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October 11, 2004

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, DC 20555

Subject: Response to Request for Additional Information (RAI) Letter No. 7 – Exelon Early Site Permit (ESP) Application for the Clinton ESP Site (TAC No. MC1122)

Re: Letter, U.S. Nuclear Regulatory Commission (N. V. Gilles) to Exelon Generation Company, LLC, (M. Kray), dated July 26, 2004, Request for Additional Information Letter No. 7 – Exelon Early Site Permit Application for the Clinton ESP Site (TAC No. MC1122)

Enclosed, as requested in the referenced letter, are responses to the requests for additional information (RAIs) associated with the seismology and geology portions of the Exelon Generation Company, LLC (EGC) ESP application. The attachments identified in the responses to specific RAIs are provided on the enclosed CD-ROM.

Please contact Eddie Grant of my staff at 610-765-5001 if you have any questions regarding this submittal.

Sincerely yours,

Marilyn C. Kray

Marilyn C. Kray
Vice President, Project Development

TPM/ERG

D073

cc: U.S. NRC Regional Office (w/ enclosures)
Ms. Nanette V. Gilles (w/ enclosures)

Enclosure: Response to RAI 2.5.1-1 through 2.5.1-5 (and associated attachments)
Response to RAI 2.5.2-1 through 2.5.2-7
Response to RAI 2.5.4-1 through 2.5.4-7

Attachments: (Note – These are provided on the enclosed CD-ROM.)
RAI 2.5.1-2 Attachment 1 (Obermeier pc 20021107)
RAI 2.5.1-2 Attachment 2 (Obermeier pc 20030110)
RAI 2.5.1-2 Attachment 3 (Obermeier pc 20030513)
RAI 2.5.1-2 Attachment 4 (Obermeier pc 20030809)
RAI 2.5.1-2 Attachment 5 (Olson 2003)
RAI 2.5.1-2 Attachment 6 (Olson et al 2004)
RAI 2.5.1-2 Attachment 7 (Green et al 2004a)
RAI 2.5.1-2 Attachment 8 (Green et al 2004b)
RAI 2.5.1-3 Attachment (Revised App B Figure 2.1-13)
RAI 2.5.1-4 Attachment (Revised App B Att 1 Figures B1-13-14-15)
RAI 2.5.2-2 Attachment (Revised App B Figure 2.2-2)
RAI 2.5.2-6 Attachment (Figures 1A-4A)
RAI 2.5.2-7 Attachment 1 (RPK Lyon 1997)
RAI 2.5.2-7 Attachment 2 (RPK Tokyo 2001)
RAI 2.5.4-1 Attachment 1 (Soil Property Info)
RAI 2.5.4-1 Attachment 2 (Revised App A Table 5-2)
RAI 2.5.4-2 Attachment 1 (EPRI Figures 7A-18&19)
RAI 2.5.4-2 Attachment 2 (EPRI Figure 6-9)
RAI 2.5.4-6 Attachment (Calc sample)

AFFIDAVIT OF MARILYN C. KRAY

State of Pennsylvania

County of Chester

The foregoing document was acknowledged before me, in and for the County and State aforesaid, by Marilyn C. Kray, who is Vice President, Project Development, of Exelon Generation Company, LLC. She has affirmed before me that she is duly authorized to execute and file the foregoing document on behalf of Exelon Generation Company, LLC, and that the statements in the document are true to the best of her knowledge and belief.

Acknowledged and affirmed before me this 11th day of October, 2004.

My commission expires 10-6-07.

Vivia V. Gallimore

Notary Public

COMMONWEALTH OF PENNSYLVANIA

Notarial Seal
Vivia V. Gallimore, Notary Public
Kennett Square Boro, Chester County
My Commission Expires Oct. 6, 2007

Member, Pennsylvania Association Of Notaries

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.1-1

In characterizing the seismic hazard of the New Madrid seismic source zone, SSAR Appendix B (Sections 2.1.5.2.1, 4.1.1.2, and Tables 2.1-3 and 4.1-1) cites the preferred magnitudes of Bakun and Hooper (2003, in press) for the New Madrid main shocks of 1811-12 as M 7.2, 7.1, and 7.4. The work of Bakun and Hooper (2003, in press) has since been revised with magnitude estimates for the 1811-12 New Madrid main shocks as M 7.6, 7.5, and 7.8 (Bakun and Hooper, 2004, Bull. Seism. Soc. Am., v. 94, no. 1, p. 64-75). Please explain what changes these revised magnitude estimates may require in the weights listed in Table 4.1-2 for the size of the expected characteristic earthquake rupture for each fault within the New Madrid seismic zone. In addition, please quantify the effect of the revised magnitude estimates on the hazard at the site by providing a graph showing two long-period (1 Hz) hazard curves, one using the magnitudes of Bakun and Hooper (2003, in press) and the second one using the magnitudes of Bakun and Hooper (2004).

EGC RAI ID: R7-1

EGC RESPONSE:

Subsequent to the submittal of the SSAR, two new articles relating to the location and estimated magnitude of the New Madrid 1811-1812 mainshocks have been published. These include the final version of the Bakun and Hopper article referred to in this RAI, as well as an article by Mueller et al. (2004). The review of these two new publications together with recent communications with the researchers indicates that there still remains uncertainty and differing views within the research community regarding the size and location of the 1811-1812 earthquakes. As described below, suggested changes due to the effects of the revision to the distributions for the maximum magnitude estimate for the New Madrid characteristic earthquakes is presented. The results of the requested sensitivity analyses to test the impact of different characteristic magnitude assessments on the median and mean rock hazard for 1 Hz spectral acceleration at the EGC ESP Site are shown in Figure 2.5.1-1-1 and discussed below.

Bakun and Hopper (2004) provide preferred estimates of the locations and moment magnitudes and their uncertainties for the three largest events in the 1811-1812 sequence near New Madrid. Their preferred intensity magnitude $\mathbf{M}_I$, which is their preferred estimate of **M**, is 7.6 (6.8 to 7.9 at the 95% confidence interval) for the 16 December 1811 event (NM1), 7.5 (6.8 to 7.8 at the 95% confidence interval) for the 23 January 1812 event (NM2), and 7.8 (7.0 to 8.1 at the 95% confidence interval) for the 7 February 1812 event (NM3). The intensity magnitude $\mathbf{M}_I$ is the mean of the intensity magnitudes estimated from individual MMI assignments. In their analysis, Bakun and Hopper (2004) consider two alternative eastern North America (ENA) intensity attenuation models, which they refer to as models 1 and 3. As indicated in the table below, these two models give significantly different results for larger magnitude earthquakes. Bakun and Hopper (2004) state that because these models are empirical relations based almost exclusively on **M** <6 calibration events "There is no way to confidently predict which relation better represents the MMI-distance data for **M** 7 earthquakes in ENA" (p. 66). They present arguments supporting their preference for model 3, but do not discount the results based on model 1.

Dr. Susan Hough (personal communication, 22 August, 2004) believes that there are insufficient data regarding the calibration of ENA earthquakes larger than **M** > 7 to rely strictly on ENA models as was done in the Bakun and Hopper (2004). She offers arguments to support **M** 7.6 (the size of the 2003 Bhuj earthquake) as a reasonable upper bound for the largest of the earthquakes in the 1811-1812 New Madrid earthquake sequence, which is more consistent with the estimates cited in Hough et al. (2000) and Mueller et al. (2004).

Mueller et al. (2004) use instrumentally recorded aftershock locations and models of elastic stress change to develop a kineamatically consistent rupture scenario for the mainshock earthquakes of the 1811-1812 New Madrid sequence. In general, the estimated magnitudes for NM1 and NM3 used in their analysis (M=7.3 and M=7.5, respectively) are consistent with those previously published by Hough et al. (2000). Their results suggest that the mainshock events NM1 and NM3 occurred on two contiguous faults, the strike-slip Cottonwood Grove fault and the Reelfoot thrust fault, respectively. The locations of the NM1 and NM3 events on the Cottonwood Grove and Reelfoot faults, respectively, are relatively well constrained. In contrast to the earlier Hough et al. (2000) study that located the NM2 earthquake on the New Madrid north fault, they suggest a more northerly location for the NM2 event, possibly as much as 200 km to the north in the Wabash Valley of southern Indiana and Illinois. Using Bakun and Wentworth's (1997) method, Mueller et al. (2004) obtain an optimal location for the NM2 mainshock at 88.43°W, 36.95°N and a magnitude of 6.8. They note that the location is not well constrained and could be fit almost as well by locations up to 100 km northwest or northeast of the optimal location. Mueller et al. (2004) conclude that the three events on the contiguous faults increased stress near fault intersections and end points, in areas where present-day microearthquakes have been interpreted as evidence of primary mainshock rupture. They note that their interpretation is consistent with established magnitude/fault area results, and do not require exceptionally large fault areas or stress drop values for the New Madrid mainshocks.

With respect to the location of the NM2 event, Bakun and Hopper (2004) also discuss the paucity of MMI assignments available for this earthquake to the west of the NMSZ and the resulting uncertainty in its location. They note that the two MMI sites closest to the NMSZ provide nearly all of the control on the location of this event and that based on these two sites, a location northeast of their preferred site would be indicated. However, they conclude that the lack of 1811-1812 liquefaction observations in western Kentucky, southern Illinois, and southern Indiana preclude an NM2 location in those areas. Bakun and Hopper (2004) follow Johnston and Schweig (1996) in selecting a preferred location on the New Madrid north fault. S. Obermeier confirmed the statement regarding the absence of liquefaction features in the Wabash Valley region that would support the more northerly location preferred by Mueller et al. (2004) (Dr. Steve Obermeier, personal communication, 24 August 2004). He noted that he had looked specifically in the area cited in the Yearby Land account that was cited by Mueller et al. (2004) and observed evidence for only small sand blows and dune sands, but did not see features of the size and origin described in that account.

Finally, recently Dr. Arch Johnston (personal communication, 31 August 2004) indicates that the estimates of Johnston (1996) are likely to be high by about 0.2 to 0.3 magnitude units. Dr. Johnston indicates that he is working on developing revised estimates for a forthcoming paper.

TABLE 2.5.1-1-1

MAGNITUDE COMPARISONS FOR NEW MADRID
1811-1812 EARTHQUAKE SEQUENCE

Study	NM1	NM2	NM3
Johnston (1996)	M 8.1 +/- 0.3	M 7.8 +/- 0.3	M 8.0 +/- 0.3
Hough et al. (2000)	M 7.2 to 7.3	M ~7.0[1] (located on the New Madrid north fault)	M 7.4 to 7.5
Mueller and Pujol (2001)	-	-	M 7.2 to 7.4 (preferred M 7.2 to 7.3)
Bakun and Hopper (2004)	M_I 7.6 (M 7.2 to 7.9) (preferred model 3)	M_I 7.5 (M 7.1 to 7.8) (preferred model 3)	M_I 7.8 (M 7.4 to 8.1) (preferred model 3)
	M_I 7.2 (M 6.8 to 7.9) (model 1)	M_I 7.2 (M 6.8 to 7.8) (model 1)	M_I 7.4 (M 7.0 to 8.1) (model 1)
Mueller et al. (2004)	M 7.3	M 6.8 (located within the Wabash Valley of southern Illinois/ southern Indiana)	M 7.5
Johnston (personal communication, 31 August 2004)	M 7.8-7.9	M 7.5-7.6	M 7.7-7.8

[1] The estimated location and magnitude of this earthquake are revised in Mueller et al. (2004).

The review of these two new publications together with discussions with the researchers (written communications from Dr. S. Hough, 22 August 2004; Dr. W. Bakun, 15 August 2004; and Dr. A. Johnston, 31 August 2004) indicates that there still remains uncertainty and differing views within the research community regarding the size and location of the 1811-1812 earthquakes. Based on our review of these new articles and communications with Drs. Bakun, Hough, and Johnston, a suggested revision to the maximum magnitude assessments for the New Madrid central fault system faults is as follows:

- Equal weight (1/3) is be given to estimates based on Bakun and Hopper (2004) and Hough et al. (2000)/Mueller et al. (2004), and the Johnston (personal communication, 31 August 2004) revisions to Johnston (1996).
- For the Bakun and Hopper (2004) estimate, we consider results from using both intensity attenuation relations (models 1 and 3). Based on Bakun and Hopper's preference for model 3, we assign weights of 0.75 to model 3 and 0.25 to model 1.
- In the case of the Hough et al. (2000)/Mueller et al. (2004) estimates and the Johnston (personal communication, 31 August 2004) estimates, we assign equal weight to the range of preferred values given for each earthquake.

The resulting characteristic magnitude distribution for each of the three faults is given in the following table. Rupture sets 1 and 2 correspond to the revised Johnston (1996) estimates, rupture sets 3 and 4 correspond to the Bakun and Hopper (2004) estimates, and rupture sets 5 and 6 correspond to the Hough et al. (2000) estimates.

TABLE 2.5.1-1-2

SUGGESTED REVISION TO THE MAGNITUDE DISTRIBUTIONS FOR CHARACTERISTIC NEW MADRID EARTHQUAKES

Characteristic Earthquake Rupture Set	Characteristic Magnitude for Individual Faults (moment magnitude [M])			Weight
	New Madrid South	Reelfoot Thrust	New Madrid North	
1	7.8	7.7	7.5	0.1667
2	7.9	7.8	7.6	0.1667
3	7.6	7.8	7.5	0.25
4	7.2	7.4	7.2	0.0833
5	7.2	7.4	7.0	0.1667
6	7.3	7.5	7.0	0.1667

The results of the requested sensitivity analyses to test the impact of different characteristic magnitude assessments on the median and mean rock hazard for 1 Hz spectral acceleration at the EGC ESP Site are shown in Figure 2.5.1-1-1. Five hazard curves are shown on each plot. The curves labeled "EGC ESP application" are taken from Appendix B of the EGC ESP Application. The curves labeled "Bakun & Hopper (2003, in press)," "Bakun & Hopper (2004) - Model 1," and "Bakun & Hopper (2004) - Model 3" were computed using only the indicated magnitude estimates for each of the New Madrid faults. The curves labeled "New distribution" were computed using the suggested updated distribution given in Table 2.5.1-1-2. The new distribution produces approximately 3 to 4 percent higher ground motions at the mean 10^{-4} and mean 10^{-5} hazard levels. These results were computed using the recurrence model and rupture sequences developed in the EGC ESP Application. The effects of alternative recurrence estimates and rupture sequences are presented in the response to RAI 2.5.2-5.

New References

Bakun, W.H. Personal communication. 15 August 2004.

Bakun, W.H., and M.G. Hopper. "Magnitudes and Locations of the 1811-1812 New Madrid, Missouri, and the 1886 Charleston, South Carolina, Earthquakes." *Bulletin of the Seismological Society of America.* Vol. 94, No. 1. Pp. 64-75. 2004

Bakun, W.H., and C.M. Wentworth. "Estimating Earthquake Location and Magnitude From Seismic Intensity Data." *Bulletin of the Seismological Society of America.* Vol. 87, pp. 1502-1521. 1997.

Hough, S. Personal communication. 22 August 2004.

Johnston, A.C. Personal communication. 31 August 2004.

Mueller, K., S. E. Hough, and R. Bilham. "Analysing the 1811-1812 New Madrid earthquakes with recent instrumentally recorded aftershocks." *Nature.* Vol. 429, pp. 284-288. 2004.

Obermeier, S. Personal communication. 24 August 2004.

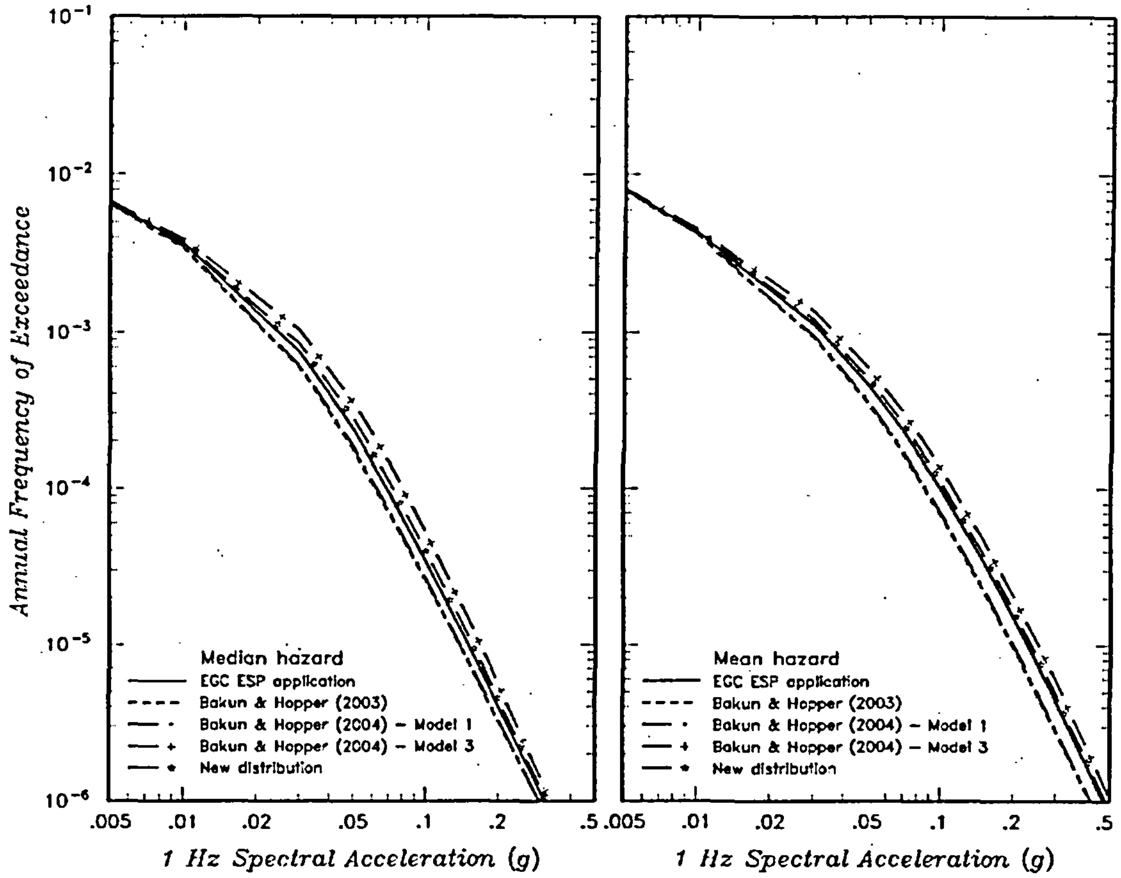

Figure 2.5.1-1-1 Median and Mean Seismic Hazard Curves for 1-Hz Spectral Acceleration Developed Using Alternative Distributions for Characteristic New Madrid Earthquakes

ASSOCIATED EGC ESP APPLICATION REVISIONS:

None

ATTACHMENTS:

None

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.1-2

In characterizing the maximum-magnitude distribution for the Wabash Valley-Southern Illinois seismic source zone, SSAR Appendix B (Sections 2.1.5.2.2, 4.1.2, and Table B.1-1) cites electronic and personal communications from S.F. Obermeier and a U.S. Geological Survey (USGS) Open-File Report, which is in preparation. Please provide a copy or detailed summary of these communications and Open-File Report in order for the staff to evaluate the assumptions and conclusion made in SSAR Appendix B concerning the characterization of the Wabash Valley-southern Illinois seismic source zone.

EGC RAI ID: R7-2

EGC RESPONSE:

The requested documents are provided in the Attachment to this RAI response.

Note that the Olson et al. (2003) Open-File Report cited in the SSAR Appendix B was revised in January 2004. This revised version is also attached and available on the web at the address listed below. Also included as attachments to this RAI response are two papers by Green et al. These papers are also available at the identified web links. The 2004a paper provides a summary of the results of the Open-File Report by Green, Obermeier, and Olson (in preparation) that was previously cited in the SSAR Appendix B. The 2004b Green et al. Open-File Report (in preparation) is a University of Michigan Department of Civil and Environmental Engineering report. The new Green et al. (2004a and 2004b) publications support general statements previously used to evaluate the maximum magnitude for the Wabash Valley-Southern Illinois source zones, and therefore, no changes have been made to the probability distribution for the maximum magnitude for these zones.

New References

Olson, S.M., R.A. Green, and S.F. Obermeier. "Geotechnical Analysis of Paleoseismic Shaking Using Liquefaction Features: Part I. Major Updating of Techniques for Analysis." U.S. Geological Survey Open-File Report 03-307. Version 1.1. Available at: http://pubs.usgs.gov/of/2003/of03-307/of03-307.pdf. Revised January 30, 2004.

Green, R.A., S.F. Obermeier, and S.M. Olson. "The Role of Paleo-Liquefaction Studies in Performance-Based Earthquake Engineering in the Central-Eastern United States." 13th World Conference on Earthquake Engineering, Vancouver, BC, Canada. Paper No. 1643. 14 p. Available at: http://wwwpersonal.engin.umich.edu/~rugreen/papers/WCEE.pdf. August 1-6, 2004a.

Green, R.A., S.F. Obermeier, and S.M. Olson. "Geotechnical Analysis of Paleoseismic Shaking Using Liquefaction Features: Part II. Field Examples." Report No. UMCEE04-08. Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, MI. Available at: http://www.personal.engin.umich.edu/~rugreen/papers/UMCEE0408.pdf. 2004b.

ASSOCIATED EGC ESP APPLICATION REVISIONS:

Revise SSAR, Appendix B, Chapter 6, References, to include the following new reference:

Obermeier, S.F. U.S. Geological Survey, Emeritus. Reston, Virginia. EqLiq Consulting. Written (electronic mail) communication to Kathryn Hanson. November 7, 2002.

ATTACHMENTS:

RAI 2.5.1-2 Attachment 1 (Obermeier pc 20021107)

RAI 2.5.1-2 Attachment 2 (Obermeier pc 20030110)

RAI 2.5.1-2 Attachment 3 (Obermeier pc 20030513)

RAI 2.5.1-2 Attachment 4 (Obermeier pc 20030809)

RAI 2.5.1-2 Attachment 5 (Olson 2003)

RAI 2.5.1-2 Attachment 6 (Olson et al 2004)

RAI 2.5.1-2 Attachment 7 (Green et al 2004a)

RAI 2.5.1-2 Attachment 8 (Green et al 2004b)

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.1-3

SSAR Figure 2.1-13 in Appendix B shows earthquake locations in the study area (southern Illinois and Indiana, western Kentucky, and eastern Missouri) for the period 1974 to 1987. However, the ESP site is not shown in Figure 2.1-13. Please revise Figure 2.1-13 to show the ESP site together with the earthquake epicenters, using a projection that can easily be related to other figures.

EGC RAI ID: R7-3

EGC RESPONSE:

SSAR, Appendix B, Figure 2.1-13 has been revised (using the EPRI-SOG 1777-1985 catalog updated through August 1, 2002 seismicity plot as a base) to show the earthquake epicenters and the EGC ESP Site location. The surface-wave focal mechanisms for historical events as reported by Herrmann (1979) and Taylor et al. (1989) are shown. In addition, focal mechanisms for two recent earthquakes, the 18 June 2002 and 28 June 2004 earthquakes also are shown as reported in the Saint Louis University Earthquake Center website (http://www.eas.slu.edu/Earthquake_Center/NEW/mechanism.html).

New References:

Saint Louis University Earthquake Center (SLUEC) "Recent Earthquake Locations and Mechanisms. Available at: http://www.eas.slu.edu/Earthquake_Center/NEW/mechanism.html. 2004.

ASSOCIATED EGC ESP APPLICATION REVISIONS:

Revise SSAR, Appendix B, Chapter 6, Reference, to include the following additional reference:

Saint Louis University Earthquake Center (SLUEC) "Recent Earthquake Locations and Mechanisms. Available at: http://www.eas.slu.edu/Earthquake_Center/NEW/mechanism.html. 2004.

Revise SSAR, Appendix B, Chapter 2, Figure 2.1-13 as provided in the attachment to this RAI response.

ATTACHMENTS:

RAI 2.5.1-3 Attachment (Revised App B Figure 2.1-13)

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.1-4

SSAR Figures B-1-13, 14, 15 of Attachment 2 of Appendix B show paleoliquefaction features at three different locations. Some of the features described in Section 1.4 of Attachment 2 of Appendix B are not readily visible in these three figures. Please provide sketches or better labeled figures to allow the staff to identify the location and extent of the sand dikes up into the silt.

EGC RAI ID: R7-4

EGC RESPONSE:

SSAR Figures B-1-13, 14, 15 of Attachment 1 (note that the reference to Attachment 2 in the RAI is incorrect) of Appendix B have been revised to provide better annotation of the location and extent of the sand dikes up into the silt.

ASSOCIATED EGC ESP APPLICATION REVISIONS:

Replace SSAR, Appendix B, Attachment 1, Figures B-1-13, -14, & -15 with new figures provided in the Attachment to this RAI response.

