Private Fuel Storage, LLC

P.O. Box C4010, La Crosse, WI 54602-4010
John D. Parkyn, Chairman of the Board

May 31, 2001

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, D.C. 20555-0001

**DATA NEEDED FOR NRC REVIEW OF LICENSE AMENDMENT # 22**
**DOCKET NO. 72-22/TAC NO. L22462**
**PRIVATE FUEL STORAGE FACILITY**
**PRIVATE FUEL STORAGE L.L.C.**

References:

1. PFS letter, Donnell to NRC, "Summary of Changes for PFSF License Application Amendment #22", dated April 16, 2001
2. April 18, 2001 meeting between PFS and the NRC in San Antonio, Texas
3. NRC letter, Brach to Parkyn, "March 30, 2001 License Application Amendment", dated May 7, 2001

In Reference 3, the NRC requested additional data needed for the completion of the review of Private Fuel Storage (PFS) License Application Amendment No. 22. The PFS response to the request for additional data is included in Enclosure 1 to this letter.

PFS believes that the information supplied in Reference 1 in conjunction with this submittal fully documents and justifies the changes made in License Amendment No. 22. However, as requested in Reference 3, PFS is providing in Enclosure 2 to this letter a stand-alone document which discusses the changes made and summarizes the effect of these changes on the adequacy of the design with supporting basis.

NMSS01Public

Rec'd at NRC PDD from NMSS with CD-ROM for entry into ADAMS

If you have any questions regarding this submittal, please contact me at 608-787-1236 or Mr. J. L. Donnell, Project Director, at 303-741-7009.

Sincerely,

John D. Parkyn, Chairman
Private Fuel Storage L.L.C.

JDP:JRJ
Enclosure

Enclosure

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# Enclosure 1

Response to NRC letter dated May 7, 2001

Each NRC request for information is repeated below followed by the PFS response.

**Seismic Hazard Analysis**

1. **Deaggregated hazard curves (mean and fractiles) for horizontal and vertical ground motion for each attenuation and site response model at all 16 frequencies.**

PFS Response

The probabilistic seismic hazard analysis was conducted for peak ground acceleration (PGA) and for 9 spectral periods. The spectral accelerations at the remaining periods in the design response spectra were obtained through the application of the procedures in REGULATORY GUIDE 1.165. Directory \SeisHaz.001 on the attached CD contains seismic hazard results for PGA and the 9 spectral periods for which probabilistic seismic hazard calculations were performed. For each of the 10 ground motion parameters 15 calculations were performed. The following naming convention applies to the calculations for the PGA.

| File Name | Calculation |
|---|---|
| TA | Horizontal hazard over all uncertainties |
| TA-AS | Horizontal hazard for Abrahamson and Silva attenuation |
| TA-BJF | Horizontal hazard for Boore et al. (1997) attenuation |
| TA-C | Horizontal hazard for Campbell (1997) attenuation |
| TA-I | Horizontal hazard for Idriss (1997) attenuation |
| TA-S | Horizontal hazard for Sadigh et al. (1997) attenuation |
| TA-SAO | Horizontal hazard for Spudich et al. (1997) attenuation |
| TA-SE | Horizontal hazard for empirical site factor only |
| TA-SN | Horizontal hazard for site response site factor only |
| TAV | Vertical hazard over all uncertainties |
| TAV-AS | Vertical hazard for Abrahamson and Silva attenuation |
| TAV-C | Vertical hazard for Campbell (1997) attenuation |
| TAV-S | Vertical hazard for Sadigh et al. (1997) attenuation |
| TAV-SE | Vertical hazard for empirical site factor only |
| TAV-SN | Vertical hazard for site response site factor only |

In the above analyses the full seismic hazard model logic tree is used. Where specific inputs are called out, that input parameter is given a weight of 1.0 and all alternatives a weight of 0. For example, the analysis TA-AS is computed by giving the attenuation relationship of Abrahamson and Silva (1997) a weight of 1.0 and the other five relationships each a weight of 0. Note that the uncertainty in the source scaling factors is still incorporated in computing the distribution in hazard given the use of only the Abrahamson and Silva (1997) relationship.

The hazard results for the individual spectral periods are identified by the file names T007 for 0.075 sec, T010 for 0.1 sec., T020 for 0.2 sec., T030 for 0.3 sec., T040 for 0.4 sec., T050 for 0.5 sec., T100 for 1.0 sec., T200 for 2.0 sec., and T400 for 4.0 sec.

For each calculation three output files are provided. The contents of the files are annotated on the following 4 pages. The first is the summary output file.

# File TA

```
**************************************************
*          Program TREE version 3.0              *
*   Copyright GEOMATRIX Consultants, July 1994   *
*             all rights reserved                *
*          written by Robert Youngs              *
**************************************************
```

*The following table lists the ground motion level (z), the mean frequency of exceedance (enu), the standard deviation (snu), the coefficient of variation and the mean magnitude and distance.*

total hazard:

Total

| iz | z | enu | snu | cov | M mean | R mean |
|---|---|---|---|---|---|---|
| 1 | 0.020 | 0.7429075E-01 | 0.4958389E-01 | 0.667 | 5.65 | 87.4 |
| 2 | 0.050 | 0.2442817E-01 | 0.1584745E-01 | 0.649 | 5.85 | 55.1 |
| 3 | 0.100 | 0.9520604E-02 | 0.5708864E-02 | 0.600 | 5.90 | 23.9 |
| 4 | 0.150 | 0.5916288E-02 | 0.3780910E-02 | 0.639 | 5.93 | 13.3 |
| 5 | 0.200 | 0.4253726E-02 | 0.2743620E-02 | 0.645 | 5.98 | 9.6 |
| 6 | 0.250 | 0.3240355E-02 | 0.2072253E-02 | 0.640 | 6.05 | 8.0 |
| 7 | 0.300 | 0.2542300E-02 | 0.1613955E-02 | 0.635 | 6.11 | 7.2 |
| 8 | 0.400 | 0.1640559E-02 | 0.1051396E-02 | 0.641 | 6.22 | 6.5 |
| 9 | 0.500 | 0.1095859E-02 | 0.7317414E-03 | 0.668 | 6.30 | 6.0 |
| 10 | 0.700 | 0.5158445E-03 | 0.3911018E-03 | 0.758 | 6.39 | 5.5 |
| 11 | 1.000 | 0.1800229E-03 | 0.1650633E-03 | 0.917 | 6.47 | 5.0 |
| 12 | 1.250 | 0.7867070E-04 | 0.8197354E-04 | 1.042 | 6.49 | 4.7 |
| 13 | 1.500 | 0.3558223E-04 | 0.4121185E-04 | 1.158 | 6.51 | 4.5 |
| 14 | 1.750 | 0.1648151E-04 | 0.2095028E-04 | 1.271 | 6.52 | 4.2 |
| 15 | 2.000 | 0.7735649E-05 | 0.1071467E-04 | 1.385 | 6.52 | 4.0 |

Contributions to Variance in Hazard Rate

| | | | precent contribution to variance from: | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| iz | z | cov | sitef | atn | sscl | msd | svfmod | wfind | fseg | tecm | cap | dip | mmax | recm | rate | b-val | magd |
| 1 | 0.020 | 0.667 | 0.043 | 0.642 | 0.008 | 0.004 | 0.000 | 0.000 | 0.000 | 0.000 | 0.002 | 0.001 | 0.031 | 0.000 | 0.010 | 0.029 | 0.230 |
| 2 | 0.050 | 0.649 | 0.094 | 0.491 | 0.031 | 0.007 | 0.000 | 0.000 | 0.001 | 0.000 | 0.008 | 0.002 | 0.054 | 0.000 | 0.025 | 0.016 | 0.271 |
| 3 | 0.100 | 0.600 | 0.080 | 0.063 | 0.020 | 0.009 | 0.000 | 0.000 | 0.001 | 0.000 | 0.027 | 0.004 | 0.148 | 0.000 | 0.080 | 0.006 | 0.561 |
| 4 | 0.150 | 0.639 | 0.063 | 0.006 | 0.012 | 0.005 | 0.000 | 0.001 | 0.001 | 0.000 | 0.034 | 0.005 | 0.188 | 0.000 | 0.104 | 0.003 | 0.577 |
| 5 | 0.200 | 0.645 | 0.069 | 0.014 | 0.012 | 0.004 | 0.000 | 0.001 | 0.000 | 0.000 | 0.036 | 0.007 | 0.211 | 0.000 | 0.119 | 0.002 | 0.526 |
| 6 | 0.250 | 0.640 | 0.084 | 0.034 | 0.014 | 0.003 | 0.000 | 0.000 | 0.000 | 0.000 | 0.037 | 0.009 | 0.225 | 0.000 | 0.129 | 0.001 | 0.463 |
| 7 | 0.300 | 0.635 | 0.102 | 0.061 | 0.017 | 0.002 | 0.000 | 0.000 | 0.000 | 0.000 | 0.037 | 0.011 | 0.232 | 0.000 | 0.137 | 0.001 | 0.400 |
| 8 | 0.400 | 0.641 | 0.141 | 0.130 | 0.023 | 0.002 | 0.000 | 0.000 | 0.000 | 0.000 | 0.033 | 0.014 | 0.226 | 0.000 | 0.143 | 0.001 | 0.286 |
| 9 | 0.500 | 0.668 | 0.175 | 0.203 | 0.029 | 0.001 | 0.000 | 0.000 | 0.000 | 0.000 | 0.028 | 0.016 | 0.205 | 0.000 | 0.141 | 0.000 | 0.201 |
| 10 | 0.700 | 0.758 | 0.212 | 0.322 | 0.037 | 0.002 | 0.000 | 0.000 | 0.000 | 0.000 | 0.019 | 0.015 | 0.159 | 0.000 | 0.128 | 0.000 | 0.106 |
| 11 | 1.000 | 0.917 | 0.222 | 0.423 | 0.044 | 0.002 | 0.000 | 0.000 | 0.001 | 0.000 | 0.012 | 0.012 | 0.115 | 0.000 | 0.113 | 0.000 | 0.056 |

| | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 12 | 1.250 | 1.042 | 0.217 | 0.465 | 0.050 | 0.002 | 0.000 | 0.000 | 0.001 | 0.000 | 0.009 | 0.010 | 0.097 | 0.000 | 0.108 | 0.000 | 0.041 |
| 13 | 1.500 | 1.158 | 0.209 | 0.488 | 0.057 | 0.002 | 0.000 | 0.000 | 0.001 | 0.000 | 0.007 | 0.008 | 0.087 | 0.000 | 0.106 | 0.000 | 0.035 |
| 14 | 1.750 | 1.271 | 0.200 | 0.499 | 0.065 | 0.003 | 0.000 | 0.000 | 0.001 | 0.000 | 0.006 | 0.007 | 0.081 | 0.000 | 0.107 | 0.000 | 0.032 |
| 15 | 2.000 | 1.385 | 0.195 | 0.499 | 0.074 | 0.003 | 0.000 | 0.000 | 0.001 | 0.000 | 0.004 | 0.007 | 0.077 | 0.000 | 0.109 | 0.000 | 0.030 |

*********************************************************************

*The following table lists the fractile hazard curves.*

Total

probability levels computed from distribution

| iz | z | enu | snu | pl: 0.0500 | 0.1000 | 0.1500 | 0.2000 | 0.3000 | 0.4000 | 0.5000 | 0.6000 | 0.7000 | 0.8000 | 0.8500 | 0.9000 | 0.9500 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.020 | 0.7417E-01 | 0.4941E-01 | 0.2512E-01 | 0.2951E-01 | 0.3388E-01 | 0.3715E-01 | 0.4266E-01 | 0.5012E-01 | 0.5754E-01 | 0.6761E-01 | 0.8318E-01 | 0.1072E+00 | 0.1259E+00 | 0.1445E+00 | 0.1778E+00 |
| 2 | 0.050 | 0.2441E-01 | 0.1582E-01 | 0.8128E-02 | 0.9772E-02 | 0.1122E-01 | 0.1230E-01 | 0.1479E-01 | 0.1738E-01 | 0.1995E-01 | 0.2344E-01 | 0.2754E-01 | 0.3388E-01 | 0.3802E-01 | 0.4467E-01 | 0.5623E-01 |
| 3 | 0.100 | 0.9521E-02 | 0.5709E-02 | 0.3162E-02 | 0.3802E-02 | 0.4365E-02 | 0.4898E-02 | 0.5888E-02 | 0.6918E-02 | 0.8128E-02 | 0.9333E-02 | 0.1122E-01 | 0.1349E-01 | 0.1479E-01 | 0.1698E-01 | 0.2089E-01 |
| 4 | 0.150 | 0.5915E-02 | 0.3778E-02 | 0.1820E-02 | 0.2239E-02 | 0.2570E-02 | 0.2884E-02 | 0.3548E-02 | 0.4169E-02 | 0.4898E-02 | 0.5754E-02 | 0.6918E-02 | 0.8511E-02 | 0.9550E-02 | 0.1096E-01 | 0.1349E-01 |
| 5 | 0.200 | 0.4250E-02 | 0.2738E-02 | 0.1288E-02 | 0.1585E-02 | 0.1820E-02 | 0.2042E-02 | 0.2512E-02 | 0.2951E-02 | 0.3548E-02 | 0.4169E-02 | 0.5012E-02 | 0.6026E-02 | 0.6918E-02 | 0.7943E-02 | 0.9550E-02 |
| 6 | 0.250 | 0.3235E-02 | 0.2066E-02 | 0.9550E-03 | 0.1202E-02 | 0.1380E-02 | 0.1585E-02 | 0.1905E-02 | 0.2291E-02 | 0.2692E-02 | 0.3162E-02 | 0.3802E-02 | 0.4677E-02 | 0.5248E-02 | 0.6026E-02 | 0.7244E-02 |
| 7 | 0.300 | 0.2537E-02 | 0.1608E-02 | 0.7413E-03 | 0.9333E-03 | 0.1096E-02 | 0.1230E-02 | 0.1514E-02 | 0.1820E-02 | 0.2138E-02 | 0.2512E-02 | 0.3020E-02 | 0.3631E-02 | 0.4074E-02 | 0.4677E-02 | 0.5623E-02 |
| 8 | 0.400 | 0.1636E-02 | 0.1048E-02 | 0.4266E-03 | 0.5623E-03 | 0.6761E-03 | 0.7762E-03 | 0.9772E-03 | 0.1175E-02 | 0.1380E-02 | 0.1622E-02 | 0.1950E-02 | 0.2399E-02 | 0.2692E-02 | 0.3090E-02 | 0.3715E-02 |
| 9 | 0.500 | 0.1093E-02 | 0.7295E-03 | 0.2399E-03 | 0.3388E-03 | 0.4169E-03 | 0.4898E-03 | 0.6166E-03 | 0.7586E-03 | 0.9120E-03 | 0.1096E-02 | 0.1318E-02 | 0.1622E-02 | 0.1820E-02 | 0.2089E-02 | 0.2512E-02 |
| 10 | 0.700 | 0.5146E-03 | 0.3902E-03 | 0.7586E-04 | 0.1230E-03 | 0.1549E-03 | 0.1862E-03 | 0.2512E-03 | 0.3311E-03 | 0.4169E-03 | 0.5129E-03 | 0.6310E-03 | 0.7943E-03 | 0.9120E-03 | 0.1072E-02 | 0.1288E-02 |
| 11 | 1.000 | 0.1796E-03 | 0.1648E-03 | 0.1413E-04 | 0.2692E-04 | 0.3548E-04 | 0.4365E-04 | 0.6607E-04 | 0.9772E-04 | 0.1318E-03 | 0.1660E-03 | 0.2138E-03 | 0.2884E-03 | 0.3467E-03 | 0.4169E-03 | 0.5248E-03 |
| 12 | 1.250 | 0.7852E-04 | 0.8183E-04 | 0.3311E-05 | 0.7586E-05 | 0.1072E-04 | 0.1349E-04 | 0.2291E-04 | 0.3715E-04 | 0.5129E-04 | 0.6761E-04 | 0.8913E-04 | 0.1318E-03 | 0.1622E-03 | 0.1995E-03 | 0.2512E-03 |
| 13 | 1.500 | 0.3552E-04 | 0.4116E-04 | 0.6761E-06 | 0.1995E-05 | 0.3162E-05 | 0.4365E-05 | 0.7943E-05 | 0.1380E-04 | 0.2042E-04 | 0.2754E-04 | 0.3890E-04 | 0.6166E-04 | 0.7762E-04 | 0.9772E-04 | 0.1230E-03 |
| 14 | 1.750 | 0.1646E-04 | 0.2093E-04 | 0.8913E-07 | 0.4571E-06 | 0.8913E-06 | 0.1349E-05 | 0.2754E-05 | 0.5012E-05 | 0.7943E-05 | 0.1148E-04 | 0.1698E-04 | 0.2951E-04 | 0.3802E-04 | 0.4786E-04 | 0.6166E-04 |
| 15 | 2.000 | 0.7723E-05 | 0.1070E-04 | 0.0000E+00 | 0.6457E-07 | 0.1905E-06 | 0.3631E-06 | 0.8710E-06 | 0.1698E-05 | 0.2951E-05 | 0.4898E-05 | 0.7762E-05 | 0.1413E-04 | 0.1862E-04 | 0.2399E-04 | 0.3090E-04 |

For ease in plotting, two other output files are provided.

# File TA.HAZ

*The first column is the ground motion level and the second column is the mean frequency of exceedance. The remaining columns are variance components.*

Total
15

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.2000E-01 | 0.7429075E-01 | 0.2458563E-02 | 0.2799368E-09 | 0.4340040E 05 | 0.2108830E-05 | 0.7700465E 04 | 0.3184284E 11 | 0.2405298E 04 | 0.7185611E 04 | 0.5657352E-03 | 0.5648266E+01 | 0.8736403E+02 |
| 0.5000E-01 | 0.2442817E-01 | 0.2511418E-03 | 0.2446344E 10 | 0.1888829E 05 | 0.4280039E 06 | 0.1365575E 04 | 0.3693643E 12 | 0.6162684E 05 | 0.4093806E 05 | 0.6813859E-04 | 0.5850622E+01 | 0.5512104E+02 |
| 0.1000E+00 | 0.9520604E-02 | 0.3259114E-04 | 0.9065085E 11 | 0.8905487E 06 | 0.1269317E 06 | 0.4826591E 05 | 0.7381950E 13 | 0.2608088E 05 | [illegible] | 0.1828271E-04 | 0.5904397E+01 | 0.2391552E+02 |
| 0.1500E+00 | 0.5916288E-02 | 0.1429528E-04 | 0.3516937E-11 | 0.4830763E-06 | 0.7604991E 07 | 0.2690566E 05 | 0.3714851E 13 | 0.1485288E 05 | 0.4537733E 07 | 0.8254971E 05 | 0.5925127E+01 | 0.1332632E+02 |
| 0.2000E+00 | 0.4253726E-02 | 0.7527452E-05 | 0.3355011E-11 | 0.2730956E-06 | 0.5324274E-07 | 0.1585880E-05 | 0.1688127E 13 | 0.8924450E 06 | 0.1562603E 07 | 0.3959934E 05 | 0.5981699E+01 | 0.9608540E+01 |
| 0.2500E+00 | 0.3240355E-02 | 0.4294234E-05 | 0.2138277E-11 | 0.1593632E-06 | 0.3870545E-07 | 0.9656305E-06 | 0.1455293E-13 | 0.5558214E-06 | 0.5837472E 08 | 0.1989103E 05 | 0.6048716E+01 | 0.8037117E+01 |
| 0.3000E+00 | 0.2542300E-02 | 0.2604852E-05 | 0.1000988E-11 | 0.9549199E-07 | 0.2843886E-07 | 0.6032144E-06 | 0.1001265E-13 | 0.3568983E-06 | 0.2497735E-08 | 0.1040925E-05 | 0.6112744E+01 | 0.7240568E+01 |
| 0.4000E+00 | 0.1640559E 02 | 0.1105434E-05 | 0.4594882E-12 | 0.3656184E-07 | 0.1543278E 07 | 0.2498338E 06 | 0.4544020E 14 | 0.1583444E-06 | 0.5587121E-09 | 0.3163360E 06 | 0.6218752E+01 | 0.6450059E+01 |
| 0.5000E+00 | 0.1095859E-02 | 0.5354455E-06 | 0.2557777E-12 | 0.1499411E-07 | 0.8311819E 08 | 0.1099725E-06 | 0.2254753E 14 | 0.7549578E 07 | 0.1539425E-09 | 0.1077605E-06 | 0.6296276E+01 | 0.6022988E+01 |
| 0.7000E+00 | 0.5158445E-03 | 0.1529607E-06 | 0.6821294E-13 | 0.2939315E 08 | 0.2325809E-08 | 0.2428193E 07 | 0.7028420E-15 | 0.1957997E 07 | 0.1936627E-10 | 0.1626831E-07 | 0.6393522E+01 | 0.5510108E+01 |
| 0.1000E+01 | 0.1800229E-03 | 0.2724589E-07 | 0.5915120E-14 | 0.3273214E-09 | 0.3312443E 09 | 0.3135975E-08 | 0.8985433E-16 | 0.3081401E-08 | 0.2000262E 11 | 0.1518610E-08 | 0.6465208E+01 | 0.5013911E+01 |
| 0.1250E+01 | 0.7867070E-04 | 0.6719660E-08 | 0.2942774E-14 | 0.6011396E-10 | 0.6713564E 10 | 0.6512066E 09 | 0.1268784E-16 | 0.7231596E 09 | 0.4239740E 12 | 0.2768075E 09 | 0.6494637E+01 | 0.4733348E+01 |
| 0.1500E+01 | 0.3558223E-04 | 0.1698416E-08 | 0.4883573E-15 | 0.1190753E-10 | 0.1435642E 10 | 0.1473878E 09 | 0.3649058E-17 | 0.1800091E-09 | 0.1006509E 12 | 0.5893808E 10 | 0.6510748E+01 | 0.4471420E+01 |
| 0.1750E+01 | 0.1648151E-04 | 0.4389143E-09 | 0.1446073E-15 | 0.2455800E-11 | 0.3259752E-11 | 0.3542269E-10 | 0.5739883E 18 | 0.4683044E 10 | 0.2529670E-13 | 0.1388257E 10 | 0.6518601E+01 | 0.4223735E+01 |
| 0.2000E+01 | 0.7735649E-05 | 0.1148041E-09 | 0.3705187E-16 | 0.5057522E-12 | 0.7718211E-12 | 0.8809431E-11 | 0.1600143E 18 | 0.1254871E 10 | 0.6523937E 14 | 0.3475689E-11 | 0.6520666E+01 | 0.3975346E+01 |

## File TA.FRC

*The first column is the ground motion level and the remaining columns are the fractile hazard curves identified by the header on the second line.*

```
Total
  15          0.0500     0.1000     0.1500     0.2000     0.3000     0.4000     0.5000     0.6000     0.7000     0.8000     0.8500     0.9000     0.9500
 0.2000E-01 0.2512E-01 0.2951E-01 0.3388E-01 0.3715E-01 0.4266E-01 0.5012E-01 0.5754E-01 0.6761E-01 0.8318E-01 0.1072E+00 0.1259E+00 0.1445E+00 0.1778E+00
 0.5000E-01 0.8128E-02 0.9772E-02 0.1122E-01 0.1230E-01 0.1479E-01 0.1738E-01 0.1995E-01 0.2344E-01 0.2754E-01 0.3388E-01 0.3802E-01 0.4467E-01 0.5623E-01
 0.1000E+00 0.3162E-02 0.3802E-02 0.4365E-02 0.4898E-02 0.5888E-02 0.6918E-02 0.8128E-02 0.9333E-02 0.1122E-01 0.1349E-01 0.1479E-01 0.1698E-01 0.2089E-01
 0.1500E+00 0.1820E-02 0.2239E-02 0.2570E-02 0.2884E-02 0.3548E-02 0.4169E-02 0.4898E-02 0.5754E-02 0.6918E-02 0.8511E-02 0.9550E-02 0.1096E-01 0.1349E-01
 0.2000E+00 0.1288E-02 0.1585E-02 0.1820E-02 0.2042E-02 0.2512E-02 0.2951E-02 0.3548E-02 0.4169E-02 0.5012E-02 0.6026E-02 0.6918E-02 0.7943E-02 0.9550E-02
 0.2500E+00 0.9550E-03 0.1202E-02 0.1380E-02 0.1585E-02 0.1905E-02 0.2291E-02 0.2692E-02 0.3162E-02 0.3802E-02 0.4677E-02 0.5248E-02 0.6026E-02 0.7244E-02
 0.3000E+00 0.7413E-03 0.9333E-03 0.1096E-02 0.1230E-02 0.1514E-02 0.1820E-02 0.2138E-02 0.2512E-02 0.3020E-02 0.3631E-02 0.4074E-02 0.4677E-02 0.5623E-02
 0.4000E+00 0.4266E-03 0.5623E-03 0.6761E-03 0.7762E-03 0.9772E-03 0.1175E-02 0.1380E-02 0.1622E-02 0.1950E-02 0.2399E-02 0.2692E-02 0.3090E-02 0.3715E-02
 0.5000E+00 0.2399E-03 0.3388E-03 0.4169E-03 0.4898E-03 0.6166E-03 0.7586E-03 0.9120E-03 0.1096E-02 0.1318E-02 0.1622E-02 0.1820E-02 0.2089E-02 0.2512E-02
 0.7000E+00 0.7586E-04 0.1230E-03 0.1549E-03 0.1862E-03 0.2512E-03 0.3311E-03 0.4169E-03 0.5129E-03 0.6310E-03 0.7943E-03 0.9120E-03 0.1072E-02 0.1288E-02
 0.1000E+01 0.1413E-04 0.2692E-04 0.3548E-04 0.4365E-04 0.6607E-04 0.9772E-04 0.1318E-03 0.1660E-03 0.2138E-03 0.2884E-03 0.3467E-03 0.4169E-03 0.5248E-03
 0.1250E+01 0.3311E-05 0.7586E-05 0.1072E-04 0.1349E-04 0.2291E-04 0.3715E-04 0.5129E-04 0.6761E-04 0.8913E-04 0.1318E-03 0.1622E-03 0.1995E-03 0.2512E-03
 0.1500E+01 0.6761E-06 0.1995E-05 0.3162E-05 0.4365E-05 0.7943E-05 0.1380E-04 0.2042E-04 0.2754E-04 0.3890E-04 0.6166E-04 0.7762E-04 0.9772E-04 0.1230E-03
 0.1750E+01 0.8913E-07 0.4571E-06 0.8913E-06 0.1349E-05 0.2754E-05 0.5012E-05 0.7943E-05 0.1148E-04 0.1698E-04 0.2951E-04 0.3802E-04 0.4786E-04 0.6166E-04
 0.2000E+01 0.0000E+00 0.6457E-07 0.1905E-06 0.3631E-06 0.8710E-06 0.1698E-05 0.2951E-05 0.4898E-05 0.7762E-05 0.1413E-04 0.1862E-04 0.2399E-04 0.3090E-04
```

The following series of figures illustrates the results in terms of the fractile hazard curves over all uncertainties and the condition mean hazard curves for specific cases. The first figure in each set of three shows the mean and fractile hazard curves over all uncertainties. The second figure shows the mean and 5$^{th}$ and 95$^{th}$-percentile fractile hazard curves over all uncertainties and the conditional mean hazard curves for each attenuation relationship. The third figure shows the mean and 5$^{th}$ and 95$^{th}$-percentile fractile hazard curves over all uncertainties and the conditional mean hazard curves for the two site factors based on empirical and site response approaches.

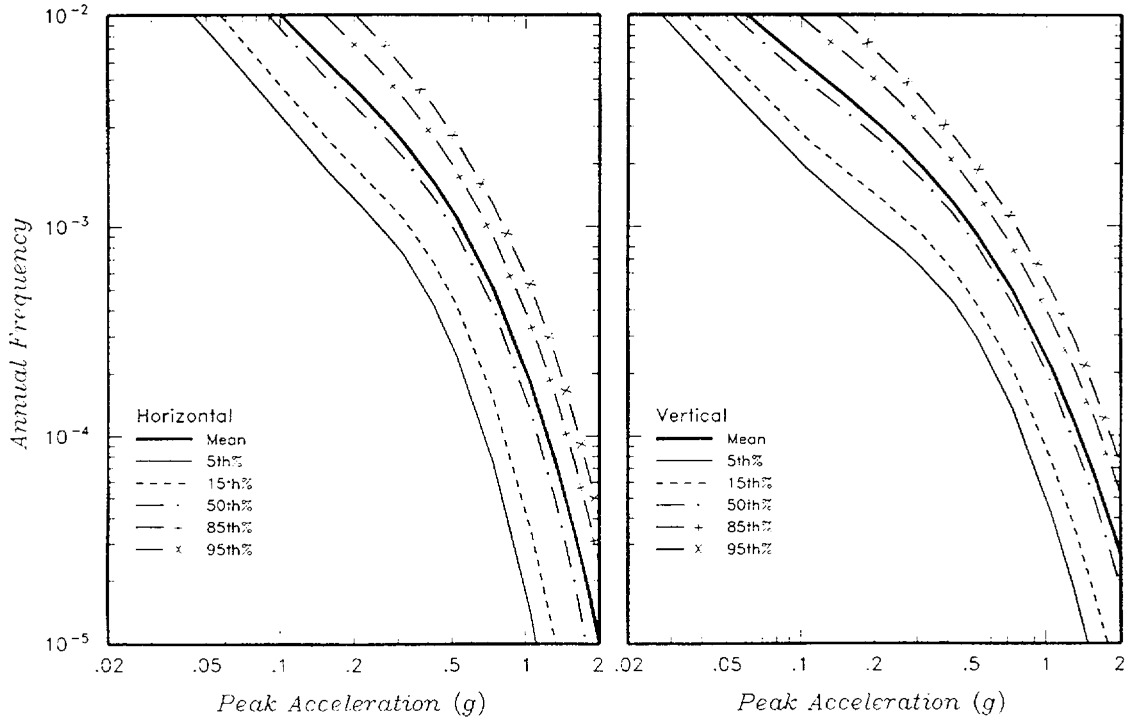


Figure SHA01-1 Fractile hazard curves for PGA

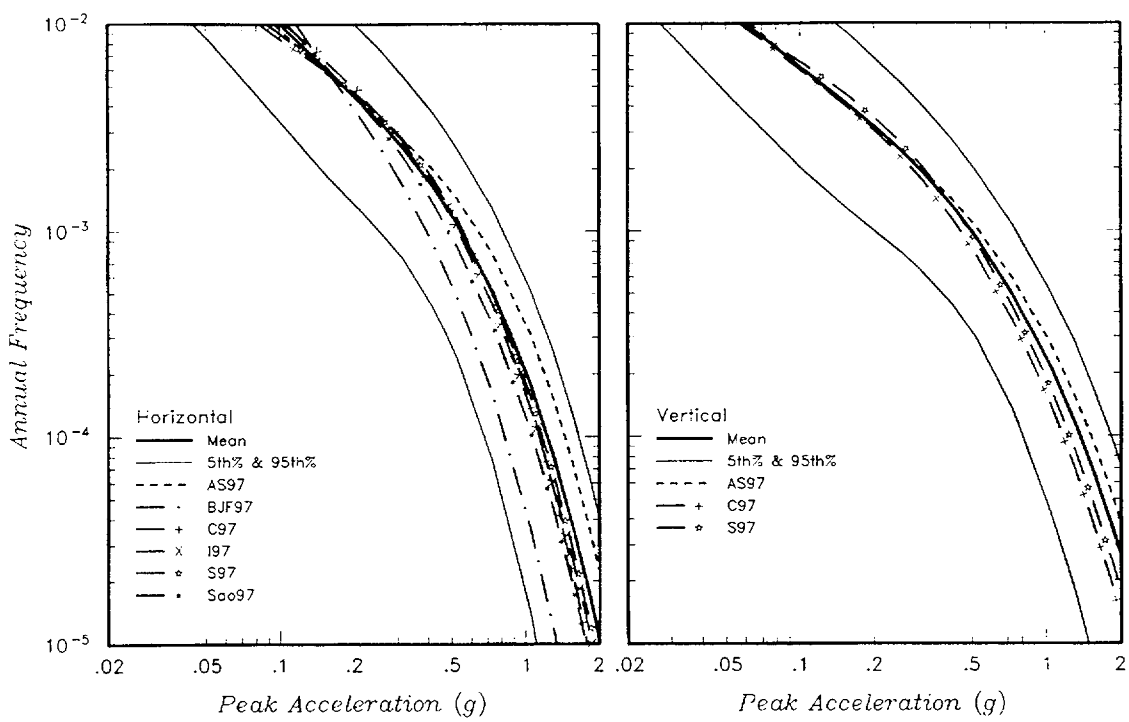


Figure SHA01-2 Conditional mean hazard curves for PGA attenuation relationships

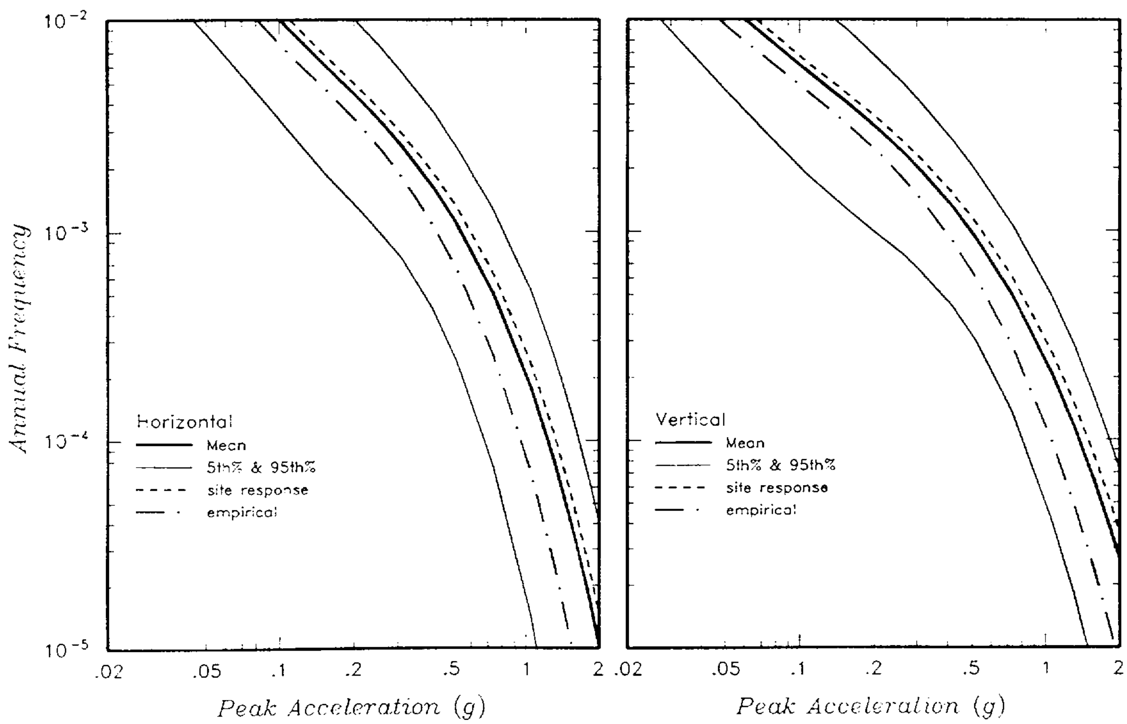


Figure SHA01-3 Conditional mean hazard curves for PGA site factors

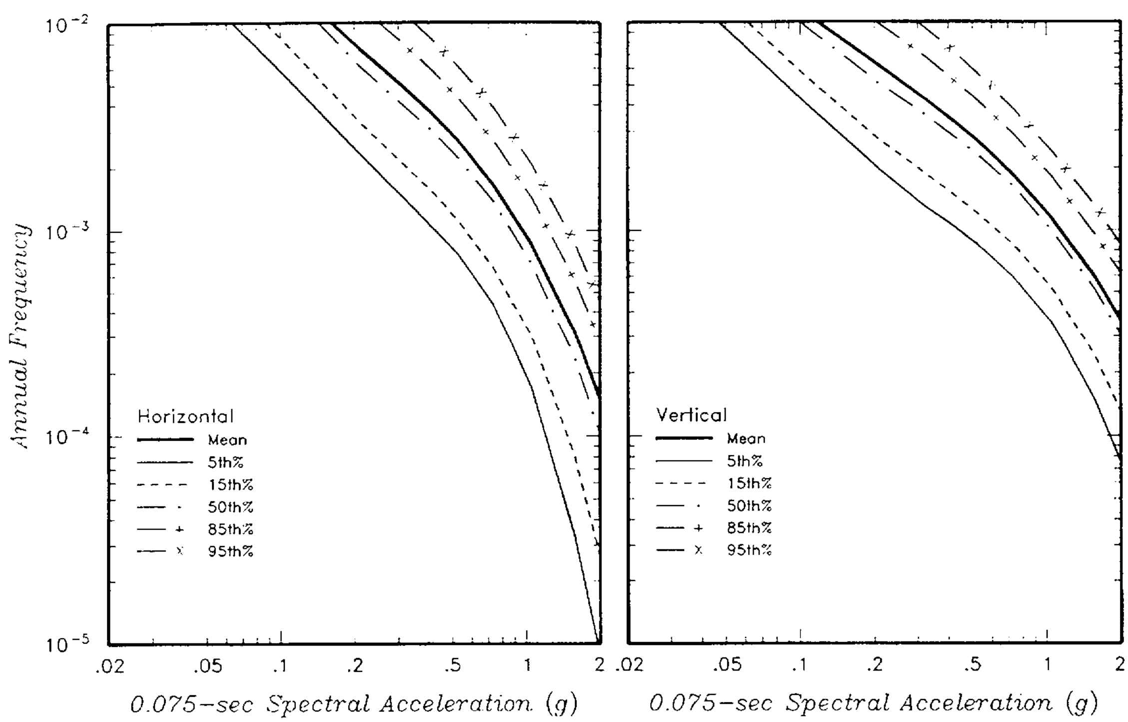


Figure SHA01-4 Fractile hazard curves for 0.075-sec SA

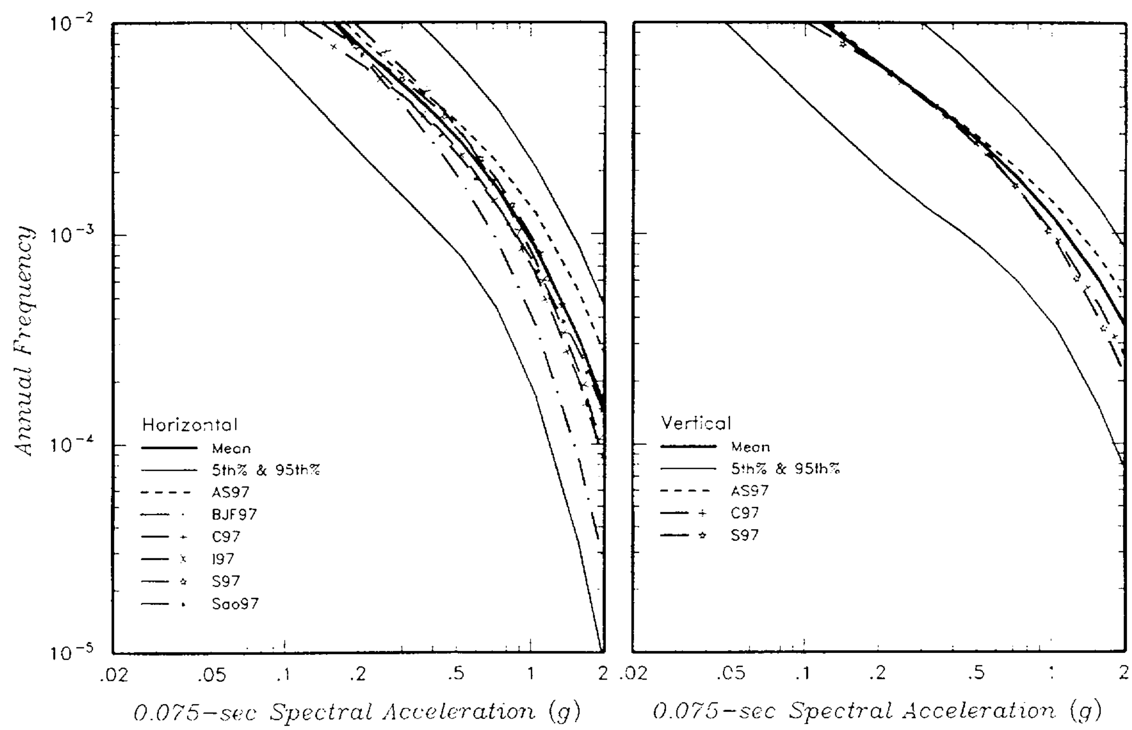


Figure SHA01-5 Conditional mean hazard curves for 0.075-sec SA attenuation relationships

