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April 23, 2009

Document Control Desk
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001

Attention: Mr. Jeffery A. Ciocco

Docket No. 52-021
MHI Ref: UAP-HF-09187

Subject: MHI's Responses to US-APWR DCD RAI No. 211-1946

Reference: 1) "Request for Additional Information No. 211-1946 Revision 1, SRP Section: 03.07.01 – Seismic Design Parameters, Application Section: 3.7.1, dated 2/25/2009.
2) "MHI's Responses to US-APWR DCD RAI No. 211-1946, UAP-HF-09112, dated 3/25/2009.

With this letter, Mitsubishi Heavy Industries, Ltd. ("MHI") transmits to the U.S. Nuclear Regulatory Commission ("NRC") a document entitled "Response to Request for Additional Information No. 211-1946, Revision 1."

Enclosed are the responses to the remaining 4 RAIs contained within Reference 1. Three additional RAI responses contained within Reference 1 were previously provided in Reference 2.

Please contact Dr. C. Keith Paulson, Senior Technical Manager, Mitsubishi Nuclear Energy Systems, Inc. if the NRC has questions concerning any aspect of this submittal. His contact information is provided below.

Sincerely,

Y. Ogata

Yoshiki Ogata,
General Manager- APWR Promoting Department
Mitsubishi Heavy Industries, LTD.

Enclosure:

1. Response to Request for Additional Information No. 211-1946, Revision 1

D081
NRO

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Enclosure 1

UAP-HF-09187
Docket No. 52-021

Response to Request for Additional Information No. 211-1946, Revision 1

April, 2009

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

4/23/2009

US-APWR Design Certification

Mitsubishi Heavy Industries

Docket No. 52-021

RAI NO.:	**NO. 211-1946 REVISION 1**
SRP SECTION:	**03.07.01 – Seismic Design Parameters**
APPLICATION SECTION:	**3.7.1**
DATE OF RAI ISSUE:	**02/25/09**

QUESTION NO. RAI 3.7.1-3:

Section 3.7.1.1 of the DCD includes a description of the generation of the artificial ground motion time histories for use as the input motion for the US-APWR standard plant. It is not clear if the artificial ground motion time histories were generated based on seed recorded time histories. The acceptance criteria of the SRP subsection 3.7.1.II.1B states that the artificial ground motion time histories that are not based on seed recorded time histories should not be used. Confirm that the artificial time histories are generated based on seed recorded time histories. Provide justification if the artificial ground motion time histories are not generated based on seed recorded time histories. Also, describe the seed motions used and how they were modified to generate the artificial time histories.

ANSWER:

Method of Generation of Artificial Time Histories

The artificial time histories were not generated using seed recorded time histories. Instead, they were synthesized using Gaussian white noise generated from random numbers. The following method was used.

(1) The 5% damped target response spectrum was converted to a power spectral density function (PSDF).

(2) The PSDF and random phase angles were combined to produce the time history.

(3) The time history was multiplied by a time domain enveloping function.

(4) The time history was modified to match the design spectra at multiple damping values: 0.5%, 2%, 5%, 7% and 10%.

(5) The peak acceleration was adjusted to match that of the target response spectrum.

(6) Iterations of steps 4 and 5 were performed until the time history matched the target response spectrum at the multiple damping values: 0.5%, 2%, 5%, 7% and 10% to the tolerance required by the Standard Review Plan 3.7.1.

In step (2), the PSDF was first converted to the Fourier Amplitude Spectrum (FAS) through Equation (1) in Appendix A to SRP Section 3.7.1, then random phase angles were assigned to the frequencies comprising the FAS to create the real and imaginary parts of the Fourier transform of the time history, and finally the inverse Fourier Transform was taken to obtain the time history. Details of the method used to spectrally match the earthquake time histories to the design response spectrum in step (4) are described in Lilhanand and Tseng (1987), and Hirasawa and Watabe (1992).

Justification for Using Synthesized Time Histories

We have generated three time histories using ground motion recordings of earthquakes as the seeds, and have shown that the characteristics of these time histories, which we refer to as modified seeded time histories, are similar to those of the artificial time history described in the DCD, which we refer to as a synthesized time history. The synthesized time history described in the DCD is therefore an appropriate time history representation of the modified Regulatory Guide 1.60 response spectrum.

We chose the Station 3 recording of the December 23, 1985 M6.8 Nahanni earthquake, and the Station WRI (Wrightwood Swarthout) and BAL (Mt Baldy) recordings of the January 14, 1994 M6.7 Northridge earthquake, as the seed ground motions for generating the time histories. The Nahanni record was chosen because it is representative of earthquake source, wave propagation and site characteristics in a region of North America that is remote from plate boundaries. These characteristics are considered to be representative of those in the Central and Eastern United States (Risk Engineering, 2001). The Northridge recordings were selected because, like the Nahanni Station 3 recording, they have the required durations and correlation (statistical independence among the three components comprising each time history).

The recorded time histories were spectrally matched to the target response spectrum at the damping values of 0.5%, 2%, 5%, 7%, and 10%, using the respMatch code of Dr. N. Abrahamson. This code is based on a low frequency modification (Abrahamson, 1992) of the same procedure (Lilhanand and Tseng, 1987) used in spectrally matching the DCD synthesized time history. This method is designed to retain non-stationary features of the input ground motion in the course of spectrally matching it to the target spectra.

The three-component accelerograms of the synthesized time histories are compared with those of each of the modified seeded time histories in Figures 1, 2, and 3. The overall characteristics of the four histories are similar. In particular, the amplitude of the accelerograms is time varying (non-stationary) and the character of this variation is similar in each set of records. For example, the amplitudes are relatively small initially and then increase to the strong shaking portion as the time progresses. The shaking levels then gradually decrease toward the end of the records.

This characteristic is evident in Figures 4, 5, and 6; which compare the normalized Arias Intensity plots of cumulative energy of the synthesized time history with those of each of the three modified seeded histories. All time histories show an initial time interval of gradual energy build up, followed by a ramp of rapid energy accumulation, and then followed by a gradual tapering of energy accumulation. There are small differences in duration (as defined in SRP Section 3.7.1-IIB) between the various sets of records, but all four sets easily satisfy the 6-second minimum SRP duration criterion based on Arias Intensity, as indicated in the legends of Figures 4, 5, and 6; where the durations are listed.

Using a 3rd order Butterworth filter and a zero-phase digital filter, all four sets of records were filtered to display the ground motions in different frequency bands. The unfiltered and band-pass filtered accelerograms of the synthesized time histories are shown in Figures 7, 8, and 9; and those of the modified seeded time histories are shown in Figures 10 though 18. Five frequency bands were selected, as listed on these figures, and the resulting ground motions clearly reveal the non-stationary nature of the high, intermediate, and low frequency motions in each set of records.

Lastly, the time-varying frequency content of each set of records was computed and displayed in spectrograms, which show the power spectral density as a function of time. Matlab's "spectrogram" function was used for the calculations; the time window of each power spectral density calculation was 0.025 second, and the power spectral density values at each of the 512 frequencies in the range of 0.195 Hz to 100 Hz were smoothed with a 400-point Hamming window. The spectrograms of the synthesized time history are compared with those of each of the three modified seeded histories in Figures 19 through 27. Variations in the power spectral density with time are observed that correspond to the variations in the band-pass filtered accelerograms described above. This non-stationary character in frequency content exists in the synthesized time histories as well as in the modified seeded time histories.

Conclusion

We have generated 3 three-component time histories using ground motion recordings as seeds, and have shown that the characteristics of these modified seeded time histories are similar to those of the synthesized time history described in the DCD. These characteristics include (1) the non-stationary amplitude represented in the shape of the Arias Intensity plot of cumulative energy and in band-pass filtered accelerograms, and (2) the non-stationary frequency content represented in band-pass filtered time histories and spectrograms. We conclude that the synthesized time history described in the DCD is therefore an appropriate time history representation of the modified Regulatory Guide 1.60 response spectrum.

References

Abrahamson, N.A. (1992). Non-Stationary Spectral Satching. Seismological Research Letters 63, p. 30.

Lilhanand, K. and W.S. Tseng (1987). Generation of Synthetic Time Histories Compatible with Multi-Damping Design Response Spectra. 9th SMiRT , page 105-110.

Hirasawa, M. and M. Watabe (1992). Generation of Simulated Earthquake Motions Compatible with Multi-Damping Design Response Spectra. Earthquake Engineering,10th WCEE, p. 807 – 810. Balkeema, Rotterdam, ISBN 90 54 10 060 5.

Munro, P. and D. Weichert (1989). The Saguenay Earthquake of Nov. 25, 1988: processed strong motion records, Geological Survey. Canada Open-file Rept. 1996.

Risk Engineering, Inc. (2001). Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-consistent Ground Motion Spectra Guidelines. NUREG/CR-6728.

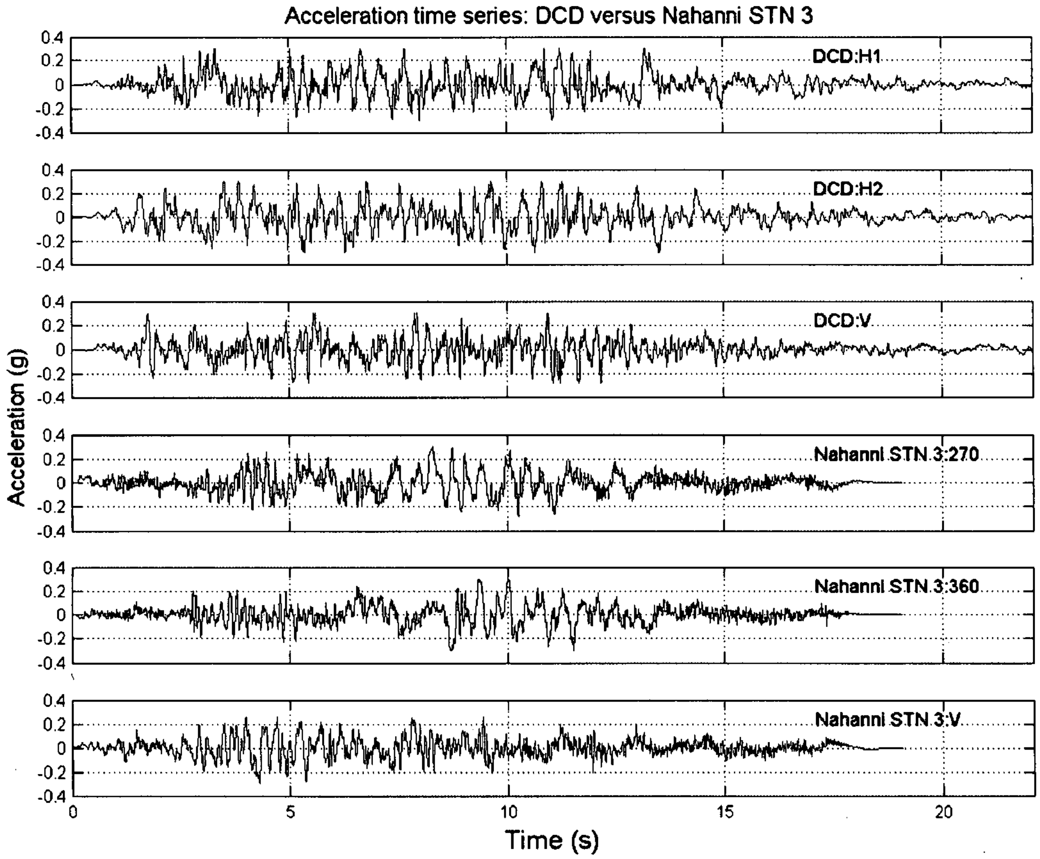


Figure 1. Three-component accelerograms of the DCD synthesized time history (top) and the Nahanni STN3 modified seeded time history (bottom).

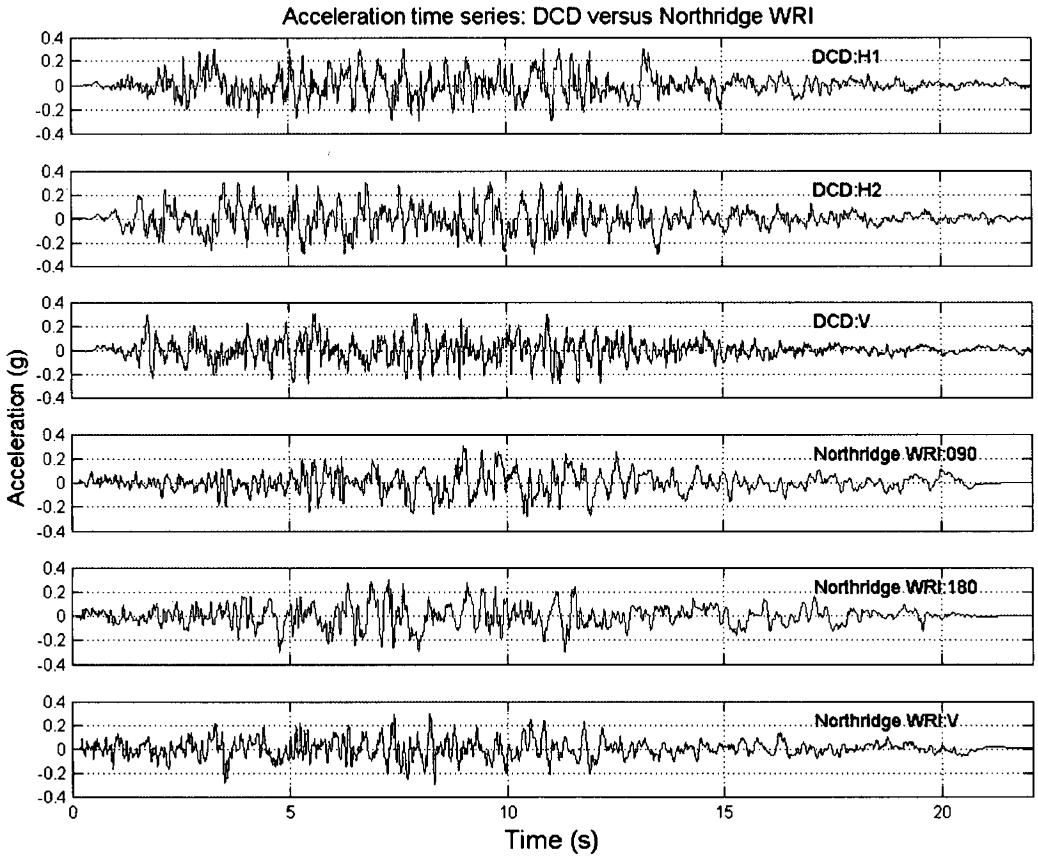


Figure 2. Three-component accelerograms of the DCD synthesized time history (top) and the Northridge WRI modified seeded time history (bottom).

