The Detroit Edison Company
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DTE Energy
Detroit Edison

10 CFR 52.79

August 6, 2010
NRC3-10-0035

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

References: 1) Fermi 3
Docket No. 52-033
2) Letter from Jerry Hale (USNRC) to Jack M. Davis (Detroit Edison), "Request for Additional Information Letter No. 36 Related to the SRP Sections 2.5.2 and 2.5.4 for the Fermi 3 Combined License Application," dated June 22, 2010

Subject: Detroit Edison Company Response to NRC Requests for Additional Information Letter No. 36

In Reference 2, the NRC requested additional information to support the review of certain portions of the Fermi 3 Combined License Application (COLA). This letter provides responses to ten of the eleven Requests for Additional Information (RAIs) identified in Reference 2. The technical work for RAI 02.05.02-15 has been completed but design verification was not completed to support submission. The response to RAI 02.05.02-15 will be provided no later than August 13, 2010.

The responses to the RAIs associated with Reference 2 are provided as Attachments 1 through 10 of this letter. Information contained in these responses will be incorporated into a future COLA submission as described in the attachments.

If you have any questions, or need additional information, please contact me at (313) 235-3341.

D095
NRO

I state under penalty of perjury that the foregoing is true and correct. Executed on the 6th day of August 2010.

Sincerely,

Peter W. Smith, Director
Nuclear Development – Licensing and Engineering
Detroit Edison Company

Attachments:
1) Response to RAI Letter No. 36 (Question No. 02.05.04-29)
2) Response to RAI Letter No. 36 (Question No. 02.05.04-30)
3) Response to RAI Letter No. 36 (Question No. 02.05.04-31)
4) Response to RAI Letter No. 36 (Question No. 02.05.04-32)
5) Response to RAI Letter No. 36 (Question No. 02.05.04-33)
6) Response to RAI Letter No. 36 (Question No. 02.05.04-34)
7) Response to RAI Letter No. 36 (Question No. 02.05.02-11)
8) Response to RAI Letter No. 36 (Question No. 02.05.02-12)
9) Response to RAI Letter No. 36 (Question No. 02.05.02-13)
10) Response to RAI Letter No. 36 (Question No. 02.05.02-14)

cc: Adrian Muniz, NRC Fermi 3 Project Manager
Jerry Hale, NRC Fermi 3 Project Manager
Bruce Olson, NRC Fermi 3 Environmental Project Manager
Fermi 2 Resident Inspector
NRC Region III Regional Administrator
NRC Region II Regional Administrator
Supervisor, Electric Operators, Michigan Public Service Commission
Michigan Department of Environmental Quality
Radiological Protection and Medical Waste Section

Attachment 1
NRC3-10-0035

Response to RAI Letter No. 36
(eRAI Tracking No. 4764)

RAI Question No. 02.05.04-29

NRC RAI 02.05.04-29

In the response to RAI 02.05.04-2, it was indicated that the observed orientation of discontinuities in the Bass Islands Group vary from horizontal to vertical, with near horizontal and near vertical joints dominating. However, the response also stated that "The orientation of the discontinuities tested is nearly horizontal, except the orientation of samples CB-C4 at 57.0 feet and RB-C3 at 46.9 feet, which were at inclined angles." You concluded in the response that both the discontinuities tested and the results are representative of the discontinuities observed within the Bass Islands Group. In accordance with 10CFR100.23, please justify why the test results from mostly horizontal discontinuities (one dominant orientation) can be representative of vertical discontinuities (another dominant orientation), and provide the basis for your conclusion.

Response

The bedrock core samples tested were obtained from borings drilled for the site investigation presented in the FSAR. Samples were obtained by rock coring, which provides a sample that is approximately vertical and cylindrical in shape, for which either horizontal or near horizontal fractures are the most appropriate for testing. Thus, the tests were performed on fractures with these orientations.

The Bass Islands Group dolomite is undeformed sedimentary bedrock at the site; therefore, depositional features, such as bedding planes, are oriented near to horizontal. Fractures along depositional features tend to have a more consistent orientation and less roughness, as they occur along preexisting planes of weakness formed as the bedrock was deposited. By comparison, vertical fractures break across depositional features, which results in a rougher fracture surface. A rougher fracture surface represents a higher strength, since the rock must ride up over or shear off the asperities for shearing to occur along the fracture surface.

Horizontal to near horizontal fractures that form along depositional features in sedimentary bedrock are more likely than vertical fractures to have either weaker material placed during original deposition of the bedrock, or deposition of weaker infilling material on a fracture following its formation. The presence of weaker original material on a fracture could result in reduced strength along the fracture. Vertical fractures typically would not contain low strength material from original deposition. Vertical fractures may have weaker infilling materials deposited on the fracture surface after fracture formation, but these infillings are more likely in horizontal fractures. Because of the higher potential for weak materials and the lower roughness of the horizontal fractures, the strength will likely be lower, and the use of the strength value from horizontal fractures should represent the lower bound.

The upper bound shear strength estimated using the Hoek-Brown criterion also supports that the testing results for the horizontal and near to horizontal fractures can be considered representative of the vertical discontinuities. The Hoek-Brown criterion estimates the strength and deformation

characteristics of the jointed bedrock mass assuming isotropic bedrock and bedrock mass behavior. Therefore, the Hoek-Brown criterion considers all the discontinuities in the bedrock mass, including horizontal, vertical and those in between. The strength parameters developed using the Hoek-Brown criterion are based on information from the bedrock at the site and empirical correlations as presented in the response to RAI 02.05.04-3 in Detroit Edison letter NRC3-09-0051 (ML100040537), dated December 23, 2009. The Hoek-Brown criterion evaluation for the Fermi 3 site estimated an upper bound friction angle of 53 degrees (FSAR Table 2.5.4-208). FSAR Section 2.5.4.2.1.2.1 addresses the properties of the Bass Islands Group, and states the following:

> "Twelve rock direct shear tests were performed along sample discontinuities to provide the residual friction angle along the discontinuities presented in Table 2.5.4-206. The residual friction angle along discontinuities ranges between 33 and 74 degrees, with a mean of 52 degrees."

FSAR Section 2.5.4.10.1 states the following regarding the Hoek-Brown criterion and the measured residual friction angle:

> "For the Bass Islands Group, the upper bound Hoek-Brown ϕ' of 53 degrees matches well with the mean residual friction angle of 52 degrees measured from rock direct shear tests on discontinuities (Table 2.5.4-206); therefore, ϕ' equal to 52 degrees is used for the Bass Islands formation."

In conclusion, the tests performed on the horizontal and near horizontal fractures in the bedrock core samples provide representative strength values along fractures in the Bass Islands Group, with the results representing the lower bound strength for the vertical fractures for the reasons discussed herein and summarized as follows:

- The roughness of the horizontal fractures will tend to be less than along vertical fractures.
- The potential presence of weak materials placed during original deposition of the sedimentary rock along horizontal fractures.

Additionally, the agreement between the friction angle measured on core samples from the Fermi 3 site and the friction angle estimated for the bedrock mass using the Hoek-Brown criterion indicates that, for the bedrock mass, the testing performed is representative of fractures at all orientations.

Proposed COLA Revision

None

Attachment 2
NRC3-10-0035

Response to RAI Letter No. 36
(eRAI Tracking No. 4764)

RAI Question No. 02.05.04-30

NRC RAI 02.05.04-30

In the response to RAI 02.05.04-3, you stated that "Guidelines are discussed in the FSAR Reference 2.5.4-201 for determination of σ'_{3max} for slopes and for shallow and deep tunnels. For Fermi 3, the equation of σ'_{3max} developed for slopes was selected." In accordance with 10 CFR 100.23, please explain why the use of "the equation of σ'_{3max} developed for slopes" would provide an adequate representation of the Hoek-Brown criteria for evaluation of foundation behavior beneath key structures.

Response

FSAR Reference 2.5.4-201 (which is corrected in the attached markup) provides two options for establishing σ'_{3max}, a slope condition or a tunnel condition. From FSAR Reference 2.5.4-201, σ'_{3max} represents "the upper limit of confining stress over which the relationship between the Hoek-Brown and the Mohr-Coulomb criterion is considered." FSAR Reference 2.5.4-201 states the following regarding evaluating σ'_{3max} for tunnels using Equation 22:

> "Equation 22 applies to all underground excavations, which are surrounded by a zone of failure that does not extend to surface."

$$\frac{\sigma'_{3\max}}{\sigma'_{cm}} = 0.47\left(\frac{\sigma'_{cm}}{\gamma H}\right)^{-0.94}$$

FSAR Reference 2.5.4-201, Equation 22

Based on this information from FSAR Reference 2.5.4-201, the σ'_{3max} for slopes was selected for the following reasons:

- Excavations in the bedrock will consist of open cut excavations extending approximately 28 feet below the bedrock surface (based on the top of the Bass Islands formation bedrock at an average elevation of 552 feet, NAVD 88 [FSAR Table 2.5.4-201], and the bottom of the Reactor/Fuel Building at elevation 523.7 feet, NAVD 88 [FSAR Table 2.5.4-224]). The open cut excavation removes all the bedrock above the base of the excavations; therefore, the equation for tunnels does not apply because there is not an underground condition with intact bedrock mass above the base of the excavation. The Category I structures are founded on exposed bedrock at the bottom of the excavation; therefore, the stress regime in the bedrock is similar to that of slopes exposed at the ground surface rather than a tunnel bored through the bedrock.
- Category I structures for Fermi 3 do not require tunnels or other underground structures within the bedrock.

Proposed COLA Revision

Proposed revision to FSAR Reference 2.5.4-201 is shown on the attached markup.

Markup of Detroit Edison COLA
(following 1 page)

The following markup represents how Detroit Edison intends to reflect this RAI response in the next submittal of the Fermi 3 COLA Revision 3. However, the same COLA content may be impacted by revisions to the ESBWR DCD, responses to other COLA RAIs, other COLA changes, plant design changes, editorial or typographical corrections, etc. As a result, the final COLA content that appears in a future submittal may be different than presented here.

"Practical Rock Engineering" notes,

11

2.5.4.13 References

2.5.4-201 Hoek, E., ~~"Practical Rock Engineering Notes,"~~ (2007 ed.), Chapter ~~10~~ Rock Mass Properties, Roc Science, http://www.rockscience.com/.

2.5.4-202 ASTM D2113-06, "Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation."

2.5.4-203 ASTM D6914-04, Standard Practice for Sonic Drilling for Site Characterization and the Installation of Subsurface Monitoring Devices."

2.5.4-204 U.S. Army Corps of Engineers, "Engineer Manual, Soil Sampling," EM 1110-2-1907, March 31, 1972.

2.5.4-205 ASTM D6151-97, "Standard Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling."

2.5.4-206 ASTM D1586-99, "Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils."

2.5.4-207 ASTM D3550-01, "Standard Practice for Thick Wall, Ring-Lined, Split Barrel, Drive Sampling of Soils."

2.5.4-208 ASTM D1587-00, "Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes."

2.5.4-209 ASTM D5079-02, "Standard Practices for Preserving and Transporting Rock Core Samples."

2.5.4-210 ASTM D4220-95, "Standard Practices for Preserving and Transporting Soil Samples."

2.5.4-211 ASTM D2488-06, "Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)."

2.5.4-212 ASTM D2487-06, "Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)."

2.5.4-213 ASTM D2216-05, "Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass."

2.5.4-214 ASTM D854-06, "Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer."

Attachment 3
NRC3-10-0035

Response to RAI Letter No. 36
(eRAI Tracking No. 4764)

RAI Question No. 02.05.04-31

NRC RAI 02.05.04-31

In the response to RAI 02.05.04-5, you provided chemical test results for groundwater sulfate and chloride concentrations, and indicated that all sample results for sulfate concentration from the monitoring wells fell into the categories of "moderate" and "severe" sulfate exposure for concrete based on ACI 349. In accordance with 10CFR100.23, provide groundwater pH values since concrete, being highly alkaline, is degraded by strong acids. Also, evaluate the potential aging effects (e.g., an increase in porosity and permeability, cracking, and/or the loss of material due to spalling or scaling) on the concrete fill due to aggressive groundwater conditions (e.g., pH < 5.5, chlorides > 500 ppm, or sulfates >1500 ppm). In addition, provide a commitment in the FSAR to ensure that ACI 349 requirements for concrete exposed to sulfate-containing solutions, including cement type, water-cement ratio, and minimum compressive strength, will be followed.

Response

For monitoring wells that had sufficient groundwater production to allow purging, the pH of the groundwater was monitored during purging until the pH values stabilized. The last pH values measured during purging are summarized in Table 1. The results in Table 1 show that the overburden groundwater and the Bass Islands Aquifer groundwater are not acidic, with measured pH greater than 7.0; therefore, the concrete will not be negatively impacted.

Table 1
Groundwater pH Measurements

Overburden Groundwater	
Monitoring Well Number	Final Purging pH Measurement
MW-387S	7.27
MW-390S	7.34
MW-391S	7.61
MW-395S	7.22
Bass Islands Aquifer Groundwater	
Monitoring Well Number	Final Purging pH Measurement
MW-381D	7.15
MW-383D	7.54
MW-384D	7.35
MW-386D	7.69
MW-387D	7.47
MW-391D	7.25
MW-393D	7.19
MW-395D	7.50

Regarding potential aging affects, the only constituent of concern for concrete is sulfate, as shown by the chemical test results for sulfate and chloride provided in the response to RAI 02.05.04-5 in Detroit Edison letter NRC3-09-0051 (ML100040537), dated December 23, 2009, and the pH results presented in Table 1. With the use of the correct cement, a well designed concrete mix, and good construction control, the concrete will not experience adverse aging effects due to the high sulfates present in the groundwater. The concrete design will be established during detailed design, but in general concrete that will be more resistant to sulfate will use a low water to cement ratio, adequate cement content, plasticizer or super plasticizer, silica fume (fly ash), and air entrainment. Control during construction to assure proper mixing of the concrete, followed by proper placement and curing will also result in concrete that is more resistant to degradation by sulfates.

Text is added to the FSAR to indicate that ACI 349 requirements for concrete exposed to sulfate-containing solutions will be implemented during detailed design.

Proposed COLA Revision

Proposed revision to FSAR Section 2.5.4.5.4.2 is shown on the attached markup.

Markup of Detroit Edison COLA
(following 2 pages)

The following markup represents how Detroit Edison intends to reflect this RAI response in the next submittal of the Fermi 3 COLA Revision 3. However, the same COLA content may be impacted by revisions to the ESBWR DCD, responses to other COLA RAIs, other COLA changes, plant design changes, editorial or typographical corrections, etc. As a result, the final COLA content that appears in a future submittal may be different than presented here.

Engineered granular backfill is compacted to achieve density that results in the backfill having a minimum ϕ' of 35 degrees. Based on correlations of strength characteristics for granular soils (Reference 2.5.4-242), the ϕ' of compacted granular soils can achieve 35 degree. Engineered granular backfill materials are placed in controlled lifts and compacted. Within confined areas or close to foundation walls, smaller compactors are used to prevent excessive lateral pressures against the walls from stress caused by heavy compactors.