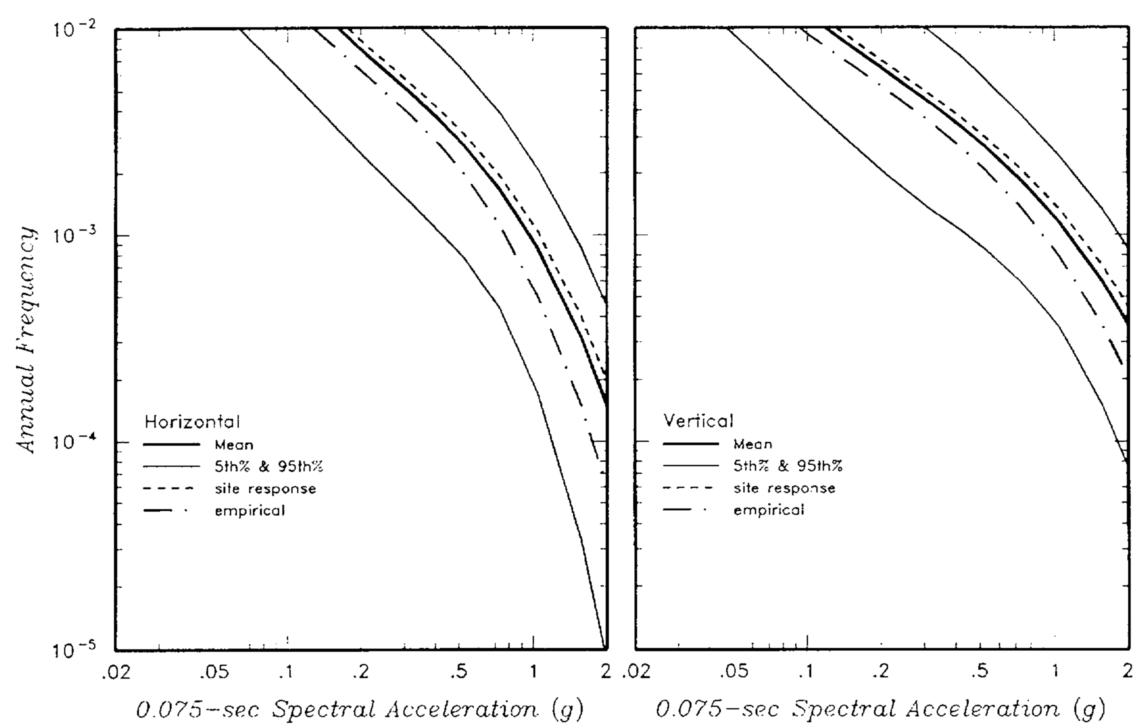


Figure SHA01-6 Conditional mean hazard curves for 0.075-sec SA site factors

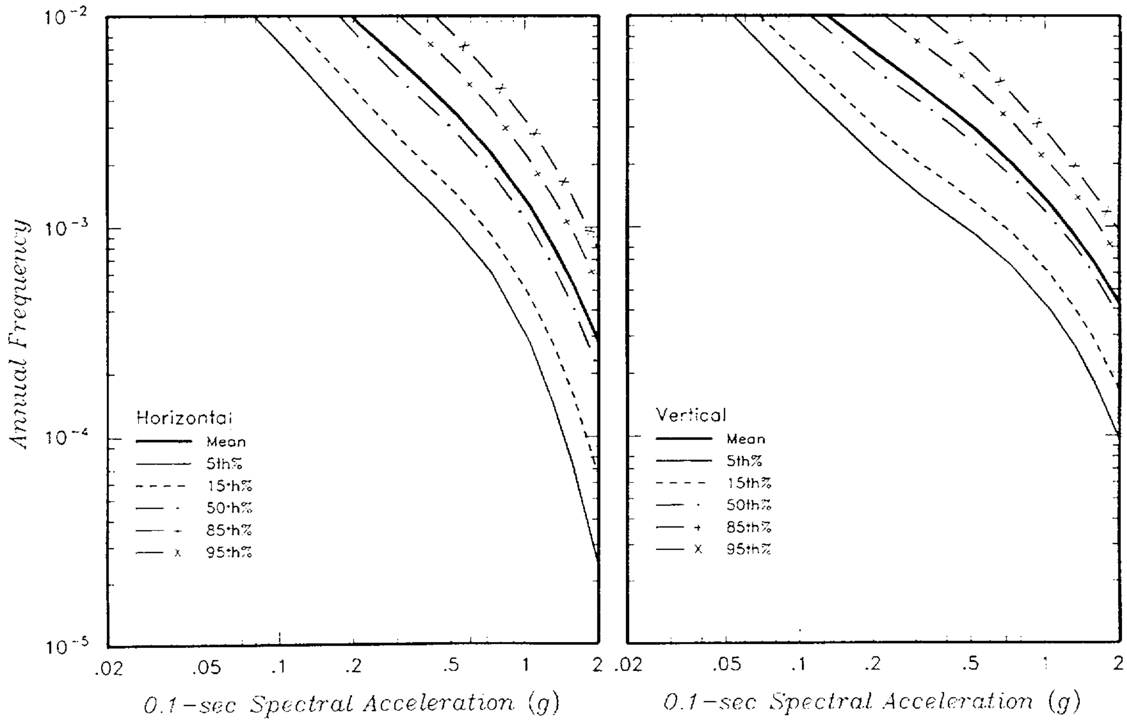


Figure SHA01-7 Fractile hazard curves for 0.1-sec SA

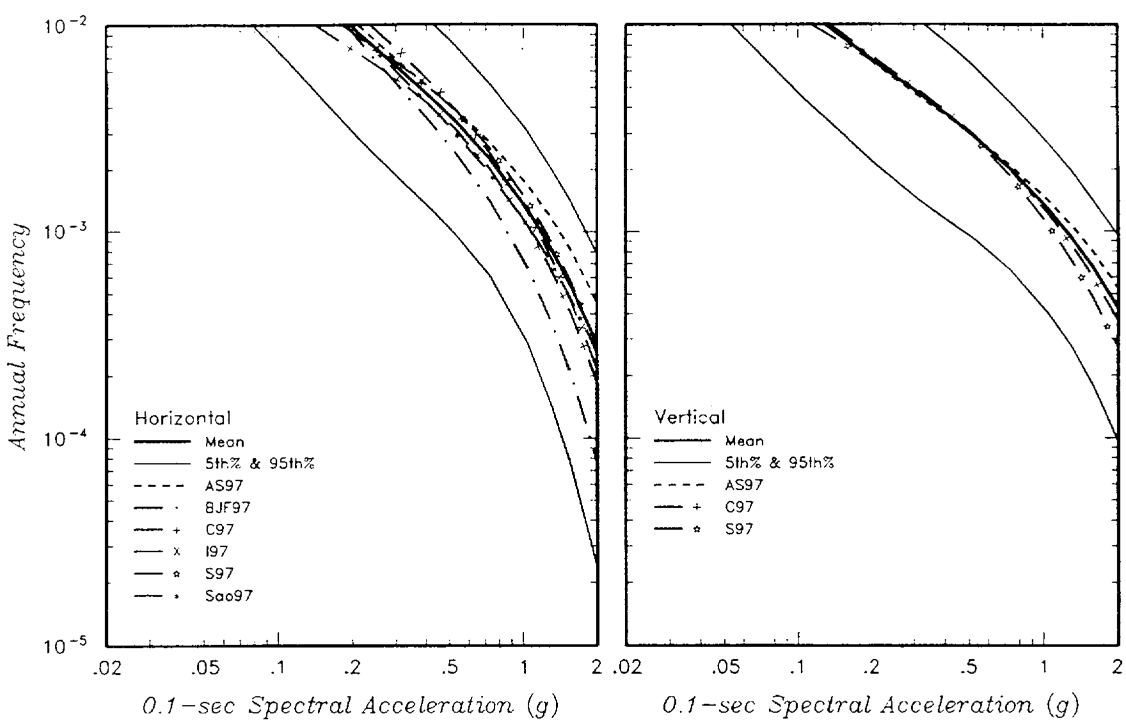


Figure SHA01-8 Conditional mean hazard curves for 0.1-sec SA attenuation relationships

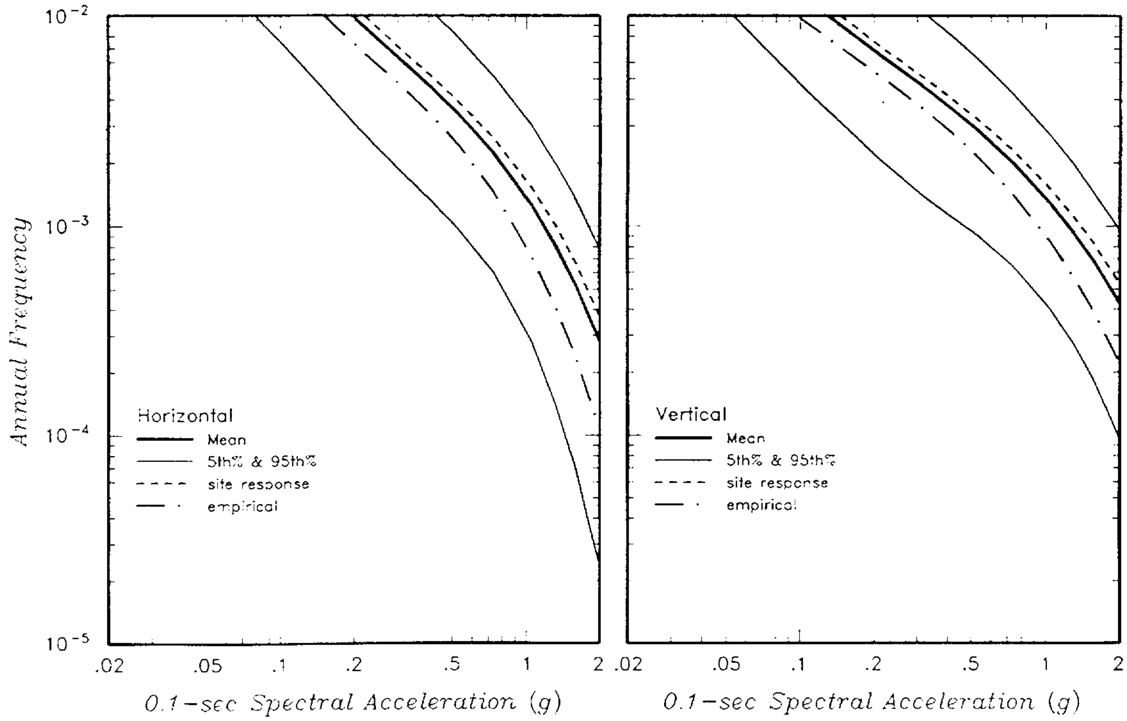


Figure SHA01-9 Conditional mean hazard curves for 0.1-sec SA site factors

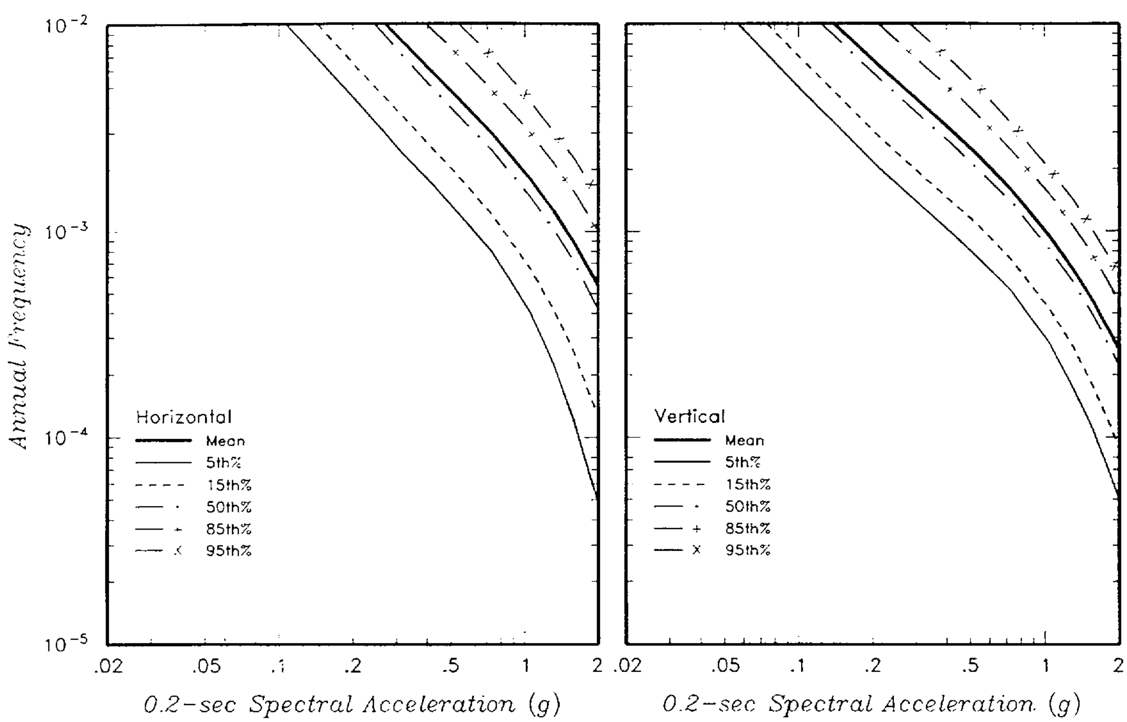


Figure SHA01-10 Fractile hazard curves for 0.2-sec SA

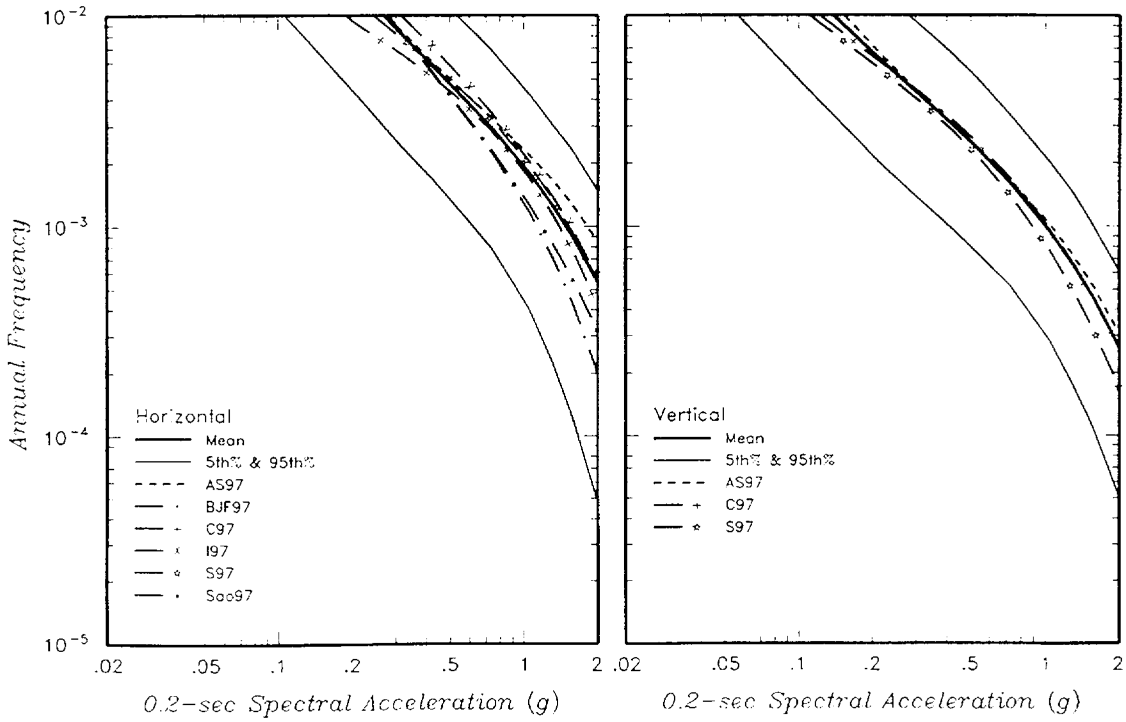


Figure SHA01-11 Conditional mean hazard curves for 0.2-sec SA attenuation relationships

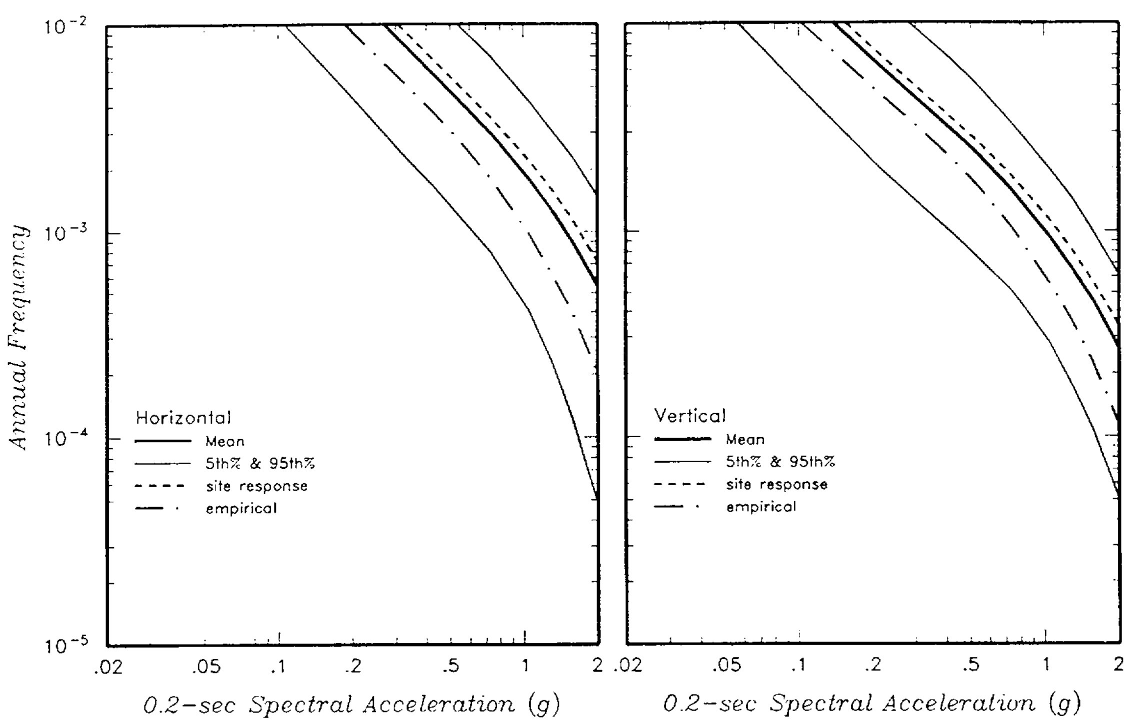


Figure SHA01-12 Conditional mean hazard curves for 0.2-sec SA site factors

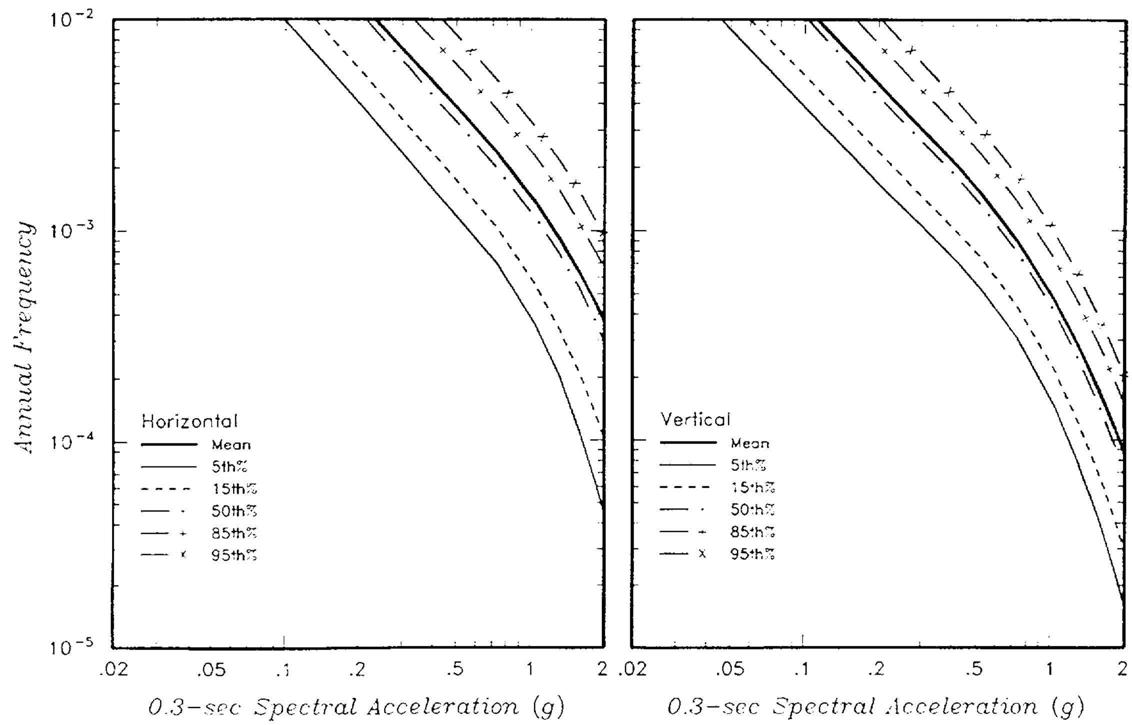


Figure SHA01-13 Fractile hazard curves for 0.3-sec SA

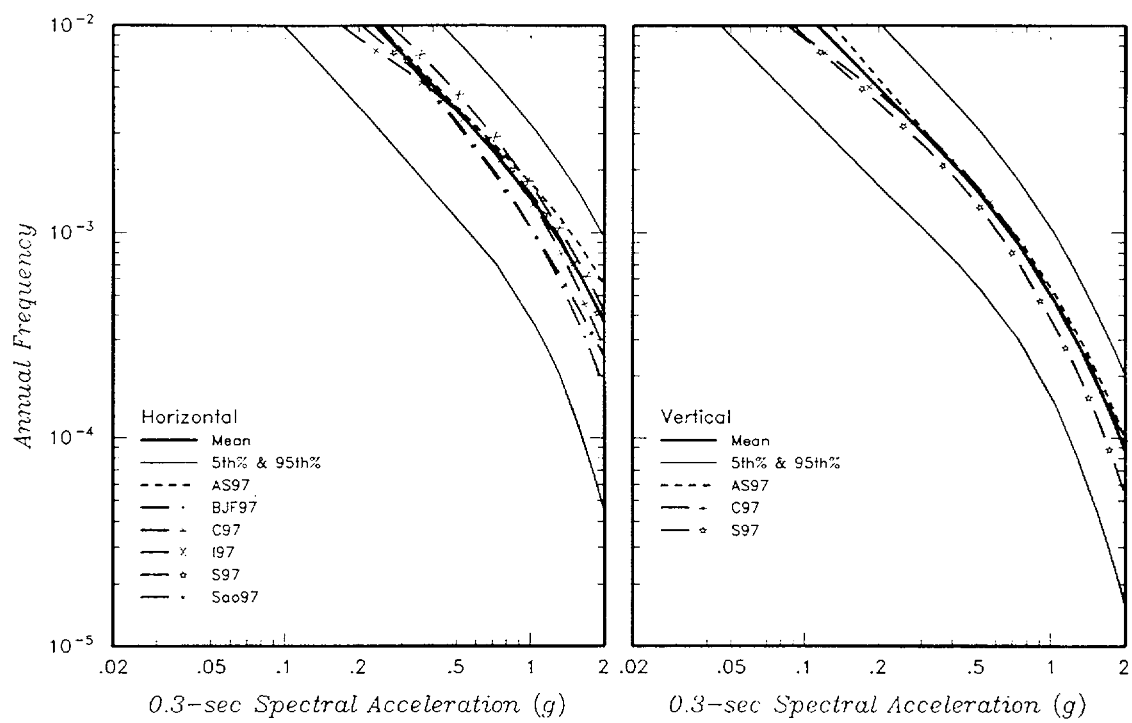


Figure SHA01-14 Conditional mean hazard curves for 0.3-sec SA attenuation relationships

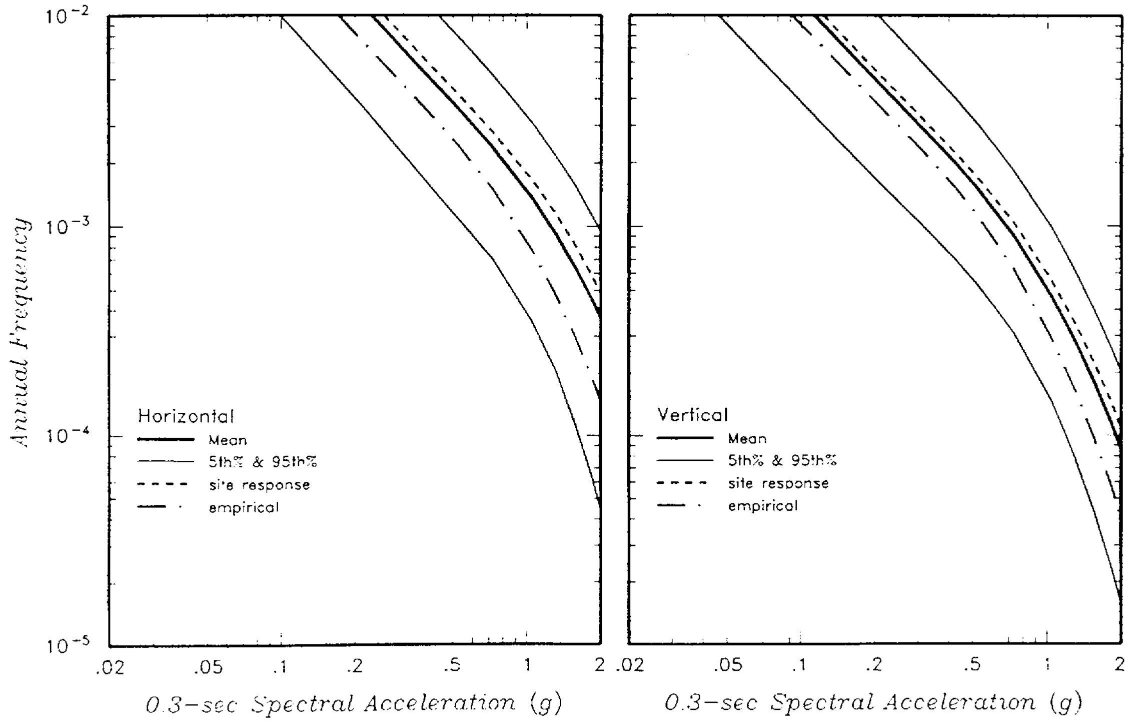


Figure SHA01-15 Conditional mean hazard curves for 0.3-sec SA site factors

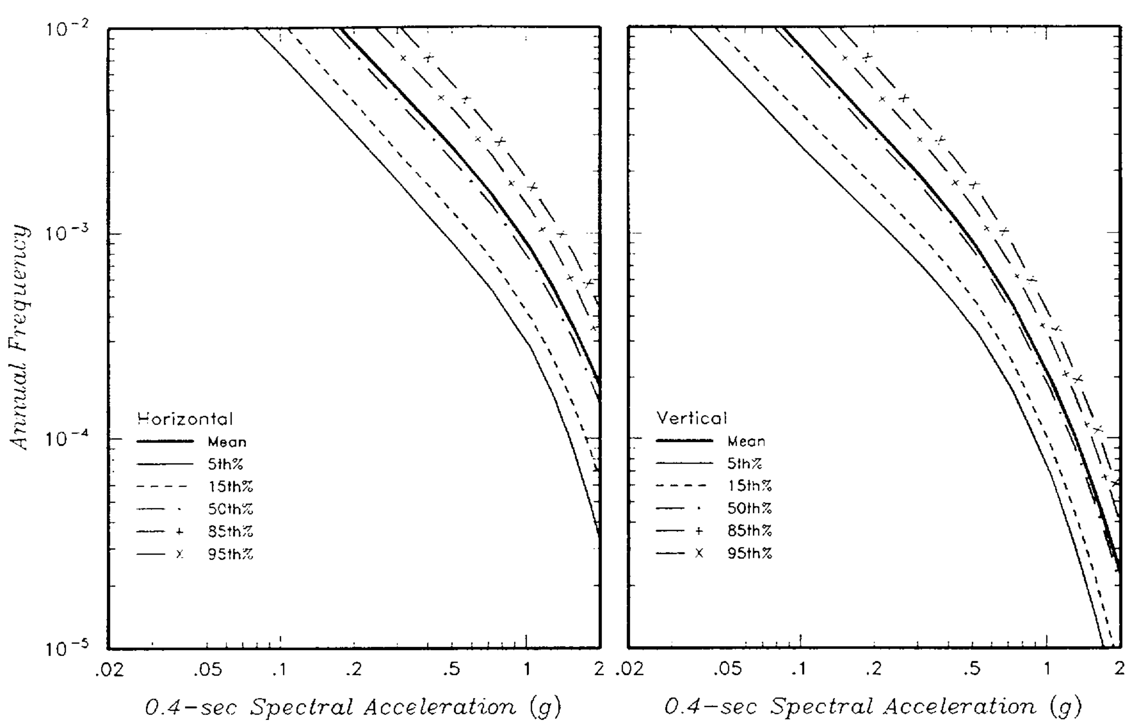


Figure SHA01-16 Fractile hazard curves for 0.4-sec SA

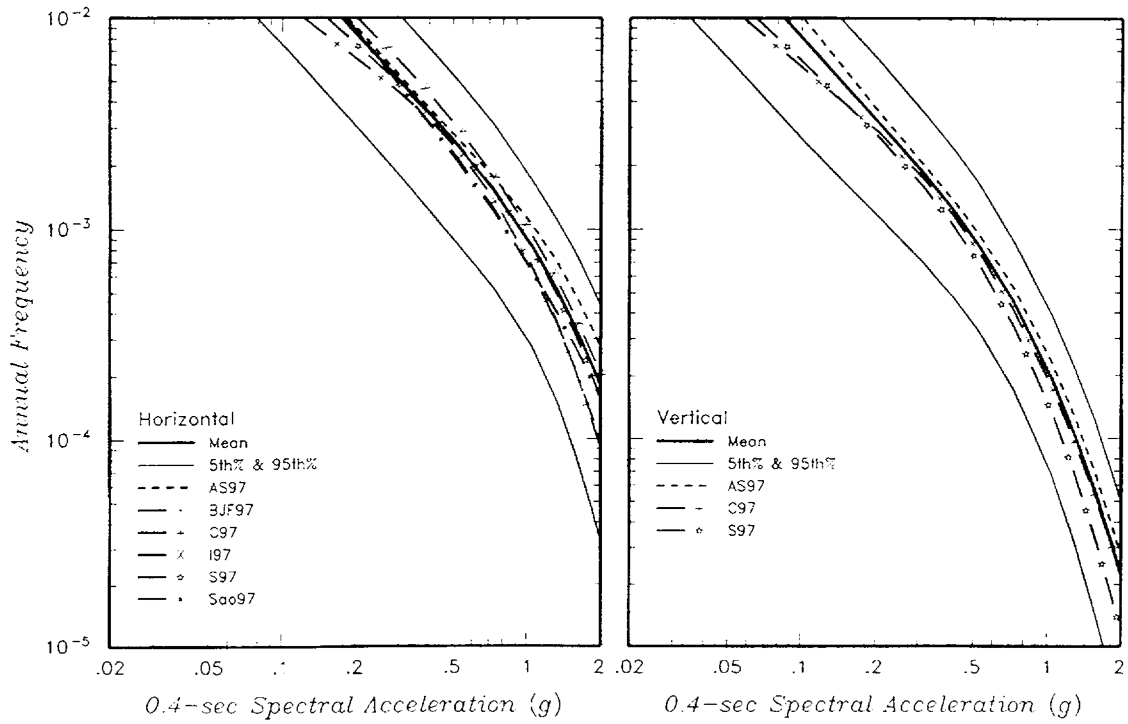


Figure SHA01-17 Conditional mean hazard curves for 0.4-sec SA attenuation relationships