ATTACHMENTS:

RAI 2.5.1-4 Attachment (Revised App B Att 1 Figures B1-13-14-15)

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.1-5

SSAR Attachment 1 to Appendix B summarizes Obermeier's paleoliquefaction studies and the recent Geomatrix field reconnaisance. The energy centers and their associated paleoliquefaction features from these studies are listed in Table B-1-1. Given the heterogeneous nature of till deposits, please explain how dike size can be reliably used to estimate the locations of the paleo-energy centers. In addition, the geo-environment (i.e., ground water level, compaction and overburden pressures) may have been significantly different from current conditions. Please explain how these potential differences in the environment are accounted for in the cyclic stress method, used by Obermeier to estimate the magnitude of paleoearthquakes.

EGC RAI ID: R7-5

EGC RESPONSE:

Obermeier et al. (2001, 2002) provides a discussion of issues related to identifying the region of strongest bedrock shaking, referred to as the "energy center" or "source region," based on paleoseismic studies. The abundance and width of dikes observed provide information that can be used to estimate the level of shaking at an individual site, which in turn is used in conjunction with more regional data on the spatial pattern and distribution of dike size to estimate the location of energy centers for prehistoric earthquakes. With respect to the applicability and use of dike width in continental settings typical of the EGC ESP site region, Obermeier et al. (2002, p. 20) notes that:

> Dike width serves as a superior parameter to locate the source region in many field settings (Obermeier, 1996a). This width generally reflects the amount of lateral spreading except where dikes are relatively small (say, less than 10 cm wide). Conceptual verification for using dike width to locate the source region is provided from a study of historical earthquakes by Bartlett and Youd (1992). Dike width works well because the development and magnitude of lateral spreading are largely independent of thickness and strength of the cap, at least for sediments that are typical of the Holocene and late Pleistocene. Maximum dike width and the sum of dike widths at a site appear to work equally well to estimate the source zone (Munson et al., 1995). A valid interpretation based on the widths of dikes obviously requires that bank erosion has not been so severe as to have destroyed dikes by lateral spreading. Problems of interpretation due to erosion are generally not serious in the meizoseismal zone of a very large magnitude earthquake because of the tendency for large lateral spreads to develop even relatively far from the stream banks.
>
> Data from historical earthquakes in the Wabash Valley region, in the forms of modified Mercalli intensities and instrumentally located epicenters (Rhea and Wheeler, 1996), suggest that using liquefaction features to locate the source region of prehistoric earthquakes is generally accurate to within a few tens of kilometers, at least for earthquakes of moderate size. The uncertainty in location probably increases with increasing magnitude because of the tendency for the epicenter of larger earthquakes to be farther removed from the area of strongest shaking (e.g., Youd, 1991). Still, it appears that the distribution and severity of liquefaction effects can be used to reasonably estimate region of strongest bedrock shaking (Pond, 1996 *[published as Pond and Martin, 1996]*).

Obermeier et al. (2002, p. 20) addresses the issue of whether paleoliquefaction features resulted from a single large earthquake or from a series of small earthquakes that were closely spaced in time by concluding that:

> The answer is generally best resolved by analysis of the regional pattern of dike widths. The attenuation pattern of maximum dike widths around a core region should be examined in orthogonal coordinates (preferably along the suspected fault axis and perpendicular to the axis). A monotonic decrease of maximum dike width in orthogonal directions around the suspected core indicates a single large earthquake. In the Wabash Valley, this approach was verified by geotechnical back-calculations of the prehistoric strength of shaking for four prehistoric earthquakes (Pond, 1996 *[published as Pond and Martin, 1996]*). The use of dike widths alone to resolve the issue of the number of events requires that the liquefaction susceptibility be reasonably uniform on a regional basis and also that the amplification or attenuation of bedrock motions be similar on a regional basis.

The more recent analysis by Green et al. (2004a and 2004b) [provided with RAI 2.5.1-2] confirm that the general conclusion of Munson et al. (1995, 1997) regarding the correlation between the spatial pattern of dike size and abundance and the location of the energy center.

The question posed in the RAI, addresses concerns regarding the uniformity and quantity (or lack thereof) of susceptible sediments in the study region. Obermeier et al. (2001, 2002) concludes that deposits of latest Pleistocene and Holocene age that have been laid down by moderate to large streams in the central and eastern United States are generally moderately susceptible. They note that this level of susceptibility applies to streams of both glaciofluvial braid-bar and Holocene point-bar origin. Deposits mapped as the Henry Formation fall into this category and are mapped along most of the major streams in the site region (Lineback, 1979). Dr. Obermeier's past mapping experience working along drainages of similar size and in the same geologic environment throughout central and southern Illinois was invaluable in providing confidence that there were comparable exposures of deposits of sufficient ages along the reaches of the drainages mapped by this study to make comparisons to the other areas where larger and more numerous paleoliquefaction features of inferred seismic origin have been mapped. Documentation of the types of deposits observed along the reaches of rivers examined during the field reconnaissance for this study is provided in the response to RAI 2.5.2-6.

There is uncertainty in estimating the energy source and magnitude of paleoearthquakes based on paleoliquefaction studies, particularly in the absence of site-specific geotechnical data and analysis of multiple sites related to a common paleoearthquake. The various factors that may influence the distribution of features and the overall completeness of the record from a paleoearthquake were discussed in Section 1.3 of Attachment 1 to SSAR Appendix B. For this reason, alternative models were used to incorporate the paleoliquefaction data into the source characterization models for probabilistic seismic hazard analysis (PSHA). For the hazard analysis, it was not assumed that the preferred location of the energy centers for paleoearthquakes as shown by McNulty and Obermeier (1999) were fixed. The alternative source models from the EPRI-SOG (1989-1991) study combined with the revised maximum magnitude distribution used to characterize those zones allow for uncertainty in the location and size of the Vincennes and Shoal Creek events. Despite the evidence for localization of a energy source in the Springfield area (i.e., the spatial pattern and possible evidence for two separate events), the Springfield event was not fixed at the inferred energy center for the middle Holocene event.

The second part of this RAI, requests additional clarification regarding how potential differences in the geo-environment (i.e., ground water level, compaction, and overburden pressures) are accounted for in using the cyclic stress method to evaluate the size of paleoearthquakes. The Olson et al. (2003, revised January 2004) [provided with RAI 2.5.1-2] and Green et al. (2004b) papers discuss these issues at length and provide recommendations for accounting for uncertainties related to these factors in analyses to back calculate the strength of earthquake shaking at individual sites. Recognizing that there may be considerable uncertainty in the estimate of peak ground acceleration and earthquake magnitude based on the analysis of a single paleoliquefaction site, that may result from unusual bedrock shaking, undefined conditions affecting the strength of shaking in alluvium (e.g., the presence of a clay layer located at depth amplifying the ground motion), in-situ test data that inconclusively define a representative value, and other factors, Olson et al. (2003, revised January 2004) recommend that individual back-calculations be integrated into a regional assessment. They note that besides allowing investigators to identify potentially anomalously back-calculation results, a regional assessment provides a means to qualitatively evaluate the effects of post-earthquake density change and aging.

In the Green et al. (2004b) paper, the authors provide case studies that illustrate the methods outlined in Olson et al. (2003, revised January 2004) for addressing these uncertainties. The Vincennes earthquake case study described in this paper illustrates the methodology, including: the appropriate selection of soil parameters for assessing liquefaction susceptibility by using field observations in sectional view at sites located in both the meizoseismal zone and beyond; and the integration of data from multiple sites into a regional assessment of the strength of shaking of a paleoearthquake, even when uncertainty exists regarding whether all the features analyzed resulted from the same causative earthquake. To qualitatively rank the geologic data quality, the authors introduce a "field data quality" (FDQ) index which relates to the quality of and confidence in geologic interpretations at an individual study site, incorporating the following factors: (1) variability of geologic setting (e.g. braid-bar, point bar, etc.); (2) depth of potential source beds at the time of the earthquake; (3) depth of the groundwater table at the time of the earthquake; (4) mechanism of ground failure (e.g., hydraulic fracturing, lateral spreading, surface oscillation); and (5) severity of liquefaction, as it relates to making proper field interpretations. They note that the quality of and confidence in the geologic interpretations are influenced by a number of factors, including the number, spacing, and locations of in-situ borings or tests; the vertical and lateral variability of sediments at the site; the method of observation (i.e., plan view versus sectional view); and the length and quality of the bank exposure at the site. These factors are discussed in detail in Appendix II to the Green et al. (2004b) paper.

New References

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ASSOCIATED EGC ESP APPLICATION REVISIONS:

None

ATTACHMENTS:

None

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.2-1

SSAR Section 2.5.2 describes the results of Exelon Generation's (Exelon's) determination of ground motion at the ESP site from possible earthquakes. Regulatory Guide (RG) 1.165 provides a method acceptable to the NRC staff with respect to the probabilistic evaluations that can be conducted to address the uncertainties associated with the Safe Shutdown Earthquake (SSE) determination. RG 1.165 specifies a target or reference probability (median 10^{-5} per year) that is used to determine the controlling earthquakes and subsequent site ground motion.

Please provide the following information related to the approach used to obtain the results in Section 2.5.2:

a) The approach described in SSAR Section 2.5.2 uses a Uniform Hazard Spectrum (UHS) at the mean 10^{-4} per year probability level as its starting point. Please justify the selection of mean 10^{-4} per year as the appropriate starting point.

b) Please provide site-specific response spectra from the controlling earthquakes at the reference probability level (median 10^{-5} per year) and demonstrate that the SSE envelops the response spectra from the controlling earthquakes at the reference probability level, or justify why this information is not needed in determining the site-specific SSE. Please also justify any reference probability level used other than median 10^{-5} per year. Appendix B to RG 1.165 discusses situations in which an alternative reference probability level may be appropriate.

c) The approach described in SSAR Section 2.5.2 incorporates component capacity or performance parameters into a scale factor used to compute the final SSE. Please justify the incorporation of equipment performance into determination of the final SSE.

EGC RAI ID: R7-6

EGC RESPONSE:

EGC RESPONSE TO ITEM a):

The first part of this RAI requests additional justification on the selection of mean 10^{-4} per year probability level as the appropriate starting point for the determination of the Safe Shutdown Earthquake (SSE). This starting point is associated with the ASCE/SEI Standard 43-05[1] (draft referenced as (ASCE, 2003) in the EGC SSAR Section 2.5), *Seismic Design Criteria for Structures, Systems and Components in Nuclear Facilities and Commentary*, performance-based approach used within the EGC ESP Application, as summarized in Section 2.5.2 and Appendix B to the EGC ESP SSAR. The technical basis for using the mean 10^{-4} per year probability level to implement the ASCE/SEI Standard 43-05 is summarized below:

- The ASCE/SEI Standard follows technical requirements in RG 1.165 except that a performance-based method is used to define the SSE rather than a hazard reference probability of median 10^{-5} per year. For the ASCE approach the starting ground

[1] This standard is similar to the Department of Energy Standard: DOE-STD- 1020-94, "Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities", U. S. Department of Energy, Washington, DC 20585.

motion level when scaled by the appropriate design factor results in a site-specific SSE such that the annual probability of unacceptable seismic effects on the plant, measured in terms of seismically induced core damage, is consistent with the risk calculated at nuclear plants in the U.S. designed to current requirements. The SSE spectrum derived using the ASCE/SEI Standard is characterized by horizontal and vertical free-field ground motion response spectra at the ground surface.

- The quantitative safety goal of the ASCE/SEI Standard is to achieve an annual frequency of seismically induced core damage that is mean 10^{-5} or lower, when conservatively estimated by taking the onset of significant inelastic deformation of structures, systems, and components (SSCs) as the measure of unacceptable performance. The quantitative goal is achieved by determining the SSE spectrum amplitude at each structural period, such that SSCs designed to this spectrum using the NRC's seismic design criteria and procedures are assured of having a mean annual frequency against loss of function of less than 10^{-5}. This performance goal determination of the SSE is consistent with the overall design objective identified in 10 CFR Part 50, Appendix S, section IV.a.1.ii which reads:

  The nuclear power plant must be designed so that, if the Safe Shutdown Earthquake Ground Motion occurs, certain structures, systems, and components will remain functional and within applicable stress, strain, and deformation limits.

- The design amplitude required to achieve this performance goal at each structural period can be calculated starting from the mean 10^{-4} annual probability level of the seismic hazard spectrum in the free field at the ground surface, or from the 10^{-5} annual probability level, or from any intermediate annual probability level. The design factor on the spectrum associated with each of these probability levels would be different, but they all would lead to the same SSE. Starting from a 10^{-4} annual probability level has precedent described in ASCE/SEI Standard 43-05.

- For the ASCE/SEI Standard, the basis for using the mean 10^{-5} annual frequency of unacceptable performance as an appropriate performance goal for generic models of SSCs is seismic probabilistic risk assessments (PRAs) of existing nuclear power generation plants. A mean 10^{-5} annual frequency of core damage from seismic events corresponds to the safest 50 percent of U.S. nuclear plants where a full seismic PRA has been performed. The following table provides these statistics.

Mean Seismic CDF for Plants Performing Seismic PRA
Table 2.2 from NUREG 1742 (EPRI Results)

Plant	Mean Seismic CDF
South Texas Project 1 & 2	1.90E-07
Nine Mile Point 2	2.50E-07
La Salle 1 & 2	7.60E-07
Hope Creek	1.06E-06
D.C. Cook 1 & 2	3.20E-06
Salem 1 & 2	4.70E-06
Oyster Creek	4.74E-06
Surry 1 & 2	8.20E-06
Millstone 3	9.10E-06
Beaver Valley 2	1.03E-05
Kewaunee	1.10E-05
McGuire 1 & 2	1.10E-05
Seabrook	1.20E-05
Beaver Valley 1	1.29E-05
Indian Point 2	1.30E-05
Point Beach 1 & 2	1.40E-05
Catawba 1 & 2	1.60E-05
San Onofre 2 & 3	1.70E-05
Columbia (WNP No. 2)	2.10E-05
TMI 1	3.21E-05
Oconee 1, 2, and 3	3.47E-05
Diablo Canyon 1 & 2	4.20E-05
Pilgrim 1	5.80E-05
Indian Point 3	5.90E-05
Haddam Neck	2.30E-04
Median of Mean Seismic CDF Value	1.20E-05
Mean of Mean Seismic CDF Value	2.50E-05

- Using the mean 10^{-5} annual frequency of core damage ensures that SSEs for future nuclear plant sites are risk-consistent with the safety performance of existing plants, which the Commission has determined to be adequately safe. The safety performance objective of developing the SSE spectrum is to ensure compliance with the public health and safety standard stated in the first paragraph in 10 CFR 100.23, as given below:

  > *This section sets forth the principal geologic and seismic considerations that guide the Commission in its evaluations of the suitability of a proposed site and adequacy of the design bases established in consideration of the geologic and seismic characteristics of the proposed site, such that there is a reasonable assurance that a nuclear power plant can be constructed and operated at the proposed site without undue risk to the health and safety of the public ...*

  The standard of no undue risk is met by deriving an SSE spectrum that results in a plant that is as safe as the plants currently operating. The results of the seismic PRA analyses summarized above demonstrate that this objective is satisfied for a mean 10^{-5} annual frequency of core damage.

- The design factor (also referred to as a scaling factor) used within the ASCE/SEI Standard to scale the mean 10^{-4} spectrum to ensure the 10^{-5} core damage frequency depends only on the hazard curve slope. The recommended design factor is given in

Section 4.3 of Appendix B to the EGC ESP SSAR, as well as Section 2.2 of the ASCE/SEI Standard. The design factor can also be derived from Equations 7.16 and 7.17 in NUREG/CR-6728. The design factor includes the assumptions that the risk reduction ratio (i.e., the ratio between the frequency of exceedance of the starting amplitude (10^{-4}) and the desired mean core damage frequency is 10; that the logarithmic standard deviation of fragility is between 0.3 and 0.6, and that a conservatively assumed High-Confidence of Low Probability of Failure (HCLPF) seismic margin for design of at least 1.0 exists.

- By starting at a specified mean annual hazard probability of 10^{-4} per year and assuming a risk reduction ratio of 10, the derived SSE is risk-equivalent to the average mean seismic CDF value of 10^{-5} given in the table presented above. The design factor together for the risk reduction ratio of 10 adjusts the spectral accelerations at a hazard probability of mean 10^{-4} to achieve a probability ratio consistent with the CDF value of 10^{-5}.

The design response spectrum calculated using the ASCE/SEI Standard is technically justified, appropriately conservative, and allowed within NRC's existing seismic regulations. Moreover, the ASCE/SEI Standard provides an SSE whose margin is explicitly quantified.

EGC RESPONSE TO ITEM b):

The EGC ESP does not rely on site-specific response spectra from the controlling earthquakes at the hazard reference probability level of median 10^{-5} per year to determine the site-specific SSE. Instead, the ASCE/SEI Standard 43-05 is implemented to determine the site-specific SSE. Application of the ASCE/SEI Standard results in site-specific SSE ground motions that are risk-consistent with the median of the mean seismic-induced core damage frequencies (CDFs) determined from probabilistic risk assessments (PRAs) of existing nuclear plants, which the Commission has determined to be adequately safe. The mean seismic CDFs for this set of plants are presented in response to Item 1 of this RAI. The basis for using the ASCE/SEI Standard 43-05 procedure instead of a hazard reference probability for the determination of the SSE is further discussed in this response.

As noted in the RAI, RG 1.165 describes procedures acceptable to NRC for determining site-specific SSEs based on the use of the hazard reference probability of median 10^{-5} per year. This reference probability was established through use of seismic hazard computations by Lawrence Livermore National Laboratories (LLNL) at 29 plant sites located in the United States, as discussed in Appendix B to RG 1.165. The reference probability is also considered applicable for hazard analyses conducted by the Electric Power Research Institute (EPRI) on behalf of utilities. The LLNL and EPRI analyses were conducted in the late 1980s and early 1990s using input to the seismic hazard computations that were applicable at the time that the computations were performed.

Studies carried out in 2003 and 2004 during the ESP Application process have found that the current understanding of seismic sources and ground motion models within central and eastern United States may result in a significant increase in seismic hazard at some sites. These changes in seismic hazard indicate a need to update the reference probability given in RG 1.165 to account for new ground motion models, new seismic source information, and better site response adjustments.

After reviewing the alternatives, EGC has concluded that a re-evaluation of the hazard reference probability in Appendix B of RG 1.165 would not achieve the regulatory stability sought by RG 1.165 and necessary for EGC to proceed with their current ESP Application or any future ESP application(s). Relative to overall industry needs, a revision to the current reference probability based on seismic information available in 2004 would remain valid only until new information becomes available on seismic sources near one or more of the 29 sites, or when new information becomes available on ground motion attenuation models. On a site-specific basis, EGC does not support development of an SSE using a reference probability that is not based on the latest seismic hazard information. Moreover, advances in technologies for determining site-specific SSEs since the late 1980s together with advances in NRC's regulation implementation policies, specifically the implementation of the Commission's Risk-Informed Regulation Policy, support the need for updating the guidance contained in RG 1.165 to comply with the current state of the practice (e.g., ASCE SEI methodology). This generic action is outside the scope of the EGC ESP submittal.

Given the perceived instability of the hazard reference probability approach and the availability of the mature risk-consistent method, described in Section 2.5.2 of the EGC ESP SSAR, the hazard reference probability of median 10^{-5} per year, as given in Appendix B of RG 1.165 (or an alternative reference probability) was not used to develop the SSE. As discussed in the response to Item 1 of this RAI, the new risk-consistent ASCE/SEI Standard 43-05 method was used to determine the SSE. This method was used for the following additional reasons:

- This procedure results in characterization of the site-specific SSE by both horizontal and vertical free-field response spectra at the ground surface. Spectra at rock level were defined by updated seismic sources for EGC ESP Site, by the 2003 EPRI ground motion model, and by EPRI-equivalent seismic hazard assessment procedures - consistent with the guidance for site-specific updating of seismic hazard assessments contained in RG 1.165. Spectra on the soil surface in the free-field were derived using site response adjustments consistent with RG 1.165 guidance, and using the current state of practice methods described in NUREG/CR-6728.
- This method results in site-specific free-field ground motions that are risk-consistent over SSCs with the risk level defined by the mean seismic induced core damage frequency (CDF) for 25 operating nuclear plants - making it analogous with use of the hazard reference probability approach in Appendix B of RG 1.165 except that a quantitative assessment of safety is used. The hazard reference probability approach in RG 1.165 is accepted because it is based on the hazard levels of past SSEs coupled with the general Commission conclusion that the population of existing plants is safe. It does not explicitly consider the degree of safety attained or the consistency of risk achieved among sites.
- By considering both the ground motion hazard and seismic design criteria of SSCs, the ASCE/SEI Standard methodology provides the annual probability of exceeding unacceptable behavior limits - that is, it provides an integrated, risk-consistent method for assuring safe seismic performance of nuclear facilities that is consistent across sites. This is in contrast to the procedures described in RG 1.165, which achieve only seismic hazard-consistent seismic ground motions between sites.

By using the performance-based approach described in the ASCE/SEI Standard 43-05, an SSE was defined that uses the updated seismic hazard at the EGC ESP Site and provides a risk to core damage that is consistent with the average of 25 operating power plants in the United States. EGC considers this methodology to be mature because it is a consensus standard, to be an acceptable methodology for complying with the governing seismic regulations, and its use to be consistent with the Commission's Risk-Informed Regulation Implementation Plan.

EGC RESPONSE TO ITEM c):

The justification for incorporation of equipment performance in determination of the final SSE is founded in the governing seismic siting regulation, 10 CFR 100.23 and the governing seismic design regulation, 10 CFR Part 50, Appendix S; in historical practice for evaluating the SSE for a site and determining its acceptability; and in NRC's Risk-Informed Regulation Implementation Plan. For the EGC ESP Site, the performance-based approach identified in ASCE/SEI Standard 43-05 was used to determine the site-specific SSE. This approach combines a conservative characterization of equipment/structure performance with ground motion hazard to establish risk-consistent SSEs, rather than only hazard-consistent ground shaking, as occurs using the hazard reference probability approach in Appendix B of RG 1.165. By following the methodology given in ASCE/SEI Standard 43-05, the derived SSE achieves a conservatively estimated mean annual risk to core damage of 10^{-5} or lower, as summarized in Item b) of the response to this RAI. This level of risk is consistent with the level of risk to core damage of other nuclear power plants operating in the United States and thereby, consistent with the hazard-based approach described in RG 1.165, which is founded in the Commission's determination that existing nuclear plants are adequately safe and pose no undue risk to the health and safety of the public.

Regulatory Requirements

10 CFR 100.23 implemented the important advance in seismic regulation of requiring "... that uncertainty inherent in estimates of the SSE be addressed through an appropriate analysis, such as a probabilistic seismic hazard analysis or suitable sensitivity analyses." In issuing Part 100.23 the NRC also provided for the use of current technologies (e.g., PSHA methodology) for determining site-specific SSEs and, importantly, intended to achieve the stability and site-to-site consistency in seismic regulation that was not achieved by the old seismic regulation, Part 100, Appendix A. The Commission addressed the seismic safety of existing nuclear plants licensed under Part 100, Appendix A by conducting safety assessments, including independent plant examinations for external events (IPEEE). Seismic IPEEEs were done by performing a seismic probabilistic risk assessment (PRA) or by performing a seismic margin assessment using seismic margin methods. The seismic assessments were based on seismic designs of existing plants and used review level earthquakes derived based on PSHA results.

The body of information developed by these various analyses confirmed the Commission's policy statement on severe accidents, that existing plants are adequately safe and pose no undue risk to the health and safety of the public. This important determination provided the foundation for establishing a target ground motion hazard level (i.e., the "reference probability" described in RG 1.165, Appendix B) for deriving SSE ground motions for future nuclear plant sites. The reference probability calculations used probabilistic seismic hazard results for plants that had been licensed under

10 CFR, Part 100, Appendix A and had used the RG 1.60 standard response spectrum appropriately scaled for site-specific controlling earthquakes, as the SSE ground motion.

By using this subset of plants with the most modern seismic design, the NRC intended to provide "...an adequate level of conservatism in determining an SSE consistent with recent licensing decisions." The reference probability (median 10^{-5} per year) was established as the median of the distribution of annual median probabilities of exceeding the SSEs for spectral responses at 5 and 10 Hz associated with 5 percent critical damping for the subset of 29 sites of plants with modern seismic design. Further, by requiring that site-specific SSEs satisfy this established reference probability, the NRC intended to provide site-to-site hazard-consistent SSE ground motions for future plants consistent on average with those of existing plants that had been determined to be adequately safe.