Evaluation and discussion of liquefaction issues related to soil backfill materials is provided in Subsection 2.5.4.8. Lateral pressures applied against foundation walls are evaluated and discussed in Subsection 2.5.4.10.

A quality control sampling and testing program is developed to verify that concrete fill and granular backfill material properties conform to the specified design parameters. Sufficient laboratory compaction and grain size distribution tests are performed to account for variations in fill material. A test fill program may be included for the purposes of determining an optimum size of compaction equipment, number of passes, lift thickness, and other relevant data for achievement of the specified compaction.

Lean concrete used as fill under the FWSC will be proportioned, tested and the placement controlled in accordance with Regulatory Guide 1.142. The lean concrete fill will have a mean 28-day compressive strength of equal to, or greater than, 2000 psi with a mean shear wave velocity of equal to, or greater than, 3600 ft/s. Compressive strength of the lean concrete will be tested in accordance with Regulatory Guide 1.142. The compressive strength of the concrete will be used to calculate shear wave velocity to ensure that the shear wave velocity of 3,600 ft/s is met. The mix design developed for the lean concrete will control erosion and leaching due to contact with site groundwater and limit settlement to specified tolerances (Table 2.0-201), including creep and shrinkage.

Insert 1 here

The quality control program for fill concrete includes requirements for compressive strength testing. Verification will be performed to confirm that compressive strength testing results comply with mix design, minimum strengths, and placement requirements. The details of the quality control program will be addressed in a design specification prepared during the detailed design phase of the project.

Insert 1

Additionally, ACI 349 requirements for concrete exposed to sulfate-containing solutions will be implemented.

Attachment 4
NRC3-10-0035

Response to RAI Letter No. 36
(eRAI Tracking No. 4764)

RAI Question No. 02.05.04-32

NRC RAI 02.05.04-32

In the response to RAI 02.05.04-22, you stated the reason for using the upper-bound Hoek-Brown effective friction angle and cohesion for the Bass Islands Group, stating that the upper-bound Hoek-Brown effective friction angle of 53 degrees matches well with the mean residual friction angle of 52 degrees measured from the rock direct shear tests on discontinuities. In accordance with 10 CFR 100.23:

a. *Please discuss why a lower value of measured effective friction angle, such as mean effective friction angle minus one standard deviation, is not used to account for variability of the tests to ensure that your Hoek-Brown criterion is conservatively applied.*

b. *Please clarify the basis for concluding that using the upper bound Hoek-Brown cohesion is appropriate based on the upper bound Hoek-Brown effective friction angle matching well with the measured mean effective friction angle.*

Response

a. *Please discuss why a lower value of measured effective friction angle, such as mean effective friction angle minus one standard deviation, is not used to account for variability of the tests to ensure that your Hoek-Brown criterion is conservatively applied.*

The measured values from direct testing of bedrock discontinuities are considered to represent lower values of strength along fractures, as stated in the response to RAI 02.05.04-2 in Detroit Edison letter NRC3-09-0051 (ML100040537), dated December 23, 2009, as repeated below:

> "The discontinuities tested were disturbed during coring of bedrock, this type of disturbance was unavoidable. Prior to testing the discontinuities of the bedrock were fitted together to bring them back to the in-situ condition."
>
> "The test results are considered representative of the discontinuities tested; however, due to discontinuity disturbance, the measured friction angles may be lower than the actual friction angle of the discontinuities. Therefore, it is concluded that the discontinuities tested and the results are representative of the discontinuities observed within the Bass Islands Group."

The mean residual friction angle of the Bass Islands Group was calculated using the test results on the fractures, and is considered appropriate for establishing the design shear strength parameter for the Bass Islands Group for the following reasons:

- The residual friction angle represents the friction angle on a fracture after enough displacement has occurred to reach the steady state resistance along the fracture. In general, the residual friction angle is representative of the lower bound value for a fracture.
- Disturbance of the fractures during bedrock coring and preparation for testing resulted in reduced measured friction angles, as discussed in the response to RAI 02.05.04-2.

Therefore, further reducing the measured residual friction angles by one standard deviation is not considered necessary. Additionally, a factor of safety is used for analysis of foundation stability using the mean shear strength parameters. The application of the factor of safety in the geotechnical practice is intended to account for the variability of the material parameters and the subsurface conditions between investigation locations.

The Hoek-Brown criterion estimates the strength and deformation characteristics of the jointed bedrock mass assuming isotropic bedrock and bedrock mass behavior; therefore, the Hoek-Brown criterion considers all the bedrock characteristics. The characteristics of the Bass Islands Group encountered during the Fermi 3 subsurface investigation are discussed in FSAR Section 2.5.1.2.3.1.2 as follows:

> "The Bass Islands Group encountered during the Fermi 3 subsurface investigation is dominantly a light gray, light brownish gray, to dark gray micritic dolomite, with the following characteristics:
>
> - The dolomite can be massive, banded, or mottled.
> - It contains pitted and vuggy zones with some pits and vugs filled with crystalline anhydrite or calcite.
> - Oolites can be found scattered in small zones throughout the Bass Islands Group.
> - Stylolites (layers associated with pressure solution) can be found throughout the unit.
> - Some zones within the Bass Islands Group have stylolites that completely surround clasts of dolomite giving the zone a brecciated appearance, but these brecciated dolomites are caused by pressure solution and not fracturing."

The engineering properties of the Bass Islands Group encountered during the Fermi 3 subsurface investigation discussed in FSAR Section 2.5.4.2.1.2.1 are as follows:

> "The results of the field and laboratory tests together with their variability are summarized in Table 2.5.4-206. The average percent recovery throughout this rock unit was 94 percent with an average rock quality designation (RQD) of 54 percent. The RQD is a measure of rock integrity determined by taking the cumulative length of pieces of intact rock greater than 4 inches long for the length of a core sampler advance and dividing by the length of the core sampler advance, expressed as a percentage. Unconfined compressive strength, q_u, and E of the intact bedrock were determined by laboratory UC tests based on testing 20 intact rock samples. The q_u ranges from 46.0 to

> 153.7 MPa (960 to 3,210 ksf), with an average of 89.5 MPa (1,870 ksf). The E ranges from 15,900 to 78,600 MPa (331,200 to 1,641,600 ksf), with an average of 43,000 MPa (898,600 ksf). Twelve rock direct shear tests were performed along sample discontinuities to provide the residual friction angle along the discontinuities presented in Table 2.5.4-206. The residual friction angle along discontinuities ranges between 33 and 74 degrees, with a mean of 52 degrees."

The information regarding the Bass Islands Group dolomite bedrock indicates that the bedrock is hard, massive bedrock that has variable fracture frequency. The massive nature of the bedrock is an indication of uniformity of the Bass Islands Group bedrock, which was observed during the investigation. RAI 02.05.04-13 in Detroit Edison letter NRC3-10-0006 (ML100570305), dated February 11, 2010, provided optical televiewer logs, which showed the joints are relatively tight.

Regarding the use of Hoek-Brown Criterion, the response to RAI 02.05.04-3 in Detroit Edison letter NRC3-09-0051 (ML100040537), dated December 23, 2009, states the following:

> "The Hoek-Brown criterion is used to estimate the strength of jointed bedrock masses, which is consistent with the bedrock encountered during the Fermi 3 subsurface investigation. The Hoek-Brown criterion is based on an assessment of interlocking rock blocks and the condition of the surfaces between these blocks. The criterion provides an estimate of equivalent angles of friction and cohesive strengths..."

The response to RAI 02.05.04-3 continues on to provide a detailed discussion of the development of the bedrock mass strengths using the Hoek-Brown criterion. For a failure in the Bass Islands group dolomite bedrock to occur it would likely have to pass through intact bedrock and along irregularly shaped fractures. Therefore, when considering the Hoek-Brown criterion of estimating the equivalent strength of interlocking bedrock blocks, correlating with the direct shear test on the samples tested along fractures is considered to represent a lower value than is available if an actual failure of the bedrock mass occurred.

b. *Please clarify the basis for concluding that using the upper bound Hoek-Brown cohesion is appropriate based on the upper bound Hoek-Brown effective friction angle matching well with the measured mean effective friction angle.*

The bearing capacity analyses of Reactor/Fuel Building and Control Building are performed using FSAR Section 2.5.4, Equation 8, which as discussed in FSAR Section 2.5.4.10.1 does not require the use of the cohesion. Therefore, the FSAR text is marked up to remove reference to the cohesion values for the Bass Islands Group and Salina Unit F bedrock.

Proposed COLA Revision

Proposed revision to FSAR Section 2.5.4.10.1 is shown on the attached markup.

Markup of Detroit Edison COLA
(following 1 page)

The following markup represents how Detroit Edison intends to reflect this RAI response in the next submittal of the Fermi 3 COLA Revision 3. However, the same COLA content may be impacted by revisions to the ESBWR DCD, responses to other COLA RAIs, other COLA changes, plant design changes, editorial or typographical corrections, etc. As a result, the final COLA content that appears in a future submittal may be different than presented here.

used for the Bass Islands formation. ~~The corresponding upper bound c' is equal to 488 kPa (10.2 ksf) based on the Hoek-Brown criterion and will be used for the Bass Islands Group.~~ For the Salina Group Unit F the Hoek-Brown, the lower bound ϕ' of 28 degrees ~~and c' of 77 kPa (1.6 ksf)~~ was used.

The bearing capacity was evaluated at each unit using the following two independent methods:

1. Ultimate Bearing Capacity using the Terzaghi approach based on strength of bedrock mass (Reference 2.5.4-243).
2. Allowable bearing pressure based on q_u of bedrock, based on Uniform Building Code (Reference 2.5.4-244).

using

For the FWSC, the

~~The~~ ultimate bearing capacity ~~according to~~ the Terzaghi approach (Method 1) is computed using Equation ~~1~~ 3 shown below:

$$q_{ult} = cN_c + 0.5\gamma' BN_\gamma + \gamma' DN_q \qquad \text{[Eq. 3]}$$

where;

q_{ult} = ultimate bearing capacity
γ' = effective unit weight of bedrock mass
B = width of foundation
D = depth of foundation below ground surface
c = cohesion intercept for the bedrock mass

The terms N_c, N_γ and N_q are bearing capacity factors given by the following equations:

$$N_c = 2N_\phi^{1/2}\left(N_\phi + 1\right) \qquad \text{[Eq. 4]}$$

$$N_\gamma = N_\phi^{1/2}\left(N_\phi^2 - 1\right) \qquad \text{[Eq. 5]}$$

$$N_q = N_\phi^2 \qquad \text{[Eq. 6]}$$

$$N_\phi = \tan^2\left(45 + \phi/2\right) \qquad \text{[Eq. 7]}$$

where:

ϕ = angle of internal friction for the bedrock mass.

As the bedrock contains fractures, this approach was used for evaluating the bearing capacity of the R/FB and CB; therefore, the ultimate bearing capacity of the R/FB and CB is computed using Equation 8 as follows:

However, in cases where the shear failure is likely to develop along planes of discontinuity or through highly fractured bedrock masses, cohesion is not relied upon to provide resistance to failure (Reference 2.5.4-243). ~~In such cases the ultimate bearing capacity can be estimated from Equation 6 as shown below~~:

Attachment 5
NRC3-10-0035

Response to RAI Letter No. 36
(eRAI Tracking No. 4764)

RAI Question No. 02.05.04-33

NRC RAI 02.05.04-33

In the response to RAI 02.05.04-23, you stated that the Terzaghi approach takes into consideration the effect of the weaker zone below the Bass Islands Group based on general bearing capacity failure behavior, and the Uniform Building Code approach considers the allowable contact pressure on unweathered bedrock under a uniaxial loading condition to assure that the foundation bedrock has sufficient capacity against rupture. However, the Terzaghi approach is based on a particular class of potential failure mode involving homogeneous foundation material, and the Uniform Building Code approach is based on a more empirically based information mainly for buildings. In accordance with 10 CFR 100.23, please provide the basis for selecting these two approaches for the possible failure modes of foundation rock units at the site.

Response

The two methods used to evaluate the bearing capacity were selected because they are considered to be appropriate evaluation techniques that allow evaluation of two general potential bedrock failure modes, as follows:

- General shear failure is addressed using the Terzaghi approach.
- Rupture of intact bedrock due to the foundation loading is addressed by the Uniform Building Code approach.

Additionally, both techniques are applied in a manner that accounted for the variation of the bedrock properties with depth. The remainder of the response provides a detailed discussion of these points.

The Fermi 3 bearing capacity evaluations accounted for variation of bedrock properties with depth, by using weighted average properties of the subsurface layers within the zone of influence of the foundation. FSAR Section 2.5.4.10.1 provides the following to establish the zone of influence considered in the bearing capacity analysis:

> "For bearing capacity analysis, it is assumed that the influence zone of the foundation level is taken to be one times the width of the foundation. Therefore, the material properties important for the bearing capacity analysis are those of Bass Islands Group and Salina Group Unit F."

Beneath the Reactor/Fuel Building (R/FB), where Salina Group F bedrock is within the zone of influence of one foundation width, both the Bass Islands Group and Salina Group F are accounted for in both approaches by analyzing one equivalent homogeneous foundation material possessing the weighted average properties of both bedrock formations.

FSAR Reference 2.5.4-243, Figure 6-1 recommends use of the Terzaghi approach to estimate the bearing capacity on densely jointed bedrock masses where the joint spacing is much smaller than the foundation width. The projected failure mode is a general shear failure shaped similar to that typically considered in the Terzaghi approach. FSAR Reference 2.5.4-243, Figure 6-1 recommends the use of Reference 2.5.4-243 Equation 6-3 which is the same as Equation 8 in FSAR Section 2.5.4.10.1.

The rock quality designation (RQD) for the Fermi 3 bedrock core recovered indicates that the average fracture spacing in the bedrock is much smaller than the foundation width. Although some of the fractures observed in the core would have occurred due to drilling, the small fracture spacing indicates the Terzaghi approach is applicable. FSAR Table 2.5.4-206 presents the rock quality designation (RQD) in the Bass Islands Group. The RQD ranged from 0 to 100 percent, with an average of 54 percent. This indicates that on average, along approximately 46 percent of each core sampler advanced in the Bass Island Group, the fracture spacing is less than 4 inches. For Fermi 3 borings, FSAR Table 2.5.4-210 presents the RQD for Salina Unit F. The RQD ranged from 0 to 100 percent, with an average of 13 percent. This indicates that on average, along approximately 87 percent of each core sampler advanced in Salina Unit F, the joint spacing is less than 4 inches.

The Terzaghi approach represents a conservative method for estimating bearing capacity because it ignores the effect of cohesion and interlocking of bedrock blocks.

The Uniform Building Code approach was used to evaluate the potential against rupture of intact bedrock due to the foundation loading. This approach presented in FSAR Reference 2.5.4-244 (Chapter 22) for foundations bearing on bedrock uses an empirical relationship with the unconfined compressive strength to estimate the allowable pressure on bedrock. Table 22.1 of FSAR Reference 2.5.4-244 indicates that, in the 1964 Uniform Building Code, the allowable contact pressure for "Massive crystalline bedrock including… dolomite" is 20 percent of the unconfined compressive strength.