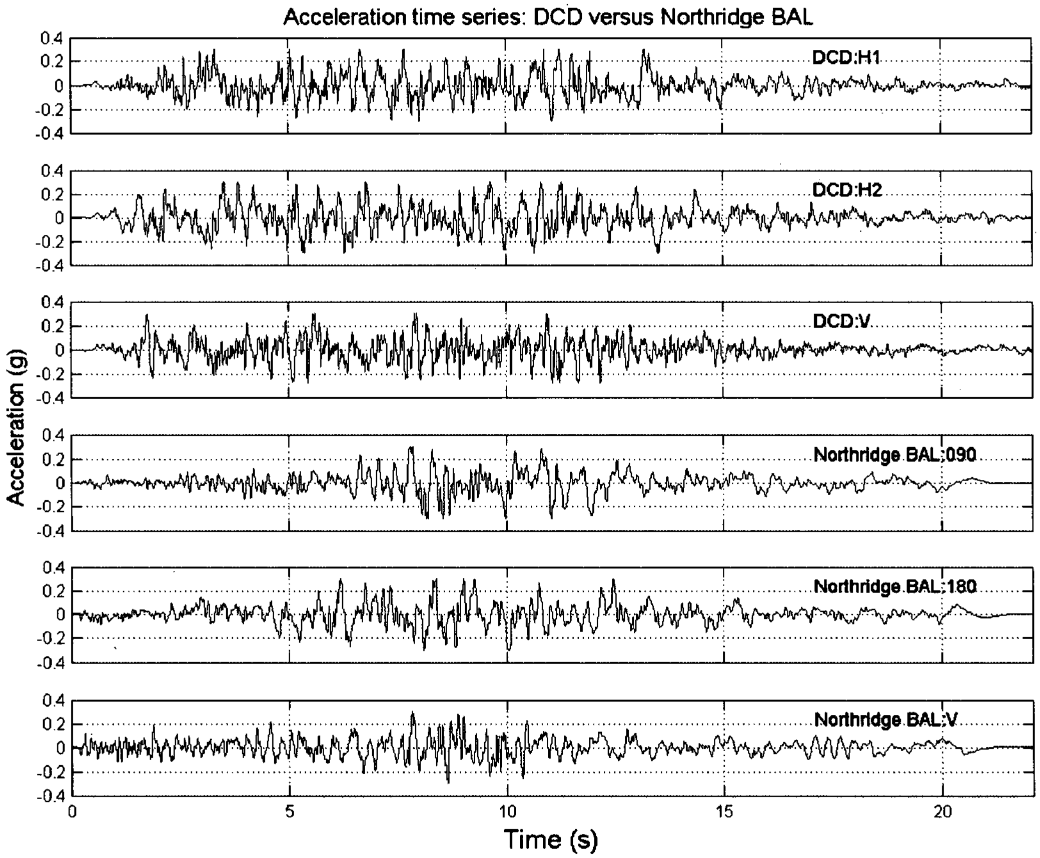


Figure 3. Three-component accelerograms of the DCD synthesized time history (top) and the Northridge BAL modified seeded time history (bottom).

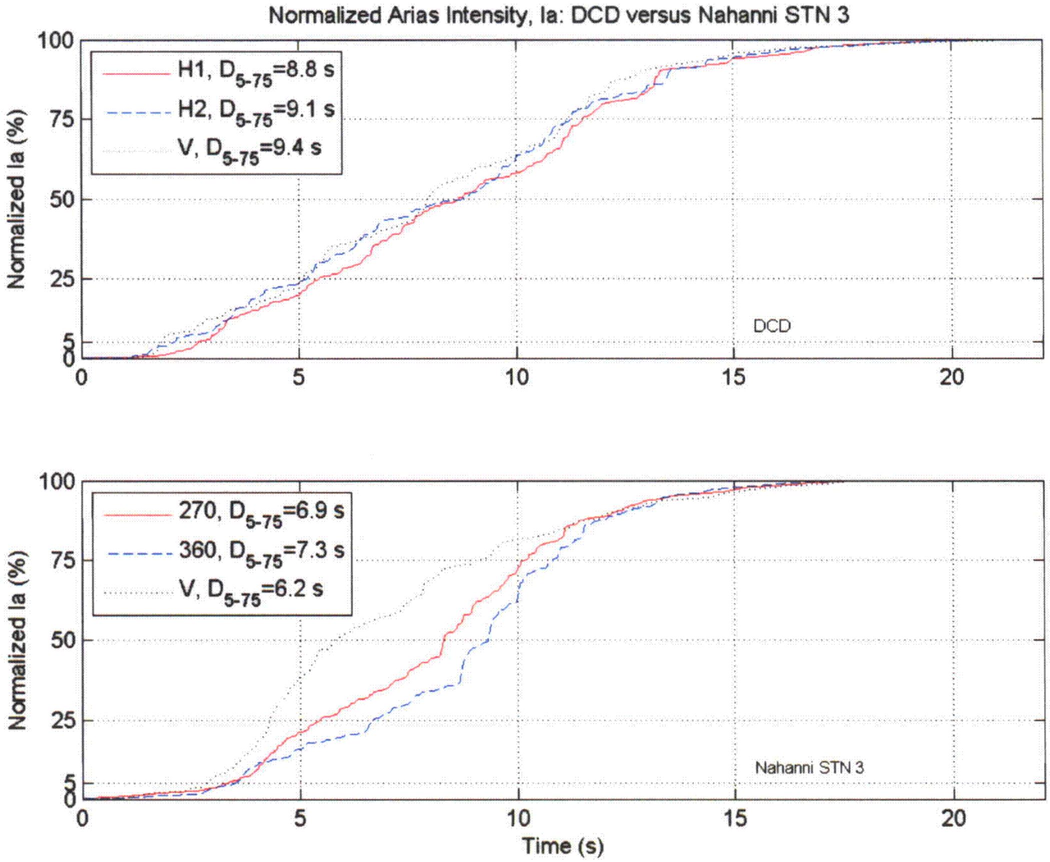


Figure 4. Normalized Arias Intensity of the three-component accelerograms of the DCD synthesized time history (top) and the Nahanni STN3 modified seeded time history (bottom). Values of duration, $D_{5\text{-}75}$, defined as the time interval between the 5% and 75% normalized Ia levels, are listed in the legends.

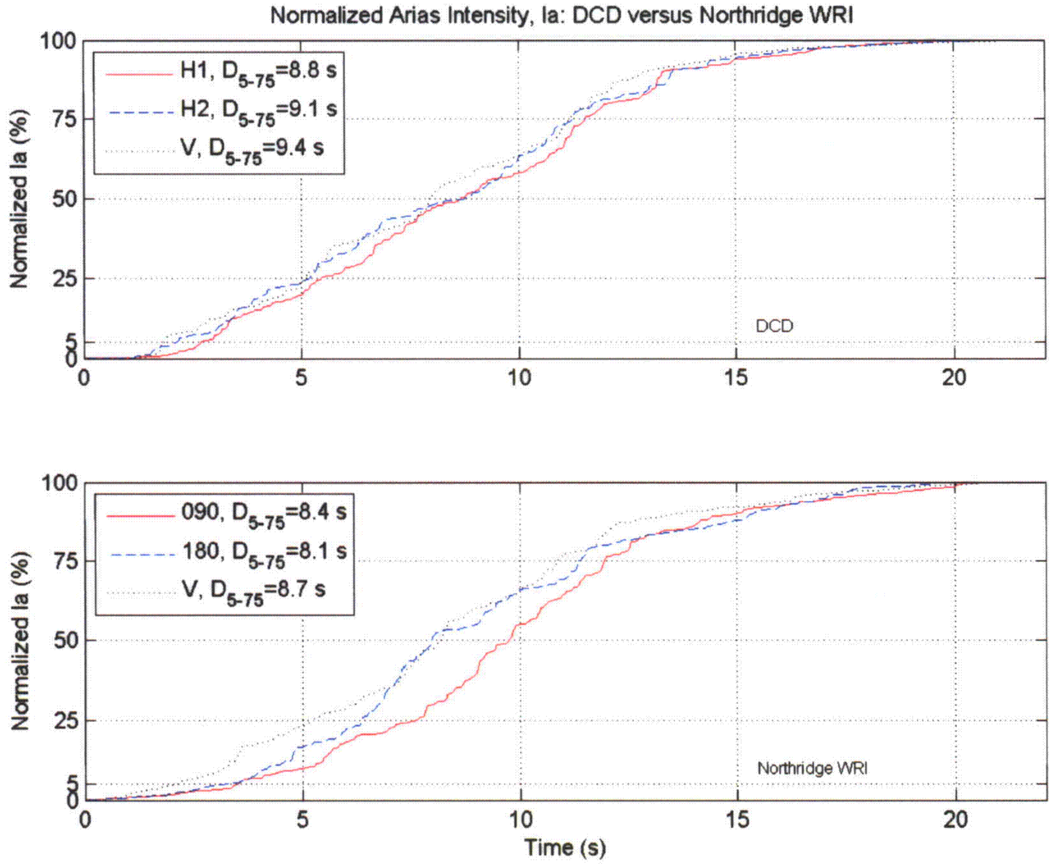


Figure 5. Normalized Arias Intensity of the three-component accelerograms of the DCD synthesized time history (top) and the Northridge WRI modified seeded time history (bottom). Values of duration, $D_{5\text{-}75}$, defined as the time interval between the 5% and 75% normalized Ia levels, are listed in the legends.

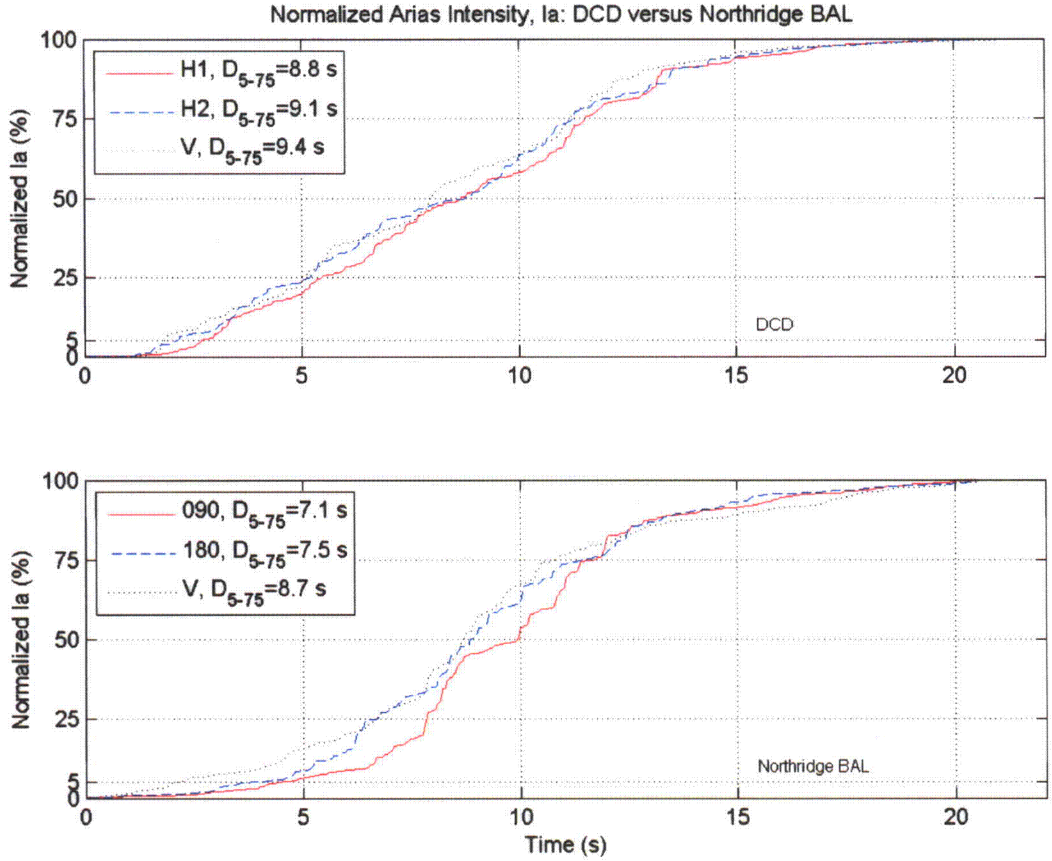


Figure 6. Normalized Arias Intensity of the three-component accelerograms of the DCD synthesized time history (top) and the Northridge BAL modified seeded time history (bottom). Values of duration, $D_{5\text{-}75}$, defined as the time interval between the 5% and 75% normalized Ia levels, are listed in the legends.

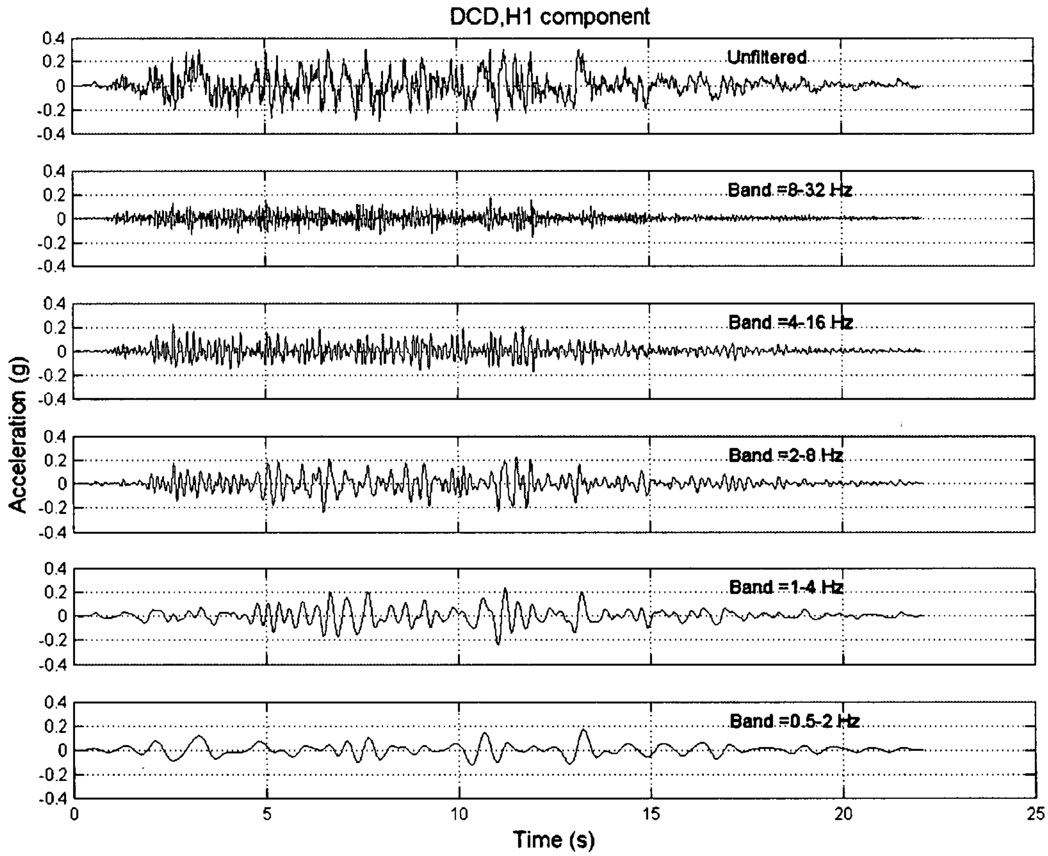


Figure 7. Unfiltered and band-pass filtered accelerograms of the DCD synthesized time history, component H1.

