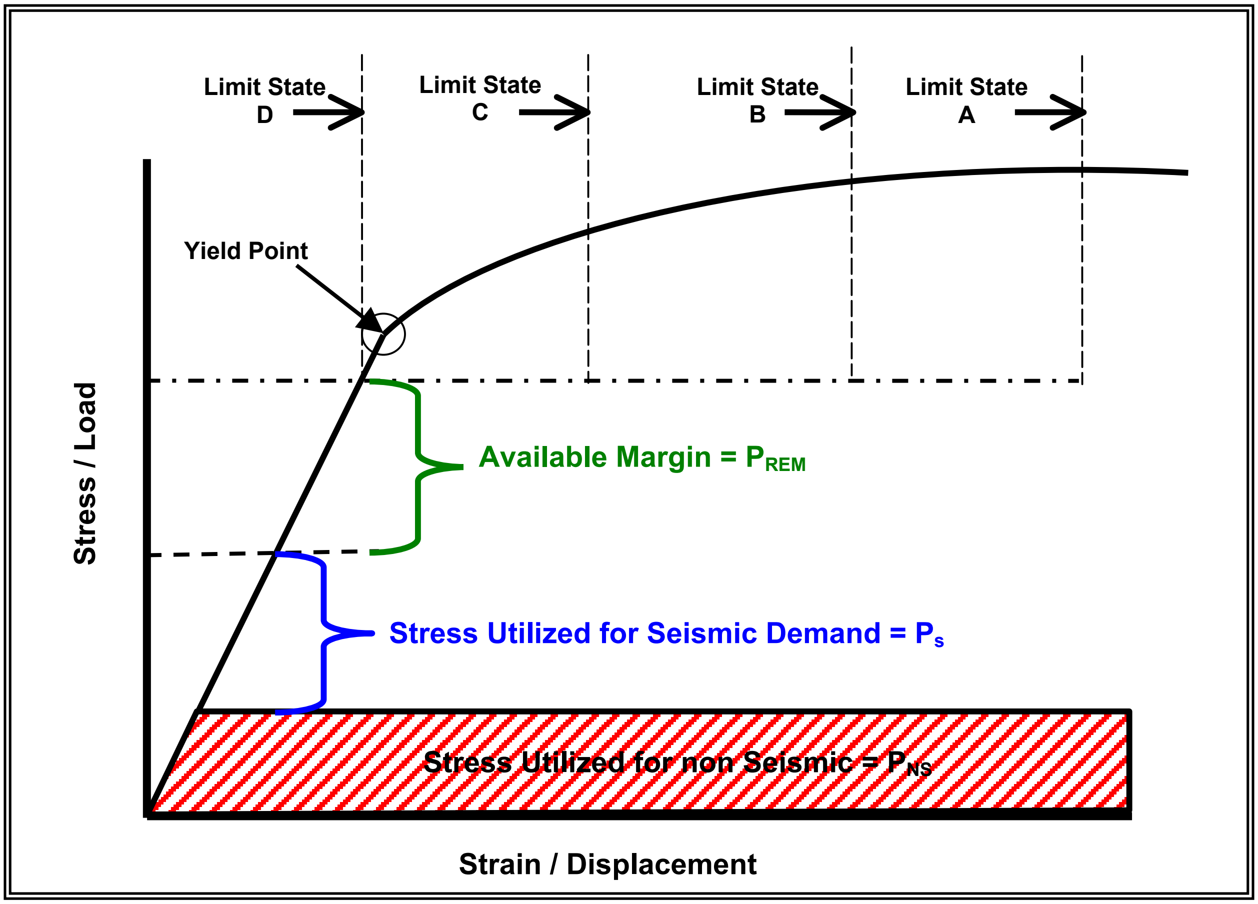


Figure B-3. Stress-Strain or Load-Displacement Diagram for Steel Members and Limit

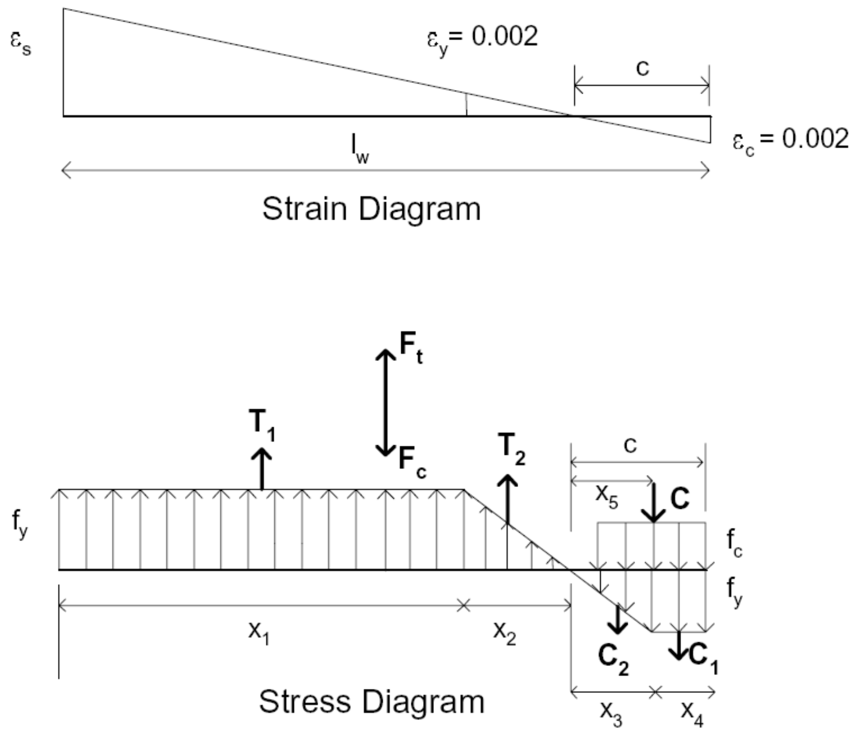


Figure B-4. Stress-Strain Diagram for Strain-Compatible Section Analysis

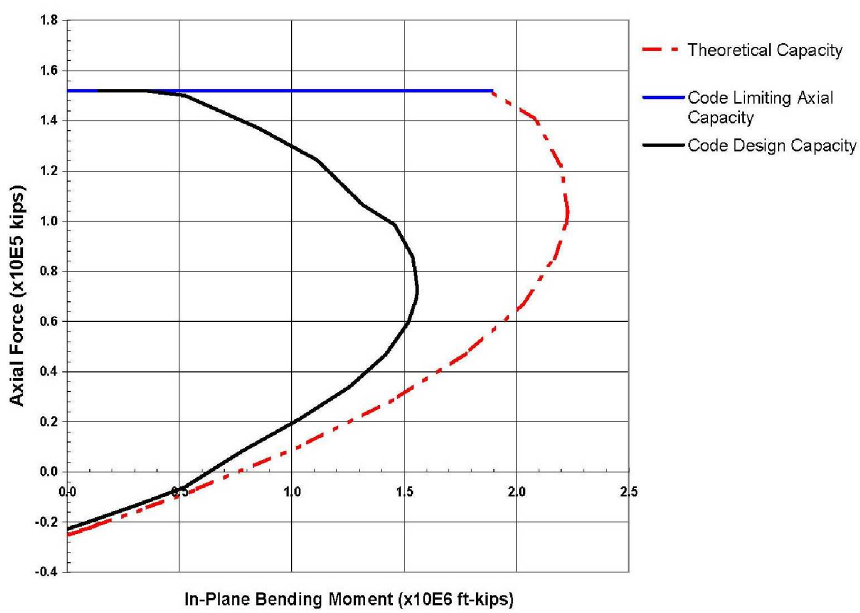


Figure B-5. Typical Interaction Diagram for Shear Walls

APPENDIX C
SOIL SPRINGS AND DAMPING

C1 SOIL SPRINGS

Soil springs (also called the foundation impedance functions) represent the interaction between the foundation basemat and the underlying soil under seismic loads. Soil springs for the models used in Tier # 1 and Tier # 2 analyses will be determined as described in Sections C1.1 and C1.2.

C1.1 SOIL SPRINGS FOR LUMPED-MASS MODELS – TIER # 1 ANALYSIS

In Tier # 1 analysis, the impedances are represented by "global" springs:

K_h = horizontal global springs, kip per foot
K_v = vertical global spring, kip per foot
K_φ = rocking springs, kip-feet per radian
K_t = torsional spring, kip-feet per radian.

Equations for these springs are given in ASCE 4-98 [DIRS 159618], Section 3.3.4.2. Impedances are determined for the median (best estimate) as well as for the upper- and lower-bound estimates of soil properties. Alternatively, impedances may be calculated from the SSI analysis. The seismic analyses using the lumped-mass model are carried out three times and the results are enveloped.

The equations for the equivalent springs in ASCE 4-98 [DIRS 159618] are for uniform half-space. At layered sites, the different properties of each layer should be taken into account. One method for determining impedances at layered sites is given by Hadjian and Ellison (1985 [DIRS 168406]).

C1.2 SOIL SPRINGS FOR FINITE ELEMENT MODELS – TIER # 2 ANALYSIS

In Tier # 2 static analysis, the supporting medium will be represented using "nodal" springs. Soil springs are given in terms of force per length per unit area (e.g., kcf). For the finite element model analysis, the springs are integrated over the tributary area of each node and then attached to nodes. The units of nodal springs are thus force per length.

When the static analysis is for long term loads (i.e., gravity loads), the springs correspond to secant stiffness and take into account: foundation size and depth, soil properties and layering geometry, and loading conditions with and without temporary loading such as earthquake loading. The spring stiffness associated with the long-term loads is based on static soil properties and is less than the corresponding spring stiffness associated with short-term loading. Typically, the spring stiffness for long-term loading is half of the spring stiffness for short-term loading.

When the static analysis is for short-duration loads (i.e., seismic loads), the soil springs should be based on dynamic soil impedances, which are based on global springs and damping coefficients.

The soil springs per unit area will be calculated from the previously discussed global springs, K_h, K_v, and K_φ, for horizontal and vertical translations and rocking, respectively. Normally, the torsional springs are neglected in the finite element analysis. The springs per unit area are obtained from the following equations:

$$k_h = K_h/A \quad \text{(Eq. C-1)}$$

$$k_v = K_v/A \quad \text{(Eq. C-2)}$$

$$k_v = K_\varphi /I_A \quad \text{(Eq. C-3)}$$

where A is the area of the basemat and I_A is the moment of inertia of the basemat area with respect to its centroidal axis. The vertical unit spring constant from Equations C-2 and C-3 will have different values. In practice, it is usually sufficient to use the value obtained from Equation C-2. If rocking motion of the structure is dominant, then the value from Equation C-3 should be used. In cases where both the translation and rocking responses are dominant, a more complex variation of the soil springs may be used. Rotational springs computed from the distributed nodal springs should be in agreement with the global rotational springs.

Distribution of the vertical springs under the foundation mat (for both short- and long-term loading) is a function of mat stiffness and of the layout and connection of the building walls to the mat. Normally, a uniform distribution is used but a parabolic distribution, especially for flexible mats, would provide a more realistic representation of soil stiffness. The parabolic distribution tends to increase the moments near the boundaries of the basemat.

C2 FOUNDATION DAMPING

C2.1 FOUNDATION DAMPING FOR LUMPED-MASS MODELS – TIER # 1 ANALYSIS

The Tier # 1 analysis using lumped-mass models must consider the effects of soil damping. The foundation damping values, termed "equivalent damping coefficient" or "dashpot" and based on the half-space theory are also given in ASCE 4-98 [DIRS 159618], Section 3.3.4.2. Again, the damping coefficients (dashpots) in ASCE 4-98 are given for a uniform half-space. The paper by Hadjian and Ellison (1985 [DIRS 168406]) also includes a methodology to calculate the approximate damping coefficient for layered sites.

