SEISMIC IMPACT ZONES DEMONSTRATION

CFR 257.63

Bottom Ash Complex

Rockport Plant
Town of Rockport, Spencer County, Indiana

January, 2018

Prepared for: INDIANA MICHIGAN POWER COMPANY - Rockport Plant
Town of Rockport, Spencer County, Indiana

Prepared by: Geotechnical Engineering Services
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AMERICAN ELECTRIC POWER
BOUNDLESS ENERGY

GERS-18-004

Seismic Impact Zone Demonstration
CFR 257.63
Bottom Ash Complex
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I certify to the best of my knowledge, information, and belief that the information contained in this seismic impact zones demonstration meets the requirements of 40 CFR § 257.63

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1 OBJECTIVE

This report was prepared by AEP- Geotechnical Engineering Services (GES) section to fulfill requirements of the new Promulgated CCR Rule CFR § 257.63. Per the New Promulgated CCR Rule, New CCR landfills, existing and new CCR surface impoundments, and all lateral expansions of CCR units must not be located in seismic impact zones unless the owner or operator demonstrates by the dates specified in paragraph (c) of the referenced section that all structural components including liners, leachate collection and removal systems, and surface water control systems, are designed to resist the maximum horizontal acceleration in lithified earth material for the site.

This report will evaluate whether the Bottom Ash Ponds (BAP) Complex at Rockport Plant is located in seismic impact zones, and if so, the report will demonstrate that the all structural components including liners, leachate collection and removal systems, and surface water control systems, are designed to resist the maximum horizontal acceleration in lithified earth material for the site

2 DESCRIPTION OF THE PLANT AND THE CCR IMPOUNDMENT

The Rockport Power Plant is located at 791 N US Highway 231, Rockport, IN 47635-8883. The coordinates of the site are 37º55’32” N latitude and 87º02’02” W longitude. A Site Location Map is included as Figure 1. The plant operates two coal fired generating units rated at 1,300 megawatts (MW) each.

Unit 1 and Unit 2 were placed in service in 1984, and 1989, respectively. A Facility Layout Plan is included as Figure 2. Coal Combustion Waste (CCW) that is produced during power generation is managed on-site with a CCW impoundment.
The facility utilizes six contiguous and hydraulically connected impoundments or cells (see Figure 2) known as the BAP Complex for CCW management. The cells are separated by internal divider dikes. The individual cells of the BAC are identified as follows:

- East Bottom Ash Pond
- West Bottom Ash Pond
- East Wastewater Pond
- West Wastewater Pond
- Reclaim Pond
- Clear Water Pond

The wastewater pond complex is a combination incised and diked earthen embankment impoundment. It is incised below grade along most of its perimeter, and is diked only on the west side of the West BA Pond, where the topography decreases in elevation toward a remnant drainage channel.

The embankments, including the west dike, have a crest elevation of 399 feet, and are approximately 30 feet wide. The west dike has a maximum height (from crest to outboard toe) of 13 feet. The inboard slope was constructed at a slope of 2 horizontal to 1 vertical (2H:1V), and the outboard slope at 2.5H:1V. The outer west dike, and the internal splitter dikes (constructed between the BA Ponds, and between each of the BA Ponds and the wastewater ponds to the south) were constructed of natural clayey soils excavated from the interior of the ponds. The inboard slopes were armored with rock riprap. Reportedly, no engineered liner systems are present in the BA Ponds or the other ponds in the wastewater pond complex.

Based on the usage of the above mentioned ponds, only the East Bottom Ash Pond and the West Bottom Ash Pond are considered CCR units. These two ponds the subjects of this demonstration report.

3 SEISMIC IMPACT ZONE DETERMINATION 257.63(a)

Per the CCR Rules Definition, a seismic impact zone means an area having a two (2%) or greater probability that the maximum expected horizontal acceleration, expressed as a percentage of the earth's gravitational pull (g), will exceed 0.10 g in 50 years.

The first step toward achieving compliance with this requirement is to identify whether the impoundment site lies within a seismic impact zone as defined above.

The determination of whether Rockport Plant area falls in a seismic impact zone and the level of the seismic acceleration is based on two approaches, the USGS web site as well as a site specific seismic analysis conducted for the plant area.

3.1 USGS MAP/WEB SITE DETERMINATION

The U.S. Geological Survey (USGS) National Seismic Hazard Mapping Program (NSHMP) Interactive Deaggregation website was used to provide the design ground acceleration relating to the design seismic event. For a 2,475-year return period (2% exceedance probability in 50 years), the website output indicates a PGA of 0.14957 g for the hard rock site (Based on URS Report recommendations, APPENDIX A). The corresponding earthquake magnitude (M) was 6.46.

