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DTE Energy
Detroit Edison

10 CFR 52.79

December 9, 2010
NRC3-10-0056

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

References: 1) Fermi 3
Docket No. 52-033
2) Letter from Peter W. Smith (Detroit Edison) to USNRC, "Detroit Edison Company Response to the Fermi 3 COLA Review Schedule Milestone Changes," NRC3-10-0031, dated September 21, 2010
3) Letter from Peter W. Smith (Detroit Edison) to USNRC, "Detroit Edison Company Response to Fermi 3 COLA Review Schedule Milestone Changes – Update," NRC3-10-0052, dated November 9, 2010

Subject: Updates to the Fermi 3 COLA Reflecting Incorporation of DCD Rev. 7 and 8 Changes to Standard Plant Site Parameter - Soil Properties Requirements

In Reference 2, Detroit Edison provided an overview of the known remaining Fermi 3 COLA deliverable items. In that letter, Detroit Edison identified that a Soil-Structure Interaction (SSI) analysis was being performed to address GEH ESBWR DCD Revision 7 and Revision 8 changes to the Standard Plant Site Parameter – Soil Properties requirements.

Following a discussion with NRC staff on October 26, 2010, Detroit Edison indicated in Reference 3 that that the Soil Properties requirement for backfill adjacent to Class I structures from DCD Revision 7 would be satisfied and a site-specific SSI analysis would not be performed to support the Fermi 3 COLA. The markups addressing the incorporation of the Soil Properties requirement of DCD Revision 7 and Revision 8 are contained in Attachment 1.

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 9th day of December 2010.

Sincerely,

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

Attachments: 1) Markup of Fermi 3 COLA, Revision 2

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 (w/o attachments)
NRC Region III Regional Administrator (w/o attachments)
NRC Region II Regional Administrator (w/o attachments)
Supervisor, Electric Operators, Michigan Public Service Commission (w/o attachments)
Michigan Department Natural Resourses and Environment
Radiological Protection Section (w/o attachments)

Attachment 1
NRC3-10-0056

Markup of the Fermi 3 COLA, Revision 2

COLA Markup	Description of COLA Markup
FSAR Table 2.0-201	Revised minimum static/dynamic bearing demand to minimum static/dynamic bearing capacity, removed at rest pressure coefficient requirement, and updated soil density requirement to reflect DCD Revision 7 changes. Updated Notes 7 and 8 to reflect DCD Revision 7 and Revision 8 changes.
FSAR Section 2.5.4.5.4.2	Removed at rest pressure coefficient requirement and updated soil density requirement to reflect DCD Revision 7 changes. Added minimum shear wave velocity requirement. Added reference to new ITAAC for backfill adjacent to Category I structures.
FSAR Section 2.5.4.7.6	Revised the depth of engineered granular backfill due to the removal of the concrete plug requirement in DCD Revision 6.
FSAR Section 2.5.4.10	Revised dynamic bearing pressures for Reactor/Fuel Building, Control Building, and Firewater Service Complex to reflect DCD Revision 6 changes not made in FSAR Revision 2.
FSAR Section 2.5.4.10.3	Removed referenced concrete plug requirement to reflect DCD Revision 6 changes not made in FSAR Revision 2.
FSAR Section 2.5.4.11	Added minimum shear wave velocity associated with seismic strains for lower bound soil properties at minus one sigma from the mean to reflect DCD Revision 7 changes.
Part 10 Section 2.4	Added new ITAAC, 2.4.2, "ITAAC for Backfill Adjacent to Category I Structures," including new Table 2.4-1.

Markup of Detroit Edison COLA
(following 14 pages)

The following markup represents changes Detroit Edison intends to reflect in a future submittal of the Fermi 3 COLA. 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.

