NOV 11 1994
35 Station 21

ENGINEERING DATA TRANSMITTAL

Page 1 of 1
1. EDT 609366

2. To: (Receiving Organization) Multifunction Waste Tank Facility	3. From: (Originating Organization) Structural analysis	4. Related EDT No.: N/A
5. Proj./Prog./Dept./Div.: Project 236A	6. Cog. Engr.: A. M. Tallman	7. Purchase Order No.: N/A
8. Originator Remarks: Review draft of seismic design response spectra for W236A. Key words: seismic design, design response spectra, ground motion		9. Equip./Component No.: N/A
11. Receiver Remarks:		10. System/Bldg./Facility: N/A
		12. Major Assm. Dwg. No.: N/A
		13. Permit/Permit Application No.: N/A
		14. Required Response Date: November 30, 1894

15. DATA TRANSMITTED					(F)	(G)	(H)	(I)
(A) Item No.	(B) Document/Drawing No.	(C) Sheet No.	(D) Rev. No.	(E) Title or Description of Data Transmitted	Approval Designator	Reason for Transmittal	Originator Disposition	Receiver Disposition
1	WHC-SD-W236A-TI-016		0	Seismic Design Spectra 200 West and East Areas DOE Hanford Site, Washington	Q	2	1	

16. KEY			
Approval Designator (F) E, S, Q, D or N/A (see WHC-CM-3-5, Sec.12.7)	Reason for Transmittal (G) 1. Approval 2. Release 3. Information 4. Review 5. Post-Review 6. Dist. (Receipt Acknow. Required)		Disposition (H) & (I) 1. Approved 2. Approved w/comment 3. Disapproved w/comment 4. Reviewed no/comment 5. Reviewed w/comment 6. Receipt acknowledged

17. SIGNATURE/DISTRIBUTION (See Approval Designator for required signatures)										(G)	(H)
Reason	Disp.	(J) Name	(K) Signature	(L) Date	(M) MSIN	(J) Name	(K) Signature	(L) Date	(M) MSIN	Reason	Disp.
1,2	1	Cog.Eng. A. M. Tallman	<i>[Signature]</i>	11/7/94	H5-53	J. M. Light	<i>[Signature]</i>		B4-08	2	
1,2	1	Cog. Mgr.T. J. Corns	<i>[Signature]</i>	11/7/94	H5-55	RL Fritz	<i>[Signature]</i>		B4-08	2	
1,2	1	QA A. K. Sharma	<i>[Signature]</i>	11/14/94	H5-50	JE Gilbert	<i>[Signature]</i>		B4-68	2	
		Safety				TC Oten	<i>[Signature]</i>		B6-50	2	
1		OSTI (2)		L8-07		TB Salzano	<i>[Signature]</i>		E6-37	2	
2		S. K. Farnworth			H5-55	GR Wagengblast	<i>[Signature]</i>		H5-55	2	
2		M. D. Stine			E6-50	GG Renfro	<i>[Signature]</i>		B4-68	2	
1		Central Files (orig + 2)		11/8/94							

18. Signature of EDT Originator <i>[Signature]</i> A. M. Tallman	Date 11/7/94	19. Authorized Representative Date for Receiving Organization <i>[Signature]</i> J. M. Light	Date 11/14/94	20. Cognizant Manager <i>[Signature]</i> J. Corns	Date 11/14/94	21. DOE APPROVAL (if required) Ctrl. No. ☐ Approved ☐ Approved w/comments ☐ Disapproved w/comments

DISTRIBUTION OF THIS DOCUMENT IS UNLIMITED

INSTRUCTIONS FOR COMPLETION OF THE ENGINEERING DATA TRANSMITTAL

(USE BLACK INK OR TYPE)

<u>BLOCK</u>	<u>TITLE</u>	
(1)*	EDT	● Pre-assigned EDT number.
(2)	To: (Receiving Organization)	● Enter the individual's name, title of the organization, or entity (e.g., Distribution) that the EDT is being transmitted to.
(3)	From: (Originating Organization)	● Enter the title of the organization originating and transmitting the EDT.
(4)	Related EDT No.	● Enter EDT numbers which relate to the data being transmitted.
(5)*	Proj./Prog./Dept./Div.	● Enter the Project/Program/Department/Division title or Project/Program acronym or Project Number, Work Order Number or Organization Code.
(6)*	Cognizant Engineer	● Enter the name of the individual identified as being responsible for coordinating disposition of the EDT.
(7)	Purchase Order No.	● Enter related Purchase Order (P.O.) Number, if available.
(8)*	Originator Remarks	● Enter special or additional comments concerning transmittal, or "Key" retrieval words may be entered.
(9)	Equipment/Component No.	● Enter equipment/component number of affected item, if appropriate.
(10)	System/Bldg./Facility	● Enter applicable system, building or facility number, if appropriate.
(11)	Receiver Remarks	● Enter special or additional comments concerning transmittal.
(12)	Major Asm. Dwg. No.	● Enter applicable drawing number of major assembly, if appropriate.
(13)	Permit/Permit Application No.	● Enter applicable permit or permit application number, if appropriate.
(14)	Required Response Date	● Enter the date a response is required from individuals identified in Block 17 (Signature/Distribution).
(15)*	Data Transmitted	
(A)*	Item Number	● Enter sequential number, beginning with 1, of the information listed on EDT.
(B)*	Document/Drawing No.	● Enter the unique identification number assigned to the document or drawing being transmitted.
(C)*	Sheet No.	● Enter the sheet number of the information being transmitted. If no sheet number, leave blank.
(D)*	Rev. No.	● Enter the revision number of the information being transmitted. If no revision number, leave blank.
(E)	Title or Description of Data Transmitted	● Enter the title of the document or drawing or a brief description of the subject if no title is identified.
(F)*	Approval Designator	● Enter the appropriate Approval Designator (Block 15). Also, indicate the appropriate approvals for each item listed, i.e., SQ, ESQ, etc.
(G)	Reason for Transmittal	● Enter the appropriate code to identify the purpose of the data transmittal (see Block 16).
(H)	Originator Disposition	● Enter the appropriate disposition code (see Block 16).
(I)	Receiver Disposition	● Enter the appropriate disposition code (see Block 16).
(16)	Key	● Number codes used in completion of Blocks 15 (G), (H), and (I), and 17 (G), (H) (Signature/Distribution).
(17)	Signature/Distribution	
(G)	Reason	● Enter the code of the reason for transmittal (Block 16).
(H)	Disposition	● Enter the code for the disposition (Block 16).
(J)	Name	● Enter the signature of the individual completing the Disposition 17 (H) and the Transmittal.
(K)*	Signature	● Obtain appropriate signature(s).
(L)*	Date	● Enter date signature is obtained.
(M)*	MSIN	● Enter MSIN. Note: If Distribution Sheet is used, show entire distribution (including that indicated on Page 1 of the EDT) on the Distribution Sheet.
(18)	Signature of EDT Originator	● Enter the signature and date of the individual originating the EDT (entered prior to transmittal to Receiving Organization). If the EDT originator is the cognizant engineer, sign both Blocks 17 and 18.
(19)	Authorized Representative for Receiving Organization	● Enter the signature and date of the individual identified by the Receiving Organization as authorized to approve disposition of the EDT and acceptance of the data transmitted, as applicable.
(20)*	Cognizant Manager	● Enter the signature and date of the cognizant manager. (This signature is authorization for release.)
(21)*	DOE Approval	● Enter DOE approval (if required) by signature or control number that tracks the approval to a signature, and indicate DOE action.

*Asterisk denote the required minimum items check by Configuration Documentation prior to release; these are the minimum release requirements.

RELEASE AUTHORIZATION

Document Number: WHC-SD-W236A-TI-016, Rev 0

Document Title: Seismic Design Spectra 200 West and East Areas DOE
Hanford Site, Washington

Release Date: 11/11/94

**This document was reviewed following the
procedures described in WHC-CM-3-4 and is:**

APPROVED FOR PUBLIC RELEASE

WHC Information Release Administration Specialist:

V. L. Birkland

11/11/94

TRADEMARK DISCLAIMER. Reference herein to any specific commercial product, process, or service by trade name, trademark, manufacturer, or otherwise, does not necessarily constitute or imply its endorsement, recommendation, or favoring by the United States Government or any agency thereof or its contractors or subcontractors.

This report has been reproduced from the best available copy. Available in paper copy and microfiche. Printed in the United States of America. Available to the U.S. Department of Energy and its contractors from:

U.S. Department of Energy
Office of Scientific and Technical Information (OSTI)
P.O. Box 62
Oak Ridge, TN 37831
Telephone: (615) 576-8401

Available to the public from:

U.S. Department of Commerce
National Technical Information Service (NTIS)
5285 Port Royal Road
Springfield, VA 22161
Telephone: (703) 487-4650

DISCLAIMER

Portions of this document may be illegible in electronic image products. Images are produced from the best available original document.