3.2 SITE SPECIFIC SEISMIC ANALYSIS

URS Company (URS), Currently AECOM, performed a site-specific seismic hazard analysis for the Rockport power plant site in Indiana. The objective of the study was to compute the design earthquake response spectrum for the site per the requirements in Chapter 21 of the ASCE 7-05 standard, which is incorporated by reference in the 2006 International Building Code (IBC).
The study also meets the requirements of the Indiana State Building Code, which amends certain sections of the IBC.

The site-specific PGA computed in URS study for a 2,475-year return period is 0.13 g, very comparable to the USGS mapped value. Excerpts of the URS (AECOM) study are included in APPENDIX A.

Based on the results of the two approaches, the design seismic acceleration of the facility is to be taken as 0.14957 g. Therefore, the BAP complex falls in a seismic impact zone and the analysis of this report will attempt to demonstrate that the Structural components including liners, leachate collection and removal systems, and surface water control systems, are designed to resist the 0.14957 g, maximum horizontal acceleration in lithified earth material.

4 DESCRIPTION OF THE FOUNDATION AND EMBANKEMENT MATERIALS 275.73(c)(1)(v)

[A description of the physical and engineering properties of the foundation and abutment materials on which the CCR unit is located.]

The description of the BAP Complex embankment and foundations soils were based on the 2016 site investigation and laboratory testing conducted by AEP Civil Engineering Laboratory.

4.1 SITE INVESTIGATION

AEP Civil Engineering Drilling crew conducted a soil site investigation of which two (2) soil test boring series (B-1605 and B-1606) that were drilled through the embankment and the foundation soils (See Figure 2), were selected for this demonstration. Representative but disturbed soil samples were collected in jars/bags and transferred to AEP Civil Engineering Laboratory for classification and testing. The Standard Penetration Resistances ($N1_{60}$-values) varied between a low of 2 to a high of 100 (refusal) blows per foot (bpf) with an average $N1_{60}$-values of 35 bpf.

The soils within the embankment were lean clay extending below the embankment with a total depth of 27-30 ft. The clay layer was underlain by fine to coarse sand deposits. Figure 4 present the soil profile interpreted from the two borings. Bedrock at the plant site is at approximate elevation of 290 ft-msl and comprised of predominantly shale.

Soil Samples from the borings at various depths were tested at AEP Civil Engineering Laboratory for the following tests:

- Moisture Content (ASTM 2216)
- Grain Size Analyses (ASTM D 422)
- Atterberg Limits (ASTM D 4318

Based on the lab soil tests results, the tested soils are non-plastic silty sand with fine content ranging from 14.5 to 28.6% with minor pockets of sandy lean clay. Laboratory test reports are included in APPENDIX B. Soil classification, index properties, and shear strength values obtained from subsurface soil investigation and laboratory tests are summarized in Table 1 below.

Table 1 Soil Properties Obtained in 2016 Investigation Laboratory Testing

Soil Boring ID	Sample Depth (ft)	USCS Classification	Fine Content (%)	Moisture Content (%)	Atterberg Limits		
					LL	PL	PI
MW-1605D	115.0-124.6ft	POORLY GRADED SAND SP	14.5	5.4	NP	NP	NP
MW-1605I	68.6-78.2ft	POORLY GRADED SAND SP	19.7	2.5	NP	NP	NP
MW-1605S	37.6-47.2ft	POORLY GRADED SAND SP	16	2.1	NP	NP	NP
MW-1606D	100.0-109.6ft	POORLY GRADED SAND SP	28.6	7.3	NP	NP	NP
MW-1606I	65.7-75.3ft	POORLY GRADED SAND SP	18.9	5.4	NP	NP	NP
MW-1606S	34.7-44.3ft	POORLY GRADED SAND SP	20.9	1.7	NP	NP	NP

APPENDIX B includes the boring logs for relevant boring 1605 and 1606 as well as the corresponding lab tests.

5 MODES OF FAILURE AND STABILITY DEMONSTRATION

Based on § 257.63 (a) part of the Rules, only East and West bottom Ash Ponds are required to be covered under this demonstration. Seismic impact zones' Structural components including liners, leachate collection and removal systems, and surface water control systems, are designed to resist the maximum horizontal acceleration in lithified earth material for the site.

5.1 FAULTS

Based on the geological survey of the Pond Complex area, there is no fault exists in the locality under the ponds dikes. This mode of failure is considered not applicable for the bottom ash pond complex.