RG 1.165 provides guidance for the determination of site-specific SSEs for ESP sites. It was recognized when RG 1.165 was issued, however, that the technologies for determining site-specific SSEs using probabilistic seismic hazard results required updating. The technological basis for updating the RG was provided by an important 5-year research program sponsored by the NRC and reported in NUREG/CR-6728. The Risk-Informed Regulation Implementation Plan clearly anticipates that RG 1.165, as well as other RGs, will be updated to fully implement risk-informed regulation practice. The ASCE/SEI Standard is a consensus implementation of the methods reported in NUREG/CR-6728 for deriving risk-consistent SSEs. Moreover, the performance-based determination of site-specific SSEs clearly is an equally acceptable method for implementing the governing seismic regulations. Consistent with the hazard-based method of determining site-specific SSEs described in RG 1.165, the performance-based method described in the ASCE/SEI Standard is founded in the Commission's determination that the existing population of nuclear plants is adequately safe and poses no undue risk to the health and safety of the public

The Performance-Based Methodology

The performance-based method described in ASCE/SEI Standard 43-05 provides requirements for developing risk-consistent SSEs based on a probabilistic seismic hazard assessment and on conservative characterizations of structure, systems, and component (SSC) seismic design criteria and risk reduction achieved by the seismic design criteria. The criterion used conservatively assumes that failure of SSCs will occur at the onset of inelastic behavior. By considering both the ground motion hazard and seismic design criteria of SSCs, the methodology provides the annual probability of exceeding acceptable behavior limits - that is, it provides an integrated, risk-consistent method for safe seismic performance of nuclear facilities consistent with the 10 CFR Part 50, Appendix S, goals for final seismic design.

The performance-based method presented in ASCE/SEI Standard 43-05 implements key recommendations contained in NUREG/CR-6728 and -6769 to provide a fully risk-consistent basis for determining site-specific seismic design basis ground motion. The ASCE Standard is an industry consensus document prepared by the Dynamic Analysis of Nuclear Structures Subcommittee of the Nuclear Standards Committee of the ASCE, a group of industry experts in the field. The standard has been approved by the technical committees and has been through a required public comment period without comments;

it has been assigned a publication number and is expected to be issued in the near future.

The Need for a Performance-Based Methodology

The performance-based methodology evolves and advances the hazard-based reference probability methodology provided in RG 1.165 by providing for the determination of site-specific SSE ground motions that are risk-consistent from site-to-site. It does not replace other elements of the guidance contained in RG 1.165.

As discussed in Item b) of the response to this RAI, updating of the seismic hazard at the EGC ESP Site identified a significant increase in hazard. Since new data have regional significance, this finding indicates that a new hazard reference probability would need to be established to replace the annual median 10^{-5} probability provided in RG 1.165. Consequently, the hazard-based criterion provided in RG 1.165 does not achieve the stability over time that it was intended to achieve.

Also, the hazard reference probability contained in RG 1.165 does not provide SSEs that are consistent with the Commission's over-arching risk-informed regulation policy. It achieves consistent seismic hazards between sites, but does not achieve uniform safety performance among units. Importantly, as stated above, because SSEs derived following RG 1.165 guidance are based on the relative hazard for 29 existing nuclear plant sites and hazard is subject to change as new data become available, inevitable changes to the computed hazard for any one or more of these 29 sites can change the basis for the reference probability. Thus, the approach lacks long-term stability that can be achieved by implementing a performance-based methodology.

Regulatory Basis for the Performance-Based Methodology

The regulatory basis for the incorporation of equipment performance into determination of the final SSE has been discussed in the response to Item a) of this RAI. It is further elaborated in this response.

The performance-based method for determining risk-consistent, SSEs is fully consistent with the NRC's Risk-Informed Regulatory Policy and Safety Goal Guidelines and with applicable regulations for early site permitting, including 10 CFR 100.23 and 10 CFR Part 50, Appendix S. This conclusion is based on the following:

- **The performance-based methodology is consistent with the requirements of 10 CFR 100.23.** NRC Regulation, 10 CFR 100.23, describes the principal geologic and seismic considerations that must be evaluated to establish the adequacy of the site SSEs. The intent of the regulation is clear - that determination of the SSEs must appropriately incorporate scientific and data uncertainties and together with the Commission's seismic design criteria and procedures, must provide adequate assurance that *a nuclear power plant can be constructed and operated at the proposed site without undue risk to the health and safety of the public.* There are no stated or unstated objectives to develop uniformity of the seismic hazard between sites or specific exceedance probabilities for site seismic hazards. Thus while providing the fundamental performance requirement, the regulation leaves the technical procedures and criteria for demonstrating no undue risk open, recognizing that the technical implementation methods would necessarily evolve with time. Instead, the regulation defines the numerous geological, seismological, and engineering characteristics that must be investigated to permit evaluation of the site and *to support evaluations performed to arrive at estimates of the appropriate SSE.* It does not prescribe or restrict the type or method of evaluation to be employed to establish the SSEs beyond the requirement that uncertainties in basic information must by considered by means of a probabilistic assessment or a sensitivity analysis. It is clear that the regulation is focused on defining the site characteristics to be considered in evaluating the suitability of a site and in estimating the SSE ground motion to achieve a satisfactory level of overall safety performance risk; it contains no language that can be interpreted to preclude use of a performance-based approach. In fact, the objective of the performance-based method is consistent with objective of 10 CFR 100.23.

- **The performance-based methodology is consistent with the requirements of Part 50, Appendix S.** 10 CFR Part 50, Appendix S defines the SSE as follows: *The Safe Shutdown Earthquake Ground Motion (SSE) is the vibratory ground motion for which certain structures, systems, and components must be designed to remain functional.* This definition clearly links the determination of the SSE to the safety performance of plant structures, systems, and components and justifies the incorporation of equipment performance into determination of the final SSE. There is no language in Part 50 Appendix S that would prohibit consideration of engineering design criteria in the assessment of a site's SSE. On the contrary, the language in the regulation supports consideration of engineering design criteria in the determination of the SSE. The NRC's seismic design criteria and requirements for future plants, and in particular the certified ALWR plants, provide plant seismic design aspects that will be adequately conservative to exceed the design assumptions upon which ASCE/SEI Standard 43-05 is based.

- **The performance-based methodology characterizes the SSE on the basis of free-field seismic motions at the ground surface.** As directed by 10 CFR 100.23(d)(1) the SSE ground motion for the site is characterized by both horizontal and vertical free-field ground motion response spectra at the ground surface. The regulatory requirement identifies the need to consider uncertainties and references Appendix S to Part 50 for minimum design requirements. However, none of the regulatory requirements states how to determine the SSE.

- **Past licensing practice considers seismic design criteria in establishing the suitability of SSE ground motion for a nuclear plant and sites.** The evolution of guidance for assessing the suitability of a plant's SSE has included consideration of seismic design criteria and procedures at every stage. From the beginning, seismic design criteria and the margin of safety achieved by the design criteria and procedures were integral to the development of guidance for evaluating the SSE. Safe seismic performance was the overriding goal that framed the guidance. Consider the following:

  - The RG 1.60 Standard Design Spectrum, adopted for design of the modern plants, specifically considered the structural frequency response of plant SSCs and the spectrum was conditioned to provide increasing response amplitudes over the range of critical structural and component frequencies.

  - Determination of site-specific peak ground accelerations (PGAs) for scaling the RG 1.60 spectrum, taking into consideration a site's seismic environment and the need to achieve reasonable consistency among sites, was a major issue in the implementation of the RG 1.60. It was well understood that typical controlling earthquakes determined following the regulatory requirements of Part 100, Appendix A would produce site-specific ground motion spectra amplitudes that would exceed the RG 1.60 spectrum at structural frequencies above about 10 Hz when the RG spectrum was scaled to what were considered to be reasonable peak ground acceleration levels that when used to scale the spectrum at 33 Hz, defined an appropriately conservative SSE for the site. The shape of the RG spectrum for frequencies greater than 10 Hz, though recognized to be lower than the amplitudes of local controlling earthquake motions in this frequency range, was nevertheless considered to provide acceptable seismic safety performance for plant components with response frequencies in this range. This reasoning led to the concept of scaling the RG spectrum with "effective PGAs" that would define a site's SSE that, in combination with the NRC's seismic design criteria and procedures, would achieve the seismic safety performance requirement of the regulation. This approach avoided compounding conservatisms across technical disciplines. The SSEs for the existing nuclear plants licensed under the requirements of Part 100, Appendix A and seismic design basis ground motions defined by RG 1.60 scaled at 33 Hz to a site-specific PGA were determined in this way.

- In its policy statement on severe accidents in nuclear power plants issued in 1985 the Commission concluded, based on available information, that existing plants pose no undue risk to the health and safety of the public. A program of independent plant examinations confirmed this conclusion for both internally initiated and externally initiated events, including seismic initiated events. The conclusion that existing plants are adequately safe provided the basis for establishing the reference probability provided in RG 1.165 for determining SSE ground motions for future plants that are, on average, consistent with the SSE ground motions for existing plants. The finding equally well provides the basis for developing the performance-based method described in ASCE/SEI Standard 43-05 for determining SSE ground motions based on the results of past PRAs.
- A recently completed NRC project, reported in NUREG/CR-6728 and -6769, provides the technical basis for revisions of RG 1.165 in order to provide guidance for the development of risk-consistent ground motion spectra. These reports recommend implementation guidance for determining site-specific uniform reliability spectra that "*... achieve approximate uniformity of seismic risk for structures, equipment, and components designed to those spectra, across a range of seismic environments, annual probabilities, and structural frequencies. By 'seismic risk' we mean the annual frequency of failure of a plant system or of its components, as opposed to 'seismic hazard', which is the annual frequency of exceedance of a level of ground motion.*" The performance-based method described in the ASCE/SEI Standard implements this NUREG/CR recommendation in a consensus standard that is intended to provide fully risk-consistent SSEs for nuclear power plants.

- **The performance-based method is consistent with the NRC's Risk-Informed Regulation Implementation Plan.** The focus of the Commission's overall Risk-Informed Regulation Policy and Safety Goal Guidelines is on achieving uniform risk of failure of critical SSCs at a site and among sites that is independent of initiating events. The performance-based approach provides a demonstration of performance-consistent SSCs in accordance with these overall policies is achieved.

- In SECY 00-0062 the staff describes its Risk-Informed Regulation Implementation Plan. The Plan describes the large number of initiatives and activities that are being implemented to provide guidance for risk-informed regulatory applications for reactor regulation. Included among these is an activity to develop standards for the application of risk-informed, performance-based regulation in conjunction with national standards committees.
- The ASCE/SEI Standard 43-05 is an example of this activity that has reached a mature state and now can be accepted by NRC for application to determine the site-specific SSE for the EGC ESP Site.
- The Skull Valley Hearing Board recently approved the design ground motion for that nuclear waste storage facility based on risk/performance. These actions can be taken as confirmation that the NRC is implementing its risk-informed policy guidelines.
- In his recent approval of 10 CFR 50.69, "Risk-informed Categorization and Treatment of Structures, Systems and Components," Chairman Diaz' expressed (in the voting record) his belief that "...performance-based approaches should be used more extensively."

Relevant Associated Activities

EGC intends to work with the nuclear power industry through EPRI and the NEI Seismic Issues Task Force, to undertake efforts to obtain generic NRC approval of a risk-informed, performance-based methodology for determining site-specific SSEs. One objective of this initiative will be to develop the technical documentation needed to revise or replace RG 1.165, particularly Appendix B, i.e., to replace the reference probability based guidance with fully performance-based guidance. The effort is expected to be based on government/industry development work and the industry consensus for a performance-based method provided in the recently approved ASCE/SEI Standard 43-05 that will be published in the near future. As these activities progress, EGC expects to coordinate any generic activity with the project specific review of this methodology, but the EGC ESP Application is not dependent on the possible generic activity.

In its ESP SSAR, EGC has requested acceptance of the use of the ASCE/SEI Standard 43-05 methodology for site-specific application to determine the SSE for the EGC ESP Site. In doing so EGC has taken the position that acceptance of the Standard Methodology for site-specific application can and should proceed independently of any review of the Standard for generic acceptance. EGC continues to hold this position.

Summary

The progression of NRC's policies, regulations, and guidance governing seismic design safety during the past 30 years has been toward achieving an integrated risk-informed, performance-based seismic design. Given these developments, incorporation of equipment performance into the determination of the SSE is consistent with 10 CFR 100.23, consistent with 10 CFR Part 50, Appendix S, consistent with past practice for developing seismic regulatory guidance, and fully responsive to the Commission's risk informing policy and the NRC Staff's Risk-Informed Regulation Implementation Plan.

New References

American Society of Civil Engineers (ASCE). "Seismic Design Criteria for Structures, Systems and Components in Nuclear Facilities and Commentary", Standard ASCE/SEI 43-05. Washington, DC. 2004. [Not yet published, draft referenced in EGC ESP SSAR Section 2.5 as (ASCE, 2003).]

U. S. Nuclear Regulatory Commission (USNRC). "Risk-Informed Regulation Implementation Plan." Washington DC. August 2001.

U. S. Nuclear Regulatory Commission (USNRC). "Policy Statement on Severe Accidents." Federal Register, Vol. 50, 32138. August 8, 1985.

ASSOCIATED EGC ESP APPLICATION REVISIONS:

None

ATTACHMENTS:

None

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.2-2

SSAR Figure 2.2-2 in Appendix B shows a comparison between three older attenuation relationships (McGuire et al., 1988; Boore and Atkinson, 1987; Nuttli, 1986-Newmark and Hall, 1982) used for the 1989 Electric Power Research Institute (EPRI) study and the four model clusters developed by the EPRI 2003 ground motion study. However, SSAR 2.2.2 in Appendix B states that Figure 2.2-2 shows the Toro et al. (1997) and Atkinson and Boore (1995) ground motion models in comparison with the original ground motion models used for the 1989 EPRI study. Please clarify this discrepancy.

EGC RAI ID: R7-7

EGC RESPONSE:

SSAR, Appendix B, Figure 2.2-2 has been revised to include the correct figure. The incorrect figure was an inadvertent duplication of the information in Figure 2.2-3 of Appendix B of the Application.

ASSOCIATED EGC ESP APPLICATION REVISIONS:

Replace SSAR, Appendix B, Chapter 2, Figure 2.2-2 with the figure provided as RAI 2.5.2-2 Attachment (Revised App B Figure 2.2-2).

ATTACHMENTS:

RAI 2.5.2-2 Attachment (Revised App B Figure 2.2-2)

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.2-3

SSAR Section 2.2 in Appendix B describes the EPRI 2003 ground motion study. Many of the ground motion relationships that make up the four clusters use different distance measurements. Since the exact distance measure used by a ground motion model can make a large difference in ground motion estimates for small distances, please clarify the original distance measure used for each attenuation relationship and the assumptions used to convert each distance measure to a common measure.

EGC RAI ID: R7-8

EGC RESPONSE:

The EPRI (2003) model for median ground motions consists of four ground motion clusters. Table 2.5.2-3-1 (below) lists the models that make up each cluster of the EPRI (2003) ground motion model and indicates the distance measures used by each model and the distance measure used for each EPRI (2003) cluster. In Clusters 1, 2, and 4, the individual models use Joyner-Boore distance except those developed by Frankel et al. (1996) and Atkinson and Boore (1995). The Frankel et al. (1996) and Atkinson and Boore (1995) models use point-source distance, the distance to the equivalent point source representing the earthquake. These two relationships were converted to Joyner-Boore distance by simulating a data set in terms of moment magnitude and Joyner-Boore distance and fitting this simulated data set. At a given Joyner-Boore distance, earthquake point source depths were simulated for a range of magnitudes using the point-source depth distributions for the CEUS proposed by Silva et al. (2002). These consist of lognormal distributions with the parameters listed in the following table.

Point-Source Depth Distribution Parameters
(from Silva et al. 2002)

Magnitude **M**	Minimum Depth (km)	Median Depth (km)	Maximum Depth (km)	$\sigma_{\ln(D)}$
4.5	2	6	15	0.6
5	2	6	15	0.6
5.5	2	6	15	0.6
6	3	7	17	0.6
6.5	4	8	20	0.6
7	4.5	9	20	0.6
7.5	5	10	20	0.6
8	5	10	20	0.6
8.5	5	10	20	0.6

For each simulation the depth and the Joyner-Boore distance were used to compute the corresponding point source distance. The median ground motion for the given magnitude and point source distance were then computed using the Frankel et al. (1996) and Atkinson and Boore (1995) relationships. The resulting simulated data sets were then fit with an appropriate functional form to provide ground motion relationships in terms of moment magnitude and Joyner-Boore distance consistent with the other relationships in Clusters 1 and 2.

The final step in the distance adjustment process was conversion to a distance measure consistent with the EPRI-SOG seismic sources. The EPRI-SOG seismic source models define the spatial (map view) location of earthquake locations in terms of points. In the PSHA conducted for the EGC ESP these point locations were assumed to represent earthquake epicenters. EPRI (2003) provided relationships to adjust epicentral distance to the appropriate distance measures (Joyner-Boore or rupture) under the assumption of random orientation of ruptures (rupture strike uniformly distributed between 0° and 360°). Adjustments are provided for two conditions, the epicenter is located in the center of the rupture and the epicenter is randomly located along the rupture (uniformly distributed over the length). The randomly located epicenter adjustment was used in the EGC ESP PSHA calculation. The adjustment is conceptually similar to the method used by Frankel et al. (1996, 2002) to account for extended source size in the PSHA. Figure 2.5.2-3-1 (below) shows examples of the EPRI (2003) epicenter to rupture distance adjustment used to conduct this PSHA. The results shown for Cluster 3 converge to the median depth to rupture for CEUS earthquakes.

The conversion from epicentral to rupture distance assuming random rupture orientations also introduces additional aleatory variability in ground motions at a given epicentral distance. EPRI (2003) provides models for this additional aleatory variability. Figure 2.5.2-3-2 (below) shows examples of the additional aleatory variability. The results for Cluster 3 include the effect of random rupture depths on the aleatory variability at small epicentral distances.

New References

Atkinson, G. M. An Alternative to Stochastic Ground-Motion Relations for Use in Seismic Hazard Analysis in Eastern North America. Seismological Research Letters. v. 72, no. 2, p. 299-306. 2001.

Sadigh, K., C.-Y. Chang, J.A. Egan, F. Makdisi, and R.R. Youngs. Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data. Seismological Research Letters. v. 68(1), p. 180-189. 1997.

Table RAI 2.5.2-3-1
Distance Measures Used in EPRI (2003) CEUS Ground Motion Model

EPRI (2003) Cluster	Cluster Distance Measure	Individual Ground Motion Model	Individual Model Distance Measure
1 - Single Corner Spectral Models	Joyner-Boore	Frankel et al (1996)	Point-source
		Silva, et al (2002) - single corner, constant stress drop	Joyner-Boore
		Silva, et al (2002) - single corner, constant stress drop with saturation	Joyner-Boore
		Silva, et al (2002) - single corner, variable stress drop	Joyner-Boore
		Hwang & Huo (1997)	Joyner-Boore
		Toro, et al (1997)	Joyner-Boore
2 - Double Corner Spectral Models	Joyner-Boore	Atkinson & Boore (1995)	Point-source
		Silva, et al (2002) - double corner	Joyner-Boore
		Silva, et al (2002) - double corner with saturation	Joyner-Boore
3 - Hybrid	Rupture	Abrahamson & Silva (2002)	Rupture
		Atkinson (2001) combined with Sadigh et al. (1997)	Rupture
		Campbell (2003)	Rupture
4 - Finite Fault/Greens Function	Joyner-Boore	Somerville et al. (2001)	Joyner-Boore

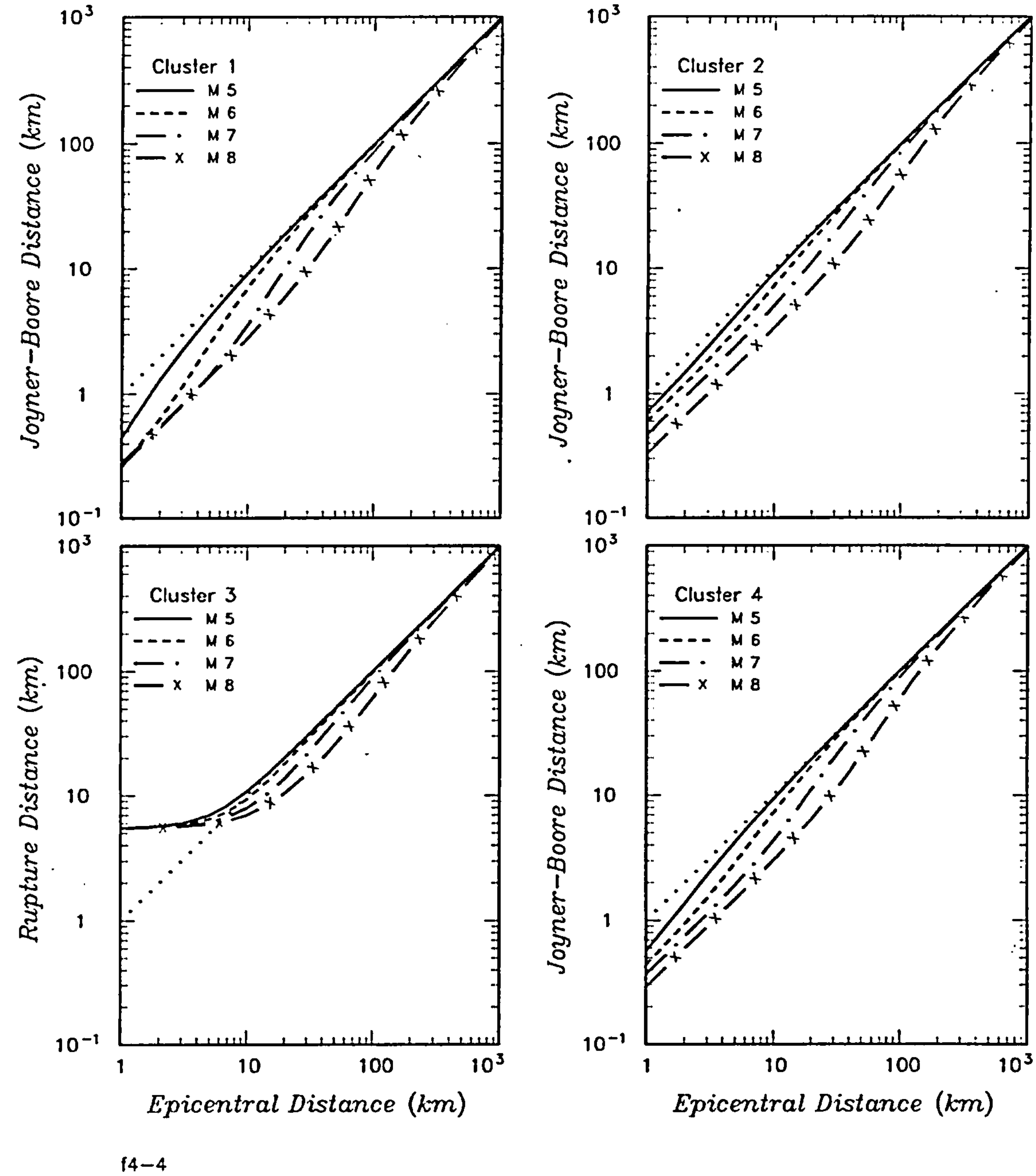
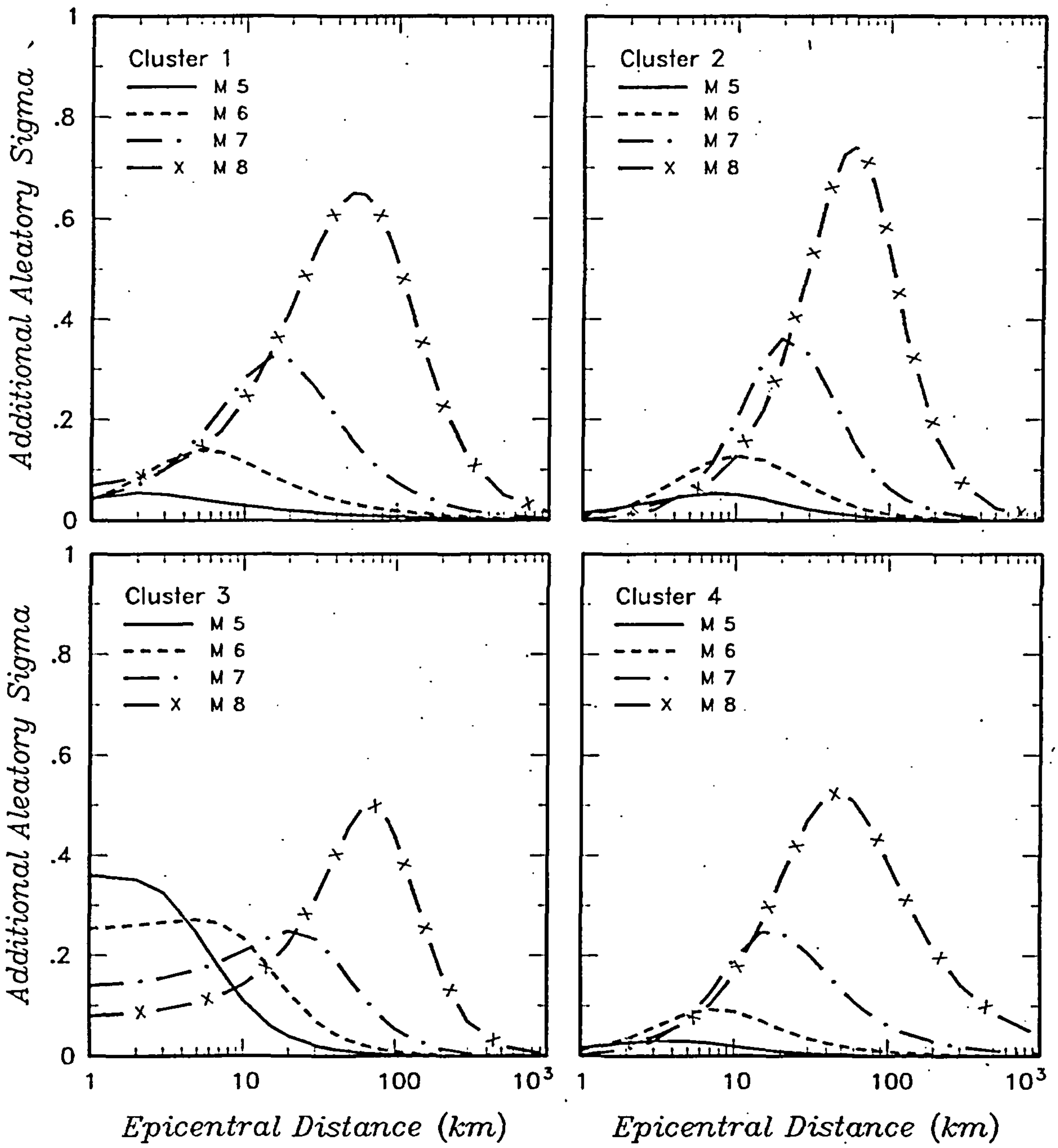