SUPPORTING DOCUMENT		1. Total Pages 74 <i>76</i>
2. Title Seismic Design Spectra 200 West and East Areas DOE Hanford Site, Washington	3. Number WHC-SD-W236A-TI-016	4. Rev No. 0
5. Key Words Seismic design, design response spectra, ground motion	6. Author Name: A. M. Tallman for Geomatrix <i>A. M. Tallman</i> Signature 8D400/DTNEE Organization/Charge Code	
7. Abstract This document presents equal hazard response spectra for the W236A project for the 200 East and West new high-level waste tanks. The hazard level is based upon WHC-SD-W236A-TI-002, Probabilistic Seismic Hazard Analysis, DOE Hanford Site, Washington. Spectral acceleration amplification is plotted with frequency (Hz) for horizontal and vertical motion and attached to this report. The vertical amplification is based upon the preliminary draft revision of Standard ASCE 4-86. The vertical spectral acceleration is equal to the horizontal at frequencies above 3.3 Hz because of near-field, less than 15 km, sources.		
		8. RELEASE STAMP OFFICIAL RELEASE BY WHC DATE NOV 11 1994 35 Station 21

DISCLAIMER

This report was prepared as an account of work sponsored by an agency of the United States Government. Neither the United States Government nor any agency thereof, nor any of their employees, makes any warranty, express or implied, or assumes any legal liability or responsibility for the accuracy, completeness, or usefulness of any information, apparatus, product, or process disclosed, or represents that its use would not infringe privately owned rights. Reference herein to any specific commercial product, process, or service by trade name, trademark, manufacturer, or otherwise does not necessarily constitute or imply its endorsement, recommendation, or favoring by the United States Government or any agency thereof. The views and opinions of authors expressed herein do not necessarily state or reflect those of the United States Government or any agency thereof.

SEISMIC DESIGN SPECTRA 200 WEST AND EAST AREAS HANFORD SITE, WASHINGTON

Prepared for Westinghouse Hanford Company

by

Geomatrix Consultants

WHC-SD-W236A-TI-016 Rev.0

Attachment: Horizontal and vertical seismic response spectra plots

MASTER

ab
DISTRIBUTION OF THIS DOCUMENT IS UNLIMITED

**SEISMIC DESIGN SPECTRA
200 WEST AND EAST AREAS
DOE HANFORD SITE, WASHINGTON**

1.0 INTRODUCTION

This report presents recommend seismic design spectra for the 200 West and 200 East areas of the DOE Hanford Site, Washington. The design spectra are based on the results of two studies. Geomatrix (1993) presents a detailed probabilistic seismic hazard analysis for the DOE Hanford Site. That analysis provided the basis for developing equal-hazard response spectra for the 200 West and 200 East Areas using generic deep soil site, empirical ground motion attenuation relationships. Section 2 of this report presents the equal-hazard spectra for the 200 West and 200 East Areas computed using the probabilistic seismic hazard model developed in Geomatrix (1993).

The second study evaluates of the applicability of generic deep soil site, empirical ground motion attenuation relationships to describe the response of the soils at the 200 West and 200 East Areas to earthquake ground motions. Preliminary evaluations were performed during the probabilistic seismic hazard analysis study and these are documented in Section 3. The preliminary analyses used generic soil profiles developed from review of investigations of subsurface conditions at various locations on the Hanford Site. Recently, subsurface investigations were completed for the Multi-function Waste Tank Facilities located at the 200 West Area (Shannon and Wilson, 1994b) and 200 East Area (Shannon and Wilson, 1994a). The results of these investigation were used together with the finalized probabilistic seismic hazard results to evaluate the dynamic response of the subsurface materials at the 200 West and 200 East Areas. This analysis is documented in Section 4. Using the results presented in Sections 2, 3, and 4, recommended design response spectra for horizontal earthquake motions are presented in Section 5.

2.0 EQUAL-HAZARD SPECTRA

2.1 SUMMARY OF PROBABILISTIC SEISMIC HAZARD MODEL

Geomatrix Consultants (1993) present a probabilistic seismic hazard model for the Hanford Site that represents the evaluations of a team of experts on seismic hazards that includes members of Geomatrix Consultants, Pacific Northwest Laboratories, Washington Public Power Supply System, and Westinghouse Hanford Company. The seismic hazard model represents the team's assessment of the current state of scientific knowledge about the seismic potential and earthquake ground motion characteristics of the Hanford region. The seismic hazard model includes contributions from all of the potential seismic sources considered relevant to hazard assessment: the Yakima folds, seismicity within the Columbia River Basalts unassociated with the Yakima folds, seismicity within the crystalline basement rocks, and the Cascadia subduction zone located west of the Cascade Range.

Geomatrix (1993) concluded that the characteristics of earthquake ground motions in the Hanford region from the Yakima fold, Columbia River Basalt, and crystalline basement sources could be represented by empirical attenuation relationships developed from California strong motion data, and the ground motion characteristics for the Cascadia subduction zone sources could be represented by empirical attenuation relationships developed from the world-wide recorded data. The selected California empirical attenuation relationships were those of Campbell (1993), Joyner and Boore (1982), and Sadigh and others (1986), and the relationships of Crouse (1991) were selected for modeling ground motions from the Cascadia sources. The selected relationships were chosen to represent ground motions on relatively deep, firm soils. Tables 2-1 through 2-4 list the coefficients of the attenuation relationships.

The Hanford DOE sites are underlain by stiff to very stiff alluvial soils overlying the Columbia River basalts. Depth to basalt varies in the range of 200 to 600 ft. The alluvial soils consist of Holocene and Pleistocene loess, sands and gravels underlain by the Ringold formation, which consists of dense silty sands to clayey to sandy gravel. Locally the Ringold

unit is cemented and exhibits shear wave velocities in excess of 4,000 ft/sec. As part of the studies conducted for the WNP-2 commercial nuclear power plant located on the Hanford Site, Washington Public Power Supply System (1985) performed comparative site response studies using the soil velocity profile at the WNP-2 site and typical firm alluvial soil profiles representative of California strong motion recording sites. The conclusion of that analysis was that the empirical strong motion data from firm alluvial sites in California was appropriate for use at Hanford. A limited re-evaluation of the Washington Public Power Supply System (1985) study, conducted during the development of the probabilistic seismic hazard model, reached the same conclusion. The results of that study are presented in Section 3 of this report. Additional studies, incorporating recent site specific data are described in Section 4.

2.2 PROBABILISTIC SEISMIC HAZARD ANALYSIS FOR THE 200 WEST AND 200 EAST AREAS

2.2.1 Analysis Procedure

Seismic hazard calculations were made for peak horizontal ground acceleration and 5%-damped response spectral accelerations at periods of 0.1, 0.2, 0.3, 0.5, 1.0, and 2.0 seconds. Fault-specific sources were modeled as segmented planer surfaces and areal source zones were modeled as a set of closely spaced parallel fault planes occupying the source regions. The probability density function for distance to earthquake rupture for each source was computed assuming earthquake ruptures were uniformly distributed over the fault planes, except for the case in which the spatial distribution in the shallow basalt is assumed to follow that observed in the recorded seismicity. The distance density functions were computed consistent with the distance measure used in each of the attenuation relationships. The rupture size of an event was specified by the relationship $\ln(\text{area}) = 2M - 7.12$ developed from the data presented by Wyss (1979). This relationship yields similar results to those given by the updated relationships provided by Wells and Coppersmith (in press). The specified relationship gives the mean rupture area for a specific magnitude rather than the median (mean log) rupture area, which is appropriate for probabilistic hazard assessment (Bender, 1984).

Distributions for the annual frequency of exceeding various levels of peak ground acceleration and spectral acceleration were developed by performing hazard computations using each end branch of the logic trees shown in Geomatrix (1993). The hazard was computed considering the contributions of earthquakes of magnitude $M \geq 5$ and larger ($m^0=5$). At each ground motion level, the complete set of results forms a discrete distribution for frequency of exceedance. The computed distributions were used to obtain the mean frequency of exceeding various levels of peak ground motion (mean hazard curve) as well as hazard curves representing various percentiles of the distributions.

2.2.2 Computed Hazard

Figures 2-1a and 2-1b present the computed mean peak hazard and the 5th- to 95th-percentile hazard curves for the 200 West and 200 East Areas, respectively, for peak acceleration and 5%-damped spectral accelerations at periods of 0.3 and 2.0 seconds. The uncertainty band varies from about one order of magnitude at low ground motion levels to over two orders of magnitude at large ground motion levels. The uncertainty in the computed hazard also increases as one considers longer periods of vibration. The distribution in computed frequency of exceedance becomes skewed at the higher ground motion levels and the mean hazard lies near the 75th-percentile of the hazard distribution.

Figures 2-2a and 2-2b show the contribution of the three shallow crustal source types - Yakima folds, shallow basalt and basement sources, and the two subduction zone source types - interface and intraslab - to the mean hazard at the 200 West and 200 East Areas, respectively. The results indicate that the Yakima fold sources are the largest contributors to the hazard for high frequency motions. At long periods, the distant Cascadia interface source becomes the dominant contributor to the site hazard.