Based on published data no active faults are known to traverse the site and no surficial evidence of faulting was observed during various field investigation conducted at the site. Figure 5 and Figure 6 present the nearest mapped fault trace considered to be active is one of a group of faults located approximately 5 miles west of the site.

5.2 Liquefaction Potential

Liquefaction is a condition where seismic ground motions cause excessive pore pressures in soils that result in a loss in shear strength. Liquefaction can cause slope instability and/or settlement. Liquefaction is most likely to occur for (1) loose sands/silts, (2) shallow groundwater conditions, and (3) strong ground motions.

Liquefaction potential analysis was performed using LiquefyPro program developed by CivilTech Software Company. The program evaluates liquefaction potential and calculates the settlement of soil deposits due to seismic loads.

LiquefyPro program is based on the most recent publications of the NCEER Workshop and SP117 Implementation. The user can choose between several different methods for liquefaction evaluation: one method for SPT and four methods for CPT data. Each method has different options that can be changed by the user. The options include Fines Correction, Hammer Type for SPT test, and Average Grain Size (D_{50}) for CPT.

The liquefaction analysis used the standard penetration (SPT) N-values recoded on the logs for the existing testing boring and monitoring wells MW-1605 and 1606. The liquefaction analysis has been performed for $N1_{60}$-values recorded in the upper 100 feet although the "RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities" (U.S.EPA, 1995) states that liquefaction is generally not likely to occur more than 50 feet below the ground surface. At the BAP Complex, groundwater is at 27 to 30 feet below the ground surface.

The results of the liquefaction analysis are summarized in Table 2 and Figure 7 and Figure 8. The detail of the analysis is included in Appendix C. The analysis shows that liquefaction is unlikely for the embankment and the foundations soils during the assumed PGA.

Table 2 Summary of Supplemental Liquefaction Potential Results

Section	Minimum Factor of Safety	Required Minimum Factor of Safety	Notes
B-1605	>1.2	1.20	None
B-1606	>1.2	1.20	None

5.3 Seismic Induced Permanent Displacement

The computer program LiquefyPro developed by developed by CivilTech Software Company was used to predict the likely magnitude of seismically-induced permanent displacements. LiquefyPro performs numerical double integration of the HEA values that are in excess of the yield acceleration values.

LiquefyPro divides the soil deposit into very thin layers and calculates the settlement for each layer. The calculations are divided into two parts, dry soil settlement and saturated soil settlement. The soil above the groundwater table is referred to as dry soil and soil below the groundwater table is referred to as saturated soil. The total settlement at a certain depth is the sum of the settlements of the saturated and dry soil. The total settlement is presented in the graphical report as a cumulative settlement curve versus depth. LiquefyPro gives settlement in both liquefied and non-liquefied zones.

The results of the permanent displacement analyses using LiquefyPro are presented graphically in Figure 7 and Figure 8. The figures indicate that the seismic induced permanent displacement are very small and range from 0 to 0.01 feet (0 to 0.12 inches).

5.4 SEISMIC SLOPE STABILITY

As a part of the factor of Structural integrity criteria assessment part of the CCR Rule (CFR §257.7 e), Terracon Inc. conducted seismic slope stability analysis in 2016 for the worst section of the bottom ash pond which is the outer dike. Factor of safety of 1.21 and 2.14 were calculated for worst case section shown in Figure 9 Figure 10 for the upstream slopes and downstream slopes, respectively. The figures show the geometry of the worst case section along with their material properties for the various soil layers, the projected slip failure, and the resulting factor of safety.

5.5 OVER TOPPING OF CREST

The west bottom ash pond is comprised of diked embankment to the west and between its respective waste water pond and adjacent east bottom ash pond that directs storm water away from the impoundment and limits runoff to that which falls directly onto the water surface. The land area to the north is an open field area that is not graded toward the Bottom Ash Complex. The east bottom ash pond has a small 13 acre catchment area that will drain into the pond. Flow into the west bottom ash pond was modeled as the pumped influent from the plant (77 ac-ft) and from the storm event (48 ac-ft) and discharged through the pond complex to the Ohio River.

The Bottom Ash Pond Complex has been determined to be a Low Hazard potential CCR impoundment. Based on this hazard classification, the design flood as determined by section 257.82(a)(3) to be the 100 year storm event that would incur 7.23 inches of precipitation in a 24 hour period. Terracon, 2015 conducted hydraulic and hydrogeologic study in which the site was modeled, however, using a greater storm (1,000-year: 10.3 inches of precipitation in 24 hrs) event to provide a more conservative analysis.