Subject (16)	DCD Site Parameter Value(1)(16)	Fermi 3 Site Characteristic	Evaluation
0% Exceedance Values			
Maximum	47.2°C (117°F) dry bulb 26.7°C (80°F) wet bulb (mean coincident)	40.1°C (104.1°F) dry-bulb with 23.3°C (73.9°F) wet bulb coincident (0% exceedance values)	The Fermi 3 site characteristic values for the 0% maximum dry bulb and wet bulb, coincident temperatures are the 100-year return period values. These values are 40.1°C (104.1°F) dry-bulb with 23.3°C (73.9°F) wet bulb coincident fall within (are less than) the DCD site parameter values for 0% exceedance.
	31.1°C (88°F) wet bulb (non-coincident)	30.0°C (86.0°F) wet-bulb (non-coincident) (0% exceedance value)	The Fermi 3 site characteristic value for the 0% maximum wet bulb temperature (non-coincident) is the 100-year return period value . This value is 30.0°C (86.0°F) wet-bulb (non-coincident) and falls within (is less than) the DCD site parameter value for 0% exceedance.
Minimum	-40°C (-40°F)	-34.9°C (-30.8°F)	The Fermi 3 site characteristic value for minimum temperature is the 100-year return period value. This value is -34.9°C (-30.8°F) and falls within (is higher than) the DCD site parameter value for 0% exceedance.
Soil Properties			
Minimum ~~Maximum~~ Static Bearing Capacity ~~Demand~~(7): Greater than or equal to the maximum static bearing demand. Maximum Static Bearing Demand:			
Reactor/Fuel Building	699 kPa (14,600 lbf/ft^2)	4,500 kPa (94,000 lbf/ft^2)	The Fermi 3 site characteristic value for allowable bearing capacity from Table 2.5.4-227 for the R/FB falls within (is greater than) the DCD site parameter value.
Control Building	292 kPa (6,100 lbf/ft^2)	14,029 kPa (293,000 lbf/ft^2)	The Fermi 3 site characteristic value for allowable bearing capacity from Table 2.5.4-227 for the CB falls within (is greater than) the DCD site parameter value.
Fire Water ~~Firewater~~ Service Complex	165 kPa (3450 lbf/ft^2)	1,532 kPa (32,000 lbf/ft^2)	The Fermi 3 site characteristic value for allowable bearing capacity from Table 2.5.4-227 for the FWSC falls within (is greater than) the DCD site parameter value.

Capacity (7): Greater than or Equal to the Maximum Dynamic Bearing Demand.

Minimum

Subject (16)	DCD Site Parameter Value(1)(16)	Fermi 3 Site Characteristic	Evaluation
Soil Properties (continued)			
~~Maximum~~ Dynamic Bearing ~~Demand~~ (continued)			
Maximum Dynamic Bearing Demand (SSE & Static):			
Reactor/Fuel Building			
Soft	1,100 kPa (23,000 lbf/ft^2)	5,980 kPa (125,000 lbf/ft^2)	The Fermi 3 site characteristic value for allowable dynamic bearing capacity for the RB/FB structure is from Table 2.5.4-227 and falls within (is greater than) the DCD site parameter value .
Medium	2,700 kPa (56,400 lbf/ft^2)		
Hard	1,100 kPa (23,000 lbf/ft^2)		
Control Building			
Soft	500 kPa (10,500 lbf/ft^2)	18,700 kPa (391,000 lbf/ft^2)	The Fermi 3 site characteristic value for allowable dynamic bearing capacity for the CB structure is from Table 2.5.4-227 and falls within (is greater than) the DCD site parameter value .
Medium	2,200 kPa (46,000 lbf/ft^2)		
Hard	420 kPa (8,800 lbf/ft^2)		
Firewater Service Complex (FWSC)			
Soft	460 kPa (9,600 lbf/ft^2)	2100 kPa (43,000 lbf/ft^2)	The Fermi 3 site characteristic value for allowable dynamic bearing capacity for the FWSC structure is from Table 2.5.4-227 and falls within (is greater than) the DCD site parameter value .
Medium	690 kPa (14,400 lbf/ft^2)		
Hard	1,200 kPa (25,100 lbf/ft^2)		