In conclusion, these two methods were selected because they are considered appropriate evaluation techniques that allow evaluation of two general potential bedrock failure modes. Additionally, the use of two distinct methods increases confidence in the estimated bearing capacities.

Proposed COLA Revision

None

Attachment 6
NRC3-10-0035

Response to RAI Letter No. 36
(eRAI Tracking No. 4764)

RAI Question No. 02.05.04-34

NRC RAI 02.05.04-34

In RAI response 02.05.04-20, you presented a liquefaction analysis for backfill to be placed adjacent to Category I structures. An assumed N_{60} of 30 blows/ft was assumed at the ground surface increasing linearly to 60 blows/ft down to a depth of 65 ft. In accordance with 10CFR100.23, please capture this liquefaction evaluation in the FSAR. Also provide details regarding how the assumed N_{60} values will be verified, and provide specific commitments to those verification methods in the FSAR. In addition, please provide the expected field backfill compaction (for example, 95% Modified Proctor, if applicable) and specify commitment in the FSAR.

Response

Discussion of the liquefaction evaluation presented in the response to RAI 02.05.04-20 in Detroit Edison letter NRC3-10-0006 (ML100570305), dated February 11, 2010, has been added to the attached markup of the FSAR text.

As part of updating the Fermi 3 FSAR to incorporate DCD Revision 7, GEH is performing a soil structure interaction analysis (SSI) for the Reactor/Fuel Building and the Control Building. Following the completion of this SSI, any site-specific requirements for the backfill adjacent to the Reactor/Fuel Building and the Control Building will be assessed and the FSAR updated accordingly.

During the detailed design which follows the COLA, the laboratory testing specified in FSAR Section 2.5.4.5.1 will be implemented to establish the required density necessary to meet the design requirements of the engineered granular backfill adjacent to Category 1 structures. To further confirm that the density selected based on the laboratory testing results satisfy the design requirements, a program will be implemented for in-place testing of the engineered granular backfill. Additionally, the detailed design will specify that testing necessary to confirm the engineered granular backfill placed during construction satisfies the design requirements.

As an example, testing to verify the N_{60} could be performed by using standard penetration test N values to convert the penetration of a standard split barrel sampler driven by a calibrated 140 pound auto hammer with a 30 inch drop to N_{60}. Test pads could also be used to coordinate in-place testing with laboratory testing and the design requirements.

Proposed COLA Revision

Proposed revisions to FSAR Sections 2.5.4.5.4.2 and 2.5.4.8 are shown on the attached markup.

Markup of Detroit Edison COLA
(following 5 pages)

The following markup represents how Detroit Edison intends to reflect this RAI response in the next submittal of the Fermi 3 COLA Revision 3. However, the same COLA content may be impacted by revisions to the ESBWR DCD, responses to other COLA RAIs, other COLA changes, plant design changes, editorial or typographical corrections, etc. As a result, the final COLA content that appears in a future submittal may be different than presented here.

geologic mapping program includes photographic documentation of the exposed surface and documentation for significant geologic features.

The details of the quality control and quality assurance programs for foundation bedrock are addressed in the design specifications prepared during the detailed design phase of the project.

2.5.4.5.4.2 Backfill Materials and Quality Control

Backfill for the Fermi 3 may consist of concrete fill or a sound, well graded granular backfill. Engineered granular backfill to be used will have a ϕ' equal to or greater than 35 degrees when properly placed and compacted. In addition, the engineered backfill is required to meet the following criteria:

i. Product of peak ground acceleration α (in g), Poisson's ratio ν and density γ
   $\alpha(0.95v + 0.65)\gamma$: 1220 kg/m^3 (76 lbf/ft^3) maximum

ii. Product of at-rest pressure coefficient κ_0 and density:
   $\kappa_0\gamma$:750 kg/m^3 (47lbf/ft^3) minimum

iii. At-rest pressure coefficient:
   κ_0: 0.36 minimum

iv. Soil density
   γ: 1900 kg/m^3 (119 lbf/ft^3) minimum

The anticipated extent of lean concrete fill and granular backfill is shown on Figure 2.5.4-202, Figure 2.5.4-203, and Figure 2.5.4-204.

Concrete fill mix designs are addressed in a design specification prepared during the detailed design phase of the project. Field observation is performed to verify that approved mixes are used and test specimens are obtained that verify that specified design parameters are reached. The foundation bedrock and concrete fill provide adequately high factors of safety against bearing capacity failure under both static and seismic structural loading. Quality Control testing requirements for bedrock include visual inspection and geologic mapping.

Engineered granular backfill sources are identified and tested for engineering properties, in accordance with recommendations from Subsection 2.5.4.5.1 and other testing as required by design specifications.

Insert 1 here

Insert 1

During the detailed design which follows the COLA, the laboratory testing specified in FSAR Section 2.5.4.5.1 will be implemented to establish the required density necessary to meet the design requirements of the engineered granular backfill adjacent to Category 1 structures. To further confirm that the density selected based on the laboratory testing results satisfy the design requirements, a program will be implemented for in-place testing of the engineered granular backfill. Additionally, the detailed design will specify that testing necessary to confirm the engineered granular backfill placed during construction satisfies the design requirements.

profiles and modulus reduction and damping curves are described in Subsection 2.5.2.6.

2.5.4.8 Liquefaction Potential

This section conforms to guidelines in RG 1.198.

All Seismic Category I structures are supported within the Bass Islands dolomite or on lean concrete fill extending to the top of bedrock. Neither the bedrock nor lean concrete fill are susceptible to liquefaction. ~~Engineered granular backfill is used to fill adjacent to all Seismic Category I structures and is not susceptible to liquefaction.~~

Insert 2 here

The existing fill, lacustrine deposits and glacial till are removed under and adjacent to all Seismic Category I structures; therefore, liquefaction analysis is not necessary.

for these soils

2.5.4.9 Earthquake Design Basis

The V_s values of soils and bedrock at the site were determined through the field exploration program using geophysical testing as described in Subsection 2.5.4.2 and Subsection 2.5.4.4. Subsection 2.5.4.7 presents the dynamic response of soil and bedrock under dynamic loading conditions. The top of generic bedrock is approximately 129.5 m (425 ft) below the existing ground surface where the V_s of bedrock (Salina Group Unit B) is greater than 2804 m/s (9200 fps). A site response analysis was performed using the above information to develop the GMRS for the site as described in Subsection 2.5.2.6.

2.5.4.10 Static Stability

In this section, the analyses performed to evaluate the stability of the safety-related structures under static loading conditions are presented. Specifically, this subsection addresses three Seismic Category I structures – R/FB, CB and FWSC. This section includes analyses of foundation bearing capacity and settlement, excavation rebound, lateral earth pressures, and hydrostatic pressures.

DCD Figure 3G.1-6 and DCD Tables 2.0-1, 3.8-8, and 3.8-13 provide information on plan dimensions, embedment depths, and loads. The R/FB mat foundation has plan dimensions of 49.0 by 70.0 m (161 by 230 ft), and bears 20.0 m (65.6 ft) below the Referenced DCD reference grade (4500 mm). As discussed in Subsection 2.5.4.5, the Referenced DCD reference grade is equivalent to a site elevation of 179.6 m (589.3 ft) NAVD 88. The base of the R/FB foundation base is thus at elevation

Insert 2

For engineered granular backfill adjacent to Seismic Category I structures, liquefaction considerations only apply below the groundwater table. Section 2.4.12.5 provides the maximum historical high groundwater level of 175.6 m (576.11 ft) NAVD 88, which is approximately 4 m (13.2 ft) below the plant grade of 179.6 m (589.3 ft) NAVD 88; therefore, liquefaction is not a consideration in the upper 4 m (13.2 ft) of the engineered granular backfill. Section 2.5.4.5.4.2 discusses placement of granular backfill adjacent to Seismic Category I structures in controlled lifts with compaction. This will result in a dense to very dense consistency engineered backfill surrounding the embedded walls of Seismic Category I structures; therefore, there is also no potential for liquefaction if the engineered granular backfill below the groundwater. For confirmation, a liquefaction analysis based on the standard penetration test (SPT) is provided to demonstrate that the engineered granular backfill is not susceptible to liquefaction.

Reference 2.5.4-252, Table 12.1 shows that for dense granular soils N_{60} is between 30 and 50 blows/foot, and for very dense granular soils N_{60} is greater than 50 blows/foot. N_{60} is the numbers of blow to drive a standard split barrel sampler the last 12 inches of the SPT using a 140 pound hammer falling 30 inches, where the hammer has a 60 percent energy efficiency. To evaluate liquefaction potential of soil, $(N_1)_{60}$ is needed, where $(N_1)_{60}$ is the N_{60} value normalized to an overburden pressure of approximately 100 kPa (1 ton per square foot) (Reference 2.5.4-253). Reference 2.5.4-253 shows that for historical data, no liquefaction was observed when $(N_1)_{60}$ is greater than 30 blows/ft.

For the engineered granular backfill, the N_{60}-value is estimated to be 30 blows/foot at the ground surface, and is increased linearly to 60 blows/foot at a depth of 65 feet. Using this distribution for N_{60} and a bounding groundwater level at 2 feet below plant grade, at all engineered granular backfill depths for the full depth of the deepest Seismic Category I structure, $(N_1)_{60}$ is greater than 30 blows/foot. With the backfill placement approach and resultant $(N_1)_{60}$ greater than 30, it is concluded that the engineered granular backfill, adjacent to all Seismic Category I structures, is not susceptible to liquefaction. If $(N_1)_{60}$ of the in-place engineered granular backfill is less than 30, a more refined liquefaction analysis will be performed to confirm there is adequate resistance against liquefaction.

New FSAR References

2.5.4-252
Youd, T.L., et al., Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of the Geotechnical and Geoenvironmental Engineering, Vol. 127, No.10, pp. 817-833, ASCE, 2001.

Attachment 7
NRC3-10-0035

Response to RAI Letter No. 36
(eRAI Tracking No. 4766)

RAI Question No. 02.05.02-11

NRC RAI 02.05.02-11

You indicated in the response to RAI 02.05.02-2 that you followed SSHAC Level 2 procedures for updating EPRI/SOG seismic sources with respect to the Fermi 3 site. In accordance with 10 CFR 100.23, please provide a detailed description on the process that you followed including a listing of the Technical Integrators (TIs) and the Review Panel members. In addition, please fully explain how you incorporated differing expert opinions. Finally, please update the FSAR to incorporate this information.

Response

As described in the response to RAI 02.05.02-2 in Detroit Edison letter NRC3-10-0006 (ML100570305), dated February 11, 2010, the Fermi 3 PSHA was conducted as a Senior Seismic Hazard Analysis Committee (SSHAC) Level 2 update to a SSHAC Level 4 study. In this context the seismic source characterization relied primarily on the assessments made by the six EPRI-SOG expert teams. These assessments were updated when new information would change an interpretation made by the EPRI-SOG expert teams.

SSHAC (FSAR Reference 2.5.2-260) describes four levels of study (Levels 1 through 4), in increasing order of participation by technical experts in the development of the PSHA inputs and related scope and effort. The choice of the level of a PSHA is driven by two factors: (1) the complexity and degree of uncertainty and contention associated with a particular issue, and (2) scheduling constraints and the amount of resources available for the study (FSAR Reference 2.5.2-260). For Level 1, 2, and 3 studies, SSHAC directs the Technical Integrator (TI) to communicate with regional and topical experts to understand the technical positions taken by various proponents of particular hypotheses. The TI team contacts members of the informed technical community in order to understand the alternatives and the technical bases behind the hypotheses and positions. By this means, the knowledge and uncertainties of the larger technical community are captured.

The SSHAC Level 2 process utilizes an individual, team, or company to act as the TI. For the Fermi 3 COLA, Geomatrix Consultants, Inc. (currently AMEC Geomatrix, Inc.) acted as the TI with additional input and review provided by the Fermi 3 Technical Advisory Board (TAB), which functioned in a participatory peer review role. A list of the key individuals, who made up the TI team and TAB are provided in Table 1, below.

The TI process used in the Fermi 3 COLA PSHA update followed guidance provided by SSHAC (FSAR Reference 2.5.2-260) and consisted of the following steps:

Step 1 Identify and Select Peer Reviewers:

A Technical Advisory Board was selected at the initiation of the project. The TAB provided peer review to the project. Their input and review was elicited during the data collection and review stages and following initial sensitivity and final hazard analyses.

Step 2 Identify available information and design analyses and information retrieval methods:

The TI team assembled all relevant technical data bases and information important to the hazard analysis, geologic, geophysical, seismological information and data sets for the Fermi 3 site region, site vicinity, site area, and site (described in FSAR Sections 2.5.1, 2.5.2, and 2.5.3), and pre-existing and new site-specific data for the Fermi 3 site (described in detail in FSAR Section 2.5.4).

Step 3 Perform analyses, accumulate information relevant to issues and develop representative community distribution:

For the Fermi 3 PSHA, this guidance was followed by reviewing published literature, available unpublished reports, documents pertinent to seismic source characterization, and by contacting researchers familiar with the seismic sources that could potentially affect the Fermi 3 site. The goal of this effort was to capture the current state-of-knowledge of the expert community, including its uncertainty. Table 2 summarizes communications with various researchers contacted during the Fermi 3 COLA study. Communications with the researchers included meetings and joint field reconnaissance, in addition to phone and email exchanges.

Through this process, the TI team was able to identify new information and key data sets and observations (published and unpublished) that suggested updates to the EPRI-SOG expert team assessments would be required. For example, the EPRI-SOG expert teams used information on the size of the largest earthquake known to have occurred in a source zone as one of the factors that influenced their assessment of maximum magnitude. Since the assessments of the EPRI-SOG teams were completed, the results of paleoearthquake research have produced new information on the size of earthquakes that have occurred in the recent geologic past. In a number of cases, this research has identified larger events than previously observed in specific source zones. The maximum magnitude distributions in specific source zones were updated to reflect this recent information.

A consistent approach was used to update all source zones. In each case, the maximum magnitude probability distribution that the EPRI-SOG teams had assigned to their sources was updated using the methodology and approaches outlined by the EPRI-SOG teams. An example of how new information on prehistoric earthquakes in southern Illinois and Indiana inferred from paleoseismic studies was incorporated into the PSHA is provided in the response to RAI 02.05.02-2.

There is no regional paleoseismic database that provides uniform coverage of the entire central and eastern United States (CEUS). The most detailed paleoliquefaction investigations have been conducted in regions that have experienced a large-magnitude historical earthquake (e.g., New Madrid seismic zone, Missouri; and Charleston, South Carolina) or in areas of elevated seismicity (e.g., the Wabash Valley and adjoining regions of southern Illinois, southern Indiana, and southeastern Missouri; North Anna, and northeastern Ohio). Where results have documented the occurrence of moderate- to large-magnitude earthquakes, this information has been applied to update the source(s) where those events occurred. In some instances, paleoliquefaction studies have not yielded conclusive evidence to demonstrate the occurrence of moderate- to large-magnitude prehistoric earthquakes or to preclude such events over a period of time that would be significant to seismic source characterization. For example, paleoliquefaction surveys were conducted along rivers in the Anna, Ohio, and northeastern Ohio regions. The results of these studies, which are outlined in the responses to RAI 02.05.01-10 and RAI 02.05.01-14 in Detroit Edison letter NRC3-10-0006 (ML100570305), dated February 11, 2010, did not identify paleoliquefaction features. The findings in the North Anna region suggested that earthquakes greater than **M** ~ 7 likely had not occurred in the past several thousand years, but the scarcity of outcrops was not sufficient to preclude moderate-magnitude events (FSAR Reference 2.5.1-350). The negative findings in these regions cannot be reliably incorporated into the seismic source characterization as there are a number of factors that could lead to an incomplete record, including the lack of susceptible deposits or conditions favorable for the formation and preservation of paleoliquefaction, scarcity of adequate outcrops to identify paleoliquefaction, and the young age of many of the sediments exposed along the surveyed streams.