Figures 2-3a and 2-3b show the relative contribution of events in different magnitude and distance intervals to the computed mean hazard at the 200 West and 200 East Areas, respectively. Histograms are presented for peak acceleration and spectral acceleration at periods of 0.3 and 2.0 seconds for a mean annual frequencies of exceedance of approximately

10^{-3} (return period of 1,000 years) and 10^{-4} (return period of 10,000 years). Each plot in the figures presents a histogram of the percent contributions of events in 0.25 magnitude unit-wide intervals. The histograms are further broken down to show the relative contributions of events in different distance ranges to the total hazard. The hazard from high frequency ground motions is dominated by the contributions from events in the range of M 5 to 6, primarily at source-to-site distances of 15 km and less. As the period of the spectral acceleration considered increases, the larger magnitude events become increasingly important, dominating the hazard at a period of 2.0 seconds. This shift in magnitude contribution results from the increase in magnitude scaling of ground motion amplitude with increasing period. Long vibration period motions exhibit a much greater increase in amplitude with increasing magnitude than short period motions. In addition, the very large, distant Cascadia interface events dominate the hazard because of the large size of the potential earthquakes and the empirically observed lower rate of attenuation of ground motions from subduction zone earthquakes compared to shallow crustal earthquakes.

2.2.3 Equal-Hazard Spectra

Figures 2-4a and 2-4b show the mean hazard curves developed for the eight periods of vibratory motion. Using these curves, the following ground motion levels were computed for return periods of 100, 500, 1,000, 2,000, 5,000, 10,000, and 100,000 years:

200 West Area								
Peak or 5%-damped Spectral Acceleration (g)								
<u>for Return Periods (yrs) of:</u>								
<u>Period</u>	<u>100</u>	<u>500</u>	<u>1,000</u>	<u>2,000</u>	<u>5,000</u>	<u>10,000</u>	<u>50,000</u>	<u>100,000</u>
PGA	0.040	0.096	0.138	0.195	0.293	0.383	0.644	0.767
0.1 sec	0.078	0.204	0.293	0.408	0.599	0.775	1.305	1.563
0.2 sec	0.096	0.252	0.366	0.511	0.762	1.004	1.738	2.152
0.3 sec	0.091	0.243	0.353	0.490	0.733	0.970	1.688	2.101
0.5 sec	0.074	0.201	0.292	0.407	0.607	0.809	1.441	1.833
1.0 sec	0.033	0.120	0.195	0.279	0.419	0.554	0.951	1.164
2.0 sec	0.013	0.056	0.086	0.114	0.184	0.239	0.419	0.530

200 East Area

Peak or 5%-damped Spectral Acceleration (g)

for Return Periods (yrs) of:

<u>Period</u>	<u>100</u>	<u>500</u>	<u>1,000</u>	<u>2,000</u>	<u>5,000</u>	<u>10,000</u>	<u>50,000</u>	<u>100,000</u>
PGA	0.039	0.092	0.132	0.186	0.280	0.372	0.645	0.775
0.1 sec	0.076	0.198	0.279	0.390	0.575	0.751	1.297	1.569
0.2 sec	0.094	0.244	0.353	0.487	0.730	0.971	1.721	2.155
0.3 sec	0.089	0.236	0.341	0.470	0.704	0.938	1.668	2.093
0.5 sec	0.073	0.196	0.283	0.395	0.587	0.781	1.400	1.794
1.0 sec	0.032	0.118	0.191	0.272	0.409	0.539	0.921	1.126
2.0 sec	0.013	0.055	0.084	0.112	0.181	0.232	0.405	0.506

Using the above values, equal-hazard curves were developed by smoothly interpolating between the ground motion levels computed at each period of vibration. Peak acceleration was assumed to conform to spectral acceleration at a period of 0.03 seconds (33 Hz). Figure 2-5 shows the resulting equal hazard spectra. With the exception of the 100-yr return period spectra, the equal-hazard spectra have very similar shapes because the hazard contains a large contribution from large magnitude events at all return periods (see Figures 2-4a,b).

TABLE 2-1
ATTENUATION RELATIONSHIP COEFFICIENTS
Sadigh and others (1986)

$$\ln(y) = C_1 + C_2 M - C_3 \ln(R + C_4 e^{C_5 M}) + C_6 + C_7 (8.5 - M)^{2.5}$$

where y is spectral acceleration in g
 $C_4 = 0.8217$, $C_5 = 0.4814$ for $M \leq 6.5$
 $C_4 = 0.6286$, $C_5 = 0.5866$ for $M > 6.5$
 R = closest distance to rupture surface

Period	C_1	C_2	C_3	C_6	C_7	Standard Error $\ln(y)$	
						$M \leq 7.21$	$M > 7.21$
0.033*	-2.611	1.1	1.75	0.0	0.0	1.39 -0.14	0.38
0.1	-2.611	1.1	1.75	0.5866	0.007	1.43 -0.14	0.42
0.2	-2.611	1.1	1.75	0.9154	0.0	1.46 -0.14	0.45
0.3	-2.611	1.1	1.75	0.9733	-0.008	1.48 -0.14	0.47
0.5	-2.611	1.1	1.75	0.9525	-0.025	1.52 -0.14	0.51
1.0	-2.611	1.1	1.75	0.6358	-0.060	1.59 -0.14	0.58
2.0	-2.611	1.1	1.75	0.1972	-0.105	1.63 -0.14	0.62

*peak ground acceleration

The above coefficients are for strike-slip earthquakes. For reverse faulting earthquakes C_1 is increased to -2.429 (20% higher motions)

TABLE 2-2
ATTENUATION RELATIONSHIP COEFFICIENTS
Joyner and Boore (1982)

$$\log_{10}(y) = C_1 + C_2(M-6) + C_3(M-6)^2 - C_4 \log(r) - C_5 r$$

where

$$r = (R^2 + C_6^2)^{\frac{1}{2}}$$

y - spectral acceleration in g

R - closest distance to surface projection of rupture

Period	C ₁	C ₂	C ₃	C ₄	C ₅	Standard Error	
						C ₆	log ₁₀ (y)
0.033*	0.43	0.23	0.0	1.0	0.0027	8.0	0.28
0.1	0.947	0.25	-0.06	1.0	0.0073	11.3	0.28
0.2	0.956	0.35	-0.09	1.0	0.0063	9.6	0.28
0.3	0.839	0.42	-0.11	1.0	0.0058	6.9	0.28
0.5	0.658	0.52	-0.14	1.0	0.0051	5.1	0.33
1.0	0.357	0.67	-0.17	1.0	0.0039	4.7	0.33
2.0	0.055	0.79	-0.20	1.0	0.0015	4.7	0.33

*peak ground acceleration

TABLE 2-3
ATTENUATION RELATIONSHIP COEFFICIENTS
Campbell (1993)

$$\ln(y) = C_0 + 0.683M + C_1 \tanh[0.647(M-4.7)] - 1.0 \ln(r) - (C_4 + C_5 M)R + 0.27F + [C_2 - 0.105 \ln(R)]S + C_3 \tanh(0.620D)$$

where

$$r = [R^2 + (0.0586e^{0.683M})^2]^{\frac{1}{2}}$$

y - spectral acceleration in g

F = 1 for reverse faulting (rake > 45°), 0 otherwise

S = 1 for rock, 0 for soil sites

Period	C ₀	C ₁	C ₂	C ₃	C ₄	C ₅	Standard Error
0.033	-3.15	0.0	0.0	0.0	0.0150	-0.000995	0.50
0.10	-2.61	0.0	0.08	0.0	0.0174	-0.000988	0.58
0.20	-2.32	0.0	-0.21	0.0	0.0139	-0.000470	0.64
0.30	-2.36	0.0	-0.42	0.0	0.0115	-0.000273	0.61
0.50	-3.36	0.75	-0.50	0.25	0.00825	0.0	0.67
1.00	-4.73	1.37	-0.41	0.57	0.00655	0.0	0.72
2.00	-6.24	1.96	-0.32	0.83	0.00496	0.0	0.52

*peak ground acceleration

TABLE 2-4
ATTENUATION RELATIONSHIP COEFFICIENTS
Crouse (1991)

$$\ln(y) = C_1 + C_2 M - C_3 \ln(R + C_4 e^{C_5 M}) + C_6 * D$$

where

y is spectral acceleration in g

D = depth of earthquake, in km (25 km was used for interface events, 55 km for intraslab events)

R = closest distance to rupture surface

Period	C ₁ *	C ₂	C ₃	C ₄	C ₅	C ₆	Standard Error ln(y)
0.033	4.611	0.65	2.09	63.7	0.128	-0.00397	0.631
0.1	0.512	1.12	1.93	1.58	0.608	0.00566	0.738
0.2	0.999	1.09	1.92	1.58	0.608	0.00531	0.675
0.3	-0.027	1.135	1.805	1.58	0.608	0.00429	0.656
0.5	-1.698	1.283	1.735	1.58	0.608	0.00311	0.659
1.0	-3.621	1.56	1.83	1.58	0.608	0.00114	0.748
2.0	-6.731	1.50	1.38	1.58	0.608	-0.00220	0.719

* C₁ converted to spectral acceleration from the values listed by Crouse (1991) for pseudo spectral velocity. Coefficients for periods of 0.3 and 0.5 seconds interpolated from coefficients given by Crouse (1991).