Subject [16]	DCD Site Parameter Value[1][16]	Fermi 3 Site Characteristic	Evaluation
Soil Properties (continued)			
Minimum Shear Wave Velocity[8]	300 m/s (1000 ft/s)	Value for each Seismic Category I structure: greater than 1,000 ft/sec for the reactor building/fuel building greater than 1,000 ft/sec for the control building greater than 1,000 ft/sec for the FWSC	The Fermi 3 site characteristic value for each Seismic Category I structure is based on the shear wave velocity of the supporting foundation material and material surrounding the embedded walls associated with seismic strains for lower bound soil properties at minus one sigma from the mean. For supporting foundation material, The ~~value~~ shear wave velocity for each structure falls within (is greater than) the DCD site parameter minimum value. As shown in **Figure 2.5.4-215** and **Figure 2.5.4-216**, the FB/RB, CB, and FWSC foundations are founded on uniform material. Therefore, the ratio of the largest to the smallest shear wave velocity over each mat foundation level does not exceed 1.7.
Liquefaction Potential			
Seismic Category I structures	None under footprint of Seismic Category I structures resulting from site-specific SSE	None at site-specific SSE under Seismic Category I structures	The Fermi 3 Category I structures are founded on bedrock or lean concrete and there is no potential for liquefaction under Fermi 3 Seismic Category I structures at the site-specific SSE ground motion.
Other than Seismic Category I structures	See Note (14)	See Evaluation column	Note (14) in **DCD Table 2.0-1** identifies a requirement to address liquefaction potential under other than Seismic Category I structures. **Subsection 2.5.4.8** provides the results of the analysis for the glacial till at the Fermi 3 site and addresses potential liquefaction under other than Seismic Category I structures. Based on the analysis provided, the glacial till is not susceptible to liquefaction.
Angle of Internal Friction (in-situ and backfill)	≥35 degrees	≥35 degrees	The Fermi 3 site characteristic value for angle of internal friction is provided in **Subsection 2.5.4.10** and falls within (is the same as) the DCD site parameter value.

Subject [16]	DCD Site Parameter Value [1][16]	Fermi 3 Site Characteristic	Evaluation
Backfill on sides of and underneath Seismic Category I structures ~~(not applicable if the fill material is concrete)~~		See Evaluation Column	The Fermi 3 site characteristic values for the backfill on the sides of seismic Category I structures are specified in Subsection 2.5.4.5.4.2 and fall within (is the same as) the DCD site parameter value. For Fermi 3, the fill material used underneath Seismic Category I structures is concrete.
i. Product of peak ground acceleration α (in g), Poisson's ratio ν and density γ	$\alpha(0.95\nu + 0.65)\gamma$: 1220 kg/m^3 (76 lbf/ft^3) maximum		
ii. Product of at-rest pressure coefficient k_o and density:	$k_o\gamma$: 750 kg/m^3 ~~(47 lbf/ft^3)~~ (47 lbf/ft^3) minimum		
~~iii. At-rest pressure coefficient:~~	~~k_o: 0.36 minimum~~		
~~iv.~~ iii. Soil density	γ: ~~1900~~ 2000 kg/m^3 ~~(119~~ 125 lbf/ft^3) minimum		

Table 2.0-201 Evaluation of Site/Design Parameters and Characteristics (Notes) (Sheet 1 of 2) [EF3 COL 2.0-1-A]

1. The site parameters defined in this table are applicable to Seismic Category I, II, and Radwaste Building structures, unless noted otherwise.
2. Probable maximum flood level (PMF), as defined in Table 1.2-6 of Volume III of **DCD Reference 2.0-4**.
3. Maximum speed selected is based on Attachment I of **DCD Reference 2.0-5**, which summarizes the NRC Interim Position on RG 1.76. Concrete structures designed to resist Spectrum I missiles of SRP 3.5.1.4, Rev. 2, will also resist missiles postulated in RG 1.76, Revision 1. Tornado missiles do not apply to Seismic Category II buildings. For the Radwaste building, the tornado missiles defined in Regulatory Guide 1.143, Table 2, Class RW-IIa apply.
4. Based on probable maximum precipitation (PMP) for one hour over 2.6 km^2 (one square mile) with a ratio of 5 minutes to one hour PMP of 0.32 as found in **DCD Reference** 2.0-3. See also **DCD Table** 3G.1-2.
5. See DCD Reference 2.0-9 for the definition of normal winter precipitation and extreme winter precipitation events. The maximum ground snow load for extreme winter precipitation event includes the contribution from the normal winter precipitation event. See also DCD Table 3G.1-2.
6. Zero percent exceedance values are based on conservative estimates of historical high and low values for potential sites. They represent historical limits excluding peaks of less than one hour: which are conservative relative to DCD Reference 2.0-4. One and two percent exceedance values were selected in order to bound the values presented in **DCD Reference 2.0-4** and available Early Site Permit applications.