As outlined in the response to RAI 02.05.02-2, the principal area of controversy that affects the hazard at the Fermi 3 site is with regard to the size of New Madrid source zone earthquakes. The 1811-1812 earthquakes represent the largest historical events in the CEUS and are among the largest events in the worldwide database for stable continental region earthquakes. In the CEUS, estimates of magnitude for prehistoric earthquakes based on paleoliquefaction results are tied in part to the magnitude bound curve developed most recently by Olson et al. (2005), which uses the 1811-1812 earthquakes to define the curve.

Publications over the past decade, in addition to recent communications with researchers, indicate that there still remains uncertainty and differing views within the research community regarding the size and location of the 1811–1812 earthquakes. The maximum magnitude distribution used to characterize the central fault system of NMSZ for this study was initially developed by members of the Fermi 3 TI team as part of a SSHAC Level 2 assessment for the Exelon Generation Company, LLC (EGC) Early Site Permit (ESP) application for the Clinton, Illinois site (FSAR Reference 2.5.2-243). In order to incorporate the then current perspectives of knowledgeable researchers, three researchers were contacted who had published very different estimates of the size of the 1811–1812

earthquakes based on evaluation of intensity data and other geologic information. The discussions with the three researchers addressed their current preferred values of the size of the 1811–1812 earthquakes and reasons for the discrepancies among the various researchers. The EGC-ESP TI team used their judgment regarding parameters and weights to be used in the seismic source model (response to RAI 02.05.02-2). In the absence of additional new information post-dating the EGC-ESP assessment, the Fermi 3 TI team adopted the parameters and weights used in the EGC-ESP study (FSAR Reference 2.5.2-243) and subsequently in the Bellefonte Units 3 and 4 COLA FSAR (FSAR Reference 2.5.2-244) (FSAR Appendix 2.5BB).

Step 4 Perform data diagnostics and respond to peer reviews:

New information identified through this process was used to evaluate and update the EPRI-SOG expert teams' source characterizations. Per SSHAC guidelines, a variety of sensitivity analyses were carried out and shared with the TAB (peer reviewers) to understand the most significant issues, sources of uncertainty, and data sets used to address the issues. Based on the results of these analyses, the TI team updated the EPRI-SOG expert team assessments to incorporate differing expert opinion and new information.

Step 5 Document process and results:

A discussion of the updates to the EPRI-SOG expert teams assessments and results of the sensitivity and final hazard analyses are documented in FSAR Section 2.5.2.

Reference

Olson, S.M., R. A. Green, and S. F. Obermeier, "Revised magnitude bound relation for the Wabash Valley seismic zone of the central United States," *Seismological Research Letters*, Vol. 76, No. 6, pp. 756-771, 2005.

Table 1
Members of the TI Team and Participatory Peer Review Board

TI Team (AMEC Geomatrix Consultants)[1]	**Area of Expertise**
Dr. Robert Youngs	Hazard Analyst Seismic Source Characterization
Ms. Kathryn Hanson	Seismic Source Characterization
Ms. Laura Glaser	Seismic Source Characterization
Ms. Valentina Montaldo Feraro	Earthquake Catalog
Technical Advisory Board (TAB) Participatory Peer Review	**Area of Expertise**
Dr. Achyut Setler	Geotechnical Engineering
Dr. Carl Stepp	Seismic Source Characterization SSHAC Process
Dr. Gonzalo Castro	Geotechnical Engineering
Dr. I. M. Idriss	Ground Motion and Geotechnical Engineering

[1] Currently AMEC Geomatrix, Inc.

Table 2
Resource Experts Contacted

Contact	Affiliation	Expertise	SSC Issues
Mark Baranoski	Ohio Division of Geological Survey	Basement structures	Evidence for reactivation of basement faults
Glenn Larsen	Ohio Division of Geological Survey	Geologic Structures	Identification and characterization of regional faults
Rick Pavey	Ohio Division of Geological Survey	Quaternary Geology	Information on evidence for Quaternary faulting, pop-up structures
E. Mac Swinford	Ohio Division of Geological Survey	Division Assistant Cheif	Referral to appropriate staff
Erick Venteris	Ohio Division of Geological Survey	Paleoliquefaction	Assessment of existing paleoliquefaction study results for identifying prehistoric earthquakes
Donovan Powers	Ohio Division of Geological Survey	GIS datasets	Obtaining digital datasets
John Esch	Michigan DEQ	Geologic Structures	Identification and characterization of faults in Michigan
Raymond Vurginovich	Michigan DEQ	Geologic Mapping and Well Database	Data compilation for source characterization
Ron Elowski	Michigan DEQ	Geologic Mapping and Well Database	Data compilation for source characterization
Steve Wilson	Michigan DEQ	Geologic Mapping	Data compilation for source characterization
Larry Organek	Michigan DEQ	Well database	Primary data for evaluating deformation history for subsurface faults in site vicinity

Roger Nelson	Michigan DEQ	Well database	Primary data for evaluating deformation history for subsurface faults in site vicinity
John Rupp	Indiana Geological Survey	Geologic Structures-Paleoliquefaction Studies in Indiana	Identification and characterization of regional faults
Mary Parke	Indiana Geological Survey	Seismic Hazard Studies in Indiana	ion of regional faults
Terry Carter	Ontario Geological Survey	Structures defined from Oil and Gas	Identification and characterization of regional fault and subsurface faults in site vicinty
Desmond Rainford	Ontario Geological Survey	Datasets in Ontario	Data compilation for source characterization
James Boyd	Ontario Geological Survey	GIS datasets	Data compilation for source characterization
Thomas Hoane	Michigan Basin Geological Society	MBGS Publications	Data compilation for source characterization
Viki Bankey	USGS	Regional Magnetic Data	Data compilation for source characterization
Kaz Fujita	Michigan State University	Seismicity in Michigan	Data compilation for source characterization
Stephen Halchuck	Geological Survey of Canada	Seismicity	Data compilation for source characterization
John Adams	Geological Survey of Canada	Seismicity	Data compilation for source characterization
Steve Obermeier	USGS	Paleoliquefaction	Primary researcher for paleoliquefaction studies in Anna, Ohio and NE Ohio regions

Margaret Hopper	USGS	Seismicity	Data compilation for source characterization
Rich Harrison	USGS	Seismic Hazards	Data compilation – paleoliquefaction data sets
Bill Harrison	Western Michigan University	Geologic Structures	Data compilation for source characterization
Abbreviations: SSC = Seismic Source Characterization DEQ = Department of Environmental Quality MBGS = Michigan Basin Geological Society USGS = U.S. Geological Survey			

Proposed COLA Revision

Proposed revisions to FSAR Sections 2.5.2.4 and 2.5.2.4.1 are shown on the attached markup.

Markup of Detroit Edison COLA
(following 13 pages)

The following markup represents how Detroit Edison intends to reflect this RAI response in the next submittal of the Fermi 3 COLA Revision 3. However, the same COLA content may be impacted by revisions to the ESBWR DCD, responses to other COLA RAIs, other COLA changes, plant design changes, editorial or typographical corrections, etc. As a result, the final COLA content that appears in a future submittal may be different than presented here.

the EPRI-SOG characterization are required based on seismicity patterns.

- Relocated EPRI events in the Anna, Ohio, seismic zone (Reference 2.5.2-220) show a better correlation with Fort Wayne rift structures (subsurface Anna-Champaign, Logan, and Auglaize faults) than was recognized by the ESTs.
- Seismicity in the Northeastern Ohio seismic zone, including several post-EPRI events, has been associated with the Akron Magnetic Boundary (Reference 2.5.2-215).
- Added historical events in the Wabash Valley seismic zone by Metzger et al. (Reference 2.5.2-212) have increased seismicity rates in source zones in southern Illinois and Indiana.
- The updated catalog does not show any earthquakes within the site region that can be associated with a known geologic structure, with the exception of the Anna and Northeastern Ohio seismic zones discussed in the bulleted items listed above.
- The closest principal sources of seismic activity are in the vicinity of Anna, Ohio, and Cleveland, Ohio. These areas lie at a distance of greater than 150 km (90 mi.) from the Fermi 3 site. Concentrations of seismicity in these areas were recognized and considered by the EPRI-SOG teams, as discussed in Subsection 2.5.2.2.1.

2.5.2.4 Probabilistic Seismic Hazard Analysis and Controlling Earthquake

This subsection describes the PSHA conducted for the Fermi 3 site. Following the procedures outlined in Appendix E, Section E.3, of Regulatory Guide 1.165, Subsection 2.5.2.4.1 and Subsection 2.5.2.4.2 discuss new information on seismic source characterization and ground motion characterization, respectively, that is potentially significant relative to the EPRI-SOG seismic hazard model (Reference 2.5.2-201). Insert 1 Subsection 2.5.2.4.3 presents the results of PSHA sensitivity analyses used to test the effect of the new information on the seismic hazard. Using these results, an updated PSHA was performed, as described in Subsection 2.5.2.4.4. The results of that analysis are used for the development of uniform hazard response spectra and identification of the controlling earthquakes (Subsection 2.5.2.4.4.2).

Insert 1

The Fermi 3 PSHA was conducted as a Senior Seismic Hazard Analysis Committee (SSHAC) Level 2 update to the SSHAC Level 4 EPRI-SOG study (References 2.5.2-260). In this context the seismic source characterization relied primarily on the assessments made by the six EPRI-SOG expert teams. These assessments were updated when new information would change an interpretation made by the EPRI-SOG expert teams. A discussion of the SSHAC Level 2 update process is provided in Subsection 2.5.2.4.1.

2.5.2.4.1 New Information Relative to Seismic Sources

This section describes potential updates to the EPRI-SOG seismic source model. Seismic source characterization data and information that could affect the predicted level of seismic hazard include the following:

- Identification of possible additional seismic sources in the site region.
- Changes in the characterization of the rate of earthquake occurrence for one or more seismic sources.
- Changes in the characterization of the maximum magnitude for other EST seismic sources.

Insert 2

Insert 3

Based on the review of new geological, geophysical, and seismological information that is summarized in Subsection 2.5.1, the review of seismic source characterization models developed for post-EPRI-SOG seismic hazard analyses (Subsection 2.5.2.2.2), and a comparison of the updated earthquake catalog to the EPRI-SOG evaluation (Subsection 2.5.2.3), the EPRI-SOG source models have been modified for the Fermi 3 COLA as follows:

- Fault sources are added for repeated large-magnitude earthquakes in the New Madrid Seismic Zone.
- The maximum magnitude distribution for the Wabash Valley – Southern Illinois source zone(s) is revised.
- The maximum magnitude distribution for selected EPRI-SOG team sources are updated based on updated earthquake catalog events.

Insert 4

Insert 5

Insert 6

2.5.2.4.1.1 Updated Characterization of Large-Magnitude New Madrid Seismic Zone Earthquakes

The NMSZ extends from southeastern Missouri to southwestern Tennessee and is located more than 700 km (435 mi.) southwest of the Fermi 3 site (Figure 2.5.1-207). The NMSZ produced a series of large-magnitude earthquakes between December 1811 and February 1812 (Reference 2.5.2-245 through Reference 2.5.2-248). A detailed discussion of recent information about the location, size, and frequency of repeated large-magnitude events that have occurred in the NMSZ is provided in Subsection 2.5.1.1.4.4.1.

The updated characterization of fault sources that are judged to be the sources for the 1811-1812 New Madrid earthquake sequence and similar paleoearthquake sequences in the NMSZ follows the characterization initially developed in the EGC ESP application (Reference 2.5.2-243) and

Insert 2

The potential updates to the EPRI-SOG seismic source model were identified and developed through a SSHAC Level 2 process. SSHAC (Reference 2.5.2-260) describes four levels of study (Levels 1 through 4), in increasing order of participation by technical experts in the development of the PSHA inputs and related scope and effort. The choice of the level of a PSHA is driven by two factors: (1) the complexity and degree of uncertainty and contention associated with a particular issue, and (2) scheduling constraints and the amount of resources available for the study (Reference 2.5.2-260). For Level 1, 2, and 3 studies, SSHAC directs the Technical Integrator (TI) to communicate with regional and topical experts to understand the technical positions taken by various proponents of particular hypotheses. The TI team contacts members of the informed technical community in order to understand the alternatives and the technical bases behind the hypotheses and positions. By this means, the knowledge and uncertainties of the larger technical community are captured.

The SSHAC Level 2 process utilizes an individual, team, or company to act as the TI. For the Fermi 3 COLA, Geomatrix Consultants, Inc. (currently AMEC Geomatrix, Inc.) acted as the TI with additional input and review provided by the Fermi 3 Technical Advisory Board (TAB), which functioned in a participatory peer review role.

The TI process used in the Fermi 3 PSHA update followed guidance provided by SSHAC (Reference 2.5.2-260) and consisted of the following steps:

Step 1 Identify and Select Peer Reviewers:

A Technical Advisory Board (TAB) was selected at the initiation of the project. The TAB provided peer review to the project at numerous stages. Their input and review was elicited during the data collection and review stages and following initial sensitivity and final hazard analyses.

Step 2 Identify available information and design analyses and information retrieval methods:

The TI team assembled all relevant technical data bases and information important to the hazard analysis including geologic, geophysical, seismological information and data sets for the Fermi 3 site region, site vicinity, site area, and site (described in Subsections 2.5.1, 2.5.2, and 2.5.3), and pre-existing and new site-specific geotechnical data for the Fermi 3 site (described in detail in Subsection 2.5.4).

Step 3 Perform analyses, accumulate information relevant to issues and develop representative community distribution:

Insert 2 Continued

For the Fermi 3 PSHA, this guidance was followed by reviewing published literature, available unpublished reports, documents pertinent to seismic source characterization, and by contacting researchers familiar with the seismic sources that could potentially affect the Fermi 3 site. The goal of this effort was to capture the current state-of-knowledge of the expert community, including its uncertainty. Table 2.5.2-227 summarizes communications with various researchers contacted during the Fermi 3 COLA study. Communications with the researchers included meetings and joint field reconnaissance, in addition to phone and email exchanges.

Through this process, the TI team was able to identify new information and key data sets and observations (published and unpublished) that suggested updates to the EPRI-SOG expert team assessments would be required. For example, the EPRI-SOG expert teams used information on the size of the largest earthquake known to have occurred in a source zone as one of the factors that influenced their assessment of maximum magnitude. Since the assessments of the EPRI-SOG teams were completed, the results of paleoearthquake research have produced new information on the size of earthquakes that have occurred in the recent geologic past. In a number of cases, this research has identified larger events than previously observed in specific source zones. The maximum magnitude distributions in specific source zones were updated to reflect this recent information.