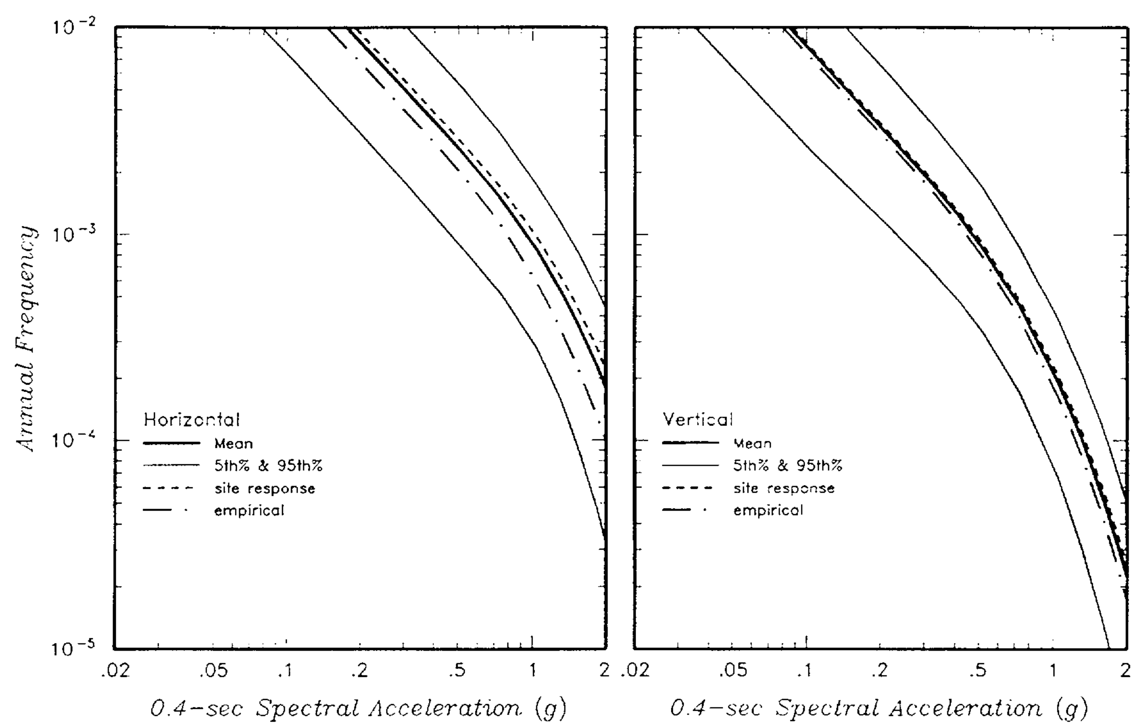


Figure SHA01-18 Conditional mean hazard curves for 0.4-sec SA site factors

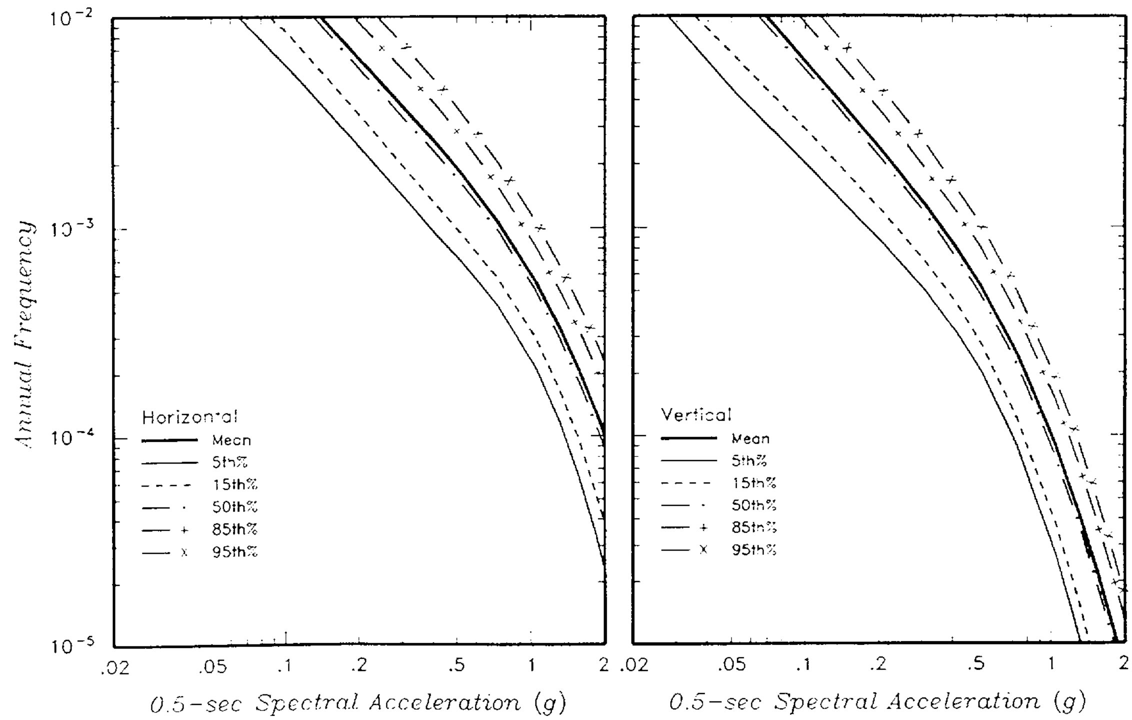


Figure SHA01-19 Fractile hazard curves for 0.5-sec SA

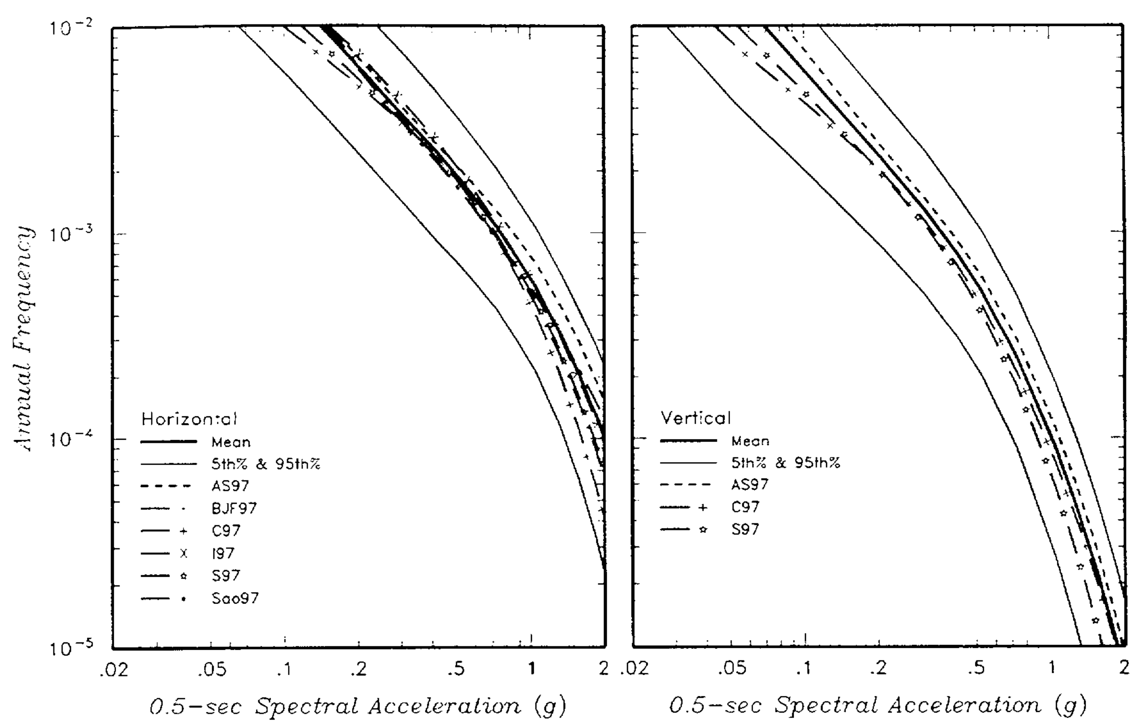


Figure SHA01-20 Conditional mean hazard curves for 0.5-sec SA attenuation relationships

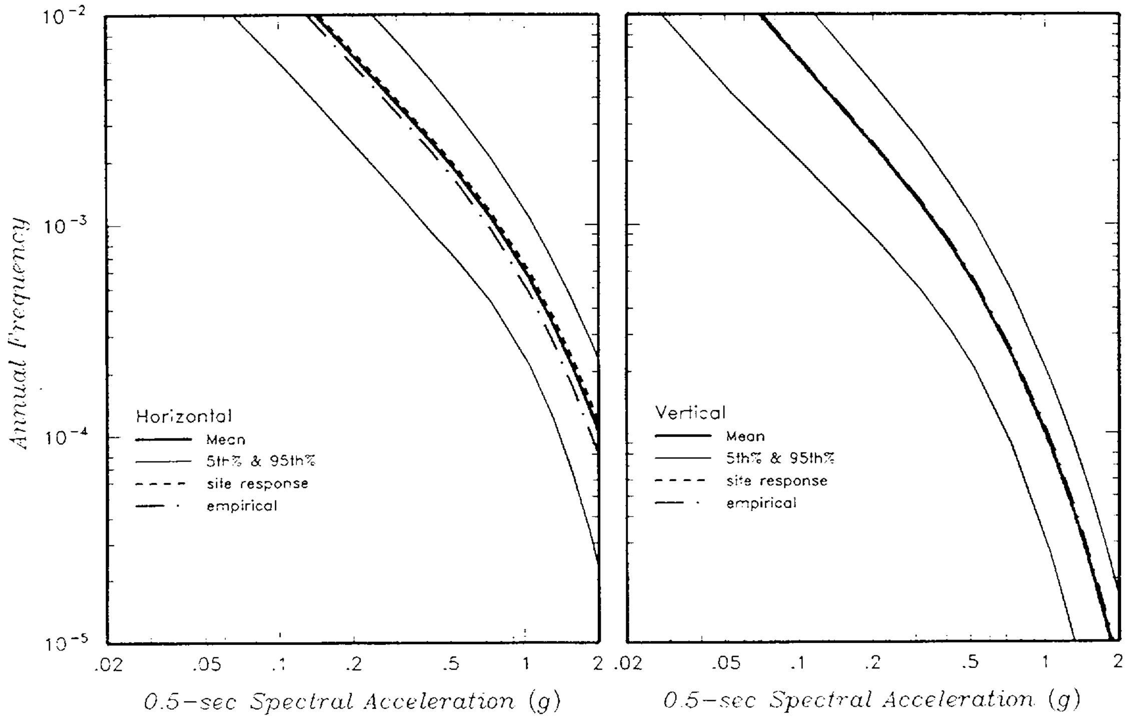


Figure SHA01-21 Conditional mean hazard curves for 0.5-sec SA site factors

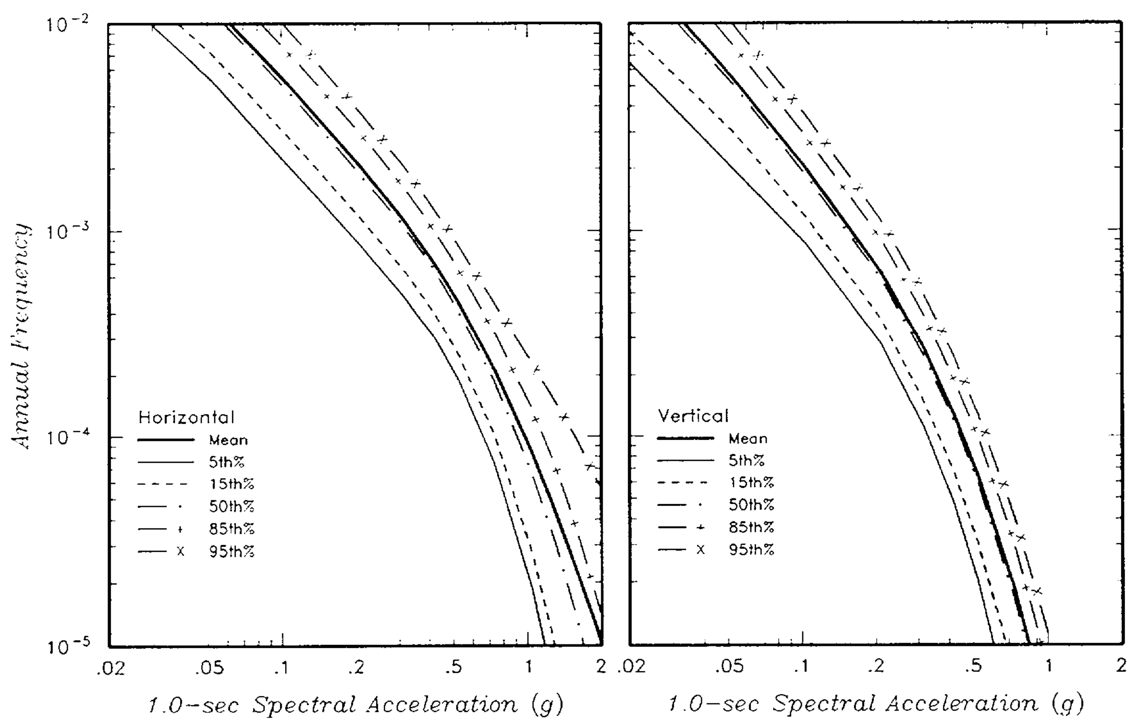


Figure SHA01-22 Fractile hazard curves for 1.0-sec SA

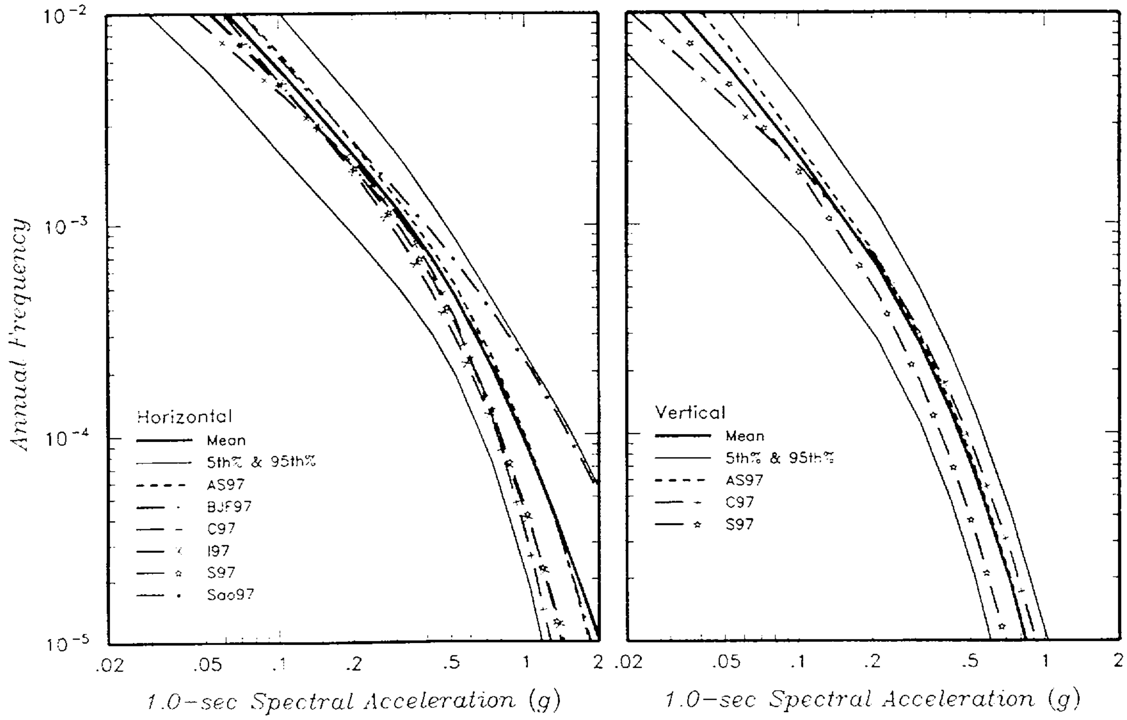


Figure SHA01-23 Conditional mean hazard curves for 1.0-sec SA attenuation relationships

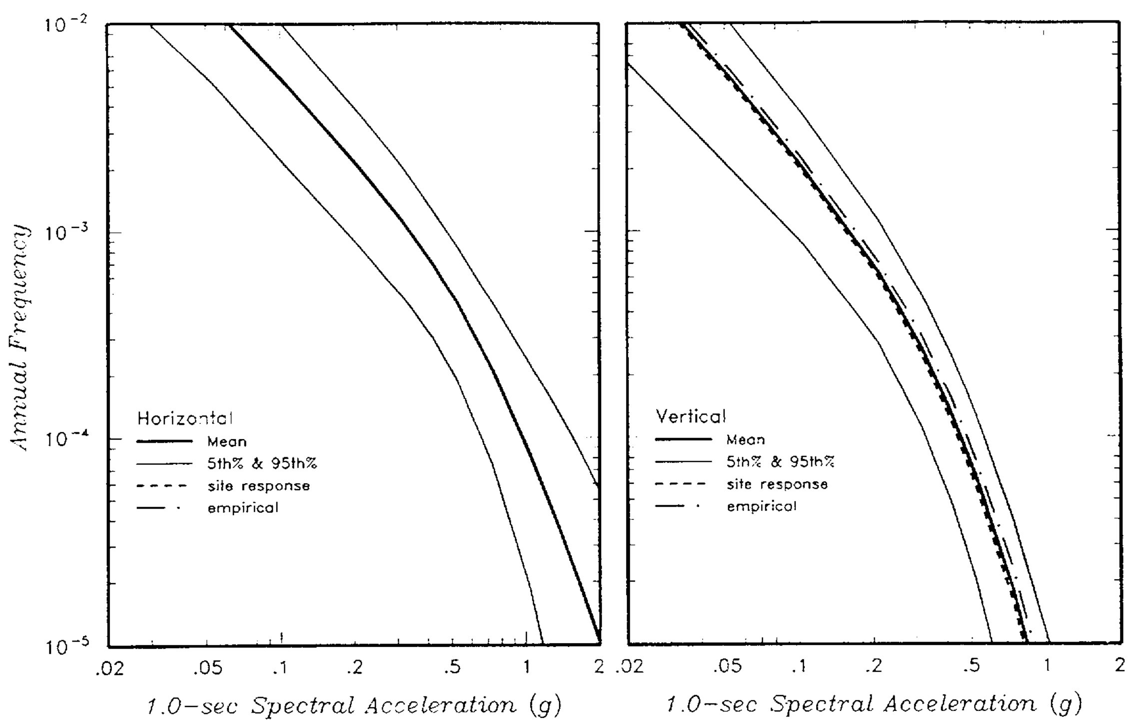


Figure SHA01-24 Conditional mean hazard curves for 1.0-sec SA site factors

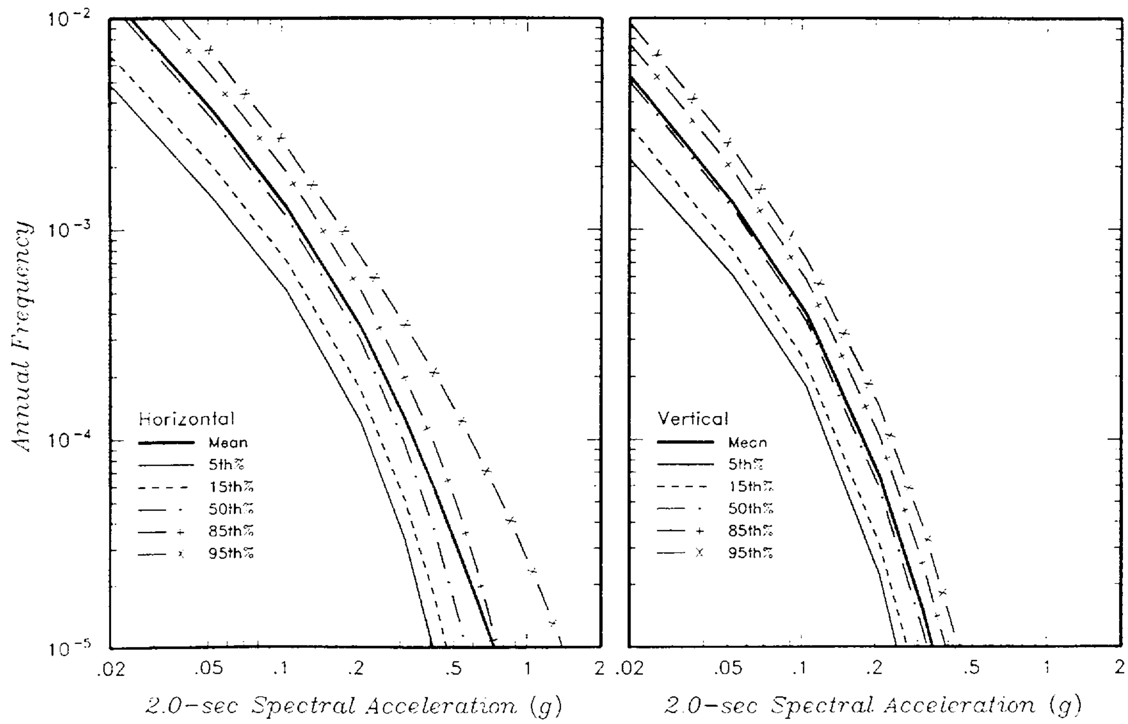


Figure SHA01-25 Fractile hazard curves for 2.0-sec SA

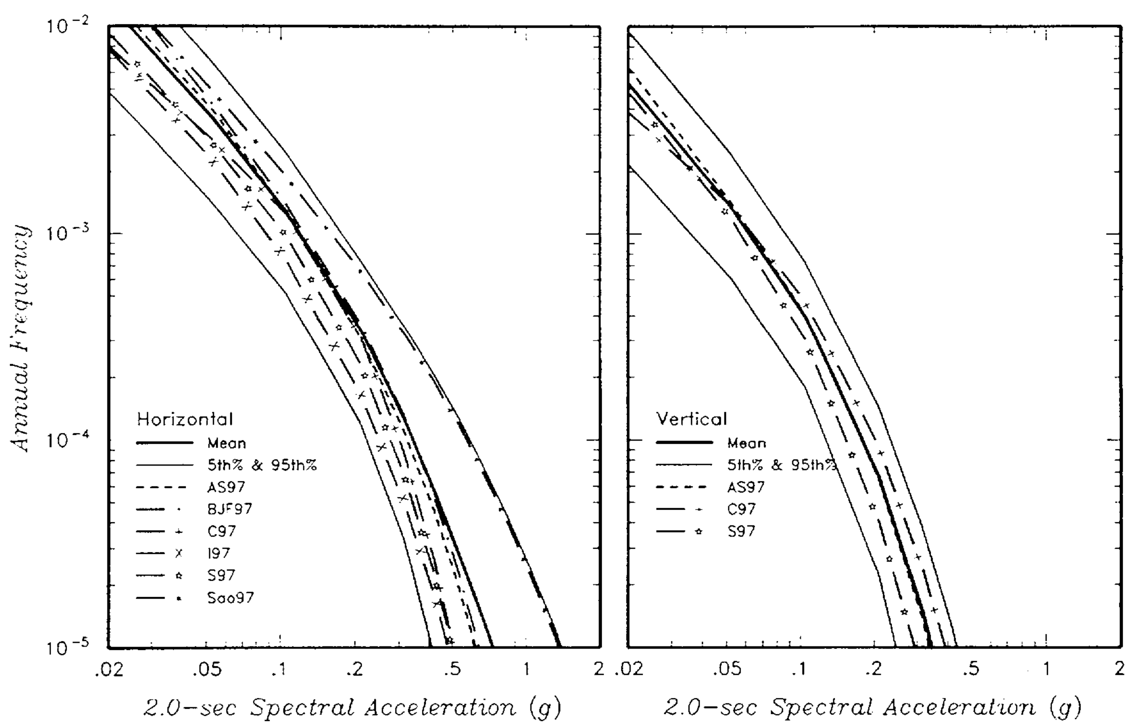


Figure SHA01-26 Conditional mean hazard curves for 2.0-sec SA attenuation relationships

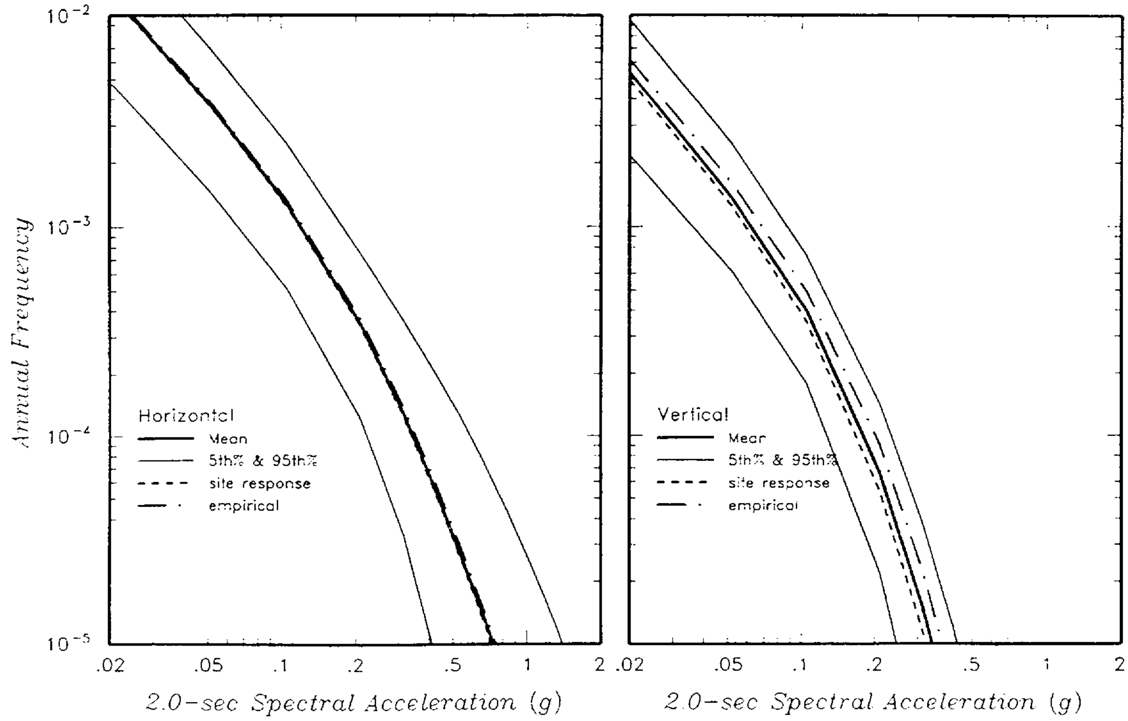


Figure SHA01-27 Conditional mean hazard curves for 2.0-sec SA site factors

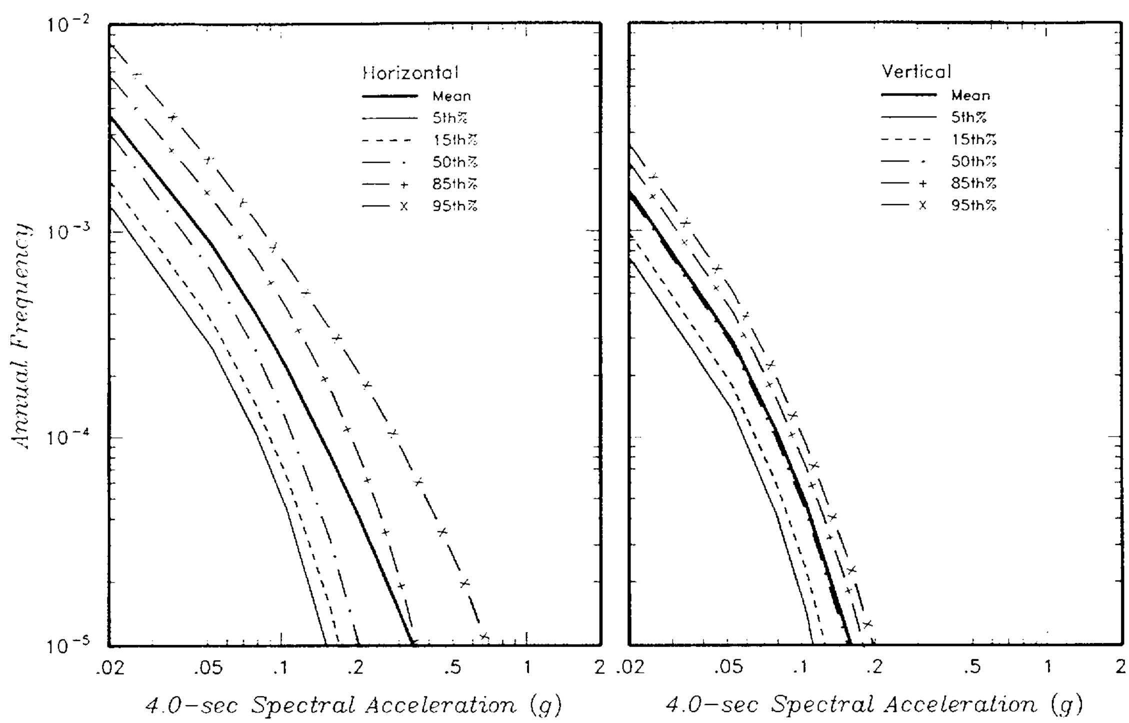


Figure SHA01-28 Fractile hazard curves for 4.0-sec SA

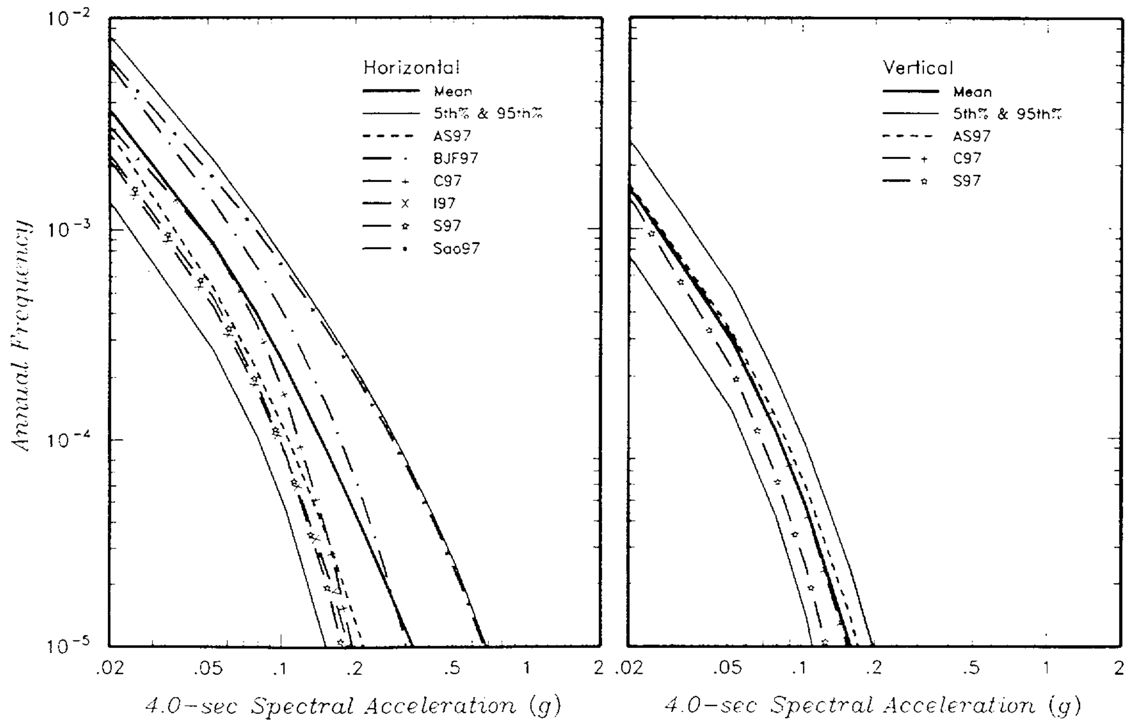


Figure SHA01-29 Conditional mean hazard curves for 4.0-sec SA attenuation relationships

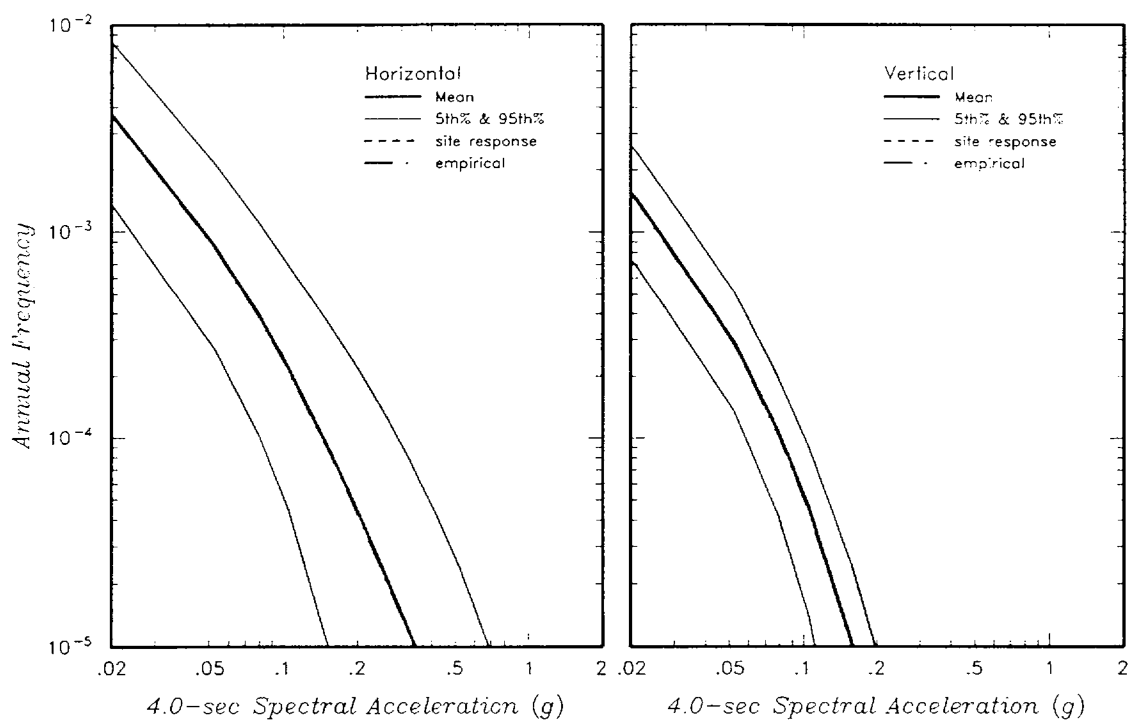


Figure SHA01-30 Conditional mean hazard curves for 4.0-sec SA site factors

2. **Site velocity measurements, the 30 random property models (all parameters - shear wave velocity, damping, modulus reduction ratio as a function of shear strain), results of simulations, and input spectra (earthquake magnitude and distance matrix of inputs).**

PFS Response
Directory **SeisHaz.002** on the attached CD contains the requested information.

Directory **SeisHaz.002** contains spreadsheet SV-VELOCITY.XLS with the data from the seismic cone tests and downhole velocity measurements. These data are also listed in Attachment A to Geomatrix (2001b)

The inputs and output of the site response analyses are contained in the following subdirectories.

Subdirectory **.\timehist** contains the recorded surface time histories scaled to a **M** 6.5 at 1 km and an **M** 7.0 at 9 km. The twelve original time histories are listed in Table F-4, Appendix F of Geomatrix (2001a). The time histories in subdirectory **.\timehist** have been scaled to the two target spectra shown on Figures F-5 and F-6 (Appendix F, Geomatrix, 2001a). The records scaled to a **M** 6.5 are designated E01M65.ACC through E12M65.ACC and the records scaled to a **M** 7.0 are designated E01M70.ACC through E12M70.ACC.

Subdirectories **.\decon.m65** and **.\decon.m70** contain the input and output files for computing the base motions for the generic WUS rock profile for **M** 6.5 and **M** 7.0 scaling, respectively. These calculations are performed using program SHAKE. The files for **M** 6.5 are designated W-D6501.* through W-D6501.* for the 12 input time histories. The files for **M** 7.0 are designated W-D7001.* through W-D7001.*. The files with extensions *.IN are the input files, those with *.OUT are the output files, and those with *.PUN contain the output base motion time histories. The options of program SHAKE used in the calculations are: option 8, read in strain-compatible modulus reduction and damping relationships; option 2, read in soil profile; option 1, read in input time history; option 3, specify location of input time history; option 4, compute strain-compatible soil properties; and option 5, compute output time histories.