See Insert "A"

7. ~~At the foundation level of Seismic Category I structures. The static bearing pressure is the average pressure. The dynamic bearing pressure is the toe pressure.~~ To compare with the maximum bearing demand, the allowable bearing pressure is ~~developed from the~~ site-specific bearing capacity divided by a factor of safety appropriate for the design load combination. ~~The maximum~~ dynamic bearing demand to be compared with the site-specific allowable dynamic bearing pressure ~~is the larger value~~ or a linearly interpolated value of the applicable range of shear wave velocities at ~~the foundation level. The shear wave velocities of soft, medium and hard soils are 300 m/sec (1000 ft/sec), 800 m/sec (2000 ft/sec) and greater than or equal to~~ 1700 m/sec (5600 ft/sec), respectively.
8. This is the minimum shear wave velocity of the supporting foundation material associated with seismic strains for lower bound soil properties at minus one sigma from the mean. The ratio of the largest to the smallest shear wave ~~velocity~~ over the mat foundation width of the supporting foundation material does not exceed 1.7.

and material surrounding the embedded walls

9. Safe Shutdown Earthquake (SSE) design ground response spectra of 5% damping, also termed Certified Seismic Design Response Spectra (CSDRS), are defined as free-field outcrop spectra at the foundation level (bottom of the base slab) of the Reactor/Fuel and Control Building structures. For the Firewater Service Complex, which is essentially a surface founded structure, the CSDRS is 1.35 times the values shown in DCD Figures 2.0-1 and 2.0-2 and is defined as free-field outcrop spectra at the foundation level (bottom of the base slab) of the Firewater Service Complex structure.
10. Values reported here are actually design criteria rather than site parameters. They are included here because they don't appear elsewhere in the DCD.
11. If a selected site has a X/Q value that exceeds the ESBWR reference site value, the COL applicant will address how the radiological consequences associated with the controlling design basis accident continue to meet the dose reference values provided in 10 CFR 52.79(a)(1)(vi) and control room operator dose limits provided in General Design Criterion 19 using site-specific X/Q values.

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7. At the foundation level of Seismic Category I structures. The dynamic bearing pressure is the toe pressure. The maximum static bearing demand is compared with the site-specific allowable static bearing pressure, which is obtained by dividing the ultimate soil bearing capacity by a factor of safety appropriate for the design load combination. The maximum dynamic bearing demand is compared with the site-specific allowable dynamic bearing pressure, which is obtained by dividing the ultimate soil bearing capacity by a factor of safety appropriate for the design load combination. When a site – specific shear wave velocity is between soft soil and medium soil the larger of the soft or medium maximum dynamic bearing demand will be used. When a site-specific shear wave velocity is between medium soil and hard soil the larger of the medium or hard maximum dynamic bearing demand will be used. Alternatively, for soils with a site-specific shear wave velocity a linearly interpolated dynamic bearing demand between soft and medium soil or between medium and hard soil can be used. The shear wave velocities of soft, medium and hard soils are 300 m/sec (1000 ft/sec), 800 m/sec (2600 ft/sec) and greater than or equal to 1700 m/sec (5600 ft/sec), respectively.

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

$\alpha(0.95\nu + 0.65)\gamma$: 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~~ 47 lbf/ft^3) minimum

~~iii.At-rest pressure coefficient:~~

~~κ_0: 0.36 minimum~~

~~iv.~~ iii. Soil density

γ: ~~1900~~ 2000 kg/m^3 (~~119~~ 125 lbf/ft^3) minimum

iv. Minimum shear wave velocity v_s associcated with seismic strains for lower bound soil properties at minus one sigma from the mean v_s: 300 m/s (1000 ft/s)

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.

in place

tests, shear wave velocity measurements

The ITAAC for backfill surrounding the embedded walls of Seismic Category I structures are provided in Part 10, Section 2.4.2.

The quality control program for granular backfill includes requirements for field density and index tests to confirm material classification and compaction characteristics are within the compliance range of materials specified and used for design. Granular backfill placement and compaction methods will be addressed in design specifications prepared in the detailed design stage of the project.