The following table provides the maximum inflows, outflows and flood elevations for the west bottom ash pond.

West Bottom Ash Pond*	
Storm Event	1000 yr.
Peak Inflow	470 cfs
Peak Outflow	35 cfs
Maximum Pool Elevation	395 ft.
Crest Elevation	399 ft.

*Reference: Terracon 2015,"Hydrologic and Hydraulic Analysis Report, Rockport Plant Bottom Ash Pond Complex, Rockport Indiana", Terracon Project No. N4155126

It can be concluded from the above results that the Bottom Ash Pond Complex has adequate hydrologic and hydraulic capacity to collect and control the peak discharge resulting from the 1000-year inflow design flood and therefore the overtopping of the crest is not anticipated.

5.6 LINER

The Ponds are CCR surface impoundments that are not equipped with a liner; therefore, this demonstration is not applicable.

5.7 LEACHATE COLLECTION AND REMOVAL SYSTEMS

The Ponds are CCR surface impoundments that are not equipped with a leachate collection and removal systems; therefore, this demonstration is not applicable.

5.8 SURFACE WATER CONTROL SYSTEMS

The surface water control structures were constructed in the late 70s and early 80s for the 2-unit operating plant with a total capacity of approximately 2,600 MW. The structures reviewed in this demonstration are all surface water control units facilitating water flow into and from the bottom ash ponds to the clear water ponds.

The components included in the demonstration can be classified into two groups:

• Group 1: components subjected to lateral loading due to the quakes used for transferring water from bottom ash ponds to waste water ponds including units used to dewater the BA ponds. The components are:

1. Energy Dissipater structure (EDS - 2 nos.) - approximately 8 plant pipes of 8 - 10 inch diameter pipes discharging into this structure and then transported into the BA pond through the Energy Dissipater troughs/Pond Discharge Inlet Chutes. EDSs are of concrete with steel dissipation flaps.
2. Energy Dissipater troughs/Pond Discharge Inlet Chutes (EDT)- These are concrete structures partially open at the top and partially covered by yellow steel boxes called Discharge Chute Covers.
3. Skimmers (SKM)- Timber structures surrounding the waste water discharge chute.
4. Waste water Discharge shaft (WWDS)- a steel and concrete prismoidal structure for routing waste water into the waste water discharge pipe.

• Group 2: Waste Water Discharge Pipe (WWDP)- Two buried 48 inch (one fiberglass and the other HDPE) pipes that transfer water under the dikes. Because they are buried they are affected by seismic waves and ground displacements.

Details of the analysis and are included in APPENDIX D. Appendix D contains the relevant calculations for the structures with the assumption that the dike stability against any seismic failure including liquefaction can be concluded. With this calculation results, the dike has been found stable. Therefore, the assumption is no more a restraint to use this calculation. The conclusion of the presented analysis indicated that

1. Based on a typical configuration, the seismic analyses of the structures are judged to meet local seismic requirements.

6 SUMMARY AND CONCLUSIONS

The Bottom Ash Pond Complex is a surface impoundment for storing CCR. The Bottom Ash Ponds within the complex are used for primary settling and storage of bottom ash. The Bottom Ash Pond Complex is located in an area having a two (2%) or greater probability that the maximum expected horizontal acceleration, expressed as a percentage of the earth's gravitational pull (g) of 0.1487 g in 50 years, which is in excess of the 0.10 g maximum horizontal acceleration in lithified earth material. Therefore, a demonstration that all structural components including liners, leachate collection and removal systems, and surface water control systems, are designed to resist the maximum horizontal acceleration in lithified earth material for the site was conducted per the requirements of CFR§257.63 – Seismic Impact Zones.

Based on the analysis conducted in this report, all structural components including liners, leachate collection and removal systems, and surface water control systems, are designed to resist the maximum horizontal acceleration and the Bottom Ash Pond Complex meets the requirements of §257.63 – Seismic Impact Zones.

7 REFERENCES

USEPA, 2015. 40 CFR Parts 257 and 261, Hazardous and Solid Waste Management System; Disposal of Coal Combustion Residuals from Electric Utilities; Final Rule. April 17, 2015. 201 pp.

Site-Specific Seismic hazard analysis for AEP power plant site, Rockport, Indiana URS corporation (currently AECOM) (2012).

Terracon 2015,"Hydrologic and Hydraulic Analysis Report, Rockport Plant Bottom Ash Pond Complex, Rockport Indiana", Terracon Project No. N4155126

Terracon, 2016. Geotechnical Engineering Report, AEP Rockport Bottom Ash Complex Professional Engineering Certification.