Figure 2-1a Computed mean and 5th- to 95th-percentile hazard curves for the 200 West Area site. Shown are results for peak horizontal acceleration and 5%-damped spectral acceleration at 0.3 and 2.0 seconds.

Figure 2-1b Computed mean and 5th to 95th-percentile hazard curves for the 200 East Area site. Shown are results for peak horizontal acceleration and 5%-damped spectral acceleration at 0.3 and 2.0 seconds.

Figure 2-2a Contributions of the five source types to mean hazard at the 200 West Area site. Shown are results for peak horizontal acceleration and 5%-damped spectral acceleration at 0.3 and 2.0 seconds.

Figure 2-2b Contributions of the five source types to mean hazard at the 200 East Area site. Shown are results for peak horizontal acceleration and 5%-damped spectral acceleration at 0.3 and 2.0 seconds.

Figure 2-3a Relative contribution of events in various magnitude and distance intervals to mean hazard at the 200 West Area site. Shown are results for peak horizontal acceleration and 5%-damped spectral acceleration at 0.3 and 2.0 seconds.

Figure 2-3b Relative contribution of events in various magnitude and distance intervals to mean hazard at the 200 East Area site. Shown are results for peak horizontal acceleration and 5%-damped spectral acceleration at 0.3 and 2.0 seconds.

Figure 2-4a Mean hazard curves for peak ground acceleration and 5%-damped spectral acceleration at seven periods of vibration for the 200 West Area site.

Figure 2-4b Mean hazard curves for peak ground acceleration and 5%-damped spectral acceleration at seven periods of vibration for the 200 East Area site.

Figure 2-5a Equal-hazard spectra (5% damping) for the 200 West Area site developed using general deep soil attenuation relationships.

Figure 2-5b Equal-hazard spectra (5% damping) for the 200 East Area site developed using general deep soil attenuation relationships.

This page intentionally left blank.

3.0 PRELIMINARY SITE RESPONSE ANALYSES

3.1 APPROACH

The purpose of the preliminary site response analyses was to re-evaluate the conclusion reached in the study by Washington Public Power Supply System (1985) that the characteristics of ground response to earthquake motions at the Hanford site are similar to those represented in the empirical strong motion data base from California used to develop the attenuation relationships used in the probabilistic seismic hazard analyses. The approach used to evaluate this issue was as follows:

- (1) Define characteristic site profiles and dynamic soil properties representative of conditions at California deep soil strong motion recording stations.
- (2) Define characteristic site profiles and dynamic soil properties representative of conditions at the DOE Hanford sites.
- (3) Select a set of analysis time histories representative of the events contributing to the site hazard.
- (4) Evaluate the relative response of the site profiles defined in (1) and (2) using the input motions identified in (3).

3.2 SITE VELOCITY PROFILES AND DYNAMIC PROPERTIES

The attenuation relationships used to develop the equal-hazard spectra presented in Section 2 are based on strong motion data recorded on firm soil site typically with soil depths in excess of 100 ft. Figure 3-1 shows the shear wave velocity profiles for a number of locations in active tectonic areas. Most of the profiles are at strong motion recording stations located primarily in the western US. These profiles may be broadly classified as S2 (Applied Technology Council, 1978), "deep cohesionless or stiff clay soils - including sites where the soil depth exceeds about 200 ft and the soil types overlying rock are stable deposits of sands graves, or stiff clays." The specific profiles are listed in Table 3-1. The velocity profiles were measured using either downhole or crosshole techniques. The depths of measured velocities range from about 100 ft to over 500 ft (Table 3-1). For those profiles that do not

terminate in rock, the estimated soil depths exceed about 100 ft. A velocity profile was defined to represent the typical shear wave velocity characteristics of a deep, firm alluvial soil site. This profile, labeled as "generic S2" on Figure 3-1, is used to represent the typical response of sites in the deep soil strong motion recording data base.

The soil profile developed to represent typical site conditions at the DOE Hanford sites was based primarily on data from studies conducted for facilities in the vicinity of the 200 East Area with consideration of velocity measurements obtained elsewhere on the Hanford Site. Figure 3-2 shows the data from the 200 East Area used in the evaluations. The three shear wave velocity profiles were used to develop the representative profile shown on Figure 3-2 as profile P1.

The measured shear wave velocities at the 200 East area are generally similar to those at other areas on the Hanford Site. Figure 3-3 compares the representative shear wave velocity profile developed for the 200 East Area with measured velocities for other sites. The velocity profiles are generally consistent, except that many of the other areas show evidence of high velocity layers at depth within the profile. These high velocity layers occur within the middle and lower Ringold formations, Miocene-Pliocene gravelly deposits that are locally cemented. The measured velocity profiles in the 200 East area do not extend far enough into the Ringold formation to penetrate the high velocity zone(s), if they are present at the 200 East area. Compression wave velocity measurements in the 200 West Area (Desser Atlas, 1985) do indicate the presence of high velocity layers within the Ringold formation at depths of 200 to 400 ft. Therefore, two alternative profiles, designated P2 and P3 on Figure 3-4, were developed to test the effect of the presence of high velocity layers in the Ringold formation on site responses. Profile P2 contains high velocity layers at the top of the middle Ringold and at the base of the lower Ringold. In profile P3 the lower high velocity layer is placed above the Ringold/basalt interface, creating two velocity reversals within the Ringold formation. Also shown on Figure 3-4 is the generic deep soil profile from Figure 3-1.

As shown on Figure 3-4, an average shear wave velocity of 8200 fps (2.5 km/sec) was selected for the base rock velocity for the Hanford soil profiles and an average velocity of 3280 fps (1 km/sec) was selected as the base rock velocity for the generic soil profile. These values were representative of the upper part of the shallow crustal velocity models discussed below. Site response calculations were then made with the 500 ft soil profiles P1, P2, and P3 compared to a 500 ft generic deep, firm soil profile.

Recent studies of the effects of local geology on site response have indicated that it is important to also consider the effects of the shallow crustal velocity structure on the site response. Therefore, relative response analyses were also performed for site velocity profiles that include the upper kilometer of the crust. Figure 3-5 shows the upper crustal velocity profiles used. The generic soil profile was extended to shallow crustal depths by attaching the generic shear wave velocity profile shown on Figure 3-1 to the upper portion of the crustal model of the San Francisco Bay area developed by Wald and others (1991). This profile is representative of typical crustal velocity profiles found in California.

The velocity profile for the basalts shown on Figure 3-5 was estimated considering surrounding well velocity and stratigraphic information. The model stratigraphy, thicknesses of basalts and interbeds, and layer density is based on data from well DC-3 (Landon, 1985), the closest well to the 200 East Area (within 5 km) that contains detailed stratigraphic information at depth. Density values were taken from a density versus depth plot for well DC-3 shown in Figure 5 from Saltus (1993). The following is a table of generalized ranges of density values used for the basalts and interbeds:

<u>Formation</u>	<u>Density (g/cm³)</u>
Grand Ronde Interbeds	2.65
Grand Ronde Basalts	2.85
Wanapum Basalts	2.82-2.88
Wanapum Interbeds	2.4-2.6
Saddle Mountains Basalts	2.80
Saddle Mountains Interbeds	2.02-2.32

Because velocity information was not available for the DC-3 well, P-wave velocities were assigned to individual layers by comparing velocities for the same or similar formations at similar depths in other wells. Digital P-wave velocity-depth profiles for two wells, Shell-33 (Yakima Minerals Well No. 1-33) and Shell-29 (Bissa No. 1-29) (Campbell, 1989; Campbell and Reidel, 1994) were used to define the compression wave velocities of various units. Both wells penetrate sub-basalt sediments (U.S. DOE, 1988). These wells are located approximately 20 km west and northwest of the 200 East Area site. Shear-wave velocity values were calculated from the assigned compression wave velocities using V_p/V_s ratios of 1.73 to 1.80 for the basalts provided by A. Rohay (pers. comm.). The V_p/V_s ratio for the interbed sediments was assumed to be equal to values obtained for the Ringold formation. Based on the velocities measured at the WNP-1, WNP-2, and WNP-4 nuclear power plant sites (WPPSS, 1985) a V_p/V_s ratio of 2.36 was used to estimate shear wave velocities for the interbed sediments.

The dynamic analyses were conducted using the equivalent-liner formulation for the dynamic soil properties. Figure 3-6 shows the strain-compatible shear modulus and damping relationships used in the analyses. These relationships were developed by the Electric Power Research Institute (EPRI) (1993) to represent the characteristics of deep firm soil deposits. The depth-dependent modulus and damping relationships represent the results of detailed evaluations of the present state of knowledge of the expected response of soils to strong ground motion.