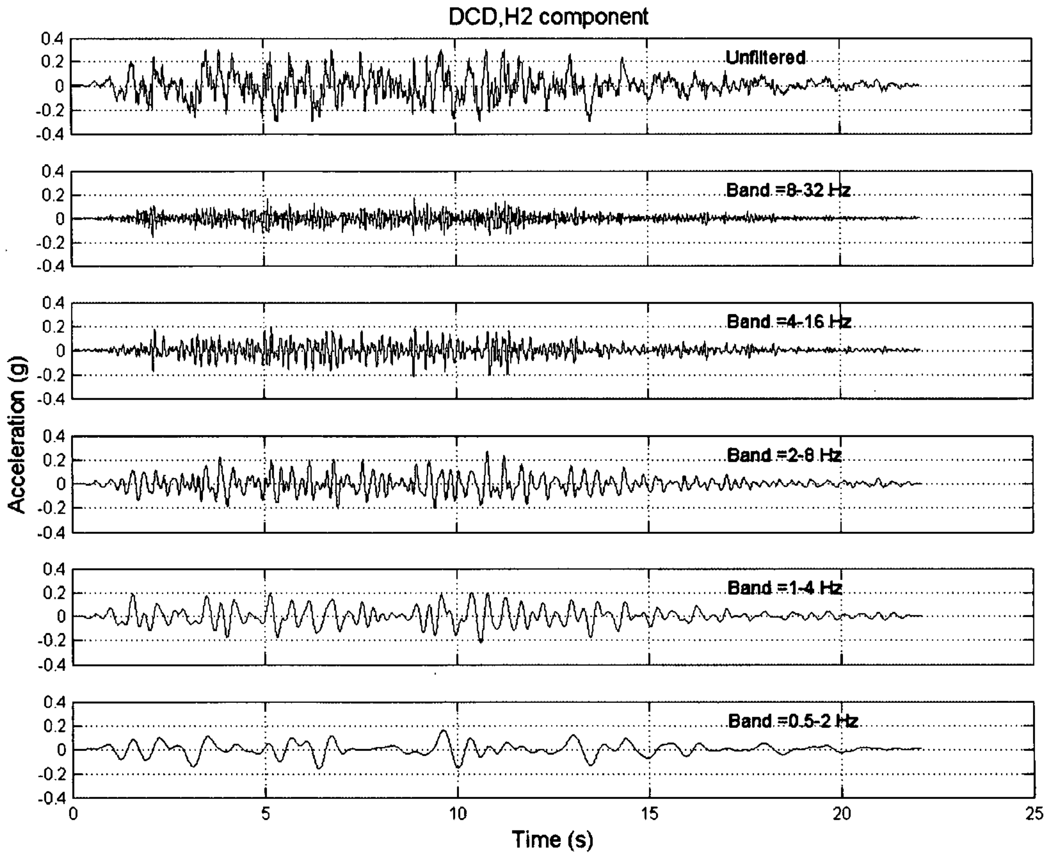


Figure 8. Unfiltered and band-pass filtered accelerograms of the DCD synthesized time history, component H2.

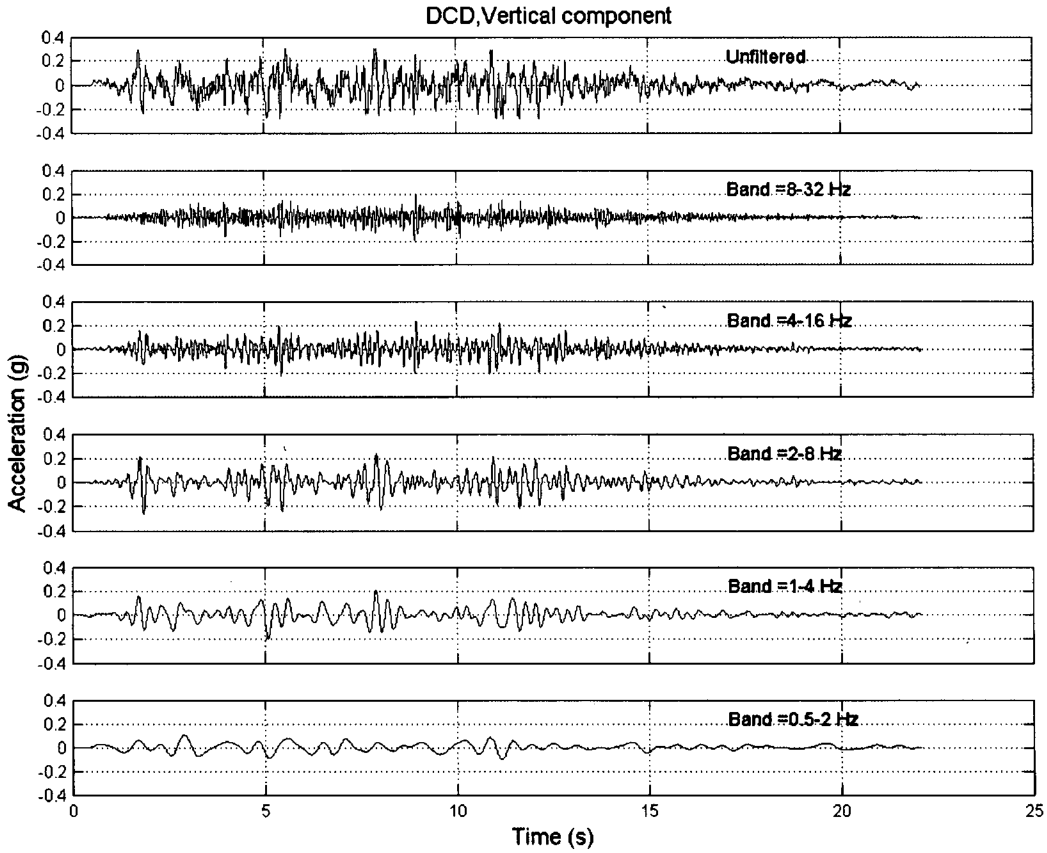


Figure 9. Unfiltered and band-pass filtered accelerograms of the DCD synthesized time history, component V.

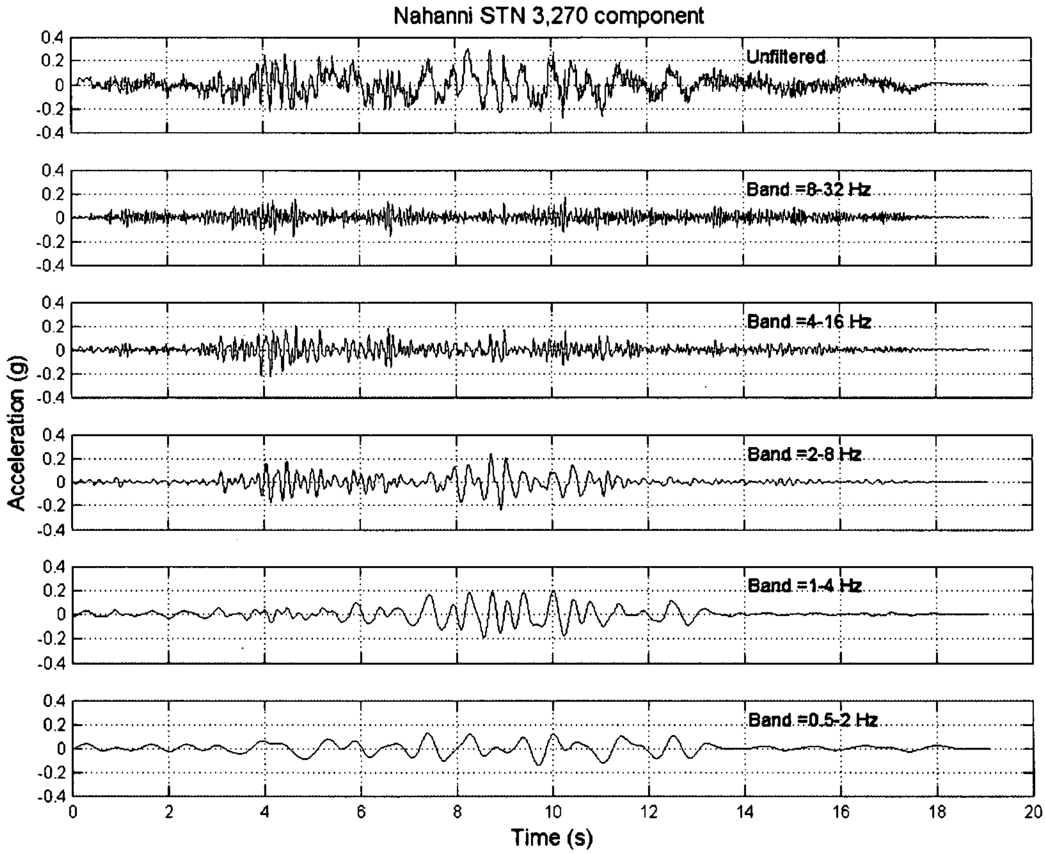


Figure 10. Unfiltered and band-pass filtered accelerograms of the Nahanni STN3 modified seeded time history, component 270.

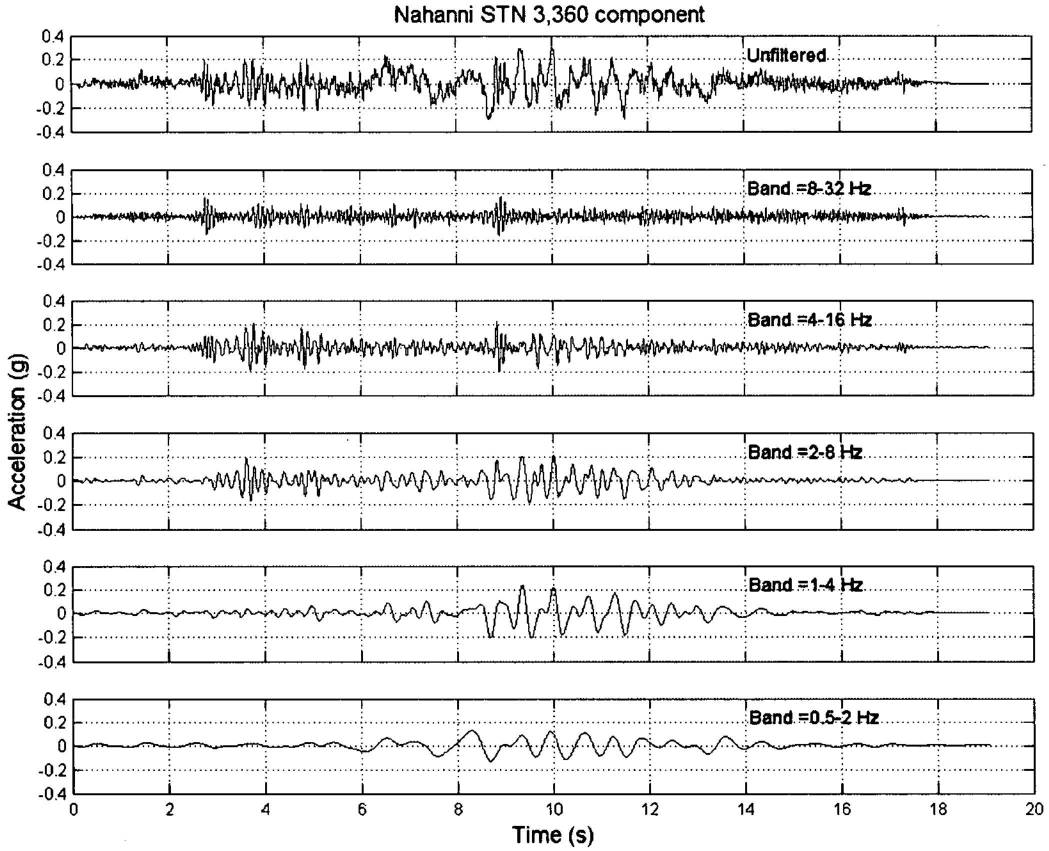


Figure 11. Unfiltered and band-pass filtered accelerograms of the Nahanni STN3 modified seeded time history, component 360.

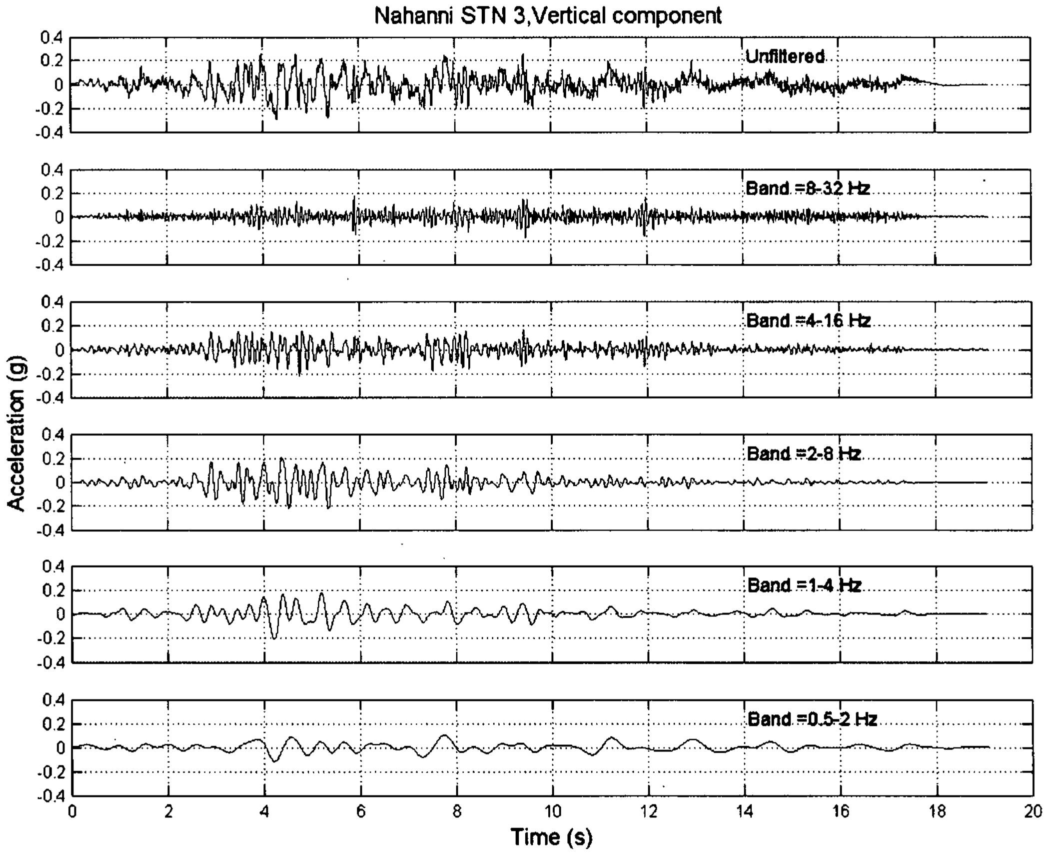


Figure 12. Unfiltered and band-pass filtered accelerograms of the Nahanni STN3 modified seeded time history, component V.