Subdirectories **.\wus.m65** and **.\wus.m70** contain the computation of surface motions for the 30 randomized WUS rock profiles for **M** 6.5 and **M** 7.0 scaling, respectively. The files in subdirectory **.\wus.m65** are designated WMxx65yy.* and those in subdirectory **.\wus.m70** are designated WMxx70yy.*. The profile simulation number is designated xx and takes on the values from 01 to 30. The input time histories are those in subdirectory **.\decon.m65** and are designated by the values of yy from 01 to 12. The files with extensions *.IN are the input files, those with *.OUT are the output files, and those with *.PUN contain the output surface time histories. Similar file designations are used in subdirectory **.\wus.m70**. The computations are performed using program SHAKE options 8, 2, 1, 3, 4, and 5.

Subdirectory **.\wus.sp** contains the computed response spectra for the surface motions in subdirectories **.\wus.m65** and **.\wus.m70**. The files have the same file names as the computed surface motions and the file extension *.050 for 5% damping. The files contain seven columns, the second is spectral period and the sixth is spectral acceleration in g's.

The following subdirectories contain the computation of surface motions for the various Skull Valley profiles

**Site Response Analyses for Spectral Ratio Cases**

| Scaling Level for Input Motion | Skull Valley Profile | WUS Profile | Subdirectory for Site Response Analysis |
|---|---|---|---|
| **M** 7 on Stansbury fault | Best Estimate – Constant Tertiary Velocity | Generic Rock | .\m-04c1.m70 |
| **M** 6.5 on East fault | Best Estimate – Constant Tertiary Velocity | Generic Rock | .\m-04c1.m65 |
| **M** 7 on Stansbury fault | Best Estimate – Increasing Tertiary Velocity | Generic Rock | .\gm-04c1.m70 |
| **M** 6.5 on East fault | Best Estimate – Increasing Tertiary Velocity | Generic Rock | .\gm-04c1.m65 |
| **M** 7 on Stansbury fault | Lower Range – Constant Tertiary Velocity | Generic Rock | .\l-04c1.m70 |
| **M** 6.5 on East fault | Lower Range – Constant Tertiary Velocity | Generic Rock | .\l-04c1.m65 |
| **M** 7 on Stansbury fault | Lower Range – Increasing Tertiary Velocity | Generic Rock | .\gl-04c1.m70 |
| **M** 6.5 on East fault | Lower Range – Increasing Tertiary Velocity | Generic Rock | .\gl-04c1.m65 |
| **M** 7 on Stansbury fault | Upper Range – Constant Tertiary Velocity | Generic Rock | .\h-04c1.m70 |
| **M** 6.5 on East fault | Upper Range – Constant Tertiary Velocity | Generic Rock | .\h-04c1.m65 |
| **M** 7 on Stansbury fault | Upper Range – Increasing Tertiary Velocity | Generic Rock | .\gh-04c1.m70 |
| **M** 6.5 on East fault | Upper Range – Increasing Tertiary Velocity | Generic Rock | .\gh-04c1.m65 |

The files in these subdirectories have a similar naming convention to those in subdirectories **.\wus.m65** and **.\wus.m70**. For example, the calculations for the best estimate profiles and **M** 6.5 scaling of the input motions are located in subdirectories **.\m-04c1.m65** and **.\gm-04c1.m65**. The files are designated M4xx65yy.* with the profile simulation number designated by xx from 01 to 30 and the input time histories designated by the values of yy from 01 to 12.

The computed response spectra for the Skull Valley profiles, the spectral ratios formed by these spectra divided by the corresponding spectra for the WUS profile, and statistics of the spectral ratios are contained in the following subdirectories.

**Response Spectral Ratio Cases**

| Scaling Level for Input Motion | Skull Valley Profile | WUS Profile | Subdirectory for Surface Spectra and Spectral Ratios |
|---|---|---|---|
| **M** 7 on Stansbury fault | Best Estimate – Constant Tertiary Velocity | Generic Rock | .\m-04c1.sp |
| **M** 6.5 on East fault | Best Estimate – Constant Tertiary Velocity | Generic Rock | .\m-04c1.sp |
| **M** 7 on Stansbury fault | Best Estimate – Increasing Tertiary Velocity | Generic Rock | .\gm-04c1.sp |
| **M** 6.5 on East fault | Best Estimate – Increasing Tertiary Velocity | Generic Rock | .\gm-04c1.sp |
| **M** 7 on Stansbury fault | Lower Range – Constant Tertiary Velocity | Generic Rock | .\l-04c1.sp |
| **M** 6.5 on East fault | Lower Range – Constant Tertiary Velocity | Generic Rock | .\l-04c1.sp |
| **M** 7 on Stansbury fault | Lower Range – Increasing Tertiary Velocity | Generic Rock | .\gl-04c1.sp |
| **M** 6.5 on East fault | Lower Range – Increasing Tertiary Velocity | Generic Rock | .\gl-04c1.sp |
| **M** 7 on Stansbury fault | Upper Range – Constant Tertiary Velocity | Generic Rock | .\h-04c1.sp |
| **M** 6.5 on East fault | Upper Range – Constant Tertiary Velocity | Generic Rock | .\h-04c1.sp |
| **M** 7 on Stansbury fault | Upper Range – Increasing Tertiary Velocity | Generic Rock | .\gh-04c1.sp |
| **M** 6.5 on East fault | Upper Range – Increasing Tertiary Velocity | Generic Rock | .\gh-04c1.sp |

The response spectra have the same name as the corresponding site response analysis output files in the associated subdirectories with the same names and have the file extension *.050.

The ratio of the response spectra for the Skull Valley profile surface motion divided by the response spectra for the WUS rock profile is given in files with the same name as the Skull Valley surface motion spectra and the file extension *.RAT. These files contain 5 columns. The first is the period number, the second is the spectral period, the third is the spectral acceleration from the Skull Valley surface motion spectrum, the fourth is the spectral acceleration from the corresponding WUS surface motion spectrum, and the fifth is the ratio of the two. These ratios are computed using program SRATIO.EXE, located in subdirectory **.\programs** on the CD. A brief user guide is located on the accompanying diskette. The input file used to compute the spectral ratios is SR.IN.

Also located in each response spectra subdirectory are statistics of the computed spectral ratios. Those in subdirectory **.\m-04c1.sp** are designated M04C1M65.*, M04C1M70.*, and M04C1.*. The first corresponds to the statistics of the spectral ratios for the best estimate, constant Tertiary velocity profile with **M** 6.5 scaling of the input motions, the second to **M** 7.0 scaling of the input motions, and the third to the combined statistics over both levels of scaling. The input files have the extensions *.INS and the output files the extensions *.STS. The calculations are performed with program SPECSTAT.EXE located on the accompanying diskette, along with a brief user guide. The files contain 11 columns. The second is the spectral period and the fourth is the median (mean log) value. These median values are shown on Figure F-13 in Appendix F of Geomatrix (2001a).

The final composite statistics input and output files are located in subdirectory **.\stat**. Files H-04C1.STS, M-04C1.STS, and L-04C1.STS contain the statistics for the high range, best estimate, and low range site profiles, respectively. File SVOWUS.STS contains the combined statistics over all cases. These are shown on Figure F-14 in Appendix F of Geomatrix (2001a).

## References

Geomatrix Consultants, Inc., 2001a, Fault evaluation study and seismic hazard assessment, Private Fuel Storage Facility, Skull Valley, Utah, Rev 1: report prepared for Stone & Webster Engineering Corporation, 3 vol., March.

Geomatrix Consultants, Inc., 2001b, Soil and foundation parameters for dynamic soil-structure interaction analysis: Geomatrix Calculation 05996.02-G(PO18)-2, Rev 1, prepared for Stone & Webster Engineering Corporation, March.

3. **Results of the soil structure interaction calculations - spectral ratio or free field vs. building structural foundation (top) motion.**

<u>PFS Response</u>
Calculation 0599602-SC-15, Revision 0 has been prepared to compare the free field motion with the Canister Transfer Building foundation motion. This calculation is attached. The plots in the body of the calculation compare the design ground motion spectra (free field) with the spectra of artificial time histories developed to simulate it, and the spectra of the response at the mat. All spectra are for 5% damping. This is done for each of the three soil cases. The EXCEL spreadsheet files containing the digitized data are provided on the diskette included with the calculation.

4. **Confirmation from Bay Geophysical's experts that the new shear wave velocities will not alter their conclusions regarding the shallow seismic reflection profiles.**

<u>PFS Response</u>
Mr. John Clark of Bay Geophysical has reviewed the report submitted by Northland Geophysical, LLC entitled "Downhole Seismic Geophysical Testing", report No. 0599602-G(PO-37)-1, dated January 31, 2001. The results of this review indicate that the velocities reported by Northland are consistent with those quoted in Bay Geophysical's final report for the PFS site.

The following is excerpted from Bay Geophysical's final report on the "High Resolution Seismic Shear Wave Reflection Profiling for the Identification of Faults at the Private Fuel Storage Facility Skull Valley, Utah" dated January, 1999, page 15:

> "Based on stacking velocity measurements (which are accurate to about 20%), and on refraction shear waves seismic measurements independently acquired by Geosphere (1997),[s]hear wave velocities in the very near surface are on the order of 500 feet per second increasing to approximately 700 to 800 feet per second at the Promontory soil and increase to between 1000 and 1500 feet per second in the Quaternary section. Below the Quaternary we estimate the shear wave velocities to be on the order of 2000 feet per second. Based on spectral analysis of the shear wave seismic data, the peak frequency of reflections range from between 100 and 150 hertz. Table 2 summarizes the upper and lower limits of the resolution and detectablity with respect to the seismic data acquired."

The shear wave velocities collected by Northland Geophysical, LLC in December 2000, and January 2001 at borings CTB-5 (OW) and CTB-5A, respectively are in general agreement with the velocities stated in Bay's report (above):

| Depth, ft | Travel Time, ms | Source Offset, ft. | Travel Distance, ft. | Average Velocity, ft/sec | Comment |
|---|---|---|---|---|---|
| 8.3 | 13.8 | 7 | 10.5 | 764 | From CTB-5(OW) |
| 50.8 | 56.7 | 7 | 50.9 | 897 | From CTB-5(OW) |
| 51.5 | 58.5 | 6.2 | 51.9 | 887 | From CTB-5A |
| 64.0 | 65.5 | 6.2 | 64.3 | 982 | From CTB-5A |
| 84.0 | 77.2 | 6.2 | 84.2 | 1091 | From CTB-5A |
| 106.5 | 87.6 | 6.2 | 106.7 | 1218 | From CTB-5A |

The values in the table above were taken graphically from figure 2 of the Northland Geophysical report: "REPORT ON DOWNHOLE GEOPHYSICAL TESTING", dated January 31, 2001.

In summary, the velocities reported from the Northland Geophysical downhole measurements DID NOT alter the conclusions reached by Bay Geophysical in the January 1999 Report with respect to resolution and calculated displacements.

**5. Complete description of the site soil characterization update including:**

- **a. site data,**
- **b. discussion of the site investigation timeline,**
- **c. complete description of the evolution of the site model, noting parameters that have remained constant as well as those that have changes,**
- **d. suite of sensitivity results that show the ramifications of changing from a "soil" model to a "rock" model,**
- **e. sensitivity results to demonstrate the sensitivity (or insensitivity) of the weighting factor (empirical vs. model).**

PFS Response

**5. a.**

A description of the revised site soil characterization with regards to the top layer of eolian silt was provided in item 4 of the PFS letter, Donnell to U.S. NRC, "Response to April 18, 2001 Meeting Issues Regarding PFSF License Application Amendment #22", dated May 1, 2001. Logs and locations for the 16 new test pits were also provided in this same letter as discussed in items 1 and 2 below under Soil Engineering. There have been no other updates to the site soil characterization.

**5. b.**

A description of the site geotechnical investigations is provided in SAR section 2.6.1.5 excerpts of which are provided below:

"Geotechnical boring programs were conducted in 1996 and 1998. The borings drilled in October 1996 were located in the pad emplacement area and along the access road corridor, as shown in Figure 2.6-2. The borings drilled in October and December of 1998 were located in the Canister Transfer Building area, as shown in Figure 2.6-18."

"In April 1999, ConeTec, Inc performed cone penetration tests (CPT) and dilatometer tests (DMT) in the pad emplacement area and the Canister Transfer Building area. The locations of these CPTs and DMTs are presented in Figure 2.6-19. The results from this subsurface investigation are presented in ConeTec (1999)."

In January 2001, 16 test pits were excavated at the site. Location of the test pits, boring logs for the test pits, and an explanation of how the information from the test pits was utilized/interpreted was provided in items 2, 3, and 4 respectively of PFS letter, Donnell to U.S. NRC, "Response to April 18, 2001 Meeting Issues Regarding PFSF License Application Amendment #22", dated May 1, 2001.

A description of the site geophysical surveys is provided in SAR section 2.6.1.10 excerpts of which are provided below:

"Results of seismic refraction and reflection surveys performed at the site in 1996 are found in Appendix 2B. Engineering properties of site materials based on the geophysical investigations are discussed in Section 2.6.1.11. The results of 1998 geophysical surveys (seismic reflection, gravity, and magnetic) are discussed in Geomatrix Consultants, Inc. (2001a) and Bay Geophysical Associates (1999). Seismic cone penetration tests were performed at the locations designated as "SEIS CPT" on Figure 2.6-19. The purpose of these tests was to measure down-hole P and S-wave velocities. The results of these tests are presented in Appendix C of ConeTec (1999), and the average velocities vs depth are shown in Figure 2.6-28."

"Two additional downhole seismic velocity surveys were conducted by Northland Geophysical, LLC on December 21, 2000 and January 19, 2001 in the vicinity of Boring CTB-5(OW) ( Northland Geophysical, LLC, 2001). One downhole seismic velocity survey was conducted from ground surface down to a depth of 50.8 ft in the observation well CTB-5(OW) on December 21, 2000. Another downhole seismic velocity survey was completed from depths of 44.0 to 106.5 ft below ground surface in hollow-stem auger borehole CTB-5A located at 15.5 ft to the northeast of Boring CTB-5(OW) on January 19, 2001. The results of the measurements taken in these downhole seismic velocity surveys have been incorporated into the analyses discussed in Section 2.6.2, Vibratory Ground Motion."

**5. c.**

Evolution of Skull Valley Site Dynamic Model is provided below:

**March 1997**

The initial ground motion assessment for the PSFS site in Skull Valley, Utah was made in March of 1997 (Geomatrix and WLA, 1997). At that time, the available site data consisted of the initial site geotechnical borings reported in Stone & Webster (1997) and the seismic reflection and refraction surveys reported in Geosphere Midwest (1997). The geotechnical data indicated a profile consisting of approximately 30 feet of silty clay, clay, and clayey silt, underlain by approximately 25 feet of dense sands, underlain by very hard silts to a maximum borehole depth of 100 feet. The shear wave refraction survey indicated two layers, a surface layer with shear wave velocities in the range of 700 to 790 ft/sec and a layer at a depth of about 45 to 50 feet with shear wave velocities in the range of 1,700 to 2,400 ft/sec. The maximum depth of penetration of the shear wave refraction survey was estimated to be 80 to 90 feet. The assessment at that time was that the site might exhibit the dynamic characteristics of both rock and soil sites during earthquake shaking. Thus, an envelope of ground motions predicted by California empirical ground motion models for soil and for rock site conditions was used to develop the initial design ground motion spectra.

**February 1999**

Geomatrix (1999a) conducted a probabilistic seismic hazard analysis (PSHA) for the site.As part of that study, ground motion models for the site were developed by comparing the response of the site profile to the response of a generic California deep soil profile (Appendix F of Geomatrix, 1999a). The relative site response analysis was conducted by developing shear wave velocity profiles for a generic California deep soil site and the Skull Valley site. The additional site data available at that time were more extensive soil geotechnical borings, a series of geological borings conducted to evaluate fault offset, and a high-resolution shear-wave reflection survey (Bay Geophysical, 1999). The high-resolution reflection survey was used to assess the offset of two principal reflectors, the Promontory soil located at a depth of approximately 45 feet and the unconformity at the top of the Tertiary Salt Lake Group sediments. Based on the geologic data for the site, the average depths to these layers were assessed to be approximately 45 and 85 feet. Bay Geophysical reported average velocities of 800 ft/sec and 1,100 ft/sec for the soils above the two marker horizons. These average velocities are consistent with those obtained by Geosphere Midwest (1997). The velocity of 800 ft/sec for the materials above the Promontory soil is at the upper limit of the velocity range of 700 to 790 ft/sec reported by Geosphere Midwest (1997) for the surficial layer. Using the layer velocities from Geosphere Midwest (1997), the average velocity to the Quaternary/Tertiary boundary is:

$$\overline{V}_S = \frac{85(\text{ft})}{45(\text{ft})/750(\text{ft/sec}) + 40(\text{ft})/2000(\text{ft/sec})} = 1063(\text{ft/sec})$$

The Skull Valley site velocity model used in Appendix F of Geomatrix (1999a) is tabulated below.

**Shear Wave Velocity Profile Used in Geomatrix (1999a)**

| Layer Thickness (ft) | Depth to Layer Base (ft) | Shear Wave Velocity (ft/sec) | Unit Weight (pcf) |
|---|---|---|---|
| 45 | 45 | 750 | 131 |
| 40 | 85 | 2,000 | 131 |
| 515 | 600 | 4,511 | 145 |
| 3,993 | 4,593 | 6,398 | 156 |
| 5,249 | 9,843 | 11,122 | 156 |

The velocities for the first two layers represent the average velocities reported by Geosphere Midwest (1997). The velocity for the third layer is based on reported velocities in the range of 1.0 to 1.75 km/sec reported for the Tertiary Salt Lake Group sediments in the Salt Lake Valley. The thickness and velocity for the bottom two layers were based on the crustal velocity model used by the University of Utah Seismographic Station to locate earthquakes in north-central Utah. Two sets of shear modulus reduction and damping relationships were used for both the California and Skull Valley soils. These relationships were developed for generic California soils by Silva et al. (1998). One set was based on the EPRI (1993) generic set of relationships for granular alluvial soils. A second set with less damping and modulus reduction was found by Silva et al. (1998) to produce better agreement with soil response in southern California. Differences in the unit weight of the soils between California and Skull Valley were not incorporated into the assessment because the differences in shear wave velocity were judged to be the controlling parameter for relative site response.

**June 1999**

Geomatrix (1999b) developed revised soil properties for soil-structure interaction analysis for input ground motions based on the 1,000-year return period surface motions defined by the PSHA (Geomatrix, 1999a). Seismic cone penetration test data obtained in April 1999 within the upper 30 feet of the site soils were used to refine the shallow shear-wave velocity profile for the site. The following table lists the best estimate velocity profile developed in Geomatrix (1999b).

Best Estimate Profile from Geomatrix (1999b)

| Layer | Depth to Layer Base (ft) | h (ft) | Vs (ft/sec) | Unit Weight (pcf) | Layer Travel Time (sec) | Cumulative Travel Time (sec) | Average Vs (ft/sec) |
|---|---|---|---|---|---|---|---|
| 1 | 10 | 10 | 540 | 85 | 0.01852 | 0.01852 | 540 |
| 2 | 12 | 2 | 720 | 92.5 | 0.00278 | 0.02130 | 563 |
| 3 | 25 | 13 | 865 | 92.5 | 0.01503 | 0.03633 | 688 |
| 4 | 45 | 20 | 1015 | 115 | 0.01970 | 0.05603 | 803 |
| 5 | 85 | 40 | 2000 | 120 | 0.02000 | 0.07603 | 1118 |
| 6 | 125 | 40 | 4511.155 | 135 | 0.00887 | 0.08490 | 1472 |
| 7 | 625 | 500 | 4511.155 | 145 | 0.11084 | 0.19573 | 3193 |
| | half space | | 6397.638 | 170 | | | |

The right-hand column of the above table lists the average shear wave velocity from the surface to the base of the various layers. As indicated, the average velocities for the top 45 feet and top 85 feet are 803 ft/sec and 1,118 ft/sec respectively. These values are consistent with those obtained by Bay Geophysical (1999) and Geosphere Midwest (1997).

The updated dynamic properties included unit weights based on the results of laboratory testing performed to date. Three sets of shear modulus reduction and damping relationships were used; the two sets used in Geomatrix (1999a) and a third set based on the published relationships of Vucetic and Dobry (1991) for clayey soils.

**March 2001**

Geomatrix (2001a and 2001b) revised the site dynamic soil properties and site ground motions. Geomatrix (2001b) developed the following updated best estimate velocity profile.

**Best Estimate Velocity Profile from Geomatrix (2001b)**

| Layer | Thickness (ft) | Layer Vs (fps) | Total Depth (ft) | Unit Weight (pcf) | Layer Travel Time (sec) | Total Travel Time (sec) | Average Velocity (fps) |
|---|---|---|---|---|---|---|---|
| 1 | 5 | 560 | 5 | 80 | 0.008929 | 0.008929 | 560 |
| 2 | 5 | 528 | 10 | 80 | 0.009470 | 0.018398 | 544 |
| 3 | 2 | 727 | 12 | 80 | 0.002751 | 0.021149 | 567 |
| 4 | 6 | 854 | 18 | 100 | 0.007026 | 0.028175 | 639 |
| 5 | 8 | 871 | 26 | 94 | 0.009185 | 0.037360 | 696 |
| 6 | 9 | 1,022 | 35 | 115 | 0.008806 | 0.046166 | 758 |
| 7 | 15 | 1,190 | 50 | 115 | 0.012605 | 0.058771 | 851 |
| 8 | 40 | 1,800 | 90 | 120 | 0.022222 | 0.080993 | 1111 |
| 9 | 35 | 2,900 | 125 | 135 | 0.012069 | 0.093062 | 1343 |
| 10 | 575 | 2,900 | 700 | 140 | 0.198276 | 0.291338 | 2403 |

The changes in the velocity profile from June 1999 to March 2001 result from the following:

- The soil layer boundaries were adjusted slightly to reflect the finalized cross sections presented in PFS (2000). Layers 1, 3, and 4 of the June 1999 model were divided into two layers reflecting differences in soil type. The average depth to the Promontory soil and the Quaternary/Tertiary boundary were revised from 45 and 85 feet to 50 and 90 feet, respectively.
- Downhole shear wave velocity measurements in boring CTB-5(OW) and CTB-5A provide additional confirmatory velocity data for the top 30 feet and additional interval velocity measurements extending to a depth of 105 feet.
- In Geomatrix (1999b), the average shear wave velocity in each layer was computed by a direct average of all of the velocity measurements obtained in that layer. In Geomatrix (2001b), velocity profiles were developed for each of the 16 cone penetration tests and the one downhole velocity boring by computing the harmonic mean of all velocity measurements within a given layer. The overall mean velocities listed in the above table were then computed by averaging the resulting layer

velocities across the 17 velocity profiles. This approach was used to develop a statistical model of the velocity variability across the site. As a result, the layer velocities within the top 30 feet differ slightly from those given in Geomatrix (1999b).

- Direct measurement of shear wave velocity in the upper portion of the Tertiary Salt Lake Group sediments indicated a velocity slightly lower than the lower limit of the range used for this unit in Geomatrix (1999a) and (1999b).

The right hand column of the above table again lists the average shear wave velocity from the surface to the base of the various layers. The average velocity from the surface to the Promontory soil layer at 50 feet is 851 ft/sec and from the surface to the Quaternary/Tertiary boundary at 90 feet is 1,111 ft/sec. These values remain close to those obtained in earlier studies.

The updated velocity model was used to revise the site design basis ground motions in Geomatrix (2001a). In addition, the dynamic properties were revised to incorporate all of the compiled unit weight data and the results of resonant column tests performed on samples of the clayey soils from depths of 8 and 20 feet. These tests (conducted in the early summer of 1999) indicate low levels of damping and shear modulus reduction in the shallow soils. Since these tests demonstrate that these soils have lower damping and less modulus reduction with increasing strain than previously assumed, it was assumed that the underlying soils also show low damping and low levels of modulus reduction. As a result, in Geomatrix (2001a), the more linear, lower damping southern California set of modulus reduction and damping relationships (Silva et al., 1998) were used for the soils at Skull Valley for layers deeper than those tested. These relationships are shown on Figure F-11 of Geomatrix (2001, Appendix F).

In summary, the evolution of the dynamic soil model for the Skull Valley PFSF site has primarily been a process of adding detail to the basic picture developed in March of 1997 and February of 1999. The information available in March of 1997 indicated two principal layers, a surface layer with a velocity of about 750 ft/sec and a layer at a depth of approximately 45 feet with a velocity of 2,000 ft/sec. The model developed in January and February of 1999 added a third layer, the Tertiary sediments located at a depth of approximately 85 feet. The average velocities of this basic model are consistent with those derived from the detailed velocity data obtained after February 1999.

**References**

Bay Geophysical, Inc., 1999, High-resolution seismic shear-wave reflection profiling for the identification of faults at the Private Fuel Storage Facility, Skull Valley, Utah.

Electric Power Research Institute (EPRI), 1993, Guidelines for determining design basis ground motions: EPRI TR-102293, v. 1-5.

Geomatrix Consultants, Inc. and William Lettis and Associates, Inc., 1997, Deterministic earthquake ground motion analysis: report prepared for Stone & Webster Engineering Corporation, CS-028233, P.O. 059901-005, March.

Geomatrix Consultants, Inc., 1999a, Fault evaluation study and seismic hazard assessment, Private Fuel Storage Facility, Skull Valley, Utah: report prepared for Stone & Webster Engineering Corporation, 3 vol., February.

Geomatrix Consultants, Inc., 1999b, Soil and foundation parameters for dynamic soil-structure interaction analysis: Geomatrix Calculation 05996.02-G(PO18)-1, Rev 0, prepared for Stone & Webster Engineering Corporation, June.

Geomatrix Consultants, Inc., 2001a, Fault evaluation study and seismic hazard assessment, Private Fuel Storage Facility, Skull Valley, Utah, Rev 1: report prepared for Stone & Webster Engineering Corporation, 3 vol., March.

Geomatrix Consultants, Inc., 2001b, Soil and foundation parameters for dynamic soil-structure interaction analysis: Geomatrix Calculation 05996.02-G(PO18)-2, Rev 1, prepared for Stone & Webster Engineering Corporation, March.

Geosphere Midwest, 1997, Seismic survey of the Private Fuel Storage Facility, Skull Valley, Utah: report prepared for Stone & Webster Engineering Corporation.

Silva, W.C., Abrahamson, N., Toro, G., and Costantino, C., 1998, Description and validation of the stochastic ground motion model: Report submitted to Brookhaven National Laboratory, Associated Universities, Inc., New York.

Private Fuel Storage Limited Liability Company (PFS), 2000, Safety Analysis Report for Private Fuel Storage Facility, Revision 8, Docket No. 72-22.

Stone & Webster Engineering Corporation, 1997, Geotechnical Data Report No. 05996.01-G(B)-Z, Rev 0.

Vucetic, M., and Dobry, R., 1991, Effect of soil plasticity on cyclic response, Proc ASCE, Journal of Geotechnical Engineering, Vol. 117, No. 1, January, pp 89-107.

**5. d.**

The change from soil to rock as the reference site condition was made because a review of the updated soil data indicates that the site velocity profile is closer to that for a generic rock site representative of the data used in the California rock empirical models and because use of rock relationships does not require the assumption that the weights developed by the Yucca Mountain Ground Motion Expert Panel for rock attenuation relationships also apply to the companion soil attenuation relationships.

The effect of using rock attenuation relationships rather than soil attenuation relationships is illustrated by the following sensitivity analysis. Seismic hazard analyses were performed using the suite of rock and soil attenuation relationships without modification for local site conditions. The hazard results were used to construct 2,000-year return period response spectra for California rock and deep soil site conditions. These spectra are compared on Figure SHA05d-1. The change from soil to rock site motions results in a slight increase in the high-frequency portion of the spectra and a decrease in the low frequency portion of the spectra.

Figure SHA05d-2 compares the PFS site-specific design response spectra developed in 1999 based on California deep soil attenuation relationships to the revised design response spectra developed in 2001 based on California rock attenuation relationships. In both the horizontal and vertical directions, the revised design spectra are significantly higher than the 1999 design spectra for spectral periods less than about 0.3 seconds (frequencies greater than about 3 Hz). At longer periods the revised horizontal spectrum is somewhat lower than the 1999 horizontal spectrum while the revised vertical spectrum is slightly higher than the 1999 vertical spectrum. As indicated in Figure SHA05d-1, the difference between the ground motions for generic California rock and soil site conditions is small at short periods, becoming larger for horizontal motions at spectral periods greater than about 0.3 seconds. These comparisons suggest that the revised site-adjustment factors for the PFS site have a measurable impact in yielding higher horizontal accelerations for the short periods (high frequencies). This impact is significantly greater than what the shift from California soil to rock attenuation relationships would in itself produce.

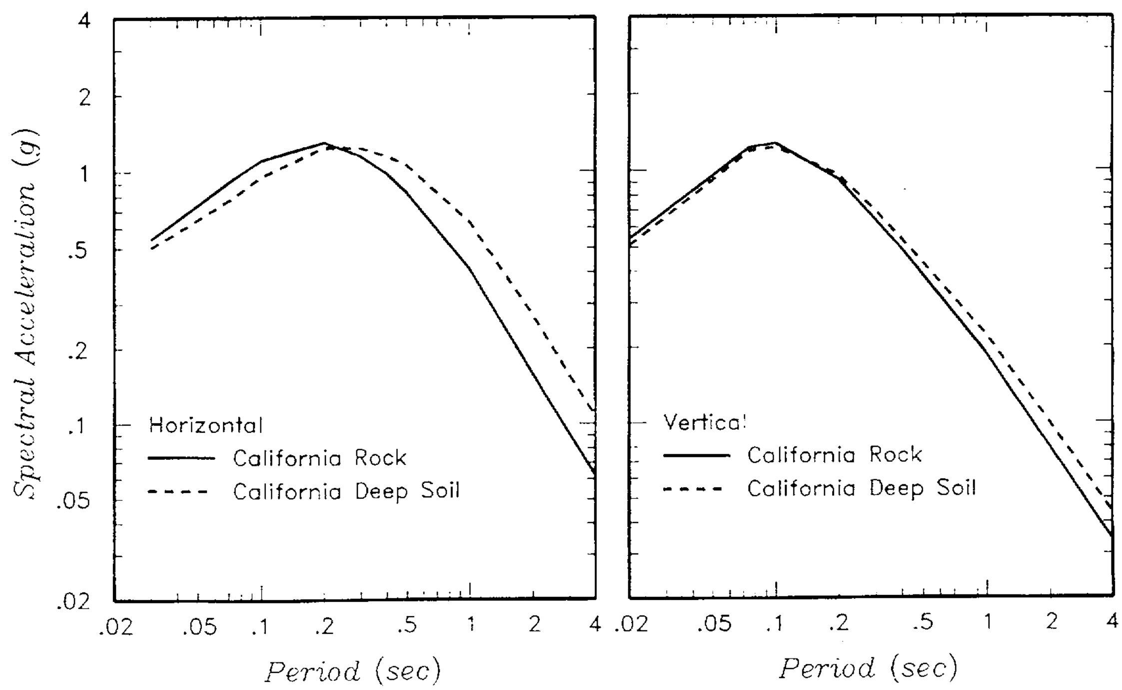


Figure SHA05d-1 Comparison of 2000-year equal hazard response spectra (5% damping) for generic California rock and deep soil site conditions (no site adjustment factors applied).

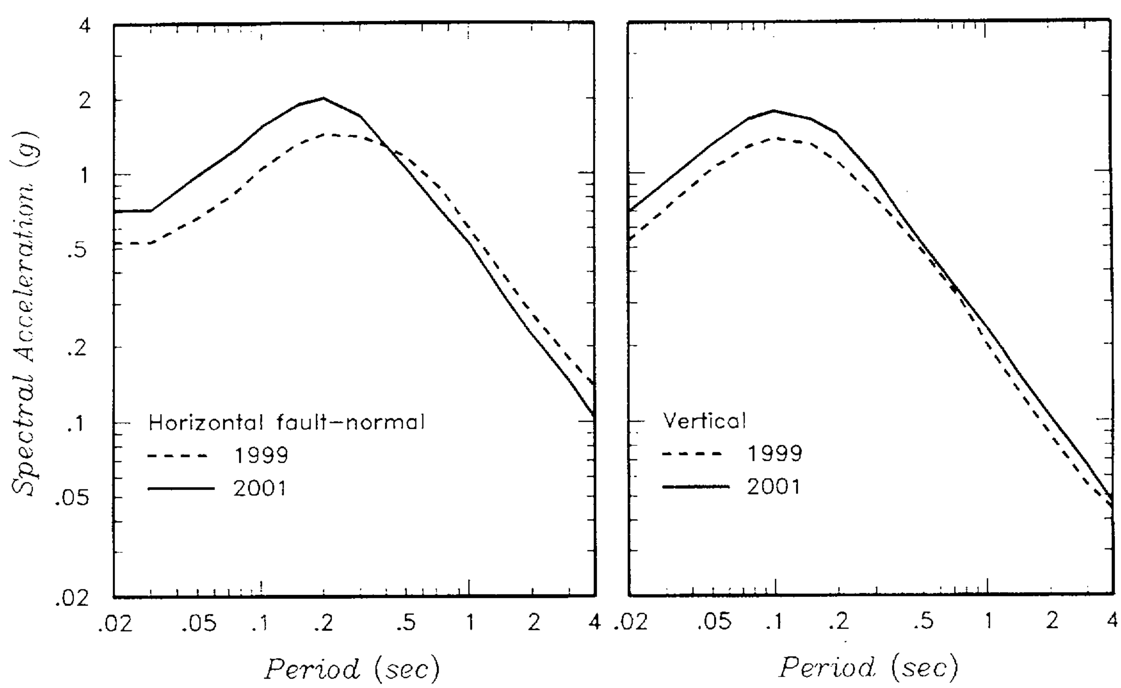


Figure SHA05d-2 Comparison of the 1999 design spectra based on California deep soil attenuation relationships and the 2001 design spectra based on California rock attenuation relationships

**5. e.**

As described in Appendix F of the revised seismic hazard analysis report, two alternative approaches were used to develop site adjustment factors to modify California rock site attenuation relationships to the site conditions at the Skull Valley PFSF site. One approach was based on relative site response analyses (designated SN in this response) and one was based on empirical strong motion data (designated SE in this response). The site hazard was computed assigning a weight of 0.67 to the site response approach and a weight of 0.33 to the empirical approach. To examine the sensitivity of the results to these weighting factors, the hazard analyses were repeated using a range of weighting alternatives. The following table lists the resulting 2,000-year return period equal-hazard response spectral accelerations (5% damping). These spectra are shown on Figure SHA05e-1.