Another issue at layered sites is the impedance mismatch between the layers, which causes reflection of the seismic waves at the layer boundaries, thus increasing the response. This effect can be approximated by reducing the damping coefficient. Hadjian and Ellison (1985 [DIRS 168406]) recommend reduction of the translational damping coefficient to 75% of its theoretical value. There is no reduction in the rotational damping coefficient. These recommendations should be followed.

In dynamic analysis using soil springs, there is a need to determine the modal damping corresponding to SSI modes. The modal damping associated with SSI modes can be calculated using the classical relationships:

$$\beta = ½ C_i/(K_iM)^{½} = C_i\omega_o/2K_i \qquad \text{(Eq. C-4)}$$

where Ci and Ki are the damping coefficient (with the translational value reduced to 75%) and corresponding stiffness, respectively, and ω_o is the undamped circular system frequency. Equation C-4 can be used for translational (all three directions) and rotational (all three directions) SSI modes using the parameters for respective direction.

C2.2 FOUNDATION DAMPING FOR FINITE ELEMENT MODELS – TIER # 2 ANALYSIS

Dynamic analysis in Tier # 2 analysis will be carried out using SASSI2000 [DIRS 182945] (Section 7), which rigorously incorporates the foundation impedance calculation.

C3 APPLICATION OF TIER # 1 ANALYSIS AT YMP

The following guidance is provided for dynamic analysis in Tier # 1, using lumped-mass models with the soil impedances. Considering the actual soil profiles at YMP and the theory of impedances described in Sections C2.1 and C2.2, the following guidelines are applicable:

Step 1. The variation of soil properties under the foundation such as variation of thickness of alluvium should be considered to compute bounding values of foundation impedance.

Step 2. Variability in soil properties should be taken into account by using the strain-compatible lower bound, median, and upper bound soil properties for each of the soil profiles identified in Step 1.

Step 3. The equivalent shear modulus and the equivalent damping coefficient should be calculated using the paper by Hadjian and Ellison (1985 [DIRS 168406]). The translational damping coefficients should be reduced to 75% of the theoretical value.

Step 4. The associated SSI damping should be calculated using Equation C-4. Damping thus determined shall not exceed 20%.

Step 5. In the dynamic model used in SAP2000 [DIRS 178238] analysis, use the foundation SSI damping for soil springs and structural damping for the structure. Use the modal damping composition method in SAP2000 to compute the composite modal damping.

Step 6. Review the results of modal analysis to ensure that damping for all SSI modes are limited to 20%. The damping for SSI modes must be limited to the values given in Section 7.2.4.2.

Step 7. Alternatively, foundation springs and damping coefficients can be used in SAP2000 [DIRS 178238] analysis. To use damping coefficients in SAP2000, a time history analysis should be performed using a direction-time integration method. Structural damping can be specified by mass and stiffness proportional damping as described in ASCE 4-98 [DIRS 159618], Section 3.1.5.2. Care must be taken in applying mass and stiffness proportional damping so as to not apply additional damping to the soil springs.

The additional caveats for the bounding calculations are:

- The soil and rock properties down to 500 ft have been provided in DTN: MO0706SCSPS5E4.002 [DIRS 181616]. These properties are given for both 35 and 110 ft depths of alluvium layers as required in Step 1. In equivalent shear modulus calculations, use either the entire depth or the depth to a point where the normal stress is less than 0.2 times the stress at the surface. Use Poisson's ratio of 0.3 as recommended in the geotechnical report (BSC 2007 [DIRS 182582]) for both alluvium and the tuff in these calculations.

- If there is any structural fill, the fill properties will be taken as equivalent to those of the alluvium. If the fill characteristics are determined to be different than those of the alluvium, additional parametric studies may need to be carried out.

APPENDIX D
SHEAR WALL STRUCTURES DESIGN PROCEDURE

D1 SHEAR WALL AND SLAB DESIGN PROCEDURE

D1.1 PURPOSE

The methodology provided herein is to be used for ITS structures in conjunction with ACI 349-01 [DIRS 181670]. It may also be used for the design of non-ITS structures in conjunction with ACI 318-02 [DIRS 158832]. In either case, the design is based on the seismic and non-seismic demand forces and moments obtained from manual calculations or computer analyses. The purpose of Appendix D, however, is to define the methodology to be used for the design of the reinforced concrete shear walls and slabs for the project structures.

Different methodologies for the design of shear walls and slabs are permitted as long as these methods comply with the requirements of ACI 349-01 [DIRS 181670].

D1.2 METHOD

The design of shear walls and slabs is accomplished using the following steps:

Step 1. Analyze the structures for the seismic and non-seismic loads.

Step 2. Define horizontal and vertical cut sections.

Step 3. Using the project load combinations, determine the controlling design loads for the cut sections.

Step 4. Starting with a minimum reinforcement, determine the required horizontal, vertical, and out-of-plane shear reinforcement, in that order. During this step, if necessary, analyze the section under in-plane shear and bending moment using the Mathcad®, Excel or similar programs. Use uniformly spaced reinforcement whenever practical.

Step 5. When necessary, check the need, and design for boundary elements or transverse reinforcement at the boundaries.

D1.3 USE OF SOFTWARE

A general-purpose finite element program is used to obtain the cut section design forces. These forces include in-plane membrane, bending and shear forces. Out of plane bending and shear are generally determined on an element basis. The same computer program is used to determine the section forces in accordance with the project criteria for loads and load combinations.

Excel® is utilized in the computation of stresses and strains on the cross sections. Formulas used in the Excel® spreadsheets are common functions: cell additions, cell multiplications, and cell divisions. Checking of the spreadsheet calculations is done by checking the formulas in the cells and then completing the mathematical computations using a hand calculator.

Mathcad® may also be utilized to compute the stresses and strains at a cut section. All Mathcad® input values and equations are stated in the calculation. Equations used in the Mathcad® template are taken from this procedure or reinforced concrete design books. Checking of the Mathcad® template is done by using a hand calculator.

D1.4 ASSUMPTIONS

The only assumptions made in the design process are those that are routinely made in the design of reinforced concrete structures. These assumptions are given in the ACI Codes, and finite element analysis and reinforced concrete design books, and therefore are not repeated here. No other special assumption is made.

D1.5 CALCULATIONS

The procedure to be followed for the reinforced concrete design calculations is given in a step-by-step approach in Tables D-1 to D-3. The indicated steps are for guidance and the order of steps can be modified, as necessary. The code referred to in the last column is ACI 349-01 [DIRS 181670]. When this procedure is used for the design of non-ITS structures, appropriate references to ACI 318-02 [DIRS 158832] will be made.

Table D-1. Horizontal Cuts

Steps	Horizontal Cuts	Code Section
	Notation:	
	ϕ = Strength reduction factor	
	= 0.9 for flexure	9.3.2
	= 0.85 for out-of-plane shear	9.3.2
	= 0.60 for in-plane shear	9.3.4
	$A_{CV} = t_w\, l_w$ = bounded by web thickness and length of section in the direction of shear force considered (gross shear area)	21.0
	A_V = area of shear reinforcement within distance s_2	11.0
	d = distance to centroid of tension (can be taken as $0.8l_w$ for walls)	11.0
	f'_c = concrete specified compressive design strength	
	f_y = reinforcement yield strength	
	l_w = length of wall (or wall segment)	
	h_w = height of wall (or wall segment)	
	s_2 = spacing of horizontal reinforcement	
	t_w = thickness of wall	
	N_u = axial force (negative for tension)	
	V_n = nominal shear strength	
	V_u = required shear strength (demand)	
	$\alpha_C = 2$ when $h_w/l_w \geq 2.0$	
	$\alpha_C = 3 - ((h_w/l_w - 1.5)/0.5)$ when $1.5 \leq h_w/l_w < 2.0$	21.6.5.3
	$\alpha_C = 3$ when $h_w/l_w < 1.5$	
	ρ_n = required horizontal reinforcement, percent (defined as ρ_h in 11)	
	ρ_v = required vertical reinforcement in percent	
Step 1	Calculate and tabulate section resultants for cut sections	—
	In-plane shear:	
	In-plane bending:	
	Axial, force:	
	Out-of-plane bending (if used):	
	Out-of-plane shear (if used):	
Step 2	Check limiting in-plane shear strengths:	
	For walls: $V_{n\,max} = 8\, A_{cv} \sqrt{f'_c}$	21.6.5.6
	For wall segments or piers: $V_{n\,max} = 10\, A_{cv} \sqrt{f'_c}$	21.6.5.6
	Show: $V_u < \phi V_{n\,max}$	21.6.5.6
	If not, revise wall thickness.	
Step 3	Set initial horizontal and vertical reinforcement as:	
	$\rho_n \geq 0.0025$, $\rho_V \geq 0.0025$	11.10.9

Table D-1. Horizontal Cuts (Continued)

Steps	Horizontal Cuts	Code Section
Step 4	Horizontal Reinforcement: Step 4.1: Calculate in-plane shear capacity, V_n, for the initial reinforcement: Equation 21-7: $V_n = A_{CV} (\alpha_C \sqrt{f'_c} + \rho_{n\,min}\, f_y)$ where the concrete capacity is given by: $V_c = A_{CV}\, \alpha_C \sqrt{f'_c}$ If there is axial tension on the cross section, use Equations 11-31 and 11-33: $V_n = 3.3\sqrt{f'_c}(0.8\, t_w l_w) + N_u d/4l_w + 0.8(A_V f_y l_w)/s_2)$ where the concrete capacity (including the effect of tension) is given by: $V_c = 3.3\sqrt{f'_c}(0.8\, t_w l_w) + N_u d/4l_w$ Use the minimum of the two as the capacity (normally Equation 21-7). [NOTE: For slender walls Equation 11-32 in the code may control and should be checked.]	21.6.5.3 11.10.6 & 11.10.9
	Step 4.2: Calculate D/C ratio. Demand, D = V_u, Capacity, C = $\phi\, V_n$ If D/C $\leq$ 1.0, section is adequate with minimum reinforcement If D/C > 1.0, section is inadequate with minimum reinforcement; go to Step 4.3	—
	Step 4.3: Calculate the required reinforcement. Using the appropriate V_c from Step 4.1: $V_u = \phi\, V_n = \phi(V_c + V_s)$ $V_s = (V_u /\phi - V_c)$ $\rho_{n\,req'd} = V_s /(A_{cv}\, f_y)$ Determine the size and spacing of the horizontal reinforcement.	11.10.9.1
Step 5	Vertical Reinforcement: Step 5.1: Set minimum reinforcement ratio: If $h_w/l_w \leq 2.0$, use: $\rho_v \geq \rho_n$ If $h_w/l_w > 2.0$, use: $\rho_v = 0.0025 + 0.5\,(2.5 - h_w/l_w)(\rho_n - 0.0025) \leq \rho_n$	21.6.5.5 11.10.9.4
	Step 5.2: Simplified Axial Force and In-plane Moment Capacity Check Assume a compressive stress block of 0.10 l_w Calculate the allowable load on the compressive stress block using: $\phi\, P_n = 0.80\, \phi\, [0.85\, f'_c\, (A_g - A_{st}) + f_y\, A_{st}]$ with the reinforcement from Step 5.1. Calculate the maximum axial load on the compression block using: $P_{uc} = P_u/10 + M_u/0.9\, l_w$ *where* P_u = Maximum factored axial load taken from calculations, and M_u = Factored moment corresponding to P_u, taken from calculations. If $P_{uc} \leq \phi\, P_n$, the section is adequate; proceed to Step 5.5. If $P_{uc} > \phi\, P_n$, the section is inadequate; go to Step 5.3.	—

Table D-1. Horizontal Cuts (Continued)