Figure 2.5.2-3-1 Example Adjustments from Epicentral to Joyner-Boore or Rupture Distance for Use with EPRI (2003) Attenuation Models



f4–6

Figure 2.5.2-3-2 Example Additional Aleatory Variability (expressed as the standard deviation or sigma of the natural log of spectral amplitude) Values to be Included When Adjusting from Epicentral to Joyner-Boore or Rupture Distance with the EPRI (2003) Attenuation Models

ASSOCIATED EGC ESP APPLICATION REVISIONS:

None

ATTACHMENTS:

None

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.2-4

SSAR Section 4.1.3 of Appendix B describes a Bayesian estimation of the maximum magnitude (M_{max}) for the central Illinois basin-background source that surrounds the site. The prior distribution for the Bayesian analysis comes from the global database of stable continental region (SCR) earthquakes of Johnston et al. (1994) and specifically from SCR earthquakes in non-extended continental crust like that of the central Illinois basin-background source. The prior distribution is adjusted for bias to imply a mean M_{max} of 6.3 with a standard error of 0.5. The prior distribution is then updated with likelihood functions developed from the paleoseismological record of the central Illinois basin-background source. The resulting posterior distribution has a mode at M_{max} 6.5. According to the database of Johnston et al. (1994), the two largest SCR earthquakes from non-extended crust through 1990 are (1) the Accra, Ghana earthquake of 1862 (M 6.75 + 0.35) and (2) the Meeberrie, Western Australia earthquake of 1941 (M 6.78 + 0.25). The reason for using Johnston's database is to increase the number of geologically analogous, seismologically indistinguishable terranes within which the rare largest possible earthquakes for those terranes have been included within the global historical record.

a) If central Illinois, Accra, and Meeberrie are seismologically indistinguishable, please explain why M_{max} for central Illinois should not be set at 6.8 and why the Bayesian analysis is necessary.

b) Please explain the bias that must be adjusted for in the database of Johnston et al. (1994) for non-extended crust and how the bias-adjusted estimate of M_{max} was calculated.

c) Does updating of the prior distribution require the assumption that the paleoseismological record of central Illinois is long enough to include the M_{max} earthquake? If yes, please explain why the assumption is valid. If updating does not require the assumption, explain why not.

d) Please explain the effect on the site hazard of estimating M_{max} for the central Illinois background source using the Bayesian analysis (i.e., with a distribution having a mode at M 6.5), as opposed to fixing M_{max} at M 6.8. Show the results of this sensitivity analysis in a graph with two short-period (10 Hz) hazard curves, one computed with the Bayesian analysis and the other using a M_{max} 6.8.

EGC RAI ID: R7-9

EGC RESPONSE:

a). The EPRI-SOG assessments of seismic source characteristics in the CEUS did not start with the assumption that maximum magnitude is the same throughout the region or even throughout regions with similar characteristics. The EPRI-SOG assessments of maximum magnitude for the central Illinois source zones need to be updated because of new information - the discovery of the Springfield paleo-earthquake. This update could have been performed using the EPRI-SOG approach - expert elicitation, but this would require a major study comparable to the original EPRI-SOG program. As an alternative, the Johnston et al. (1994) Bayesian approach was used. The Johnston et al. (1994) Bayesian approach was developed as part of a study specifically focused on the assessment of maximum magnitude in Stable Continental Regions (SCR). It provides a quantitative approach based on evaluation of a worldwide database of SCR earthquakes and crustal domains. This approach provides a reasonable method for assessing the uncertainty in maximum magnitude.

The Bayesian approach for estimating maximum magnitude developed by Johnston et al. (1994) does not start from the assumption that all SCR domains have the same maximum magnitude. Instead it assumes that there are characteristics that control the maximum size of an earthquake that can occur in an individual SCR domain and these characteristics vary from domain to domain, just as the maximum size of earthquakes varies for other source types (e.g., plate-boundary faults, subduction zones). The statistical analysis presented in Chapter 5 of Johnston et al. (1994) explored the utility of using the characteristics of the SCR domains as predictors of maximum magnitude. The first step in this process was the development of "super domains" by "pooling" the data for domains that "*cannot, with the information available, be considered different.*" The primary objective of pooling was to increase the earthquake sample size for a given super domain to provide a more constrained estimate of maximum magnitude (see the response to part b below). The resulting super domains were distinguishable from each other using the tectonic, geologic, and seismologic information gathered as part of the project. The prior distribution from Johnston et al. (1994) used in the EGC ESP probabilistic seismic hazard analysis (PSHA) assessment of maximum magnitude for central Illinois was based on grouping all of the 15 non-extended crust super domains and estimating the statistics of the maximum magnitudes of that group of domains. These 15 super domains all had the common characteristic of non-extended crust, but differ in other characteristics that may or may not be related to differences in maximum magnitude, such as crustal age, state of stress, and orientation of stress relative to structure. The Johnston et al. (1994) analysis did not assume that all of the non-extended crust super domains are identical, and thus would have the same maximum magnitude.

b). The bias adjustment described in Johnston et al. (1994) is the correction (positive) from the largest observed earthquake $M_{max}^{observe}$ to an estimate of the "true" maximum magnitude $\underline{m^u}$. Chapter 5 of Johnston et al. (1994) describes this bias adjustment and its dependence on sample size. For large sample sizes (>100 events) the correction is small. The mean value of $M_{max}^{observe}$ for non-extended crust obtained in Chapter 5 of Johnston et al. (1994) was **M** 6.2. Chapter 6 of Johnston et al. (1994) describes the application of the bias correction to this value. Accounting for catalog completeness, the estimated average number of events per non-extended crust super domain was 120. As a result, the bias correction was small, 0.1 magnitude units, and the estimated mean value of $\underline{m^u}$ for non-extended crust domains was set to **M** 6.3.

c). The updating of the prior distribution in the Bayesian analysis does not require that the period of record is long enough to contain the maximum magnitude. The likelihood function estimates the relative likelihood of various possible maximum magnitude values given the observed sample. The likelihood function has the form:

$$L[m^u] = \begin{cases} 0 & \text{for } m^u < M_{max}^{observed} \\ \left[1 - \exp\left\{-b\ln(10)(m^u - m_0)\right\}\right]^{-N} & \text{for } m^u \geq M_{max}^{observed} \end{cases}$$

where N is the observed number of earthquakes in the magnitude range of m_0 to $M_{max}^{observe}$, and b is the Gutenberg-Richer b-value. Example likelihood functions are shown on Figure 2.5.2-4-1. The effect of $M_{max}^{observe}$ is to set the likelihood function to zero for all smaller magnitudes - m^u is ≥ $M_{max}^{observe}$. The sample size N controls the likelihood that m^u is near $M_{max}^{observe}$. If N is small (e.g., a short period of recording relative to the frequency of large earthquakes), a wide range of values of m^u have similar likelihoods. If N is large, then the likelihood that m^u is near $M_{max}^{observe}$ is high.

d). The results of the requested sensitivity analyses to test the impact of the maximum magnitude assessment for central Illinois sources on the median and mean rock hazard for 10 Hz spectral acceleration at the EGC ESP site are shown in Figure RAI 2.5.2-4-2. Each plot contains two hazard curves. The curves labeled "EGC ESP Application" are taken from the Appendix B and show the hazard computed using the posterior distribution for maximum magnitude shown on Figure 4.1-5 of Appendix B of the EGC ESP SSAR. The curves labeled "M_{max} = M 6.8" were computed setting the maximum magnitude for all central Illinois source to the single value of **M** 6.8. The two maximum magnitude distributions yield nearly the same hazard, with the single value of **M** 6.8 maximum magnitude producing approximately 2 to 3 percent higher ground motions at the mean 10^{-4} and mean 10^{-5} hazard levels. The small difference in hazard results is not surprising given that the mean maximum magnitude from the posterior distribution is **M** 6.65.

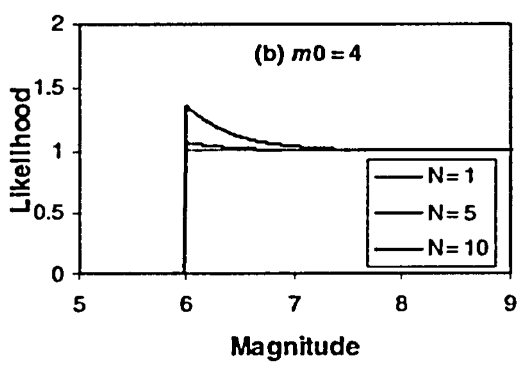


Figure 2.5.2-4-1 Example Likelihood Functions for a Value of $M_{max}^{observe}$ Equal to 6.0.

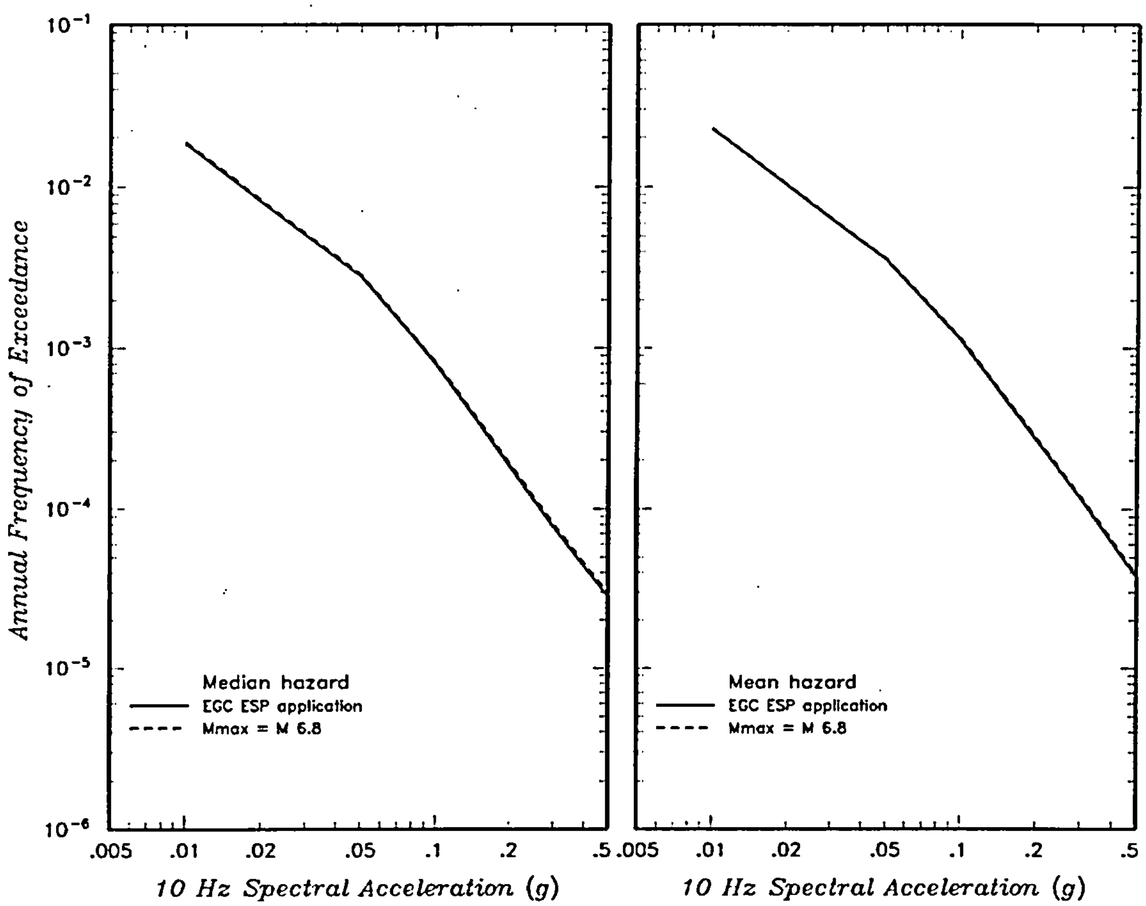


Figure 2.5.2-4-2 Effect of Maximum Magnitude Assessment for Central Illinois Source on the Seismic Hazard at the EGC ESP Site

ASSOCIATED EGC ESP APPLICATION REVISIONS:

None

ATTACHMENTS:

None

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.2-5

In Appendix B, Section 4.1.1.3, page B-4-7, the SSAR states that the New Madrid paleoseismological data of Tuttle et al. (2002)

> indicate that the Reelfoot fault (RF) has ruptured in all three sequences, but the New Madrid North fault (NN) and New Madrid South fault (NS) sources may not have produced large earthquakes in all three sequences. These observations were used to set the relative frequency of event sequences on the central New Madrid fault sources. The model used consists of: ruptures of all three sources NN, RF, and NS one third of the time, rupture of the NN and RF one-third of the time, and rupture of the NS and RF one third of the time.

However, Tuttle et al. (2002) conclude that all three sources (RF, NN, and NS) ruptured in each of the three sequences, but that one third of the time the NN rupture may have been smaller than in 1811-1812, and one third of the time NS may have been smaller than 1811-1812. Tuttle et al. (2002) also conclude that these smaller earthquakes are at least M 7 events. Please clarify this discrepancy.

EGC RAI ID: R7-10

EGC RESPONSE:

For the seismic source model developed for the New Madrid characteristic earthquakes in Appendix B of the EGC ESP SSAR, Figure 6 of Tuttle et al. (2002) was used to infer that previous ruptures of the New-Madrid North and New Madrid South faults may have been approximately one magnitude unit smaller than the estimated site of the 1811-1812 ruptures. The magnitudes for the 1811-1812 sequence shown on Figure 6 of Tuttle et al. (2002) were those developed by Johnston (1996). The information presented on Figure 6 of Tuttle et al. (2002) was used to infer the relative size of ruptures of the New Madrid North and New Madrid South faults in the 1450 and 900 sequences compared to the 1811-1812 ruptures. Thus, if the size of the 1811-1812 ruptures on these faults were in the low magnitude **M** 7 range (e.g., values estimated by Bakun and Hopper, 2003), then the size of previous ruptures would have been below magnitude **M** 7. These smaller ruptures, which would be considered dependent events, were not included in the hazard calculations as characteristic earthquakes. The rupture model developed for the New Madrid characteristic earthquake sources in the EGC ESP Application consisted of three possible sequences, each occurring with a relative frequency of 1/3. One sequence consisted of full ruptures of all three New Madrid faults; one sequence consisted of full rupture of the New Madrid North and Reelfoot thrust faults, with the rupture of the New Madrid South fault being approximately one magnitude unit smaller than the 1811 rupture (this smaller dependent event was not included in calculating the hazard); and one sequence consisted of full rupture of the New Madrid South and Reelfoot thrust faults, with the rupture of the New Madrid North fault being approximately one magnitude unit smaller than the 1811 rupture (this smaller dependent event was not included in calculating the hazard).

A sensitivity analysis was conduced to evaluate the effect of assuming that all rupture sequences consist of three full-sized ruptures of the three New Madrid faults. Figure 2.5.2-5-1 compares the resulting median and mean hazard curves for 1-Hz spectral acceleration to those shown in the EGC ESP Application. Assuming that all rupture sequences consist of full-sized ruptures results in approximately 7 to 8 percent higher ground motions at the mean 10^{-4} and mean 10^{-5} hazard levels.

As described in the response to RAI 2.5.1-1, the magnitude estimates for the 1811-1812 earthquakes continue to evolve and an updated distribution for the size of the 1811-1812 earthquakes was developed. In addition, recent discussions with Dr. Tuttle indicate that she considers that the difference between the size of the 1811-1812 earthquakes and those of the 900 and 1450 sequences are likely to be smaller than what was portrayed in Figure 6 of Tuttle et al. (2002). Consequently, a revised model for New Madrid sequences was developed consisting of two alternative models for earthquake sequences. In Model A, all ruptures are similar in size to the 1811-1812 earthquakes. Model B is similar to the model used in PSHA for the EGC ESP Application in that 1/3 of the sequences contain a smaller rupture of the New Madrid North fault and 1/3 of the sequences contain a smaller rupture of the New Madrid South fault. However, the difference in magnitude from the 1811-1812 ruptures was set to be no more that ½ magnitude unit, and no ruptures were allowed to be less than **M** 7. In addition, all three earthquakes were included in the hazard calculation in all rupture sequences. Model A (always full ruptures) was given a weight of 2/3 and Model B a weight of 1/3 based on Dr. Tuttle's expression of the difficulties in estimating the size of the pre 1811-1812 ruptures and her judgment that the difference between the rupture sizes was likely smaller that proposed in Tuttle et al. (2002). The hazard resulting from this revised model for rupture sequences combined with the updated magnitude distribution (response to RAI 2.5.1-1) is shown by the curves labeled "Revised magnitudes and sequences" on Figure 2.5.2-5-1. These results produce approximately 9 to 10 percent higher ground motions at the mean 10^{-4} and mean 10^{-5} hazard levels.

Finally, revisions to the earthquake recurrence estimates are suggested by analyses conduced for this response. In the EGC ESP Application, the lognormal distribution was used to represent the distribution of repeat times for characteristic earthquakes. Attachment 2 to Appendix B of the EGC ESP SSAR introduces the Brownian Passage Time (BPT) model developed by Matthews et al. (2002) and used by Working Group (2003) to assess the probabilities of large earthquakes in the San Francisco Bay region as the preferred model for estimating real-time recurrence of repeating earthquakes on individual faults. As stated in the EGC ESP Application, Matthews et al. (2002) showed that the lognormal model produces similar estimates of hazard to those obtained using the BPT model for elapsed times less than the mean repeat time and the lognormal model was used in the EGC ESP analysis because of its simpler form. While the lognormal and BPT models yield similar estimates of hazard given known parameters (the mean and standard deviation of repeat times between events), analyses conducted for this response show that the models do not yield the same parameters when they must be estimated from limited data. This is illustrated on Figure 2.5.2-5-2. Shown is the likelihood distribution for the mean time between repeating earthquakes estimated using the BPT and lognormal models. The top plot shows the case when the sample size is 50 closed intervals. For this case, the two models yield very similar results. The bottom plot shows the case for New Madrid where there are two closed intervals (900 to 1450 and 1450 to 1812) and one open interval (1812 to 2004). For this case, the uncertainty estimates for the mean repeat time are different.

Because of the differences shown on Figure 2.5.2-5-2, the analyses presented in Attachment 2 to Appendix B of the EGC ESP SSAR were repeated using the BPT model. The result was slightly lower equivalent earthquake frequencies for the real-time model. The Poisson recurrence rates were also revised by considering only the 1811-1812, 1450, and 900 earthquake sequences to be consistent with the data used in the real-time model estimates. This revision produced slightly larger earthquake frequencies for the Poisson model. Finally, the weight assigned to the real-time model was increased from 0.5 to 0.6, to reflect the weight assigned by the Working Group (2003) to real-time models on faults that clearly demonstrate repeated earthquakes (the San Andreas and Hayward faults). The overall effect of these small changes to the earthquake recurrence rates combined with the revised characteristic magnitude distributions and rupture sequence models is shown by the curves labeled "Revised magnitudes, sequences, and recurrence rates" on Figure 2.5.2-5-1. These results produce approximately 7 to 8 percent higher ground motions at the mean 10^{-4} and mean 10^{-5} hazard levels compared to those developed in the EGC ESP Application.

The assessment of the size of the 1811-1812 earthquakes and the likely scenarios for future ruptures continues to be an area of active research, and thus it is possible that the assessments presented in the ESP Application will undergo future evolution. It is expected, however, that the effects of these changes will be on the order of those presented in the sensitivity analyses presented in this response, and the calculated 1-Hz ground motions corresponding to the mean hazard in the 10^{-4} to 10^{-5} range will vary from those presented in the ESP Application by plus or minus 10 percent or less. A revision to the EGC ESP Application, therefore, is not warranted at this time.

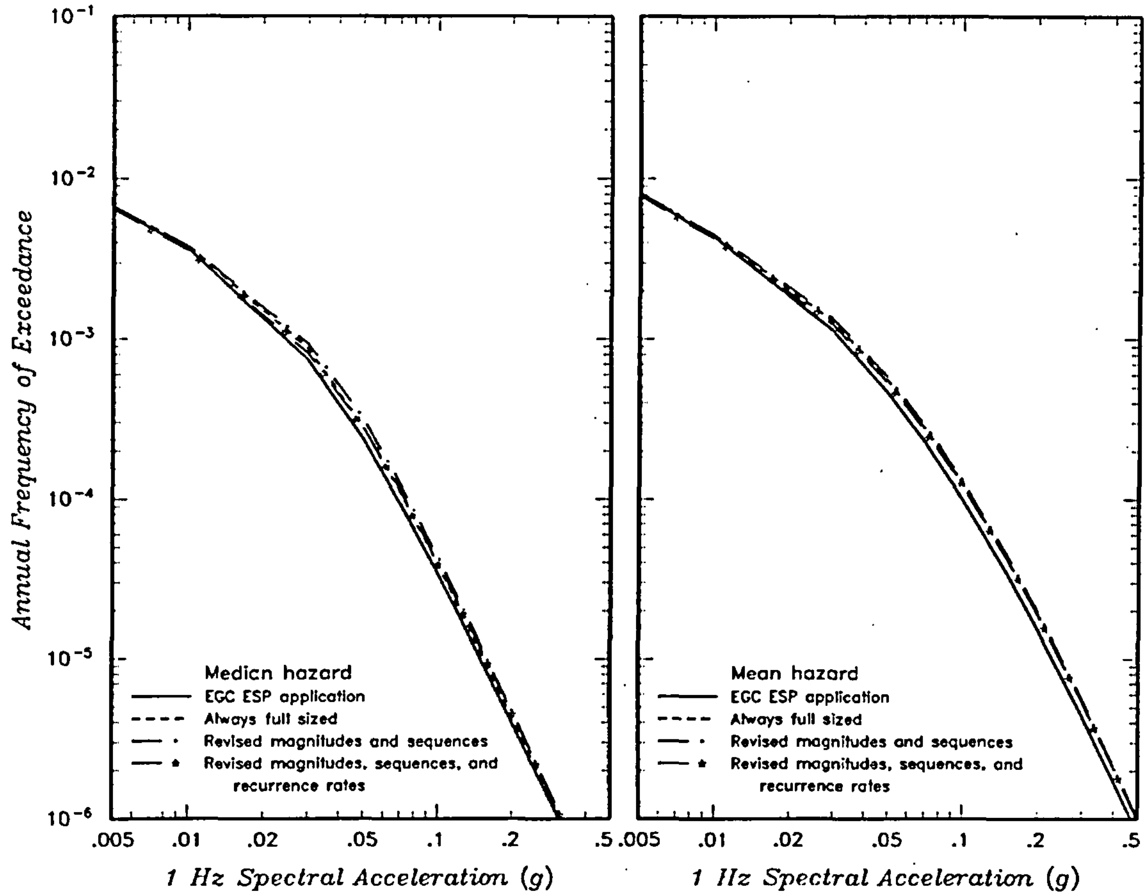


Figure 2.5.2-5-1 Median and Mean Seismic Hazard Curves for 1-Hz Spectral Acceleration Developed Using Alternative Models for Characteristic New Madrid Earthquakes.