The details of the quality control and quality assurance programs for concrete fill and granular backfill are addressed in the specifications prepared during the detailed design phase of the project.

2.5.4.5.5 Control of Groundwater during Excavation

Control of groundwater and dewatering during excavation is presented in Subsection 2.5.4.6.2.

2.5.4.5.6 Geotechnical Instrumentation

The Fermi 3 excavation support and seepage control system will be continually monitored during excavation activities for movement and/or deflection. Real time data acquisition techniques may be used for collection and graphical representation of the data. An instrumentation and monitoring program developed during the project detailed design phase may include inclinometers, piezometers, seismographs, survey points, and construction inspection documentation.

Rebound or heave, less than 12.7 mm (0.5 inch), as presented in Subsection 2.5.4.10, is expected from foundation excavation; therefore heave monitoring is not needed.

As discussed in Section Subsection 2.5.4.10.2, settlement is predicted to be well within the design limits in the ESBWR DCD. Settlement is expected to occur during the construction phases of the project instead of during post construction because the Seismic Category I structures are founded on bedrock, which will compress elastically as the loads are applied. To confirm the settlement predictions, the following monitoring plan will be implemented.

- Benchmarks will be established at the corners of selected Seismic Category I structures as the foundation mats are constructed. These will be monitored before and periodically during construction of the basemats and sidewalls prior to placement of the backfill materials.
- Additional bench marks will be installed approximately 1 meter (3 feet) above site grade and connected to the sidewalls directly above the

The modulus reduction and damping curves for glacial till are needed for developing the GMRS. The shear modulus and damping curves for glacial till are chosen from published correlations (Reference 2.5.4-229). As shown in Table 2.5.4-204, the plasticity index of glacial till ranged from 7 to 27 percent with a mean value of 14 percent. The shear modulus reduction and damping curves with plasticity index equal to 15 and 50 were selected for glacial till as discussed in Subsection 2.5.2.5.1.2. The modulus reduction and damping curves were then randomized as shown on Figure 2.5.2-259 and Figure 2.5.2-260 as discussed in Subsection 2.5.2.5.1.3.

Measured shear modulus reduction and damping data from RCTS testing and published curves for a range of plasticity index values are plotted for comparison on Figure 2.5.4-226. The measured modulus reduction and damping curves from the RCTS tests are well within the randomized plasticity index 15 and 50 curves as shown on Figure 2.5.2-259 and Figure 2.5.2-260.

2.5.4.7.6 Shear Modulus Reduction and Damping Curves for Granular Backfill and Concrete Fill

Engineered granular backfill is not used to support any Seismic Category I structures. Engineered granular backfill is mainly used to backfill adjacent to the sidewalls of structures or to backfill beneath other structures with foundation levels above bedrock, except the Turbine Building, which is founded on lean concrete.

20 m (65.6 ft)

The shear modulus and damping curves for granular backfill are chosen from published correlations. The depths of engineered granular backfill range from 0 to approximately ~~11.3 to 11.6 m (37 to 38 ft)~~. The density of the engineered granular backfill is expected to be from dense to very dense. Therefore, shear modulus reduction and damping curves for sand from 6.1 to 15.2 m (20 to 50 ft) were selected for engineered granular backfill as shown on Figure 2.5.4-227.

Shear modulus reduction and damping curves for lean concrete fill are discussed in Subsection 2.5.2.5.

2.5.4.7.7 Ground Motion Response Spectra

The seismic velocity profiles are shown on Figure 2.5.4-220 through Figure 2.5.4-225. The Ground Motion Response Spectra (GMRS) and Foundation Input Response Spectra (FIRS) based on these velocity

159.6 m (523.7 ft) NAVD 88. The 4.0 m (13.1 ft) thick foundation is designed for soil pressures of 699 kPa (14,600 psf) (static) and ~~5,400~~ 1,100 kPa (~~112,800~~ 23,000 psf) (dynamic).

The CB mat foundation has plan dimensions of 23.8 by 30.3 m (78 by 99 ft) and bears 15.0 m (48.9 ft) below the final site elevation. The base of the CB foundation is thus at elevation 164.7 m (540.4 ft) NAVD 88. The 3.0 m (9.8 ft) thick CB mat is designed for allowable soil bearing pressures of 292 kPa (6,100 psf) (static) and ~~2,400~~ 420 kPa (50,200 8,800 psf) (dynamic).