Steps	Horizontal Cuts	Code Section
	Step 5.3: Strain-Compatible Section Analysis. Perform a strain-compatible section analysis using Mathcad® or Excel®, with the reinforcement from Step 5.1 and determine the stress and strain distribution on the section. In this calculation, use the controlling axial compression and the in-plane bending moment and, with the following assumptions: (1) linear strain distribution (plane sections remain plane) (2) a parabolic stress-strain distribution for concrete with a peak stress of f_c' at a strain of 0.002, and a peak limiting strain of 0.003 (see ACI 349-01 [DIRS 181670]): $f_c = 0.85\ [1 – (\varepsilon_c / \varepsilon_0 -1)^2]\ f_c'$ or $f_c = 0.85\ [2\varepsilon_c / \varepsilon_0 – (\varepsilon_c / \varepsilon_0)^2]\ f_c'$ where ε_c = concrete strain and ε_0= 0.002 Alternatively, an equivalent rectangular stress distribution may be used in these calculations. (3) An elasto-plastic, bi-linear stress-strain distribution for the reinforcing steel, with the peak stress at the yield strength (f_y) (i.e., when the reinforcement strain is equal to or greater than the yield strain, the stress is set equal to f_y).	10.2 & 10.3
	Step 5.4: Evaluate the analysis results from Step 5.3: WALLS WITH H_W/L_W < 2.0: If $\varepsilon_c \leq 0.003$, the section is adequate, If $\varepsilon_c > 0.003$, increase vertical reinforcement until $\varepsilon_c \leq 0.003$ WALLS WITH H_W/L_W > 2.0: If $\varepsilon_c \leq 0.002$, the section is adequate If $\varepsilon_c > 0.002$ but $\varepsilon_c \leq 0.003$, there are two options: Increase vertical reinforcement until $\varepsilon_c \leq 0.002$ Provide a boundary element (Step 6) Steps 5.3 and 5.4 are iterative and may even necessitate increasing the wall thickness. Once a solution is reached, determine the reinforcement size and spacing.	10.2 & 10.3 (See Appendix D-2)
	Step 5.5: Check available vertical reinforcement for out-of-plane moment. When the minimum reinforcement controls, determine the percentage of minimum reinforcement required by in-plane forces: Calculate the in-plane moment capacity corresponding to maximum concrete strain of ε_c = 0.002, with the actual axial force (i.e., use Mathcad® or Excel® spreadsheet), M_{max} Determine the percentage of reinforcement required for in-plane loads as $\rho_{n\ req'd} = M_u / M_{max}$ where M_u is actual in-plane moment Subtract $\rho_{n\ req'd}$ from the actual reinforcement; the balance is available for out-of-plane bending	
	Step 5.6: Determine the required additional vertical reinforcement (if any). Considering out-of-plane moments: Determine out-of-plane reinforcement required using the equation $M_u = \phi\ \rho_{req'd}\ f_y\ b\ d^2\ [1-0.59\ \rho_{req'd}\ f_y/f'_c]$ *or* $M_u = \phi\ A_s\ f_y\ (d-a/2)$ where *d* = effective depth of the section in inches, M_u is the out-of-plane design moment (axial force is ignored) Provide additional vertical reinforcement considering the results of Steps 5.3, 5.4, and 5.5.	—

Table D-1. Horizontal Cuts (Continued)

Steps	Horizontal Cuts	Code Section
Step 6	BOUNDARY ELEMENTS This step will be reached only if $h_w/l_w > 2.0$, and the option of boundary element is selected in Step 5.3. If required, provide boundary elements and transverse reinforcement.	21.6.1 21.4.4.1 & 21.4.4.3
Step 7	OUT-OF-PLANE SHEAR – Step 7.1: Calculate the nominal shear strength provided by the concrete: $V_c = 2\sqrt{f'_c}\ b\ d$ where *b* = width of section (usually unit width, 12 in.) *d* = effective depth of the section in inches.	11.3
	Step 7.2: Check demand-capacity ratio: D/C = $V_u / \phi V_c$ where V_u = the factored out-of-plane shear, $\phi = 0.85$ If D/C $\leq$ 1.0 No shear reinforcement is needed If D/C > 1.0 go to Step 7.3	11.1
	Step 7.3: Design for shear: consider the following options Review Equations 11-4 through 11-7 to assess if the allowable shear can be increased, If not provide shear reinforcement or increase wall thickness	11.3
Step 8	Shear Friction Shear-friction is an appropriate consideration for shear transfer across a plane where a weakness may exist. In shear wall structures, the only plane where shear-friction calculations should be performed would be the wall–basemat junction. Normal construction joints need not be checked as these joints are prepared to provide a monolithic construction. Step 8.1: Calculate the shear-friction capacity: $V_n = A_{vf} f_y \mu < 0.2 f_c' A_c$ where A_{vf} = Area of shear-friction reinforcement, (reinforcement in the compression zone less the reinforcement required for out-of-plane bending is available for shear-friction) μ = Coefficient of friction. Use 1.0 for shear-friction check at the wall–basemat junction where the surface is intentionally roughened. Step 8.2: Check adequacy of shear-friction reinforcement: $V_c \leq \phi V_n$ Step 8.3: Net tension check: If there is net tension on the cross section, subtract the reinforcement required for the net tension from the area of vertical reinforcement before calculating the shear-friction capacity.	—
Step 9	Tabulate reinforcement requirements and D/C ratios. Also, check uniformity of the overall reinforcement sizes and spacing.	—

NOTE: Code sections are referenced from ACI 349-01 [DIRS 181670], unless noted otherwise.

Table D-2. Vertical Cuts

Steps	Vertical Cuts	Code Section
Step 1	Calculate and tabulate controlling section resultants for cut sections Axial, force: Out-of-plane bending: Out-of-plane shear:	—
Step 2	Determine the required horizontal reinforcement. Considering the out-of-plane moments: Required reinforcement is calculated from the equation: $M_u = \phi\ \rho_{req'd}\ f_y\ b\ d^2\ [1\text{-}0.59\ \rho_{req'd}\ f_y/f'_c]$ where d = effective depth of the section in inches, M_u is the out-of-plane design moment If $\rho_{req'd}$ calculated from the previous equation is less than the existing horizontal reinforcement in one curtain, no additional rebar is needed. Otherwise, provide additional reinforcement so that the total rebar will be adequate for out-of-plane bending. If there is net horizontal tension, provide additional reinforcement to resist the tension. This additional reinforcement may be distributed to either face.	—
Step 3	Check out-of-plane shear following a process similar to the horizontal cut section (Table D-1).	—
Step 4	Tabulate reinforcement requirements and D/C ratios. Also, check uniformity of the overall reinforcement sizes and spacing	—

NOTE: Code sections are referenced from ACI 349-01 [DIRS 181670], unless noted otherwise.

Table D-3. Special Provisions for Torsion

Steps	Special Provisions For Torsion	Code Section
Step 1	**Identify Significant Torsion Cases**. Torsional moments may be significant in some cases, especially for slabs. Identify elements/sections with significant torsional moment, M_{xy}. (See Appendix B3 for a discussion on torsion for wall and slab design.)	—
Step 2	**Check code threshold for neglecting torsion**: $\mathbf{T_n = \sqrt{f'_c}\ (A_{cp}^2/p_{cp})}$ where A_{cp} = concrete area in 11.6.1 p_{cp} = perimeter of concrete cross section in 11.6.1 If $\mathbf{T_u \leq \phi T_n}$, torsion can be neglected If $\mathbf{T_u > \phi T_n}$, consider torsion as described in Step 3.	11.6
Step 3	**Design for Torsion**. If torsion is significant, the method advocated by MacGregor (1997 [DIRS 130532]) is a reasonable approach. The process is as follows: • Add the torsional moment (absolute value) to out-of-plane bending in each direction • Re-design the section using the total moment	—

NOTE: Code sections are referenced from ACI 349-01 [DIRS 181670], unless noted otherwise.

D1.6 RESULTS

The calculations performed using the procedure given herein will be documented separately using project procedures.

D2 BOUNDARY ELEMENTS IN WALLS AND SLABS

D2.1 INTRODUCTION

- Boundary elements in shear walls and slabs are required to confine the concrete when the compressive strain exceeds a critical value during the design basis earthquake. In walls, this occurs as a result of cantilever action where the wall is subject to in-plane bending due to lateral loads and axial compression due to gravity loads. In slabs, the axial load is usually small and concrete compression is due to in-plane bending of the slab as a deep beam, supported at the two ends. In either case, the significant parameter is the magnitude of the compressive stresses and strains in the concrete, which determine the need for boundary elements. In this appendix, both walls and slabs are treated similarly.

- Boundary element requirements for YMP ITS reinforced concrete structural walls are evaluated in accordance with the applicable codes listed in the project structural design criteria (BSC 2007 [DIRS 179641], Section 4.2.11.4.1). The structural design criteria stipulate that ITS reinforced concrete structures be designed in accordance with ACI 349-01 [DIRS 181670].

- ACI 349-01 [DIRS 181670], 21.6.1, recognizes the inherent strength and stiffness of low aspect ratio shear walls and diaphragms and exempts the walls with aspect ratios (h_w/l_w) less than 2.0 from the provisions of 21.6.6 - boundary elements for structural walls. By extension, diaphragms with aspect ratios less than 2.0 are exempt from the provisions of 21.6.7. These interpretations have been incorporated into the design process for the ITS structure shear walls and diaphragms.

  Walls and diaphragms with h_w/l_w ratios greater than 2.0 may be evaluated for boundary elements in accordance with ACI 349-01 [DIRS 181670], 21.6.6 and 21.6.7.

  ACI 349-01 [DIRS 181670], 21.6.6.1, requires that special boundary elements are required at boundaries and edges around openings of structural walls where the maximum extreme fiber compressive stress, corresponding to factored forces including earthquake effect exceeds $0.2f_c'$. This stress is calculated based on the gross uncracked concrete section. The compressive stress of $0.2f_c'$ is a carry over from ACI 318-02 and is used as an index value to determine the requirements for boundary elements and does not necessarily describe the actual state of stress that may develop at the critical section. The Uniform Building Code (ICBO 1997 [DIRS 100323]), has replaced the $0.2f_c'$ check for wall boundary elements with other provisions.

  The YMP design team has developed an alternative method to determine boundary element requirements for the ITS shear walls and diaphragms. This method involves the computation of concrete strains for the design forces utilizing the stress-strain diagrams for concrete and reinforcing steel.

  The first part of this write up reviews the current code provisions in ACI 349-01 [DIRS 181670], 21.6.1, 21.6.6, and 21.6.7, for boundary elements at shear walls and diaphragms, and discusses the limitations of their applicability to the ITS shear wall structures. This is followed by a review and justification for the alternative method proposed by the YMP design team to evaluate boundary element requirements at shear walls.

D2.2 CODE INTERPRETATION AND APPLICATION TO ITS STRUCTURES

- ACI 349-01 [DIRS 181670], 21.6.1. ACI 349 adopted a revised version of ACI 318-89 [DIRS 167041], Chapter 21, in 1992. To eliminate needless transverse reinforcement providing concrete confinement, walls with height to length ratios (h_w/l_w) less than 2.0 were exempted from boundary elements. ACI 349-01 [DIRS 181670], Section 21.6.1, requires that for shear walls with h_w/l_w ratios less than 2.0, the provisions of Section 21.6.6 (i.e., boundary elements for structural walls may be waived).

  ACI 349-01 [DIRS 181670], Section R21.6, justifies the above exemption from boundary elements by considering the behavior of the low-rise shear walls, which, during earthquakes, respond predominantly in shear, with insignificant bending deformation. Because boundary elements are required to provide adequate confinement of concrete in the compression zone, shear walls with aspect ratios (h_w/l_w) less than 2.0 do not require them.

  The project concurs with the above provision and its justification and, therefore, the shear walls with aspect ratios less than 2 are not provided with boundary elements.

- ACI 349-01 [DIRS 181670], Section 21.6.6.1, requires that structural walls with h_w/l_w ratios greater than 2.0 shall have special boundary elements at boundaries and edges around openings of structural walls where the maximum extreme fiber compressive stress, corresponding to factored forces including earthquake effect, exceeds 0.2 f_c'. The compressive stress is computed using the gross concrete section properties.