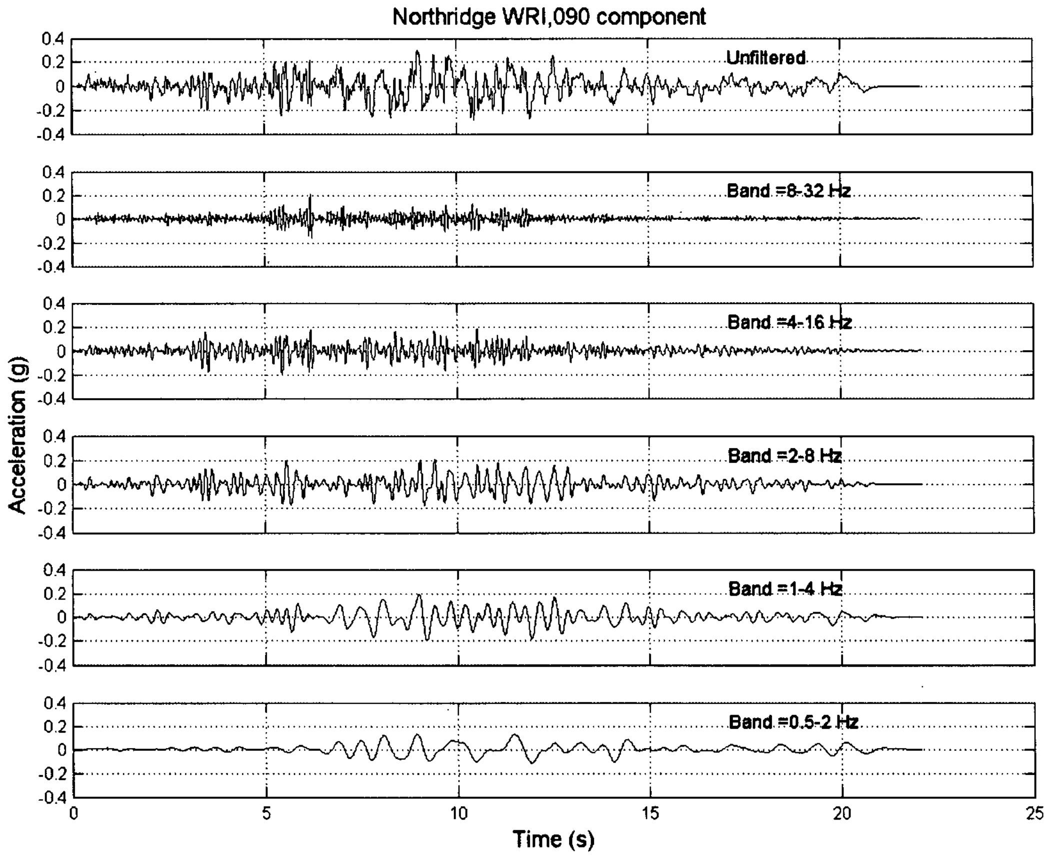


Figure 13. Unfiltered and band-pass filtered accelerograms of the Northridge WRI modified seeded time history, component 090.

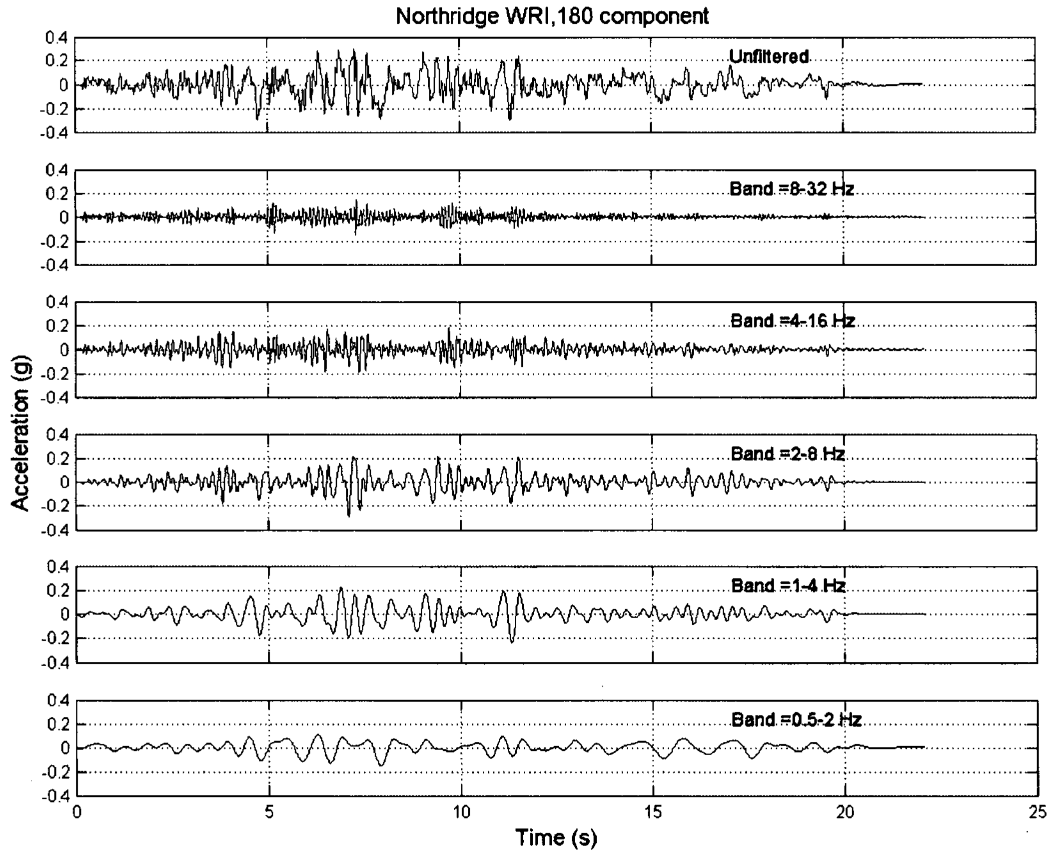


Figure 14. Unfiltered and band-pass filtered accelerograms of the Northridge WRI modified seeded time history, component 180.

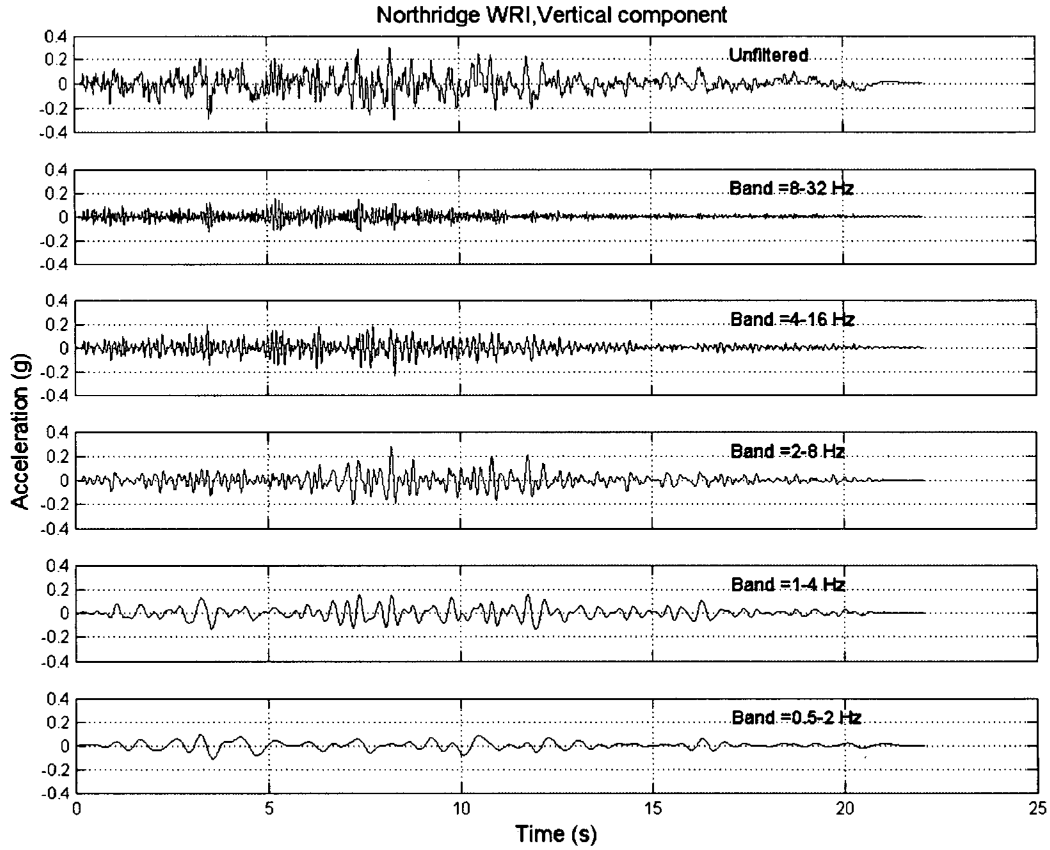


Figure 15. Unfiltered and band-pass filtered accelerograms of the Northridge WRI modified seeded time history, component V.

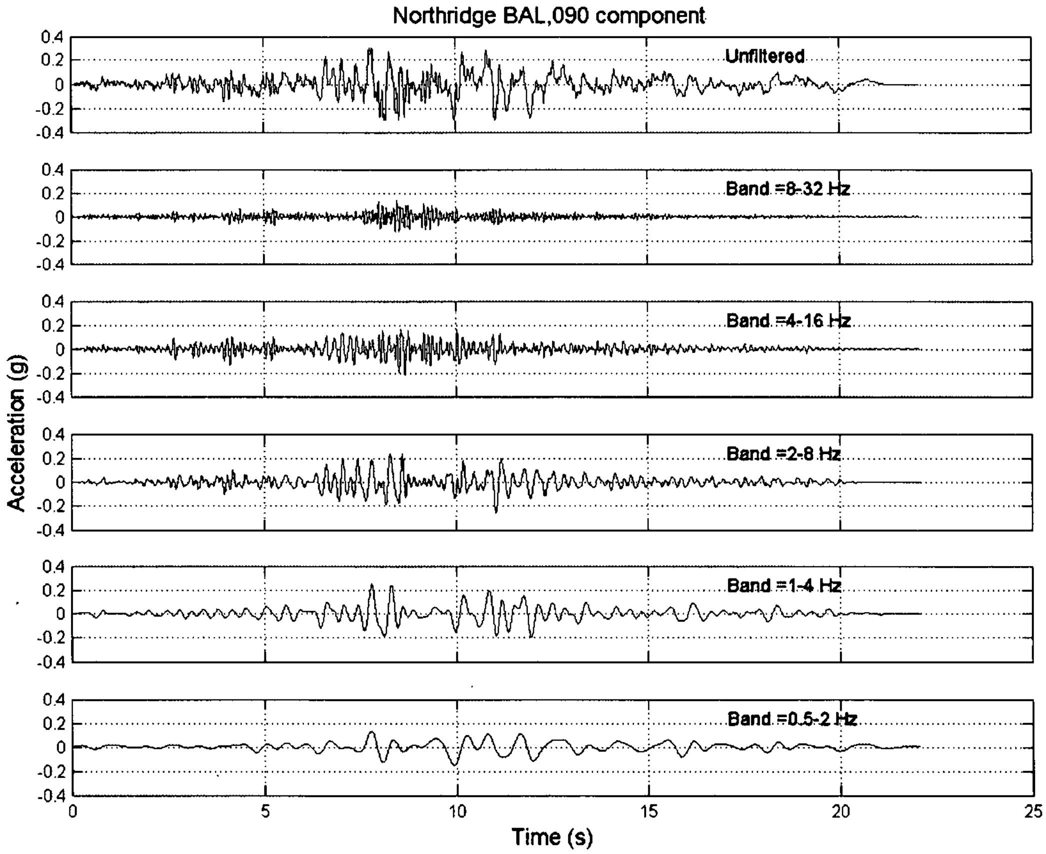


Figure 16. Unfiltered and band-pass filtered accelerograms of the Northridge BAL modified seeded time history, component 090.

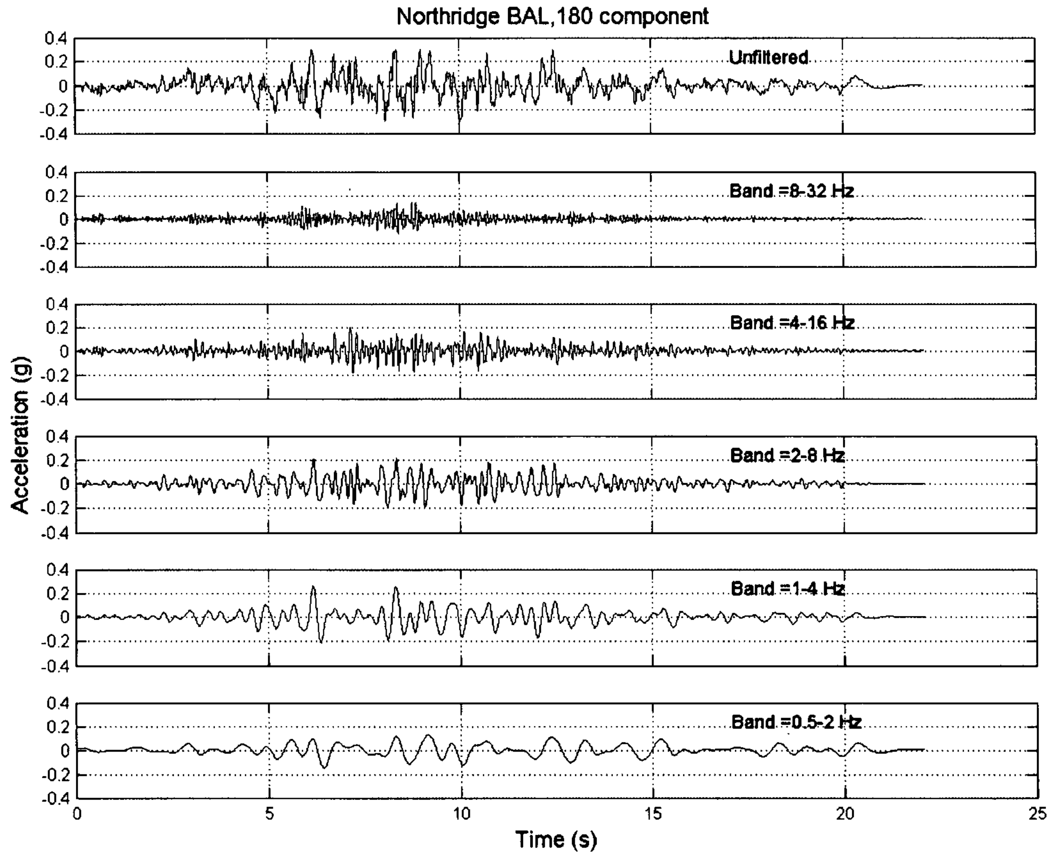


Figure 17. Unfiltered and band-pass filtered accelerograms of the Northridge BAL modified seeded time history, component 180.