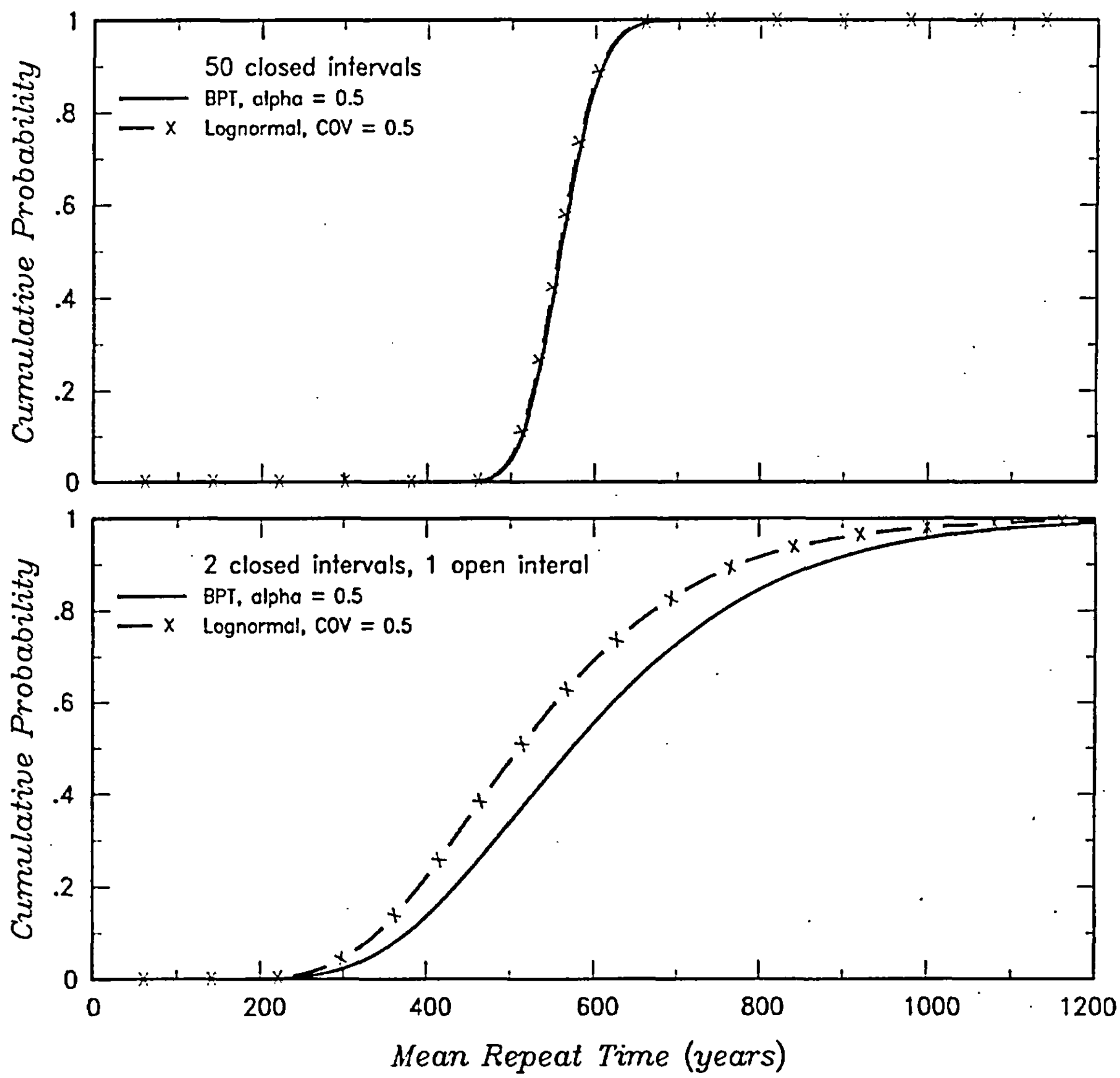


Figure 2.5.2-5-2 Comparison of Uncertainty Distributions for Mean Repeat Time Based on the Lognormal and BPT Distributions

ASSOCIATED EGC ESP APPLICATION REVISIONS:

None

ATTACHMENTS:

None

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.2-6

SSAR Attachment 2 of Appendix B indicates that Geomatrix performed reconnaissance-level investigations of streams within about 30 miles of the ESP site. Along Salt Creek, they identified four sites with features considered likely to be seismic in origin, with the closest being 11.5 miles from the ESP site. No other sites with paleoliquefaction features were identified in the study area. The consultants concluded that, given how few features there were, the exposure was sufficient to suggest that there have been no repeated moderate- to large-magnitude events in the vicinity of the ESP site in latest Pleistocene to Holocene time and that the late Holocene record in particular is sufficient to demonstrate the absence of such events in the past approximately 6 to 7 thousand years.

a) Given the proximity of the Salt Creek paleoliquefaction sites to the ESP site, please provide a detailed map showing where exposure was present and the quality of the exposure (such as bank heights, material, and lengths of the exposures).

b) Explain why the streams northwest and southeast of the ESP site were not examined in this study.

c) Were other sites, besides river bank exposures, used to confirm the absence of liquefaction features in the vicinity of the ESP site? If so, please describe these sites. If not, please justify your conclusion that there have been no repeated moderate- to large-magnitude events in the vicinity of the ESP site in latest Pleistocene to Holocene time.

d) The geographic distribution of the Henry Formation appears to have been an important guide to which stream reaches were searched for paleoliquefaction features. Please explain the characteristics and importance of the Henry Formation, including describing where it belongs in the time-distance diagram of SSAR Figure B-1-9.

EGC RAI ID: R7-11

EGC RESPONSE:

a) Maps showing the general ages and extent of exposed deposits along the riverbanks examined during this study are provided in Figures 2.5.2-6-1A through 1C (Mackinaw River), 2A through 2E (Sangamon River), 3A through 3C (Salt Creek), and 4A (North Salt Creek). The ages of the deposits are inferred based on general height above stream level, field observations regarding weathering and soil profile development, and descriptions and mapping provided in the Soil Conservation Surveys for DeWitt, Ford, and McLean Counties (issued 1991, 1990, 1998, respectively). Detailed information regarding bank height, material, and length of exposures was noted for key localities of older deposits (see Table B-1-5, Attachment 1, Appendix B, EGC ESP SSAR).

b) We did not conduct reconnaissance investigations in the areas to the northwest and southeast for both logistical and technical reasons. The field reconnaissance conducted as part of this study was designed to supplement the previous mapping and reconnaissance that had been conducted in the region by McNulty and Obermeier (1999). The relatively large Mackinaw River in the northern part of the study area was judged to be a good candidate location to evaluate evidence for paleoearthquakes that might be associated with reactivation of basement structures related to the La Salle anticlinorium. The upper reach of Salt Creek above the reservoir was judged to be the most likely drainage to record events to the east of the site. The reconnaissance was conducted over a two-week period in late fall. Late fall was chosen as the time when the stream level would be expected to be at its lowest providing the most extensive exposures along the larger drainages and prior to the start of the late fall rainy season.

With the identification of the two ages of clastic dikes in the vicinity of Farmer City in the initial stages of the field study, a focused effort was made to constrain the location and size of the causative paleoearthquakes by reconnaissance in the region surrounding the observed dikes. This entailed revisiting some stretches of the Salt Creek (below the reservoir) and the upper Sangamon River that had been surveyed previously by McNulty and Obermeier (1999) to check for new exposures and to look more specifically for deposits of latest Pleistocene age that would record the oldest event identified at SC-25.

We also surveyed portions of the North Fork of Salt Creek and the uppermost reaches of the Sangamon River where Henry Formation deposits are mapped (Lineback, 1979). In the region southeast of the site Henry Formation deposits are not mapped along the middle section of the Sangamon River (Lineback, 1979) that lies between the stretches mapped by McNulty and Obermeier (1999) and this study (see Figures B-1-6 and B-1-11, Attachment 1, Appendix B, EGC-ESP SSAR). Given the apparent absence of significant deposits of Henry Formation in this area and the coverage provided by previous mapping further to the southeast, additional surveys of this stretch were considered unnecessary. Reconnaissance of the region northwest of the site (e.g., the area in the vicinity of Downs) also was not conducted. The upper reaches of the drainages in this area would not be as readily accessible by canoe and there likely would be more limited exposure than is present along the larger drainages. Therefore, the EGC ESP surveys focused on more promising areas north of the site.

Although there are regions to the northwest and southeast of the site within 25 miles of the site that have not been examined, the coverage provided by the previous mapping and the mapping done as part of this study provides sufficient coverage to support the conclusion that paleoearthquakes comparable to the postulated Springfield event have not occurred within a radius of approximately 25 miles of the site post-hypsithermic time (post-6-7 ka). A moderate to large event located within the 25-mile radius to the southeast of the site likely would have been recorded along the examined reaches of Salt Creek and the Sangamon River. A moderate to large event within the 25-mile radius northwest of the site also likely would have been recorded along the examined reaches of the Mackinaw River, Salt Creek, or Sugar Creek.

c) The majority of the reconnaissance mapping effort focused on examining riverbank exposures, as this approach provides the most efficient method of finding sufficient exposures over an extensive area. Additionally, gravel pits shown on the 1:24,000 topographic map also were examined in some areas, chiefly in the region southwest of Mahomet. Locations of these gravel pits are indicated on Figure 2.5.2-6-2D. Typically, in most of the older or abandoned gravel pits where the walls of the gravel pit are degraded and covered with slopewash, colluvium, and vegetation, there are few or no good exposures. Active gravel pits elsewhere in the 25 mile-radius region that were not visited during this study could provide additional exposures, but as noted in the discussion above, it is expected that if earthquakes comparable to the Springfield earthquake had occurred in this area, evidence would have been recorded along the surveyed stretches of the streams.

d) The Henry Formation, which consists of stratified sand and gravel that occurs above the Sangamon Geosol is included in the Mason Group of latest Wisconsin age as shown on Figure B-1-9 (Attachment 1, Appendix B, EGC-ESP SSAR). Hansel and Johnson (1996, p. 58) state that:

> The Henry Formation is interpreted to be (1) outwash deposited adjacent to or leading away from the glacier, (2) nearshore sand and gravel deposited in beaches, spits, bars, and deltas in glacial and postglacial lakes, and (3) eolian sand derived from glaciofluvial, fluvial, and nearshore lake sediments deposited in dunes and sheets on and adjacent to those sediments. ... Most of the Henry Formation was deposited during the Wisconsin Episode, predominantly during the Michigan Subepisode between about 26,000 and 11,000 radiocarbon years ago...

As noted by Obermeier et al. (2001) the typical setting for paleoliquefaction features is in a valley of a moderate to a large river, on the modern floodplain or on a terrace that is only a few meters higher, where the depth to the water table is less than several meters and where there are thick sandy deposits. In the study area, the Henry Formation sediments, where they are overlain by a cap of finer silt and where they are at or below the water table, are generally susceptible to liquefaction. As an initial screening tool, we used the existing Quaternary map of the region (Lineback, 1979) to identify stretches along the larger streams in the region where the Henry Formation is shown as favorable locations for reconnaissance.

New References

U.S. Department of Agriculture (USDA). *Soil Survey of Ford County, Illinois.* Soil Conservation Service. Illinois Agricultural Experiment Station Soil Report No. 128. 1990.

U.S. Department of Agriculture (USDA). *Soil Survey of De Witt County, Illinois.* Soil Conservation Service. Illinois Agricultural Experiment Station Soil Report No. 137. 1991.

U.S. Department of Agriculture (USDA). *Soil Survey of McLean County, Illinois.* Natural Resources Conservation Service. Illinois Agricultural Experiment Station Soil Report No. 159. 1998.

ASSOCIATED EGC ESP APPLICATION REVISIONS:

None

ATTACHMENTS:

RAI 2.5.2-6 Attachment (Figures 1A-4A)

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.2-7

SSAR Subsection 2.5.2.6 describes an alternative approach to that recommended in RG 1.165 for determining the Safe Shutdown Earthquake (SSE) ground motion spectrum. Please provide the following information regarding this approach:

a) The approach described in SSAR Section 2.5.2 targets a performance goal of mean 10^{-5} per year of "unacceptable performance of nuclear structures, systems and components as a result of seismically initiated events." Please justify the selection of mean 10^{-5} per year as an appropriate performance goal and describe in detail what this probability represents.

b) The approach described in SSAR Section 2.5.2 starts with the risk equation and ends with a scale factor multiplier that is used to achieve the target performance goal. Please provide the details of this derivation and describe how the use of the scale factor achieves the target performance goal. In addition, please provide the details (beyond those provided in NUREG/CR-6728 and the ASCE Draft Standard, SSAR References 118 and 119) of the assumptions made for each of the key parameters such as the seismic margin ratio, combined standard deviation, amplitude ratio, and hazard curve slope.

EGC RAI ID: R7-12

EGC RESPONSE:

EGC RESPONSE TO ITEM a):

The approach described in Section 2.5.2 of the EGC ESP SSAR is based on the recently approved ASCE/SEI Standard 43-05, *Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities and Commentary.* This standard uses a mean 10^{-5} probability per year of "unacceptable performance of nuclear structures, systems, and components as a result of seismically initiated events" for nuclear power plants. As noted subsequently in this response, the quantitative goal of this performance-based approach is to achieve an annual frequency of seismically induced core damage frequency (CDF) that is 10^{-5} or lower, when conservatively estimated by calculating the annual frequency of onset of significant inelastic deformation (FOSID) of structures, systems, and components (SSCs).

Justification for the use of mean 10^{-5} per year as an appropriate performance goal is based on work that was published in 2002 as NUREG-1742, as summarized below.

- The selection of mean 10^{-5} annual frequency of exceedance as an appropriate performance goal for generic models of SSCs is based on the results from seismic probabilistic risk assessments (PRA) that were performed for 25 operating nuclear facilities using an SSE ground motion spectrum. These PRAs achieved an annual mean CDF of 10^{-5} or higher for seismic core damage for 50 percent of the operating power plants. The computed results were provided previously in the response to RAI 2.5.2-1. The summary table shows that a mean 10^{-5} annual frequency of core damage from seismic events corresponds to 50 percent of U.S. nuclear plants where a full seismic PRA has been performed.

- The annual frequency of onset of significant inelastic deformation (FOSID) of structures, systems, and components is generally much less than failure of the SSC. Failure results in large inelastic deformations - leading to loss of containment or other unacceptable performance. As long as the SSCs remain essentially elastic in their performance - or have limited inelastic response - performance during the seismic event is considered acceptable. It is generally recognized that core damage frequency (CDF) is typically less than the highest SSC failure frequency - indicating that by using CDF as a basis for design, the approach is conservative relative to other SSCs.
- By following the ASCE/SEI Standard 43-05 method, the target performance goal annual frequency is achieved so long as the seismic demand and structural capacity evaluations have sufficient conservatism to achieve both of the following:
    - Less than approximately a 1 percent probability of unacceptable performance for the SSE , and
    - Less than approximately a 10 percent probability of unacceptable performance for a ground motion equal to 150 percent of the SSE.

  Plants reviewed and approved using the USNRC Standard Review Plan guidelines have achieved at least these levels of conservatism.
- The mean 10^{-5} annual frequency of core damage represents a means for achieving safe plant design. Safe plant design is the underlying goal of developing the selected SSE spectrum as reflected in the first paragraph in 10 CFR 100.23:

  > *This section sets forth the principal geologic and seismic considerations that guide the Commission in its evaluations of the suitability of a proposed site and adequacy of the design bases established in consideration of the geologic and seismic characteristics of the proposed site, such that there is a reasonable assurance that a nuclear power plant can be constructed and operated at the proposed site without undue risk to the health and safety of the public ...*

  The requirement for no undue risk is met by determining an SSE spectrum that results in a plant that is as safe as the safest plants currently operating. The results of the seismic PRA analyses summarized above indicate that this objective is satisfied for a mean 10^{-5} frequency.

EGC RESPONSE TO ITEM b):

The SSE Spectrum shown in Section 2.5.2 of the EGC ESP SSAR was determined following the procedures, including assumptions, given in the ASCE/SEI Standard 43-05, *Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities and Commentary*. The derivations for the risk equation and the scale factor in the ASCE/SEI Standard 43-05 are included Section C2.2.1.2 of the Commentary to the Standard. This derivation involves convolving the seismic hazard and the fragility curve to determine the mean probability of unacceptable performances (P_F).

A number of references provide relevant background information and details that support the development and use of the methods given in ASCE/SEI Standard 43-05, including:

- Two papers co-authored by Dr. Robert Kennedy and provided to NRC staff prior to the Site Meeting in Chicago on May 18, 2004. Copies of these two papers are again included as attachments to this response. Appendix C of the Paper RPK Tokyo gives the derivation of the risk equation.
- Section 7 of NUREG/CR-6728, *Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-consistent Ground Motion Spectra Guidelines,* by McGuire et al. (2001). Section 7.2 of this document provides a detailed discussion of the concepts and goals for seismic risk consistency, factors affecting seismic risk, and the risk equation. The figures in this section (e.g., Figure 7-5) demonstrate graphically the concept of the risk calculation.
- Section 2 and Appendix C of DOE-STD-1094-94, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities,* U.S. Department of Energy, April 1994 with Change Notice # 1 January 1996. Of note is the DOE's use of a consistent risk level for wind and flood design, as well as for seismic design.

A full understanding of the derivation of the methods used in ASCE/SEI Standard 43-05 requires careful study of each of these references. In recognition that multiple references are used in the development of the ASCE/SEI Standard 43-05 method and based on comments from the NRC staff during a meeting on September 16, 2004, EGC is currently preparing a summary that will document the various steps involved in the derivation of the approach used in ASCE/SEI Standard 43-05, including use of the scale factor to achieve the target performance goal. This information will be submitted at a later date. Note, however, the basis of the ASCE/SEI Standard 43-05 method was developed by industry specialists in the area of risk evaluation, and this method was previously adopted by the Department of Energy and now by ASCE. As such, the ASCE/SEI Standard 43-05 is an industry-accepted approach. The summary paper will necessarily be limited to explaining steps that may not be obvious in the use of the ASCE/SEI Standard 43-05 and not serve as a basis for approving or rejecting the basis of the Standard.

Relative to the use of the ASCE/SEI Standard 43-05 method at the EGC ESP Site, Item b) of this RAI also requests details for the assumptions made for each of the key parameters assumed when using the ASCE/SEI Standard 43-05. The assumptions made during use of the ASCE/SEI Standard 43-05 for the EGC ESP Site are summarized below:

- ***Seismic Margin Ratio:*** The seismic margin on design (F_P) was conservatively assumed to be 1.0 consistent with the ASCE/SEI Standard 43-05 recommendations. This margin represents the ratio of the High-Confidence of Low Probability of Failure (HCLPF) seismic capacity to demand based on the Design Response Spectrum.
- ***Combined Standard Deviation:*** The logarithmic standard deviation (β) was selected to be in the typical range of nuclear SSCs from 0.3 to 0.6. This value is also consistent with the recommendations of ASCE/SEI Standard 43-05.
- ***Design Factor (DF):*** The design factor is the ratio of spectral acceleration of the design response spectrum to the spectral acceleration of the mean 10^{-4} uniform hazard spectrum. The Design Factor (DF) is a function of the hazard curve slope factor (A_R). Within the EGC ESP SSAR and ASCE/SEI Standard 43-05, this factor is referred to as the Design Factor (DF); other documents (e.g., DOE-STD-1020-94 and NUREG/CR-6728) refer to it as a scale factor. This ratio is given in column DF_2 of Table 4.3-1 in Appendix B of the EGC ESP SSAR but not less than 1.0. Values

range from 1.00 to 1.45, depending on frequency. In the 5 to 10 Hz range, the DF values range from 1.04 to 1.08, respectively.

- ***Hazard Curve Slope Factor (A_R):*** The hazard curve slope factor is the ratio between the spectral accelerations at 10^{-5} and 10^{-4}. This slope factor can be determined for the frequencies using the data in Table 4.3-1 within Appendix B of the EGC ESP SSAR. Results indicate that the slope factor ranges from approximately 1.8 to 3.0 depending on frequency - with the slope factor generally increasing for decreasing frequency. The slope factor varies from approximately 2.08 to 2.00 for frequencies of 10 and 5 Hz, respectively.

The Design Factor (DF) is based on achieving a probability ratio (R_P) of 10. The probability ratio defines the ratio between the free-field seismic hazard at the ground surface and the risk of unacceptable behavior of SSCs. DOE-STD-1020-94 refers to the probability ratio as the risk reduction ratio and indicates that this ratio introduces additional reduction in the risk of unacceptable performance below the annual risk of exceeding the SSE (or Design Basis Earthquake).

ASSOCIATED EGC ESP APPLICATION REVISIONS:

None

ATTACHMENTS:

RAI 2.5.2-7 Attachment 1 (RPK Lyon 1997)

RAI 2.5.2-7 Attachment 2 (RPK Tokyo 2001)

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.4-1

SSAR Chapter 3.1 in Appendix A states that, due to the expected consistency between the ESP and CPS sites, the scope of the drilling and sampling program consisted of two boreholes drilled to 100 feet below ground surface on the perimeter of the ESP footprint and two deep boreholes drilled into rock at the center of the footprint. In addition, four Cone Penetrometer Tests (CPT) were also performed between the borehole locations. Section 3.1.1 states, "If any significant soil property variations (for example, different soil types or different blowcounts from the [standard penetration test] SPT had been revealed during the drilling and sampling program, additional explorations would have been added to resolve the observed difference." Please describe the criteria used to determine whether the Uniform Hazard Spectrum difference in soil properties was significant enough to require additional exploration. In addition, provide a table showing a comparison between the static and dynamic soil properties for the CPS and ESP sites. For each soil property provide the sample size, average value, standard deviation, and range of values.

EGC RAI ID: R7-13

EGC RESPONSE:

[A] The first part of this RAI deals with the similarity of geotechnical conditions between the EGC ESP Site and the CPS Site and what criteria were used to require additional explorations to resolve difference between the two sites. The EGC ESP SSAR indicates that additional explorations would have been added if significant differences in soil properties were encountered. When planning the exploration program for the EGC ESP Site, results of a review of available information suggested that soil conditions would be the same at the two sites. The available information included the geologic history for the area and drilling and sampling that had been done for the CPS Site. The geologic information indicated that the regional processes that formed the soil profile at the two sites were the same. The information from the drilling and sampling program at the CPS Site included a number of explorations within and beyond the EGC ESP Site, as discussed in Section 3.1.1 of Appendix A to the EGC ESP SSAR. Soil cross-sections developed from the CPS Site explorations and presented in the USAR extended through the EGC ESP Site. These soil cross-sections also suggested that the essentially same soil stratigraphy existed at each site. Given that the geologic formation and stratigraphy for a site were essentially the same, the expectation was that only small variations in soil properties would be encountered during the EGC ESP Site explorations and that these variations would be within the normal range of variation that occur at any site with similar geologic history.

Although the soil properties were generally expected to be the same, the potential for some variations in soil conditions was recognized during the planning process. These variations typically result from localized differences in the geologic processes creating the soil profile at a specific location, such as a stream channel. During the planning for the field exploration program, the explorations were located in areas where gaps in information existed relative to the needs of the EGC ESP Site program. Also during the planning, a strategy was developed for collecting soil samples. In this strategy as the explorations were advanced, soil retrieved during the drilling and sampling was visually monitored to see if the soil color and texture were consistent with what was reported in the USAR for CPS Site. Blowcounts were also compared qualitatively in the field to confirm that their values were also in the same range. This combination of consistency, color, and texture were then judged qualitatively in the field to decide whether the material was essentially the same. For example, if the blowcount reported in the USAR was significantly different (e.g., an order of magnitude greater) than what was recorded during the EGC ESP Site exploration program and the texture of the material was fine-grained rather than a sand, the field task leader for the EGC ESP Site explorations was prepared to take additional soil samples to investigate this difference.

Once the fieldwork was complete and the laboratory testing program was initiated, the following comparisons and criteria were implemented to identify similarities and/or differences in subsurface conditions between the EGC ESP and CPS Sites:

- Comparison of visual-manual field descriptions of the soil samples collected during the EGC ESP Site investigation with each other and with CPS Site data. These comparisons were performed to evaluate whether each stratigraphic unit encountered at the CPS Site was present or absent at the EGC ESP Site, and to identify similarities and/or differences in thickness of and contact elevations between these units. Soil descriptions compared include apparent soil gradation, plasticity, presence of inclusions and bedding, color, consistency (soft to hard, loose to dense), and moisture condition. This comparison differed from that done in the field from the standpoint that all the information was available for review, rather than the individual comparison done in the field as the boring was drilled and sampled. Details of these comparisons are presented in Section 5.2.1 and 5.2.2 of SSAR Appendix A. As described, the same stratigraphic units were identified between each EGC ESP site investigation location and the CPS site, with moderate variations in contact elevations and unit thicknesses.
- Comparison of data from laboratory tests performed on samples from the EGC ESP Site with each other and with CPS Site test data. These comparisons were performed to identify similarities and/or differences in engineering properties of the stratigraphic units between the two sites, and within the EGC ESP Site. Test results compared include Atterberg Limits, in-situ dry density, moisture content, undrained shear strength, and consolidation properties. Results of these comparisons are presented in Section 5.2.2 and 5.2.3 of SSAR Appendix A. Standard Penetration Test (SPT) blowcounts at the two sites were also compared, as summarized in Section 5.2.1 of SSAR Appendix A. Shear and compression wave velocity data from the seismic CPT and suspension logging tests were also compared with applicable data from the CPS Site (from downhole and uphole logging tests), as described in Section 5.2.4 of SSAR Appendix A.