The upper crustal materials below a depth of 500 ft, including the soils in the sedimentary interbeds within the basalt sequence, were assumed to respond linearly to the input ground motions. Lanning (1985) and Pujol and Smithson (1991) report Q values in the range of 20 to 50 for the upper portion of the basalt sequence in the Columbia Plateau. These represent compression wave Q values measured at frequencies in the range of 10 to 30 Hz. The results presented by Pujol and Smithson (1991) indicate that compression wave Q in the shallow basalts is approximately proportional to frequency. Viscoelastic theory suggests that shear wave Q is approximately one half of compression wave Q if one assumes there is no energy

dissipation in isotropic (bulk) compression. Thus, Q values at frequencies of 10 to 30 Hz would be approximately 10 to 25.

Q can be equated to two other representations of energy loss in wave propagation analysis. If liner viscoelastic wave propagation modeling is used (such as the site response analyses performed for this study using program SHAKE), then the material damping is equivalent to $1/2Q$. Thus, Q values in the range of 10 to 25 correspond to damping ratios of 2 to 5 percent. Q can also be related to the high frequency attenuation parameter κ developed by Anderson and Hough (1984) by the relationship $\kappa = H/QV_s$, where H is the portion of the crust over which the energy loss occurs. The appropriate value of H is 1 to 2 km (Silva and Darragh, 1992). The average shear wave velocity of the velocity profile for the basalts shown on Figure 3-5 is 2.6 km/sec. The resulting values of κ range from 0.015 to 0.038 seconds. The average value of κ obtained from analysis of typical California rock site strong motion recordings is 0.04 seconds (Silva and Darragh, 1992).

The measured Q in the basalt sequence is likely due to a combination of material damping and scattering from the large velocity contrasts in the alternating layers of basalt and sediments. The amount of energy loss due to scattering from this velocity structure was estimated using the site response techniques developed by Silva (1989). The attenuation of high frequency waves due to reflections from the interbeds was found to be equivalent to a κ value of 0.007 seconds. Therefore, it was assumed that the total energy loss in the upper kilometer of the basalts could be divided into a scattering κ of 0.007 seconds and an material damping κ of 0.008 to 0.031 seconds. The equivalent Q values for material damping are 20 to 50, corresponding to damping ratios of 1 to 4 percent. These values were used for material damping in the basalt sequence in the site response analyses.

Silva and Darragh (1992) found that Q for western US rocks could be estimated by the relationship $Q = \tau V_s$, with τ equal to 7 sec/km. Assuming that Q is proportional to shear wave velocity in the Wald and others crustal model, a value of τ of 10 sec/km produces the a value κ of 0.04 seconds, the average value of κ for WUS rock sites reported by Silva and

Darragh (1992). The resulting Q values and equivalent damping values for the upper 2 km of the Wald and others (1991) velocity model are:

Depth Range (km)	Shear Wave Velocity (km/sec)	Q	Damping Ratio
0.0 - 0.1	1.0	10	5.0%
0.1 - 0.5	1.95	20	2.5%
0.1 - 1.0	2.48	25	2.0%
1.0 - 2.0	2.77	28	1.8%

These damping values were used to conduct site response for the WUS crustal model.

3.3 SELECTION OF ANALYSIS RECORDS

Analysis of the relative contributions of various size event to the hazard (Figures 2-3a and 2-3b) indicates events of magnitude 6 or less occurring at distances of 15 km or less contribute most to the hazard over much of the frequency range of interest to short period structures. Table 3-2 lists eight recordings of magnitude ~6 earthquakes obtained on rock sites. Figure 3-7 shows a comparison of the response spectra of these events to the spectra of a magnitude 6 event predicted using the empirical rock site attenuation relationships developed by Geomatrix (1992). The spectra were all normalized to a peak acceleration of 0.25 g, representative of the level of ground motions with exceedance probabilities of 10^{-4} to 10^{-3} and representative of the median ground motion levels for a magnitude 6 event at a distance of 10 km predicted by the empirical attenuation relationships.

The accelerograms listed in Table 3-2 were all recorded at the surface of typical rock sites in California. Figure 3-8 shows a suite of rock site velocity profiles for the WUS strong motion recording stations listed in Table 3-3. Also shown on Figure 3-8 is a "generic" rock site velocity profile representative of the profiles shown on Figure 3-8. This rock site profile was used to de-convolve the free field recordings listed in Table 3-2 to base rock conditions for input into the soil site response analyses. For those site response analyses that considered only the response of the soil profiles, the rock motions were de-convolved to a depth of 50 ft, the point at which the generic rock velocity profile reached a shear wave velocity of 1 km/sec.

For those site response analyses that considered the response of the upper kilometer of crust, the rock motions were de-convolved through a velocity profile consisting of the Wald and others (1991) crustal model with the top 0.1 km replaced by the generic rock velocity profile shown on Figure 3-8. The generic rock profile of Figure 3.8 has an average velocity of 1.1 km/sec, very similar to the velocity of the top 0.1 km of the Wald and others (1991) model.

3.4 SITE RESPONSE ANALYSES

The site response analyses were conducted using the computer program SHAKE (Schnabel and others, 1972). The approach followed was to make comparative analyses of the response of the Hanford profile and the generic California deep soil profile to the same input motions. Each time history listed in Table 3-2 was used in a paired analysis, one computation with a Hanford profile and one computation with the generic deep soil profile. The computed surface motions for each profile were used to compute 5%-damped response spectra. Spectral ratios were then computed by dividing the response spectral ordinates computed for Hanford profile response motion by the response spectral ordinates computed for the generic profile response motion. Figure 3-9 shows the resulting spectral ratios for the response of Hanford profile P1 compared to the response of the generic deep soil profile for the eight individual input motions. Spectral ratios greater than 1.0 indicate that the motions at the surface of the Hanford profile would be larger than those at the surface of a generic deep soil profile representative of the data used to develop the attenuation relationships used to develop the equal-hazard spectra shown in Section 2. The eight individual response ratios were then used to compute a geometric mean response ratio to represent the typical difference in soil response.

The following sets of relative response analyses were performed considering only the response of the soil profiles:

Case	<u>Hanford Profiles</u>		<u>Generic Deep Soil Profiles</u>	
	<u>Velocity Profile</u>	<u>Shear Modulus and Damping Curves</u>	<u>Velocity Profile</u>	<u>Shear Modulus and Damping Curves</u>
1	P1	EPRI (1993)	Generic	EPRI (1993)
2	P2	EPRI (1993)	Generic	EPRI (1993)
3	P3	EPRI (1993)	Generic	EPRI (1993)
4	P1A	EPRI (1993)	Generic	EPRI (1993)

Figure 3-10 presents the median relative response ratios computed for these analyses cases.

Figure 3-10a shows the effect of the assumption of the presence of high velocity layers in the middle and lower Ringold formation on the relative response. If these layers are present, they result in only minor changes in the computed ground response.

Figure 3-10b shows the effect of the base rock shear wave velocity used for the Hanford profile on the computed relative response. The base rock velocity of profile P1 (Figure 3-4) was changed to 1 km/sec, the value for typical western US rock sites. The modified profile, P1A was then used to compute relative response ratios. The results indicate that the higher response for the Hanford profiles shown on Figure 3-10a is largely due to the high shear wave velocity of the basalts and the resulting velocity contrast at the soil/basalt interface. Thus it is important to consider the effects of the upper portion of the crust in evaluating the relative response.

Accordingly, the following relative response analyses were performed using velocity profiles that incorporate the upper kilometer of the crust:

Case	<u>Hanford Profiles</u>		<u>Generic Deep Soil Profiles</u>	
	<u>Velocity Profile(*)</u>	<u>Shear Modulus and Damping Curves</u>	<u>Velocity Profile</u>	<u>Shear Modulus and Damping Curves</u>
5	P1 + basalt(1 %)	EPRI (1993)	Generic + Wald	EPRI (1993)
6	P1 + basalt(4 %)	EPRI (1993)	Generic + Wald	EPRI (1993)
7	P2 + basalt(1 %)	EPRI (1993)	Generic + Wald	EPRI (1993)

* damping ratio in basalt sequence

The results of these relative response analyses are shown on Figure 3-11.

Figure 3-11a shows the relative response ratios computed using Hanford profiles P1 and P2. The results indicate that the response of the Hanford velocity profile is similar to that of the generic WUS deep soil profile when the response of the upper crust is considered. The change in the spectral ratios from those shown on Figure 3-11a results from less amplification in the basalt sequence that in the Wald and others (1991) crustal model due to the lack of a velocity gradient.

Figure 3-11b shows the effect of the use of the range of values for material damping in the basalt sequence defined in Section 3.2 on the computed spectral ratios. As one would expect, an increase in the damping ratio assigned to the basalt sequence results in a reduction in the relative response between the Hanford profile and the generic WUS profile, although the effect is not large. The effect of various damping ratios was also examined using the stochastic (RVT) ground motion analysis method of Silva (1989). The analyses were performed using the WUS generic deep soil site velocity model shown on Figure 3-5 and the Hanford crust model shown on Figure 3-5 with the soil properties replaced by preliminary results obtained for the 200 East site. The damping in the Wald and others (1991) rock profile was modeled using the average κ of 0.04 seconds for WUS rock sites (Silva and Darragh, 1992) and the damping in the basalt sequence was modeled using κ values of 0.01 to 0.04 seconds. Ground motions were computed for M 6 earthquakes occurring at distances of 5 and 10 km using both profiles and ratios of the resulting spectral accelerations computed as above. Figure 3-12 shows the resulting spectral ratios. As indicated, the Hanford profile shows lower response than the generic deep soil WUS profile. The use of κ values in the range of 0.01 to 0.04 second results in less than a 30 percent difference in the ratio of spectral response.