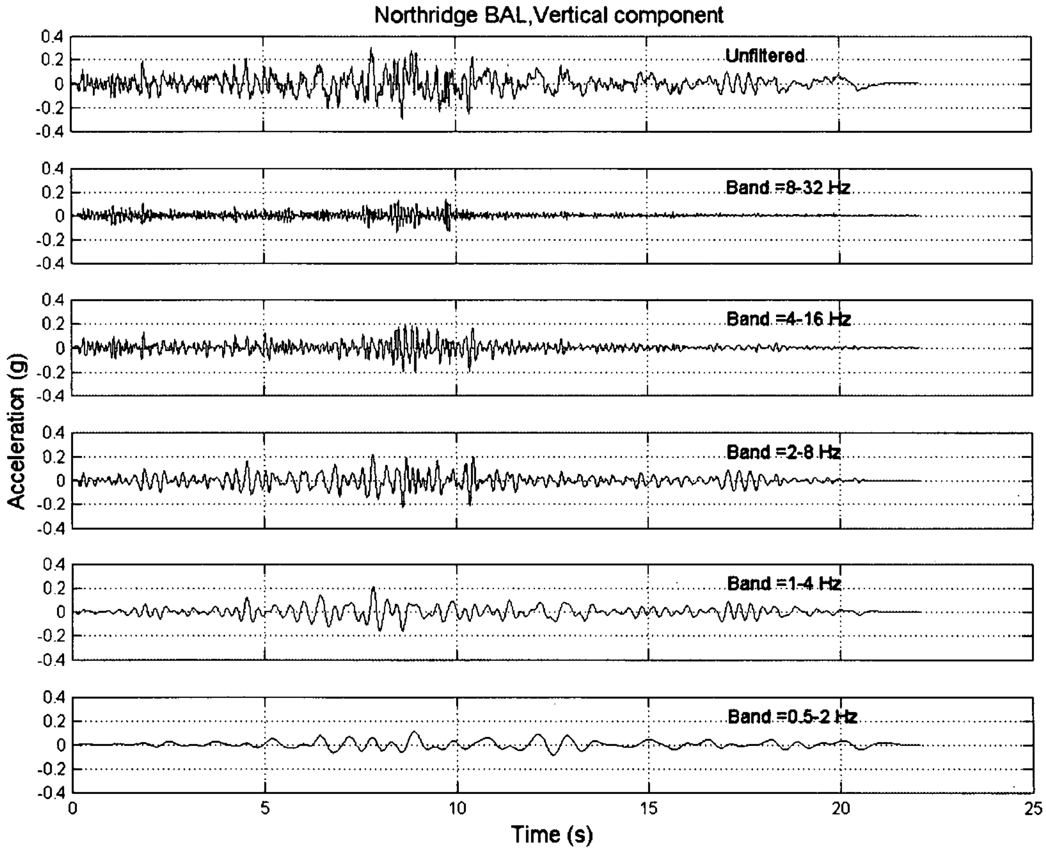


Figure 18. Unfiltered and band-pass filtered accelerograms of the Northridge BAL modified seeded time history, component V.

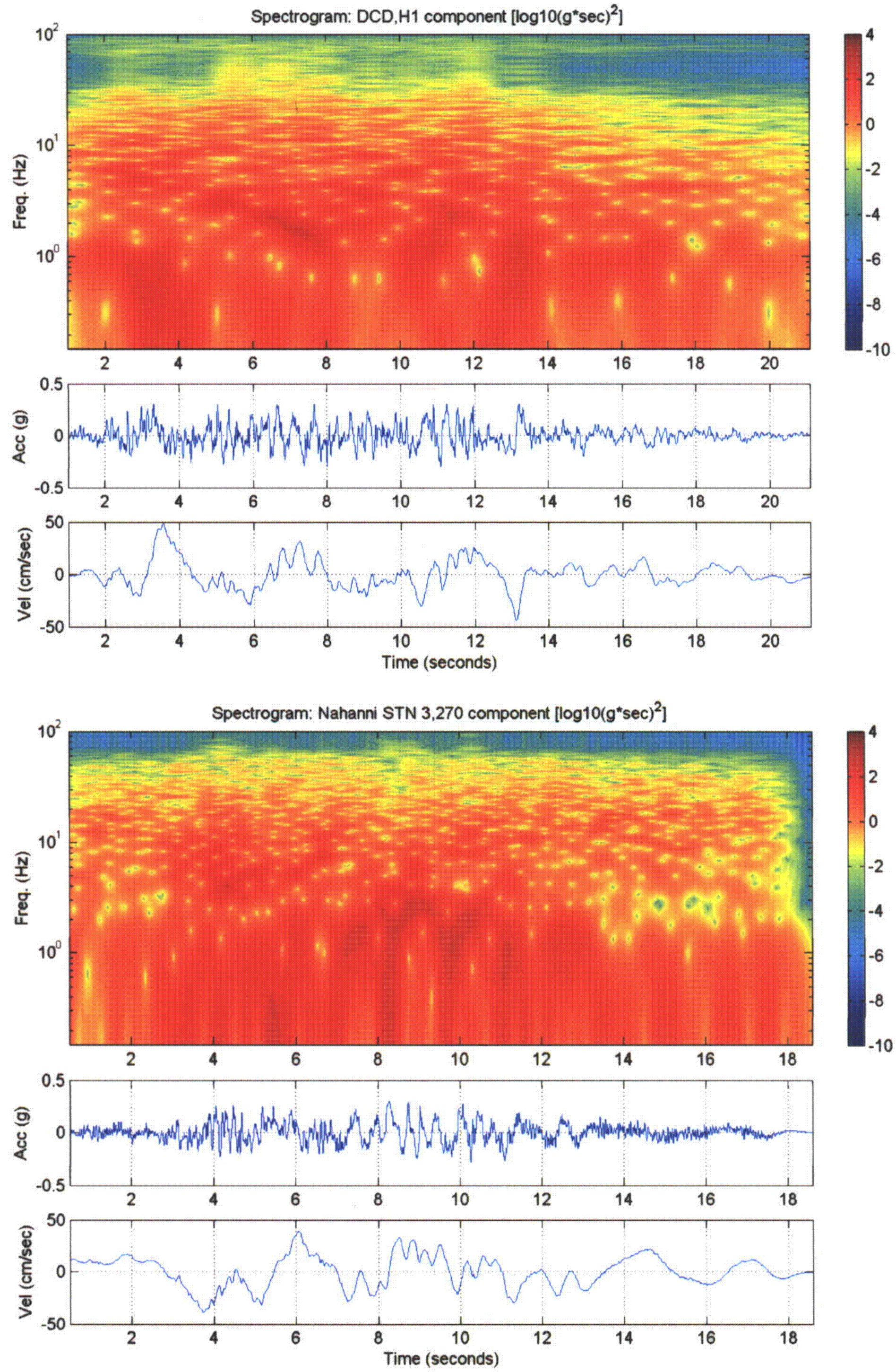


Figure 19. Spectrograms of DCD synthesized time history (top) and the Nahanni STN3 modified seeded time history (bottom) for components H1 and 270, respectively.

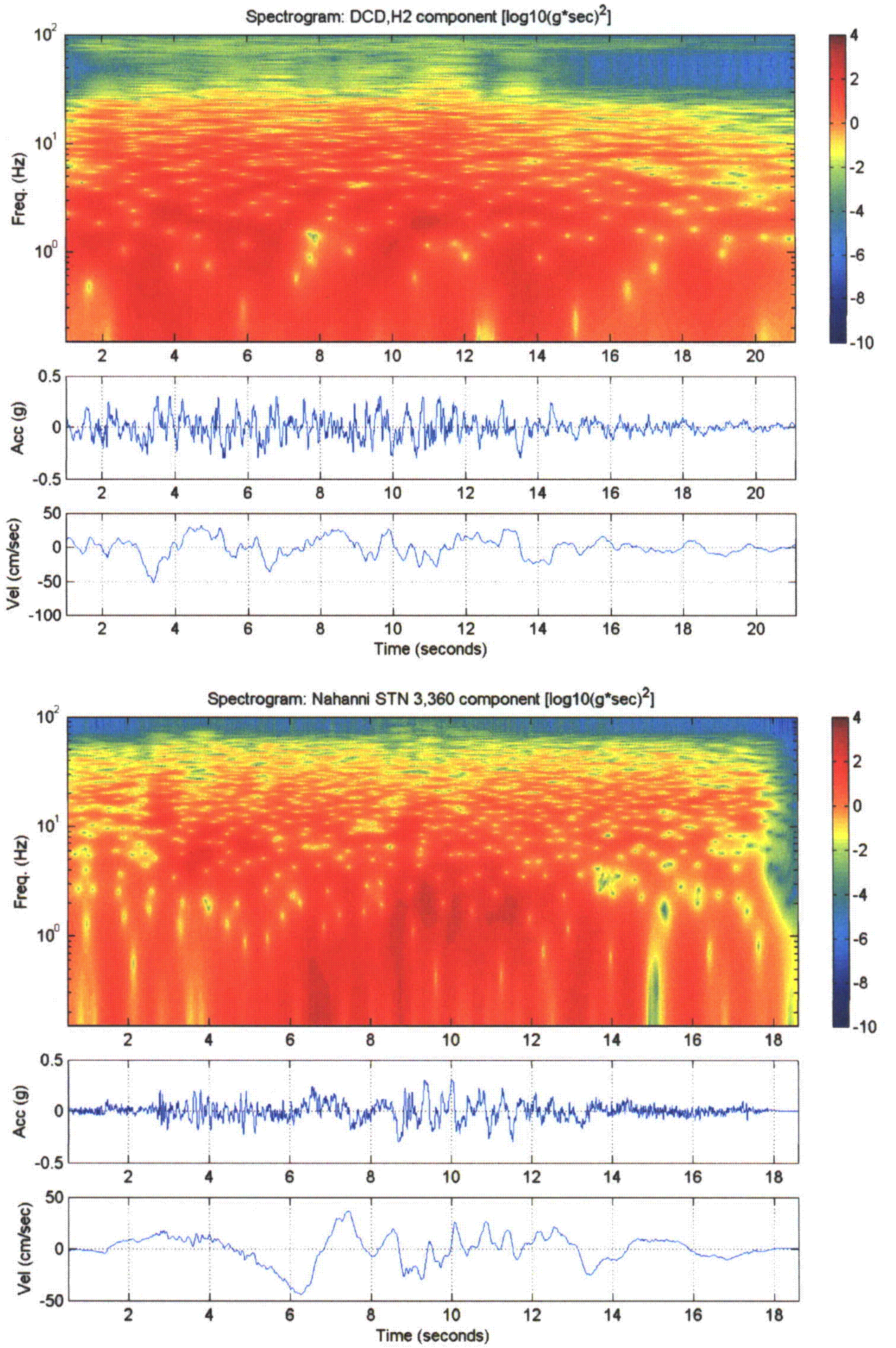


Figure 20. Spectrograms of DCD synthesized time history (top) and the Nahanni STN3 modified seeded time history (bottom) for components H2 and 360, respectively.

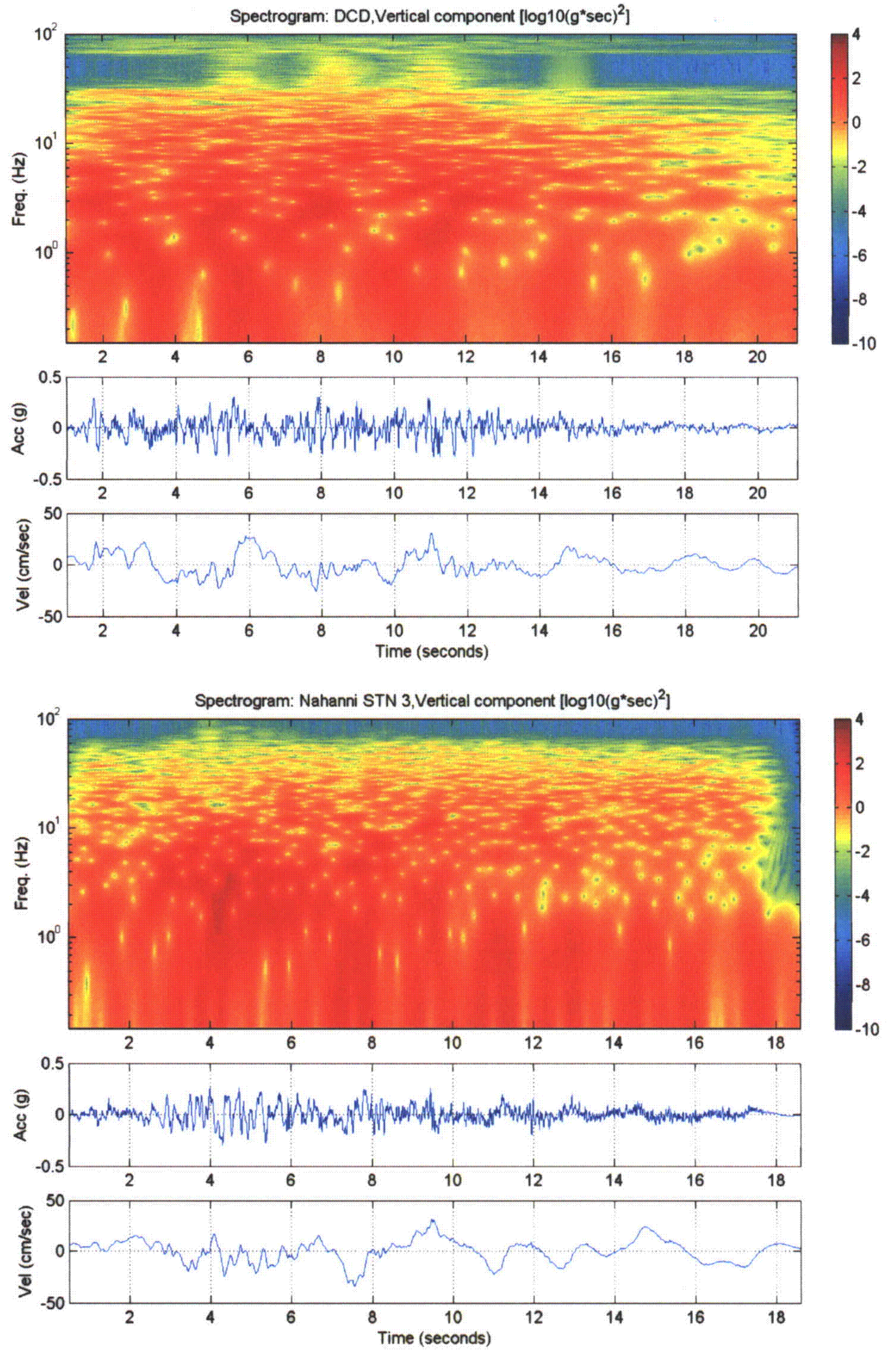


Figure 21. Spectrograms of DCD synthesized time history (top) and the Nahanni STN3 modified seeded time history (bottom) for component V.