  Commentary to ACI 349-01 [DIRS 181670], Section 21.6, does not address this provision. However, as noted earlier, this provision is based on ACI 318-99 [DIRS 118294], Section 21.6.6.3, and the commentary to that section states "The compressive stress of 0.2 f_c' is used as an index value and does not necessarily describe the actual state of stress that may develop at the critical section under the influence of the actual inertia forces for the anticipated earthquake intensity." ACI 318-99 [DIRS 118294], Section 21.6.6.3, is applicable to non-ITS structures. Consequently, it reflects a design where a response modification factor, R, is used to establish the design seismic forces. The expected, elastically calculated seismic forces are divided by the R factor for determining the seismic design forces.

  For a shear wall structure designed by the Uniform Building Code (ICBO 1997 [DIRS 100323]), the wall design forces are established by dividing the base shear force with the "response modification factor," R, equal to 4.5 or 5.5. Therefore, the corresponding wall seismic design forces are about 20% of the expected elastically calculated value. For concrete stresses computed using these reduced loads, the $0.2f_c$' stress check may be an appropriate check for boundary element requirements. However, this check would be inappropriate for shear walls designed on the basis of R = 1.0. Therefore, ACI 318-99 [DIRS 118294], Section 21.6.6.3, is not appropriate for nuclear structural shear walls for which elastic response to Safe Shutdown Earthquake shaking is required.

The SEAOC blue book provisions (SEAOC 1996 [DIRS 177779]), that have been adopted in the Uniform Building Code (ICBO 1997 [DIRS 100323]), have eliminated the 0.2 $f_c^{'}$ check for boundary elements and uses provisions that include moment curvature demand keyed to maximum concrete strain of 0.003 to check the need for boundary elements.

- ACI 349-01 [DIRS 181670], 21.6.7, gives direction on how the boundary elements for structural diaphragms should be proportioned. It does not provide a stress criteria or limitation on length–width ratio. Nor is there a commentary for these diaphragms. However, as stated in ACI 349-01 [DIRS 181670], 21.6.1, the provisions that are applicable to walls are also applicable to structural diaphragms. Therefore, the strain criteria proposed for the walls in Section 3 is judged to be equally applicable to slab diaphragms, regardless of the span length–width ratio. However, consistent with the walls, the check for boundary elements is not necessary if the length–width ratio of the structural diaphragm is less than 2.0.

D2.3 PROJECT METHODOLOGY

- In light of the discussions above, the proposed methodology to evaluate walls, piers and structural diaphragms for boundary element is as follows:

  - Determine h_w/l_w for the wall. In accordance with ACI 349-01 [DIRS 181670], 21.6.1, if h_w/l_w is less than 2.0, boundary elements are not required. Evaluation of piers within a wall is not necessary provided that the wall shear deformation will control the drift of wall pier(s) and large flexural compression will not develop in the pier(s) due to flexure.

  - For walls with h_w/l_w greater than 2.0, and piers within a wall, and diaphragms, compute the maximum compressive stress using gross concrete section properties and compare this value with $0.2f_c^{'}$. In accordance with ACI 318-99 [DIRS 118294], Section 21.6.6.3, if the maximum compressive stress is less than 0.2 $f_c^{'}$, boundary element is not required. In the review above, it was concluded that applying this provision to walls designed using R = 1.0 is very conservative. However, continued use of this provision is acceptable as a means of filtering out walls and piers that will not require boundary elements.

- Elements with h_w/l_w greater than 2.0, and walls, piers and diaphragms with maximum compressive stress in excess of 0.2 $f_c^{'}$, will be further evaluated for boundary element requirements in accordance with the following procedure:

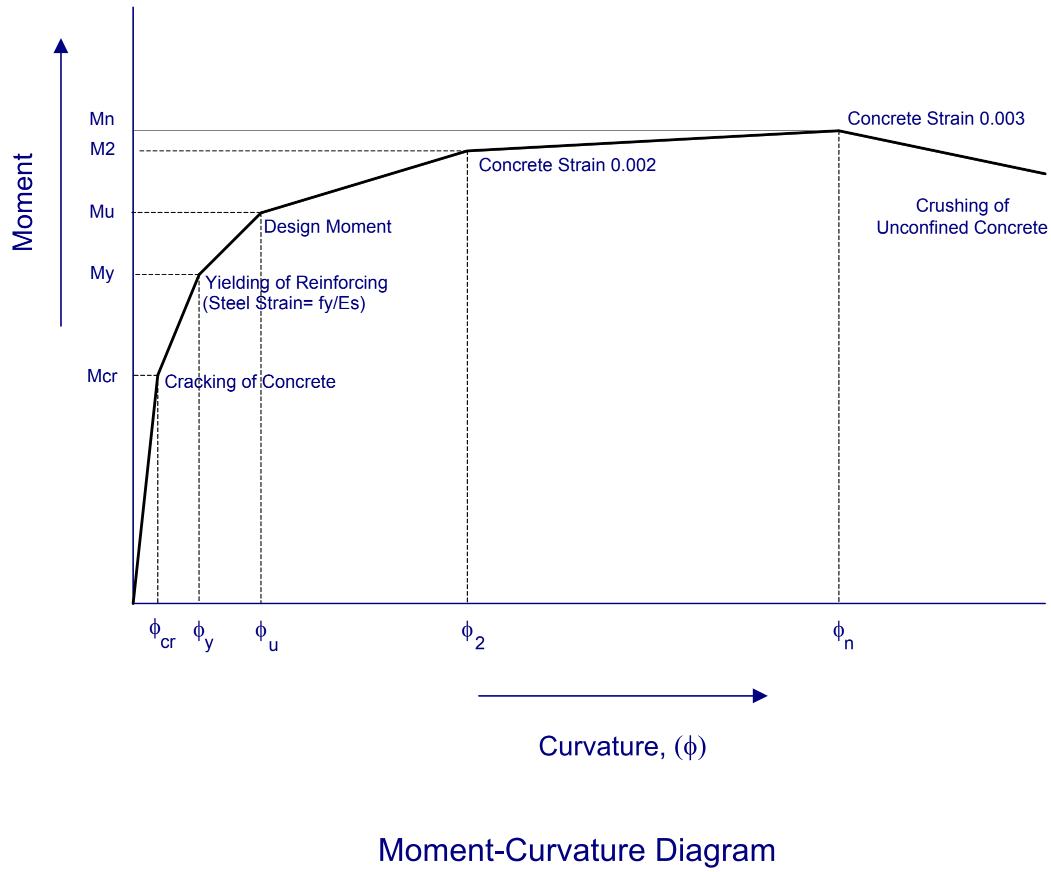
  - Analyze the cross section taking into account concrete cracking to determine the maximum concrete strain,

- If the maximum concrete strain is less than 0.002, the section is adequate,
- If the maximum concrete strain is greater than 0.002, either provide boundary elements or revise the design so that the maximum concrete strain is less than 0.002.

Justification for this approach is based on the fact that unconfined concrete can resist strains in the order of 0.002 without any deterioration. The SEAOC blue book (SEAOC 1996 [DIRS 177779]), states that tests show spalling of unconfined concrete initiates at strains on the order of 0.004. In ACI 318-99 [DIRS 118294], Sections 10.3.2, 0.003, is used as the limiting strain. Thus, restricting the design strains to 0.002 will provide a minimum safety factor of 1.5 to reach the conservative limiting value of 0.003 and a factor of 2 to reach 0.004. These safety factors are considered adequate, especially in view of the fact that the seismic design basis does not allow inelastic behavior under the design basis earthquake (i.e., the R-factor of the Uniform Building Code (ICBO 1997 [DIRS 100323]) is set to 1.0).

The conservatism associated with the concrete strain limit can be better illustrated by reviewing the typical moment-curvature diagram as shown in Figure D-1. In this figure, curvature of the cross section is plotted against the applied moment for a given axial load. Various stages of the behavior of the typical wall are indicated on this diagram together with the maximum concrete strains. Under the design loads, the maximum concrete strain is normally less than the strain limit of 0.002. If moment is increased (while the axial load is kept constant) the concrete strain limit of 0.002 will be reached next. At this point, the curvature is in the order of twice the curvature under the design loads. If the applied moment is further increased, the maximum concrete strain will eventually reach 0.003 and concrete (which is unconfined) may start spalling.

The area under the moment-curvature diagram is indicative of the energy dissipation capacity of the wall or the slab. As can be seen from Figure D-1, there is a significant reserve energy capacity between the design level and the level at which maximum concrete strain reaches 0.003. This figure shows that the proposed design methodology adequately protects against unacceptable behavior under the seismic loads and provides more than adequate factor of safety against concrete spalling under the beyond design basis earthquake forces.



NOTE: The moment curvature diagram for a typical shear wall shows a minimum safety factor of 1.5 exists between curvatures corresponding to concrete strain of 0.002 and the code limitation value of 0.003, thus eliminating the need for a boundary element.

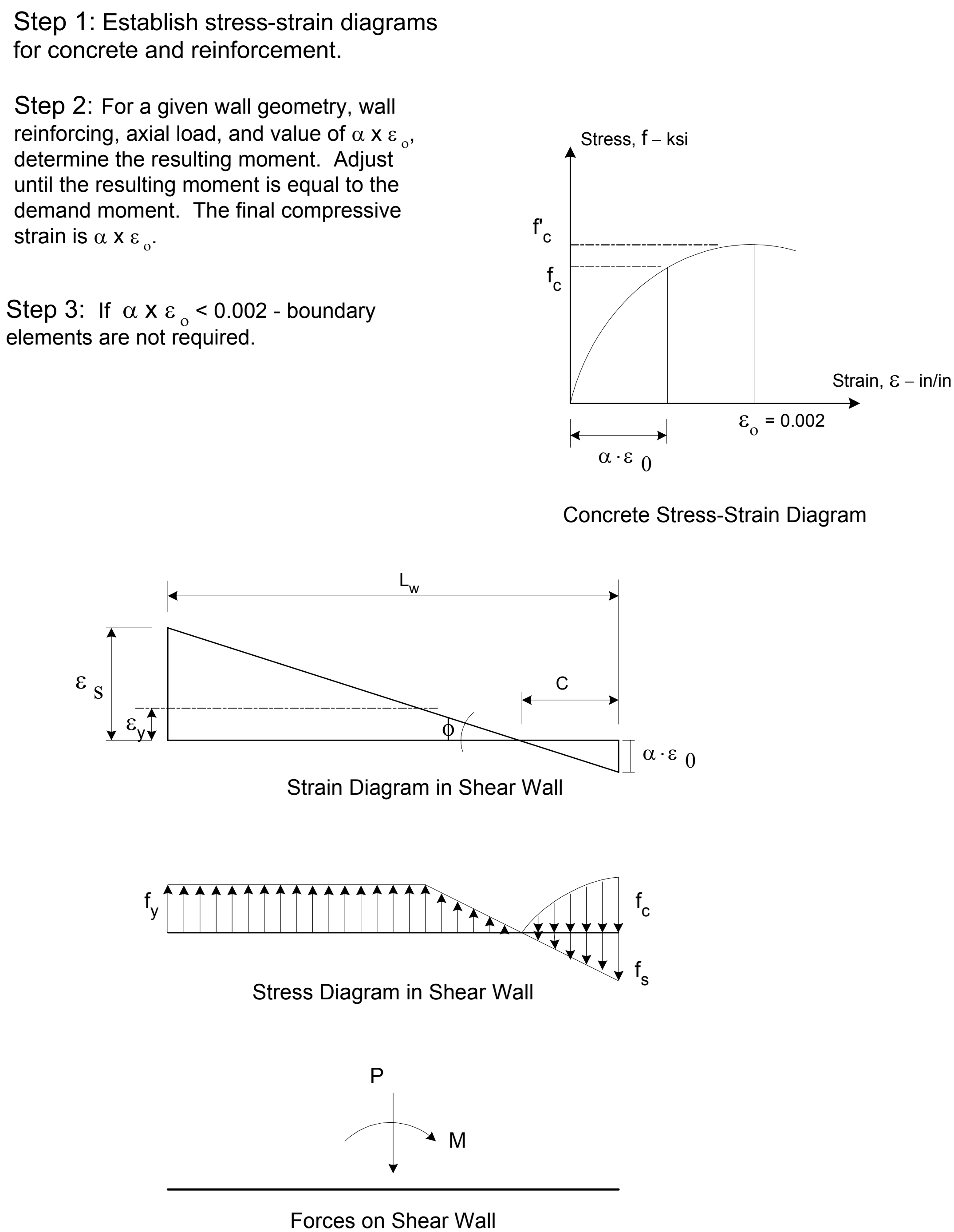
Figure D-1. Typical Moment-Curvature Diagram

Therefore, it is concluded that the proposed system of design to determine the need for boundary elements is adequate. If the calculated maximum concrete strain exceeds 0.002 under the design basis earthquake in combination with other applicable loads, then boundary elements will be provided or the section will be re-designed so that the maximum concrete strain is less than 0.002.