The comparisons shown above indicate that ground motions recorded at Hanford sites underlain by deep soil deposits are expected to be similar in character to ground motions representative of the empirical deep soil site strong motion data base used to develop the empirical attenuation relationships used in the probabilistic seismic hazard analysis presented in Section 2. Because of the differences in the velocity structure of the upper crust between typical WUS recording sites and sites at Hanford, the evaluation of relative response should

include consideration of the response of the upper crust in combination with the response of the soil deposits. These comparisons, shown on Figure 3-11, indicate that the empirical deep soil attenuation relationships may be slightly conservative for evaluating ground motions at the Hanford site. However, because the spectral ratios are relatively close to 1 and they are based on somewhat limited data, it is judged appropriate to use the equal-hazard developed from empirical deep soil site attenuation relationships to develop design spectra for the Hanford sites.

TABLE 3-1

FIRM, DEEP ALLUVIAL SOIL VELOCITY PROFILES SHOWN ON FIGURE 3-1

Profile Name	Source*	Profile Depth (ft)
Hollywood Storage	USGS OFR 82-833	98
Halls Valley		100
CIT Athenaeum	USGS OFR 84-681	79
Santa Barbara Cnty Courthse	USGS OFR 84-681	85
Agnews State Hosp	**	164
Capitola	**	99
Fremont Mission SJ	**	236
Gilroy 2 EPRI hole	**	700
Holiday Inn	USGS OFR 80-378	95
APEEL 2E-John Muir School	USGS OFR 91-311	98
Gilroy 3-Sewage Treat Plant	USGS OFR 91-311	197
Gilroy 4-San Ysidro School	USGS OFR 91-311	98
Stanford Linear Accelerator	USGS OFR 91-311	98
Oakland 1700 Webster	**	374
Santa Ana Courthouse	UCLA Eng. Dept. 64-15	4,500
445 Figueroa St., LA	NUREG/CR-1643	400
1800 Century Pk 761.960 y City	NUREG/CR-1643	401
3345 Wilshire Blvd, LA	NUREG/CR-1643	236
8244 Orion Blvd (Holiday Inn)		1,000
15910 Ventura Blvd, LA	NUREG/CR-1643	320
CIT Miliken Library, Pasadena	NUREG/CR-1643	35
Cholame-Shandon Array 5	USGS OFR 82-407	183
Taft Lincoln School	USGS OFR 82-407	197
Melendy Ranch Barn, Hollister	NUREG/CR-1643	340
Esso Refinery, Managua		164
El Centro	USGS OFR 84-562	802
Lotung LSST	EPRI NP-6154 Project 2225, vol. 1	197
Menlo Park VA	**	197

TABLE 3-1 (continued)

Profile Name	Source*	Profile Depth (ft)
Terminal Island	USGS OFR 80-378	70
Richmond City Hall	**	236
Public Utilities Bldg, LB	UCLA Eng. Dept. 64-15	8,300
LA Chamber of Commerce	UCLA Eng. Dept. 64-15	8,200
Cholame-Shandon Array 2		210

- * USGS OFR - U.S. Geological Survey Open File Report
UCLA - University of California, Los Angeles, Department of Engineering
NUREG - U.S. Nuclear Regulatory Commission NUREG
EPRI - Electric Power Research Institute

**Thiel, C. C. and J. F. Schneider (1993). "Investigations of thirty-three Loma Prieta earthquake strong motion recording sites." United States/Japan Loma Prieta earthquake project, *California Universities for Research in Earthquake Engineering and Building Contractors Society of Japan*.

TABLE 3-2
ROCK RECORDINGS USED IN SITE RESPONSE ANALYSES

Record #	Earthquake*	Date	M _w	M _L	Station	Record Code	Comp. (°)	EPD (km)	CLD (km)	PGA (g)
1	Coalinga Aftershock	83-07-21	5.9	6.0	Transmitter Hill	CO567	360	6.0	9.5	0.96
2	Coalinga Aftershock	83-07-21	5.9	6.0	Oil Field Fire Sta.; Pad	CO565P	360	8.5	11.3	0.22
3	Coalinga Aftershock	83-07-21	5.9	6.0	Oil Field Fire Sta.; Free Field	CO565F	360	8.5	11.3	0.19
4	Coalinga Aftershock	83-07-21	5.9	6.0	Skunk Hallow	CO561	360	10.0	12.4	0.23
5	Coalinga Aftershock	83-07-21	5.9	6.0	Temporary Station 03	CO546T03	000	13.3	15.3	0.12
6	Whittier Narrow	87-10-01	6.0	5.9	Garvey Res.	WHGARV	330	0.6	12.2	0.48
7	Whittier Narrow	87-10-01	6.0	5.9	Whittier Narrow Dam, Up Stream	WHWHDAMU	152	0.9	12.3	0.30
8	Whittier Narrow	87-10-01	6.0	5.9	4407 Jasper Street	WHJASP	040	8.1	14.9	0.22

* Both Earthquakes are thrust events.

EPD - Epicentral Distance

CLD - Distance to rupture surface/hypocenter

MHC-SD-M236A-TI-016
Revision 0

TABLE 3-3
WESTERN US ROCK SITE VELOCITY PROFILES SHOWN ON FIGURE 3-8

ROCK PROFILE SOURCES AND DEPTHS	
Profile Name	Source
Piedmont Jr. High School	1
Yerba Buena Island	2
APEEL 7-Pulgas Water	3
APEEL 10-Skyline	3
Gilroy-Gavilon College	3
APEEL 3E-Hayward CSUH	3
Anza - PFO	5
Anza - KNW	5
Gilroy #1	3
Lexington Dam	1
Pacific Heights, SF	1
Sierra Pt, So. SF	1
Haviland Hall, U.C.B	1
Point Bonita	1
Rincon Hill, SF	1
UCSC Lick Observ.	4
Site 300, Lawrence Liv. Lab	1
Hollister, SAGO South	1
Diamond Heights, SF	1
Memorial Stadium, U.C.B	1
Monterey City Hall	1
Belmont, Davis Dr.	1
Patterson Pass	4
Gilroy #6	3
San Francisco VA Hosp.	1
Martinez	4

Sources:

- 1) Agbabian Asso. (1993) "Borehole Geophysical Measurements at 24 Soil and Rock Sites." EPRI RP 3014-18.
- 2) Redpath Geophysics (not released)
- 3) USGS OFR 91-311
- 4) Agbabian Associates (not released)
- 5) Fletcher et al. (1990) "Near-surface velocities and attenuation at two boreholes near Anza, California, From Logging Data." Bull. Seis. Soc. Am. 80(4), 807-831.

Figure 3-1 Generic deep soil shear wave velocity profile (heavy solid curve) developed to represent the shear wave velocity profiles for western US recording stations listed in Table 3-1.

Figure 3-2 Published shear wave velocity profiles for locations in the 200 East Area and the selected representative profile P1 used in the site response analyses,

Figure 3-3 Comparison of the profile selected to represent the DOE Hanford sites with published velocity profiles from various locations on the Hanford Site: 200 West Area from URS Blume (1980); 400 Area from Bechtel (1970); WNP-1, WNP-2, and WNP-4 from WPPSS (1985); Shagit/Hanford from Northwest Energy Services Company (1981).

Figure 3-4 Alternative shear wave velocity profiles used to examine the effect of potential high velocity layers in the Ringold formation on site response. Also shown is the generic deep soil profile from Figure 3-1.

Figure 3-5 Upper crustal profiles use in the site response analyses. The Hanford profile is profile P1 from Figure 3-2 combined with the estimated velocities in the upper Columbia River Basalt group (labeled basalt). The western US profile is the generic soil profile from Figure 3-1 combined with the crustal model developed by Wald and others (1991) for the San Francisco Bay area (labeled Wald).

Figure 3-6 Generic strain-compatible shear modulus and damping curves developed by the Electric Power Research Institute (1993) to represent the properties of deep firm soil deposits.

Figure 3-7 Response spectra for selected magnitude 6 rock recordings (Table 3-2) compared to empirical rock spectra (Geomatrix, 1992). All spectra have been normalized to a peak acceleration of 0.25 g.

Figure 3-8 Generic rock shear wave velocity profile (heavy solid curve) developed to represent the shear wave velocity profiles for western US rock site recording stations listed in Table 3-3.

Figure 3-9 Ratio of the response of Hanford Profile P1 to the response of the generic soil profile for the eight input motions.