The FWSC mat foundation has plan dimensions of 20 by 52 m (65.6 by 171 ft) and is embedded 2.4 m (7.7 ft) below the final site elevation. The base of the FWSC foundation is thus at elevation 177.3 m (581.6 ft) NAVD 88. The 2.5 m (8.2 ft) thick FWSC mat is designed for allowable soil bearing pressures of 165 kPa (3,450 psf) (static) and ~~670~~ 1,200 kPa (~~14,000~~ 25,100 psf) (dynamic).

The stability of the R/FB, CB, and FWSC foundations were evaluated for the various design conditions, which included Referenced DCD reference grade, maximum design groundwater elevation, and the total static dead plus live loads. Bearing capacity and foundation settlement potential were evaluated for the foundations using currently accepted methods and practices. Lateral earth pressures were calculated for the situation where compacted gravel backfill is placed against buried concrete walls (R/FB and CB only). The lateral earth pressures were based on the at-rest lateral earth pressure condition.

Table 2.5.4-226 summarizes building sizes, depths, and loadings for buildings in the power block area. The information was used for stability analyses in the following sections.

2.5.4.10.1 Bearing Capacity

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.

Table 2.5.4-208 shows the Mohr-Coulomb parameters, based on Hoek-Brown criterion. 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

2.5.4-230 and Table 2.5.4-231, respectively, for excavation rebound, and total (settlement from the rebounded position) foundation settlements. Only settlements under Seismic Category I structures are shown in these tables. The calculated total and differential settlements in Table 2.5.4-232 are within the acceptance criteria required in the Referenced DCD.

2.5.4.10.3 Lateral Earth Pressures

Static and seismic lateral earth pressures are addressed for Fermi 3 below-ground walls. From the Referenced DCD, the lateral soil pressure at rest is applied to external walls for R/FB and CB. Therefore, the R/FB and CB walls are assumed to not yield due to the lateral earth pressure applied to them. The at-rest pressure is the appropriate earth pressure to use for design of the walls per the Referenced DCD. For the Firewater Service Complex, the lateral soil pressure is not considered since it has no below-grade walls.

For a conservative analysis, the engineered granular backfill was assumed to be resting on the R/FB and CB walls from finish grade to bottom of foundation ~~with concrete plug as per the Referenced DCD requirements~~. Therefore, properties of engineered granular backfill were used for calculating lateral earth pressure from plant grade to the bottom of foundation. It is expected that the ϕ' of the engineered granular backfill is a minimum of 35 degree; therefore $\phi' = 35°$ was used for lateral pressure analysis. The saturated and unsaturated unit weights of 21.2 and 20.4 kN/m^3 (135 and 130 pcf), respectively, was conservatively assumed for the engineered granular backfill.

Hydrostatic pressures are conservatively based on the groundwater table being 0.6 m (2 ft) below grade [El. 179.0 m (587.3 ft), NAVD 88]. A surcharge pressure of 24 kPa (500 psf) is used. Considering the small to medium sized compaction equipment normally used for compaction of backfill behind rigid retaining walls, a 24 kPa (500 psf) compactive surcharge pressure is appropriate for the additional compaction lateral earth pressures that are developed (Reference 2.5.4-245).

2.5.4.10.3.1 Static Lateral Earth Pressures

The at-rest static lateral earth pressure σ_h for a given depth z is calculated as follows (Reference 2.5.4-246):

$$\sigma_h = K_0 \sigma_0' + u \qquad \text{[Eq. 9]}$$

is at least 2.25. The selection of shear strength parameters used in the bearing capacity evaluation is discussed in Subsection 2.5.4.2.1.

Results of the geophysical surveys for shear wave velocity are presented in Subsection 2.5.4.4.1 and shear wave velocity profiles are summarized in Subsection 2.5.4.7.2. The minimum shear wave velocity of the supporting foundation material associated with seismic strains for lower bound soil properties at minus one sigma from the mean is greater than 1,000 fps as discussed in Subsection 2.5.4.7.2.

For backfill surrounding Seismic Category I embedded walls, the minimum shear wave velocity associated with seismic strains for lower bound soil properties at minus one sigma from the mean is 1,000 fps as discussed in Subsection 2.5.4.5.4.2.