Analysis of the cross section is achieved either by the Mathcad® software or by an Excel® spreadsheet. In this analysis, the following approximations are made:

- Linear strain distribution (plane sections remain plane)
- A parabolic stress-strain distribution for concrete with a peak stress of f_c' at a strain of 0.002, and a peak limiting strain of 0.003.
- An elasto-plastic, bi-linear stress-strain distribution for the reinforcing steel, with the peak stress at the yield strength (f_y).

Figure D-2 provides more details regarding strain-compatible section analysis.



NOTE: Determination of the maximum compressive strain in concrete for the design forces.

Figure D-2. Strain-Compatible Section Analysis

D2.4 SUMMARY

Boundary elements for shear walls are required to confine the concrete in portions of structures where the peak compressive strain may exceed a limiting value of 0.003 in ACI 318-99 [DIRS 118294] during the design basis earthquake.

Application of the ACI 349-01 [DIRS 181670], 21.6.1, provision ($h_w/l_w < 2.0$) to eliminate boundary elements is appropriate. This provision has been incorporated in the design of ITS shear wall structures.

The ACI 349-01 [DIRS 181670], 21.6.6.1, provision, which requires that boundary elements shall be provided when the maximum extreme fiber compressive stress exceeds $0.2f_c'$, is too conservative for walls designed with a response modification factor of R = 1.0. Since this is a very conservative check, the project team may continue to use it as a filter to eliminate boundary elements at walls and piers.

Any walls and piers where boundary elements cannot be eliminated using the checks in ACI 349-01 [DIRS 181670], 21.6.1 and 21.6.6.1, must be evaluated using the approach discussed in this write-up. The same principles are applicable to the design of structural diaphragms.

Code ACI 349-06, section 21.7.6.2 has incorporated the 0.002 strain limitation in determining the need for boundary elements. This change is based on technical evaluations similar to what are presented in this paper.

D3 DESIGN OF WALLS AND SLABS FOR TORSION

D3.1 BACKGROUND

Refined analysis of structures by finite element models have brought about issues that were not generally dealt with in non-ITS design. The M_{xy} moment for plate elements in reinforced concrete is an example. This moment occurs for the equilibrium of the cross section and is actually a "torsional moment" (or twisting moment) on the cross section. This torsion is resisted by "shear flow" and it is inappropriate to approximate its effects as linear out-of-plane shear (triangular distribution) on the cross section.

Torsion is significant for frame structures where geometry may result in torsional loads. For example, a girder, which supports integral beams on one side, will be subject to significant torsion and the beam design must consider this torsion in order to preclude premature failure. Typical cases where torsion may be significant are illustrated in the commentary to ACI 318-02 [DIRS 158832].

In the case of shear wall structures, torsion on the wall and slab cut sections is normally insignificant. That is because the gravity loads are resisted mainly by out-of-plane shear, bending and axial loads, and seismic loads are resisted by in-plane shear and bending. As a result, little torsional moment is generated in the shear wall structures. For this reason the torsional moment has generally been ignored in traditional shear wall structure design process.

D3.2 CODE PROVISIONS

ACI 318-02 [DIRS 158832] deals with torsion. The basic philosophy of design is to neglect small amounts of torsion (termed "threshold torsion"). For nonprestressed members, the threshold torsion is approximately 25% of the cracking torque. It has been calculated and confirmed by tests that the reduction in shear capacity due to neglecting torsion, in the order of 25% of the cracking torque, is about only 3% (ACI 318-02 [DIRS 158832], R11.6.1) and therefore it is not significant.

If the calculated torsion exceeds the threshold value, then the section (e.g., beams) must be designed to resist the torsion along with the direct shear. Provisions for design of beam shear and torsion are given in the code.

Similar explicit criteria do not exist in the code for slabs and walls, probably for the reason that these effects are not significant, except for special cases. However, it is reasonable to use the same threshold value for consideration of torsion in the design of slabs and walls.

D3.3 PROPOSED APPROACH

Since torsion is generally insignificant for walls and slabs, a torsion check need not be made a routine part of the design process. Instead, selective consideration of elements or members where torsion has been observed to be significant is sufficient. With this philosophy, the following step-by-step approach is recommended.

Step 1. Design the walls and slabs without the consideration of torsion.

Step 2. Review the cut sections (or elements) to identify where torsion is most significant.

Step 3. Calculate the threshold torsion for the cut section (or elements) using the provisions of ACI 318-02 [DIRS 158832], 11.6.1, to determine whether torsion should be explicitly incorporated into the design.

Step 4. Compare the calculated torsion for the cut section (or element) with the threshold torsion. If the calculated torsion is less than the threshold torsion, no further action is required.

Step 5. If the calculated torsion is greater than the threshold torsion, consider the torsion in design. The method advocated by MacGregor (1997 [DIRS 130532]) is an acceptable approach. In this method, the torsion (M_{xy}) is added to out-of-plane bending moments in each direction to determine the total required reinforcement. Then the section is designed for the total bending moment. The method of calculating out-of-plane shear from the twisting moment by triangular distribution is not correct and should not be used.

Step 6. Once the total required reinforcement is determined, and out-of-plane shear reinforcement is not required due to direct shear (as it should be in most cases), design is complete. There is no need to consider the effect of torsional shear on the direct shear stress in the out-of-plane direction. This is justified since adequate reinforcement has been provided in both in-plane directions to preclude torsional cracking.

Step 7. In the rare cases where both torsion is significant and that out-of-plane direct shear requires reinforcement, a more detailed design process is necessary taking into consideration of all forces. Alternatively, section thickness may be increased to preclude this condition from happening.

D3.4 SUMMARY AND CONCLUSIONS

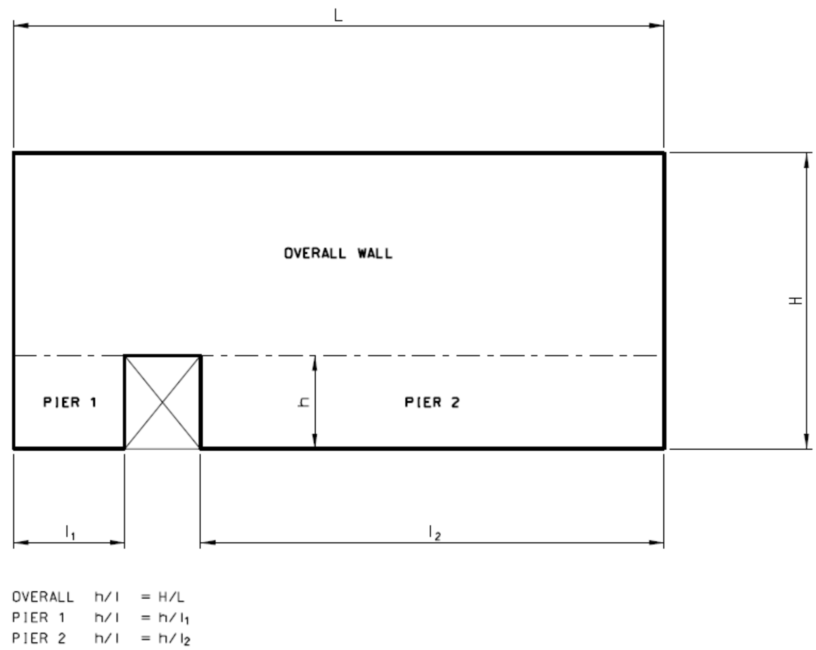
Torsion is normally insignificant for slabs and walls unless there are unusual geometric configurations and/or loading conditions. Therefore, it is not necessary to incorporate the torsion into the routine wall and slab design process.

ACI 318-02 [DIRS 158832], 11.6.1, provides a threshold limit on torsion, below which consideration of torsion may be neglected in member design. Using this provision, it may be demonstrated that torsion can be neglected in most cases of wall and slab design.

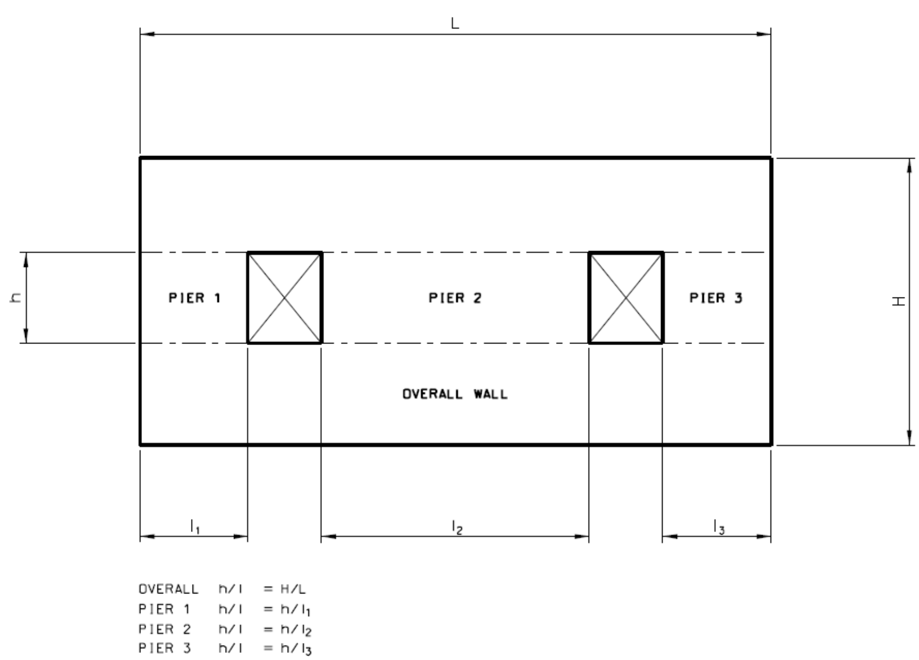
In cases where torsion cannot be neglected, a step-by-step approach is provided to check such cases and include the torsion in the design process.

D4 GUIDANCE ON DETERMINATION OF H/L_W RATIOS

Figure D-3 provides guidance on computing H/L_W ratios. This information must be adjusted for specific walls and openings on a case-by-case basis.