Figure 3-10 Geometric mean of response ratios for Hanford profiles (Figure 3-4) compared to generic deep soil profiles. The response analyses were computed using the EPRI (1993) soil modulus and damping relationships (Figure 3-6). (a) Effect of Hanford velocity profile assumptions on relative response. (b) Effect of base rock velocity on relative response.

Figure 3-11 Geometric mean of response ratios for Hanford profiles (Figure 3-5) compared to generic deep soil profiles when the response of the upper kilometer of crust is included in the analysis. The response analyses were computed using the EPRI (1993) soil modulus and damping relationships (Figure 3-6). (a) Effect of Hanford velocity profile assumptions on relative response. (b) Effect of damping in basalt sequence on relative response.

Figure 3-12 Effect of assumed κ for basalt sequence on relative response. Ground motions were computed using RVT (Silva, 1989) for a M 6 earthquake at distances of 5 and 10 km.

4.0 SITE RESPONSE ANALYSES FOR THE 200 WEST AND 200 EAST AREAS

This section presents an updated analysis of the response of Hanford soils specifically for the 200 West and East Areas and the proposed waste tanks site. The analysis uses the site-specific velocity and material properties obtained by Shannon and Wilson (1994a, 1994b).

4.1 SITE VELOCITY PROFILES AND DYNAMIC PROPERTIES

The generic deep soil profiles described in Section 3 (Figures 3-1 and 3-5) were again used to represent the properties of typical Western US deep soil recording sites. Shannon and Wilson (1994a, 1994b) have developed a recommended set of dynamic properties for the Multi-function Waste Tank Facilities located at the 200 West and East Areas. These properties were used in the relative response analyses.

Figure 4-1 compares the velocity profiles developed by Shannon and Wilson (1994a, 1994b) with the profiles developed in Section 3.2. The 200 West and 200 East profiles are similar to the Hanford profile P1, except for higher velocities near the surface. Shannon and Wilson (1994a, 1994b) measured velocities to a depths of 140 ft in the 200 West and 200 East areas. Below these depths, they used previous measurements to develop velocity profiles, and thus obtain very similar values to those presented in Section 3.2.

Figure 4-2 shows the shear modulus reduction and damping relationships recommended by Shannon and Wilson (1994a, 1994b) compared to the EPRI (1993) relationships used in the analyses described in Section 3. The site-specific modulus reduction and damping relationships fall within the lower ranges of the generic curves developed by EPRI (1993).

4.2 SELECTION OF ANALYSIS RECORDS

As discussed in Section 3.3, events of magnitude 6 or less occurring at distances of 15 km or less contribute most to the hazard over much of the frequency range of interest to short period

structures. Therefore, the accelerograms listed in Table 3-2 were again used for the relative response analyses. Because of the interest in ground motion levels with exceedance frequencies near 10^{-4} , the time histories were normalized to a peak acceleration of 0.35 g before input into the analysis.

4.3 SITE RESPONSE ANALYSES

The site response analyses were again conducted using the computer program SHAKE (Schnabel and others, 1972). The following sets of relative response analyses were performed:

Case	Hanford Profiles		Generic Deep Soil Profiles	
	Velocity Profile	Shear Modulus and Damping Curves	Velocity Profile	Shear Modulus and Damping Curves
8	P1	EPRI (1993)	Generic	EPRI (1993)
9	200 West	200 West	Generic	EPRI (1993)
10	200 East	200 East	Generic	EPRI (1993)
11	200 East	EPRI (1993)	Generic	EPRI (1993)
12	P1 + Basalt(1%)	EPRI (1993)	Generic + Wald	EPRI (1993)
13	200East + Basalt	200 East	Generic + Wald	EPRI (1993)

Figures 4-3 through 4-5 present the median relative response ratios computed for the paired profiles listed above.

Figure 4-3 shows that similar spectral ratios were obtained for input rock motions scaled to 0.25 g and 0.35 g. These results, together with the comparisons shown on Figure 3-12, indicate that the computed spectral ratios not very sensitive to the level motion within the range of ground motions of interest.

Figure 4-4 shows results obtained considering only the response of the soil deposits (Cases 9 through 11). Figure 4-4a compares the spectral ratios computed using the P1 profile to those computed using the site-specific soil properties developed by Shannon and Wilson (1994a, 1994b). The results obtained using the site-specific properties indicate lower spectral response

ratios than the preliminary P1 profile. The comparisons shown on Figure 4-4b indicate that the difference between the relative response ratios computed using the site-specific properties developed by Shannon and Wilson (1994a, 1994b) and those developed in Section 3 is largely due to the difference in the modulus reduction and damping relationships.

Figure 4-5 shows the relative response ratios computed considering the response of the upper kilometer of the crust. The results are similar to those presented in Section 3, Figure 3-11, and indicate that ground motions recorded on deep soil sites in the 200 East and 200 West areas may tend to be slightly less than motions recorded on typical deep soil sites in California.

Based on these comparisons, it is again concluded that the ground motion characteristics at the 200 West and 200 East Area site are adequately represented by empirical attenuation relationships developed using California deep soil site strong motion data.

Figure 4-1 Shear wave velocity profiles used in relative site response analyses for the 200 West and 200 East Areas. Shown are the velocity profiles developed by Shannon and Wilson (1994a, and 1994b). Also shown are the P1 profile from Figure 3-2 and the generic deep soil profile from Figure 3-1.

Figure 4-2 Strain-compatible shear modulus and damping curves recommended by Shannon and Wilson (1994a, 1994b) for the waste tank sites near 200 West and 200 East Areas compared to the relationships developed by EPRI (1993) from Figure 3-6.

Figure 4-3 Comparison of response spectral ratios computed using input motions scaled to 0.25 and 0.35 g.

Figure 4-4 Geometric mean response ratios for Hanford profile P1 and the 200 West and 200 East site-specific profiles computed considering only the response of the soil deposit. (a) Comparison of relative response ratios computed using the generalized profile P1 and the site-specific profiles for the 200 West and 200 East areas. (b) Influence of modulus and damping relationships on relative response for the 200 East site-specific profile.

Figure 4-5 Geometric mean response ratios computed considering the response of the upper kilometer of crust. Shown are the results for the P1 profile and the 200 East Area waste tank profile, both combined with the basalt sequence velocity profile (Figure 3-5).

5.0 DESIGN SPECTRA

The analyses presented in Sections 3 and 4 indicate that the equal-hazard spectra presented in Section 2 are appropriate for use in evaluating the seismic response of Hanford facilities. The hazard levels of primary interest are in ground motions with return periods of 2,000 to 10,000 years. Figure 5-1 compares the spectral shapes developed from the equal-hazard spectra for these return periods shown on Figures 2-5a and 2-5b. As indicated on the figure, there is very little difference between the spectral shapes for the two sites and the two return periods.

The equal-hazard response spectra for damping ratios of 0.5%, 2%, 7%, 10%, and 12% were developed from the hazard results computed for 5% damping using relationships between spectral amplification developed by Newmark and Hall (1978) and Sadigh (1983). At all damping levels the differences between the spectral shapes for the two return periods and the two sites are less than five percent.

The results shown on Figures 5-1 and 5-2, as well as examination of the equal-hazard spectra shown on Figures 2-5a and 2-5b indicate that a uniform spectral shape is applicable to ground motions with return periods of 500 years and longer at both the 200 West and 200 East sites. Accordingly, it is recommended that a single spectral shape be used as to develop design spectra. Figure 5-3 shows the spectral shapes for the six damping ratios developed by enveloping the spectral amplifications shown on Figure 5-2. The recommended spectral amplification factors are:

Spectral Acceleration Amplification Factor for Damping Ratio (%):						
<u>Period</u>	<u>0.5</u>	<u>2.0</u>	<u>5.0</u>	<u>7.0</u>	<u>10.0</u>	<u>12.0</u>
0.033	1.000	1.000	1.000	1.000	1.000	1.000
0.100	3.776	2.769	2.098	1.864	1.594	1.447
0.200	4.828	3.497	2.625	2.312	1.972	1.809
0.300	4.644	3.370	2.532	2.229	1.907	1.747
0.500	3.876	2.827	2.128	1.883	1.604	1.469
1.000	2.630	1.944	1.467	1.292	1.114	1.028
2.000	1.097	0.815	0.625	0.556	0.494	0.462

Figure 5-4 compares the recommended spectral shape for 5% damping to the Newmark and Hall (1978) spectral shape for soil sites. As can be seen, the two spectral shapes are generally similar.

Figure 5-1 Equal-hazard spectral shapes (5% damping) for the 200 West and 200 East Area sites developed using general deep soil attenuation relationships.

Figure 5-2 Equal-hazard spectral shapes for the 200 West and 200 East Area sites for damping ratios of 0.5%, 2%, 5%, 7%, 10%, and 12%.

Figure 5-3 Recommended design spectral shapes for the 200 West and 200 East Areas based on equal-hazard spectra for damping ratios of 0.5% to 12%.

Figure 5-4 Comparison of the recommended design spectral shape and the Newmark and Hall (1978) medial spectral shape for soil sites.