**Effect of Weights Assigned to Site Adjustment Factors On 2000-year Return Period Spectra**

| Period (sec) | Weights on Empirical (SE) and Site Response (SN) Site Adjustment Factors | | | |
|---|---|---|---|---|
| | SE 1.0 SN 0.0 | SE 0.5 SN 0.5 | SE 0.33 SN 0.67 | SE 0.0 SN 1.0 |
| **Horizontal Spectral Acceleration (g)** | | | | |
| 0.03 | 0.554 | 0.670 | 0.708 | 0.765 |
| 0.075 | 0.999 | 1.185 | 1.247 | 1.366 |
| 0.1 | 1.159 | 1.454 | 1.543 | 1.694 |
| 0.2 | 1.363 | 1.832 | 1.986 | 2.229 |
| 0.3 | 1.218 | 1.569 | 1.679 | 1.873 |
| 0.4 | 1.060 | 1.226 | 1.279 | 1.375 |
| 0.5 | 0.965 | 1.026 | 1.045 | 1.081 |
| 1 | 0.475 | 0.475 | 0.475 | 0.475 |
| 2 | 0.169 | 0.165 | 0.164 | 0.162 |
| 4 | 0.0658 | 0.0664 | 0.0667 | 0.0671 |
| **Vertical Spectral Acceleration (g)** | | | | |
| 0.02 | 0.540 | 0.655 | 0.696 | 0.760 |
| 0.075 | 1.265 | 1.546 | 1.629 | 1.792 |
| 0.1 | 1.324 | 1.647 | 1.754 | 1.961 |
| 0.2 | 1.056 | 1.336 | 1.428 | 1.590 |
| 0.3 | 0.777 | 0.912 | 0.960 | 1.044 |
| 0.4 | 0.621 | 0.652 | 0.663 | 0.684 |
| 0.5 | 0.519 | 0.512 | 0.509 | 0.505 |
| 1 | 0.237 | 0.227 | 0.223 | 0.217 |
| 2 | 0.0990 | 0.0906 | 0.0878 | 0.0823 |
| 4 | 0.0363 | 0.0366 | 0.0368 | 0.0370 |

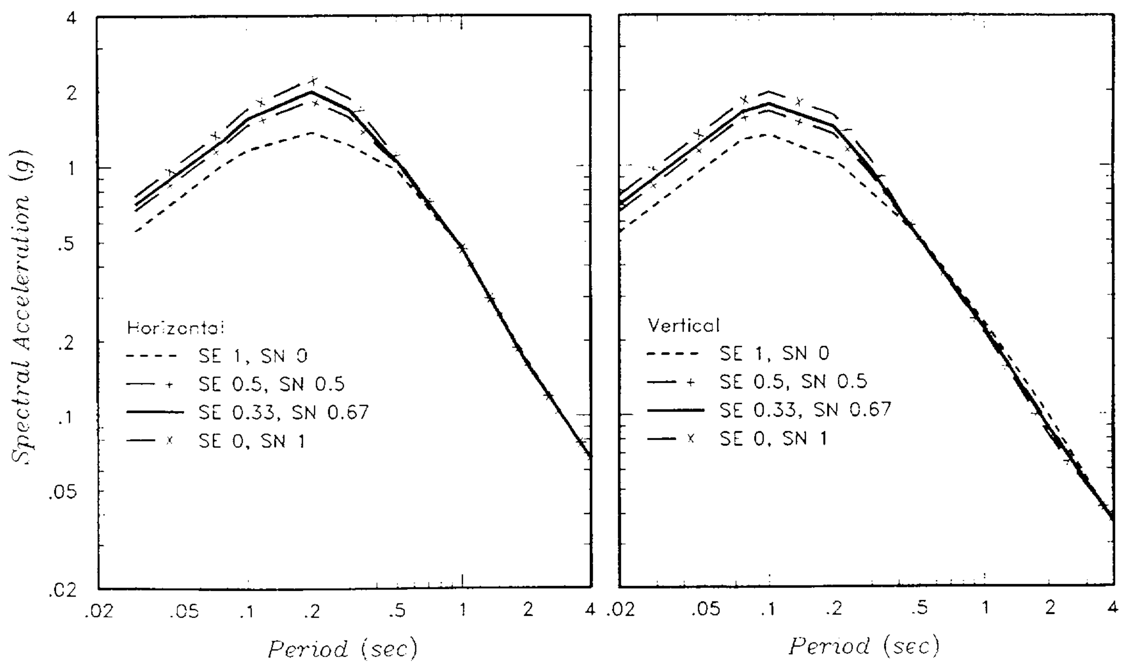


Figure SHA05e-1 Effect of weights assigned to site response factors on 2000-year equal hazard response spectra (5% damping)

The design ground motion spectra are sensitive to the relative weights assigned to the empirical and site response site adjustment factors for spectral periods less than about 0.5 seconds (spectral frequencies greater than about 2 Hz).

**6. Complete revised hazard analysis report (or at least a complete section 6).**

PFS Response
A complete page replacement revision to the Fault Evaluation Study and Seismic Hazard Assessment was provided to the NRC with PFS letter, Donnell to U.S. NRC, "Calculation Package and Report Submittal", dated April 5, 2001. **As requested, a complete copy of Section 6 is attached. Please note that only pages 88 through 93A were affected by Revision 1.**

**7. Well data for soil below 30 ft.**

PFS Response
The following data have been obtained for the sediments below a depth of 30 feet.

**Soil Borings**
Four borings (A-1, D-4, CTB-1, and CTB-5(OW)) were drilled to depths in excess of 100 feet and 25 borings were drilled to depths of approximately 50 to 75 feet (see Figure 2.6-

5 sheets 1 through 14 and Figures 2.6-21 through 2.6-23 of the SAR (PFS, 2001). Logs of these borings are included in Attachment 1 of Appendix 2A of the SAR. Standard penetration test (SPT) blow counts were obtained, generally, at 5-foot intervals in these borings. The data from these borings show a consistent picture across the site. Between depths of 30 and 50 feet, the sediments consists of dense sands with blow counts generally in the range of 70 to over 100 blows/foot. A layer of sandy gravel is often encountered at a depth of 50 ± 5 feet. The top of this layer is marked by the Promontory soil, which represents an erosional unconformity. Below the sandy gravel, the sediments are very hard silts with some very dense sands. The SPT blow counts in these materials are generally well in excess of 100 blows/foot. Below a depth of approximately 90 feet, an ash marker horizon is encountered that indicates penetration into the Tertiary sediments of the Salt Lake Group.

**Seismic Survey Data**
Two seismic survey studies were performed at the site. Geosphere Midwest (1997) performed both seismic refraction and seismic reflection surveys across the site. The shear wave refraction survey indicated two layers, a surface layer with shear wave velocities in the range of 700 to 790 ft/sec and a layer at depth of about 45 to 50 feet with shear wave velocities in the range of 1,700 to 2,400 ft/sec. The maximum depth of penetration of the shear wave refraction survey was estimated to be 80 to 90 feet. The seismic refraction survey indicated that the depth to the basement rocks was in the range of 600 to 800 feet. Bay Geophysical (1999) conducted a high-resolution shear-wave reflection survey. The results of this survey were used to assess the offset of two principal reflectors, the Promontory soil located at a depth of approximately 50 feet and the unconformity at the top of the Tertiary Salt Lake Group sediments at a depth of approximately 90 feet. Bay Geophysical (1999) reported average velocities of 800 ft/sec and 1,100 ft/sec for the soils above the two marker horizons. As discussed in the response to 5c, these values are consistent with those reported by Geosphere Midwest (1997). The depths and offsets of the shallow marker horizons were calibrated using geologic borings (Geomatrix, 1999a).

**Downhole Velocity Data**
Northland Geophysical (2001) conducted confirmatory downhole shear and compression wave velocity measurements in borings CTB-5(OW) and CTB-5A to a maximum depth of 106.5 feet. The data from boring CTB-5(OW) were measured in a PVC-cased boring to a maximum depth of 50.8 feet. The shear and compression wave velocity data from this boring are very consistent with the velocities obtained from the seismic cone penetration tests in the depth range of 0 to 30 feet (Geomatrix, 2001b). The data from boring CTB-5(OW) show a trend of gradually increasing velocity in the depth range of 30 to 50 feet. Boring CTB-5A was drilled adjacent to CTB-5(OW) using a hollow-stem auger and shear wave velocities were measured with the geophones clamped within the auger stem. The measurements were initiated at the bottom of the hole at a depth of 106.5 feet and continued upward until the data quality began to deteriorate above 44 feet. As a result, there was a very limited range of overlap between the measurements in borings CTB-5(OW) and CTB-5A. The velocities in the two borings were similar (see Figures 3 and 4 of Northland Geophysical, 2001). The data from boring CTB-5A show

fairly uniform velocities for the depth range of 55 to 95 feet and an increase in velocity below 95 feet.

**References**

Bay Geophysical, Inc., 1999, High-resolution seismic shear-wave reflection profiling for the identification of faults at the Private Fuel Storage Facility, Skull Valley, Utah.

Geomatrix Consultants, Inc., 1999a, Fault evaluation study and seismic hazard assessment, Private Fuel Storage Facility, Skull Valley, Utah: report prepared for Stone & Webster Engineering Corporation, 3 vol., February.

Geomatrix Consultants, Inc., 2001b, Soil and foundation parameters for dynamic soil-structure interaction analysis: Geomatrix Calculation 05996.02-G(PO18)-2, Rev 1, prepared for Stone & Webster Engineering Corporation, March.

Geosphere Midwest, 1997, Seismic survey of the Private Fuel Storage Facility, Skull Valley, Utah: report prepared for Stone & Webster Engineering Corporation.

Northland Geophysical, LLC, 2001, Results of downhole seismic geophysical testing, Private Fuel Storage Facility, Skull Valley Utah: Report prepared for Stone & Webster, ESSOW No. 05996.02-G011 (Rev. 1), January 31.

Private Fuel Storage Limited Liability Company (PFS), 2001, Safety Analysis Report for Private Fuel Storage Facility, Revision 21, Docket No. 72-22, La Crosse, WI.

8. **More site specific data (i.e., beyond the one existing deep well) for the soil between 30 ft and the Tertiary strata or provide an analysis that shows that the applicant has captured the uncertainty of the soil properties sufficiently such that any new information will not again significantly change the ground motions (i.e., sensitivity study of the site response model that would incorporate the variability of the soil parameters expected for this site).**

PFS Response

As discussed in the response to Number 7, the data for the soils below a depth of 30 feet indicate that they consist of dense sands and hard silts. Geomatrix (2001a and 2001b) developed the best estimate velocity profiles for the site, as shown in Table SHA08-1. The two alternative profiles shown in that table (constant Tertiary velocity and increasing Tertiary velocity) reflect the uncertainty as to whether or not the velocity in the deeper Tertiary sediments remains approximately constant with depth or increases with depth at a rate that denotes a significant gradient.

The velocities shown in the depth range of 30 to 125 feet in the best estimate profiles shown in Table SHA08-1 were based primarily on the downhole velocity measurements in boring CTB-5(OW) and CTB-5A reported in Northland Geophysical (2001). The velocities are in agreement with the layer average velocities obtained from the two geophysical surveys(Geosphere Midwest, 1997, and Bay Geophysical, 1999).

The relative site response analyses presented in Appendix F of Geomatrix (2001a) used the three velocity profiles for the Skull Valley soils listed in Table SHA08-2. The best estimate profile includes replacement of the upper 5 feet of soils by a soil-cement layer. The plus and minus on the layer depths reflects the randomness in layer boundaries included in the simulations of the site profiles described in Appendix F of Geomatrix (2001a). Lower and upper range profiles were also analyzed. The velocities assigned to these profiles represent the assessed uncertainty in the average layer velocities for the site (the uncertainty in the layer velocities for any single profile is modeled through the simulation process). For layers 1 through 6, the range in average velocities reflects the statistics of the seismic cone and downhole velocity measurements in the upper 30 feet. At 30 feet, the uncertainty in the average velocity is approximately ±5%. This uncertainty was applied to average velocity in layer 7. The uncertainty in the average velocity in layer 8 (depth range of 50 to 90 feet) was assigned a value of ±0.1 times the best estimate velocity, reflecting the low variability observed in the velocities of the shallower soils. The uncertainty in the average velocity of the soil-cement and the Tertiary sediments was assigned a value of $\pm\sqrt{1.5}$ times the best estimate velocity. This value is the minimum variability in shear wave velocity recommended by ASCE (1986) for soil-structure interaction and is likely to be a conservative value based on the data obtained at the site for other sediment layers.

The velocity profiles developed in Geomatrix (2001a) assumed a constant average velocity within the major soil layers below a 30-foot depth, with the velocity in each layer increasing with depth. In the following section, several sensitivity analyses are presented showing the effect of alternative interpretations of the variation in shear wave velocity with depth below 30 feet. The alternative average velocity profiles are tabulated in Table SHA08-3 and shown on Figures SHA08-2, SHA08-3, and SHA08-4.

Alternative Profile 1 replaces the constant average velocities in the 35- to 50-foot and 50- to 90-foot depth ranges with gradually increasing average velocities with depth. The change from the base case profile is similar to the change that resulted from the detailed seismic cone data for the upper 30 feet compared to the constant velocity of 750 ft/sec used in Geomatrix (1999a). Table SHA08-4 presents the variation in average shear wave velocity from the surface to various depths within the profile. The resulting values for the depth ranges 0-50 feet and 0-90 feet are consistent with those for the best estimate profile and with the original geophysical surveys. Alternative Profile 2 is a variation on Alternative 1 in which a velocity gradient is introduced for the transition into the Tertiary sediments at a depth of 90 feet.

A possible interpretation of the variation in shear wave velocity with depth obtained by Northland Geophysical (2001) is that there are several higher velocity layers present where there is transition into older units. There is a suggestion of this at depths between 35 and 40 feet and between 50 and 60 feet. The blow count data for the site also indicate the presence of very hard layers underlain by layers with somewhat lower blow counts. Alternative Profile 3 modifies the base case profile to introduce high velocity layers at depths between 35 and 40 feet and between 50 and 60 feet. Table SHA08-5 presents the variation in average shear wave velocity from the surface to various depths within the profile. The resulting values for the depth ranges 0-50 feet and 0-90 feet remain consistent with those for the best estimate profile. Alternative Profile 4 is a variation on Alternative 3 in which a high velocity layer in introduced at the transition into the Tertiary sediments. The possible presence of such a layer is suggested by the results presented on Figure 4 of Northland Geophysical (2001).

Alternative Profiles 1 through 4 are used in sensitivity analyses to assess the impact of alternative interpretations of the detailed velocity variation with depth below 30 feet on the relative site response analysis. For each average profile, 30 random velocity profiles were generated using the approach outlined in Appendix F of Geomatrix (2001a). These profiles are shown on Figures SHA08-5 through SHA08-9. A fifth alternative profile was defined in order to compare the importance of the velocity profile details in the upper 30 feet to site response to the importance of the velocity profile details below 30 feet. (This alternative used the same velocity profile as the base case for depths below 50 feet.) Alternative Profile 5 replaces the shallow portion of the base case profile with a uniform velocity of 850 ft/sec, the average velocity for the top 50 feet of the base case profile. Figure SHA08-10 shows the 30 random profiles generated using the Alternative Profile 5 average velocity.

For each of the six analysis cases, the base case and the five alternative profiles, 240 site response analyses were conducted using the 24 input time histories developed in Appendix F of Geomatrix (2001a). The response spectra for the computed surface motions were divided by the corresponding spectra for a WUS rock site. The median (mean-log) of the 240 spectral ratios was then computed. Figure SHA08-11 compares these spectral ratios. The spectral ratios obtained using Alternative Profiles 1 through 4 show only minor differences from the base case results. Alternative 5 results in a large change in the computed spectral ratios. These results indicate that the details of the velocity profile in the upper 30 feet of soil (where the lowest velocities are experienced) are much more significant to the computation of relative site response than those below 30 feet (where the higher velocities are experienced), and that for levels below 50 feet the details of the velocity distribution assumed have little effect on the predicted relative site response.

**References**

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Northland Geophysical, LLC, 2001, Results of downhole seismic geophysical testing, Private Fuel Storage Facility, Skull Valley Utah: Report prepared for Stone & Webster, ESSOW No. 05996.02-G011 (Rev. 1), January 31.

**Table SHA08-1**
**Best Estimate Velocity Profile from Geomatrix (2001a)**
**Constant Tertiary Velocity**

| Layer | Thickness (ft) | Layer Vs (fps) | Total Depth (ft) | Unit Weight (pcf) | Layer Travel Time (sec) | Total Travel Time (sec) | Average Velocity (fps) |
|---|---|---|---|---|---|---|---|
| 1 | 5 | 560 | 5 | 80 | 0.008929 | 0.008929 | 560 |
| 2 | 5 | 528 | 10 | 80 | 0.009470 | 0.018398 | 544 |
| 3 | 2 | 727 | 12 | 80 | 0.002751 | 0.021149 | 567 |
| 4 | 6 | 854 | 18 | 100 | 0.007026 | 0.028175 | 639 |
| 5 | 8 | 871 | 26 | 94 | 0.009185 | 0.037360 | 696 |
| 6 | 9 | 1,022 | 35 | 115 | 0.008806 | 0.046166 | 758 |
| 7 | 15 | 1,190 | 50 | 115 | 0.012605 | 0.058771 | 851 |
| 8 | 40 | 1,800 | 90 | 120 | 0.022222 | 0.080993 | 1111 |
| 9 | 35 | 2,900 | 125 | 135 | 0.012069 | 0.093062 | 1343 |
| 10 | 575 | 2,900 | 700 | 140 | 0.198276 | 0.291338 | 2403 |

**Increasing Tertiary Velocity**

| Layer | Thickness (ft) | Layer Vs (fps) | Total Depth (ft) | Unit Weight (pcf) | Layer Travel Time (sec) | Total Travel Time (sec) | Average Velocity (fps) |
|---|---|---|---|---|---|---|---|
| 1 | 5 | 560 | 5 | 80 | 0.008929 | 0.008929 | 560 |
| 2 | 5 | 528 | 10 | 80 | 0.009470 | 0.018398 | 544 |
| 3 | 2 | 727 | 12 | 80 | 0.002751 | 0.021149 | 567 |
| 4 | 6 | 854 | 18 | 100 | 0.007026 | 0.028175 | 639 |
| 5 | 8 | 871 | 26 | 94 | 0.009185 | 0.037360 | 696 |
| 6 | 9 | 1,022 | 35 | 115 | 0.008806 | 0.046166 | 758 |
| 7 | 15 | 1,190 | 50 | 115 | 0.012605 | 0.058771 | 851 |
| 8 | 40 | 1,800 | 90 | 120 | 0.022222 | 0.080993 | 1111 |
| 9 | 35 | 2,900 | 125 | 135 | 0.012069 | 0.093062 | 1343 |
| 10 | 175 | 2,900 | 300 | 140 | 0.060345 | 0.153407 | 1956 |
| 11 | 200 | 4,000 | 500 | 140 | 0.050000 | 0.203407 | 2458 |
| 12 | 200 | 5,000 | 700 | 140 | 0.040000 | 0.133062 | 5261 |

**Table SHA08-2**
**Best Estimate and Upper and Lower Range Velocity Profiles for Skull Valley PFSF Site Response Analyses (Geomatrix, 2001a)**

| Layer | Depth to Base of Layer (ft) | Lower Range Average Layer Shear Wave Velocity (fps) | Best Estimate Average Layer Shear Wave Velocity (fps) | Upper Range Average Layer Shear Wave Velocity (fps) |
|---|---|---|---|---|
| 1 | 5±2 | 1,225 | 1,500 | 1,837 |
| 2 | 10±1 | 500 | 528 | 556 |
| 3 | 12±1 | 687 | 727 | 767 |
| 4 | 18±1 | 832 | 854 | 876 |
| 5 | 26±1 | 858 | 871 | 884 |
| 6 | 35±1 | 974 | 1,022 | 1,070 |
| 7 | 50±5 | 1,034 | 1,190 | 1,248 |
| 8a | 70±5 | 1,620 | 1,800 | 1,980 |
| 8b | 90±5 | 1,620 | 1,800 | 1,980 |
| 9 | 125 | 2,368 | 2,900 | 3,552 |
| 10 | 300 | 2,368 | 2,900 | 3,552 |

Table SHA08-3
Alternative Average Velocity Profiles Used in Sensitivity Analyses

| Layer | Depth to Base of Layer (ft) | Base Case Average Layer Shear Wave Velocity (fps) | Alternative 1 Average Layer Shear Wave Velocity (fps) | Alternative 2 Average Layer Shear Wave Velocity (fps) | Alternative 3 Average Layer Shear Wave Velocity (fps) | Alternative 4 Average Layer Shear Wave Velocity (fps) | Alternative 5 Average Layer Shear Wave Velocity (fps) |
|---|---|---|---|---|---|---|---|
| 1 | 5±2 | 1,500 | 1,500 | 1,500 | 1,500 | 1,500 | 1,500 |
| 2 | 10±1 | 528 | 528 | 528 | 528 | 528 | 850 |
| 3 | 12±1 | 727 | 727 | 727 | 727 | 727 | 850 |
| 4 | 18±1 | 854 | 854 | 854 | 854 | 854 | 850 |
| 5 | 26±1 | 871 | 871 | 871 | 871 | 871 | 850 |
| 6a | 30±1 | 1,022 | 1,022 | 1,022 | 1,022 | 1,022 | 850 |
| 6b | 35±1 | 1,022 | 1,100 | 1,100 | 1,022 | 1,022 | 850 |
| 7a | 40±2 | 1,190 | 1,200 | 1,200 | 1,400 | 1,400 | 850 |
| 7b | 45±3 | 1,190 | 1,300 | 1,300 | 1,100 | 1,100 | 850 |
| 7c | 50±5 | 1,190 | 1,400 | 1,400 | 1,100 | 1,100 | 850 |
| 8a | 60±5 | 1,800 | 1,600 | 1,600 | 2,000 | 2,000 | 1,800 |
| 8b | 70±5 | 1,800 | 1,800 | 1,800 | 1,600 | 1,600 | 1,800 |
| 8c | 80±5 | 1,800 | 2,000 | 2,000 | 1,600 | 1,600 | 1,800 |
| 8d | 90±5 | 1,800 | 2,200 | 2,200 | 1,600 | 1,600 | 1,800 |
| 9a | 95±2 | 2,900 | 2,900 | 2,350 | 2,900 | 3,100 | 2,900 |
| 9b | 100±2 | 2,900 | 2,900 | 2,500 | 2,900 | 2,200 | 2,900 |
| 9c | 110 | 2,900 | 2,900 | 2,650 | 2,900 | 2,200 | 2,900 |
| 9d | 125 | 2,900 | 2,900 | 2,800 | 2,900 | 2,200 | 2,900 |
| 10 | 700±100 | 2,900 | 2,900 | 2,900 | 2,900 | 2,900 | 2,900 |

**Table SHA08-4**
**Average Shear Wave Velocity versus Depth for Alternative Profile 1**

| Layer | Thickness (ft) | Layer Vs (fps) | Total Depth (ft) | Unit Weight (pcf) | Layer Travel Time (sec) | Total Travel Time (sec) | Average Velocity (fps) |
|---|---|---|---|---|---|---|---|
| 1 | 5 | 560 | 5 | 80 | 0.008929 | 0.008929 | 560 |
| 2 | 5 | 528 | 10 | 80 | 0.009470 | 0.018398 | 544 |
| 3 | 2 | 727 | 12 | 80 | 0.002751 | 0.021149 | 567 |
| 4 | 6 | 854 | 18 | 100 | 0.007026 | 0.028175 | 639 |
| 5 | 8 | 871 | 26 | 94 | 0.009185 | 0.037360 | 696 |
| 6 | 4 | 1,022 | 30 | 94 | 0.003914 | 0.041274 | 727 |
| 7 | 5 | 1,100 | 35 | 115 | 0.004545 | 0.045819 | 764 |
| 8 | 5 | 1,200 | 40 | 115 | 0.004167 | 0.049986 | 800 |
| 9 | 5 | 1,300 | 45 | 115 | 0.003846 | 0.053832 | 836 |
| 10 | 5 | 1,400 | 50 | 115 | 0.003571 | 0.057404 | 871 |
| 12 | 10 | 1,600 | 60 | 120 | 0.006250 | 0.063654 | 943 |
| 13 | 10 | 1,800 | 70 | 120 | 0.005556 | 0.069209 | 1011 |
| 14 | 10 | 2,000 | 80 | 120 | 0.005000 | 0.074209 | 1078 |
| 15 | 10 | 2,200 | 90 | 120 | 0.004545 | 0.078755 | 1143 |

**Table SHA08-5**
**Average Shear Wave Velocity versus Depth for Alternative Profile 3**

| Layer | Thickness (ft) | Layer Vs (fps) | Total Depth (ft) | Unit Weight (pcf) | Layer Travel Time (sec) | Total Travel Time (sec) | Average Velocity (fps) |
|---|---|---|---|---|---|---|---|
| 1 | 5 | 560 | 5 | 80 | 0.008929 | 0.008929 | 560 |
| 2 | 5 | 528 | 10 | 80 | 0.009470 | 0.018398 | 544 |
| 3 | 2 | 727 | 12 | 80 | 0.002751 | 0.021149 | 567 |
| 4 | 6 | 854 | 18 | 100 | 0.007026 | 0.028175 | 639 |
| 5 | 8 | 871 | 26 | 94 | 0.009185 | 0.037360 | 696 |
| 6 | 4 | 1,022 | 30 | 94 | 0.003914 | 0.041274 | 727 |
| 7 | 5 | 1,022 | 35 | 115 | 0.004892 | 0.046166 | 758 |
| 8 | 5 | 1,400 | 40 | 115 | 0.003571 | 0.049738 | 804 |
| 9 | 5 | 1,100 | 45 | 115 | 0.004545 | 0.054283 | 829 |
| 10 | 5 | 1,100 | 50 | 115 | 0.004545 | 0.058829 | 850 |
| 12 | 10 | 2,000 | 60 | 120 | 0.005000 | 0.063829 | 940 |
| 13 | 10 | 1,600 | 70 | 120 | 0.006250 | 0.070079 | 999 |
| 14 | 10 | 1,600 | 80 | 120 | 0.006250 | 0.076329 | 1048 |
| 15 | 10 | 1,600 | 90 | 120 | 0.006250 | 0.082579 | 1090 |

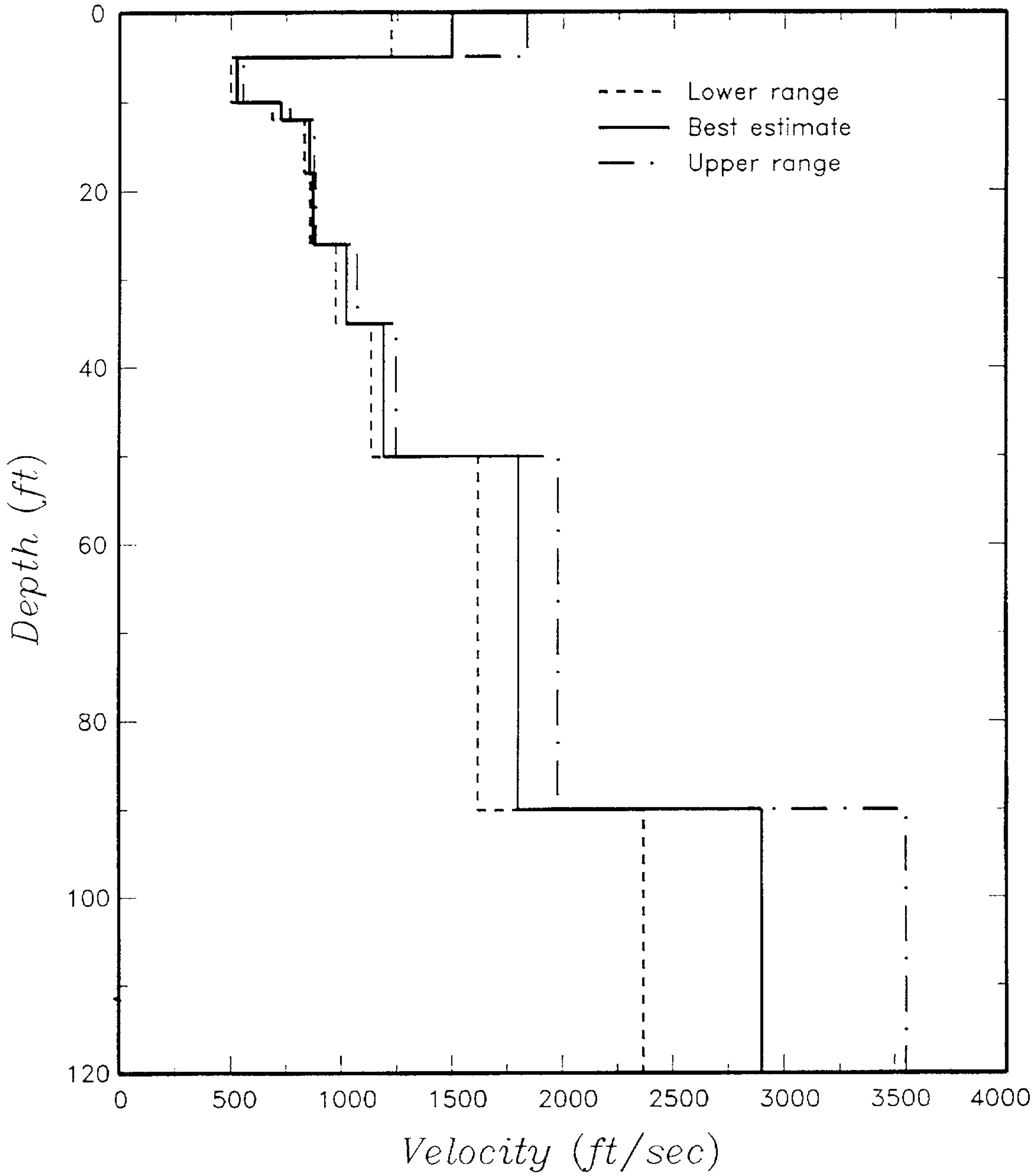


Figure SHA08-1 Shear wave velocity profiles used in Geomatrix (2001a)

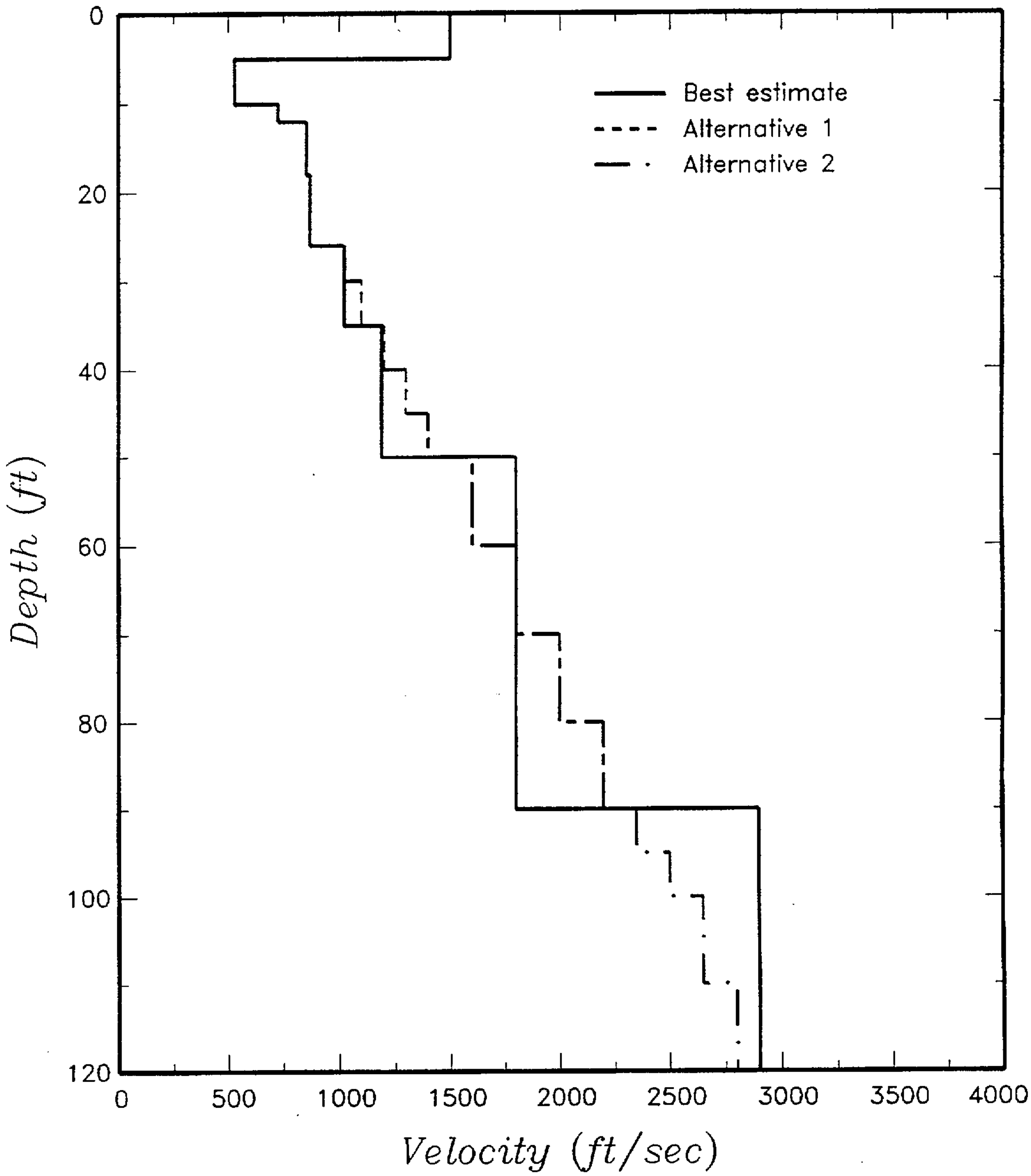

Figure SHA08-2 Alternative shear wave velocity profiles 1 and 2 used in sensitivity analyses

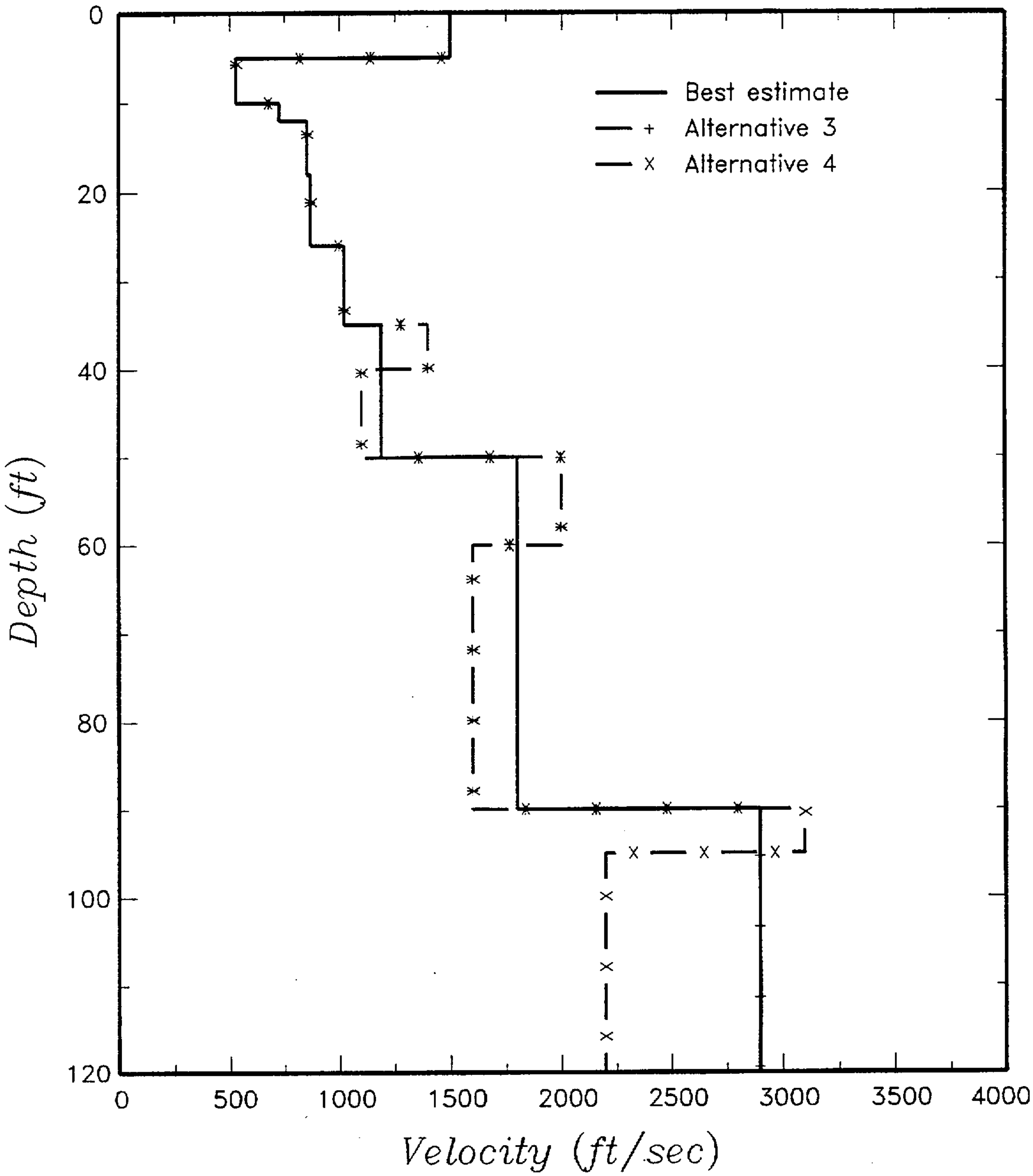

Figure SHA08-3 Alternative shear wave velocity profiles 3 and 4 used in sensitivity analyses