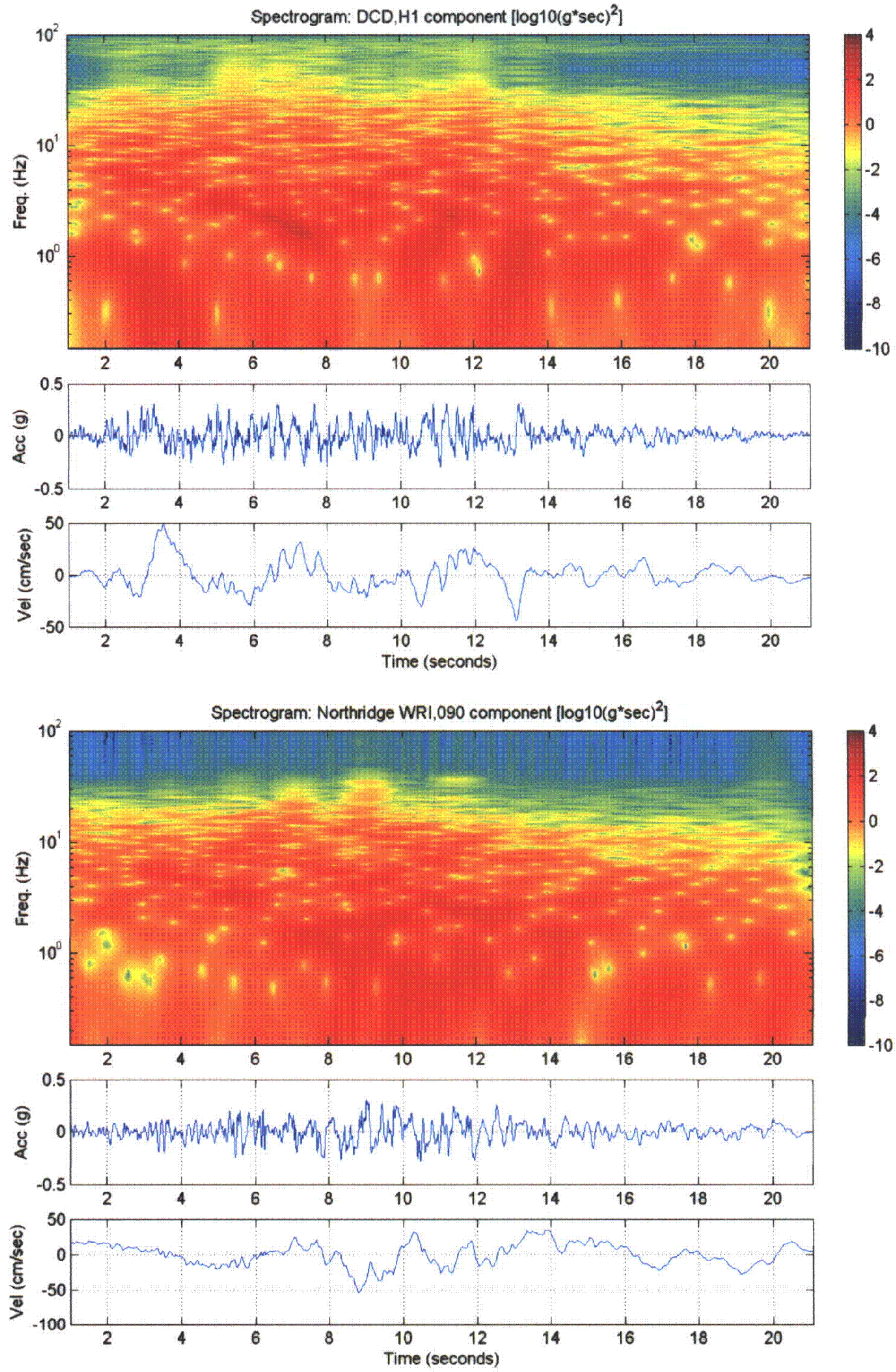


Figure 22. Spectrograms of DCD synthesized time history (top) and the Northridge WRI modified seeded time history (bottom) for components H1 and 90, respectively.

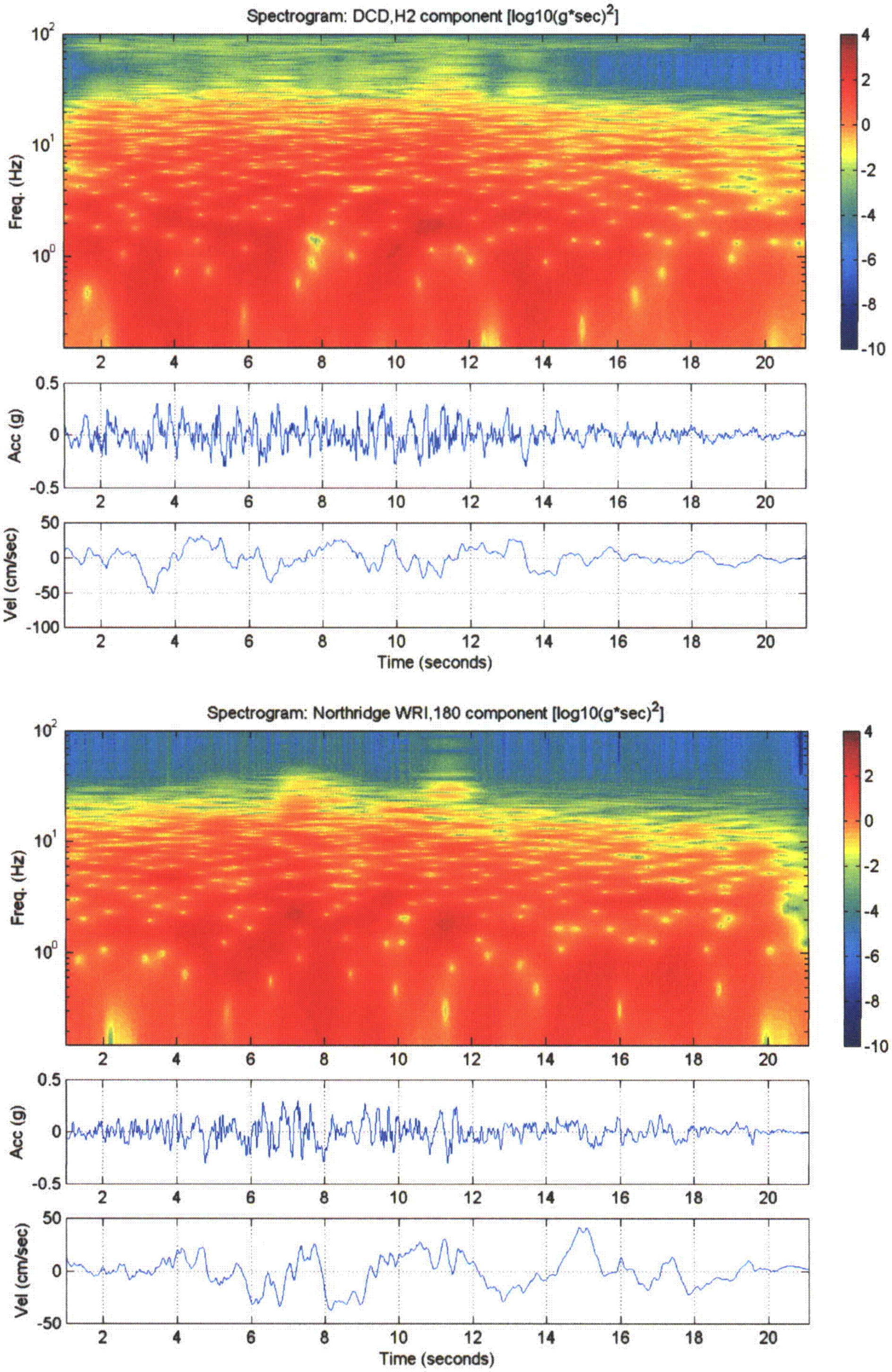


Figure 23. Spectrograms of DCD synthesized time history (top) and the Northridge WRI modified seeded time history (bottom) for components H2 and 180, respectively.

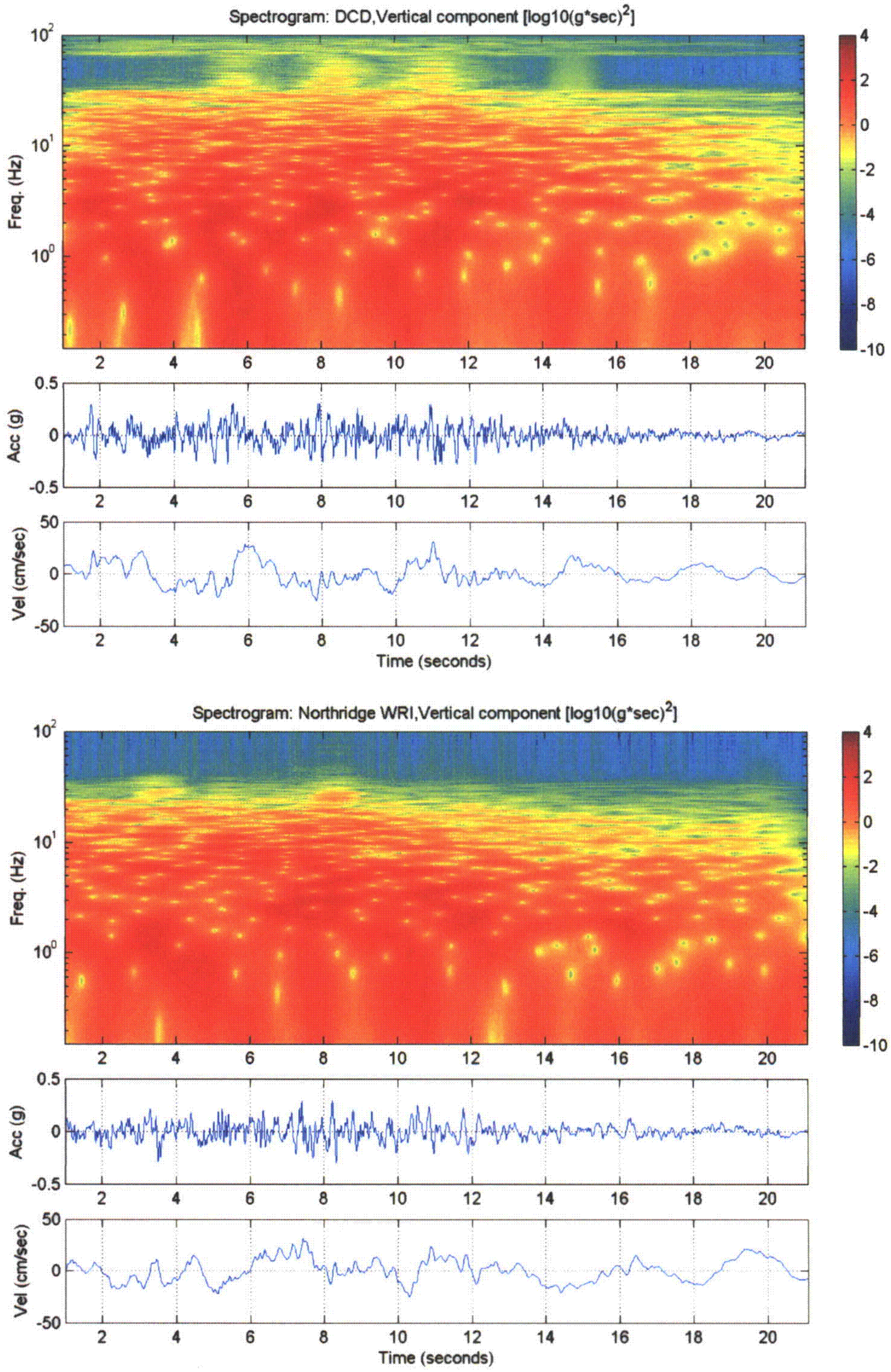
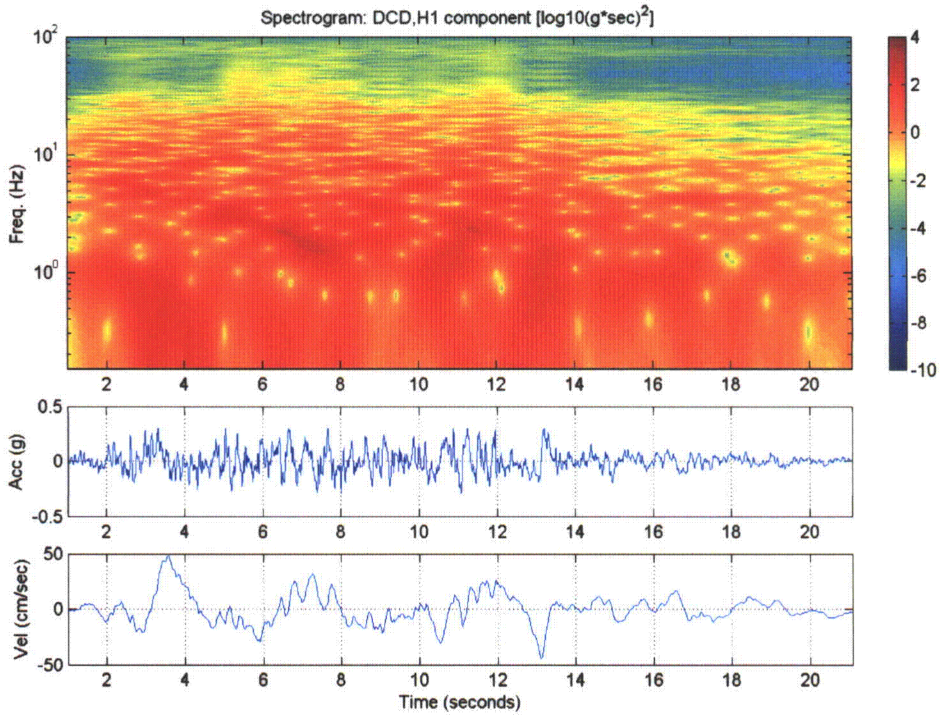


Figure 24. Spectrograms of DCD synthesized time history (top) and the Northridge WRI modified seeded time history (bottom) for component V.