Step 4 Perform data diagnostics and respond to peer reviews:

New information identified through this process was used by the TI team to evaluate and update the EPRI-SOG expert teams' source characterizations. Per SSHAC (Reference 2.5.2-260) guidelines, a variety of sensitivity analyses were carried out and shared with the TAB (peer reviewers) to understand the most significant issues, sources of uncertainty, and data sets used to address the issues (see discussion in Subsection 2.5.2.4.3). Based on the results of these analyses, the TI team updated the EPRI-SOG expert team assessments to incorporate differing expert opinion and new information.

Step 5 Document process and results:

A discussion of the updates to the EPRI-SOG expert teams assessments and results of the sensitivity and final hazard analyses are documented in Section 2.5.2.

Insert 3

the SSHAC Level 2 process, which included

Insert 4

Three fault sources are included in the updated characterization of the central fault system of the NMSZ: (1) the New Madrid South (NS) fault, (2) the New Madrid North (NN), and (3) the Reelfoot fault (RF). The most significant updates of source parameters for the NMSZ since the 1986 EPRI-SOG study that stem from the results of paleoliquefaction studies are the reduction in the mean recurrence interval to approximately 500 years, and consideration of clustered event sequences. The principal area of controversy that affects the hazard at the Fermi 3 site is with regard to the size of New Madrid source zone earthquakes. The 1811-1812 earthquakes represent the largest historical events in the CEUS and are among the largest events in the worldwide database for stable continental region earthquakes. In the CEUS estimates of magnitude for prehistoric earthquakes based on paleoliquefaction results are tied in part to the magnitude bound curve developed most recently by Olson et al. (Reference 2.5.2-307), which uses the 1811-1812 earthquakes to define the curve.

Publications over the past decade, in addition to recent communications with researchers, indicates that there still remains uncertainty and differing views within the research community regarding the size and location of the 1811– 1812 earthquakes. The maximum magnitude distribution used to characterize the central fault system of NMSZ for this study was initially developed as part of a SSHAC Level 2 assessment for the Exelon Generation Company, LLC (EGC) Early Site Permit (ESP) application for the Clinton, Illinois site (Reference 2.5.2-243). In order to incorporate the then current perspectives of knowledgeable researchers who had published very different estimates of the size of the 1811– 1812 earthquakes based on evaluation of intensity data and other geologic information, three individuals were contacted to discuss their current preferred values and reasons for the discrepancies among the various researchers (Appendix 2.5BB, Table 2.5.2-207). The results of this evaluation are discussed in Subsection 2.5.2.4.1.1.

Insert 5

The most significant update of source parameters for the Wabash Valley-Southern Illinois source zone(s) since the EPRI-SOG 1986 study is the estimate for maximum magnitude. The results of this evaluation are discussed in Subsection 2.5.2.4.1.2.

Insert 6

The results of this evaluation are discussed in Subsection 2.5.2.4.1.3.

2.5.2-303 Gray, H.H., C.H. Ault, S.J. Keller, and D. Harper, "Bedrock Geologic Map of Indiana," Indiana Geological Survey Miscellaneous Map 48, 2002.

2.5.2-304 Slucher, E.R., E.M. Swinford, G.E. Larson, and D.M. Powers, "Bedrock Geologic Map of Ohio," Ohio Geological Survey, Map BG-1, version 6.0, 2006.

2.5.2-305 Johnston, A.C., and E.S. Schweig, "The Enigma of the New Madrid Earthquakes of 1811-1812," *Annual Review of Earth and Planetary Sciences,* Vol. 24, 1996.

2.5.2-306 Hough, S.E., and S. Martin, "Magnitude Estimates of Two Large Aftershocks of the 16 December 1811 New Madrid Earthquake," *Bulletin of the Seismological Society of America*, Vol. 92, No. 8, 2002.

Insert 7

Insert 7

2.5.2-307

Olson, S. M., R. A. Green, and S. F. Obermeier, "Revised magnitude bound relation for the Wabash Valley seismic zone of the central United States," *Seismological Research Letters*, Vol. 76, No. 6, pp. 756-771, 2005.

Table 2.5.2-227
Resource Experts Contacted

Contact	Affiliation	Expertise	SSC Issues
Mark Baranoski	Ohio Division of Geological Survey	Basement structures	Evidence for reactivation of basement faults
Glenn Larsen	Ohio Division of Geological Survey	Geologic Structures	Identification and characterization of regional faults
Rick Pavey	Ohio Division of Geological Survey	Quaternary Geology	Information on Evidence for Quaternary faulting, pop-up structures
E. Mac Swinford	Ohio Division of Geological Survey	Division Assistant Chief	Referral to appropriate staff
Erick Venteris	Ohio Division of Geological Survey	Paleoliquefaction	Assessment of existing paleoliquefaction study results for identifying prehistoric earthquakes
Donovan Powers	Ohio Division of Geological Survey	GIS datasets	Obtaining digital datasets
John Esch	Michigan DEQ	Geologic Structures	Identification and characterization of faults in Michigan
Raymond Vurginovich	Michigan DEQ	Geologic Mapping and Well Database	Data compilation for source characterization
Ron Elowski	Michigan DEQ	Geologic Mapping and Well Database	Data compilation for source characterization
Steve Wilson	Michigan DEQ	Geologic Mapping	Data compilation for source characterization
Larry Organek	Michigan DEQ	Well database	Primary data for evaluating deformation history for subsurface faults in site vicinity
Roger Nelson	Michigan DEQ	Well database	Primary data for evaluating deformation history for subsurface faults in site vicinity
John Rupp	Indiana Geological Survey	Geologic Structures-Paleoliquefaction Studies in Indiana	Identification and characterization of regional faults
Mary Parke	Indiana Geological Survey	Seismic Hazard Studies in Indiana	Identification and characterization of regional faults
Terry Carter	Ontario Geological Survey	Structures defined from Oil and Gas	Identification and characterization of regional fault and subsurface faults in site vicinity
Desmond Rainford	Ontario Geological Survey	Datasets in Ontario	Data compilation for source characterization
James Boyd	Ontario Geological Survey	GIS datasets	Data compilation for source characterization
Thomas Hoane	Michigan Basin Geological Society	MBGS Publications	Data compilation for source characterization
Viki Bankey	USGS	Regional Magnetic Data	Data compilation for source characterization

Table 2.5.2-227
Resource Experts Contacted

Contact	Affiliation	Expertise	SSC Issues
Kaz Fujita	Michigan State University	Seismicity in Michigan	Data compilation for source characterization
Stephen Halchuck	Geological Survey of Canada	Seismicity	Data compilation for source characterization
John Adams	Geological Survey of Canada	Seismicity	Data compilation for source characterization
Steve Obermeier	USGS	Paleoliquefaction	Primary researcher for paleoliquefaction studies in Anna, Ohio and NE Ohio regions
Margaret Hopper	USGS	Seismicity	Data compilation for source characterization
Rich Harrison	USGS	Seismic Hazards	Data compilation – paleoliquefaction data sets
Bill Harrison	Western Michigan University	Geologic Structures	Data compilation for source characterization

Abbreviations:
SSC = Seismic Source Characterization
DEQ = Department of Environmental Quality
MBGS = Michigan Basin Geological Society
USGS = U.S. Geological Survey

Attachment 8
NRC3-10-0035

Response to RAI Letter No. 36
(eRAI Tracking No. 4766)

RAI Question No. 02.05.02-12

NRC RAI 02.05.02-12

FSAR Section 2.5.2 (Revision 2) includes multiple figures that are either truncated in the lower portions (e.g., FSAR Figure 2.5.2-261, and others) or do not include tick marks on the x axes (e.g., FSAR Figures 2.5.2-222 through 2.5.2-227). Please make the appropriate corrections to these FSAR figures. This request is in accordance with 10 CFR 100.23.

Response

FSAR Figure 2.5.2-214 is revised in this response to fully show the horizontal scale. Figures 2.5.2-237 and 2.5.2-238 are revised in this response to more clearly present the grid lines. Figures 2.5.2-261 and 2.5.2-262 are revised in the response to RAI 02.05.02-14.

FSAR Figures 2.5.2-222 through 2.5.2-228 will be revised, as needed, to more clearly denote the grid lines and the horizontal and vertical axes in the response to RAI 02.05.02-15, which will be provided to the NRC no later than August 13, 2010.

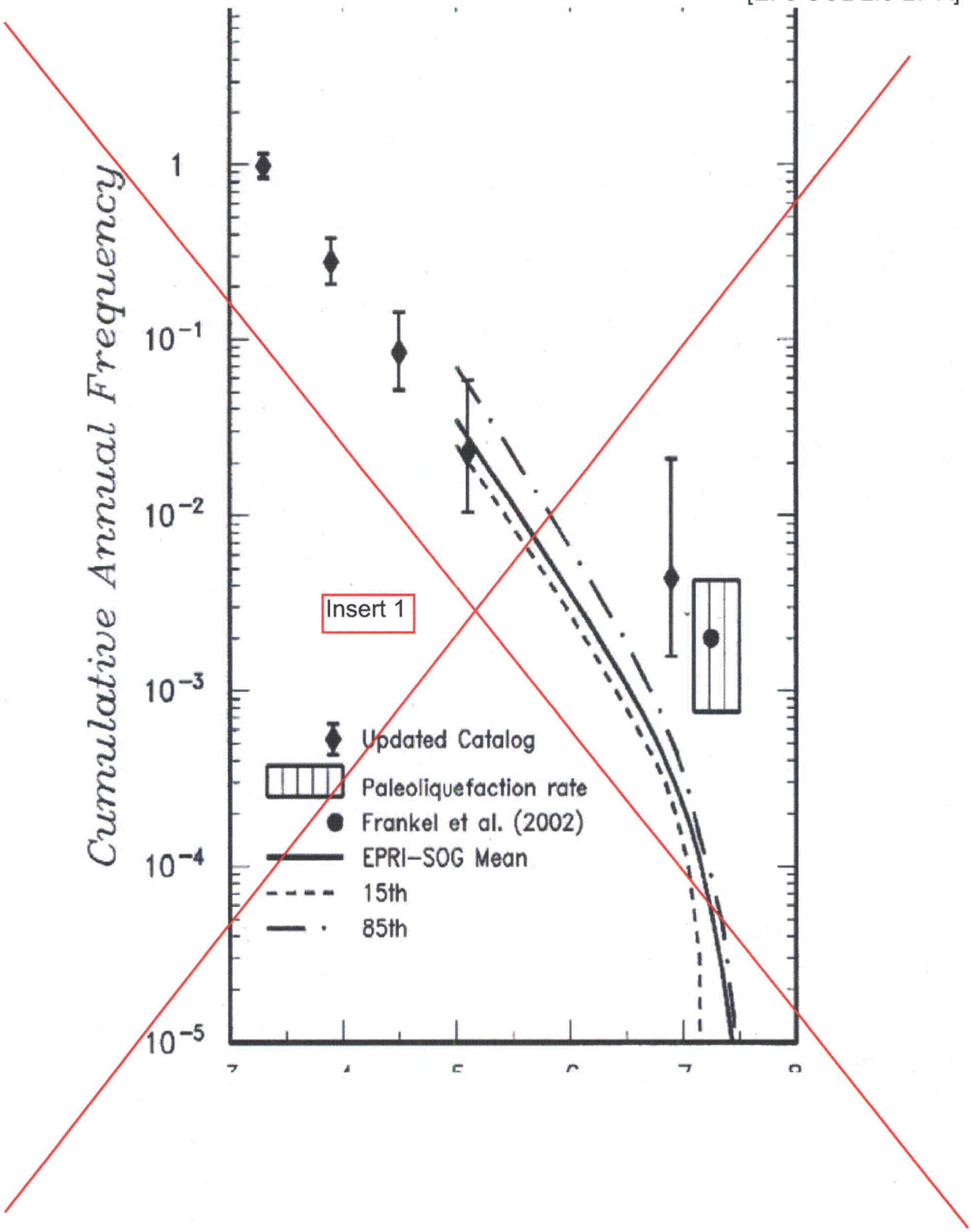
Proposed COLA Revision

Proposed revisions to FSAR Figures 2.5.2-214, 2.5.2-237, and 2.5.2-238 are shown on the attached markup.

Markup of Detroit Edison COLA
(following 6 pages)

The following markup represents how Detroit Edison intends to reflect this RAI response in the next submittal of the Fermi 3 COLA Revision 3. However, the same COLA content may be impacted by revisions to the ESBWR DCD, responses to other COLA RAIs, other COLA changes, plant design changes, editorial or typographical corrections, etc. As a result, the final COLA content that appears in a future submittal may be different than presented here.