Figures

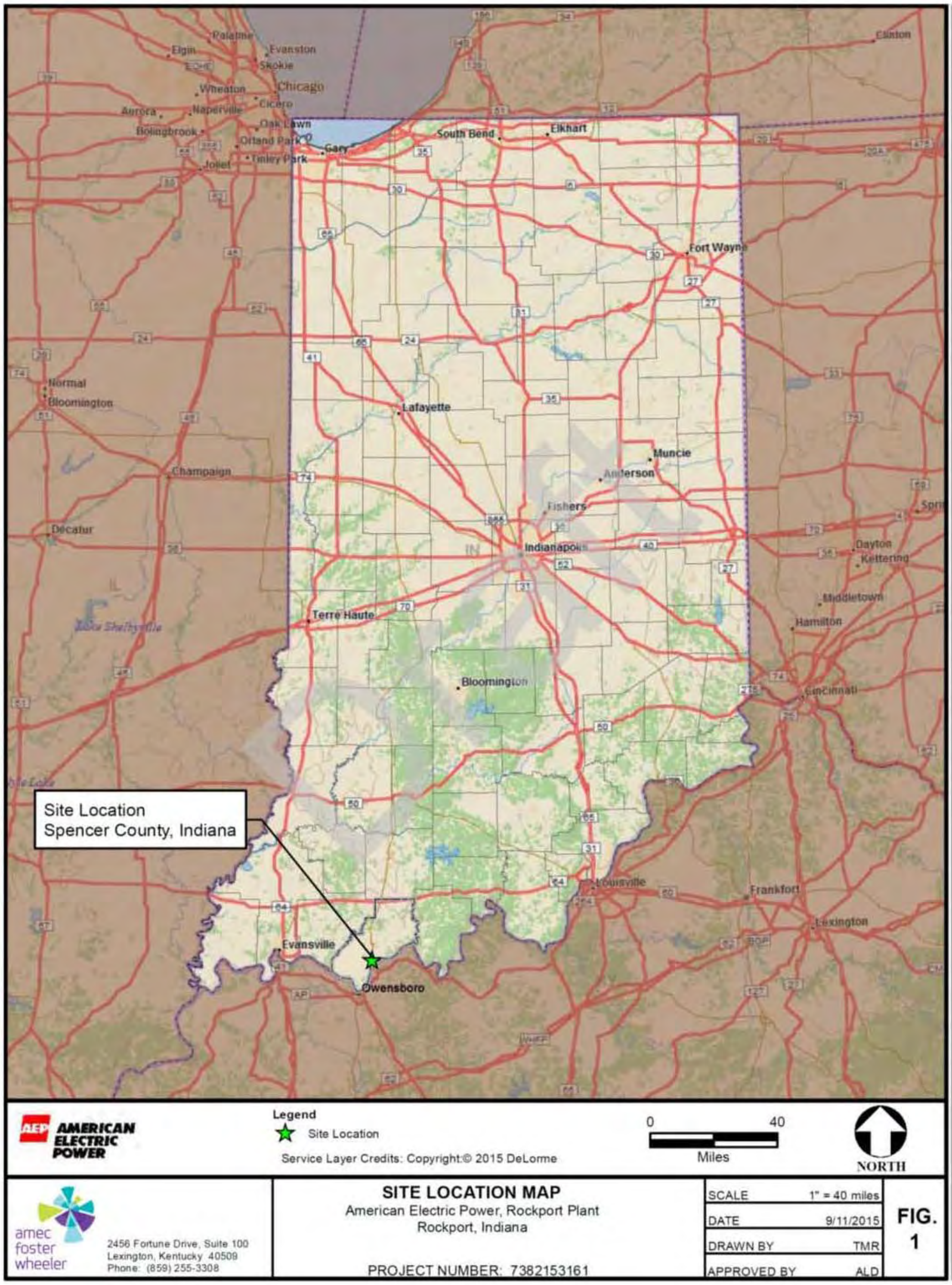

Figure 1 Rockport Power Station's Bottom Ash pond Complex Location Map