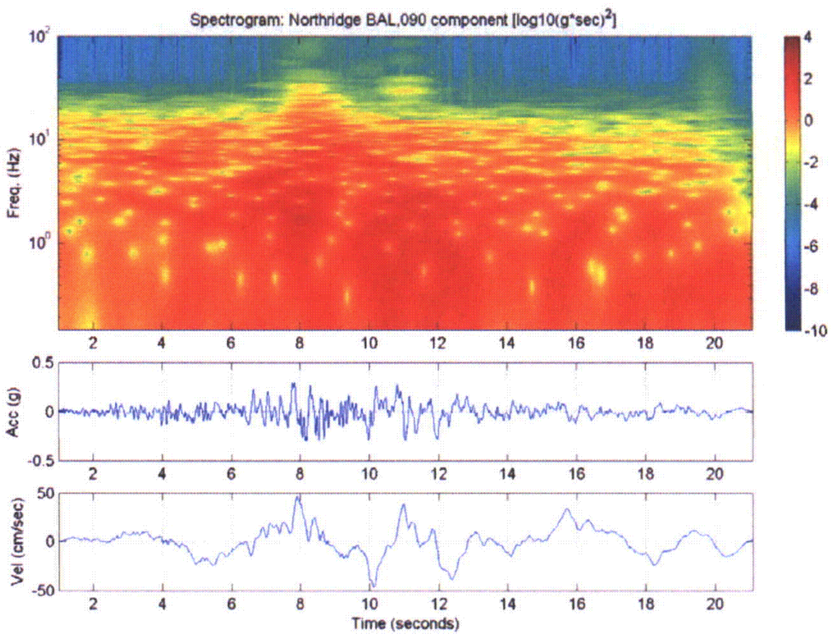


Figure 25. Spectrograms of DCD synthesized time history (top) and the Northridge BAL modified seeded time history (bottom) for components H1 and 90, respectively.

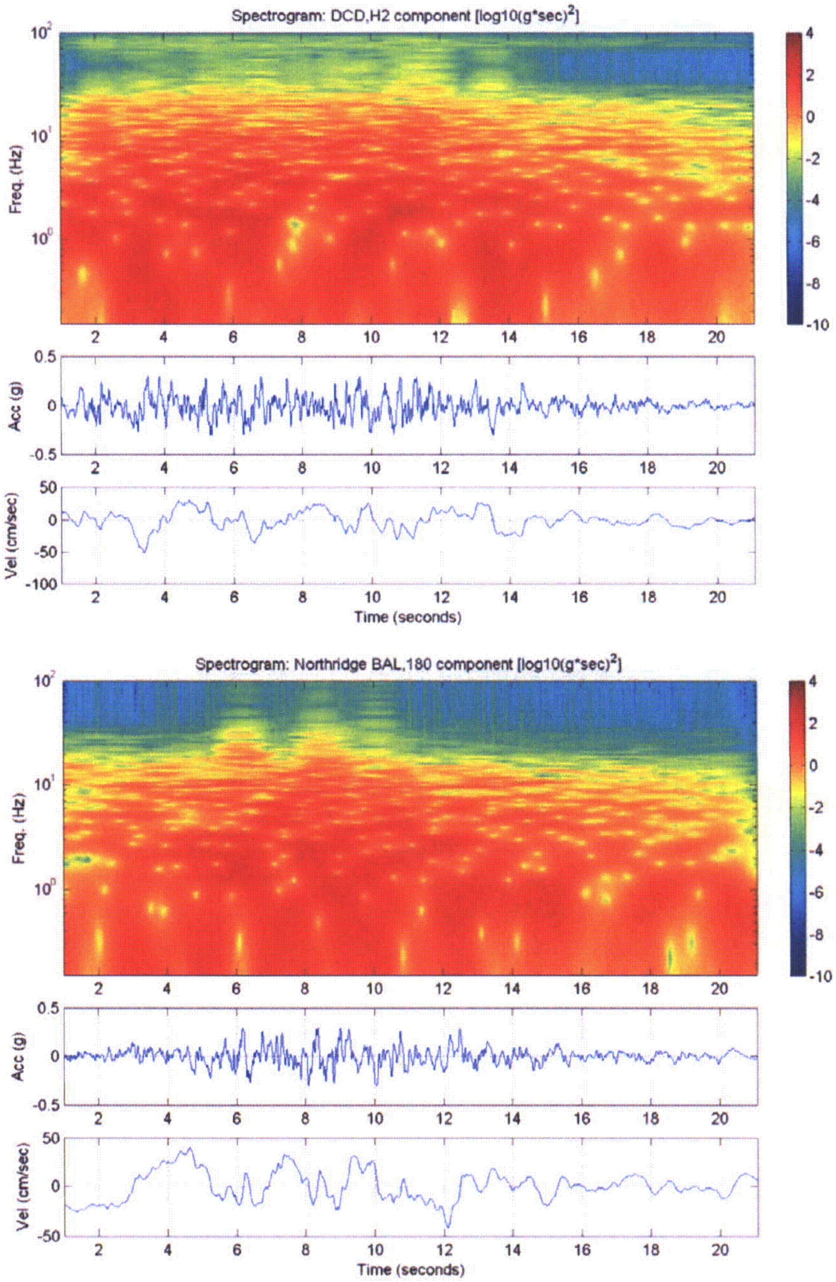


Figure 26. Spectrograms of DCD synthesized time history (top) and the Northridge BAL modified seeded time history (bottom) for components H2 and 180, respectively.

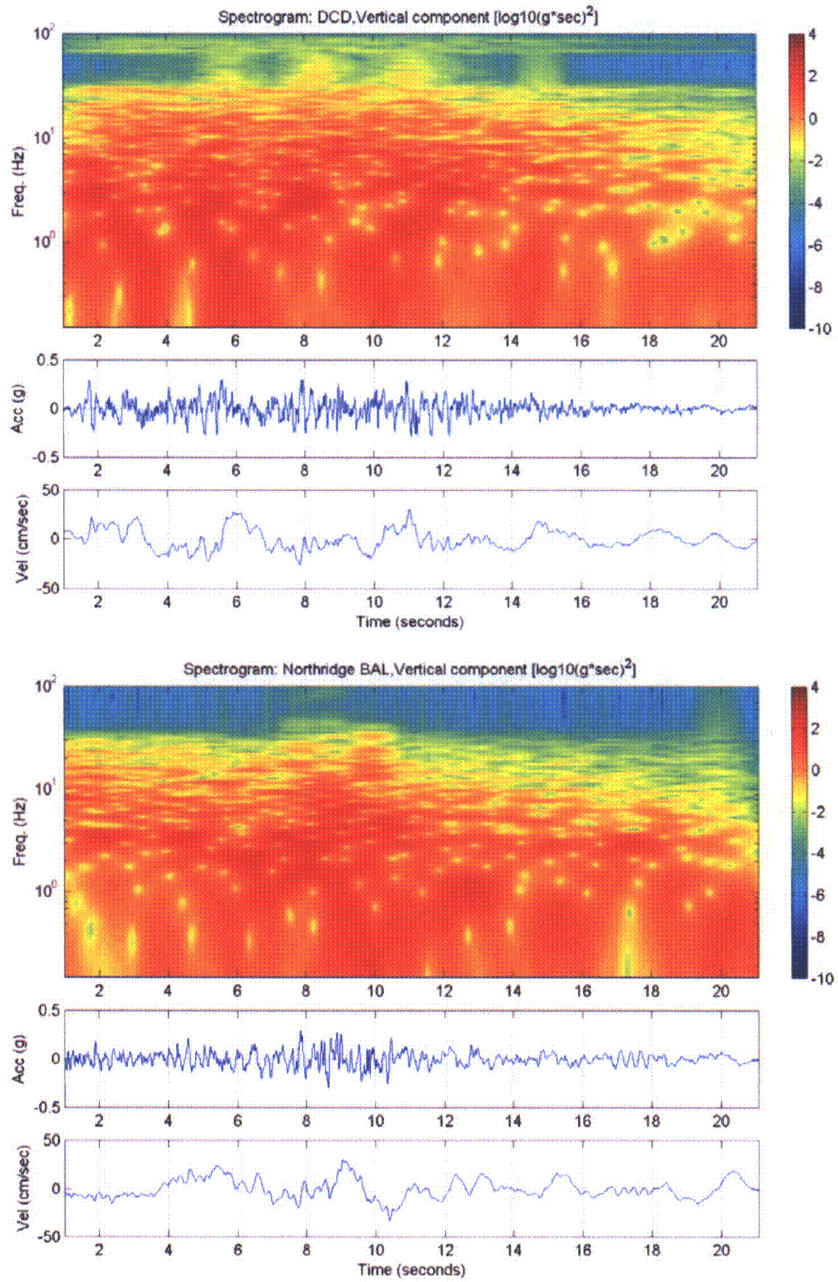


Figure 27. Spectrograms of DCD synthesized time history (top) and the Northridge BAL modified seeded time history (bottom) for component V.

Impact on DCD

There is no impact on the DCD.

Impact on COLA

There is no impact on the COLA.

Impact on PRA

There is no impact on the PRA.

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

4/23/2009

US-APWR Design Certification

Mitsubishi Heavy Industries

Docket No. 52-021

RAI NO.:	**NO. 211-1946 REVISION 1**
SRP SECTION:	**03.07.01 – Seismic Design Parameters**
APPLICATION SECTION:	**3.7.1**
DATE OF RAI ISSUE:	**02/25/09**

QUESTION NO. RAI 3.7.1-5:

Section 3.7.1.2 of the DCD states that the damping values presented in Tables 3.7.3-1(a) and 3.7.3-1(b) are in accordance with RG 1.61 and ASCE 4-98. The SRP acceptance criteria 3.7.1.II.2 states that the damping values used in the analyses of Category I SSCs are considered acceptable if they are in accordance with RG 1.61. The staff has not reviewed and endorsed ASCE 4-98 for the damping application. Currently this ASCE standard is under revision. Provide justification for all the damping values in Tables 3.7.3-1(a) and 3.7.3-1(b) that are either different from or are not specified in RG 1.61. In particular, justify the damping values shown for welded aluminum structures, and state the percentage of critical damping values to be used for conduit systems with other than maximum cable fill.

Also, the primary shield wall and other walls inside containment are fabricated as steel-concrete (SC) module walls described in Section 3.8.3.1.5 of the DCD. Discuss the methodology for calculating the stiffness and damping values used in modeling the module walls for seismic analysis of the containment structures. Provide justification including test data to demonstrate the appropriateness of the stiffness and damping values used in the seismic analysis and design.

ANSWER:

The US-APWR standard design does not use welded aluminum structures. Table 3.7.3-1(a) and Table 3.7.3-1(b) will be revised to delete damping values for these structures.

Table 4 of RG 1.61 specifies the use of 5% and 3% damping values for SSE and OBE (where required) analyses of empty conduits and related supports. Tables 3.7.3-1(a) and 3.7.3-1(b) will be revised to add these damping values. The damping values presented in Tables 3.7.3-1(a) and 3.7.3-1(b) are in accordance with RG 1.61 (see additional discussion below for Steel Concrete (SC) damping values). The reference to ASCE 4 for damping will be deleted from DCD Section 3.7.1.2 since it is redundant.

The following discussion provides further clarification for the basis for the stiffness and damping values used for the SC modules.

The validation of the lumped mass stick model of the Containment Internal Structure (CIS) is described in DCD Subsection 3.7.2.3.10.3 and repeated in items i) through iii) below.

i) Fixed-base FE model

DCD Figure 3.7.2-10 shows the fixed-base FE model for the CIS which is compared with the three-dimensional stick model. To verify the three-dimensional stick model, the FE model is used to estimate its rigidity by both static and dynamic analyses.

ii) Rigidity estimation by static analysis

Comparisons of static deformations are made between the three-dimensional stick model and the FE model.

iii) Comparison of ISRS

Comparisons of ISRS are made between the three-dimensional stick model and the FE model at various points for various elevations.

The section properties of the CIS stick model (Figure 3H.2-1 of DCD) are calculated from a static analysis of the CIS FE model (Figure 3.7.2-2 of DCD). The shear areas are directly calculated from beam formula for shear deformation to match the lateral displacement imposed on the FE analysis under a simultaneously constrained condition of the vertical direction for the grid points at the floor levels. The moment of inertias, combined with the above shear areas, are adjusted so that the stick model deformation corresponds with the total deformation of the FE analysis without constraint of the floor grid points.

The rigidities of the CIS stick model are shown in Table 3H.2-5 of the DCD, and the deformations of the stick model in the NS and the EW directions are shown in Figure 3H.3-3 compared with the deformations of the FE model under the same load condition. It is clear that the deformation of the stick model reasonably correspond with those of the FE analysis in each direction.

Comparisons of ISRS between the three-dimensional stick model and the FE model at various points at various elevations are shown in Figure 3H.3-4. It is clear that the ISRS of the stick model reasonably correspond with that of the FE analysis at each point.

The element properties of the FE model are defined and calculated as follows.

The primary shield wall, face plates, middle plates and web plates are modeled by plate elements which have only elastic in-plane rigidity of steel, and the concrete is modeled by brick elements which have elastic rigidity of concrete. SC members are modeled by shell elements which have composite elastic rigidities of steel face plates and concrete. The properties are calculated by the following equations which are described in DCD Subsection 3.8.3.4.1:

- Axial and Shear Stiffnesses of SC Modules:

  $\Sigma EA = E_cA_c + E_sA_s,\qquad \Sigma GA = G_cA_c + G_sA_s$

  $A_c = L(t - 2t_s),\ A_s = 2Lt_s,\ G_c = E_c/2(1+ \nu_c),\ G_s = E_s/2(1+ \nu_s)$

- Bending Stiffness of SC Modules:

  $\Sigma EI = E_c I_c + E_s I_s$

  $I_c = L(t - 2t_s)^3/12,\ I_s = Lt^3/12 - I_c$

  Where:

  E_c or E_s = modulus of elasticity for concrete or steel

  ν_c or ν_s = Poisson's ratio for concrete or steel

  L = length of SC module

  t = thickness of SC module

  t_s = thickness of plate on each face of SC module

The above stiffnesses based on elastic properties and composite behavior of steel and concrete are acceptable and have been verified by many test results. See DCD Reference 3.8-27, which shows the 1/10th scale model test results of a CIS in a PWR which used SC modular walls to provide validation of the above evaluation method. This reference provides test results that show the equivalent viscous damping of the SC modules at design load level (SSE) of 5 percent. This generally remained constant up to the load level at which the SC module steel plates started yielding. Therefore, it is reasonable to use 4 percent damping for the OBE based on these test results.