Figure 2.5.2-214 Earthquake Recurrence Rates for New Madrid Seismic Sources
[EF3 COL 2.0-27-A]



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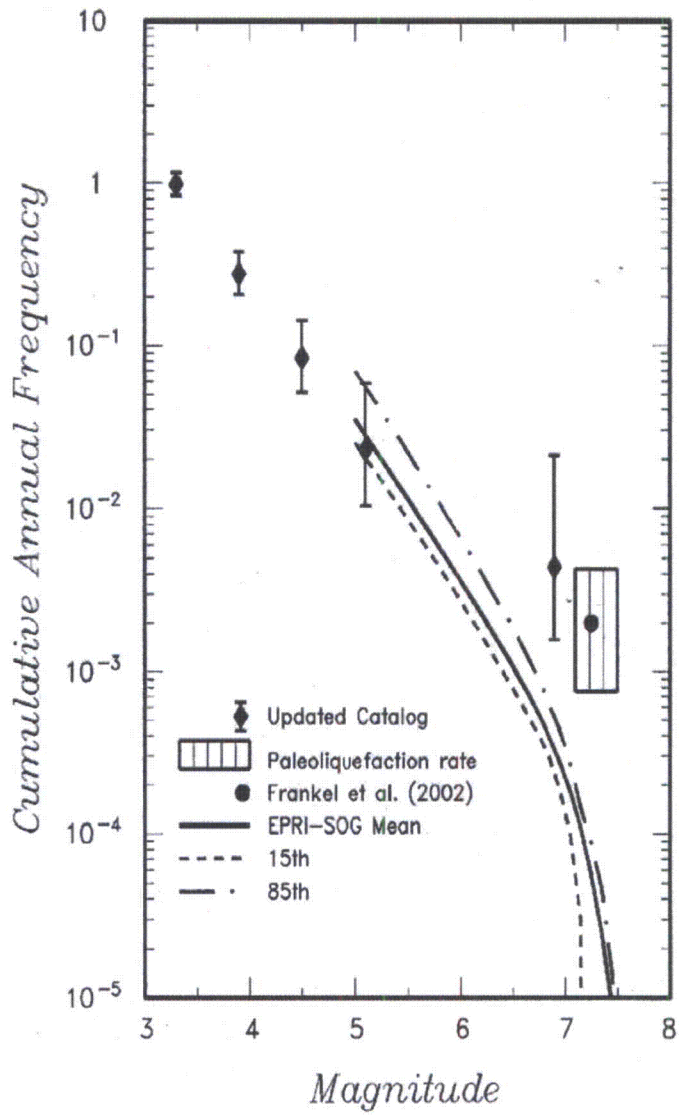
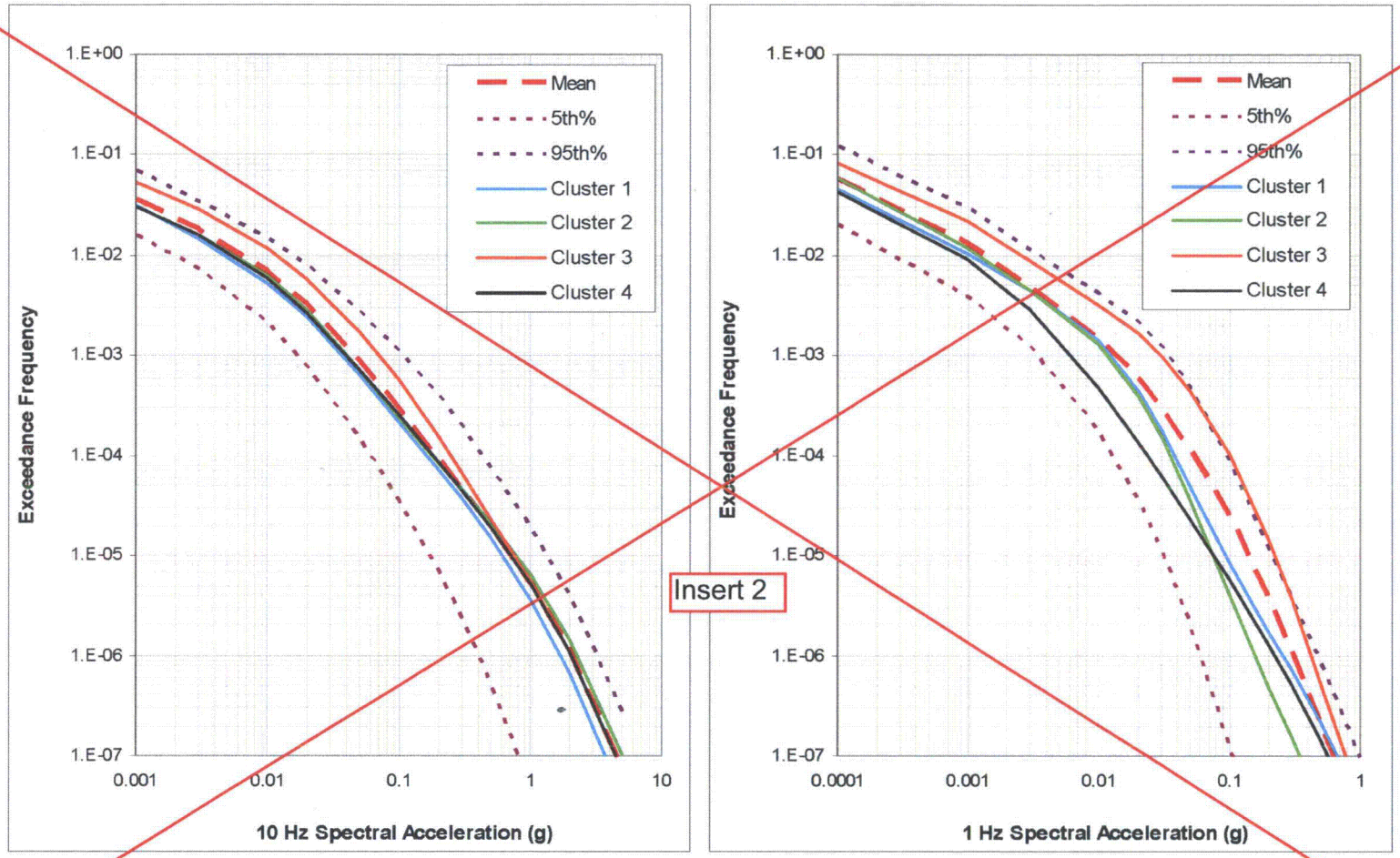


Source: Reference 2.5.2-244

Figure 2.5.2-237 Effect of Alternative EPRI (2004) Ground Motion Cluster Median Models on the Hazard Computed for the Fermi 3 Site [EF3 COL 2.0-27-A]



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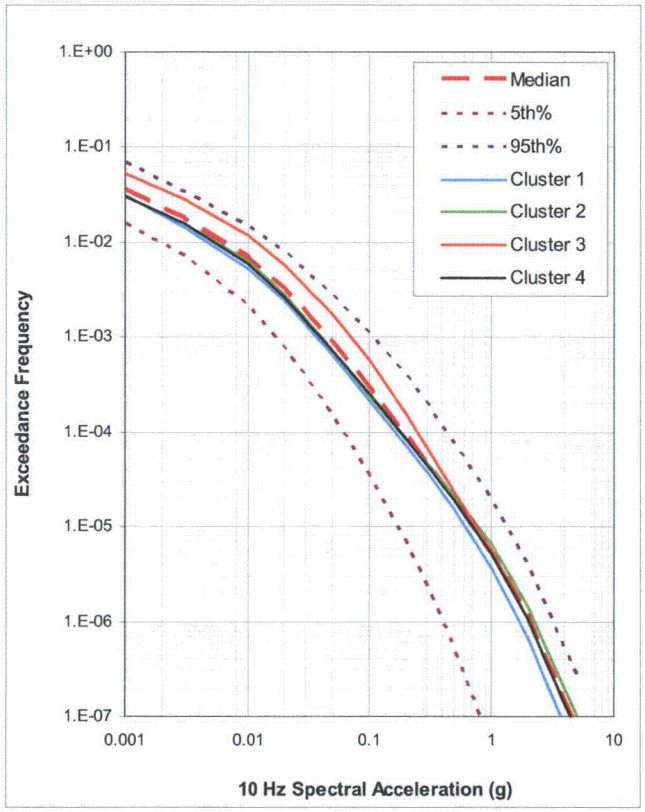
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Exceedance Frequency
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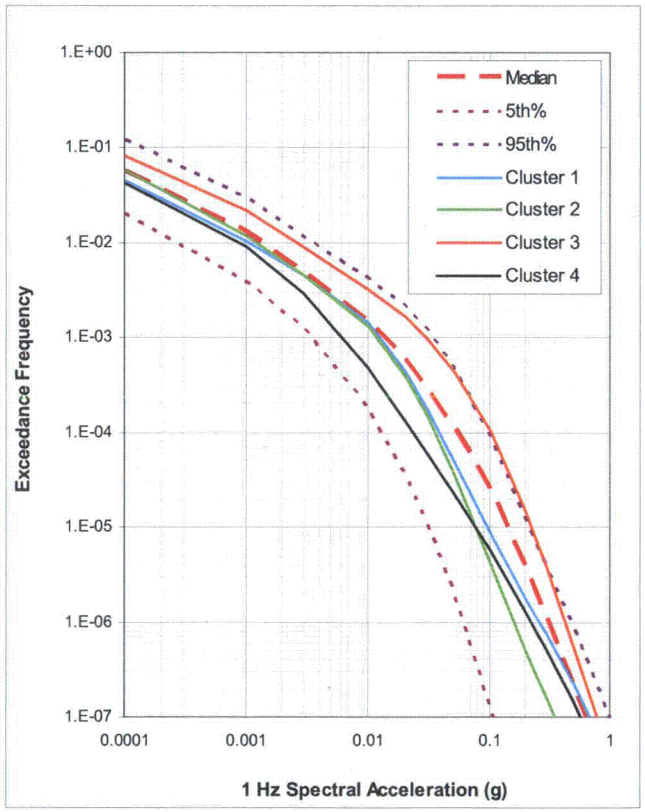
Cluster 2
Cluster 3
Cluster 4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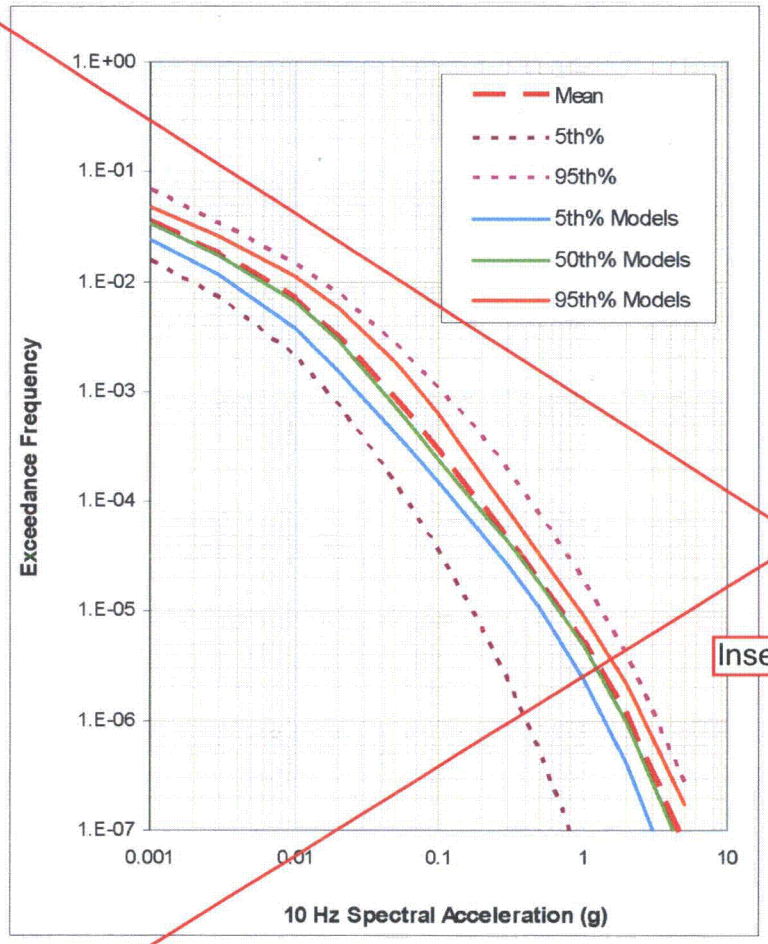
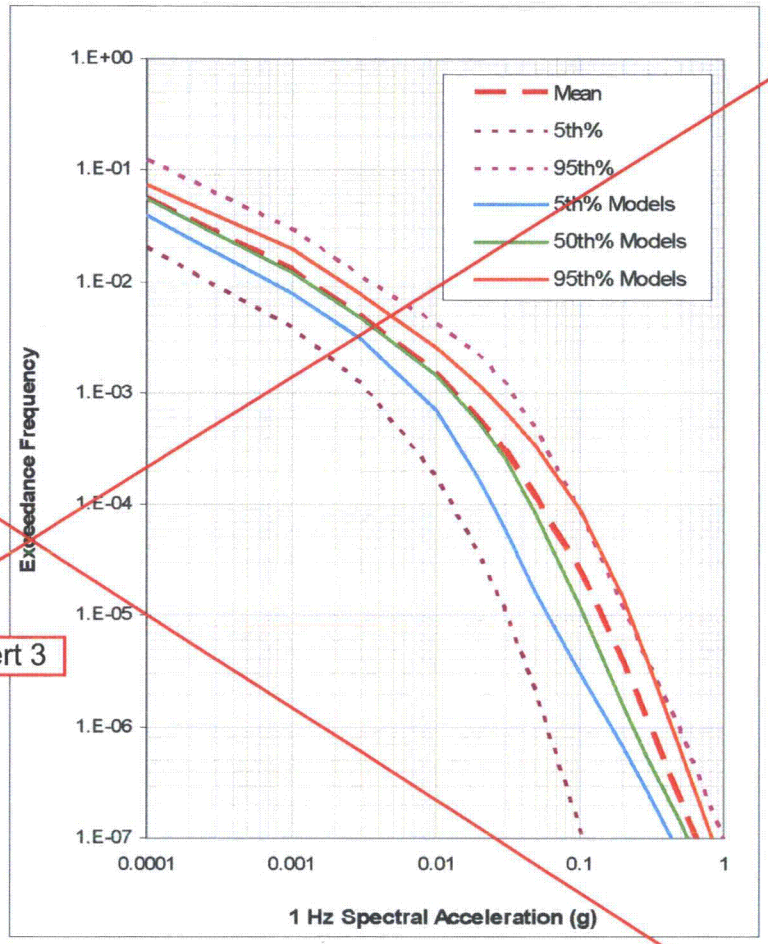
0.01
0.1
1
1 Hz Spectral Acceleration (g)
Median
5th%
95th%
Cluster 1
Cluster 2
Cluster 3
Cluster 4

Figure 2.5.2-238 Effect of Uncertainty in the EPRI (2004) Ground Motion Cluster Median Models on the Hazard Computed for the Fermi 3 Site [EF3 COL 2.0-27-A]





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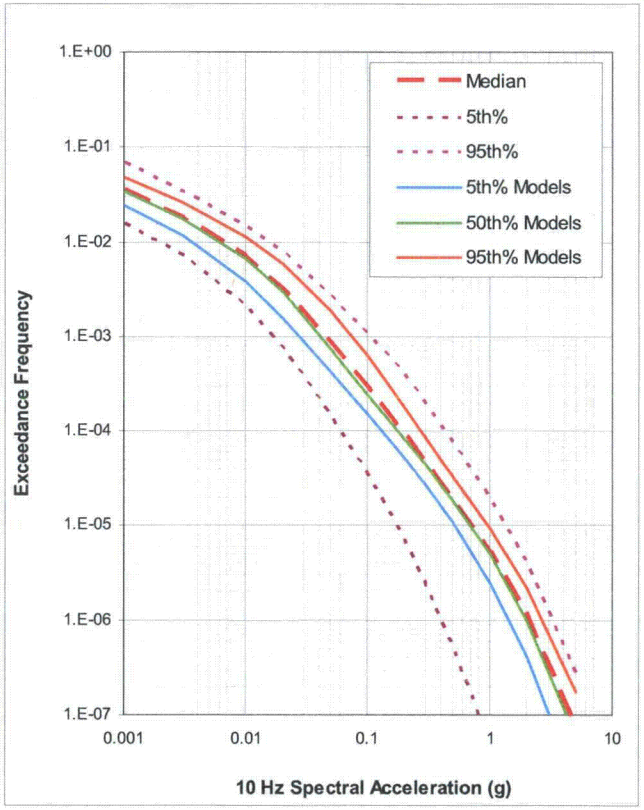
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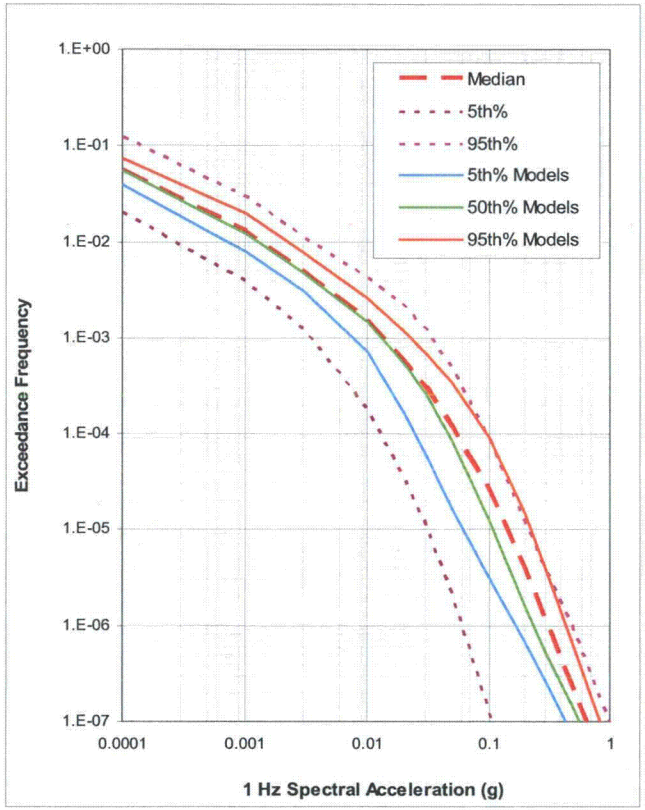
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Exceedance Frequency
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1 Hz Spectral Acceleration (g)
Median
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50th% Models
95th% Models