The static stability analyses are presented in Subsection 2.5.4.10. The design criteria for static stability analyses are identified in Subsection 2.5.4.10 and are compared to site parameters in Table 2.0-201. Discussion of the assumptions and methods of analyses for the static stability analyses are provided in Subsection 2.5.4.10.

Subsection 2.5.4.8 discusses the liquefaction potential of soils encountered and fill at the site. It is concluded that there are no liquefiable soils under and adjacent to all Seismic Category I structures.

DCD Table 2.0-1 requires that that $\phi' \geq 35°$. Seismic Category I structures are founded on bedrock or lean concrete extending to bedrock. The angle of internal friction of bedrock is greater than 35 degree based on laboratory direct shear tests performed on samples with discontinuities from the Bass Islands Group and empirical correlations using Hoek-Brown criterion. Engineered granular backfill is used to backfill adjacent to all Seismic Category I structures and based on compaction requirements the angle of internal friction of engineered granular backfill should be greater than 35 degrees.

The design criteria required for the foundation settlement for Seismic Category I structures are addressed in Subsection 2.5.4.10.2. The calculated foundation settlements of all Seismic Category I structures were demonstrated to be less than the maximum settlement values specified in the Referenced DCD.

The computer program used in the settlement analysis (Subsection 2.5.4.10.2) was validated by comparing the results obtained from computer program to solutions obtained from theoretical equations.

2.5.4.12 Techniques to Improve Subsurface Conditions

The R/FB and CB are founded on bedrock. Based on the stability analysis presented on Subsection 2.5.4.10, no subsurface improvement

2.4 SITE-SPECIFIC ITAAC

The Site Specific ITAAC are provided in the following sections. Site specific systems were evaluated against selection criteria in **FSAR Section 14.3**. If a site-specific system described in the FSAR does not meet an ITAAC selection criterion, only the system name and the statement "No entry for this system" is provided.

Seismic

2.4.1 ITAAC FOR BACKFILL UNDER CATEGORY I STRUCTURES

Seismic

Not applicable since no compactable backfill will be placed under Fermi 3 Category I structures.

2.4.2 ITAAC FOR BACKFILL SURROUNDING SEISMIC CATEGORY I STRUCTURES

Compactable backfill surrounding Fermi 3 Seismic Category I structures. The ITAAC for backfill surrounding the embedded walls of Seismic Category I structures are provided in Table 2.4-1.

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Table 2.4-1
ITAAC for Backfill Adjacent to Seismic Category I Structures

Design Commitment	Inspections, Tests, and Analyses	Acceptance Criteria
1. Shear wave velocity of the backfill material surrounding Seismic Category I structures, associated with seismic strains for lower bound soil properties at minus one sigma from the mean, is greater than or equal to 1,000 feet per second.	Field measurements and analyses of shear wave velocity in backfill will be performed.	An engineering report exists that concludes that the shear wave velocity within backfill material surrounding Seismic Category I structures, associated with seismic strains for lower bound soil properties at minus one sigma from the mean is greater than or equal to 1,000 feet per second.
2. The engineering properties of backfill material surrounding Seismic Category I structures are equal to or exceed the Design Control Document requirements.	Laboratory tests and field measurements to evaluate the engineering properties of the backfill will be performed. Laboratory tests will include: • Direct Shear Tests • Relative Density and/or Proctor Tests • Sieve Analyses • Moisture Content Field measurement will include: • Standard Penetration Tests (SPT) • In-place Density Tests	An engineering report exists that concludes that the engineering properties of backfill material surrounding Seismic Category I structures are equal to or exceed the Design Control Document requirements as follows: • Angle of Internal Friction ≥ 35 degrees • Product of peak ground acceleration, α, (in g), Poisson's ratio, ν, and density, γ,: $\alpha(0.95\nu+0.65)\gamma$: 1220 kg/m^3 (76 lbf/ft^3) maximum • Product of at-rest pressure coefficient, k_0, and density, γ,: $k_0\gamma$: 750 kg/m^3 (47 lbf/ft^3) minimum • Soil density: γ: 2000 kg/m^3 (125 lbf/ft^3) minimum