6.0 REFERENCES

- Anderson, J.G., and S.E. Hough, 1984, A model for the fourier amplitude spectrum of acceleration at high frequencies: Bulletin of the Seismological Society of America, v. 74, p. 1969-1993.
- Applied Technology Council, 1978: Tentative provisions for the development of seismic regulations for buildings: Applied Technology Council, ATC-3-06, Washington D.C., 599 p.
- Bechtel, 1970, Site investigation report for the Fast Flux Test Facility, Richland, Washington: Report prepared for WADCO, Richland, Washington.
- Bender, B., 1984, Seismic hazard estimation using a finite fault rupture model: Bulletin of the Seismological Society of America, v. 74, m. 5, p. 1899-1923.
- Campbell, K.W., 1993, Empirical prediction of near-source ground motion from large earthquakes: Proceedings of the International Workshop on Earthquake Hazard and Large Dams in the Himalaya: Sponsored by the Indian National Trust for Art and Cultural Heritage (INTACH), January 15-16, New Delhi, India.
- Campbell, N.P, 1989, Structural an stratigraphic interpretation of rocks under the Yakima fold belt, Columbia Basin, based n recent surface mapping and well data: *in* Reidel, S.P., and P.R. Hooper, eds., Volcanism and tectonism in the Columbia River flood-basalt province: Geological Society of America, Special Paper 239, p. 209-222.
- Campbell, N.P, and S.P. Reidel, 1994, Further exploration for gas warranted in Columbia basin: Oil & Gas Journal, May 2, p. 127-132.
- Crouse, C.B., 1991, Ground-motion attenuation equations for earthquakes on the Cascadia subduction zone: Earthquake Spectra, v. 7, p. 210-236.
- Dames & Moore, 1988, Seismic design data, Grout Vaults, Hanford, WA: report prepared for Kaiser Engineers, Job. No. 10805-282-016, October.
- Dames & Moore, 1989a, Seismic design data, Process Condensate Treatment Facility, Hanford, WA: report prepared for Kaiser Engineers, Job. No. 10805-383-016, August.
- Dames & Moore, 1989b, Seismic design data, Hanford Waste Vitrification Plant, Hanford, WA: report prepared for Kaiser Engineers, Job. No. 10805-383-016, November.
- Dresser Atlas, 1985a Report of borehole seismic analysis, well RRL-05: report prepared for Rockwell International, May 2.

- Dresser Atlas, 1985b Report of borehole seismic analysis, well RRL-8: report prepared for Rockwell International, May 2.
- Dresser Atlas, 1985c Report of borehole seismic analysis, well RRL-16: report prepared for Rockwell International, May 2.
- Electric Power Research Institute (EPRI), 1993, Guidelines for determining design basis ground motions: EPRI TR-102293, Project 3302, 5 vol., November.
- Geomatrix Consultants, 1992, Seismic Ground Motion Study for the San Francisco Bay Bridge: Report prepared for the California Department of Transportation, March.
- Geomatrix Consultants, 1993, Probabilistic seismic hazard analysis, DOE Hanford Site, Washington: report prepared for Westinghouse Hanford Company, December.
- Joyner, W.B., and D.M. Boore, 1982, Prediction of earthquake response spectra: US Geological Survey Open File Report 82-977.
- Landon, R.D., 1985, Deep borehole stratigraphic correlation charts: SD-BWI-DP-35, Rev. 3, Rockwell Hanford Operations, Richland, Washington.
- Lanning, E., 1985, PhD. Thesis, summarized in University of Washington, 1985, Annual technical report, 1985, on earthquake monitoring of eastern Washington: Geophysics Program, University of Washington, Seattle, report prepared for the U.S. Department of Energy under contract no. EY-76-S-06-2225.
- Newmark, N.H., and W.J. Hall, 1978, Development of criteria for seismic review of selected nuclear power plants: U.S. Nuclear Regulatory Commission Report NUREG/CR-0098.
- Northwest Energy Services Company, 1981, Geophysical Investigations Skagit/Hanford Nuclear Project Site, Hanford Site, Washington: Preliminary Safety Analysis Report, Appendix 2L, Amendment No. 23, December.
- Pujol, J., and Smithson, S., 1991, Seismic wave attenuation in volcanic rocks from VSP experiments: Geophysics, v. 56, p. 1441-1455.

- Sadigh, K., 1983, Considerations in the development of site-specific spectra: Proceedings of Conference XXII; Site-Specific Effects of Soil and Rock on Ground Motions and the Implications for Earthquake Resistant Design, U.S. Geological Survey Open-File Report 83-845, 423-458.
- Sadigh, K., J.A. Egan, and R.R. Youngs, 1986, Specification of ground motion for seismic design of long period structures (abs.): Earthquake Notes, v. 57, n. 1, p. 13. relationships printed in W.B. Joyner and D.M. Boore, 1988, Measurement, characterization, and prediction of strong ground motion: *in* Earthquake Engineering and Soil Dynamics II - Recent Advances in Ground Motion Evaluation, ASCE Geotechnical Special Publication 20, p. 43-102.
- Saltus, R.W., 1993, Upper-crustal structure beneath the Columbia River Basalt Group, Washington: Gravity interpretation controlled by borehole and seismic studies: Geological Society of America Bulletin, v. 105, p. 1247-1259.
- Schnabel, P.B., J. Lysmer, and H.B. Seed, 1972, SHAKE: a computer program for earthquake response analysis of horizontally layered sites: Earthquake Engineering Research Institute, University of California, Berkeley, Report EERC 72-12.
- Shannon and Wilson, Inc., 1994a, Geotechnical Investigation KEH W-236A, multi-function waste tank facility, Hanford Site, Richland, Washington: report prepared for SCM Consultants, H-1053-05, January.
- Shannon and Wilson, Inc., 1994b, Geotechnical Investigation KEH W-236A, multi-function waste tank facility, 200 West area addition, Hanford Site, Richland, Washington: report prepared for Kaiser Engineers Hanford Company, Document No. W236A-T1-MT5, H-1070-50, April.
- Silva, W., 1989, Site dependent specification of strong ground motion; in Dynamic Soil Property and Site Characterization, Proceedings of the Workshop sponsored by the National Science Foundation and the Electric Power Research Institute, Palo Alto, California.
- Silva, W.J. and R.B. Darragh, 1992, Engineering characterization of strong ground motion recorded at rock sites: Final Draft Report Submitted to the Electric Power Research Institute, EPRI RP 2556-48.
- URS Blume, 1980, Z Plant report
- Wald, D.J., D.V. Helmberger, and R.H. Heaton, 1991, Rupture model of the 9189 Loma Prieta earthquake from the inversion of strong-motion and broadband teleseismic data: Bulletin of the Seismological Society of America, v. 81, p. 1540-1572.

Washington Public Power Supply System (WPPSS), 1985, Site-specific response spectra for the WNP-2 power plant: Final Safety Analysis Report for Unit WNP-2, Appendix 2.5Q, Amendment No. 36, December.

This page intentionally left blank.

Attachment

HORIZONTAL AND VERTICAL SEISMIC
RESPONSE SPECTRA PLOTS

This page intentionally left blank.

Horizontal Spectral Acceleration Amplification Factor
for Damping Ratio (%):

<u>Frequency</u>	<u>0.5</u>	<u>2.0</u>	<u>5.0</u>	<u>7.0</u>	<u>10.0</u>	<u>12.0</u>
30.3	1.000	1.000	1.000	1.000	1.000	1.000
10.0	3.776	2.769	2.098	1.864	1.594	1.447
5.0	4.828	3.497	2.625	2.312	1.972	1.809
3.33	4.644	3.370	2.532	2.229	1.907	1.747
2.0	3.876	2.827	2.128	1.883	1.604	1.469
1.0	2.630	1.944	1.467	1.292	1.114	1.028
0.5	1.097	0.815	0.625	0.556	0.494	0.462

Vertical Spectral Acceleration Amplification Factor*
for Damping Ratio (%)

<u>Frequency</u>	<u>0.5</u>	<u>2.0</u>	<u>5.0</u>	<u>7.0</u>	<u>10.0</u>	<u>12.0</u>
30.3	1.000	1.000	1.000	1.000	1.000	1.000
10.0	3.776	2.769	2.098	1.864	1.594	1.447
5.0	4.828	3.497	2.625	2.312	1.972	1.809
3.33	4.644	3.370	2.532	2.229	1.907	1.747
2.0	2.584	1.885	1.419	1.225	1.069	0.979
1.0	1.753	1.296	0.978	0.861	0.743	0.685
0.5	0.731	0.543	0.417	0.371	0.329	0.308

* Vertical spectral amplification factor is based on preliminary draft revision of Standard ASCE 4-86. The vertical spectral acceleration is equal to the horizontal at frequencies above 3.3 Hz (0.30 sec) and 0.6667 times the horizontal below 3.3 Hz. The ground motion at the Hanford Site is dominated

This page intentionally left blank.

This page intentionally left blank.

(2D) II Print II 2 Nov 1994 II 41411.plt II seed=41411

This page intentionally left blank.