Attachment 9
NRC3-10-0035

Response to RAI Letter No. 36
(eRAI Tracking No. 4766)

RAI Question No. 02.05.02-13

NRC RAI 02.05.02-13

FSAR Section 2.5.2.5.1.2 states that attenuation models for CEUS hard rock assume a shallow crustal κ (attenuation parameter) value of approximately 0.006 seconds, which refers to a point at elevation 48 m (156 ft) at the Fermi 3 site. The FSAR further states that the material above this elevation will contribute additional damping and add to the total site κ. Using FSAR Section 2.5.2 Equation 11, you calculated an additional κ value of 0.013 seconds based on an average shear wave velocity of 5700 fps for the materials above elevation 48 m (156 ft). You then subtracted the hard rock κ value of 0.006, which yields a remaining κ of 0.007 seconds. In accordance with 10 CFR 100.23, please provide the following:

a. *Please confirm if the κ value of 0.013 seconds represents an additional damping contribution from the materials above elevation 48 m (156 ft).*

b. *If the κ value of 0.013 seconds does represent an additional damping contribution (and the κ value of 0.006 seconds represents the contribution from the crystalline rocks beneath elevation 48 m), please explain why you subtracted 0.006 seconds from the 0.013 seconds value.*

Response

The following responds to both questions (a) and (b) in RAI 02.05.02-13.

FSAR Section 2.5.2 Equation 11 represents the relationship between average shear wave velocity and *total* site κ, not just an additional damping contribution from the materials above elevation 48 m (156 ft). The text of the FSAR will be amended to clearly indicate this relationship. The CEUS (Reference 2.5.2-259) hard rock ground motion models already include a shallow crustal κ value of 0.006 seconds; thus, 0.006 seconds was subtracted from the total site κ value of 0.013 seconds obtained from Equation 11 to obtain the value of κ contributed by the materials above elevation 48 m (156 ft) at the Fermi 3 site.

Proposed COLA Revision

Proposed revision to FSAR Section 2.5.2.5.1.2 is shown on the attached markup.

Markup of Detroit Edison COLA
(following 2 pages)

The following markup represents how Detroit Edison intends to reflect this RAI response in the next submittal of the Fermi 3 COLA Revision 3. However, the same COLA content may be impacted by revisions to the ESBWR DCD, responses to other COLA RAIs, other COLA changes, plant design changes, editorial or typographical corrections, etc. As a result, the final COLA content that appears in a future submittal may be different than presented here.

viscoelastic wave-propagation modeling used in program SHAKE, the material damping, ξ, is obtained by the relationship:

$$\xi = \frac{1}{2Q_S} \qquad \text{[Eq. 7]}$$

Parameter Q_S is also related to the high-frequency attenuation parameter K developed by Anderson and Hough (Reference 2.5.2-283) by the relationship:

$$\kappa = \frac{H}{Q_S V_S} \qquad \text{[Eq. 8]}$$

where H is the thickness of the crust over which the energy loss occurs, typically taken to be 1 to 2 km (0.6 to 1.2 mi.) (Reference 2.5.2-284). Silva and Darragh (Reference 2.5.2-284) find that Q_S is proportional to shear-wave velocity:

$$Q_S = \gamma V_S \qquad \text{[Eq. 9]}$$

where γ is the constant of proportionality. Using this assumption, the amount of high-frequency attenuation in the ith layer of a velocity profile, k_i, is given by the relationship:

$$\kappa_i = \frac{H_i}{\gamma V_{Si}^2} \qquad \text{[Eq. 10]}$$

where H_i is the layer thickness and V_{Si}, is the layer shear-wave velocity. Given the total value of k appropriate for the site, one can solve for the corresponding value of γ. Using the resulting value of γ and Equations 7, 8, and 10, the appropriate damping values for each layer are then obtained.

The attenuation models for CEUS hard rock are developed assuming a shallow crustal k of approximately 0.006 second (Reference ~~2.5.2-283).~~ 2.5.2-259 This point is placed at elevation 48 m (156 ft.). The material above this elevation will contribute additional damping and thus add to the total site k. EPRI (Reference 2.5.2-285) gives the following relationship between k and site shear-wave velocity: total site

$$\log(\kappa) = 2.2189 - 1.0930\log(V_S) \qquad \text{[Eq. 11]}$$

1737 m/s (5700 fps)

where V_S is shear-wave velocity in fps and k is in seconds. The average shear-wave velocity of the rocks above elevation 48 m (156 ft.) is ~~5700 fps.~~ Using this value in Equation 11 yields a ~~k~~ value of 0.013 seconds. Subtracting the hard rock value of 0.006 yields a ~~remaining k~~ total site of 0.007 seconds. If this value is attributed to the top ~~131~~ 121 m (396 ft.) of dolomite, the damping values computed using the above equations ~~will be~~ are in the range of 3 to 7 percent. ~~This value appears~~ These values appear to be large in comparison with the low strain damping values typically assigned to soft rock materials. Silva et al. (Reference 2.5.2-286), as modified by Silva (Reference 2.5.2-287), proposed modulus reduction and damping relationships for soft rock that have low-strain damping values on the order of 3 percent. These would be expected to apply to relatively low velocity rocks. The Salina Group Unit F layer at the Fermi site is perhaps in the upper range of soft rock velocities. A set of modulus reduction and damping relationships used by EPRI (Reference 2.5.2-285) to model the behavior of soft rock that has low-strain damping values on the order of 1 percent or less. Based on these values, it was assumed that the low-stain damping in the softest rock layer, Salina Group Unit F is in the range of 1 percent to 3 percent. Using Equations 7, 8, and 10, damping values were computed for the remaining rock layers assuming that Q_S is proportional to V_S. The resulting values are listed in Table 2.5.2-221 along with the corresponding values of a k for each layer. The result is that the assigned values of damping add an additional k of 0.001 to 0.003 seconds.

The value of k assigned to a site profile is a measure of the total damping due to both material damping and scattering effects. To account for this in a one-dimensional (1-D) site response model, the conversion from k to material damping should account for the scattering (reflection) of waves off layer boundaries, particularly velocity reversals. In addition to those present in the initial velocity model, the process of profile randomization to account for site variability, discussed in Subsection 2.5.2.5.1.3, will introduce additional velocity reversals. The amount of k that is attributed to scattering in the site velocity profiles was assessed by comparing the median response of the randomized velocity profiles to a simple model with uniform velocity layers. The process used is shown on Figure 2.5.2-257. The randomized velocity profiles are used to compute the response of the site with the value of k set to zero in the rock layers under

Attachment 10
NRC3-10-0035

Response to RAI Letter No. 36
(eRAI Tracking No. 4766)

RAI Question No. 02.05.02-14

NRC RAI 02.05.02-14

In response to RAI 02.05.02-5, you stated that you conducted the Fermi site response analysis prior to completion of the laboratory dynamic testing performed on the glacial till. Furthermore, you stated that you selected the shear modulus reduction and damping relationships for the glacial till from literature. You then used those literature-based shear modulus reduction and damping curves in your site response analysis. Please update relevant FSAR figures that compare the shear modulus reduction and damping curves used in the site response analysis to the dynamic laboratory test results, similar to the figures provided in response to RAI 02.05.02-5. This request is in accordance with 10 CFR 100.23.

Response

Figures 2.5.2-261 and 2.5.2-262 in the FSAR have been updated to present the dynamic laboratory testing for the glacial till with the damping relationships used in the site response analysis. Figures 2.5.2-261 and 2.5.2-262 in the FSAR are similar to Figures 2 and 3 of the response to RAI 02.05.02-5 in Detroit Edison letter NRC3-10-0002 (ML100130382), dated January 11, 2010. The FSAR has been updated with text that discusses the dynamic laboratory testing, and Figures 2.5.2-261 and 2.5.2-262.

Proposed COLA Revision

Proposed revisions to FSAR Sections 2.5.2.5.1.2, 2.5.2.5.1.3, Figure 2.5.2-261, and Figure 2.5.2-262 are shown on the attached markup.

Markup of Detroit Edison COLA
(following 8 pages)

The following markup represents how Detroit Edison intends to reflect this RAI response in the next submittal of the Fermi 3 COLA Revision 3. However, the same COLA content may be impacted by revisions to the ESBWR DCD, responses to other COLA RAIs, other COLA changes, plant design changes, editorial or typographical corrections, etc. As a result, the final COLA content that appears in a future submittal may be different than presented here.

Poisson's ratio of 0.2 for lean concrete, the assigned values of Young's modulus and bulk density translate into a shear wave velocity of approximately 3600 fps. The FIRS analysis profile for the FWSC was constructed by placing approximately 9 m (30 ft.) of lean concrete on the top of the Bass Islands Group rocks.

The velocities and average layer thickness of the GMRS and three FIRS analysis profiles are listed in Table 2.5.2-220.

2.5.2.5.1.1 Density

Table 2.5.2-220 lists the average unit weight of the subsurface materials. These are taken from Table 2.5.4-202.

2.5.2.5.1.2 Shear Modulus and Damping

The top layer of the GMRS analysis profile consists of glacial till. The average plasticity index (PI) of this material is 14. As discussed in Subsection 2.5.4.7.5, a representative set of modulus reduction and damping relationships for this material were selected to be those developed by Vucetic and Dobry (Reference 2.5.2-281) for clays with a PI of 15. These relationships are shown on Figure 2.5.2-256. The curves developed by Vucetic and Dobry (Reference 2.5.2-281) are based in large part on remolded samples. In order to account for possible aging effects producing more linear behavior in the till, a second set of modulus reduction and damping relationships were also used. These consist of the Vucetic and Dobry (Reference 2.5.2-281) relationships for clays with a PI of 50 and are shown on Figure 2.5.2-256. ~~The laboratory test results presented in Subsection 2.5.4.7.5 fall within the range of these two published relationships and within the range of randomized modulus reduction and damping relationships used in the site response analyses.~~

Insert 1

The remaining portion of the GMRS profile consists of dolomites and claystones with shear wave velocities in excess of 3,000 fps. This material is expected to remain essentially linear at the levels of shaking defined by the rock hazard. The damping within these materials was established using the following procedure.

The site response analyses were conducted using an updated version of program SHAKE originally developed by Schnabel et al. (Reference 2.5.2-282). The energy lost in shear-wave propagation was measured by the shear-wave quality factor, Q_S, which can be equated to two other representations of energy loss in wave-propagation analysis. For the linear

Insert 1

Subsequent to completion of the site response analyses, dynamic test results were obtained for the glacial till. These test results are discussed in Section 2.5.4.7.5 and are presented on Figure 2.5.4-226. As shown on Figure 2.5.4-226, the laboratory test results generally fall within the range of the Vucetic and Dobry (Reference 2.5.2-281) shear modulus and damping relationships for PI =15 to PI = 50. Section 2.5.2.5.1.3 discusses a comparison of the laboratory test results with the randomized shear modulus and damping relationships used in the site response analyses, concluding that the laboratory test results fall within the range of randomized soil dynamic property relationships. In addition, the results of the site response analyses presented in Section 2.5.2.5.3 indicate that the site amplification functions are relatively insensitive to the choice between the relationships for PI =15 to PI = 50. Thus, it is concluded that the response of the glacial till at the Fermi 3 site is appropriately modeled using the published Vusetic and Dobry (Reference 2.5.2-281) relationships.

The modulus reduction and damping relationships were also randomized, as shown on Figure 2.5.2-261, Figure 2.5.2-262, and Figure 2.5.2-263. The standard deviation in the modulus reduction and damping were set so that the randomized relationships fell within recommended bounds provided by Silva (Reference 2.5.2-287). The damping ratio curves were limited to a maximum of 15 percent damping as recommended in Appendix E of Regulatory Guide 1.208.

Insert 2

The damping in the sedimentary rocks beneath the soil profile was computed using the randomized sedimentary rock layer velocities and thicknesses and the selected values of κ.

2.5.2.5.2 Acceleration Time Histories for Input Rock Motions

Response spectra were developed for each DE, as described in Subsection 2.5.2.4.4.3. Thirty time histories were developed for each DE from the time history sets given in McGuire et al. (Reference 2.5.2-270). Table 2.5.2-222 lists the time history sets used. The selected time histories were scaled to approximately match the target DE spectrum using a limited number of iterations of the program RASCALS (Reference 2.5.2-289). Figure 2.5.2-264 shows the response spectra for the 30 time histories scaled to match the HF and LF DEL and DEH spectra for mean 10^{-4} ground motions.

The purpose of randomization of the site properties is to account for natural variability in defining the site response. Part of the natural variability is variability in the ground motions of an individual earthquake. That is why only weak scaling of the time histories was performed. The weak scaling produces recordings that have, in general, the desired relative frequency content of the DE spectra while maintaining a degree of natural variability. The use of three DE for both HF and LF motions along with a large number of recordings provides adequate coverage of the frequency band of interest. The acceleration time histories represent free field outcropping motions for generic CEUS hard rock.