Typical Wall with One Opening



Typical Wall with Two Openings

Figure D-3. Typical Wall H/L_W Ratios

D5 SHEAR WALL DESIGN PROCEDURE FOR TIER #1

The procedure used to design reinforced concrete shear walls is given in a step-by-step approach in Sections D5.1 through D5.2. The code referred to throughout Section D5 is ACI 349-01 [DIRS 181670] and the following notations are used:

NOTATION

		Code Section
ϕ	Strength reduction factor	
	0.9 for flexure	9.3.2
	0.85 for out-of-plane shear	9.3.2
	0.60 for in-plane shear	9.3.4
	0.85 for shear across a joint	9.3.4
	0.9 for net tension	9.3.2
	0.7 – 0.9 for combined flexure and axial load	9.3.2
A_{cv}	area bounded by web thickness and length of section in the direction of the shear force considered (gross shear area) = $t_w * l_w$	21.0
d	distance to centroid of tension (taken as $0.8l_w$ for walls)	11.0 and 11.10.4
f'_c	concrete compressive design strength (5,000 psi for this calculation)	
f_y	reinforcing steel yield strength (60 ksi for this calculation)	
l_w	length of wall or wall segment	21.0
h_w	height of wall or wall segment	21.0
t_w	thickness of wall segment	
F_t	axial demand force in tension	
F_c	axial demand force in compression	
V_u	in-plane shear demand force	
M_z	in-plane demand moment (moment resulting from in-plane shear forces)	
V_z	out-of-plane demand shear force	

NOTATION (CONTINUED)

		Code Section
M_y	out-of-plane demand moment (moment resulting from out-of-plane shear forces)	
A_h	out-of-plane design acceleration (in g's)	
H	height of wall between floors (diaphragms)	
V_c	nominal shear strength provided by concrete	11.0
V_n	nominal shear strength	11.0
V_s	nominal shear strength provided by shear reinforcement	11.0
ε_c	concrete strain	
ε_s	reinforcing steel strain	
d'	concrete cover to rebar centerline	
α_c	coefficient defining the relative contribution of concrete strength to wall strength	21.6.5.3
ρ_n	required horizontal reinforcing, percent.	
ρ_v	required vertical reinforcing, percent.	

D5.1 SHEAR WALL DESIGN PROCEDURE

This section describes the methodology to be used in designing shear walls on the YMP per the requirements of ACI 349-01 [DIRS 181670].

Horizontal reinforcing is designed to satisfy the code requirements based on in-plane shear forces.

Vertical reinforcing is designed to satisfy the following code requirements:

- Minimum reinforcing based on horizontal reinforcing requirements
- Reinforcing required for shear friction considering the net tension force in combination with in-plane shear force
- Reinforcing required for axial forces in combination with in-plane and out-of-plane bending moments.

The process steps used to perform these design requirements are described in detail in Section D5.1.1.

D5.1.1 Process Steps

Code Section

Compute controlling section forces for shear wall or shear wall segment (pier). NA

Controlling section forces are given in the template, D5.2.

Out-of-plan shear forces and moments are computed in the spreadsheet as:

$$V_z = A_h * t_w * 0.15 * \frac{H}{2} \qquad \text{(0.15 = concrete unit weight – kcf)}$$

$$M_y = A_h * t_w * 0.15 * \frac{H^2}{8}$$

Step 1 Compute limiting in-plane shear strength of walls / piers.

For walls: $V_{n\max} = 8A_{cv}\sqrt{f'_c}$ 21.6.5.6

For individual wall segments or piers: $V_{n\max} = 10A_{cv}\sqrt{f'_c}$ 21.6.5.6

Check $V_u \leq \phi V_n$ 11.1

Increase wall thickness for walls not meeting the above criteria.

Step 2 Compute horizontal reinforcing requirements.

2a: Check ACI 349-01 [DIRS 181670] equation 21-7 requirements. 21.6.5.3

$$V_n = A_{cv}(\alpha_c \sqrt{f'_c} + \rho_n f_y)$$

Compute $V_c = A_{cv}\alpha_c \sqrt{f'_c}$

Compute $V_s = \dfrac{V_u}{\phi} - V_c$ 11.1.1

Compute reinforcing required $\rho = \dfrac{V_s}{(A_{cv})(f_y)}$

Compare computed reinforcing requirements with the minimum reinforcing requirements $\rho_{\min} = 0.0025$, the larger value governs. 11.10.9.2

2b: Check ACI 349-01 [DIRS 181670] equation 11-31 requirements. 11.10.6

Compute $V_c = 3.3\sqrt{f'_c}hd + \dfrac{F_t d}{4l_w}$

(note the sign for F_t is negative in tension)

Compute $V_s = \dfrac{V_u}{\phi} - V_c$ 11.1.1

Compute reinforcing required $\rho = \dfrac{V_s}{(0.8l_w t_w)(f_y)}$

NOTE: $0.8l_w$ is the value taken for d in accordance with section 11.10.4 11.10.4

Compare computed reinforcing requirements with the minimum reinforcing requirements $\rho_{\min} = 0.0025$. The larger value governs. 11.10.9.2

2c: Check ACI 349-01 [DIRS 181670] equation 11-32 requirements

Compute $V_c = \left[0.6\sqrt{f'_c} + \frac{l_w\left(1.25\sqrt{f'_c} + 0.2\frac{F_t}{l_w h}\right)}{\frac{M_u}{V_u} - \frac{l_w}{2}} \right] hd$ 11.10.6

When $\frac{M_u}{V_u} - \frac{l_w}{2}$ is less than zero these requirements do not apply. 11.10.6

Compute $V_s = \frac{V_u}{\phi} - V_c$ 11.1.1

Compute reinforcing required $\rho = \frac{V_s}{(0.8 l_w t_w)(f_y)}$

NOTE: $0.8l_w$ is the value taken for d in accordance with section 11.10.4 11.10.4

Compare computed reinforcing requirements with the minimum reinforcing requirements $\rho_{\min} = 0.0025$, the larger value governs. 11.10.9.2

2d: Compute the horizontal reinforcing requirements by selecting the larger of the three values computed in steps 2a, 2b, and 2c.

2e: Select the horizontal shear reinforcing to be provided such that:

$A_s(provided) \geq A_s(required)$

2f: Compute the demand / capacity ratio for horizontal shear requirements.

Step 3 Compute vertical reinforcing requirements.

3a: Determine minimum vertical shear reinforcing requirements

For $\frac{h_w}{l_w} \leq 2$ $\rho_v = 0.0025 > \rho_n$ 21.6.5.5

For $\frac{h_w}{l_w} > 2$ $\rho_v = 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_n - 0.0025)$ 11.10.9.4

3b: Compute vertical reinforcing requirements considering shear friction.

In-plane and out-of-plane bending moments do not influence the shear friction capacity of a wall. The reduction in shear friction capacity of a shear wall resulting from the tension component of a bending moment is offset by the increase in shear friction capacity resulting from the compression component of the bending moment. Therefore, only the net tension force and in-plane shear acting on a section are considered in evaluating the shear friction reinforcing requirements.

Compute in-plane shear per foot of wall $= \dfrac{V_u}{l_w}$

Compute transverse shear per foot of wall = V_z

Compute resultant shear, V_r, of values computed above.

Compute the limiting shear friction capacity:

$V_n = 0.2 f'_c A_c \leq 800 A_c$ 11.7.5

Check $\phi V_n > V_r$ if not increase wall thickness.

Compute shear friction reinforcing requirements:

$V_n = A_{vf} * f_y * \mu$ 11.7.4.1

μ is taken as 1.0 for concrete placed against hardened concrete with the surface intentionally roughened as specified in 11.7.9

Compute $A_{vf} = \dfrac{V_r}{2 * \phi * \mu * f_f}$ where A_{vf} is the vertical steel required on each face of the section.

Compute reinforcing steel required for the net tension force on the section.

$$A_t = \frac{F_t}{2 * \phi * f_y * l_w}$$

Compute total vertical steel required per face on a per foot basis as:

$$A_v = A_{vf} + A_t$$

Compute ρ_v required as: $$\rho_v = \frac{2A_v}{12 * t_w}$$

3c: Compute the required vertical reinforcing by selecting the larger of the values computed in steps 3a and 3b.

3d: Select vertical steel reinforcing to be provided such that

$$A_s(provided) \geq A_s(required)$$

3e: Perform strain-compatible section analysis for combined bending and axial loads on the wall section based on the reinforcing selected in step 3d.

Limit concrete strain, ε_c, to 0.002.

Considering the wall as a beam with distributed reinforcing:

For the tension plus bending case use a trial and error solution (goal seek option in Excel®) to solve for the neutral axis location such that we have an equilibrium condition.

Using the neutral axis location compute the corresponding bending moment capacity.

Perform a similar calculation for the compression plus bending loading condition.

3f: Compute a demand capacity ratio as:

M_z / M_u where M_z is the applied in-plane moment and M_u is the minimum of the moment capacities computed above in the tension plus bending and compression plus bending conditions.

3g: Compute reinforcing steel requirements for out-of-plane bending loads.

3h: Compute total vertical steel reinforcing requirements considering axial loads, in-plane bending loads and out-of-plane bending loads based on the results of steps 3f and 3g.

Step 4 Check requirements for boundary elements

Compute the h_w / l_w for the wall or wall segment being considered.

For walls or wall segments with $h_w / l_w \leq 2$ no boundary elements are required per ACI 349-01 [DIRS 181670], 21.6.1 21.6.1

For walls or wall segments with $h_w / l_w > 2$ the concrete compressive strains were limited to 0.002 in step 3e. No boundary elements are required.

Step 5 Check out-of-plane shear requirements

Compute the transverse shear capacity of the concrete:

$$V_c = 2\sqrt{f'_c}bd \qquad 11.3.1$$

Compute demand capacity ratio as:

$$D/C = \frac{V_z}{\phi V_c}$$

For D/C <1.0 wall thickness is OK, no shear reinforcement required.

For D/C >1.0 increase wall thickness or provide shear reinforcement.

Step 6 Summary

Summarize the results found in steps 1 through 5.

D5.2 SHEAR WALL DESIGN TEMPLATE

The spreadsheets (Figures D-4a through D-4c), provided in this section, are based on the methodology defined in Section D5.1. No special assumptions are made. The following spreadsheets represent a typical wall:

WALL	1	SHEAR WALL	1-a

Design Loads			Shear Wall Section Properties			Concrete & Rebar Properties		
Axial Force (-Tension)	**Ft =**	-3130 kips	**Height of Wall (segment)**	**h_w =**	15 ft	**Concrete Design Strength**	**f'c =**	5000 psi
Axial Force (+Comp)	**Fc =**	16062 kips	**Ht of Wall Between Floors**	**H =**	32 ft	**Concrete Strain**	**ε_c =**	0.002
In plane shear	**Vu =**	16123 kips	**Length of Wall (Segment)**	**l_w =**	67.5 ft	**Rebar Yield Strength**	**fy =**	60 ksi
In plane Moment	**Mz =**	131765 ft-kips	**Thickness of Wall**	**t_w =**	6 ft	**Rebar Yield Strain**	**ε_y =**	0.002
			Shear Area of Wall (Segment)	**Acv = l_w*t_w =**	405 ft^2	**Min Steel Required**	**ρ_{min} =**	0.0025
Design Acceleration	**A_h =**	1.36 g				**Concrete Cover**		6 in
Out of plane shear	**Vz =**	19.58 kips/ft (A_h*t_w*150/1000)*H/2						
Out of plane Moment	**My =**	156.67 ft-kips/ft (A_h*t_w*150/1000)*H^2/8						

1.0 Check Shear on gross section - ACI 349: 21.6.5.6

Nominal Shear Capacity = 8*Acv*(f'c) $^{1/2}$ — Vn (kips) = 32991

Factored Shear Load = Vu / ϕ — Vu/ϕ (kips)= 26871.7
(ϕ = .6 per ACI 349 - 9.3.4)

Demand Capacity Ratio
Check Code 21.6.5.6 Demand/Capacity Ratio — D/C = (Vu/ϕ)/Vn — D/C = 0.81 **SHEAR WALL THICKNESS OK**

2.0 Horizontal Reinforcing Requirements

2.0a) ACI 349 - 21.6.5.3 Requirements

Determine α_c : h_w / l_w = 0.22 — α_c = 3 — α_c=3 for h_w/l_w<1.5, α_c varies linearly from 3 for h_w/l_w=1.5 to 2 for h_w/l_w=2.
Determine Concrete Shear Capacity Vc=Acv*α_c*(f'c) $^{1/2}$ — Vc = 12371.5 kips
Determine Shear Carried by Steel Vs=Vu/ϕ -Vc — Vs = 14500.1 kips