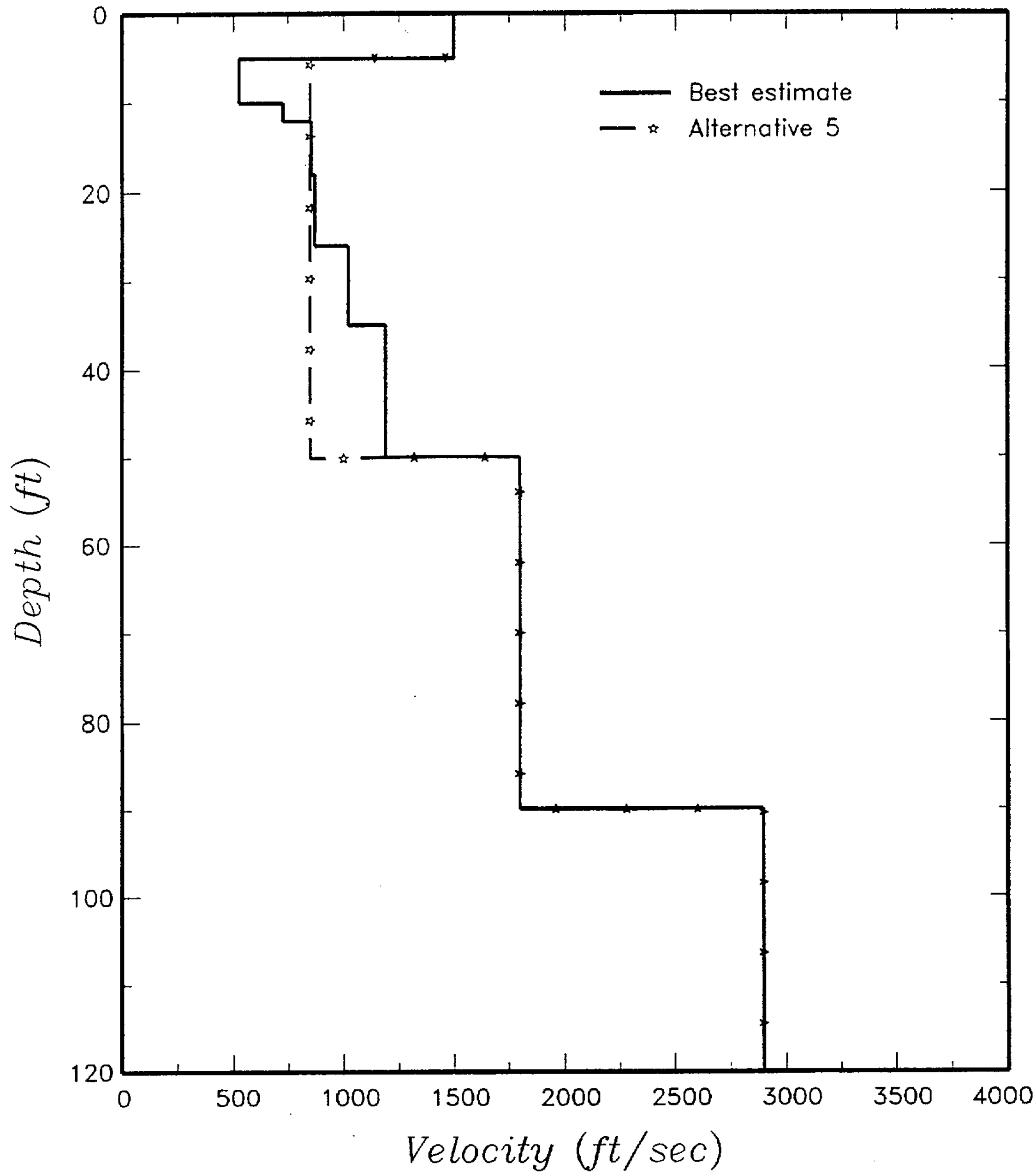


Figure SHA08-4 Alternative shear wave velocity profile 5 used in sensitivity analyses

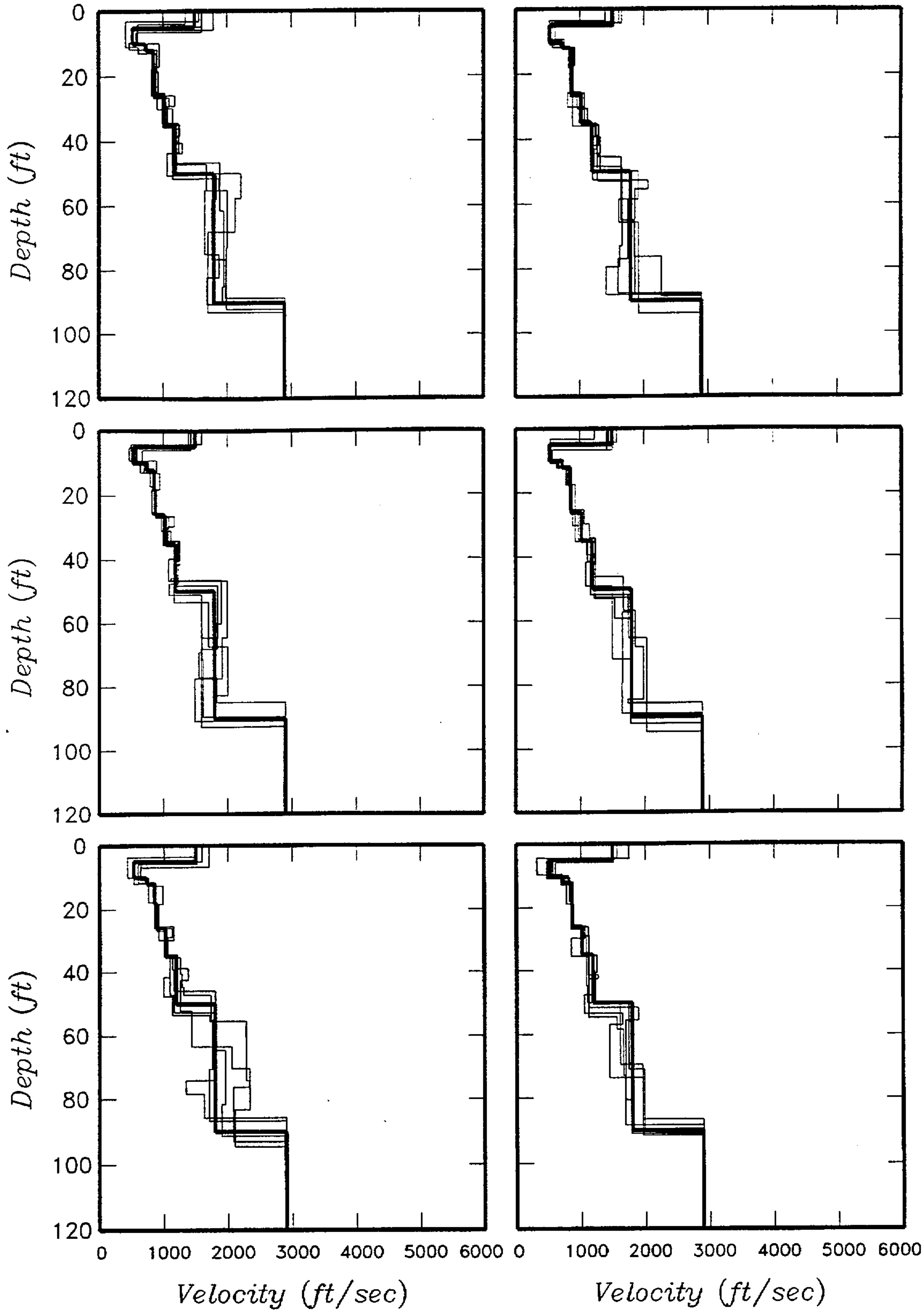


Figure SHA08-5 Thirty randomized shear wave velocity profiles for base case

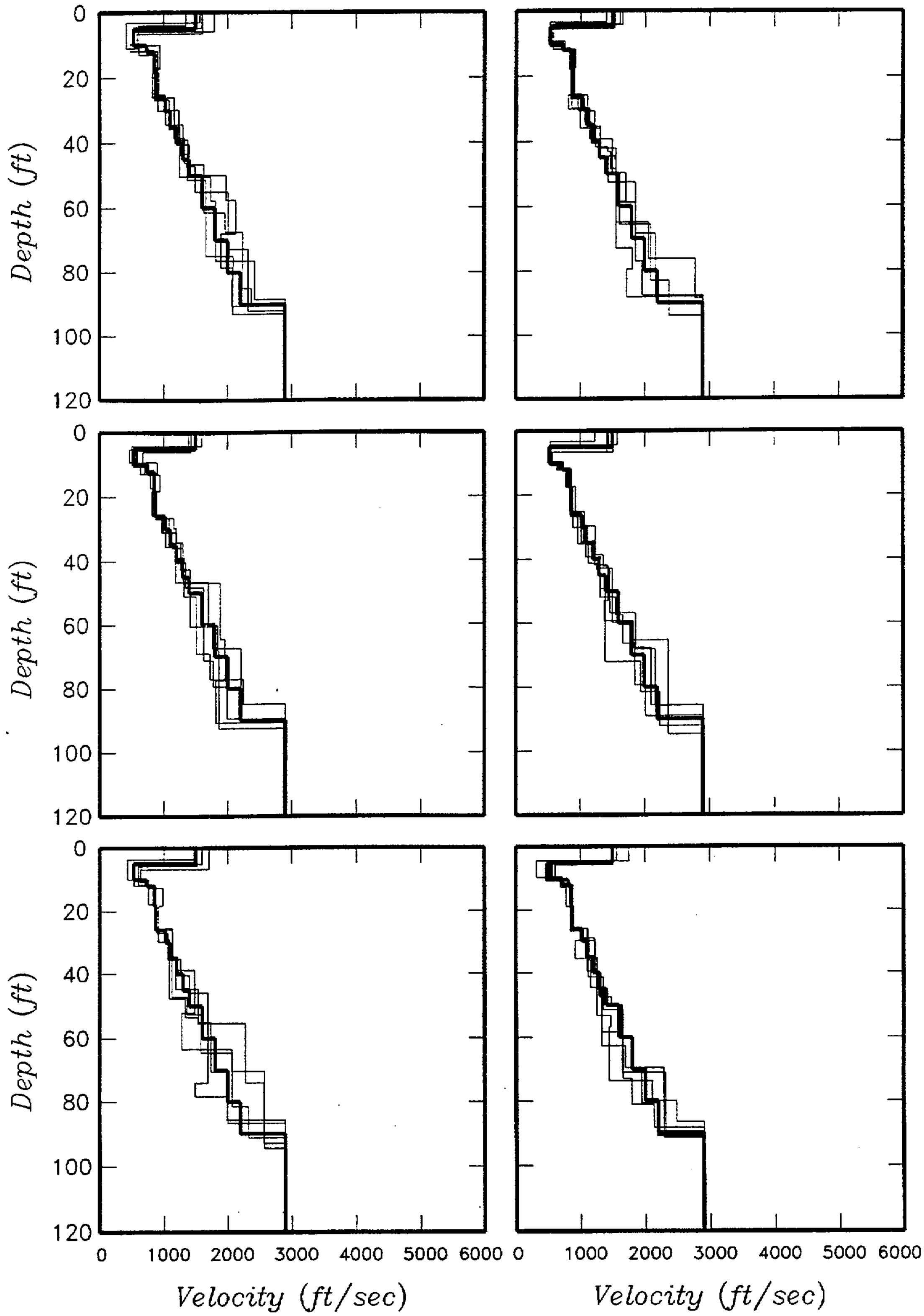


Figure SHA08-6 Thirty randomized shear wave velocity profiles for Alternative 1

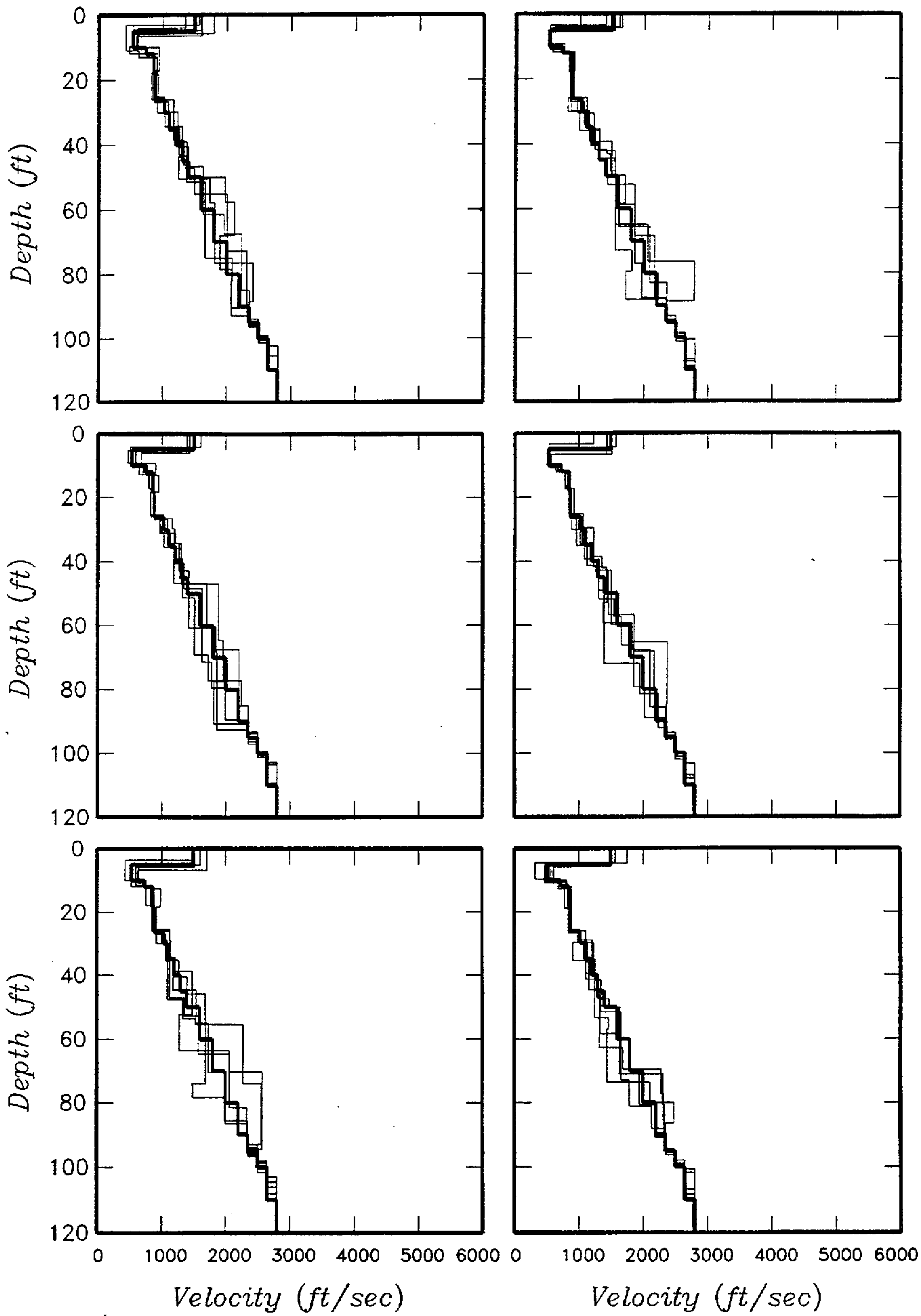


Figure SHA08-7 Thirty randomized shear wave velocity profiles for Alternative 2

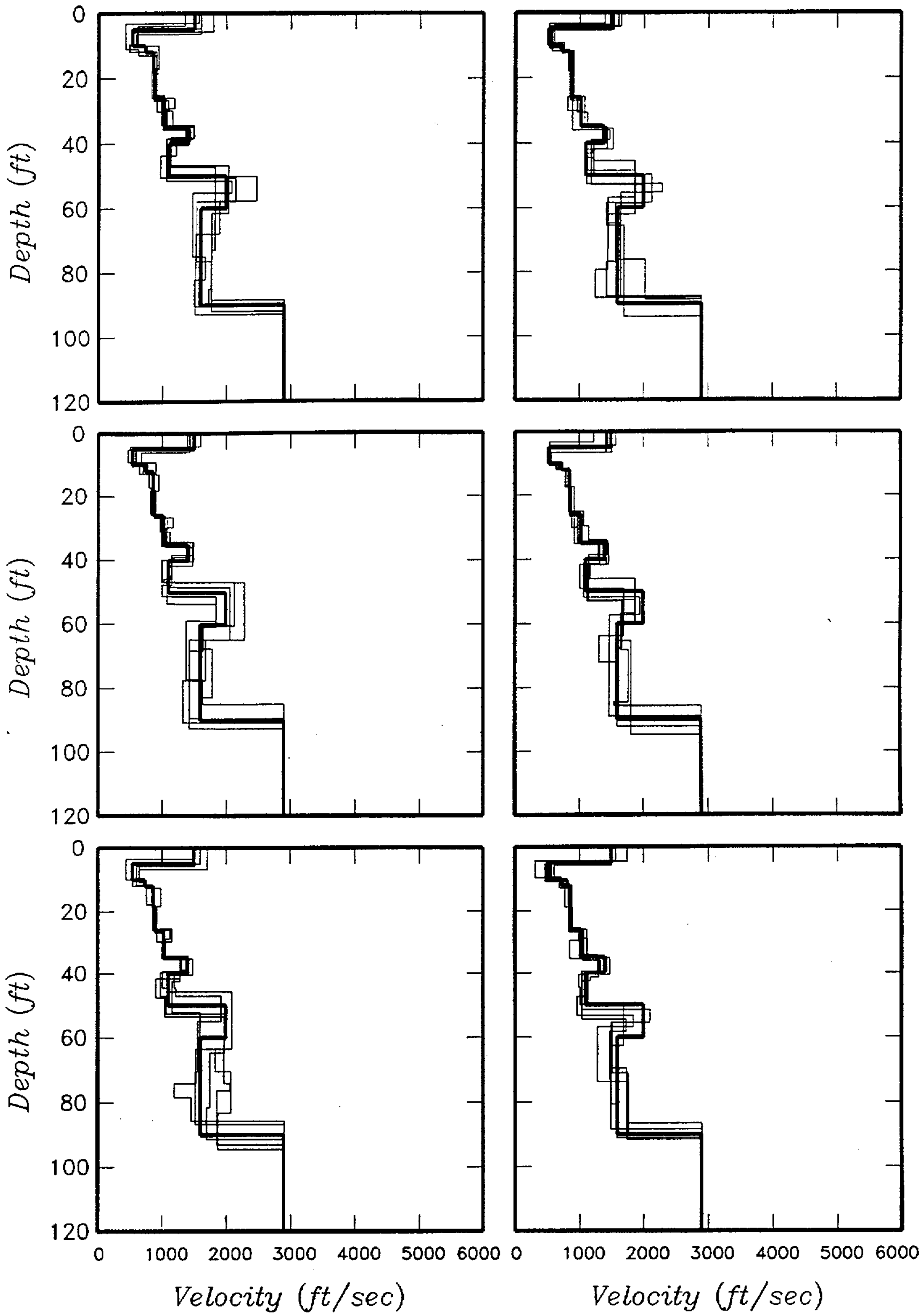


Figure SHA08-8 Thirty randomized shear wave velocity profiles for Alternative 3

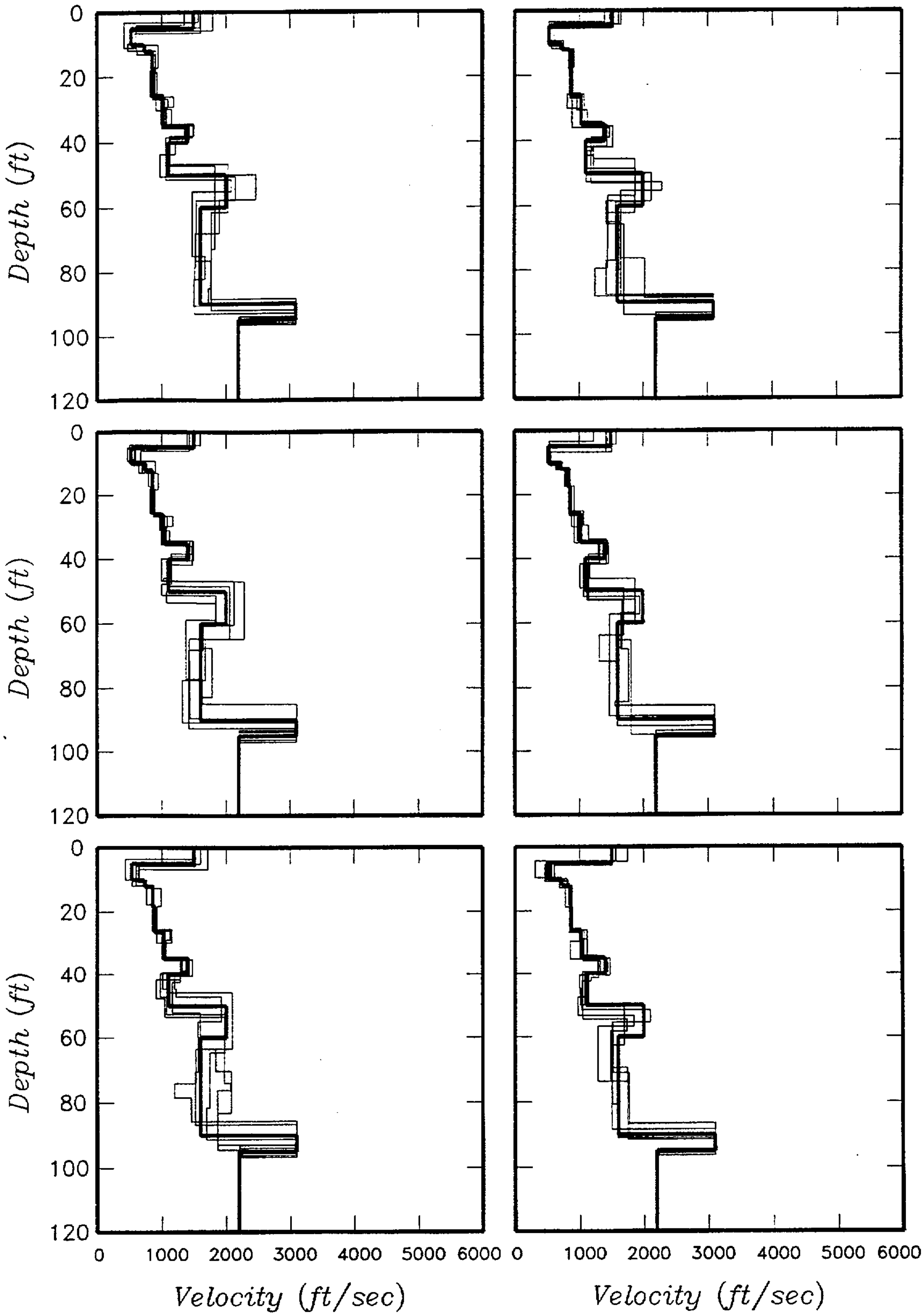


Figure SHA08-9 Thirty randomized shear wave velocity profiles for Alternative 4

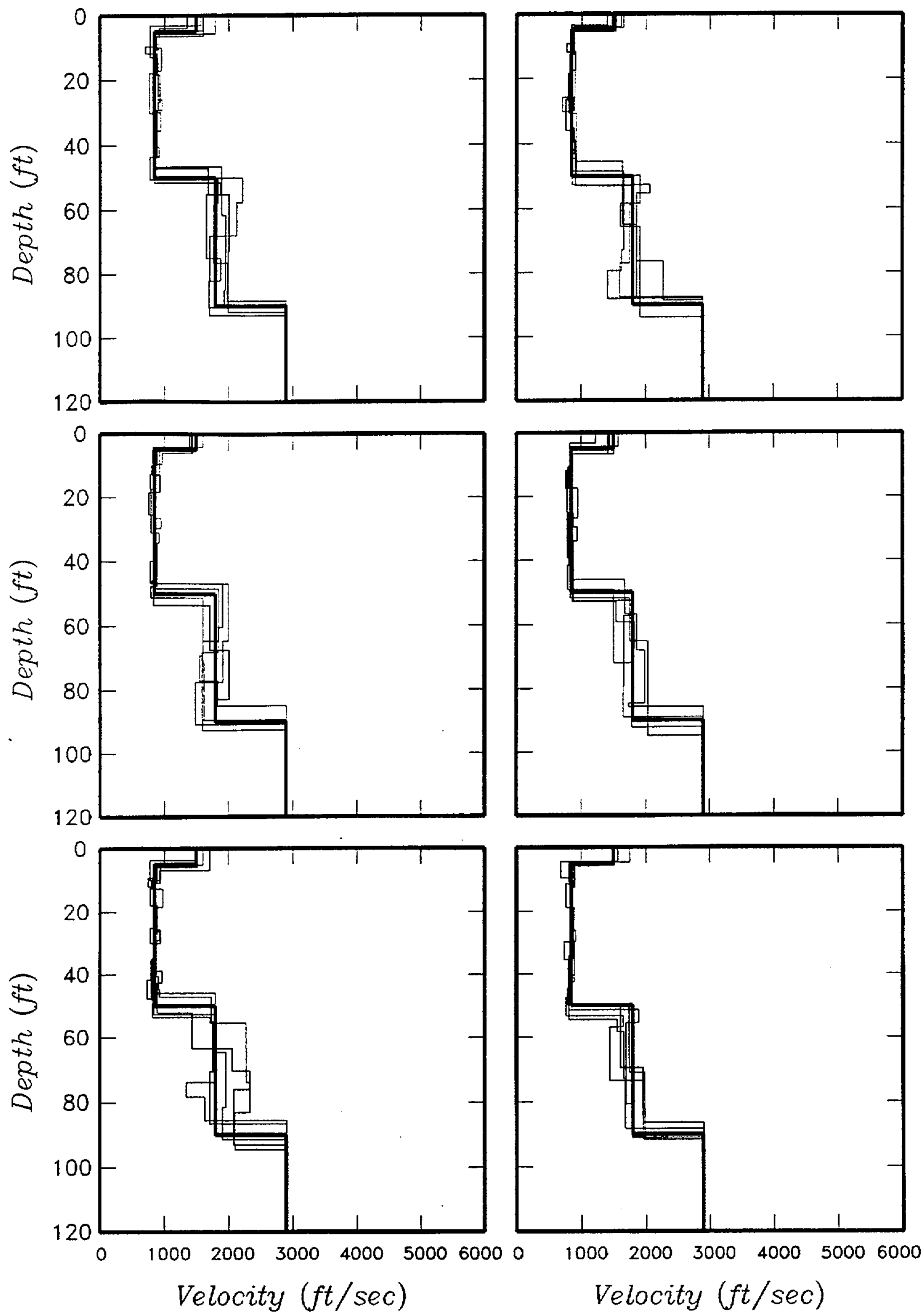


Figure SHA08-10 Thirty randomized shear wave velocity profiles for Alternative 5

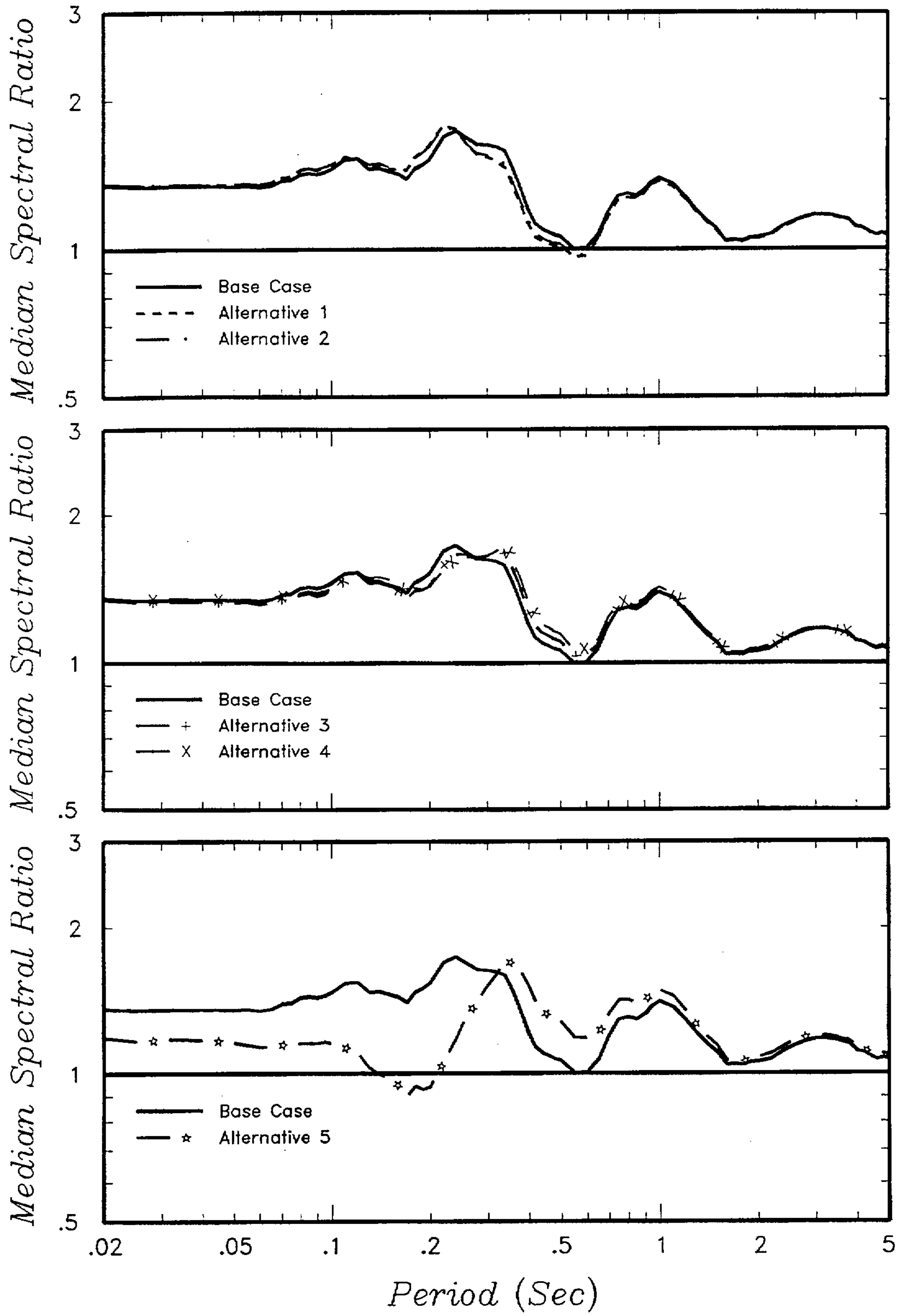


Figure SHA08-11 Comparison of relative site response spectral ratios for the base case and alternative shear wave velocity profiles

**Soil Engineering:**

1. **A site plan showing location of any new borings and test pits used to support PFS analyses.**

PFS Response

A Figure showing the location of the 16 new test pits was provided with PFS letter, Donnell to U.S. NRC, "Response to April 18, 2001 Meeting Issues Regarding PFSF License Application Amendment #22", dated May 1, 2001. As requested in the conference call of May 9, 2001 between PFS, S&W, NRC and the CNWRA, PFS has incorporated these locations onto SAR Figure 2.6-19, which is attached. This revised Figure 2.6-19 and supporting text (if required) will be included in the next License Amendment.

2. **Logs for any new borings or test pits used to support PFS analyses.**

PFS Response

Test pit logs for the 16 test pits that were excavated in January 2001 to obtain bulk soil samples for soil-cement tests were provided with PFS letter, Donnell to U.S. NRC, "Response to April 18, 2001 Meeting Issues Regarding PFSF License Application Amendment #22", dated May 1, 2001. These logs and supporting text (if required) will be included in the next License Amendment.

3. **Revised analyses of the stability of the storage pads to include a clear identification of the potential failure modes and failure surfaces, and the material strengths required to satisfy the regulatory requirement, considering the critical failure modes and failure surfaces.**

PFS Response

Calculation No. 05996.02–G(B)-04, Revision 8, entitled "Stability Analysis of Storage Pads" and Calculation No. 05996.02–G(B)-13, Revision 5, entitled "Stability Analysis of the Canister Transfer Building Supported on a Mat Foundation" are attached. These calculations have been revised to clarify the critical failure modes, failure surfaces, and the required material strengths. All references to these calculations and supporting text (if required) will be updated in the next License Amendment.

**Design of Facility:**

**Storage Pads**

1. **Assessment of the edge effects on the stability of the Storage Pads under new seismic loads.**

PFS Response
Calculation No. 05996.02–G(B)-04, Revision 8, entitled "Stability Analysis of Storage Pads" is attached. This calculation has been revised to address the stability of the pads on the outer edge of the storage pad array. All references to this calculation and supporting text (if required) will be updated in the next License Amendment.

**Cask Transfer Building**

1. **General description of the major structural elements of the CTB. This should include the reinforced concrete walls, columns, roof and slab and the structural steel elements including the roof support beams.**

PFS Response
The CTB is a reinforced concrete structure in which the fuel canisters are transferred from the shipping casks to the storage casks. It is supported on a 5' thick rectangular base mat 279.5' long by 240' wide. There is a 1.5' deep shear key around the perimeter of the mat to help resist seismic sliding forces. The main portion of the building is 90' high and contains the three transfer cells. The perimeter shear walls are 2' thick and support the roof and the two cranes that are used in the canister transfer operations. The perimeter walls and the 8" thick roof provide tornado missile protection for the transfer cells. The reinforced concrete roof slab is poured on 1-1/2" deep metal decking. A steel frame supports the vertical roof dead weight, snow, and seismic loads. The decking spans approximately 5 feet to 16" deep steel beams. The 16" deep roof beams span up to 30 feet in the north-south direction to the main roof girders. Five foot deep steel girders spanning 65' in the E-W direction carry the vertical roof loads to embedded plates set in the building's concrete walls. Horizontal seismic load from the roof mass is transferred to the building's walls by diaphragm action of the roof slab.

There are three openings (22' wide) through the wall on the west side of the transfer cells to allow cask transporter access to each transfer cell. These openings are tornado missile protected during canister transfer operations by 1-foot thick rolling doors fabricated from steel plate and concrete.

On the east and west sides of the main building are lower roofed areas that house the transporter aisle (on the west side) and an office area with mechanical rooms (on the east side). The rail bay (used for the off loading of shipping casks) extends out from the main portion of the building on both the east and west sides under the low roofed areas. The low roofs are 30 feet above grade and are constructed of 8" thick reinforced concrete supported on a structural steel frame similar to the high roof except the main girders have shorter spans and are 36" deep. The low roofs provide lateral support for the N-S perimeter walls of the main building.

SAR Chapter 3 provides a complete discussion of the principal design criteria for the CTB. The input criteria for the design basis ground motion is the only change to Chapter 3 that affects the CTB. All other criteria remain as previously reviewed and approved by the NRC in the SER.

SAR Chapter 4, Section 4.7.1 provides a discussion of the CTB design including design specifications, design bases and safety assurance, structural design (design load combinations and analysis methods). Although changes have been made to the CTB design, the essential elements of the design remain as previously reviewed and approved by the NRC in the SER.

A detailed explanation of the changes made to the CTB including analyses revisions and their correlation to the previous design was provided in PFS letter, Donnell to U.S. NRC, "Summary of Changes for PFSF License Application Amendment #22", dated April 16, 2001 and in PFS letter, Donnell to U.S. NRC, "Response to April 18, 2001 Meeting Issues Regarding PFSF License Application Amendment #22", dated May 1, 2001.

**2. New calculation package (SC) for Design of Tornado Doors on cells in canister transfer building (CTB).**

PFS Response
Calculation No. 05996.02–SC-14, Revision 0, entitled "Design of Rolling Doors at Canister Transfer Cells" has been completed and is attached. Reference to this calculation and supporting text (if required) will be provided in the next License Amendment.

**3. New SC for Design of roof steel members.**

PFS Response
Calculation No. 05996.02–SC-12, Revision 0, entitled "Design of Canister Transfer Building Structural Steel", has been completed and is attached. Reference to this calculation and supporting text (if required) will be provided in the next License Amendment.

**4. Updated letter from Ederer, Incorporated on impact of new seismic levels.**

PFS Response
A copy of the latest letter from the crane manufacture (EDERER) was provided with PFS letter, Donnell to U.S. NRC, "Response to April 18, 2001 Meeting Issues Regarding PFSF License Application Amendment #22", dated May 1, 2001.

**5. Updated G(B)-11 Dynamic Settlements of the soils underlying the site.**

PFS Response
Calculation No. 05996.02–G(B)-11, Revision 3, entitled "Dynamic Settlements of the Soils Underlying the Site", has been updated and is attached. Revision 3 of this calculation incorporates the effects of changing the design ground motion from a peak horizontal ground acceleration of 0.53g to 0.11g. We also prepared a supplement to the cask storage pad static settlement calculation. The supplement is identified as Calculation No. 05996.02–G(B)-21, Revision 0, entitled "Supplement to Estimated Static Settlement of Cask Storage Pads", and a copy is attached. The purpose of this calculation is to refine the estimate of static settlement, incorporating the results of the consolidation tests that were performed on samples obtained from the Canister transfer Building area and which are included in Attachment 6 of Appendix 2A of Revision 6 to the SAR. In addition, this calculation demonstrates that the estimated differential settlement between the edge of the cask storage pads and the crushed rock surface will be less than ¾ inches, to document the rationale for the change from constructing the pads so that the tops were 3.5 inches above grade to making them flush with the ground surface. All references to these calculations and supporting text (if required) will be updated in the next License Amendment.