2.5.2.5.3 Site Amplification Functions

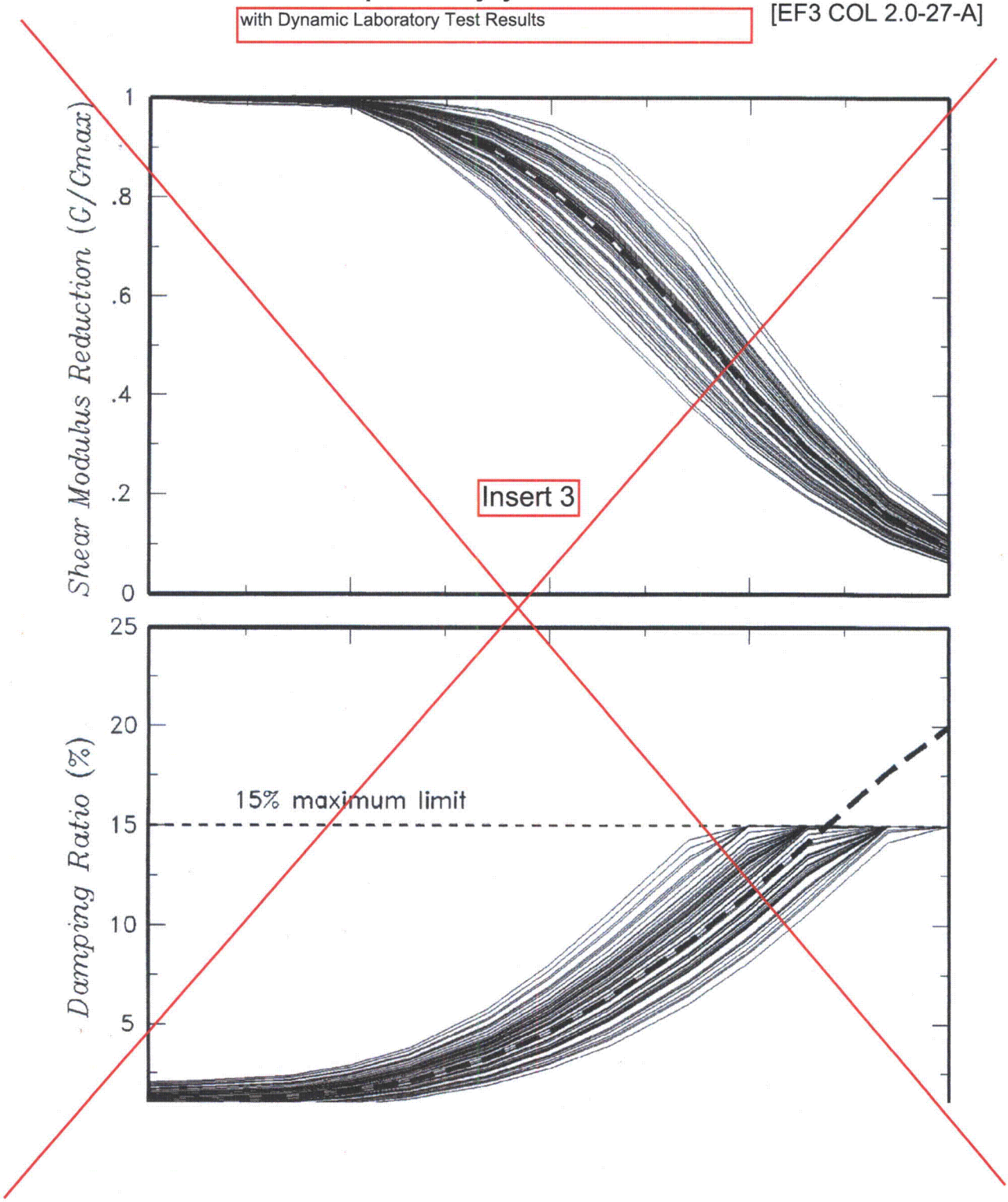
Site amplification functions were developed for each DE. The 60 randomized velocity profiles were paired with the 60 sets of randomized modulus reduction and damping curves (one profile with one set of modulus reduction and damping curves). Each of the 30 scaled time histories was used to compute the response of two profile-soil property curves sets. For each analysis, the response spectrum for the computed

Insert 2

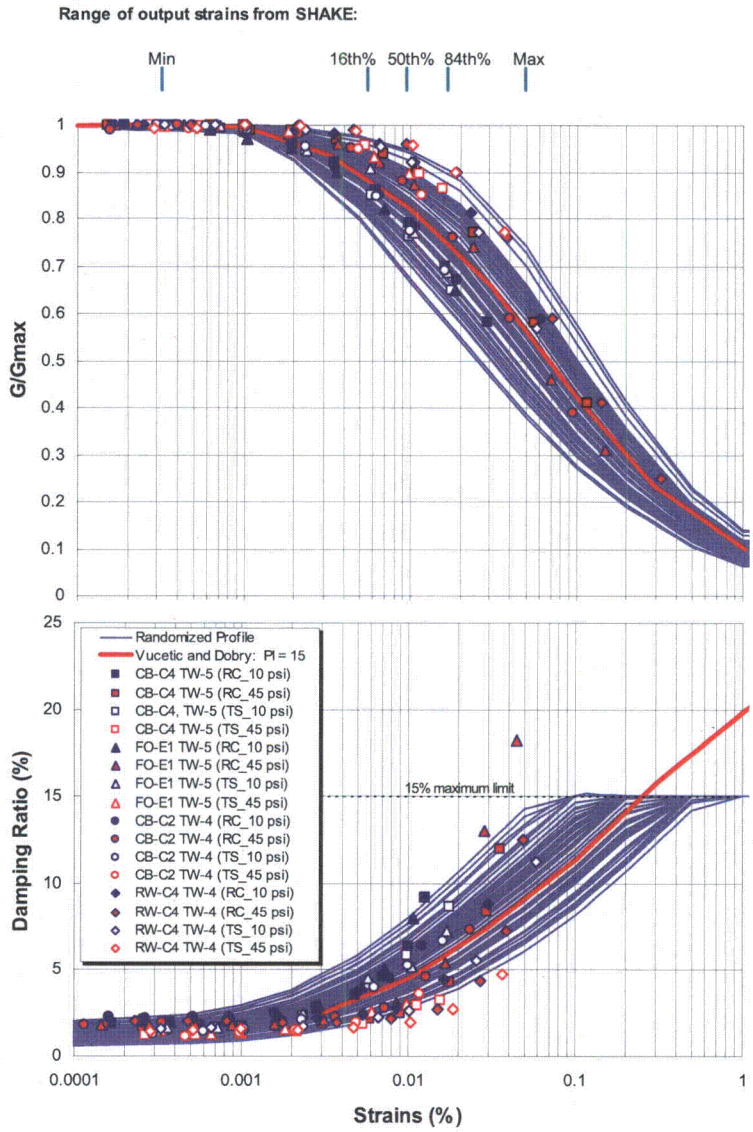
As discussed in Section 2.5.2.5.1.2, laboratory tests results for the glacial till were obtained after completion of the site response analyses. These test results are presented in Section 2.5.4.7.5 and are shown on Figure 2.5.4-226. The dynamic test results for the glacial till are overlain on the randomized shear modulus and damping relationships for PI = 15 and PI = 50 on Figures 2.5.2-261 and 2.5.2-262, respectively. The individual symbols on Figures 2.5.2-261 and 2.5.2-262 denote individual test results, with the blue and red colors indicating the confining pressures of 69 and 310 kPa and (10 and 45 psi), respectively. The solid and open symbols are test results from resonant column (RC) and torsional shear (TS) testing, respectively. Also shown at the top of Figures 2.5.2-261 and 2.5.2-262 are the range of shear strains computed in the site response analyses for the 10^{-5} level of high frequency input motion (Figure 2.5.2-272). Only the strain levels for the high frequency input motions are shown because the limited thickness of the till primarily effects the high frequency response of the site.

The comparisons on Figures 2.5.2-261 and 2.5.2-262 show that the dynamic laboratory test results for the glacial till fall within the range of the randomized modulus reduction and damping relationships used in the site response analyses.

Figure 2.5.2-261 **Randomized Shear Modulus (G) Reduction and Damping Relationships for Clayey Soils with a PI of 15 ~~Used~~ for Glacial Till** with Dynamic Laboratory Test Results [EF3 COL 2.0-27-A]



Insert 3



Notes:
1) RC_10psi = Resonant Column Test with Isotropic Confining pressure of 10 psi
2) RC_45psi = Resonant Column Test with Isotropic Confining pressure of 45 psi
3) TS_10psi = Torsional Shear Test with Isotropic Confining pressure of 10 psi
4) TS_45psi = Torsional Shear Test with Isotropic Confining pressure of 45 psi
5) Tests performed on thin-wall tube samples (designated with TW-4 and TW-5) from Borings CB-C4, FO-E1, CB-C2 and RW-C4

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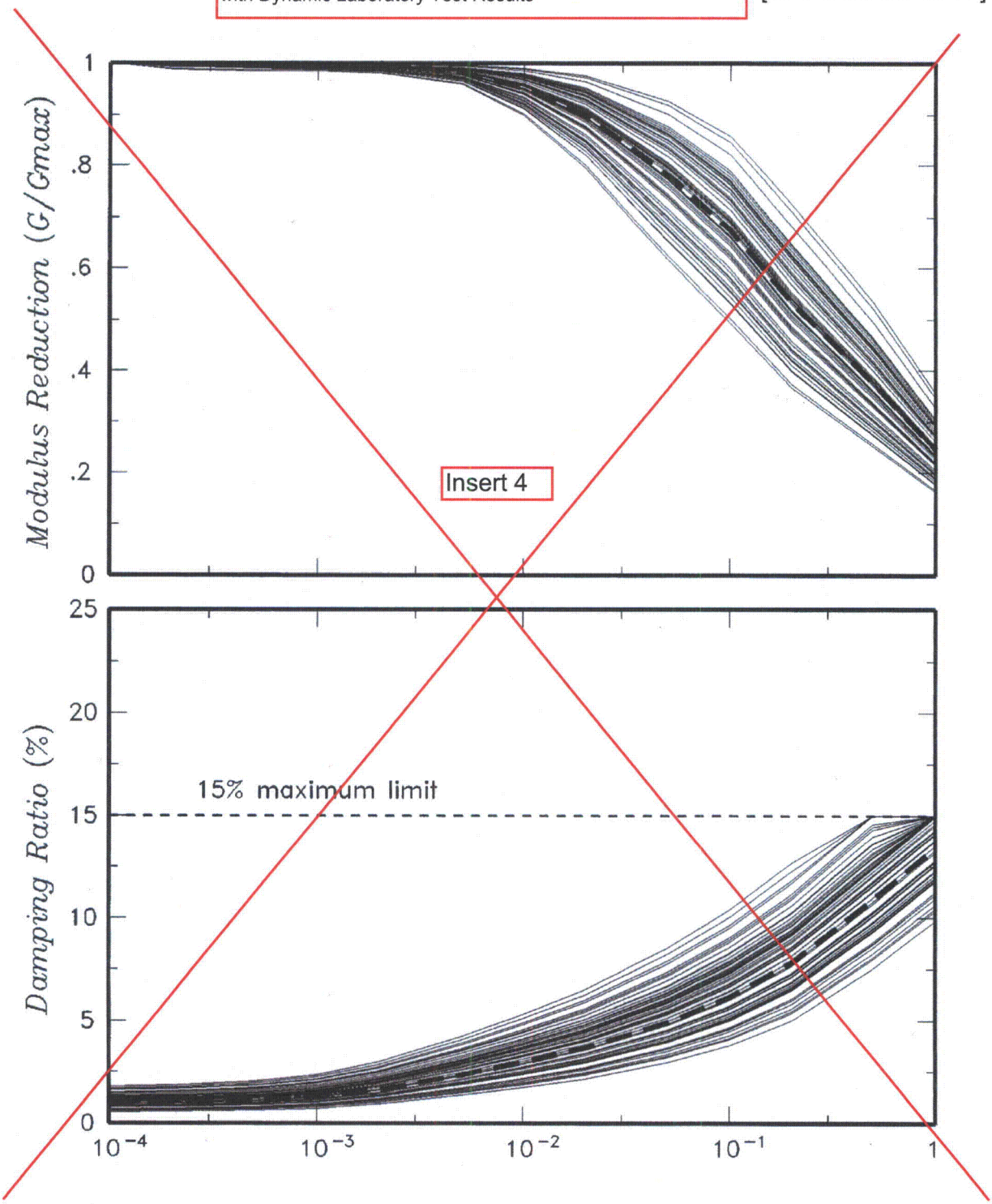
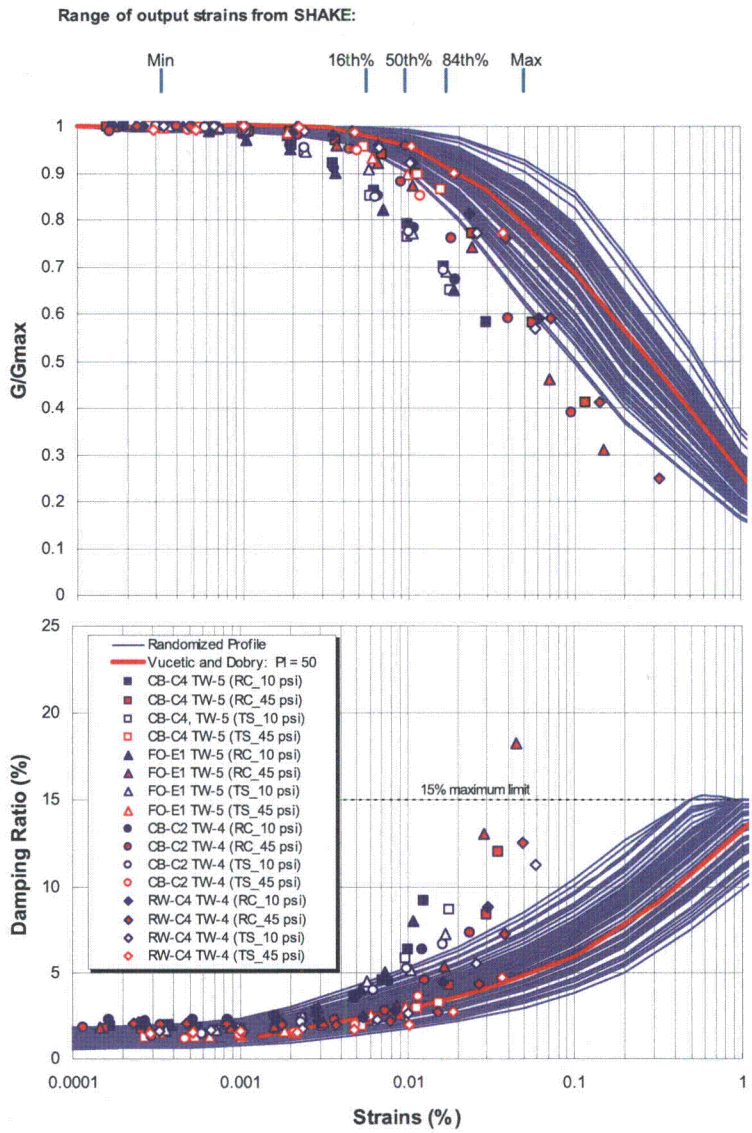
Figure 2.5.2-262 **Randomized Shear Modulus (G) Reduction and Damping Relationships for Clayey Soils with a PI of 50 ~~Used~~ for Glacial Till** with Dynamic Laboratory Test Results [EF3 COL 2.0-27-A]



Insert 4



Notes:
1) RC_10psi = Resonant Column Test with Isotropic Confining pressure of 10 psi
2) RC_45psi = Resonant Column Test with Isotropic Confining pressure of 45 psi
3) TS_10psi = Torsional Shear Test with Isotropic Confining pressure of 10 psi
4) TS_45psi = Torsional Shear Test with Isotropic Confining pressure of 45 psi
5) Tests performed on thin-wall tube samples (designated with TW-4 and TW-5) from Borings CB-C4, FO-E1, CB-C2 and RW-C4

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