Determine Required Shear Reinforcing ρ=Vs/(fy*Acv) — ρ = 0.0041

21.6.5.3 Shear Reinforcing Requirements — **ρ_n = 0.0041** > .0025 MINIMUM STEEL DOES NOT GOVERN

2.0b) ACI 349 - 11.10.6 Equation 11-31 Requirements
Determine Concrete Shear Capacity — Vc = 10261.0 kips — Vc=3.3*(f'$_c$)$^{0.5}$*(0.8*t_w*l_w)+F_t*0.8*l_w/(4*l_w)

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc — Vs = 16610.7 kips

Determine Required Shear Reinforcing — ρ = 0.0059 — ρ=Vs/(0.8*l_w*t_w*144*fy)

11.10.6 - Equation 11-31 Shear Reinforcing Requirements — **ρ_n = 0.0059** > .0025 MINIMUM STEEL DOES NOT GOVERN

2.0c) ACI 349 - 11.10.6 Equation 11-32 Requirements
Check Bounding Case Mu/Vu - l_w/2 = -25.58 **Mu/Vu-lw/2 < 0 equation 11-32 NOT APPLICABLE**

Determine Concrete Shear Capacity — Vc= N/A — Vc=[0.6*f'$_c$$^{0.5}$+$l_w$*(1.25*f'$_c$$^{0.5}$+0.2*Ft*1000/($l_w$*$t_w$*144))/(Mu/Vu-$l_w$/2)]*$t_w$*0.8*$l_w$*144/1000

Determine Shear Carried by Steel — Vs = N/A — Vs=Vu/ϕ -Vc

Determine Required Shear Reinforcing Requirements — ρ =N/A — ρ=Vs/(0.8*l_w*t_w*144*fy)

11.10.6 - Equation 11-32 Shear Reinforcing Requirements — **ρ_n = N/A** **EQUATION 11-32 NOT APPLICABLE**

2.0d) Horizontal Shear Reinforcing Requirements (max of 2a, 2b, 2c) — **ρ_n = 0.0059**

2.0e) Select Horizontal Shear Reinforcing Asn required per ft on each face = **2.56** **in^2/ft each face** (ρ_n*12*t_w*12/2)

Use 2-#11's@12"c/c EF Asn provided = 3.12 in^2/ft each face **ρ_n (prov)= 0.00722**

2.0f) Check Demand / Capacity Ratio for In-Plane Shear:
D/C = 0.76 — D/C=(Vu/ϕ)/[Vc+(Asn*2*fy*l_w*t_w*144/(12*t_w*12))]

3.0 Vertical Reinforcing Requirements

3.0a) ACI 349 21.6.5.5 and 11.10.9.4 - Minimum vertical reinforcing ratio : — hw/lw = 0.22
If h_w/l_w>2.0, use: $\rho_{v\,min}$ =0.0025+0.5(2.5-h_w/l_w)(ρ_n-0.0025)<=ρ_n
ρ_v (min) = 0.0059 — If h_w/l_w<=2.0, use: ρ_v>=ρ_n

3.0b) Check Shear Friction Requirements

In plane shear per foot of wall: 238.86 kips/ft (Vu/l_w)

Figure D-4a. Shear Wall Design Template

Transverse shear per ft of wall: 19.58 kips/ft (Vz)

Resultant Shear 239.66 kips/ft $[(\text{in-plane shear})^2+(\text{transverse shear})^2]^{0.5}$

Calculate limiting shear friction strength at joint per ACI 349-11.7.5:

Vn<.2f'cAv for f'c = 5000 psi Vn=1000Av
Vn<800Av The limiting value of 800Av controls

Vn (MAX) = 691.2 kips/ft **Vn(MAX>Resultant shear ………OK**

Calculate shear friction reinforcing requirements per ACI 349 - 11.7.4.1:

$Vn = Avf*fy*\mu$ μ = 1.0 for concrete placed against hardened concrete intenionally roughened to a full amplitude of 1/4 inch (ACI 349-11.7.9)

$Avf = Vu/2*\phi*\mu*fy$ (steel required per face)
Avf = 2.35 in^2/ft (steel required on each face for shear friction))

Calculate steel required for net Tension force

$At = T/2*\phi*fy*lw$
At = 0.45 in^2/ft (steel required on each face for direct tension)

Steel Requirements for Shear Friction Avf + At

Av = 2.80 in^2/ft (steel required on each face for shear friction + direct tension)

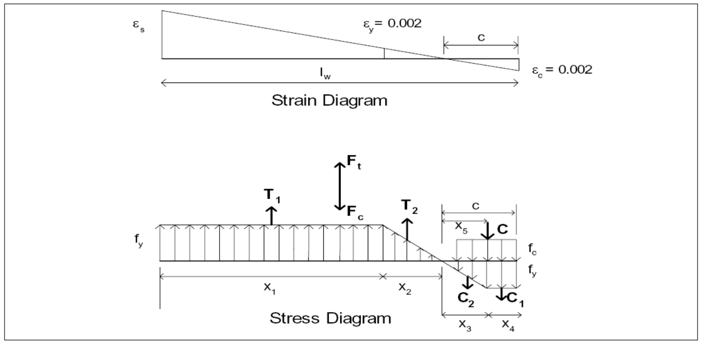
ρv(req'd) = 0.0065 $(\rho_{v\ req'd}=(2*Av)/(12*t_w*12)$

3.0c) Vertical Reinforcing Requirements (max of 3a,3b) **ρ_v = 0.0065**

3.0d) Select Vertical Reinforcing **Asv required per ft on each face = 2.80 in^2/ft each face** $(\rho_{v\ min}*12*t_w*12/2)$

Use 2-#11@12"c/c EF **Asv provided = 3.12 in^2/ft each face** **ρ_v (prov)= 0.00722**

3.0e) Perform Strain-Compatible Section Analysis - For Axial Force (Tension) Ft = -3130 kips



ϕ : Strength reduction factor (= 0.9 for tension, ACI 349 9.3.2.2), ϕ = 0.9
c : Distance from compression face to neutral axis (ft)
l_w : Length of wall (ft)
$\varepsilon_{s\_max} = [(l_w\text{-c}) / c] * \varepsilon_c$
$X_1 = l_w - c - X_2$ (ft) (length of wall with tension steel reinforcing steel strain > ε_y)
$X_2 = X_3 = (\varepsilon_y / \varepsilon_c) * c$ (ft) (length of wall with tension / compression steel reinforcing steel strain < ε_y)
$X_4 = c - X_3$ (ft) (length of wall with compression steel reinforcing steel strain > ε_y)
$T_1 = X_1 * As * fy$ (kips)
$T_2 = C_2 = 0.5 * X_2 * As * fy$ (kips)
$C_1 = X_4 * As * fy$ (kips)
$C = 0.85 * 0.8 * c * t_w * f'_c * 144$ (kips)
Balanced condition : Tension = Compression, $T_1+T_2-C_2-C_1-C+Ft/\phi = 0$.
M_u : Total moment capacity = $\phi\{T_1(X_2+X_1/2) + T_2(X_2*2/3) + C(c\text{-}0.8*c/2) + F_t(l_w/2\text{-}c)/\phi + C_2(X_3*2/3) + C_1(X_3+X_4/2)\}$, (kip-ft)

Using Goal Seek

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
5.91	61.59	0.0208	55.68	5.91	0.00	20845	1107	0	17367	0.000	609333

Verified that it equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
5	62.5	0.0250	57.50	5.00	0.00	21528	936	0	14688	3362	609199
5.5	62.0	0.0225	56.50	5.50	0.00	21153.6	1029.6	0	16157	1519	608899
6	61.5	0.0205	55.50	6.00	0.00	20779.2	1123.2	0	17626	-324	609505

Perform Strain-Compatible Section Analysis - For Axial Force (Comp) Fc = 16062 kips

Figure D-4b. Shear Wall Design Template

ϕ : Strength reduction factor (= 07 to 0.9 for comp, ACI 349 9.3.2.2), ϕ = 0.7898
If Fc<=0, ϕ=0.9; if Fc>0, ϕ=0.9-0.2*Fc/(0.1*Acv*144*f'$_c$/1000)

Using Goal Seek

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
12.37	55.13	0.0089	42.76	12.37	0.00	16008	2316	0	36344	0.000	1013358

Verified that it equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
12	55.5	0.0093	43.50	12.00	0.00	16286.4	2246.4	0	35251	1371	1012350
12.5	55.0	0.0088	42.50	12.50	0.00	15912	2340	0	36720	-472	1013807
13	54.5	0.0084	41.50	13.00	0.00	15537.6	2433.6	0	38189	-2315	1016059

3.0f) Check Demand / Capacity Ratio for In-Plane Moment:

Vertical Reinforcing Ratio Provided by Design : ρ_v (prov)= 0.0072 2-#11@12"c/c EF $\rho_v = (2*Asv)/(12*t_w*12)$
D/C = (in-plane moment Mz) / (Min of Mu-ten, Mu-comp) D/C = 0.22 **Section Adequate**
Vertical Reinforcing Ratio Required for In-Plane Moment: $\rho_{vl\,req}$= 0.0016 $\rho_{v\,req} = \rho_v*(D/C)$

3.0g) Consider Out-of-Plane Moment:

Vertical Reinf Ratio Required for Out-of-Plane Bending: $\rho_{vt\,req}$= 0.00067 (per face) $\rho f_y bd$^2*(1-.59$\rho f_y/f'_c$)=Mu/ϕ solve for ρ

REF. "REINFORCED CONCRETE MECHANICS AND DESIGN"
James G MacGregor, Third Edition, 1997. Section 4-3
TIC 242587 [DIRS 130532]

3.0h) Total Reinforcing required for axial force, in-plane and out of plane bending ρ_v = 0.0022 $= \rho_{vl}+\rho_{vt} > 2\rho_{vt}$

D/C = ρ_v(reqd) / ρ_v (prov) D/C = 0.31 **Section Adequate** = ρ_v(req'd) / ρ_v(prov)

4.0 Boundary Elements:

h_w / l_w = 0.22 <2, No Need To Check for Boundary Elements

5.0 Out-of-Plane Shear:

Nominal Shear Strength Provided by The Concrete Vc=2*(f'c)$^{1/2}$*b*d Vc = 112.0 kips/ft width of wall

Check Demand / Capacity Ratio: D/C = 0.21 No Shear Reinforcement Required
D/C = (out-of-plane shear Vz) / (0.85*Vc)

6.0 Tabulate Reinforcement Requirements And D/C Ratios:

Use 6 ft thick wall with 2-#11's@12"c/c EF Horizontal Reinforcement
2-#11@12"c/c EF Vertical Reinforcement.

For Shear on Gross Section:	**D/C =**	**0.81**
For In-Plane Shear:	**D/C =**	**0.76**
For Out-of-Plane Shear:	**D/C =**	**0.21**
Bending + axial Loads	**D/C =**	**0.31**

Boundary Elements **D/C=** **BOUNDARY ELEMENTS NOT REQUIRED**

Figure D-4c. Shear Wall Design Template

APPENDIX E
INCORPORATION OF CONCRETE CRACKING INTO THE FINITE ELEMENT MODEL

As noted in Section 7.2.9, concrete cracking may need to be modeled to obtain accurate seismic responses. For the typical shear wall structures at the YMP, cracking should be considered in the analysis of the slabs. The walls of these structures are subject to small out-of-plane bending and, therefore, the out-of-plane bending moments are normally not sufficient to cause cracking. The in-plane moments can be large; but because of their height-to-length ratio (squat walls), again cracking is not a significant factor.

For the slab cracking, two distinct cases should be considered:

Case 1. Reinforced concrete slabs without supporting steel beams and/or girders.

Case 2. Slabs with supporting steel beams/girders.