The decision regarding the need for additional explorations was made after review of the qualitative and quantitative comparisons described above. The qualitative and quantitative comparisons focused first on whether the stratigraphic unit was the same, and then whether the engineering properties within the unit were the same. The following criteria were applied for this purpose:

- The criteria applied to confirm the presence and depth of each stratigraphic unit at the EGC ESP Site were primarily qualitative, consisting of observed similarities of soil descriptions between the sites. These descriptions included soil color, texture, and consistency in terms of denseness or hardness indicated qualitatively by SPT blowcounts, pocket penetrometer strengths, and ability to indent with the logger's thumb. With this information and the geologic history of the area, it was possible to reasonably conclude that the stratigraphic unit was the same.

- A somewhat more quantitative approach was used to confirm similarity of stratigraphic unit engineering properties between the sites. EGC ESP Site engineering property data for each unit were plotted or tabulated along with the CPS Site data (as presented in Sections 5.2.2 through 5.2.4 of SSAR Appendix A), and the results were compared for similarity. This comparison focused on the typical range of properties within a stratigraphic unit recorded at the CPS Site versus the range in the same properties recorded at the EGC ESP Site. If the range in properties and the mean values appeared to be roughly the same, it is reasonable to conclude that the engineering properties for the unit were the same - within the normal level of uncertainty.

While no hard numerical acceptance criteria were employed for this comparison (i.e., if the range or mean differed by a certain number), the EGC ESP Site data were generally shown to fall within the range of CPS Site results for each stratigraphic unit, for each engineering property. There were some variations, but these were not considered significant enough to alter the conclusion that subsurface conditions are similar between the sites, and within the EGC ESP Site, as described in Sections 5.2.2 through 5.2.4 of SSAR Appendix A.

Since it was concluded that subsurface conditions are similar between the EGC ESP and CPS Sites, as well as within the EGC ESP Site, additional investigation was not considered necessary to support the ESP Application.

Modifications will be made to SSAR Appendix A, Section 3.1.1 to clarify this process.

[B] The second part of this RAI requests statistical comparisons of static and dynamic soil properties from the CPS and EGC ESP Sites. Many of these comparisons are shown graphically in Figure 5-7 through Figure 5-18 of Appendix A to the EGC ESP SSAR - where Atterberg Limits, moisture content, and density are compared for the two sites. These graphical comparisons show that in most cases the data obtained from the CPS Site and the EGC ESP Site are within the same range for the same stratigraphic units.

Tabulated statistical summaries of geotechnical test results requested by the RAIs are provided in Attachment 1 to this response. Separate tables have been prepared which summarize results from each stratigraphic unit. Results for each of the tested EGC ESP Site samples are included in these tables, as are results for samples from the CPS Site investigation that were collected from the vicinity of the plant site. The summary tables include results for the following static properties *(associated table column names follow in parentheses)*:

- Atterberg Limits *(Plastic Limit, Plasticity Index, and Liquid Limit)*
- Undrained Shear Strength *(Su-UU Test, and Su-UC Test)*
- Moisture Content *(Moisture Content)*
- Dry Density *(Dry Density)*
- Consolidation Parameters *(Initial Void Ratio, Compression Index, Recompression Index, and Preconsolidation Pressure)* - for Illinoian till only.

Dynamic soil properties were not included in the statistical summary tables for the following reasons.

- First, the dynamic laboratory tests performed on overburden soils during the CPS Site investigation consist of one resonant column test, 14 dynamic triaxial compression tests, and 3 shockscope tests. These test methods are different than the more modern combined resonant column/torsional shear (RC/TS) tests performed for the EGC ESP Site investigation, and therefore it is our opinion that the RC/TS test results should not be statistically compared with the CPS site test results. Specifically, in the case of the CPS resonant column test data, the shearing strain at the appropriate confining pressure for this sample is too high for the type of equipment used, indicating that the data cannot be relied upon. In the case of the cyclic triaxial test equipment, significant seating effects occurred in equipment used in the late 1960s and 1970s. These seating effects usually resulted in shear modulus values that were unreasonably low. In the absence of a companion resonant column test on these samples, it was not possible to judge which samples were most affected. Also, most cyclic triaxial tests were conducted on reconstituted soil samples. The final test method cited in the CPS USAR is the shockscope test. This is a nonstandard testing method that is no longer used within the profession.
- Second, the CPS Site dynamic data that are most comparable to the EGC ESP Site are from uphole and downhole logging tests performed in borehole P-14 at the CPS Site, as summarized on Figure 2-369 of the CPS USAR (CPS 2001). This figure gives only average compression and shear wave velocities from the tests that were conducted, and these average velocities are summarized for several stratigraphic units. Individual velocity values which could be compared to the results of seismic cone and suspension logging results obtained at the EGC ESP Site are not available. Therefore, a rigorous statistical comparison with parameters as requested in the RAI could not be performed for this data.

Although rigorous statistical comparisons using the downhole logging test performed at Borehole P-14 could not be performed, the interpreted results of this test will be added to SSAR Appendix A, Table 5-2, for comparative purposes.

The information presented in Attachment 1 to this RAI response, together with the comparisons shown graphically in Figure 5-7 through Figure 5-18, provide support to the conclusion that the conditions at the EGC ESP Site and the CPS Site are for practical purposes the same, and that the large body of information developed for the CPS Site can be used to augment the information collected during the exploration programs completed for the EGC ESP Site. In cases where the statistical comparisons do not appear to be favorable, both the qualitative description of the samples and the visual plots of the data are consistent within the range in variation that might be expected for the particular stratigraphic unit.

ASSOCIATED EGC ESP APPLICATION REVISIONS:

Revise SSAR Appendix A, Chapter 3.1.1, paragraph 2, last sentence from:

If any significant soil property variations (for example, different soil types or different blowcounts from the SPT) had been revealed during the drilling and sampling program, additional explorations would have been added to resolve the observed differences.

To read:

<Insert Paragraph Break> If any stratigraphic units were encountered in the EGC ESP Site investigations that were significantly different than observed at the CPS Site, or if significant variations in subsurface conditions were observed among the EGC ESP Site investigation locations, additional explorations would have been considered to resolve the observed differences. For this application a significant difference in stratigraphy would involve a soil color, texture, or consistency that was outside the range of normal variation for the particular soil type. Likewise, if field and laboratory test data (as described in Section 5) had indicated significant differences in stratigraphic unit engineering properties within the EGC ESP Site or between the EGC ESP and CPS Sites, additional explorations would have been considered. For this application a significant difference in field or laboratory properties would involve ranges and means that were outside the range of normal variation for the particular soil type. However, from results of qualitative and quantitative data comparisons it was concluded that subsurface conditions within and between the EGC ESP and CPS Sites were similar, as described in Section 5. Available information about the geologic history of the EGC ESP and CPS Site and the exploration information reported in the USAR for from the CPS Site also suggested that no significant variations should have been expected. On the basis of this information, it was concluded that no additional investigations were considered necessary.

Replace SSAR Appendix A, Table 5-2 with the new Table 5-2 provided in RAI 2.5.4-1 Attachment 2.

ATTACHMENTS:

RAI 2.5.4-1 Attachment 1 (Soil Property Info)

RAI 2.5.4-1 Attachment 2 (Revised App A Table 5-2)

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.4-2

Section 2.5.4.2 of the SSAR states that the EPRI modulus and damping curves were used for the site response calculations, because a much larger database was used to develop the EPRI curves and, therefore, average EPRI results are expected to be representative of conditions at the Exelon Generation Company (EGC) ESP site. It is the staff's understanding that the EPRI soil nonlinear property curves were primarily based on the results of regression studies of recorded ground motions at California soil sites, not on the results of laboratory test data. In addition, the EPRI curves are typically considered to be appropriate for sands and non-plastic (low plasticity index [PI]) silts. In view of these two observations, please provide a detailed justification for using the EPRI modulus and damping curves for the ESP site. Also, the laboratory test data (Figure 5-21 of Appendix A) for hysteretic damping appears to be relatively high at low strain compared to the generic data from the EPRI report. Please comment on these results.

EGC RAI ID: R7-14

EGC RESPONSE:

The first part of this RAI requests justification for the use of the EPRI modulus reduction and material damping curves at the EGC ESP Site. In response to this request, information on the development of the EPRI modulus reduction and material damping curves from the EPRI (1993) set of reports is summarized. A summary of discussions with one of the investigators involved in the EPRI (1993) project is also provided. This information indicates that the development of the EPRI modulus reduction and damping curves differs from that suggested in the RAI and that use of the EPRI modulus reduction and material damping curves at the EGC ESP Site is justified.

The modulus reduction and material damping curves that were used for the site response analyses at the EGC ESP Site, referred to as the EPRI curves, were obtained from Figures 7.A-18 and 7.A-19 of Appendix 7.A, *Modeling of Dynamic Soil Properties* in Volume 2: Appendices for Ground Motion Estimation of the EPRI (1993) set of reports. Copies of these figures are provided as Attachment 1 to this response. Section 7.A indicates that the curves in Figures 7.A-18 and 7.A-19 were originally developed by extending a simple hyperbolic model representing the nonlinear change in soil shear modulus and material damping with shearing strain. The nonlinear relationship upon which the hyperbolic model is based was observed during some of the early dynamic soil properties testing programs conducted in late 1960s and early 1970s by Hardin and Drnevich (1972) and Seed and Idriss (1970). As discussed in Section 7.A.5 of Volume 2 (EPRI, 1993), this extended model incorporated the Cundall-Pyke hypothesis for cyclic loading (Pyke, 1979), as well as other refinements for low- and high-strain material damping, total stress degradation and degeneration, and effective stress modeling.

Through a series of parametric evaluations, the curves shown in Figures 7.A-18 and 7.A-19 were developed for use in generic site response studies in eastern North America. These curves were reported to be compatible with each other and consistent with published data. The discussion of Figures 7.A-18 and Figure 7.A-19 indicates "it is intended that these curves represent soils in the general range of gravelly sands to low plasticity silty or sandy clays." Information in Section 5.2.2 of Appendix A to the SSAR and shown in Figure 5-1 in the same appendix indicates that soils at the EGC ESP Site fall within the range of "gravelly sands to low plasticity silty or sandy clays" identified in EPRI (1993).

Volume 1 of the EPRI (1993) set of reports includes a summary set of modulus reduction and material damping curves (Figure 6-9) for use in the quantification of site effects. A copy of Figure 6-9 is provided as Attachment 2 to this response. These curves were plotted from those in Figures 7.A-18 and Figure 7.A-19, and therefore are the same with one exception. The material damping ratio curves for depths of 50 feet or in Figure 6-9 were assigned the same curve as for 51 to 120 feet. No explanation is provided in either Section 6.3.4 of Volume 1 or Section 7.A.6 of Volume 2 for this change. This issue was discussed with Dr. Walter Silva, the technical lead for work described in Section 6 of Volume 1 to the EPRI report "Guidelines for Determining Design Basis Ground Motions" (EPRI, 1993a). According to Dr. Silva, there was concern at the time (circa 1993) that the material damping in the upper 50 feet calculated by the simple model could be too high and until further experience was obtained with the extended hyperbolic model presented in Appendix 7.A, the curves from 51-120 feet would define a cap on the material damping curves. Following the publication of the EPRI (1993) set of reports, it was concluded that the material damping values estimated by the simplified model were reasonable and could be used. This use is documented in Silva et al. (1996) where material damping values consistent with those in Figure 7.A-19 are presented. Silva et al. (1996) also commented that use of this model resulted in reasonable comparisons to recorded ground motions at a northern California site. The use of the EPRI damping curves at less than 50 feet was also adopted during studies completed for NUREG/CR-6728 (McGuire et al., 2001).

In summary, these discussions indicate that the EPRI nonlinear soil curves were developed primarily from results of laboratory testing programs that had been completed since the late 1960s and not by regression analyses of recorded ground motions. Subsequent to the development of the curves, regression analyses were conducted to confirm that use of the curves in site response analyses produces reasonable ground response predictions. The discussions also note that the EPRI curves were originally developed for "gravelly sands to low plasticity silty or sandy clays" and that these descriptions are consistent with the soils at the EGC ESP Site.

In response to the observation that the laboratory testing data (Figure 5-21 of Appendix A) for hysteretic damping appears to be relatively high at low strain compared to the generic data from the EPRI report, the following clarification is provided.

The information in Figure 5-21 does indeed suggest that the hysteretic damping from the laboratory tests was higher than the EPRI curves. The same conclusion can be made from the information in Figure 5-24. However, the curves that are above the EPRI damping curves are related to the resonant column tests. The fact that the resonant column data shown in Figures 5-21 and 5-24 are solid, bold lines masks the response from the torsional shear tests - suggesting that all the damping values are too high at low shearing strain amplitudes. However, material damping values from the torsional shear tests are very consistent with the EPRI damping curves, as shown in Figures 4.2-2 through 4.2-6 of Appendix B to the EGC ESP SSAR.

The higher damping from the resonant column tests has been recognized for a number of years, and was noted in the EPRI (1993) set of reports. It is attributed to rate of loading effects. Typical frequencies of loading for the resonant column test range from 100 to 200 Hz for these soils. As the frequency increases from the torsional shear testing (frequencies of 0.1 to 10 Hz) to resonant column (frequencies of 100 to 200 Hz), the absolute value of damping increases by several percent. This trend is shown in Figures B.15, C.15, D.15, E.15, and G.15 of Attachment A-7 to the Appendix A report. The observed frequency effect on damping is the reason combined resonant column/ torsional shear (RC/TS) tests are conducted. The frequency of loading for the torsional shear tests ranges from 0.1 to 10 Hz and therefore is much more consistent with predominant frequencies of earthquake loading.

New References

Hardin, B.O. and V.P. Drnevich. "Shear Modulus and Damping in Soils: Design Equations and Curves." *Journal of the Soil Mechanics and Foundation Division.* American Society of Civil Engineers. Vol. 98, No. SM7, pp. 667-692. 1972.

Personal communication between Dr. Robert Youngs, Geomatrix Consultants Inc., and Dr. Walter Silva, Pacific Engineering and Analysis, regarding the development history and use of the EPRI modulus reduction and damping curves in Section 6, Volume 1 of EPRI (1993a). September 2004.

Pyke, R.M. "Nonlinear Models for Irregular Cyclic Loadings." *Journal of the Geotechnical Engineering Division.* American Society of Civil Engineers. Vol. 105, No. GT6, pp. 715-726. 1979.

Seed, H.B. and I.M. Idriss. *Soil Modulus and Damping Factors for Dynamic Response Analyses*, Report No UCB/EERC 70-10. Earthquake Engineering Research Center. University of California, Berkeley. 1970.

ASSOCIATED EGC ESP APPLICATION REVISIONS:

Revise SSAR, Chapter 2, Section 2.5.2.5, 3rd paragraph, 2nd bullet, second sentence, from:

In general, the modulus and damping data are consistent with the EPRI (1993a) relationships, except that the laboratory data tend to show higher damping levels at very low shearing strains.

To read:

In general, the modulus and damping data are consistent with the EPRI (1993a) relationships, except that the resonant column data tend to show higher damping levels at very low shearing strains. The higher damping from the resonant column tests is attributed to rate-of-loading effects. Damping values from torsional shear tests, which are conducted at frequencies of loading more consistent with predominant free-field ground motions, is very consistent with EPRI damping values.

Revise SSAR, Chapter 2, Section 2.5.2.5, 5th paragraph, from:

The site response analyses were conducted using randomized shear wave velocity profiles and soil modulus and damping relationships to account for variation in the dynamic soil properties across the EGC ESP Site. The depth to hard rock was also randomized to reflect its uncertainty. The site response also assumed that the sedimentary rock below 300 ft remains linear during earthquake shaking. Damping in the rock was based on published information. Additional details about the generation of profiles for the site response analyses are included in Sections 4.2.1 and 4.2.2 of Appendix B.

To read:

The site response analyses were conducted using randomized shear wave velocity profiles and soil modulus reduction and material damping relationships to account for variation in the dynamic soil properties across the EGC ESP Site. In the absence of consensus within the profession and since these were free-field ground response analyses, rather than soil-structure analyses described in the SRP Section 3.7.2, material damping in the randomized sets of material damping curves was not capped at 15 percent. The depth to hard rock was also randomized to reflect its uncertainty. This randomization process resulted in 60 independent soil columns that were used in evaluating site response effects. The site response also assumed that the sedimentary rock below 300 ft remains linear during earthquake shaking. Damping in the rock was based on published information. Additional details about the generation of profiles for the site response analyses are included in Section 4.2.1 and 4.2.2 of Appendix B.

Revise SSAR, Chapter 2, Section 2.5.4.2, 4th paragraph, from:

Dynamic properties obtained for the EGC ESP Site were considered but not used explicitly for the site response studies described previously in Section 2.5.2.6. Rather, the EPRI modulus and damping curves were used as the base case for the site response analyses. The rationale for using the EPRI curves rather than the EGC ESP Site data was that a much larger database was used to develop the EPRI curves and, therefore, average EPRI results are expected to be representative of conditions at the EGC ESP site if an extensive dynamic testing program had been conducted. It is important to note that the dynamic test results for the EGC ESP Site are very consistent with the EPRI curves, indicating that use of the EPRI curves is acceptable. A comparison of the EPRI and EGC ESP Site cyclic test results is included in Figures 5-20 and 5-21 of Appendix A.

To read:

Dynamic properties obtained for the EGC ESP Site were considered but not used explicitly for the site response studies described previously in Section 2.5.2.6. Rather, the EPRI modulus and damping curves were used as the base case for the site response analyses. According to EPRI (1993a), the EPRI modulus reduction and material damping curves were developed to account for the variations in soil shear modulus and material damping that occur with shearing strain and soil confining pressure - with soil confining pressure being approximated within the set of curves by the depth below the ground surface. EPRI (1993a) indicates that these curves are appropriate for use in "gravelly sands to low plasticity silty or sandy clays", which is consistent with the soil conditions at the EGC ESP Site. The rationale for using the EPRI curves rather than the EGC ESP Site data was that a much larger database was used to develop the EPRI curves and, therefore, average EPRI results are expected to be representative of conditions at the EGC ESP Site. It is important to note that the dynamic test results for the EGC ESP Site are very consistent with the EPRI curves, indicating that use of the EPRI curves is acceptable. A comparison of the EPRI and EGC ESP Site cyclic test results is included in Figures 5-20 and 5-21 of Appendix A.

Revise SSAR, Appendix A, Section 5.2.4.2, 8th paragraph, from:

In view of the good comparison between the measured modulus and damping data for the samples from the EGC ESP Site and the published EPRI values of modulus ratio and damping ratio, it was concluded that the conditions at the EGC ESP Site could be adequately represented by the EPRI soil model when developing a site response model, as discussed in both Section 2.5 and Appendix B of the SSAR. Variations noted between the published EPRI curves and those obtained by laboratory testing reflect the normal variation that can be expected when testing soil samples. These variations are accounted for during ground response modeling by introducing a variation between the upper and lower bound modulus and damping ratio curves.

To read:

In view of the good comparisons between the measured modulus and damping data for the samples from the EGC ESP Site and the published EPRI values of modulus ratio and damping ratio, it was concluded that the conditions at the EGC ESP Site could be adequately represented by the EPRI soil model when developing a site response model, as discussed in both Section 2.5 and Appendix B of the SSAR. According to EPRI (1993), the EPRI modulus and damping curves were developed to account for the variations in soil shear modulus and material damping with shearing strain and soil confining pressure - with soil confining pressure being approximated within the set of curves by the depth below the ground surface. EPRI (1993) indicates that these curves are appropriate for use in "gravelly sands to low plasticity silty or sand clays", which is consistent with the soil conditions at the EGC ESP Site. Variations noted between the published EPRI curves and those obtained by laboratory testing reflect the normal variation that can be expected when testing soil samples, including the effects of soil disturbance as represented by the shear wave velocity ratio tabulated in Table 5-3. These variations are accounted for during ground response modeling by introducing sets of randomized modulus reduction and material damping curves that account for uncertainty in these curves through the use of variability terms explicitly determined from a study testing of rock and soil samples (Silva et al., 1996), as discussed in Section 4.2.2 of Appendix B in the EGC ESP SSAR.

Revise SSAR, Appendix B, Section 4.2.1, 4th paragraph, from:

A set of shear modulus reduction and damping tests were performed on samples taken from borings at the EGC ESP Site, as described in the EGC ESP Geotechnical Report (SSAR Appendix A). Figures 4.2-2 through 4.2-6 show the test results compared to the generic modulus reduction (G/Gmax) and damping relationships developed by EPRI (1993). (Note that one test sample produced what are considered to be erroneous values of modulus reduction and high damping values, as discussed in Appendix A to the SSAR. The test data from that sample were not included in developing the site dynamic properties and are not shown here.) In general, the site data are consistent with the EPRI (1993) relationships, except that the site data tend to show higher damping levels at very low shear strains. The EPRI (1993) curves are shown together on

Figure 4.2-7, illustrating the effect of increasing confining pressure (increasing depth) on the nonlinear behavior of soils.

To read:

A set of shear modulus reduction and damping tests were performed on samples taken from borings at the EGC ESP Site, as described in the EGC ESP Geotechnical Report (SSAR Appendix A). Figures 4.2-2 and 4.2-6 show the test results compared to the generic modulus reduction (G/Gmax) and damping relationships developed by EPRI (1993). (Note that one test sample produced what are considered to be erroneous values of modulus reduction and high damping values, as discussed in Appendix A to the SSAR. The test data from that sample were not included in developing the site dynamic properties and are not shown here.) In general, the site data are consistent with the EPRI (1993) relationships, except that the resonant column data tend to show higher damping levels at very low shearing strains. The higher damping from the resonant column tests is attributed to rate-of-loading effects. Damping values from torsional shear tests, which are conducted at frequencies of loading more consistent with predominant free-field ground motions, is very consistent with EPRI damping values. The EPRI (1993) curves are shown together on Figure 4.2-7, illustrating the effects of increasing confining pressure (increasing depth) on the nonlinear behavior of soil. According to EPRI (1993), the EPRI modulus and damping curves were developed to account for the variations in soil shear modulus and material damping with shearing strain and soil confining pressure - with soil confining pressure being approximated within the set of curves by the depth below the ground surface. EPRI (1993) indicates that these curves are appropriate for use in "gravelly sands to low plasticity silty or sand clays", which is consistent with the soil conditions at the EGC ESP Site.

ATTACHMENTS:

RAI 2.5.4-2 Attachment 1 (EPRI Figures 7A-18 &19)

RAI 2.5.4-2 Attachment 2 (EPRI Figure 6-9)

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.4-3

Section 2.5.4.10 (Page 2.5-29) of the ESP SSAR states that, while static stability considerations are not explicitly addressed for the EGC ESP site, high allowable bearing values and low compressibility are expected at the EGC ESP site because of the similarity in soil conditions to those occurring at the CPS site. The static stability discussions in the CPS USAR indicate that allowable bearing pressures range from 25 to 60 tons per square foot (tsf). Based on the bearing values given in the CPS USAR, the minimum site characteristic value for bearing pressures at the EGC ESP site is 25 tsf. This value is listed in the plant parameter envelope (PPE) table (Table 1.4-1). Please provide more detail regarding the method(s) used to determine the bearing capacities for the CPS site. Specifically, discuss how the minimum value of 25 tsf was determined.

EGC RAI ID: R7-15

EGC RESPONSE:

The method used for calculation of ultimate bearing capacities and factors of safety for the CPS site is summarized in Section 2.5.4.10.2 of the CPS USAR (CPS, 2001). As cited in the USAR, "These factors of safety were calculated by conventional bearing capacity analyses assuming a local shear failure condition. It was assumed that the subsoil beneath the foundations is uniform and the mats under the various components of the station are structurally independent with respect to foundation loading and support."

As described in Section 6.2 of SSAR Appendix A, the ultimate bearing capacities for the Category I Structures at the CPS Site (except for the ultimate heat sink outlet structure, which is located near the shore of Lake Clinton) range from 39.9 to 60.6 tsf, as calculated from the bearing capacity analyses outlined above. This range in ultimate bearing capacities corresponds to CPS building foundation elevations ranging from 692 to 702 feet above MSL. These elevations are approximately 35 to 40 feet below the ground surface. During construction of the CPS Facility, the soil was excavated to a depth of approximately 55 feet below the ground surface to remove soils that could be compressible. Approximately 20 feet of highly compacted granular backfill was placed between the base of the excavation and the foundation level for the CPS Facility foundations.

The values of ultimate bearing capacity given in the CPS USAR represent, therefore, a condition where the foundation is placed on approximately 20 feet of highly compacted granular fill over the highly overconsolidated Illinoian Till unit. The undrained shear strength of this layer is likely to be greater than 15,000 pounds per square foot based on an average of the unconsolidated undrained strengths obtained at the CPS Site. This combination of depth below the ground surface and the heavily overconsolidated state of the Illinoian Till result in the high ultimate bearing capacities given in the CPS USAR. For example, the cohesion term alone in the bearing capacity equation results in an ultimate bearing capacity of over 36 tsf.