Figure 1 below shows a comparison of the deformation between test and calculation (Table 1 below) results based on the above evaluation method. The calculated stiffness corresponds to that of test results near the load level of the SSE.

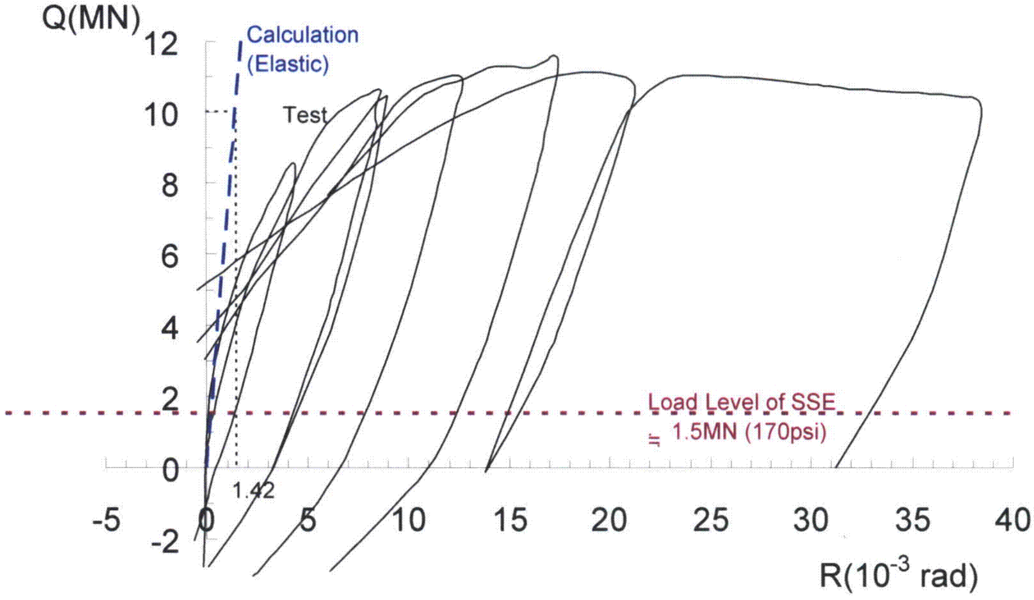
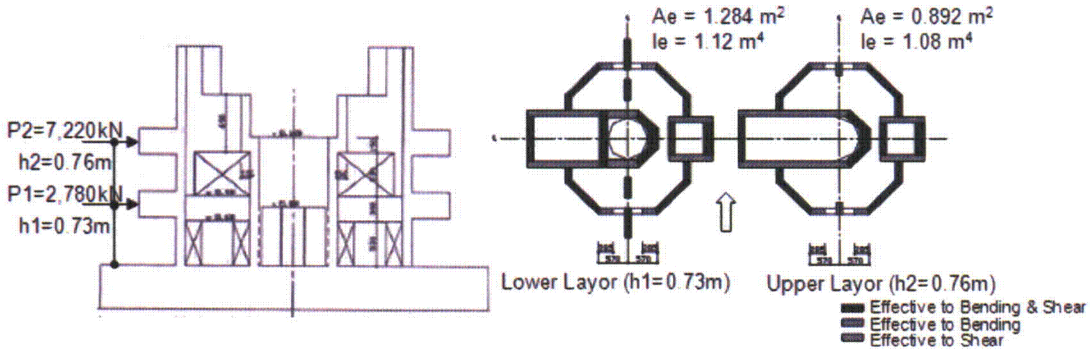


Figure 1 Comparison of Load- Distortion Angle Relationship between Calculation and Test

Table 1 Calculation of Horizontal Rigidity (Displacement at Q = 10MN) based on Elastic Material Property

Height	Load	Shear Force	Shear Stress	Moment	Shear Area	Moment of Inertia	Shear Disp.	Bending Disp.	Total Disp.	Rotation Angle
h	P	Q	τ	M	Ae	Ie	δs	δb	δ	R
m	kN	kN	N/mm^2	kNm	m^2	m^4	mm	mm	mm	10^{-3}
0.76	7,220	7,220	8.09	5,487	0.892	1.08	1.644	0.478	2.122	1.424
0.73	2,780	10,000	7.79	12,787	1.284	1.12	0.790	0.148	0.937	

[NOTE] Young's modulus of concrete = 16.8 kN/mm^2, Poisson's ratio = 0.167, R = δ/(h1+h2)
Ae & Ie are the concrete equivalent values based on FE analyses.



Reference

1. Akiyama, H., Sekimoto, H., Tanaka, M., Inoue, K., Fukihara, M., and Okuda, Y. 1/10th Scale Model Test of Inner Concrete Structure Composed of Concrete Filled Steel Bearing Wall. 10th International Conference on Structural Mechanics in Reactor Technology, 1989 (DCD Reference 3.8-27).

Impact on DCD

See Attachment 1 for the mark-up of DCD Tier 2, Section 3.7, Revision 2, changes to be incorporated:

- Change the third sentence of the first paragraph of Subsection 3.7.1.2 to the following: "The specified damping coefficients are in accordance with RG 1.61 (Reference 3.7-15), and are based on consideration of the material, load conditions, and type of construction used in the structural system."

- Delete the first line of Table 3.7.3-1(a): Welded aluminum structures (%) 4

- Change the 10th line of Table 3.7.3-1(a) to the following:

  "Full conduits & related supports (%)...7"

- Insert the following as the 11th line of Table 3.7.3-1(a):

  "Empty conduits & related supports (%)...5"

- Delete the first line of Table 3.7.3-1(b): Welded aluminum structures (%) 3
- Change the 10th line of Table 3.7.3-1(b) to the following:

  "Full conduits & related supports (%)... 5"
- Insert the following as the 11th line of Table 3.7.3-1(b):

  "Empty conduits & related supports (%).. 3"

Impact on COLA

There is no impact on the COLA.

Impact on PRA

There is no impact on the PRA.

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

4/23/2009

US-APWR Design Certification
Mitsubishi Heavy Industries
Docket No. 52-021

RAI NO.:	**NO. 211-1946 REVISION 1**
SRP SECTION:	**03.07.01 – Seismic Design Parameters**
APPLICATION SECTION:	**3.7.1**
DATE OF RAI ISSUE:	**02/25/09**

QUESTION NO. RAI 3.7.1-6:

In Section 3.7.1.1 (a) of the DCD, it is stated that the Nyquist frequency of 100 Hz is considered to assure that seismic analysis will capture the responses of SSCs in the high frequency range. Define what is considered the high frequency range and discuss the criteria used to assure that an adequate number of discrete mass degrees of freedom is considered in dynamic modeling of SSCs to capture the seismic response in the high frequency range.

ANSWER:

The high frequency range discussed in Section 3.7.1.1(a) of the DCD refers to frequency components of earthquake excitation which may exceed the certified seismic design response spectra (CSDRS) in the frequency range extending up to 50 Hz. This range is consistent with guidance given in NRC DG-1175 and NRC Interim Staff Guidance on "Seismic Issues Associated with High Frequency Ground Motion in Design Certification and Combined License Applications". The artificial time histories are developed with a time step of 0.005 second, resulting in a Nyquist frequency of 100 Hz, to assure sufficient resolution to capture the responses of the structures in this high frequency range. The discrete mass structural models used for dynamic analysis of seismic Category I structures, systems, and components (SSCs) are developed following the dynamic modeling acceptance criteria set in SRP 3.7.2 II A (iv) to ensure their ability to capture the seismic response in the high frequency range at each direction of the input seismic excitation. The models have sufficient number of degrees of freedom to adequately represent the contribution of all significant modes of vibration with frequencies less than 50 Hz.

The adequacy of the dynamic modeling in the high frequency range is verified based on comparison of the results of the analysis performed on discrete mass structural models and distributed mass finite element (FE) models on a fixed base. The modal analyses results for natural frequencies and mass participation factors are compared to ensure a sufficient correlation between the two models in representing the seismic response at frequencies below 50 Hz. The mode shapes with significant mass participation obtained from the fixed base modal analyses of lumped mass stick models are plotted in the direction of response and inspected to ensure that they are reasonable. Good correlation of deformation results obtained from the analyses of the lumped mass stick models compared with the FE model indicates adequate modeling of the discrete mass degrees of freedom.

Subsection 3.7.2.3.10 of the DCD and the responses to questions RAI 3.7.2-3, 17, 18, and 19 of RAI 212-1950 provide further explanations on dynamic model verification and capturing of the dynamic response of the structure.

Impact on DCD

There is no impact on the DCD.

Impact on COLA

There is no impact on the COLA.

Impact on PRA

There is no impact on the PRA.

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

4/23/2009

US-APWR Design Certification

Mitsubishi Heavy Industries

Docket No. 52-021

RAI NO.:	**NO. 211-1946 REVISION 1**
SRP SECTION:	**03.07.01 – Seismic Design Parameters**
APPLICATION SECTION:	**3.7.1**
DATE OF RAI ISSUE:	**02/25/09**

QUESTION NO. RAI 3.7.1-7:

In Section 3.7.1.3 of the DCD, a value of 15 ksf is specified for the required allowable static bearing capacity for seismic Category I structures basemats. Provide the bases and justification for the 15 ksf value. Also, a minimum factor of safety of 2 is proposed for the ultimate bearing capacity versus the allowable dynamic bearing capacity. Provide the bases and justification for the proposed minimum factor of safety of 2.

ANSWER:

Section 3.7.1.3 of the DCD specifies a required allowable bearing capacity of 15 ksf for seismic Category I structures, based on the maximum static bearing pressure of 11.3 ksf calculated for the foundation mat supporting the reactor building (R/B), prestressed concrete containment vessel (PCCV), and containment internal structure on a common mat. The value of 15 ksf provides additional margin above the factor of safety that would be achieved if a required allowable bearing capacity of 11.3 ksf were specified. The value of 11.3 ksf for the R/B-PCCV-containment internal structure on their common foundation mat is the highest value of the bearing pressures calculated under the foundations of other US-APWR standard plant seismic Category I structures, as shown in the table below.

The site-specific safety factor for dynamic bearing capacity is established by dividing the ultimate bearing capacity of the site by the maximum allowable dynamic bearing capacity. Ultimate bearing capacity is defined as the bearing stress that results in rupture or shear failure of the underlying soil mass and/or excessive, unacceptable settlements of the foundation. Subsection 3.7.1.3 of the DCD provides the minimum value of 2 for the required safety factor for dynamic bearing capacity only as a suggested reference value. The actual value for the required safety factor for dynamic (and static) bearing capacity is assigned on site-specific basis considering uncertainties that include but are not limited to the knowledge of subsurface conditions, measurements of the soil strength parameters, complexity of the soil behavior, environmental changes, variability of water table, and assumptions incorporated in mathematical models used to calculate foundations bearing pressures and the bearing capacity of the site.

Building	Seismic Category	Weight (kips)	Footprint Dimensions (ft)		Static Bearing Pressure (ksf)
			EW	NS	
R/B	I	734600	213'-4"	308'-11"	11.3
East PS/B	I	34300	114'-10"	69'-4"	4.3
West PS/B	I	34300	114'-10"	69'-4"	4.3

Impact on DCD

There is no impact on the DCD.

Impact on COLA

There is no impact on the COLA.

Impact on PRA

There is no impact on the PRA.

This completes MHI's responses to the NRC's questions.

The strong motion duration is defined as the time required for the Arias Intensity to rise from 5% to 75% in accordance with SRP 3.7.1 (Reference 3.7-10). The uniformity of the growth of this Arias Intensity has been examined and is acceptable. The duration of motion of the US-APWR artificial time histories with respect to the time duration needed to achieve 5% and 75% Arias intensities is summarized in Table 3.7.1-5.

The COL Applicant is to verify that the site-specific ratios *V/A* and AD/V^2 (*A, V, D*, are PGA, ground velocity, and ground displacement, respectively) are consistent with characteristic values for the magnitude and distance of the appropriate controlling events defining the site-specific uniform hazard response spectra. These parameters are examined to assure that they are consistent with the values determined for the low and high frequency events described in Appendix D of RG 1.208 (Reference 3.7-3).

The COL Applicant is to provide site-specific design ground motion time histories and durations of motion.

3.7.1.2 Percentage of Critical Damping Values

Damping coefficient values representing percentages of critical damping are assigned to the linear-elastic models to quantify the dissipation energy in the dynamic system. Table 3.7.3-1(a) presents the values of damping coefficients used for the SSE seismic analysis of seismic category I and II systems and subsystems. The specified damping coefficients are in accordance with RG 1.61 (Reference 3.7-15), ~~ASCE 4-98 (Reference 3.7-9),~~ and are based on consideration of the material, load conditions, and type of construction used in the structural system.

The values of the SSE damping coefficients specified in Table 3.7.3-1(a) are based on the expectation that the response of the linear elastic structure attributed to load combinations that include the SSE is close to applicable stress limits. This is considered acceptable for the US-APWR standard plant seismic design where, as described in RG 1.61 (Reference 3.7-15) Section 1.2, the design-basis ISRS represent the envelope of the in-structure responses obtained from multiple analyses conducted to consider a range of expected site soil conditions. However, this does not apply for the site-specific seismic analysis that use site-specific site properties since it is possible that the predicted structural response to the load combinations that include an SSE is significantly below the stress limits. In these cases, the SSE values in Table 3.7.3-1(a) may overestimate the actual dissipation of energy in the linear dynamic system and, thus, result in a non-conservative estimate of the structural response for frequencies close to the resonant frequencies. To prevent non-conservative results, the COL Applicant is to review the resulting level of seismic response and determine appropriate damping values for the site-specific calculations of ISRS that serve as input for the seismic analysis of seismic category I and seismic category II subsystems. In accordance with Section 1.2 of RG 1.61 (Reference 3.7-15), no verifications of seismic response are required if the lower damping values listed in Table 3.7.3-1(b) are used as input for computation of ISRS. In accordance with RG 1.61 (Reference 3.7-15), the damping values in Table 3.7.3-1(b) are also intended for use in site-specific OBE analyses, if the site-specific OBE is higher than 1/3 of the site-specific SSE.

The damping values in Table 3.7.3-1(a) and Table 3.7.3-1(b) are applicable to all modes of vibration of a structure constructed of the same material.