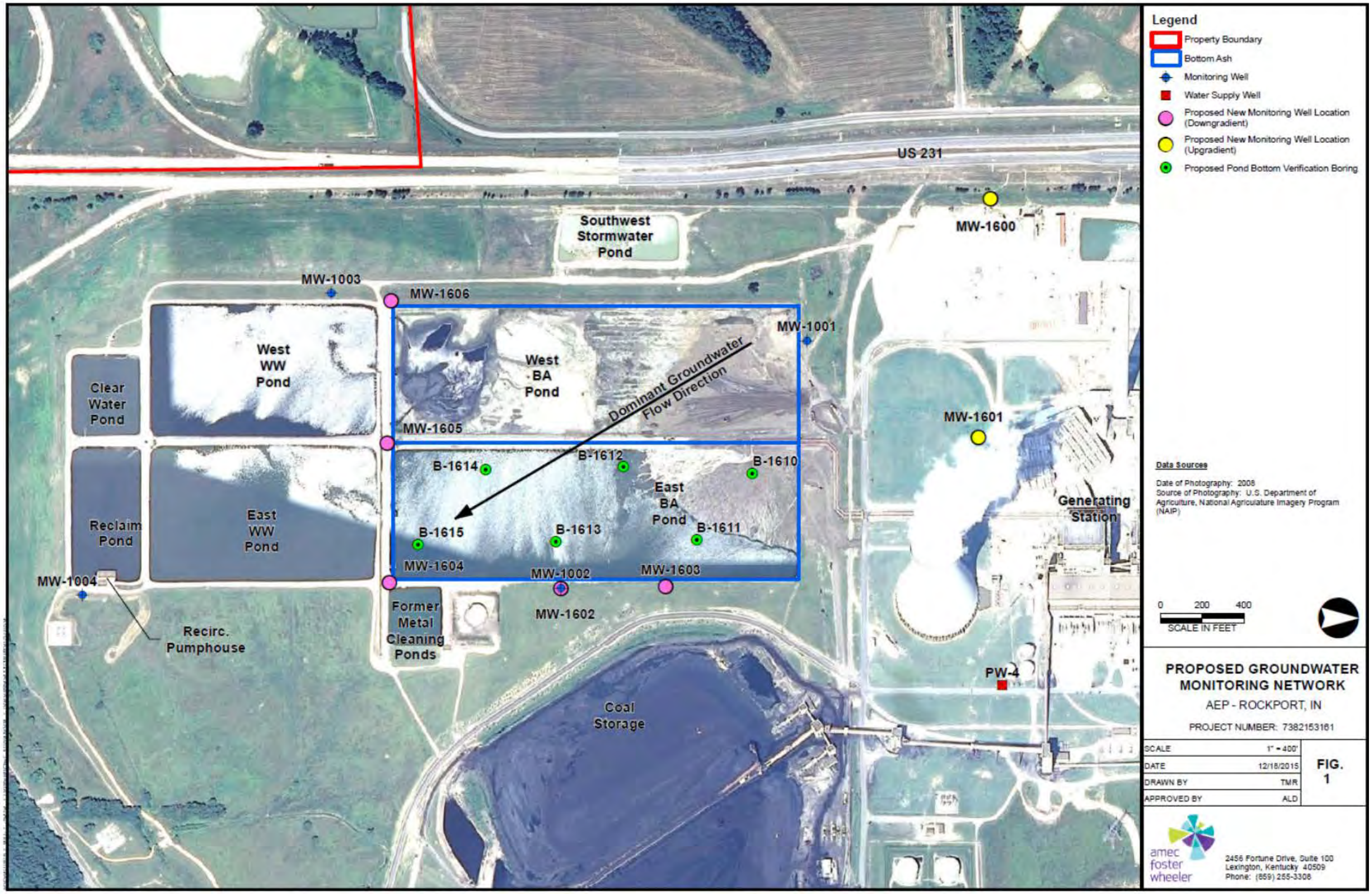

Figure 2 Rockport Power Station's BAP Plan View (Includes Borings location)