A minimum bearing capacity of 25 tsf was selected for inclusion in the Plant Parameters Envelope (SSAR, Table 1.4-1). For foundations of future structures at the EGC ESP Site are of a similar size and if they are founded at similar elevations as the CPS structures (with excavation to approximately 55 feet below the ground surface and backfilling with highly compacted granular fill), ultimate bearing capacities of 39.9 to 60.6 tsf may be expected. A minimum bearing capacity of 25 tsf provides a factor of safety greater than 1.5 compared to the minimum calculated ultimate bearing capacity (39.9 tsf) for the CPS Category 1 structures. The 25 tsf ultimate bearing capacity can be met as long as the soil is excavated to the top of the Illinoian Till, and the average undrained strength in the Illinoian Till is approximately 10,000 psf. The actual foundation depth, size, and shape; the specific structure location; and settlement limits will be considered to confirm the final ultimate bearing capacity at COL.

Modifications will be made to the text of SSAR Section 2.5.4.10, as noted.

ASSOCIATED EGC ESP APPLICATION REVISIONS:

Revise SSAR Section 2.5.4.10, page 2.5-29, paragraph three, from:

The static stability discussions in the CPS USAR cover... design of the CPS structures. The bearing capacity discussion indicates that allowable bearing pressures range from 25 to 60 tsf, that settlements were less than predicted, and that conventional methods were used to estimate lateral earth pressures. The very high bearing capacities...

To read:

The static stability discussions in the CPS USAR cover... design of the CPS structures. Ultimate bearing capacities for the CPS structures were computed with conventional methods assuming a local shear failure condition, as described in Section 2.5.4.10.2 of the CPS USAR. The resulting ultimate bearing capacities for the category I structures (except for the Ultimate heat sink outlet structure, which is located near the shore of Lake Clinton) range from 39.9 to 60.6 tsf. Section 2.5.4 of the CPS USAR also indicates that post-construction settlements were less than predicted, and that conventional methods were used to estimate lateral earth pressures. The very high bearing capacities...

Revise SSAR Section 2.5.4.10, page 2.5-29, paragraph four, first sentence from:

Based on the bearing values given in the CPS USAR, the minimum characteristic value for bearing pressures at the EGC ESP Site is 25 tsf.

To read:

Based on the bearing values given in the CPS USAR, the minimum characteristic value for bearing capacity at the EGC ESP Site is 25 tsf.

ATTACHMENTS:

None

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.4-4

In Table 5-2, "Summary of Shear and Compression Wave Velocity Test data," of Appendix A, "Geotechnical Report," to the SSAR, the range in shear wave velocities for some of the stratigraphic units is large. In addition, as indicated in Figure 5-6 of Appendix A, the SPT blow counts for Borings B1 and B4 (not shown in Figure 5-5) appear to be significantly different from those of B2 and B3 in the upper 100 feet of the site. Please justify the appropriateness of using a single "average" soil column for the site response analyses used to develop the site design response spectrum (DRS) rather than including a number of different base-case soil columns.

EGC RAI ID: R7-16

EGC RESPONSE:

As suggested by this RAI, there is variation in the shear wave velocities and Standard Penetration Tests (SPT) blowcounts with depth at the EGC ESP Site. These variations are shown in Figures 5-6 and 5-19 for SPT blowcounts and shear wave velocity measurements, respectively. This type of material property changes is observed at most sites - though to greater and less degree depending on the particular site. The changes result from changes in the depositional conditions during formation of the soil profile and the geologic history of the site following deposition. For the EGC ESP Site, the geologic history includes the advance and retreat of a substantial thickness of ice during the last ice age. This ice loaded the material located below approximately 50 feet, which led to very dense or hard soil conditions (i.e., overconsolidation) by the ice load. One of the consequences of the ice loading is that natural variability of the soil existing below 50 feet after initial formation has been reduced. In contrast the soil in the upper 50 feet was formed by fluvial and aolian processes, resulting in more variability both vertically and horizontally.

In recognition of the natural variability of the soil, the standard approach for site response analyses is to account for the likely variation in soil layering and soil properties within a specific layer by considering different combination of soil property and soil profile conditions that could exist at a site. One method for evaluating these variations is by manually creating independent soil columns, as suggested in the RAI. The alternative that was taken during the EGC ESP Site ground motion response studies was to statistically create a large number of profiles, or realizations, and conduct the site response analyses using these profiles. This approach is discussed in Section 4.2 of Appendix B of the EGC ESP SSAR. As noted in Section 4.2.2 (4th paragraph), sixty (60) profiles were used in the site response analyses. Figure 4.2-9a and Figure 4.2-9b shows these realizations in the upper 500 feet of soil profile. Each realization represents a different column of soil to a depth of approximately 300 feet and rock below.

A similar approach was used to address the potential uncertainty in the EPRI modulus reduction and material damping curves (Figure 4.2-14 through Figure 4.2-18 of Appendix B) - with each depth interval represented by 60 independent sets of modulus reduction and material damping curves. The 60 velocity profiles and the 60 modulus reduction and material damping curves were combined randomly to defined 60 columns, with each column having a different set dynamic soil properties (i.e., low-strain shear modulus, modulus reduction versus shearing strain curves, and material damping versus shearing strain curves) and each with a different soil layers locations. These variations were selected based on the range of conditions reported in statistical studies of site response (e.g., Silva et al. 1996; Toro, 1996). These 60 columns account for the potential variability in soil layering and properties that could occur at the site. These columns were then used with 30 different earthquake records (i.e., each record assigned to 2 soil profiles) to compute the transfer function used to convert rock spectra to the SSE at the ground surface.

The randomization process described in Section 4.2 of Appendix B of the EGC ESP SSAR is consistent with procedures identified in EPRI (1993) and Silva et al. (1996). This approach is described as method 2B in NUREG/CR-6728 (McGuire et al., 2001). The primary difference from the manual process of creating individual columns (i.e., different base-case soil columns) is that the randomization process creates a series of profiles representative of the statistical variation of soil properties across a site.

In summary, the randomization process used to develop the transfer functions at the EGC ESP Site allows the uncertainty in soil layering and soil properties to be considered during the evaluation of site response effects. Use of transfer functions developed through this evaluation method provides a reasonable and accepted basis for adjusting rock motions to the surface of a soil site.

ASSOCIATED EGC ESP APPLICATION REVISIONS:

Revise SSAR, Chapter 2, Section 2.5.2.5, 5th paragraph, as provided in the response to RAI 2.5.4-2.

Revise SSAR, Appendix B, Section 4.2.4, beginning of first paragraph from:

Sixty response analyses were performed with program SHAKE6 to compute the site amplification function for each deaggregation earthquake. The 60 randomized velocity profiles were paired with the 60 sets of randomized modulus reduction and damping curves (one profile with one set of modulus reduction and damping curves). Each of the 30 scaled time histories were used to compute the response of two profile-soil property curve sets.

To read:

Sixty response analyses were performed with program SHAKE6 to compute the site amplification function for each deaggregation earthquake. The 60 randomized velocity profiles were paired with the 60 sets of randomized modulus reduction and damping curves (one profile with one set of modulus reduction and damping curves) to define 60 soil columns, each characterized by a set of shear wave velocities, modulus reduction curves, and material damping curves. Each of the 30 scaled time histories were used to compute the response of two profile-soil property curve sets.

ATTACHMENTS:

None

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.4-5

In Table 5-2, "Summary of Shear and Compression Wave Velocity Test data," of Appendix A, "Geotechnical Report," to the SSAR, the range in shear wave velocities for some of the stratigraphic units is large. In addition, as indicated in Figure 5-6 of Appendix A, the SPT blow counts for Borings B1 and B4 (not shown in Figure 5-5) appear to be significantly different from those of B2 and B3 in the upper 100 feet of the site. Please justify the appropriateness of using a single "average" soil column for the site response analyses used to develop the site design response spectrum (DRS) rather than including a number of different base-case soil columns.

EGC RAI ID: R7-17

EGC RESPONSE:

As discussed in Section 5.2.4.2 of Appendix A to the EGC ESP SSAR, the ratio of shear wave velocity measured in the laboratory to that measured in the field is an indication of the accumulated disturbance that occurs to soil samples when they are removed from the ground, transported to the laboratory, set up in the laboratory, and tested in equipment that may not replicate the stress state and loading conditions in situ during a seismic event. These effects are well-established and are discussed in Section 5.2.4.2.

Only two of the 6 samples tests had velocity ratios (i.e., laboratory to field) of less than 86 percent. Four of the other till samples had velocity ratios greater than 86 percent. The sample with a velocity ratio of 0.68 (referred to in the RAI) was an obviously disturbed sample as evidenced by the shape of its modulus reduction and damping ratio curves and is discussed in Section 5.2.4.2. The other sample with a low velocity ratio (i.e., 0.76) was from a depth of 242 feet below the ground surface. This depth results in relatively large, unavoidable stress relief as the sample is brought to the ground - which may be part of the reason for the lower velocity ratio.

The lower velocity ratios for samples from a depth of 208 and 242 feet had no bearing on the site response studies described in Section 4.2 of Appendix B of the EGC ESP SSAR. As summarized in Section 2.5.2.5 of the EGC ESP SSAR and as discussed in more detail in Section 4.2 of Appendix B to the EGC ESP SSAR, the EPRI (1993) modulus reduction and damping ratio curves were used rather than the results of the laboratory tests. As shown in Figure 4.2-6 of Appendix B, the modulus reduction and damping ratio curves are for practical purposes identical to the EPRI values for the sample from 242 feet, even though this sample had a velocity ratio of 0.76. This observation is consistent with recent work (Anderson, 2003) that has shown that even with a reduced velocity ratio, the shape of the modulus reduction curve is unaffected while the effects on the damping ratio are small. This observation is also counter to the proposal to adjust the results of laboratory derived modulus and damping curves by some factor related to the velocity ratio. The consensus of a number of individuals (e.g., Professor K.H. Stokoe who conducted the combined resonant column/torsional shear tests for the EGC ESP project) is that reliable application of such corrections is not currently supported.

In summary, while there continues to be debate on the importance of sample disturbance on the shapes of the modulus reduction and damping curves, use of the EPRI (1993) modulus reduction and damping curves, together with the explicit modeling of uncertainty in these curves, is considered to provide an adequate basis for computing site response at the EGC ESP Site. Since the laboratory test results were not used in the site response model, the lower velocity ratio noted in the RAI does not affect the results of the site response analysis.

New Reference

Anderson, D.G. "Laboratory Testing of Nonlinear Soil Properties: I & II." Report prepared by CH2M HILL, Seattle, Washington for the Lifeline Research Program, Pacific Earthquake Engineering Research Center, University of California at Berkeley. December 2003.

ASSOCIATED EGC ESP APPLICATION REVISIONS:

Revise SSAR, Appendix A, Section 5.2.4.2, 8th paragraph, as provided in the response to RAI 2.5.4-2.

Revise SSAR, Appendix A, Chapter 8, References to add the following new reference:

Silva W.J., N. Abrahamson, G. Toro, and C. Costantino. "Description and Validation of the Stochastic Ground Motion Model." Report prepared by Pacific Engineering and Analysis, El Cerrito, CA for the Engineering Research and Applications Division, Department of Nuclear Energy, Brookhaven National Laboratory, Contract No. 770573. 1996.

ATTACHMENTS:

None

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.4-6

Section 6.1 of Appendix A describes the method used to determine the potential for soil liquefaction at the ESP. Please provide a copy of a sample liquefaction analysis from one of the four ESP Site borehole locations and clearly show how the Factor of Safety (FOS) was determined for the different soil layers. Since each of the four boreholes sampled soil layers with FOS less than 1.1, provide a description of the methods that will be used to mitigate the potential for liquefaction. In addition, describe the extent (i.e., depth below ground surface, and thickness) of the non-cohesive (silts and sands) soil layer over the site area, particularly the ESP footprint.

EGC RAI ID: R7-18

EGC RESPONSE:

[A] A sample liquefaction calculation (for Borehole B-1 at the 38.5-foot depth interval) is provided in the attachment to this RAI response. The methodology followed in this analysis was the Youd et al. (2001) procedure for determining liquefaction potential using the Standard Penetration Test (SPT) method. Additional detail on the methodology used for the liquefaction analyses at the EGC ESP Site is provided in EGC ESP SSAR Appendix A, Section 6.1.1. Interpretations of these results are provided in Section 6.1.2 of the same appendix.

[B] The second part of this RAI requests a description of potential methods that may be used to mitigate liquefaction potential. Potential methods to manage liquefiable soils during construction are discussed in EGC ESP SSAR Appendix A, Section 6.1.2, paragraph 2. As noted, one method would involve removal of liquefiable materials to a depth of 55 to 60 feet below the ground surface, similar to what was done at the CPS Site, and replace the excavated material with heavily compacted granular backfill. The alternative approach would be to use some type of ground improvement method to improve liquefiable soils. The types of ground improvement methods that could be considered include:

- Use of vibro-densification methods. One common technique for mitigating the potential for liquefaction is to densify the soil using an impact or vibratory procedure. The vibratory procedure typically involves use of a vibrating probe to densify soils. Depths in excess of 90 feet below the ground surface can be reached. Probing is usually done at relatively close spacing, and can involve repeated passes over an area to achieve the required level of improvement. SPTs or Cone Penetrometer Tests (CPTs) are often performed as part of the densification program to check on the degree of densification being achieved.
- Use of stone columns. These columns are constructed of highly compacted gravel. They are usually placed every 5 to 10 feet, and they typically have a diameter of 2 to 3 feet. The stone column mitigates liquefaction by densifying the soil between the columns and by providing a system for draining excess porewater pressures caused by the earthquake during and following shaking.
- Use of in-place soil cement mixing. This approach involves mixing granular soil and cement in place to create a cement-treated soil. The cement binds the sand particles together to reduce the potential for liquefaction. Methods of mixing the soil in place

include jet grouting and in place soil mixing. The mixing can produce either a stabilized area or a series of cells which confine the material that liquefies. By confining the liquefiable material, the tendency for lateral spreading and bearing failure is reduced; however, it is still necessary to consider the settlement that results as porewater pressures dissipate.

- Use of earthquake drains. Recent studies have found that by placing drains at close spacing the tendency for liquefaction to develop can be mitigated. The drains provide a path for porewater to escape as the shaking occurs.

In summary, a variety of methods are available for improving soils. Most of these methods have been tested in severe earthquakes, and have demonstrated that they can successfully control the potential for and consequences of liquefaction.

[C] The third part of this RAI is a request for a description of the extent (i.e., depth below ground surface, and thickness) of the non-cohesive (silts and sands) soil layer over the site area, particularly the ESP footprint. The extent of non-cohesive soils at the EGC ESP site is generally limited to outwash intervals in the Wisconsinan till, Interglacial zone, and upper 15 feet of the Illinoian till. Interbedded silt and sand layers were encountered from 62 to 72 feet bgs at Borehole B-2, sand was encountered from 43 to 60 feet bgs at Borehole B-3, and clayey sand was encountered from 49 to 59 feet bgs at Borehole B-4. Additional thinner layers of non-cohesive sands and silts were observed in shallower intervals in B-1 and B-2.

Not all of these non-cohesive soils are considered liquefiable for the design conditions. At all four boreholes, the Illinoian till becomes uniform and cohesive with depth, below the outwash deposits. Even though the Illinoian till typically has a USCS classification of ML (silt), the corresponding Atterberg limits indicate that it is borderline clay, as described in SSAR Appendix A, Section 5.2.2.4. Therefore, the cohesive, uniform silts of the Illinoian till are not considered liquefiable.

ASSOCIATED EGC ESP APPLICATION REVISIONS:

None

ATTACHMENTS:

RAI 2.5.4-6 Attachment (Calc sample)

NRC Letter Dated: 07/26/2004

NRC RAI No. 2.5.4-7

Appendix B of the ESP application describes the process used to generate the soil surface design ground response spectrum. In particular, Figures 4.2-14 through 4.2-18 present the results of the randomizations for soil shear modulus degradation and hysteretic damping curves used for the site response calculations. Please explain how these curves were used in the randomization process with respect to both the different depth ranges and the soil types occurring within those depth ranges. For example, the boring logs indicate that some soils are clays and some soils are silty sands over a particular depth range. Secondly, the damping curves used in the calculations do not incorporate the 15 percent damping cutoff as recommended in SRP Section 3.7.2. Since the calculations are nonlinear, and since some of the randomizations can lead to relatively high strains, it is not clear how this influenced the computed site amplification factor. Please provide clarification regarding the use of high strain values in the randomization process.

EGC RAI ID: R7-19

EGC RESPONSE:

1) The first part of this response deals with the use of randomized sets of modulus reduction and material damping curves to represent different depth ranges at the EGC ESP Site, and the soil types within the depth ranges, particularly relative to the soil classifications in the boring logs. As identified in the RAI, the soil boring logs indicate that the soil layering at the EGC ESP Site consists of both clays and sands.

The curves shown in Figure 4.2-14 through 4.2-18 are, however, dependent only on the depth range and not on the material type. This is consistent with the development of the EPRI modulus reduction and material damping curves (EPRI, 1993) - where the standardized curves are based only on depth interval, thereby avoiding the need to link the modulus reduction curves and damping curves to the soil boring log, as suggested by the RAI. The randomized modulus reduction and material damping curves are then combined with randomized low-strain shear wave velocity profiles to define a large number of independent soil columns for the site.

The independence of the modulus reduction and material damping curves from the specific soil type results from the information presented in Section 7.A.6 of Volume 2 of the EPRI (1993) set of reports, which concludes that the primary variable contributing to the variation in shape and absolute value of the modulus reduction and material damping curves is the depth of the soil below the ground surface, rather than soil type. The depth of the soil is, in turn, an indication of the effective confining pressure on the soil sample.

Since there is evidence that the type of soil also has some effect on the shape and magnitude of the modulus reduction and material damping curves - though it can be considered a secondary effect - a range of unique modulus reduction and material damping curves is computed within each depth interval (i.e., 0 to 20, 21 to 50, 51 to 120, etc.) through a randomization process. The computation performed for the EGC ESP project resulted in 60 modulus reduction curves and 60 material damping curves with each of the five depth intervals, as discussed in Section 4.2 of Appendix B to the EGC ESP SSAR. The range represented by each of the 60 sets of curves is intended to cover the uncertainties in the shape and absolute value of the modulus reduction and material damping ratio curves resulting from a number of different effects, including the particular soil type and characteristics, the stress history for the soil, sample disturbance associated with the laboratory testing of soil samples, and random variability that is typically observed in laboratory testing programs.

The 60 sets of modulus reduction curves and the 60 sets of material damping ratio curves for each depth interval were randomly assigned to 60 sets of randomized shear wave velocity profiles. These velocity variations account for the spatial variability in velocity that could occur with depth, as well as the variation in soil layer thickness. A standard deviation of 0.2 (log normally distributed) was used in developing the variation in shear wave velocity based on work by Toro (1996); a +/- 20 percent variation in layer thickness was assumed in the randomization of layer thickness.

By combining the randomized velocity profiles with the randomized modulus reduction and material damping curves, 60 profiles or soil columns were developed, each with a randomly assigned modulus and material damping curve. These combinations of soil property curves were used in the computer program SHAKE to develop over 180 unique transfer functions representing the modification of seismic wave propagation between the input motion at hard rock level and the ground surface for each hazard level (10^{-4}, 10^{-5}) and frequency range (5-10 Hz and 1-2.5 Hz). Thirty earthquake records were used when conducting these analyses. The 30 earthquake records were obtained from McGuire et al. (2001) and represented earthquakes occurring in central Illinois (DEL), the Wabash Valley-southern Illinois region (DEM), and a characteristic New Madrid earthquake (DEH). These earthquakes were also selected to match the high (5 to 10 Hz) and low frequencies (1 to 2.5 Hz) rock spectral shapes. The recordings were scaled to approximately match the target response spectra for the three deaggregation earthquakes developed for target spectra (see Figures 4.2-21 and 4.2-22 of Appendix B to the EGC ESP SSAR).

Although this approach does not explicitly relate soil type and layer thickness to soil boring logs, it keys the analyses to shear wave velocities that were measured at the EGC ESP Site, and these velocities are a direct indication of the soil type represented on the boring logs. Input to these response analyses also includes the soil unit weights for each soil layer. These unit weights were obtained from laboratory testing work conducted on samples obtained from the site and portrayed in the boring logs.

In summary, the randomization process for modulus reduction and material damping curves, as well as the low-strain shear wave velocity and thickness profiles, results in combinations of soil stiffness and damping conditions that account for the possible variations in soil type, soil layer thickness, and dynamic soil properties for the EGC ESP Site. By keying the analyses to the shear wave velocity measured at the EGC ESP Site, variations in soil type are included in the analyses.

2) The question regarding the use of a 15 percent damping cutoff for free-field site response calculations involves a fundamental issue as to whether to apply a 15 percent cutoff during free-field, site response analyses. Usually the 15 percent damping in SRP Section 3.7.2 pertains to soil-structure interaction (SSI) problems, where geometric or radiation damping in some modes of response (e.g., transnational) can become very large in lump-mass analyses. For soil-structure interaction analyses, the high damping can result in significant reductions in response spectra if no limitations are placed on damping. In order to avoid unconservative response calculations for SSI, a limit on

radiation or geometric damping is usually required and sometimes this limitation is achieved by limiting the contributions from material damping.

Though the use of a 15 percent cap on material damping for use in free-field, site response analyses has been suggested at times, there is no evidence in laboratory testing programs that material damping should be capped at 15 percent (e.g., EPRI, 1993). Results of work described in NUREG/CR-6728 (McGuire et al., 2001) also do not suggest that a 15 percent cap was necessary to achieve reasonable comparisons between recorded and predicted site response. Finally, the standard-of-practice when conducting free-field analyses also does not limit the material damping in any model of site conditions to 15 percent.

For a stiff site such as occurs at the EGC ESP Site, the 15 percent cutoff is expected to have little effect except perhaps in the shallowest soil layers, where the shear modulus is lowest. The SHAKE program uses an equivalent linear rather than a nonlinear soil model - with a constant shear modulus and material damping used in each iteration. The analysis is iterated until the equivalent shearing strain during the iteration is consistent with the shearing strain used to estimate the shear modulus and material damping within a certain degree of accuracy. Where soils are relatively stiff and peak ground acceleration only moderate, such as occurs at the EGC ESP Site, the equivalent shearing strains will often be low enough that damping ratios do not exceed 15 percent. Only the upper 50 feet or so of soil profile at the EGC ESP Site, where the shear modulus is reduced, could the site response potentially be affected by the damping cutoff.

However, to support the response to this RAI, a series of supplemental computer runs were conducted using the low-strain shear wave velocity profile, the SHAKE program, and modulus reduction curves as discussed in the EGC ESP SSAR. For these supplemental analyses, the material damping curves in the EPRI soil model were capped at 15 percent. The results of a representative set of these analyses compared to the mean transfer functions shown in Figure 4.2-23 and Figure 4.2-24 from Appendix B of the EGC ESP SSAR. This comparison indicated that the 15 percent damping cap results in no more than a 2 percent increase in the transfer function for the 10^{-5} hazard level motions and much less for the 10^{-4} hazard level motions for the EGC ESP Site. These effects are considered negligible.

In summary, a 15 percent cutoff for material damping during free-field ground response studies using the equivalent linear computer program SHAKE was not applied. Nevertheless, analyses have been conducted to evaluate the potential effect of this cap. When this cutoff is imposed during a series of supplemental response analyses, the results indicated only a minor effect on the transfer function, which will result in an increase of the transfer functions of less than 2 percent.

ASSOCIATED EGC ESP APPLICATION REVISIONS:

Revise SSAR, Appendix B, Section 4.2.2, next to last paragraph from:

Equations 4-6 and 4-7 were used to define the variability of G/Gmax and damping ratio about the average curves presented in Figure 4.2-7. Figures 4.2-14 through 4.2-18 show the 60 sets of simulated G/Gmax and damping curves.

To read:

Equations 4-6 and 4-7 were used to define the variability of G/Gmax and damping ratio about the average curves presented in Figure 4.2-7. Note that these curves are not tied directly to the soil type or consistency shown in the boring logs. Rather they are related only to depth interval, as discussed in EPRI (1993). However, since the modulus reduction curves and material damping curves are later tied to the randomizations of the shear wave velocity measured at the site, the effects of different soil types represented in boring logs is explicitly accounted for in the analyses. Figures 4.2-14 through 4.2-18 show the 60 sets of simulated G/Gmax and damping curves.

ATTACHMENTS:

None

RAI ATTACHMENTS

The following attachments are provided on the enclosed CD-ROM.