**6. Updated SC-4 Impedance Functions for CTB.**

PFS Response
Calculation No. 05996.02–SC–4, Revision 2, entitled "Development of Soil Impedance Functions for Canister Transfer Building" was submitted to the NRC with PFS letter, Donnell to U.S. NRC, "Calculation Package Submittal", dated April 13, 2001.

**7. Assessment of the design changes to the slab in terms of load transfer from the walls to the slab and resulting loads on soils. Emphasis should be on the pad areas extending beyond the building walls.**

PFS Response
The CTB's base mat slab has been widened to increase the building's resistance to overturning. The narrow direction width of the mat (east-to-west), which was 145' (except at the track bay where it was 215'), has been increased to 240'. Bending stresses in the mat for the two primary load cases have been evaluated for the revised seismic

accelerations using the new mat configuration. The bending stresses in the mat have magnitudes similar to those from the previous arrangement except for six small localized areas of load concentration. The high stresses are concentrated at the outer corners of the building directly below the walls (not at the mat corners). The mat elements at the ends of the 'gusset' walls have higher bending moments than the previous analysis. These are the north end wall (1-Line), the inner gusset wall (8-Line), and the south end wall (11-Line). The increase is on the order of 1.5 for Load Case 1 (maximum downward y-acceleration) and 3.5 for the overturning load case (Load Case 2). The high base mat bending is concentrated in approximately 15' wide areas. Attached sketch 0599602-SKA-401A provides a floor plan of the Canister Transfer Building and shows the location of the six areas discussed above.

The initial design required #8 bars placed at 6" on centers under the walls. We are now considering the use of larger diameter reinforcing bars (#11 or larger) placed in two layers in the base mat for the six highly stressed areas identified above.

The resulting foundation loadings of the Canister Transfer Building are presented in SAR Table 2.6-11. SAR Table 2.6-10 presents a summary of the dynamic bearing capacity analyses for these loadings. This table includes $q_{actual}$, the average bearing stress beneath the mat over the effective loaded area due to these foundation loadings. The effective loaded area, defined as B' x L', was calculated using standard geotechnical engineering techniques, originally developed by Meyerhof (1953), to account for eccentric and inclined loads on foundations in bearing capacity analyses. Comparison of these results with those presented in Rev. 3 of PFS Calculation 05996.02-G(B)-13, which used the 0.53g earthquake, indicates that, due to the increase in the size of the mat, the soil stresses under the mat have decreased. The effective soil bearing pressures range from 0.99 ksf to 2.92 ksf for the current configuration (240' x 279.5' mat) and design earthquake with a peak horizontal ground acceleration of 0.711g vs 2.15 ksf to 3.59 ksf for the previous configuration (~165' x ~265' mat) and earthquake with a peak horizontal ground acceleration of 0.53g.

Results of the ANSYS finite element analysis were examined to assess local bearing pressures. The worst overturning case (maximum accelerations in the east direction, 0.4 times the maximum acceleration in the north direction, and 0.4 times the maximum acceleration in the upward direction) produces the worst bending moments in the mat, and was selected for this evaluation. For this load case, SAR Table 2.6-10 indicates that the ultimate soil bearing pressure is 14.1 ksf (for Case IIIB). Local bearing pressures were calculated by taking the force in the elements connecting the base mat to the soil model, and dividing by the tributary area. It was found that at several points the local bearing pressures exceeded 14.1 ksf (maximum of 26.7 ksf).

To assess the consequences of this local overstress, the force in the overstressed elements was only allowed to increase to a value equal to the ultimate bearing pressure times the tributary area. This resulted in higher forces in adjacent elements, some of which increased to levels exceeding the ultimate bearing pressure. Again the forces in the overstressed elements were limited to be consistent with the maximum bearing pressure.

This process was repeated until the forces in all the elements were equal to or less than the maximum bearing pressure. It was found that the maximum bending moments increased by only 5% when this redistribution of bearing pressure was done. Based on this study, it is concluded that bearing pressures in some isolated areas (particularly under the walls at the corner of the building) may exceed the global ultimate bearing pressure, but there are minimal increases in the stresses in the mat, as these high stresses in these local areas are redistributed to adjacent sections of the mat.

**Reference**

Meyerhof, G. G., 1953, The bearing capacity of foundations under eccentric and inclined loads, Proceedings, Third International Conference on Soil Mechanics and Foundation Engineering, Zurich, Vol. I, pp. 440-445.

**8. Assessment of fire impact on the new design of the CTB.**

<u>PFS Response</u>
An assessment of the effect of the CTB fire scenarios on the new roof steel was provided in item 9 of PFS letter, Donnell to U.S. NRC, "Response to April 18, 2001 Meeting Issues Regarding PFSF License Application Amendment #22", dated May 1, 2001. This information will be included in the SAR in the next License Amendment.

**9. Assessment of the drop of a cask onto the slab of the CTB.**

<u>PFS Response</u>
A drop of a cask onto the CTB slab has been evaluated by Holtec International in Report No. HI-2012653, Revision 1, entitled "PFSF Site-Specific HI-STORM Drop/Tipover Analyses". This report is proprietary, therefore a copy of the updated report is provided under separate cover. Reference to this revised report and supporting text (if required) will be included in the next License Amendment

## 10. NRC Questions Regarding Certain SAR References

At the April 18, 2001 Meeting in San Antonio, the NRC requested that PFS determine if the several references in the latest SAR revision (Revision No. 21, included in PFSF License Application Amendment #22) are correct. The references in question are repeated below by SAR chapter and followed by the PFS response.

### SAR Chapter 2

CEC, 2001, PFSF Calculation 05996.02-G(PO17)-2, Rev 3, Storage Pad Analysis and Design, prepared by International Civil Engineering Consultants, Inc, for Stone and Webster Engineering Corp, Denver, CO.

Stone & Webster Engineering Corporation (SWEC), 1998a, Calculation No. 05996.02-SC-6, Revision 0, Finite Element Analysis of Canister Transfer Building.

Stone & Webster Engineering Corporation (SWEC), 1999e, Calculation No. 05996.02-G(B)-3, Revision 3, Estimate Static Settlement of Storage Pads.

Stone & Webster Engineering Corporation (SWEC), 2000a, Calculation No. 05996.02-G(B)-11, Revision 1, Dynamic Settlements of the Soils Underlying the Site.

Stone & Webster Engineering Corporation (SWEC), 2000b, Calculation No. 05996.02-G(C)-14, Revision 1, Static Settlement of the Canister Transfer Building Supported on a Mat Foundation.

Stone & Webster Engineering Corporation (SWEC), 2001a, Calculation No. 05996.02-G(B)-5, Revision 3, Document Bases for Geotechnical Parameters Provided in Geotechnical Design Criteria.

Geomatrix Consultants, Inc., 2001c, PFSF Calculation 05996.02-G(PO18)-2, Rev 1, Soil and Foundation Parameters for Dynamic Soil-Structure Interaction Analyses, 2,000-Year Return Period Design Ground Motions, prepared for Stone and Webster Engineering Corp., Denver, CO, 53 pp, March 2001.

<u>PFS Response</u>
The above references were checked and determined to be correct, with the exception of Stone & Webster Engineering Corporation (SWEC), 2001a, Calculation No. 05996.02-G(B)-5, Revision 3, Document Bases for Geotechnical Parameters Provided in Geotechnical Design Criteria. This should be Revision 2. The NRC noted that the first reference listed above (Calculation 05996.02-G(PO17)-2, prepared by CEC), is listed as Revision 3 in SAR Chapter 2, but as Revision 2 in SAR Chapter 4 (Reference 16). It is Revision 3, and the Chapter 4 reference to this calculation needs to be changed to refer to Revision 3

**SAR Chapter 3**

9. Private Fuel Storage Facility Storage Facility Design Criteria, Section 4.0, Geotechnical Design Criteria, Revision 2.

12. Deterministic Earthquake Ground Motion Analysis, Private Fuel Storage Facility, prepared by Geomatrix Consultants, Inc. and William Lettis & Associates, SWEC Report No. 0599601-G(P05)-1, Revision 0.

32. Stone and Webster Topical Report, SWECO 7703, "Missile-Barrier Interaction", September 1977

<u>PFS Response</u>
The above references were checked and determined to be correct, with the exception of Reference 9, which should be updated to refer to Revision 3 of the PFSF Design Criteria, which was approved in October 2000. The Chapter 3 textual references to the PFSF Design Criteria (for depth to bedrock and depth required for protection against frost) remain correct. Reference 12, the original deterministic earthquake analysis which was at one time Appendix 2D of the PFSF SAR, is no longer referenced in the Chapter 3 text and should be deleted from the list of Chapter 3 references.

**SAR Chapter 4**

7. Multi-Cask Seismic Response at the PFS ISFSI, Holtec International, Holtec Report HI-971631, Revision 1.

22. Private Fuel Storage Facility Storage Facility Design Criteria, Section 4.0, Geotechnical Design Criteria, Revision 2.

47. PFSF Calculation No. 05996.02 SC-7, Design of Reinforcing Steel for Canister Transfer Building, Revision 0, Stone & Webster.

56. Seismic Qualification Analysis 200 Ton Bridge Crane, PFSF, No. ANA-QA-147, Anatech Corporation, Revision 0, November 1998.

57. Seismic Qualification Analysis 150 Ton Semi-gantry Crane, PFSF, No. ANA-QA-148, Anatech Corporation, Revision 0, November 1998.

60. Holtec Report HI-992134, HI-STORM Thermal Analysis for PFS RAI, Rev. 0, dated February 9, 1999.

62. PFSF Calculation No. 05996.02 SC-10, Seismic Restraints for Spent Fuel Handling Casks, Revision 1, Stone & Webster.

63. Ederer Incorporated letter from S. Hertel to W. Lewis of Stone & Webster, Impacts of the Revised Seismic Accelerations on the Cranes for the Skull Valley Project, Document No. F2621L0045H, dated March 23, 2001.

PFS Response

The above references were checked and determined to be correct, with the exception of Reference 22, which is no longer referenced in the Chapter 4 text and should be deleted from the list of Chapter 4 references. The NRC asked if Reference 7 (Holtec Report HI-971631) has been superceded by Holtec Report HI-2012640 (which is Reference 61 of Chapter 4). Reference 7 applies to the cask stability analysis performed for the original deterministic design earthquake, and this report is referenced by HI-2012640. Therefore, it is appropriate to retain Reference 7. The NRC asked if References 56 and 57 have been updated. They have not. As discussed in SAR Section 4.7.2.5.3, "The PFSF overhead and semi-gantry cranes have been seismically analyzed in accordance with ASME NOG-1 to ensure they will remain in place and support the load during and after a seismic event. The analyses were performed for both cranes by Anatech Corporation to qualify the crane designs for the original PFSF deterministic design earthquake (0.67g horizontal, 0.69g vertical – See Section 8.2.1.1)... In addition, the cranes were reviewed by Ederer for their seismic stability based on the current PFSF design basis ground motion of 0.711g horizontal and 0.695g vertical (See Section 3.2.10.1.1) and resulting response spectra curves... PFS will have Ederer formally update the seismic analysis for both cranes as part of the final detailed engineering phase of the crane design and fabrication." This seismic analysis is currently underway. The NRC also asked if Reference 60 (Holtec Report HI-992134) has been updated. It has been supplemented by Holtec Report HI-2002413 (Reference 85 of Chapter 4), but not itself updated, and remains a valid reference.

The NRC asked what happened to the following documents that are not listed in SAR Chapter 4 in the latest revision:

a) PFSF Calculation No. 05996.01-SC-1, Evaluation of Concrete Storage Pad Target Hardness, Stone & Webster.

b) PFSF Calculation No. 05996.02-SC-8, Crane Decoupling Evaluation – Canister Transfer Building, Stone & Webster.

c) PFSF Calculation No. 05996.02 SC-4, Development of Soil Impedance Functions for Canister Transfer Building, Revision 2, Stone & Webster.

Calculation c) above is currently listed as Reference 42 of Chapter 4. While PFS records indicates that above calculations a) and b) were submitted to the NRC, they were not previously referenced in SAR Chapter 4. The target hardness methodology for evaluating a storage cask drop event is not the methodology used by Holtec for its analyses of HI-STORM storage cask drop analyses, and is not relevant to the PFSF licensing basis. The crane decoupling evaluation was performed to demonstrate that a discrete model of the crane does not need to be included in the seismic analysis of the Canister Transfer Building. This calculation b) will be updated when Anatech Corporation (the vendor for both of the Canister Transfer Building cranes) completes their design of the cranes and the masses and natural frequencies of the cranes have been established.

**SAR Chapter 8**

8. Holtec Report No. HI-971631, Multi-Cask Response at the PFS ISFSI, Revision 0, dated May 19, 1997.

28. Geomatrix Consultants, Inc, Deterministic Earthquake Ground Motions Analysis, Private Fuel Storage Facility, Skull Valley, Utah, prepared by Geomatrix Consultants, Inc. and William Lettis & Associates, Inc., GMX#3801.1 (Rev. 0), March 1997.

42. Holtec Report No. HI-992277, Multi-Cask Response at the PFS ISFSI, From 2000 Year Seismic Event, Revision 0, dated August 20, 1999.

44. PFSF Calculation No. 05996.02-UR-5, Dose Rate Estimates from Storage Cask Inlet Duct Clearing Operations, Revision 2, Stone & Webster.

45. PFSF Calculation No. 05996.01-UR-3, Postulated Release of Removable Contamination from Canister Outer Surfaces - Dose Consequences, Revision 2, Stone & Webster.

72. J&R Engineering Company, Inc. fax from R. Johnston to DW Lewis of Stone & Webster, J&R Engineering Drawing No. 1481L001, Rev. B, "Preliminary Layout TL250-40 Commonwealth Edison," with revisions to suit PFSF, dated June 15, 2000.

74. PFSF Calculation No. 05996.02-UR(D)-13, Dose Calculation at 500 Meters for the HI-STORM BWR Canister for Postulated Accident Conditions, Revision 0, Stone & Webster.

PFS Response

The above references were checked and determined to be correct, with the exception of Reference 8. This reference to Holtec Report HI-971631 should be updated to Revision 1 (the same as Reference 7 of SAR Chapter 4). The NRC asked if Reference 8 has been superceded by Holtec Report HI-2012640 (Reference 82 of SAR Chapter 8), and if Reference 8 is different than Reference 42. Reference 8 applies to the original deterministic design earthquake. SAR Section 8.2.1.2 states: "Results of the initial HI-

STORM cask stability analysis for the PFSF deterministic design earthquake are documented in Reference 8. Holtec has also performed a cask stability analysis for the PSHA design basis ground motion (Reference 82), described below." Reference 42 is Holtec Report No. HI-992277, Multi-Cask Response at the PFS ISFSI, From 2000 Year Seismic Event, Revision 0, dated August 20, 1999. This cask stability analysis was performed for the previous horizontal and vertical design basis ground motions, but the results remain relevant, as discussed in SAR Section 8.2.1.2, which states that "Previous cask stability analyses (e.g., Reference 42) determined that the tipping potential exceeds the sliding potential." The NRC also asked if Reference 28 of Chapter 8 is different than Reference 12 of Chapter 3. The answer is no. These refer to the same report, which established the original deterministic earthquake ground motions.

Conclusion

Several of the above-listed references should be revised, as discussed above. These will be corrected in the next license amendment.

# Enclosure 2

Response to NRC letter dated May 7, 2001

## PFSF LICENSE APPLICATION AMENDMENT #22: ASSOCIATED CHANGES

### INTRODUCTION

The purpose of this document is to describe the changes associated with Amendment #22 to the PFSF License Application (Reference 1), explain the reasons for the changes, provide a summary of the effects of the changes, discuss the continuing adequacy of the PFSF design in light of the changes, including supporting bases (calculations, analyses, and other design basis documents that demonstrate the adequacy of the design), and summarize the revisions made to the License Application documents to reflect the changes. This document includes information previously provided in Reference 2 regarding changes associated with Amendment #22 and reasons for the changes, and expands upon that information. As discussed in Reference 2, the changes associated with License Application Amendment #22 can be grouped into four major categories:

- Revised design basis ground motions
- Revised storage cask/pad spacing
- Revised Canister Transfer Building design
- Other miscellaneous changes

Each category of changes is addressed separately below.

### METHODOLOGY FOR IMPLEMENTING THE CHANGES

In order to ensure that all potentially necessary changes were identified and evaluated and the effects of changes thoroughly understood and incorporated into the design and licensing processes, PFS engaged the efforts of the technical teams that had been involved in the generation of the original licensing basis and the ongoing PFSF design and licensing effort in the areas potentially affected by the changes, including both those participating in developing the design basis documents and the users of the design outputs. Since members of those teams had been involved in the previous analyses, calculations and reports required to support the facility design and licensing basis, their first-hand, in-depth knowledge enabled them to assess not only the direct impacts of the changes being proposed, but also less obvious or indirect effects that could cascade into other areas or disciplines.

Once the proposed changes had been thoroughly assessed and discussed with project management, representatives of the relevant teams were assigned responsibility for preparing analyses, calculations and reports assessing the effects of the proposed changes on the facility design and licensing basis. The results of analyses, calculations and reports of one team were submitted and communicated to project management, which would notify other teams and technical disciplines of information that could impact their areas of expertise and assigned responsibilities. Matrices were developed to identify the revisions that would be required to the PFSF Licensing Application documents (LA, SAR, ER, and EP) as a result of the various proposed changes and to document the effects of the changes, so that the status of the revisions could be tracked to ensure that the affected documents were updated as necessary. Once the need for a revision was identified in one section of a licensing document, reviews were performed to help ensure that other related sections that could possibly be impacted by the revision would be identified, evaluated, and, if necessary, revised for consistency to reflect the proposed change.

Through this process, the PFS project has developed a high degree of confidence that all necessary design changes associated with License Application Amendment #22 have been identified and implemented.

## REVISED DESIGN BASIS GROUND MOTIONS

Reasons for Changes to the Design Basis Ground Motions

Re-evaluation of previously collected test data for the PFSF site indicated that some of the data that had not been completely incorporated into the PFSF "Fault Evaluation Study and Seismic Hazard Assessment", prepared for PFS by Geomatrix Consultants, Inc., needed to be incorporated. Specifically:

1. The seismic shear wave velocity profiles obtained during the 1999 cone penetration testing program at the site for the top 30 feet of soil were evaluated by Geomatrix and incorporated into the calculation "Soil and Foundation Parameters for Dynamic Soil-Structure Interaction Analysis, 2000-Year Return Period Design Ground Motions." However, Geomatrix concluded at the time that these velocity profiles were consistent with the average velocity profile used in the "Fault Evaluation Study and Seismic Hazard Assessment" and that revisions to that Assessment were not required.

2. The unit weight for the soil for both the Skull Valley and generic California deep soil profiles used in the original "Fault Evaluation Study and Seismic Hazard Assessment" was 131 $lb/ft^3$. The appropriate unit weight for the soil at the PFSF varies from 80 $lb/ft^3$ near the surface to 115 $lb/ft^3$ at a depth of 26 ft. It was initially concluded that this difference in unit weight was not a significant contributor to the outcome of the "Fault Evaluation Study and Seismic Hazard Assessment".

A re-evaluation of the above two items determined that the "Fault Evaluation Study and Seismic Hazard Assessment" needed to be revised to include these differences. When the Assessment was revised to account for these differences, it predicted new Peak Ground Accelerations (PGA) of 0.711g horizontal and 0.695g vertical.

## Effects of the Revised Design Basis Ground Motions

The change in the design basis ground motions impacted a number of calculations, reports and analyses which assess the seismicity of the PFSF site, or which use the design basis ground motions as an input. Personnel from PFS, Geomatrix Consultants, Inc. and Stone & Webster developed plans for reevaluating site soil characteristics and the "Fault Evaluation Study and Seismic Hazard Assessment". Once the revised design basis ground motions had been calculated, the following additional teams were assigned responsibilities in their areas of expertise to determine the effects of the updated seismic loads: Holtec International to reanalyze HI-STORM storage cask stability in light of the revised design basis ground motions and response time-history, International Civil Engineering Consultants, Inc. to reanalyze the storage pads, Stone & Webster to reanalyze the stability of the storage pads and Canister Transfer Building for the new seismic loads, and Ederer Inc. and Stone & Webster to reanalyze and make necessary changes to the design of the Canister Transfer Building cranes and the building itself.

The reassessment of the PFSF design basis ground motions, and establishment of revised design basis ground motions, necessitated revisions to the following:

Geomatrix Consultants, Inc. (Geomatrix)

- Calculation No. 05996.02–G(PO18)–2, Revision 1, entitled "Soil and foundation parameters for dynamic soil-structure interaction analysis, 2000-year return period design ground motions".
- Calculation No. 05996.02–G(PO18)–3, Revision 1, entitled "Development of Time Histories for 2000-year return period design spectra".
- Fault Evaluation Study And Seismic Hazard Assessment, Revision 1, March 2001
- Development of Design Basis Ground Motions for the Private Fuel Storage Facility, Revision 1, March 2001.

Holtec International (Holtec)

- Multi Cask Response at the PFS ISFSI from 2000-Yr Seismic Event (Revision 2), Holtec Report No. HI-2012640.

International Civil Engineering Consultants, Inc (CEC)

- Calculation No. 05996.02–G(PO17)–2, Revision 3, entitled "Storage Pad Analysis and Design".

Stone and Webster (S&W)

- Calculation No. 05996.02–SC–4, Revision 2, entitled "Development of Soil Impedance Functions for Canister Transfer Building".
- Calculation No. 05996.02–SC-5, Revision 2, entitled "Seismic Analysis of Canister Transfer Building".
- Calculation No. 05996.02–SC-10, Revision 1, entitled "Seismic Restraints for

Spent Fuel Handling Casks".

- Calculation No. 05996.02–G(B)-04, Revision 7, entitled "Stability Analysis of Cask Storage Pads".
- Calculation No. 05996.02– G(B)-11, Revision 2, entitled "Dynamic Settlements of the Soils Underlying the Site".
- Calculation No. 05996.02–G(B)-13, Revision 4, entitled "Stability Analysis of the Canister Transfer Building Supported on a Mat Foundation".

Ederer Inc.,

- Performed an evaluation to assess impacts of the revised design basis ground motions on the Phase 1 design of each crane (The results of this analysis are included in an attachment to Reference 3).

Adequacy of the PFSF Design to Accommodate the Revised Design Basis Ground Motions

The design of the Canister Transfer Building was modified to accommodate the revised design basis ground motions. The area of the concrete base mat was enlarged to maintain the desired factor of safety against sliding and overturning for the increased seismic loads, and the depth of the perimeter key was increased from 1.0 to 1.5 ft. Soil cement was added around the Canister Transfer Building base mat to make the free-field soil profile for the building consistent with that for the storage pad emplacement area and to help resist sliding forces due to the higher design basis ground motions. The soil cement extends out from the mat a distance equal to one mat dimension in each direction. The revised design was demonstrated to be adequate in ensuring the stability of the Canister Transfer Building against sliding and overturning forces due to the design basis ground motions in Calculation No. 05996.02–G(B)-13. This calculation established criteria for the soil cement surrounding the building base mat to ensure the sliding stability of the building. These soil cement design criteria are discussed further in this enclosure under the heading of "Soil Cement" in the section entitled "Other Miscellaneous Changes". Calculation No. 05996.02–SC-5 determined that the building structural design was adequate for the revised seismic loading. Calculation No. 05996.02–SC-10 demonstrated the adequacy of the design of the seismic restraints to support the casks in the Canister Transfer Building in the event an earthquake were to occur during canister transfer operations.

The design of the storage pads was modified by increasing their length to accommodate the larger cask transporter, as further discussed below. Calculation No. 05996.02–G(B)-04 demonstrated the stability of the longer storage pads when subjected to the revised design basis ground motions, and established criteria for the soil cement underlying and surrounding the storage pads to ensure the sliding stability of the pads. These soil cement design criteria are discussed further in this enclosure under the heading of "Soil Cement" in the section entitled "Other Miscellaneous Changes". Holtec Report No. HI-2012640 demonstrated the stability of HI-STORM storage casks staged on a pad when subjected to the revised design basis ground motions, considering the longer pad and revised cask spacing. Holtec's analysis considered soil-structure interaction by modeling the pad foundation with a layer of soil cement immediately underlying the pad, supported by native soil. The analysis identified maximum tipping and sliding displacements during the design basis ground motions of less than 4 inches, which is acceptable. Forces on the pads from cask/pad interaction determined in the cask stability analysis were used by CEC as input to their storage pad analysis, and factored into the design of the storage pads. The adequacy of the pad concrete and reinforcement design to withstand the revised design basis ground motions, considering the revised pad length and cask spacing, was demonstrated in CEC Calculation No. 05996.02–G(PO17)-2.

Ederer Inc., the vendor for the Canister Transfer Building overhead bridge crane and semi-gantry crane, performed an evaluation to assess impacts of the revised design basis ground motions on the Phase 1 design of each crane. Ederer Inc. determined that modifications to the previous crane structures design would be to required to accommodate the new accelerations. While both cranes will generally fit into the same envelope, the girder sections of both cranes will deepen. The results of this analysis were included in an attachment to Reference 3, and were incorporated in SAR Section 4.7.2.5.3.

Changes to the PFSF License Application Documents

The following is a list of the sections of the PFSF License Application documents (i.e., SAR, ER, EP, and LA) that were updated to address the effects of the revised design basis ground motions and to incorporate the results of the above revised reports and analyses:

- SAR Section 2.6.1.10 was revised to describe the additional downhole seismic velocity surveys that were conducted by Northland Geophysical, LLC in December 2000 and January 2001, and to indicate that the results of these surveys were incorporated into the analyses discussed in Section 2.6.2.
- SAR Section 2.6.1.11.3 was revised to state that the soil cement will nominally extend 2 ft below the bottoms of most of the pads, and that it will have a minimum required thickness of 1 ft and a maximum thickness of 2 ft.
- Section 2.6.1.12 of the SAR was revised to update the discussions of the results of dynamic stability analyses of the storage pads and the Canister Transfer Building resulting from changes to the PFSF site design basis ground motions. SAR Section 2.6.1.12.1 identified the revised storage pad dimensions and the requirement for the unconfined compressive strength of the soil cement surrounding the storage pads to provide sufficient resistance to sliding. Section 2.6.1.12.2 identified revised Canister Transfer Building basemat dimensions and the required 1.5 ft deep perimeter key around the Canister Transfer Building base mat, in addition to 5 ft deep soil cement surrounding the base mat to provide sufficient resistance to sliding for the revised design basis ground motions.
- SAR Section 2.6.2 was revised to incorporate results of the revised site seismic response analyses. Section 2.6.2.1 was revised to reflect the additional geotechnical investigations performed in 2001.

- SAR Sections 2.6.4.7 and 2.6.4.9 were updated to reflect the new design basis ground motions.
- SAR Section 2.6.4.11 was updated to include a discussion of the soil cement around the Canister Transfer Building. Requirements applicable to the soil cement underlying the storage pads were also identified.
- Changes to maintain consistency were also made to other subsections of Section 2.6 of the SAR.
- SAR Section 2.7 was revised to identify the new peak ground accelerations.
- Numerous SAR Chapter 2 tables and figures were revised to account for changes resulting from the revised design basis ground motions. Figure 2.6-5, "Pad Emplacement Area Foundation Profile 'A-A' was revised to reflect the most recent determination of locations and depths of the various soil layers.
- Information was provided in Appendix 2G explaining why the conclusions of the Appendix had not changed, even though the design basis ground motion values were revised.
- Section 3.2.10.1.1 of the SAR was revised to reflect the site-specific horizontal and vertical response spectra associated with the new design basis ground motions.
- Changes to maintain consistency were also made to other subsections of Section 3.2.10 of the SAR.
- Section 4.2.1.5.1(H) of the SAR, which evaluates the structural design of the storage cask under seismic conditions, was updated to reflect the results of the HI-STORM storage cask stability analyses based on the new seismic response spectra.
- SAR Section 4.2.3.5.1 was revised to reflect the dynamic analyses of the storage pads for the new design basis ground motions.
- SAR Sections 4.7.1 and 4.7.2 were updated to address changes in the Canister Transfer Building design, and in the design of the overhead bridge and semi-gantry cranes, resulting from the new seismic loads.
- SAR Section 8.2.1 was revised to reflect the new design basis ground motions and the results of the HI-STORM storage cask stability analyses based on the new seismic response spectra.
- The discussion of the stability of a loaded cask transporter under seismic conditions (Section 8.2.6.2) was updated for the new design basis ground motions.
- Section 2.6 of the PFSF Environmental Report, which includes a summary of the geotechnical and seismic information in Chapter 2 of the SAR, was updated to be consistent with the information presented in the SAR.
- Section 2.6.5 of the ER was revised to incorporate the changes made to the velocity profiles and resulting changes to the site response analyses and idealized soil profiles that were used in the soil-structure interaction analyses.

- ER Section 2.6.8 was updated to identify the new design basis ground motions.
- Changes to maintain consistency were also made to other subsections of Section 2.6 of the ER.
- Several ER Chapter 2 tables and figures were revised to account for changes resulting from the revised seismic analysis and design basis ground motions.

## REVISED STORAGE CASK/PAD SPACING

Reasons for Changes to the Cask/Pad Spacing

In the process of preparing the procurement specification for the PFSF storage cask transporter, it was determined that current transporter designs have become larger than those evaluated in the previous PFSF design (the PFSF design for the Canister Transfer Building and cask storage area had been based on a transporter used at Point Beach). The dimensions of the new generation transporters that have been designed to date for use with HI-STORM storage casks are substantially larger than those provided by the designer/fabricator of the Point Beach transporter.

Based on extensive discussions with two transporter vendors, it was concluded that the PFSF design should accommodate transporter dimensions of up to 17'-4" wide and approximately 25 ft long. With the previous 15 ft center-to-center spacing between storage casks, the clearance between the outside edge of the transporter and an adjacent cask could be as little as 3 inches, assuming worst case cask placement tolerances. Such limited clearances would make cask placement difficult and time consuming. Increasing the cask spacing to 16 ft would increase the most limiting clearance to 1'-3", improving operational ease in placing the casks. Therefore, it was decided to increase the length of each pad from 64 ft to 67 ft, which provides for the 16 ft center-to-center cask spacing in the pad length direction (north-south). The cask spacing in the pad width direction (east-west) remains at 15 ft. Since there are 20 pads in a column in the cask storage area, from north to south, the total additional length required to accommodate the new pad size is 20 x 3 ft = 60 ft. This 60 ft distance was accommodated by reducing the 150 ft space between the north and south pad quadrants to 90 ft, therefore not impacting the overall outer dimensions of the cask storage area or the location of the Restricted Area (RA) fence.

In discussions with the transporter vendors it was determined that the anticipated diagonal length of this larger transporter is approximately 30 ft. Since the aisles between the pads were previously 30 ft wide, the increased cask transporter diagonal length could involve contact with one or both pads on either side of the aisle. Both vendors recommended a minimum aisle width of 35 ft to enable the transporter to turn without interference with a pad edge. Therefore, it was decided to increase the aisle spacing between columns of pads from 30 ft to 35 ft. Reducing the 150 ft space that previously existed between the east and west pad quadrants to 35 ft allowed the aisle width to be increased to 35 ft between each of the 25 columns of pads with no change in the overall outer dimensions of the cask storage area or the location of the RA fence.

As a result of discussions with the cask transporter vendors, it was also decided not to construct the storage pads 3.5 inches above grade to accommodate potential settling. Rather, the pads will be constructed so that their tops are level with grade. Any settling of the storage pads is now expected to be minimal (1.75 inches maximum over 40 years), and would be addressed by scraping crushed aggregate from between pads so that the aggregate layer would be flush with the top of the pads, should the need arise.

Effects of the Revised Storage Cask/Pad Spacing

Following discussions with the cask transporter vendors, Stone & Webster recommended modifications to the cask/pad spacing and pad dimensions to project management. After the proposed modifications were evaluated, Holtec was assigned to assess the impact of the proposed changes on the storage cask array radiation dose rate analyses and thermal analyses which they had performed, since cask/pad spacing are an input to these analyses. Once the Holtec radiation dose rate analyses were completed and it was determined that array dose rates associated with the proposed cask/pad spacing would continue to comply with regulatory requirements, Stone & Webster revised its calculations that used the results of Holtec's storage cask array dose rate analysis as input. Stone & Webster also reanalyzed effects on PFSF construction of the revised storage pad layout, including impacts on soil cut and fill volumes, concrete volumes, soil cement volumes, quantities of imported materials, construction truck traffic and related noise levels, air pollution impacts of construction, and doses to workers that will construct pads while loaded storage casks are placed and stored in nearby quadrants. Calculations performed to assess impacts on PFSF construction are listed under a following subsection entitled "Quantities of Imported Construction Materials and Number of Truck Trips". Changes in the cask/pad spacing necessitated revisions to the following reports and analyses:

Holtec International

- Radiation Shielding Analysis for the Private Fuel Storage Facility (Revision 2), Holtec Report No. HI-971645.
- Additional Thermal Evaluation of the HI-STORM 100 System for Deployment at Skull Valley (Revision 1), Holtec Report No. HI-2002413.

Stone and Webster

- Calculation No. 05996.02–UR–5, Revision 2, entitled "Dose Rate Estimates from Storage Cask Inlet Duct Clearing Operations".
- Calculation No. 05996.02–UR(D)–8, Revision 1, entitled "Dose Rate Calculations at PFSF Locations Potentially Accessible to Wildlife and Estimates of Annual Doses to Individual Animals".
- Calculation No. 05996.02–UR(D)–11, Revision 1, entitled "Personnel Dose Rate Estimates During Construction of the Storage Pads at the Private Fuel Storage Facility".

- Calculation No. 05996.02–UR(D)–12, Revision 1, entitled "Dose Rates From the 4000 Storage Cask PFSF Array Representative of PFSF Typical Spent Fuel, Assumed to be PWR Fuel Having 35 GWd/MTU Burnup and 20 Year Cooling Time".

Adequacy of the PFSF Design with Regards to the Revised Storage Cask/Pad Spacing

Based on communication with the storage cask vendors, the 16 ft center-to-center storage cask spacing in the north-south direction has been determined to be adequate to enable efficient placement and pickup of storage casks during cask loading and unloading operations with the new generation cask transporters. In addition, the 35 ft spacing between columns of pads has been determined to provide adequate room for turning the cask transporters, which have a diagonal length of approximately 30 ft, without interference with the edge of a pad.

The radiation analysis of the PFSF array performed by Holtec, based on a full complement of 4,000 storage casks, determined that the design of the PFSF is adequate to ensure that the annual exposure to any real individual located beyond the controlled area boundary will be in compliance with the 25 mrem criterion specified in 10 CFR 72.104. The PFSF site-specific thermal analysis performed by Holtec for normal conditions of operation, which considered the revised cask spacing, determined that the peak cladding temperature would be below the HI-STORM SAR long-term maximum allowable cladding temperature.

Changes to the PFSF License Application Documents

The following is a list of the sections of the PFSF licensing documents that were updated to address the new cask/pad spacing and to incorporate the results of the revised reports and analyses identified above:

- Technical Specification Design Feature 4.2.3 was revised to specify the new storage cask spacing requirements.
- A number of figures were revised to show the new cask/pad spacing, such as the PFSF General Arrangement drawing that appears in the SAR, ER, and EP.
- Holtec reanalyzed dose rates at the RA fence, the owner controlled area boundary, and at the nearest residence (approximately 2 miles from the PFSF) using an assumed array of 4,000 HI-STORM storage casks based on the new cask/pad spacing (Holtec Report HI-971645, Revision 2). Maximum doses (at the north RA fence and OCA boundary) and doses at the nearest residence increased marginally (less than 5%) from the previous dose analysis. The results of this dose assessment are discussed primarily in SAR Section 7.3.3.5, with accompanying changes to SAR Sections 7.4 (Estimated Onsite Collective Dose Assessment) and 7.6 (Estimated Offsite Collective Dose Assessment). The dose rates calculated in Holtec's recent dose assessment were used to reevaluate offsite dose rates in ER Section

4.2.9.1.1, doses to construction workers in ER Section 4.1.9, and doses to wildlife postulated to spend time at the RA fence in ER Section 4.2.9.