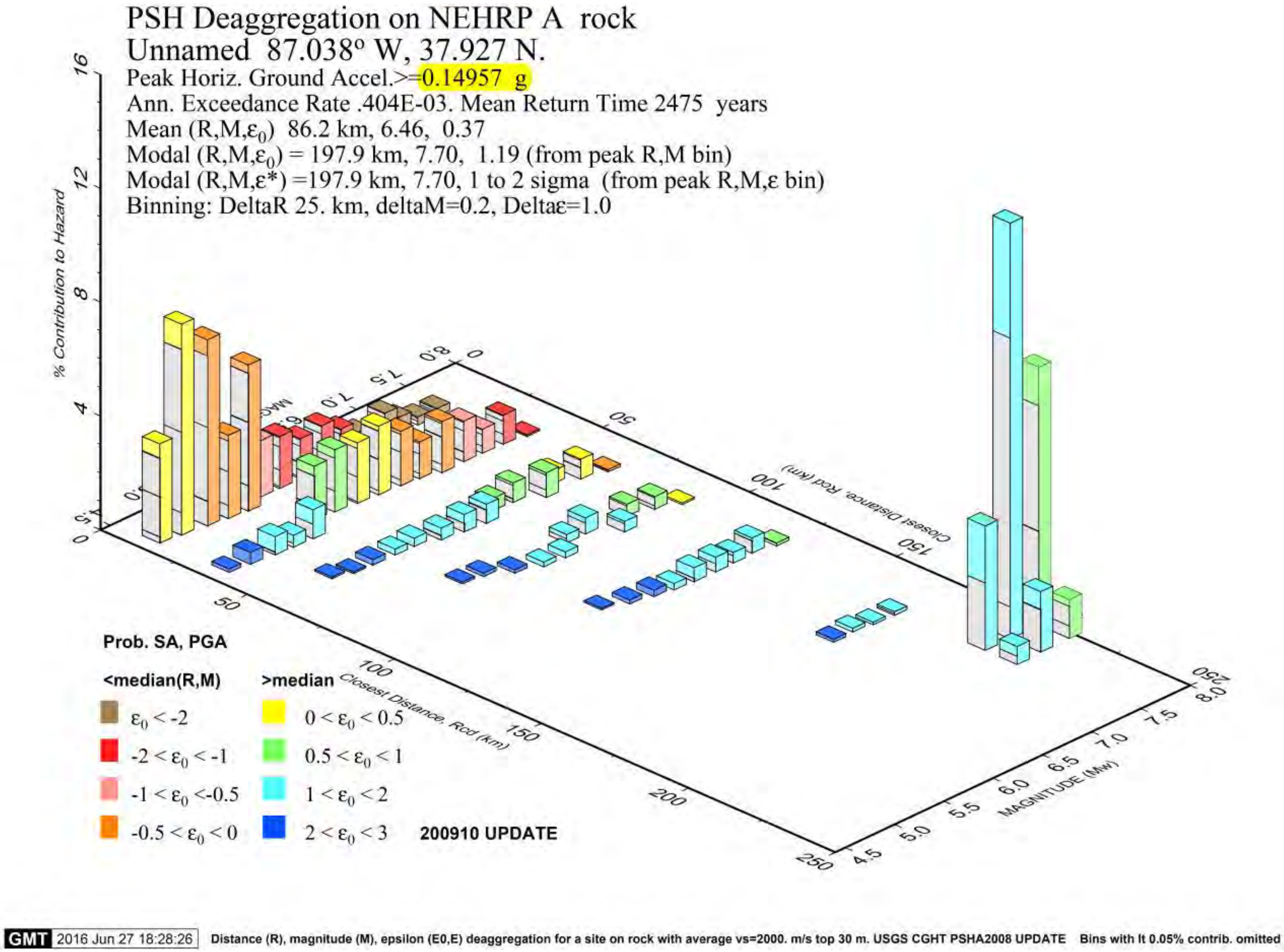

Figure 3 Maximum expected Earthquake Magnitude and horizontal acceleration based on U.S. Geological Survey Web Site

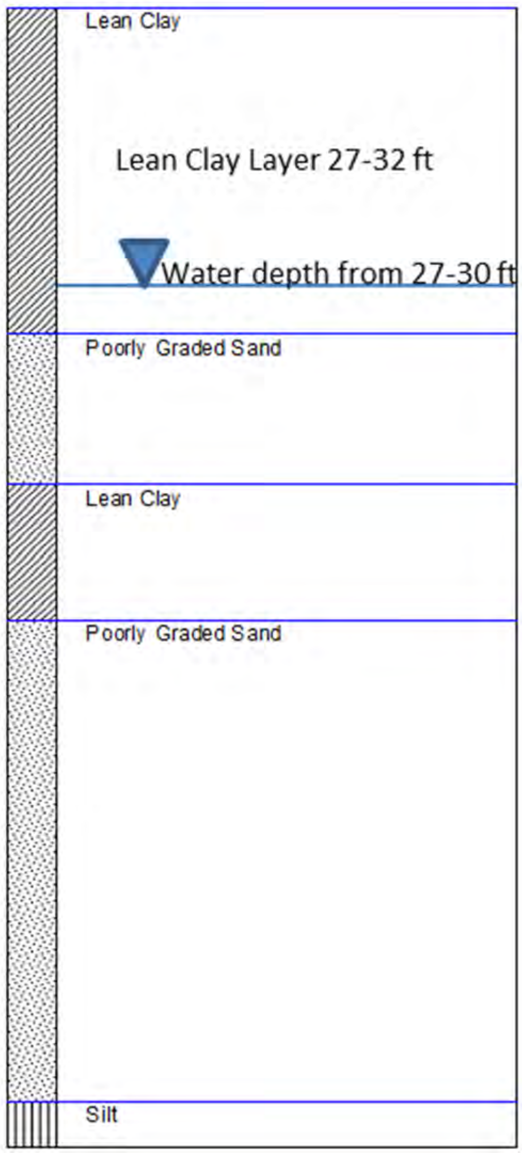

Figure 4 Soil Profile Interpreted from the Two Borings.