RAI 2.5.1-2 Attachment 1 (Obermeier pc 20021107)

RAI 2.5.1-2 Attachment 2 (Obermeier pc 20030110)

RAI 2.5.1-2 Attachment 3 (Obermeier pc 20030513)

RAI 2.5.1-2 Attachment 4 (Obermeier pc 20030809)

RAI 2.5.1-2 Attachment 5 (Olson 2003)

RAI 2.5.1-2 Attachment 6 (Olson et al 2004)

RAI 2.5.1-2 Attachment 7 (Green et al 2004a)

RAI 2.5.1-2 Attachment 8 (Green et al 2004b)

RAI 2.5.1-3 Attachment (Revised App B Figure 2.1-13)

RAI 2.5.1-4 Attachment (Revised App B Att 1 Figures B1-13-14-15)

RAI 2.5.2-2 Attachment (Revised App B Figure 2.2-2)

RAI 2.5.2-6 Attachment (Figures 1A-4A)

RAI 2.5.2-7 Attachment 1 (RPK Lyon 1997)

RAI 2.5.2-7 Attachment 2 (RPK Tokyo 2001)

RAI 2.5.4-1 Attachment 1 (Soil Property Info)

RAI 2.5.4-1 Attachment 2 (Revised App A Table 5-2)

RAI 2.5.4-2 Attachment 1 (EPRI Figures 7A-18&19)

RAI 2.5.4-2 Attachment 2 (EPRI Figure 6-9)

RAI 2.5.4-6 Attachment (Calc sample)

Kathryn Hanson

From: Stephen Obermeier [sobermei@yahoo.com]
Sent: Thursday, November 07, 2002 9:46 AM
To: Kathryn Hanson
Subject: revised value of M for magnitude bound method.

Kathryn,

Using a value of 7.6 for the 1811-12 New Madrid eq, the magnitude bound method for the Vincennes eq of ~ 6000 yr BP yields M 7.3.

You might want to contact BuddySchweig for a reference for the value of the 1811-12 eq. Latest thinking according to the big wheels is that the value was 7.6 +/- 0.5.

Also, please see attachment for a brief resume, as you requested.

Steve

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Technical Editing Guidance Reference Source Documentation

Source Document No. SSAR-330-SD-Obermeier-2003a

Document Title:	Written (electronic mail) Communication to Kathryn Hanson.
Originating Organization:	U. S. Geological Survey, Emeritus, Reston, Virginia; EqLiq Consulting.
Author:	Obermeier, S.F.
Document No:	
Volume/Page/Date:	January 10, 2003.
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Describe the relevance of the reference to the report topic.	
SSAR – Appendix B	

99AR-330

Kathryn Hanson

From: Stephen Obermeier
Sent: Friday, January 10, 2003 10:12 AM
To: Kathryn Hanson
Subject: Vincennes eq re-evaluation

Kathryn,

Please see the attachment.

Steve

Kathryn, Jan 10, 03

A few days ago we spoke about the results of re-analysis of the paleo-eq that was centered near Vincennes, IN, about 6,100 yr BP. The re-analysis was done by Russell Green, using the energy solution developed - and evaluated - in his PhD thesis. I sent you the email address for that thesis a few days ago. In his development of a totally new method, I think that he showed that ALL previous energy-based methods had serious flaws, and he developed appropriate new relations. The man is a clearcut genius.

The previous interpretations of magnitude for the Vincennes earthquake, discussed by Pond in his PhD thesis at Virginia Tech, were based on the magnitude-bound method, the cyclic stress method, and the energy-stress method. The magnitude-bound method was based on M for the largest New Madrid earthquake of 1811-12 being M~8, which is now in question. The cyclic stress method Pond used utilized the existing relations for pga that had been developed for the central US, as well as existing relations for magnitude scaling factor. The energy stress method was a composite of the cyclic stress method with existing energy attenuation relations that had been used by many researchers. Using all those methods, Pond consistently got a value of M~7.5 for the Vincennes paleo-earthquake.

Russell and I both think the energy-stress method that Pond used had serious flaws, both conceptually and in use of existing energy attenuation relations (which, it turns out, should not be used for liq analysis). And again, there was the matter of the questionable tie-in of the magnitude-bound method with the value of M for the largest New Madrid eq of 1811-12. That leaves only the cyclic stress method that Pond used as a serious candidate for being valid.

Russell just went through re-analysis of the Vincennes eq using the more recent attenuation relations of pga for the central US (four of them) developed by Somerville et al. (2001), by Campbell (2001), by Atkinson and Boore (1997) and Toro et al. (1997); Russell and I also went through the 50 or so boring logs in Pond's thesis to select appropriate SPT values for the re-analysis. Russell also used the most recent magnitude scaling factors, suggested by Youd and Idriss. For all these solutions, for the cyclic stress method the best estimate of M for the Vincennes eq ranged from 7+ to 7.5. Using the energy-based solution that he developed, and which I trust most of all (because it totally circumvents use of the magnitude scaling factor, which is a large questionable factor in the use of the cyclic stress method in the central US), he obtained a value of M~7.5 for each of the four newer attenuation relations.

The final draft of the paper with this analysis should be completed within a week. The paper has a large scope, ranging from proper selection of field sites for engineering testing (both locally and regionally), evaluation of the validity of the field test data, to proper geotechnical analysis of the data. I will keep you posted about availability of the paper, which we plan to release first as a USGS Open-File Report on the www. Authors are Olson, Green, and me. (Scott Olson is another supersmart and experienced geotech PhD, recently from U of IL; I just tag along with these geniuses.)

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Technical Editing Guidance Reference Source Documentation

Source Document No. SSAR-331-SD-Obermeier-1998

Document Title:	Written (electronic mail) Communication to Kathryn Hanson.
Originating Organization:	U. S. Geological Survey, Emeritus, Reston, Virginia; EqLiq Consulting.
Author:	Obermeier, S.F.
Document No:	
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Describe the relevance of the reference to the report topic.	
SSAR – Appendix B	

SSAR - 331

Kathryn Hanson

From: Stephen Obermeier [sobermei@yahoo.com]
Sent: Tuesday, May 13, 2003 8:14 AM
To: Kathryn Hanson
Cc: Scott Olson2; Rich Harrison; John Hill; Russell
Subject: summary and critique of eqs in IN-IL

To: Kathryn HAnson, Geomatrix, San Francisco
From: Steve Obermeier
CC: Russell Green, U of Michigan
Scott Olson, URS, St Louis
John Hill, Assoc. Dir., IN Geol Survey
Rich Harrison, USGS Reston

Kathryn,

Attached is a writeup with the information you requested, plus other background information that I believe is relevant regarding the analysis of paleo-earthquakes in IN-IL. Russell Green has revewed the writeup, and he says that if you have any questions concerning ground motion anaysis, you can email or phone him (734-764-3668).

Steve

IL-IN paleoearthquakes – a critique

A Brief Summary And Critique of Previous Paleoseismic Interpretations in IN-IL

Questions have arisen recently concerning the bases and validity of interpretations for paleoseismic earthquakes in IN-IL, especially in central IL as well as in the vicinity of the IL-IN border. Of particular interest has been the mid-Holocene earthquake that has been interpreted as centered near Vincennes ("the Vincennes earthquake"), and the mid-Holocene earthquake interpreted as being centered NE of Springfield, IL.

Much has been written the past 10 years in technical journals and reports available to the public, in publications having the lead authors of Stephen Obermeier, Patrick and Cheryl Munson, or Eric Pond. The purpose of this writeup is to provide a summary and critique of the geologic and technical information in their reports, as well as to summarize the provisional findings of on-going geotechnical research on paleoseismic interpretations in the region by Russell Green, Scott Olson, and Steve Obermeier. (Dr. Russell Green is a geotechnical engineering professor (specialties in seismic liquefaction, ground motions, and seismic structural design) at U of Michigan; Dr. Scott Olson is a design engineer at URS Corporation in St. Louis (specialties in seismic liquefaction, seismic ground failure, and seismic design).

Two techniques have been used for analysis of the value of M for paleo-earthquakes in the region: the magnitude-bound method, and the cyclic stress method or some variant of it (e.g., the energy-stress method of Pond, 1996, which is written up in Pond and Martin, 1996). (References that are cited are listed at the end of this writeup.)

Brief Discussion of Methods Used for Analysis of M in IN-IL

The Magnitude-Bound Method. The magnitude-bound method is based on comparison with liquefaction effects from historic earthquakes in the same region (i.e., the same tectonic setting). The method is based on locating of the most distal effects of liquefaction from the epicenter or else from the "energy center." (The energy center is the region of strongest bedrock shaking, and thereby a much more meaningful parameter than epicenter for analysis of larger magnitude earthquakes.) Obermeier et al. (1993; USGS Prof. Paper 1536) first used this method for eqs in IN-IL by assuming the shaking characteristics (basically the frequency content and amplification of pga) at distal sites of liquefaction in the Wabash Valley were the same regardless of whether the energy center was in the NMSZ or in IN-IL (i.e., in the Wabash Valley or anywhere else in IN-IL). This distinction of "distal sites" is very important, because the depth to bedrock in the NMSZ is much, much greater than in IN-IL.

For use of the method in the Wabash Valley, calibration points that were used were Obermeier et al (1993; in Prof. Paper 1536) were taken from eqs that occurred in the NMSZ in years 1811-12 and in 1895 (near Charleston, MO). The value of M for the 1811-12 eq centered at New Madrid was taken to be 8.3, which was the value advocated by Arch Johnston at the time; the range of liq effects was taken to be somewhere between

240 and 275 km from the town of New Madrid, which were the most distal locales distances of liq that were historically reported in IL-IN. The 240 km site was just across the Miss River from St. Louis, and was well documented in a letter found in the St. Louis U. archives (actually, a warehouse), a copy of which was forwarded to me from either Otto Nuttli or Bob Hermann. The distance of 275km was at the confluence of Wabash and Ohio Rivers, and was reasonably well documented at the time of the 1811-12 eqs by observations of sand blows. Several years ago I found very young, small dikes very near that confluence, which tends to confirm that historic observation.

For the 1895 eq of M~6.8, entered near Charleston, MO, the maximum range of liq effects was almost certainly less than 20-30 km. An attempt was made in USGS Prof Paper 1536 (Obermeier et al., 1993) to account for differing ground motion amplification factors in the NMSZ versus the much shallower bedrock conditions in IN and IL. The program RATTLE was used by Art Frankel (USGS) and SHAKE was used by Jimmy Martin (VA TECH). The results of RATTLE (amplification factor X 1.9) were somewhat higher than SHAKE (amplification factor X 1.1), but it should be noted that RATTLE included frequencies up to 50 Hz whereas SHAKE only went up to 25 Hz. Regardless of which is more correct, from a practical viewpoint the location of the data point the calibration point of the magnitude-bound curve is not much affected (which can be seen by Fig. 11 in Prof. Paper 1536). A point of relevance is that Russell Green tells me that slight amplification is probable for the setting of deep alluvium at low values of g – which is in essence is the value that was used to initially develop Fig. 11.

Eric Pond (1996; pages 198-201) later added another calibration data point to the curve, for a M~ 5.6 eq that struck near Charleston, MO, in year 1851. The eq produced a single liq feature. Thus three data points at much different values of M were available to develop a calibration curve. This calibration curve is shown as Fig. 9B, in Obermeier and Pond (1999).

The question must be asked of whether those data points in Fig. 9B for the M 6.8 and the 5.6 eqs are reasonable, at least in terms of distance from the energy center. To answer that question, two properties are especially relevant: liquefaction susceptibility and bedrock attenuation. The sediments in the NMSZ and in IN-IL having highest liq susceptibility are typically well graded (engineering sense) medium-sized sands. These sands were laid down by fluvial processes. Other sand deposits, worldwide, are much more susceptible (e.g., uniformly sized, very fine sands such as those found in many beach/dune settings.). Thus it is not surprising that that the data points for the M 5.6 and 6.8 eqs in the NMSZ lie within the range for other eqs, worldwide.

It should also be noted that the sediments at the most distal sites of liq that were used for the calibration were all almost certainly hundreds if not thousands of years in age, and thus the factors of aging were likely comparable for both calibration points as well as the data that were used for estimation of the magnitude of the mid-Holocene eqs in the area of IN-IL.

Since the time when Pond (1996) first used the curve to estimate values of M in the region of IN-IL, the value of M (8.3) for the 1811-12 NMSZ eq has come into serious question. Many researchers favor a value of 7.6-7.7 +/- 0.5 as the value of that eq (see Wheeler and Perkins, 2000), although Arch Johnston apparently still favored a value of M~8.

Part of the reason for favoring a value < M 8 appears to come from analogy with the M 7.7 Buhj (Gujarat) eq of 2001, which struck vicinity of the India/Pakistan border. A writeup I read a year or so ago noted that the farthest liquefaction features developed at essentially the same maximum that was observed from the NMSZ eqs of 1811-12. That observation from the Buhl eq, plus the contention by some researchers that both the Buhj and the NMSZ eqs were in the same tectonic setting, appeared to give credence to the notion the largest NMSZ eq of 1811-12 was M~ 7.6-7.7.

However, Seth Stein of Northwestern University plus a co-author (in a GSA article about 2 years ago) have called into question the validity of comparison of the tectonic setting for the Buhj eq with eqs in the NMSZ. Of much credence to me are the comments by a very competent USGS geologist who has spent much time in the field in the vicinity of the Buhj eq during the past 15 years, doing structural geology and other studies; that person has extensively discussed the field setting of the Buhj eq with me, and he says that the structural scenario by Stein and another is undoubtedly the more correct. Regardless of who is correct, there are serious doubts in comparing the tectonic settings of the Buhj and NMSZ eqs.

So, there arises the question of which magnitude to use for the 1811-12 eqs for the calibration curve of the magnitude-bound method. Regardless of whether M 8 or M 7.6/7.7 is used, the data point extends far beyond that from any form other locales, worldwide (see Obermeier and Pond, 1999, Fig. 9B). That seems reasonable, given the well known attenuation relations for the central/eastern US.

Let us return now to the question of the interpretation of the value of M for the eq centered near Vincennes, IN, in 6,100 yr BP. The maximum distance of liq effects from the energy center of that eq has been interpreted by Munson and Munson (1996) to be at least 170 km (I address this matter of distance below.) Using a value of M 7.6/7.7 for the 1811-12 eq to define the calibration point for the magnitude-bound curve and a distance of 170 km yields a value of M~7.5 for the Vincennes eq.

On a related matter, note the data point from the 1988 Sanguenay eq, in Fig. 9A in Obermeier and Pond (1999). Even if the calibration curve is determined by using that data point in conjunction with the value of M 7.6/7.7 for the 1811-12 NMSZ eq, a value of 7+ is yielded for the Vincennes eq.

Obviously, if the value of M for the 1811-12 eqs in the NMSZ was M~8, then the interpreted value for the Vincennes eq would be slightly higher than those cited in the two preceding paragraphs.

The Cyclic-Stress Method (i.e., the Seed-Iris method). The cyclic stress method also was used by Pond (1996) to estimate the value of M for the Vincennes eq. His back-calculated result was M~ 7.7. His methodology is being thoroughly reviewed by Green, Olson, and Obermeier. Two factors that he used are being evaluated: (1) the N value he selected (taken from SPT testing at some 13 sites, mainly located from N to S along the Wabash River; (2) the use of SHAKE for estimating ground motion amplification. In addition, more modern attenuation relations that have been developed by seismologists for the region are being used in our analyses, as well as more recent values of Magnitude Scaling Factors (MSF); in addition, an energy-based solution developed by Green in his thesis is being used, and that method does not require value of MSF. Whereas our evaluation of the Vincennes eq is not yet complete, it is very far along.

For a representative value of N1 at each of the sites, Pond used the highest of the minimum value of N1 that was observed in multiple borings at a site of liquefaction. Typically, 3 to 5 borings were taken by Pond at each site. This approach was taken by Pond because one can be confident that only the most susceptible sediment liquefied at most sites of liquefaction.

In our review, we feel that some of these minimum values used by Pond are likely to be too high because boring locations were made back from the stream bank, typically 50 ft or so from where observations of liq features were seen in vertical view in the stream bank. This lack of our confidence results from the highly variable nature of the fluvial deposits, especially where they were laid down as (glaciological) braid-bar deposits. (It should be noted that this problem of variability is not so severe in point-bar deposits of the Wabash River.) Because of this problem of variability, we typically selected values of N1 that were present in multiple borings at a site of liquefaction.

Green feels that the method in which Pond used SHAKE introduced some uncertainties, because energy from frequencies higher than about 10 Hz were not considered. For re-analysis, Green used the method recommended by NEHRP, but Green says that method also has serious uncertainties for use in the central US. Green and Cameron (2003) recently presented a paper at the 2003 Pacific Conference on Earthquake Engineering illustrating this problem with the NEHRP procedure. Still, for analysis of the Vincennes eq, Green used the NEHRP procedure because nothing more credible is available.

Green also used attenuation relations for bedrock motions that have been proposed by Somerville et al (2001), by Campbell (2001), by Atkinson and Boore (1997), and by Toro et al (1997).

The preliminary result of the analysis is obviously very long, and as I noted, not ready for publication. Suffice it to say, though, that a value of M for the Vincennes eq for all the solutions listed are above M 7, with some in excess of M 7.5.

Two other items are in order here. One concerns a method that Pond developed as part of his thesis, called the "energy-stress" method. We believe that because of problems listed above, plus others, that the results of the energy-stress method are not reliable.

Finally, a few words are in order concerning whether the sites of liquefaction where Pond conducted testing for the Vincennes eq were, in fact, caused by shaking of the Vincennes eq. I bring up this matter because a few persons have very recently raised this question. Since then I have gone through the (extensive) data in the report in Munson and Munson (1996), and I address this issue below in more detail. But, the short answer is that there can be little doubt of the validity of their initial interpretation, especially south of Vincennes. And, I should note that using only sites from south of Vincennes, the back-calculated value of M for the Vincennes eq lies in the range that I cited above.

Brief Discussion of Liquefaction Caused by The Vincennes and The Springfield Earthquakes

The Vincennes Earthquake. The aerial extent of this earthquake was determined by Munson and Munson (1996) by using a combination of factors, mainly radiocarbon dating, stratigraphy and weathering, and artifacts from Indians. The Munsons are archeologists, and a note is in order to explain their qualifications to do this interpretation.

Pat Munson is a professor at Indiana University, and Cheryl is employed at an archeological institute at the university. Both have been long-employed in these positions, and both are very experienced in working in field archeology in the central US, studying and dating Indian artifacts. To that end, they have developed a very strong understanding and knowledge of fluvial sedimentation processes and dates of fluvial terraces in IN and IL; they have achieved these qualities by their own effort plus working with the acknowledged experts in stratigraphy/weathering processes in the central US. In addition, both are extremely conscientious in data collection and interpretations, and they also bring along the quality of whip-smart intelligence. The suite of skills they have is essential for making interpretations in the central US in regard to dating of liquefaction, largely because of the overall paucity of materials for radiocarbon dating that can be used for close bracketing of an occurrence of liquefaction at many sites. I am very comfortable in saying that the Munsons are unequivocally the most qualified persons to do dating and regional analysis of liquefaction effects in IN-IL, and probably the entire central US; an examination of the report by Munson and Munson (1996) should demonstrate that point. It is indeed fortunate that the Munsons did this work.

Now, back to the Vincennes earthquake and the regional extent of liquefaction from that eq (see Munson & Munson, beginning on p.44). There are only two locales where the time occurrence of liq can be closely bracketed within the region of liq associated with the Vincennes eq. One of these is south of Vincennes at site BR, located ~ 75 km S of Vincennes, where a radiocarbon age of 6,100 yr BP. The other is from OH1 and OH3 (about 1000 m from OH1) on Eel River, located ~ 90 km NNE of Vincennes, where nut shells and wood pieces slightly post-dating the event yielded data of 5,690 yr BP and 6,190 yr BP, respectively.

If the data from OH and BR were from separate eqs that were closely spaced in time, then one might expect sand blows in the Wabash River region (which commonly have gravels) to show the evidence of different episodes of shaking. Separate layering of gravels has not been observed, but likely would have had it been present.
It should also be noted that going N of Vincennes, along the Wabash River, there is a progressive decrease in abundance and sizes of liquefaction features, although the plot is certainly not smooth and there is what may be an anomalous higher concentration in the distance range of 100-150 km NNE of Vincennes (see Munson & Munson, Table 6-1). Still, if a sizeable eq had taken place near the Eel River (which is located about 25-40 km from the Wabash River, and more or less parallels it), then it is doubtful that such a progression would be seen.

However, in support of another eq located along Eel River at the OH sites, it can be seen in Munson et al (1997) that there is a concentration of closely spaced liq sites that was discovered by the Munsons (see Munson et al; 1997, for the concentration in Fig. 6). A significant number of the liq features are relatively sizeable, being dikes in the 0.15-0.5 m range. But, in a phone conversation with Pat Munson within the past few days, he informed me that (1) the quality and number of exposures along the Eel River was extremely high because channelization downstream had caused the river to aggressively cut, both down and laterally, and (2) the sediments along Eel River are very probably more liquefiable than those typically bordering the Wabash River in that part of the world; whereas the sediments along the Wabash are commonly gravel-bearing those along in Eel River are typically packages of very thick, finer-grained clean sands – and therefore more likely to form liquefaction features. Concerning multiple source regions vs. one near Vincennes, it should also be noted that there is a similar concentration of liq features to the WSW of Vincennes, along the Little Wabash River (see Obermeier, 1998, Fig. 4); the symmetry of the dikes on nearly opposite sides of Vincennes argues strongly for an energy center near Vincennes in my opinion.

In summary, I think the evidence supporting the original interpretation of the Munsons is quite strong north of Vincennes, and nearly unequivocal south if Vincennes,

I should note that in Pond's thesis (which is the same as the report by Pond and Martin, 1996)) he has incorrectly drawn the southern limit of liquefaction from the Vincennes eq. The southern limit along the Wabash River extends at least to the confluence of the Wabash and Ohio Rivers, as shown in Obermeier (1998), and perhaps much farther south in IL.

The Springfield Earthquake. The question has arisen whether the liq features that are in the vicinity of Springfield, IL, might have been from an eq centered far away. If another eq had made the liq effects around Springfield, then there should either be evidence for an orderly progression of liq effects toward Springfield from other eq sources, or else there should be evidence for a "bounce" of seismic energy off the Moho. In answer, it can be seen clearly in Obermeier (1997) and in McNulty and Obermeier (1999) that there is a concentration of larger-sized dikes in what is named the Springfield Earthquake, and certainly no field evidence for an orderly progression of liq effects toward Springfield.

And, I have done sufficient field work in the region to know that there is no good geologic reason related to liquefaction susceptibility that can explain that pattern. Concerning the idea of a bounce, no such effects was observed from the NMSZ eqs of 1811-12, and therefore invoking that model would seem to have little support.

Another reason for believing that an energy center is located within the region of liq shown for the Springfield is that there are two episodes of time-separated liq that are clearly present in the region. These times were not widely spaced, being perhaps only a few hundred years. In the areas of liq features within 100 km S and SE of the Springfield region, no evidence has ever been found for two episodes. To my mind, that also indicates the uniqueness of earthquake that produced the Springfield area liq. Simply because no one has found a readily identifiable fault in the region to associate with liq features around Springfield in not a valid reason to discount that region as having the source eq: for example, despite having spent millions of dollars looking for the fault(s) that caused the strong eq (M=7+) in Charleston, SC, in the year 1886, no fault has been definitively identified as causing that eq or the other previous strong eqs that have occurred there in Holocene time. Thus, for all the reasons I have discussed above, I strongly suspect a local source for the Springfield area liq features.

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SSAR-422

PHONE CONVERSATION RECORD

PROJECT: 7935 Exelon ESP

DATE: 9 August 2003
9:00 am (PDT)

RECORDED BY: Kathryn L. Hanson

TO: Steve Obermeier (812) 649-4474

Subject: Paleoliquefaction studies in South Carolina and implications for evaluations of paleoliquefaction features in Central and Southern Illinois

1) Scott Olson, Obermeier, and Stark (SRL 2001) critique on aging factors--good review of previous work.

2)South Carolina studies--Group at Georgia Tech under Paul Mayne (blasting sites and measuring properties with time). Steve is not familiar with work on Savannah River Site done by Arango et al. He noted that effects of aging in So Carolina are nil because of ground water conditions (naturally acidicwaters cause chemical dissolution-- losing volume because of etching and dissolution). Need to pick site effectively to measure effects of aging.

Scott Olson and Obermeier reviewed work of Ke Hu(published in the SRL, v. 73, 2002)--strongly recommended that work was flawed and should not be published. Field methods for collecting data were problematic.

S. Obermier's general conclusion-Work in South Carolina and effects of aging irrelevant to work in Central Illinois.

Studies in Central United States.

1) Wabash Valley- Part 1 Techniques. OFR due out on web in a week. Part 2 Field Examples-- first draft of paper to be done in about a week focuses on Vincennes earthquake.

Picked out representative minimum number of blow counts (taking into account methods of failure)-data from Pond's thesis. Used Russell's new energy based method, and Seed method with various scaling factors and attentuation relationships. Vincennes (> M7 to 7.5)--unknowns-used NEHRP recommended factor.

considered aging effects influence- for low blow counts -extreme upper limits factor of 2 or 3. For moderately susceptible sites like those in southern Illinois--factor is significantly less --maybe a factor of 1 or 2 --not very significant.

evaluated whether features are from 1 or more earthquakes. Conclusion--most features in Vincennes and to west from one large event.