E1 SLABS WITHOUT SUPPORTING BEAMS AND/OR GIRDERS

For this case, the slabs are considered to be cracked and the recommendations of ASCE/SEI 43-05 [DIRS 173805] are followed. That is:

$$E_cI_{cr} = 0.5\ E_cI_g = (0.5\ E_c)I_g \qquad \text{(Eq. E-1)}$$

where E_c is the concrete modulus of elasticity, I_{cr} is the cracked moment of inertia and I_g is the gross moment of inertia. Thus, in the computer input, the modulus of elasticity of such slabs are reduced by one-half and the actual slab thickness is entered.

If the slab is modeled as a single-degree-of-freedom system (e.g., a lollipop), the frequency is calculated using the reduced moment of inertia.

Equation E-1 is used in lieu of a more detailed stiffness calculation. The effective stiffness used in ACI 349 [DIRS 181670] is based on weighted average of cracked and uncracked moments of inertia. For the slabs in the ITS structures at the YMP, a more appropriate approach may be to simply consider the average of the cracked and uncracked section modulus. This is proper since less than one-half of the span in either direction is expected to crack under the seismic loads. This consideration leads to the following equation:

$$I_e = (I_{cr} + I_g) / 2 \qquad \text{(Eq. E-2)}$$

Equation E-2 will typically result in an effective stiffness of about $0.6E_cI_g$, depending on the reinforcement ratio. Considering the uncertainties involved in the effective stiffness calculations, both equations E-1 and E-2 are acceptable for accounting concrete cracking in the finite element modeling.

E2 SLABS WITH SUPPORTING BEAMS AND/OR GIRDERS

In the case of the slabs with supporting beams and/or girders, there is a need to perform a separate analysis to determine the properties of the "equivalent slab." This is accomplished as follows:

Step 1: Model the slab with finite elements and beams/girders with beam elements. In this analysis, the average of cracked and uncracked moments of inertia for the slab may be used per Equation E-2:

$$I_e = (I_{cr} + I_g) / 2$$

where I_e is the effective moment of inertia. However, rather than adjusting the slab moment of inertia, the modulus of elasticity can be adjusted as before:

$$E'_c \times I_g = E_c \times I_e \qquad \text{(Eq. E-3)}$$

$$E'_c = E_c \times (I_{cr} + I_g) / (2.0 \times I_g) \qquad \text{(Eq. E-4)}$$

Equation E-4 gives the value of the modified modulus of elasticity to be used in the analysis. The average moment of inertia concept is based on studies for one way action that predicts the behavior of beams accurately (simple span and two-span continuous beams, T-beams). Extension of this concept to two-way action slabs is reasonable as the purpose here is to define an approximate model for the slab that will result in accurate fundamental frequency.

Step 2: Using the actual thickness of the slab, adjust the mass density to account for the total dead weight.

Step 3: Perform a modal analysis of the composite model to determine its fundamental frequency.

Step 4: Remove the beam elements (or simply disable them) and perform modal analysis by adjusting the modulus of elasticity until the fundamental frequency matches that obtained in Step 3.

Step 5: If the slab model will be part of the structural model to perform a dynamic analysis, input the slab properties together with the converged modulus of elasticity for the plate elements that make up this slab segment. Similar studies may be performed for other slabs to obtain a representative range of slabs for the overall model.

Step 6: If the slab segment will be modeled as a single-degree-of-freedom system, determine a spring constant that will give the same fundamental frequency as the converged analysis. This spring can then be located in the center of the slab and connected to midpoints of the supporting walls on all four sides.

The equivalent slab models or the single-degree-of-freedom systems developed by this six-step approach will result in accurate stress analysis for these slabs and in-structure response spectra.

APPENDIX F
SEISMIC REQUIREMENTS FOR MECHANICAL EQUIPMENT

F1 SCOPE

Section 7.5 of this document covers seismic analysis of systems and components which includes mechanical equipment. Section 8 covers the seismic design of ITS SSCs.

This appendix is provided to document design requirements for specific mechanical equipment currently being specified. It is expected that the list of equipment will grow over time and this appendix will be updated, as required.

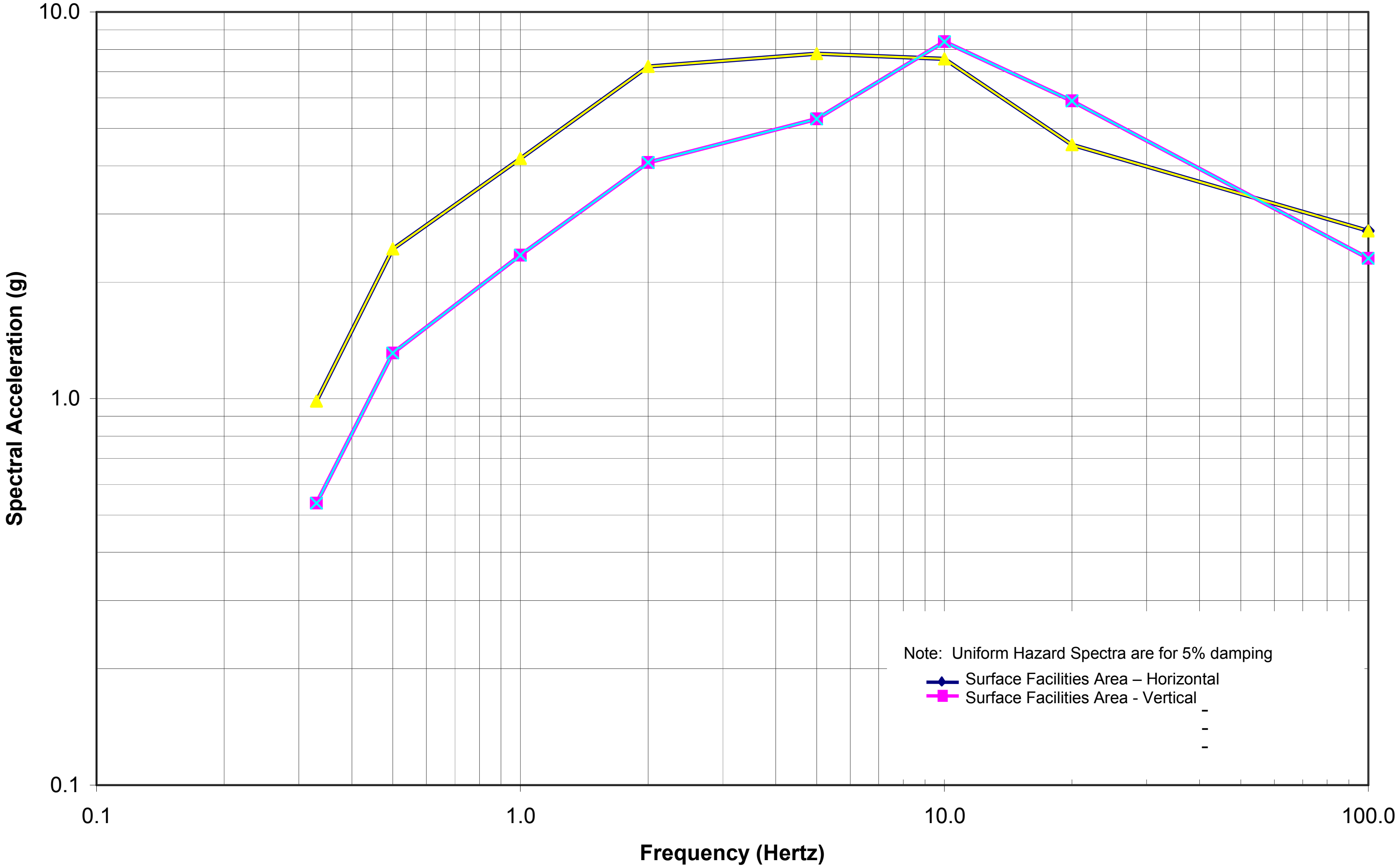
F2 EQUIPMENT REQUIREMENTS

Table F1 shows the seismic requirements for the following equipment:

- Aging Overpack and Aging Cask (A.O.)
- Shielded Transfer Cask-Vertical (STC)
- Shielded Transfer Cask-Horizontal (STC)
- Tractor and Trailer (included with STC Horizontal
- Transporter for Vertical A.O.
- Spent Fuel Transfer Machine (SFTM)

The seismic requirements were established as a result of discussions between engineering and the PCSA group personnel. The requirements are intended to maintain the functionality of each equipment after the design basis earthquake (DBGM-2). In addition, some of the equipment must be evaluated for BDBGM in order to demonstrate that their annual probability of exceedance (APE) for the failure modes listed in the table are less than or equal to 2E-6.

Figure F1 shows the horizontal and vertical response spectra for the extreme seismic event (APE 2E-6 MO0706HCUHSSFA.000 [DIRS 182465]). Table F1 lists the seismic requirements for selected mechanical equipment. Table F2 shows the digitized response spectra for the extreme seismic event (APE 2E-6 MO0706HCUHSSFA.000 [DIRS 182465]).



Source: MO0706HCUHSSFA.000 [DIRS 182465]

Figure F-1. Uniform Hazard Ground Acceleration Spectra for Extreme Seismic Event (APE 2E-6) – Surface Facilities Area – 5% Damping

Table F1. Seismic Requirements for Selected Equipment

Equipment	Design Criteria	Evaluation Criteria	Miscellaneous Criteria	Comments
A.O. **Aging Overpack or Aging Cask**	DBGM-2 • No tip over • Design to applicable code • Thermal criteria • Leakage criteria	BDBGM • No tip over • Designed to Code • Meets thermal criteria • Meets leakage criteria Extreme Seismic Event (APE 2E-6) • No tip over • DPC integrity maintained	Consider seismic amplification from ground spectra at the equipment base for all seismic events	Secured in transporter must sustain drop/slap down • Consider transporter event • Consider extreme seismic event • Demonstrate that the slap down velocity is higher than the seismic induced velocity. Vendor to provide impact velocity information for the entire spectrum of events determined by the vendor.
STC Vertical **Shielded Transfer Cask**	DBGM-2 • Designed to Code Note: Vertical STC not taken out of building	—	—	—
STC Horizontal **Shielded Transfer Cask**	DBGM-2 • Shall sustain rollover & drop • Sealing criteria	BDBGM • Must sustain rollover and drop • Meets sealing criteria	Consider seismic amplification from ground spectra at the equipment base for all seismic events	The extreme seismic event is not considered because of the limited number of operations per year.
Tractor & Trailer	The tractor/trailer system will be designed such that it does not affect the integrity of the horizontal STC	—	—	The extreme seismic event is not considered because of the limited number of operations per year.
Transporter for Vertical A.O.	Inside of the building: • DBGM-2 • Design to applicable code • No tip over Outside of the building: • DBGM-2 • Design to code • No tip over • Design such that it does not affect the integrity of the Aging Overpack	—	—	The extreme seismic event is not considered as it is a part of the criteria for evaluation A.O's.
SFTM **(Spent Fuel Transfer Machine)**	DBGM-2 Design for seismic load from WHF spectra on top of basemat grade.	BDBGM Designed for seismic load from WHF spectra on top of basemat grade (no failure).	—	—

Table F2. Uniform Hazard Spectra

Uniform Hazard Ground Acceleration Spectra for Extreme Seismic Event for 2E-6 annual probability of exceedance, Surface Facilities Area

Site-Wide Uniform Hazard Spectra HORIZONTAL			Site-Wide Uniform Hazard Spectra VERTICAL		
PERIOD(S)	**FREQ(HZ)**	**GMOTION**	**PERIOD(S)**	**FREQ(HZ)**	**GMOTION**
0.01	100.00	.27137E+01	0.01	100.00	.23036E+01
0.05	20.00	.45324E+01	0.05	20.00	.58842E+01
0.10	10.00	.75612E+01	0.10	10.00	.83697E+01
0.20	5.00	.78000E+01	0.20	5.00	.52884E+01
0.50	2.00	.72244E+01	0.50	2.00	.40745E+01
1.00	1.00	.41768E+01	1.00	1.00	.23470E+01
2.00	0.50	.24355E+01	2.00	0.50	.13102E+01
3.00	0.33	.98388E+00	3.00	0.33	.53600E+00