- Holtec reevaluated the site-specific HI-STORM storage cask thermal performance based on the revised cask/pad spacing (Holtec Report HI-2002413, Revision 1). The results of this thermal assessment are discussed in SAR Section 4.2.1.5.2.
- Changing the length of the storage pads increased the volume of concrete associated with the pads, which impacted the quantity of imported solid construction materials (ER Table 4.1-6) as well as the water volumes drawn from the on-site well(s) during PFSF construction (ER Section 4.5.4). This change in concrete volumes due to the storage pad changes had a minor impact on the traffic during PFSF construction which relates to construction noise levels evaluated in ER Section 4.1.7 and air quality, discussed in ER Section 4.1.3.
- Chapter 4 of Appendix B to the License Application, "Decommissioning Cost Estimate", was revised to address the change in storage pad dimensions, which increases the pad surface area (by less than 5%) that could potentially require decontamination.
- The volume of earthwork, discussed in ER Section 4.1.5.2, was revised to account for changes associated with the new pad spacing/layout. This affected the fugitive dust emissions (Tables 4.1-4 and 4.1-5) and quantity of water required to be trucked in for soil compaction and dust control (ER Sections 4.1.7 and 4.5.4).
- Revisions to reflect the storage pads situated with the tops of the pads flush with grade were made to SAR Figure 4.2-7, SAR Sections 2.6.1.12.1 and 4.2.3.5.3, and to ER Section 3.1.

## REVISED CANISTER TRANSFER BUILDING DESIGN

Reasons for Changes to the Canister Transfer Building Design

Changes in the design of the Canister Transfer Building were made to ensure that the building is capable of withstanding the revised design basis ground motions, to ensure that the building can accommodate the larger cask transporters, to address crane hook approach requirements, and to enhance building constructability, as follows:

- Increased the area of the base mat, and increased the depth of the perimeter key from 1.0 ft to 1.5 ft. This was done to maintain the desired factor of safety against sliding and overturning for the increased seismic loads.
- Changed the strain-dependent soil properties beneath the building because of higher seismic accelerations.
- Provided, as a result of discussions with the transporter vendors, improved access to the transfer cells to avoid the 90-degree turns required with the original design to enter the transfer cells. Three additional doors were

incorporated into the West wall of the transporter aisle to make access into the transfer cells easier. It was decided that the tornado missile boundary should be moved from column line A.8 (transporter aisle west wall) to column line C (transporter aisle east wall). Since it is no longer needed as a missile barrier, the concrete wall on column line A.8 was replaced with a steel frame and metal sided wall, as was the wall on column line F (office area).

- Widened the doors in the west walls of each transfer cell from 20 ft to 22 ft, and increased the height to accommodate a potentially larger cask transporter.
- Moved the building north wall 5 ft in the north direction to accommodate crane hook approach requirements.
- Changed, as the result of a constructability review, the roof beams from reinforced concrete to structural steel. The roof slabs were reevaluated and the thickness reduced from 1 foot to 8 inches, while still satisfying tornado missile protection criteria. These changes will reduce construction time and cost.
- Increased the width of the transporter aisle by 7 ft to accommodate larger transporters.

Following is a description of the revised Canister Transfer Building design:

The CTB is a reinforced concrete structure in which the fuel canisters are transferred from the shipping casks to the storage casks. It is supported on a 5 ft thick rectangular base mat 279.5 ft long by 240 ft wide. There is a 1.5 ft deep shear key around the perimeter of the mat to help resist seismic sliding forces. The main portion of the building is 90 ft high and contains the three transfer cells. The perimeter shear walls are 2 ft thick and support the roof and the two cranes that are used in the canister transfer operations. The perimeter walls and the 8 inch thick roof provide tornado missile protection for the transfer cells. The reinforced concrete roof slab is poured on 1-1/2 inch deep metal decking. A steel frame supports the vertical roof dead weight, snow, and seismic loads. The roof decking spans are approximately 5 ft, and are supported by 16 inch deep steel beams. The 16 inch deep roof beams span up to 30 ft in the north-south direction to the main roof girders. Five ft deep main roof steel girders spanning 65 ft in the E-W direction carry the vertical roof loads to embedded plates set in the building's concrete walls. Horizontal seismic load from the roof mass is transferred to the building's walls by diaphragm action of the roof slab.

There are three openings (22 ft wide) through the wall on the west side of the transfer cells to allow cask transporter access to each transfer cell. These openings are tornado missile protected during canister transfer operations by 1-foot thick rolling doors fabricated from steel plate and concrete.

On the east and west sides of the main building are lower roofed areas that house the transporter aisle (on the west side) and an office area with mechanical rooms (on the east side). The rail bay (used for the off loading of shipping casks) extends out from the main portion of the building on both the east and west sides under the low roofed areas.

The low roofs are 30 ft above grade and are constructed of 8 inch thick reinforced concrete supported on a structural steel frame similar to the high roof except the main girders have shorter spans and are 36 inches deep. The low roofs provide lateral support for the N-S perimeter walls of the main building.

Effects of the Revised Canister Transfer Building Design

The proposed modifications to the design of the Canister Transfer Building were evaluated by project management, who assigned Stone & Webster to assess the effects of the proposed modifications and capability of the building to meet design criteria in light of the revised design basis ground motions. Changes in the Canister Transfer Building design necessitated generation of the following new calculations:

Stone and Webster

- Calculation No. 05996.02–SC-14, Revision 0, entitled "Design of Rolling Doors at Canister Transfer Cells".
- Calculation No. 05996.02–SC-12, Revision 0, entitled "Design of Canister Transfer Building Upper and Lower Roof Steel".

The Canister Transfer Building seismic analysis (Calculation No. 05996.02–SC-5, Revision 2) and the calculation that assessed the seismic stability of the Canister Transfer Building (Calculation No. 05996.02–G(B)-13, Revision 4) were performed as a result of the increased design basis ground motions, and evaluated the revised Canister Transfer Building design. In addition, revisions to two calculations are in progress and will be completed in the near future: S&W Calculation No. 05996.02–SC–6, entitled "Finite Element Analysis of Canister Transfer Building" is being revised to reflect the final design configuration of the building. The results of this analysis will be used to revise the calculation which determines the design of the building reinforcing steel - S&W Calculation No. 05996.02–SC–7, entitled "Design of Reinforcing Steel for Canister Transfer Building".

Adequacy of the Canister Transfer Building Design

As discussed in the previous section on revised design basis ground motions, the seismic analysis of the Canister Transfer Building (Calculation No. 05996.02–SC-5, Revision 2) determined the adequacy of the building structural design to withstand the revised design basis ground motions. Calculation No. 05996.02–G(B)-13, Revision 4, determined that the Canister Transfer Building design is adequate to ensure the stability of the building against sliding and overturning forces due to the revised design basis ground motions. Calculation No. 05996.02–SC-10, Revision 1, demonstrated that the design of the seismic restraints is adequate to support the casks in the Canister Transfer Building in the event an earthquake were to occur during canister transfer operations. Calculation 05996.02–SC-14, Revision 0, determined that the design of the east and west sliding doors of the three canister transfer cells is adequate to ensure these doors are capable of resisting the required loadings, which include the design basis ground motions for all doors and the tornado-missile loadings for the west doors.

Calculation 05996.02–SC-12, Revision 0, determined that the roof steel members are adequate to resist the required loadings while remaining within allowable stresses.

The Canister Transfer Building shall be designed for the load combinations and to the applicable codes and standards specified in Chapter 3 of the PFSF SAR (Principal Design Criteria). In regards to the revised finite element analysis and reinforcement design, SAR Section 4.7.1.5.3 states the following: "A detailed analysis of the final design configuration of the building will be performed using the ANSYS computer program (Reference 45) with a 3-dimensional finite element model. The results of this analysis will be used to design the reinforcing steel for the concrete walls, slabs, beams and columns (pilasters) of the building. This detailed analysis and design will be done for the load combinations for the building set forth in Section 3.2.11.4 and will be performed in accordance with the applicable codes and standards identified in Section 3.2.11.4..... In addition, the detailed ANSYS analysis will follow the same general approach as the ANSYS analysis previously performed for the conceptual design configuration of the building..... Some changes to the amount of reinforcing steel are anticipated, but it is expected that the results of the analysis and design will be similar to those for the conceptual design configuration for the building", which were previously submitted and reviewed by the NRC.

Changes to the PFSF License Application Documents

- SAR Chapter 3 provides a complete discussion of the principal design criteria for the CTB. The input criteria for the design basis ground motion is the only change to Chapter 3 that affects the CTB. All other criteria remain as previously reviewed and approved by the NRC in the SER.
- SAR Figures 4.1-1, 4.3-1, and 4.7-1, and ER Figure 3.1-5 were revised to reflect the changes in the Canister Transfer Building design. Figure 4.7-7 was revised to show the seismic support struts with the casks positioned 1 ft further from the west wall of the canister transfer cells to accommodate crane hook approach requirements.
- SAR Chapter 4, Section 4.7.1 provides a discussion of the CTB design including design specifications, design bases and safety assurance, structural design (design load combinations and analysis methods). Although changes have been made to the CTB design, the essential elements of the design remain as previously reviewed and approved by the NRC in the SER.
- SAR Section 4.7.1.5.3 was changed to indicate that the finite element analysis of the Canister Transfer Building that was performed with the ANSYS computer code for the previous building design (S&W Calculation No. 05996.02–SC–6, entitled "Finite Element Analysis of Canister Transfer Building") will need to be revised to reflect the final design configuration of the building. The results of this analysis will be used to revise the design of the building reinforcing steel (S&W Calculation No. 05996.02–SC–7, entitled "Design of Reinforcing Steel for Canister Transfer Building").

## OTHER MISCELLANEOUS CHANGES

As identified in Reference 2, the following miscellaneous changes were made in Amendment #22 to the PFSF License Application:

- Soil Cement
- RAI Incorporation
- Quantities of Imported Construction Materials and Number of Truck Trips
- Technical Specifications
- Chapter 1 of the License Application
- Environmental Permits

Of these changes, only the changes to the soil cement impacted the facility design. PFS assigned Stone & Webster to reevaluate the depth of the Eolian silt layer based on the most recent field test data (discussed below), and modify the soil cement design as necessary to ensure stability of the storage pads and Canister Transfer Building in light of the revised design basis ground motions. Geomatrix provided Holtec with information on the characteristics of the soil foundation of the storage pads for input to the cask seismic stability analysis performed by Holtec, which considered soil cement around the storage pads. Holtec accounted for the effects of the soil cement underlying the storage pads on the cask seismic stability analysis, and in their analyses of the hypothetical storage cask tipover event and postulated storage cask vertical end drop accident. CEC's analyses of the storage pads, and pad design, also accounted for the effects of soil cement around the storage pads.

**Soil Cement**

The surficial layer of eolian silt extends across the entire site, including the pad emplacement area and the area surrounding the Canister Transfer Building. The eolian silt, in its *in situ* loose state, is not suitable for founding the structures at the site. The basemat of the Canister Transfer Building will be founded on the silty clay/clayey silt layer beneath the eolian silt. It was originally intended that the cask storage pads also would be founded on the silty clay/clayey silt layer. However, instead of excavating and spoiling the eolian silt from the pad emplacement area and replacing it with structural fill (as necessary), the eolian silt will be mixed with sufficient portland cement and water and compacted to form a strong soil cement subgrade to support the cask storage pads. Soil cement will also be utilized around the Canister Transfer Building. The required characteristics of the soil cement will be engineered during detailed design and constructed to meet the necessary strength requirements.

Reasons for Changes in the Soil Cement Design

The discussion which follows provides the rationale for the changes in the depth of soil cement across the PFSF site and for changes in the soil cement strength requirements.

*Depth of Eolian Silt Layer and Soil Cement*

In January 2001, 16 test pits were excavated at the PFSF site in the pad emplacement area to obtain soil samples for use in the laboratory analyses necessary to design the soil cement. It was observed from these test pits that the depth of the eolian silt was shallower than previously believed (approximately 2 ft on average rather than 3 ft). The borings previously performed in this area obtained soil samples at depths from grade to 2 ft and from 5 ft to 7 ft. Therefore, as later observed in the test pits, the interface between the eolian silt and the silty clay/clayey silt fell between the samples collected in the borings. The soil unit descriptions from Trench T-2 in the pad emplacement area (Plate 3, Geomatrix "Fault Evaluation Study and Seismic Hazard Assessment") also corroborated the soil cement test pit observations; i.e., the fine sandy silt (eolian deposit) overlying the sandy clayey silt (combic B soil horizon developed on Bonneville deep-water sediment) is not expected to extend much deeper than approximately 2 feet from the ground surface. Furthermore, these observations are verified by Atterberg limits tests that have recently been performed on the samples obtained from these test pits, which indicate that those collected below depths of 2 ft are exclusively cohesive clayey silt/silty clay with high plasticity indices.

Our previous interpretation of the eolian silt boundary assumed that this boundary lay where the initial spike in the cone penetration tip resistance bottomed out. This assumption was made in order to obtain a conservative upper-bound estimate of the amount of soil cement required for the soil improvement of the noncohesive eolian silt. This increase in tip resistance was previously assumed to represent a layer of slightly cemented eolian silt. However, as observed in the January 2001 soil cement test pits and in Trench T-2, the interface between the noncohesive eolian silt and the cohesive clayey silt/silty clay more closely corresponds to the initial increase in the cone tip resistance, along with an accompanying steep increase in the sleeve skin friction resistance. These observations are consistent with the experience of soil classification using electric CPT data which indicate that sandy soils (noncohesive) tend to produce high cone resistance and low friction ratio, whereas soft clay soils (cohesive) tend to produce low cone resistance and high friction ratio (p.51, Lunne, Robertson and Powell, 1997). Therefore, it is expected that the transition from the noncohesive soil to the cohesive soil will be characterized by a steep increase of the cone skin friction resistance.

Based on the correlations and evaluations discussed above, the transitional boundary between the surficial noncohesive eolian silt and underlying cohesive clayey silt/silty clay presented in SAR Figure 2.6-5, Sheets 1 through 14, was re-interpreted to be consistent with the soil cement test pit observations and laboratory classification test results performed on soil samples from the test pits in the pad emplacement area. The interpretation also considered that the measurement of sleeve friction ($f_s$) is often less accurate and less reliable than the cone resistance (p.51, Lunne, Robertson and Powell, 1997). The boundary was re-interpreted based on consideration of the consistency between various cone penetration tests to obtain a smoothed boundary instead of interpreting each cone penetration test discretely. This re-interpretation of the eolian silt boundary reduced the estimated amount of eolian silt, resulting in the need for less soil cement under the cask storage pads (see SAR Figure 4.2-7).

During construction of the storage pads, all eolian silt in the quadrant under construction will be excavated. The eolian silt will be mixed with sufficient cement and water and compacted to produce soil cement across the pad area, up to the design elevations of the bottoms of the storage pads. The layer of soil cement beneath the storage pads will have a minimum thickness of 12 inches and a maximum thickness of 24 inches. In the event that the eolian silt layer extends to a depth greater than 2 ft below the elevations of the bottoms of the storage pads, compacted clayey soils will be used to raise the elevation of the subgrade that will support the soil cement layer to an elevation of 2 ft or less below the design elevations of the bottoms of the pads. This will ensure that the layer of soil cement does not exceed a thickness of 2 ft. This is the maximum permissible thickness of the soil cement layer, since the storage cask tipover and drop analyses (discussed below) were performed assuming a 2.0 ft thick layer of soil cement underlying the storage pads.

*Strength of Soil Cement and Minimum/Maximum Thickness Requirements*

The soil cement underlying the pads shall have a minimum unconfined compressive strength of 40 psi to ensure that there is an adequate factor of safety against sliding of an entire column of pads (S&W Calculation 05996.02-G(B)-04, Revision 7). This layer of soil cement is required to be no greater than 2 ft thick and have a static modulus of elasticity less than or equal to 75,000 psi to ensure that the decelerations from a hypothetical storage cask tipover event or vertical end drop accident do not exceed HI-STORM design criteria (SAR Section 3.2.11.3).

Following construction of the storage pads on top of this layer of soil cement, additional soil cement will be placed around and between the cask storage pads, extending from the bottoms of the pads to a level that is 28 inches above the bottoms of the storage pads. The remaining 8 inches, from the top of the soil cement up to grade, will be filled with coarse aggregate, placed and compacted to be flush with the tops of the pads to permit easy access by the cask transporter. The soil cement placed around the sides of the storage pads was required to have a minimum unconfined compressive strength of 340 psi to ensure sliding stability of individual pads (based on S&W Calculation 05996.02-G(B)-04, Revision 7, as discussed in SAR Section 2.6.1.12.1).

The Canister Transfer Building basemat will be founded on the silty clay/clayey silt layer that is below the eolian silt. The revised design calls for soil cement to be placed around the Canister Transfer Building base mat to make the free-field soil profile for the building consistent with that for the storage pad emplacement area and to help resist sliding forces due to the higher design basis ground motions. Soil cement will surround the foundation mat and will extend outward from the mat to a distance equal to the associated mat dimension; i.e., approximately 240 ft out from the mat in the east and west directions and approximately 280 ft out in the north and south directions. Existing soils (eolian silt and silty clay/clayey silt) will be excavated to a depth of approximately 5 ft 8 inches below grade, mixed with cement, and placed and compacted around the foundation mat. The soil cement placed around the Canister Transfer Building foundation mat will be 5 ft thick and have a minimum unconfined compressive strength of 250 psi to ensure that there is an adequate factor of safety against sliding of the Canister Transfer Building (based on Calculation 05996.02-G(B)-13, Revision 4, as discussed in SAR Section 2.6.1.12.2). The top 8 inches will be filled with compacted coarse aggregate, similar to that used in the pad emplacement area.

S&W Calculation 05996.02-G(B)-04, Revision 8, is discussed in Enclosure 1 of this letter, and is attached. This calculation shows that the soil cement design in License Application Amendment #22 is conservative and that the strength of the soil cement between pads can be reduced. The thickness requirements of soil cement under the storage pads and between the storage pads have not changed. To ensure an ample margin over the minimum shear strength required to obtain a factor of safety of 1.1 against sliding, the unconfined compressive strength of the soil cement underlying the storage pads is required to be a minimum of 40 psi, as previously specified. S&W Calculation 05996.02-G(B)-04, Revision 8, determined that soil cement is not required to be placed between the storage pads to resist sliding of the pads, because there is sufficient shear strength at the interfaces between the concrete pads and the underlying soil cement and between that soil cement layer and the underlying clayey soils so that the factor of safety against sliding during the design basis ground motions exceeds the minimum required value. Whereas soil cement between pads was previously required to have a minimum unconfined compressive strength of 340 psi to resist pad sliding during the revised design basis ground motions, pad stability is now ensured with no reliance on soil cement between pads. Nevertheless, the storage pads will be surrounded with soil cement to provide an adequate subbase for support of the storage cask transporter and so that PFS can effectively use the eolian silt at the site. The soil cement placed between the storage pads will provide additional margin against any potential pad sliding over that provided by the underlying soil cement layer. The unconfined compressive strength of the soil cement between the storage pads is required to be a minimum of 50 psi to provide an adequate subbase for support of the cask transporter. However, it will likely be at least 250 psi to satisfy the durability requirements associated with environmental considerations (i.e., freeze/thaw and wet/dry cycles) within the frost zone (30 inches from the surface of the ground). The actual unconfined compressive strength and mix requirements for the soil cement between the storage pads will be based on the results of standard soil cement laboratory tests. As noted in Enclosure 1, the results of S&W Calculation 05996.02-G(B)-04, Revision 8, will be used to revise the PFSF Licensing Application documents in the next license amendment.

<u>Effects of Changes in Soil Cement</u>

Changes in the soil cement resulted in generation/revision of the following analysis:

Holtec International

- Holtec International "PFSF Site-Specific HI-STORM Drop/Tipover Analyses, (Revision 0), Holtec Report HI-2012653.

Also, using the revised design basis ground motions, Stone & Webster developed calculations that assess the seismic stability of the storage pads and the Canister Transfer Building which involve soil cement (Calculation No. 05996.02–G(B)-04, Revisions 7 and 8, and Calculation No. 05996.02–G(B)-13, Revisions 4 and 5). The design of the soil cement was based on the results of these calculations.

Adequacy of the Soil Cement Design

The adequacy of the design of the soil cement surrounding and underlying the pads to ensure the sliding stability of the pads under seismic conditions is demonstrated by S&W Calculation 05996.02-G(B)-04, Revision 8. Revision 8 of this calculation determined that there is sufficient shear strength at the interfaces between the concrete pad and the underlying soil cement and between that soil cement layer and the underlying clayey soils that the factor of safety against sliding exceeds the minimum required value, with no credit for the soil cement placed between storage pads above the bottom of the pads. The underlying layer of soil cement is also required to have a static modulus of elasticity less than or equal to 75,000 psi to ensure that decelerations of a cask resulting from a hypothetical storage cask tipover event or vertical end drop accident do not exceed design criteria (SAR Sections 3.2.11.3 and 8.2.6). Holtec Report HI-2012653 demonstrated the adequacy of the pad design and the design of the soil cement underlying the pads to ensure decelerations of a storage cask do not exceed the 45g HI-STORM design criteria in the event of a hypothetical storage cask tipover event or vertical end drop accident, as discussed below.

The large extent of soil cement in the storage pad emplacement area allows the soil cement layer to be considered as part of the free field soil profile for the site response analyses. The properties of the soil cement, higher shear wave velocity and higher density than the existing soils in the area, help to minimize the response at the surface of the site caused by the design basis ground motions. Soil cement was added around the Canister Transfer Building foundation mat to make the free field soil profile for the building consistent with that for the storage pad emplacement area (as discussed in SAR Section 2.6.4.11), and to help resist sliding forces, in conjunction with the building's perimeter key, due to the revised design basis ground motions. The adequacy of this design feature is demonstrated in Calculation No. 05996.02–G(B)-13, Revision 5, which determined that the design of the soil cement surrounding the Canister Transfer Building (in conjunction with the building's perimeter key) is adequate to ensure the stability of the Canister Transfer Building under seismic conditions.

*Hypothetical Storage Cask Tipover and Vertical End Drop Events, Considering the Maximum Stiffness (Thickness and Elastic Modulus) of Soil Cement Underlying a Storage Pad*

The tipover and vertical end drop analyses documented in the HI-STORM FSAR assume a storage pad concrete thickness of 36 inches, a pad concrete compressive strength of 4,200 psi (at 28 days), reinforcement at the top and bottom (both directions) of the pad consisting of 60 ksi yield strength ASTM material, and a soil effective modulus of elasticity of 28,000 psi. The PFSF pads are 36 inches thick, the pad concrete compressive strength shall not exceed 4,200 psi (at 28 days), and the pad reinforcing steel is 60 ksi yield strength ASTM material. The soil foundation beginning not more than 2 ft below the ISFSI pad concrete has an effective soil Young's Modulus less than 28,000 psi. However, the soil cement mixture extending a maximum of 2 feet directly below the ISFSI pad has an effective Young's Modulus not to exceed 75,000 psi. To ensure that the HI-STORM storage cask 45g limit at the top of the fuel is met, PFSF site-specific cask tipover and vertical drop events were analyzed by Holtec International (Holtec Report 2012653) using the same methodology and computer codes used in the analyses discussed in the HI-STORM FSAR. The results of these analyses are discussed in PFSF SAR Sections 4.2.1.5.1E, and 8.2.6. The site-specific hypothetical tipover analysis demonstrated that the maximum deceleration at the top of the active fuel region is below the HI-STORM design basis value of 45g assuming 2.0 ft of soil cement underlying the storage pad with a static modulus of elasticity of 75,000 psi. The site-specific vertical end drop analysis determined that the maximum cask deceleration remained below 45g for a 9 inch drop height for the same soil cement conditions. These analyses demonstrated the adequacy of the design of the soil cement underlying the storage pads for HI-STORM storage cask tipover/end drop events. As a result of the vertical end drop analysis, PFSF Technical Specification 4.2.5, "Cask Transporter", was revised to require that the cask transporter be designed to mechanically limit the lifting height of a storage cask to a maximum of 9 inches (the previous maximum permissible lift height was 10 inches).

Holtec assumed in the PFSF site-specific analyses of storage cask tipover and drop events (Holtec Report 2012653) that the nominal 28 day compressive strength of the HI-STORM overpack concrete is 3,000 psi. This is lower than the 4,000 psi minimum concrete strength specified in the HI-STORM FSAR (Table 1.D.1), and also lower than the 4,200 psi cask concrete compressive strength assumed in the tipover and drop analyses in the HI-STORM FSAR. Assuming 3,000 psi cask concrete compressive strength for the PFSF site-specific tipover and vertical end drop analyses, Holtec Report 2012653 determined decelerations of 43.82g at the top of the active fuel region for a non-mechanistic tipover event, a deceleration of 36.15g for a vertical end drop from a height of 6.5 inches, and a deceleration of 45.15g for a vertical end drop from a height of 10 inches, as discussed in Section 8.2.6.2 of the PFSF SAR. Two additional HI-STORM storage cask concrete compressive strengths were evaluated in the tipover simulations in Holtec Report 2012653: 3,600 psi and 4,200 psi. For the hypothetical storage cask tipover event, the analyses determined maximum decelerations at the top of the active fuel region of 45.0g for the case with 3,600 psi cask concrete, and 45.9g for the case with 4,200 psi cask concrete. Holtec performed an evaluation of the effects of using 3,000 psi concrete in a HI-STORM storage cask (Reference 4) and determined that the only numerically significant use of concrete strength appears in the evaluation of the overpack resistance to the 8 inch diameter penetrant tornado missile. In Reference 4, Holtec states that the cask shielding effectiveness and thermal conductivity of concrete will not be affected by use of a reduced strength concrete, since the density of the concrete is inconsequentially affected by variations in the concrete strength (which is primarily a function of the water-cement ratio). Appendix 3.G in the HI-STORM FSAR details the tornado missile evaluation. Holtec performed a calculation for the 8 inch diameter penetrant tornado missile impacting the side of a HI-STORM storage cask assumed to have concrete with a 3,000 psi compressive strength, using the same methodology as HI-STORM FSAR Appendix 3.G. The calculation determined that a reduction in the compressive strength of the concrete will lead to a slightly larger depth of penetration than that identified in the HI-STORM FSAR. However, Holtec's calculation (Reference 4) demonstrated that the HI-STORM storage cask with 3,000 psi concrete provides an effective containment barrier for the canister after being subjected to a side missile strike, since the side concrete will not be penetrated by the missile and there will be no damage to the canister.

Changes to the PFSF License Application Documents

- Sections of the License Application documents revised to address changes related to soil cement include: Technical Specifications 4.2.5 and 5.5.4, SAR Figures 2.6-5, 4.2-7, and SAR Sections 2.6.1.6, 2.6.1.7, 2.6.1.11.4, 2.6.1.12.1, 2.6.1.12.2, 2.6.4.11, 3.2.10.1.7, 3.2.11.3, 4.2.1.5.1, and 8.2.6, ER Section 4.1.5.2, and EP Section 2.4.1.

**RAI Incorporation**

PFS's responses to the NRC's Third Round EIS Request for Additional Information (RAI), NRC Letter, M. Delligatti to J. Parkyn, dated October 24, 2000 were incorporated in the licensing documents, as applicable.

- ER Chapter 7 was updated to include the results of the cost benefit analyses performed in response to the RAI. These analyses account for changes to the PFS membership and the date when it is anticipated that the PFSF will become operational (the latter part of 2003). Several revisions were also made to ER Chapter 1 as a result of these analyses.
- Information regarding the proposed project schedule was updated in several sections of the licensing documents (SAR Section 1.1, ER Sections 1.3 and 3.2.1, and LA Section 1.8)
- ER Section 1.2 was updated regarding the remaining fuel assembly storage capacity in the PFS member fuel pools (accounting for changes in the PFS membership), and the projected dates for loss of full-core offload capability.
- ER Figure 2.5-2 was updated to reflect the latest information in the Utah Division of Water Rights database concerning water wells within 5 miles of the PFSF site.

The above changes had no impact on the PFSF design.

**Quantities of Imported Construction Materials and Number of Truck Trips**

Changes were made in the PFSF Environmental Report to the quantity of imported material and number of truck trips (imported solid material truck trips and water truck trips) required to support PFSF construction in ER Section 4.1.7 and Table 4.1-3. The imported material quantities that were substantially modified were the common fill material, water and cement needed to produce soil cement, materials needed to produce concrete (sand, large aggregate, and cement), and water that will be trucked-in for sprinkling to control fugitive dust emissions and water for soil compaction. Water for concrete production is obtained from the on-site well(s) and is not imported.

It was previously assumed that the common fill material would be imported. The current design utilizes a site earthwork balance where no common fill material is imported. The volume of soil affected during the various construction phases has changed, impacting the quantity of water that needs to be trucked-in for soil compaction and dust control. The overall concrete volume for the facility construction has increased as a result of the increase in size of the Canister Transfer Building base mat and increase in length of the storage pads. There has also been a net increase in the site soil cement quantities, primarily as a result of the emplacement of soil cement around the Canister Transfer Building.

Effects of Changes in Quantities of Imported Materials and Number of Truck Trips

The following calculations needed to be revised to address changes in the quantities of imported materials and the number of truck trips:

- The revised water requirements for PFSF construction were determined in S&W Calculation 05996.01-P-002, Rev. 5, entitled "Miscellaneous Design Data Required for PFSF Licensing Documents."
- The revised number of truck trips (imported material and water) was determined in S&W Calculation 05996.01-SY-7, Rev. 5, entitled "Truck Traffic Estimates on Skull Valley Road."
- The effect of the revised truck trips on traffic noise levels was determined in S&W Calculation No. 05996.01-E(B)-03, Rev. 3, entitled "Traffic/Sound Levels – Skull Valley Road Construction Thru Operation."
- The effect of the revised truck trips on air quality was determined in S&W Calculation No. 05996.01–E(B)-04, Rev. 4, entitled "Estimated PFSF Facility Construction Related Air Pollutant Emissions and Impacts".

Changes to the PFSF Licensing Documents

The following changes to the License Application documents were required as a result of the changes in the quantities of imported material and number of truck trips described above:

- Sections 4.1.7.1 through 4.1.7.3 of the ER discuss the effects of noise and traffic for the three construction phases of the PFSF. Truck trip quantities along with noise levels generated from the trips were revised in these sections. These values are also reflected in Tables 4.1.3 and 4.1.6 of the ER.
- Information regarding the volume of concrete production, quantities of earthwork affected, quantities of aggregate, and construction traffic levels associated with Phase 1 facility construction was used to revise information

on air pollution and air quality impacts in ER Section 4.1.3, and ER Tables 4.1-4 and 4.1-5.

- ER Section 4.5.2 was revised to clarify that concrete for the Intermodal Transfer Point would be obtained from commercial sources and no extra water for concrete production would need to be provided for by PFS. ER Sections 4.5.4 and 4.5.5 were updated to reflect the quantities of water needed for PFSF construction and operation, calculated in S&W Calculation 05996.01-P-002, Rev. 5. Section 4.5.5 provides a breakdown of water required to be trucked in and water that will be obtained from the onsite well(s). The lifetime average withdrawal rate from the onsite well(s) was updated, and the revised rate reflected in other sections of the Licensing Application documents. It was determined that no changes were needed to ER Section 4.5.7, "Radius of Influence for Proposed PFSF Water Well".

The changes associated with quantities of imported material and number of truck trips were the result of changes in construction planning and design changes to the storage pads, pad layout, and Canister Transfer Building, and had no impact on the PFSF design.

**Technical Specifications**

PFSF Technical Specification Design Feature 4.2.5, "Cask Transporter", prescribed that the cask transporter was to be designed such as to ensure that it does not begin to tip during the PFSF design basis ground motions. However, this was not consistent with the Technical Specification for the design basis tornado-driven missile for the cask transporter which utilized the drop height limitation. Therefore, for consistency this specification was revised to require that the cask transporter be designed to ensure that the transporter not tip over in the event of the PFSF design basis ground motions, and any tipping must be limited to ensure that the storage cask does not temporarily rise above its analyzed drop height of 9 inches. This now applies the same criteria to the design basis ground motions for the cask transporter that are specified for the design basis tornado-driven missile.

PFSF Technical Specification Design Feature 4.2.6, "Storage Pads", prescribed requirements for the storage pads to ensure that the pads and underlying soil are not harder than the reference storage pad upon which the design basis tipover and vertical end drop accidents are based in the HI-STORM FSAR. This specification was originally extracted from Appendix B, Section 3.4.6 of the HI-STORM 100 Cask System Certificate of Compliance (C of C) No. 72-1014. Holtec International revised the corresponding specification in their HI-STAR storage system C of C, and submitted a proposed amendment to this section of the HI-STORM C of C, which would permit site specific analyses to determine that the 45g deceleration HI-STORM design criteria is not exceeded for hypothetical storage cask tipover and postulated vertical end drop events. PFS revised Design Feature 4.2.6 accordingly, requiring that "The storage pads and underlying foundation shall be verified by analysis to limit cask deceleration during design basis drop and hypothetical tipover events to ≤45 g's at the top of the

CANISTER fuel basket. Analyses shall be performed using methodologies consistent with those described in the HI-STORM 100 FSAR." This change is reflected in SAR Section 3.2.11.3. Technical Specification 5.5.4, "Onsite Cask Transport Evaluation Program", was revised to be compatible with the revised Design Feature 4.2.6.

Technical Specification Design Feature 4.2.3 was revised to specify the new storage cask spacing requirements, as stated previously.

The changes to the Technical Specifications had no impact on the PFSF design.

### License Application Chapter 1

Certain information in Chapter 1 of the License Application was updated. For example, the list of the PFS Board of Managers was updated (Section 1.10) to be current. Similarly, the financial information was updated (Section 1.6) to correspond to the information presented by PFS in the licensing proceeding.

The changes to Chapter 1 of the License Application had no impact on the PFSF design.

### Environmental Permits

PFS updated Chapter 9 of the Environmental Report (Environmental Approvals and Consultation) to take into account the results of the wetland and stream survey conducted by PFS to determine if any jurisdictional waters of the United States are present along the proposed railroad alignment (PFS had committed to such an update in Reference 5). This survey concluded that there are no jurisdictional waters of the United States, wetlands or other kinds of water, along the proposed railroad alignment. PFS believes this survey along the rail corridor reflects the characteristics of the entire area around the facility, which has minimal drainage features as compared to the railroad alignment itself. Because of this determination, concurred in by the U.S. Army Corps of Engineers, various Federal and State permits required under Clean Water Act previously identified in Chapter 9 are not required. Chapter 9 was updated to reflect this determination and was generally updated as well to reflect PFS's current identification of required permits and status towards obtaining those permits.

The changes to Chapter 9 of the Environmental Report had no impact on the PFSF design.

## References


1. PFS letter, Parkyn to U.S. NRC, License Application Amendment #22, dated March 30, 2001.

2. PFS letter, Donnell to the U.S. NRC, Summary of Changes for PFSF License Application Amendment #22, dated April 16, 2001.

3. PFS letter, Donnell to the U.S. NRC, Response to April 18, 2001 Meeting Issues Regarding PFSF License Application Amendment #22, dated May 1, 2001.

4. PFS letter, Donnell to U.S. NRC, PFSF Site-Specific HI-STORM Evaluation, dated May 31, 2001.

5. PFS letter, Donnell to U.S. NRC, Responses to Third Round EIS Request for Information, dated November 7, 2000.

# Attachments

1. CD containing data in support of responses to Seismic Hazards Analysis data requests 1 and 2 and a diskette containing a brief user guide

2. Copyright Notice dated May 30, 2001 for Geomatrix computer program files

3. S&W Calculation No. 05996.02–SC-15, Revision 0, entitled "Additional Information for NRC Review"

4. Section 6 of the Geomatrix Fault Evaluation Study And Seismic Hazard Assessment, Rev 1, March 2001

5. Safety Analysis Report Figure 2.6-19, Locations of Cone Penetration, Test Pit and Dilatometer Tests

6. S&W Calculation No. 05996.02–G(B)-04, Revision 8, entitled "Stability Analysis of Storage Pads"

7. S&W Calculation No. 05996.02–G(B)-13, Revision 5, entitled "Stability Analysis of the Canister Transfer Building Supported on a Mat Foundation"

8. S&W Calculation No. 05996.02–SC-14, Revision 0, entitled "Design of Rolling Doors at Canister Transfer Cells"

9. S&W Calculation No. 05996.02–SC-12, Revision 0, entitled "Design of Canister Transfer Building Upper and Lower Roof Steel"

10. S&W Calculation No. 05996.02–G(B)-11, Revision 3, entitled "Dynamic Settlements of the Soils Underlying the Site"

11. S&W Calculation No. 05996.02–G(B)-21, Revision 0, entitled "Supplement to Estimated Static Settlement of Cask Storage Pads"

12. S&W Sketch 0599602-SKA-401A, floor plan of the Canister Transfer Building showing the location of the six areas with increased base mat bending.