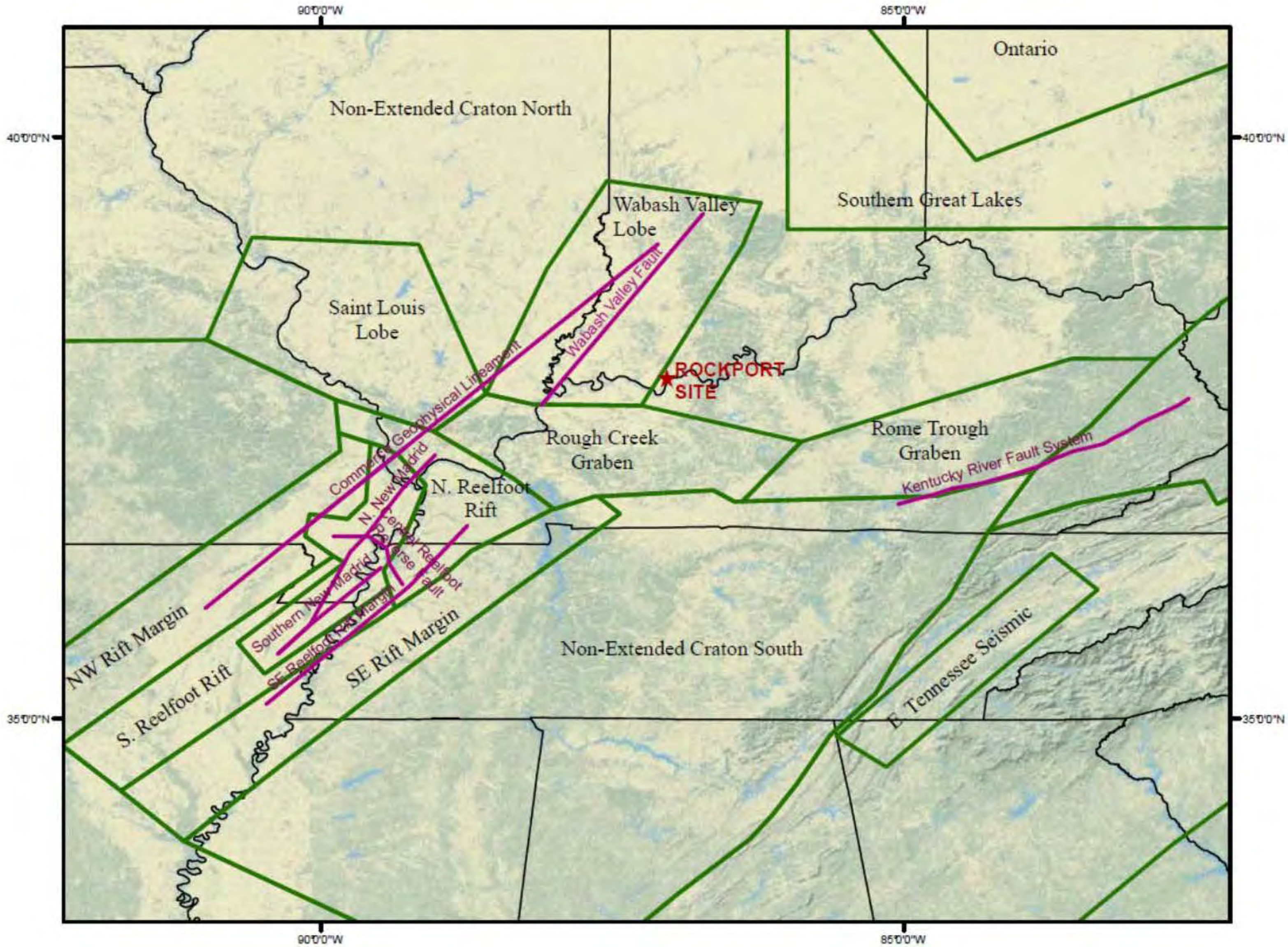

Figure 5 Regional Faults Location Map

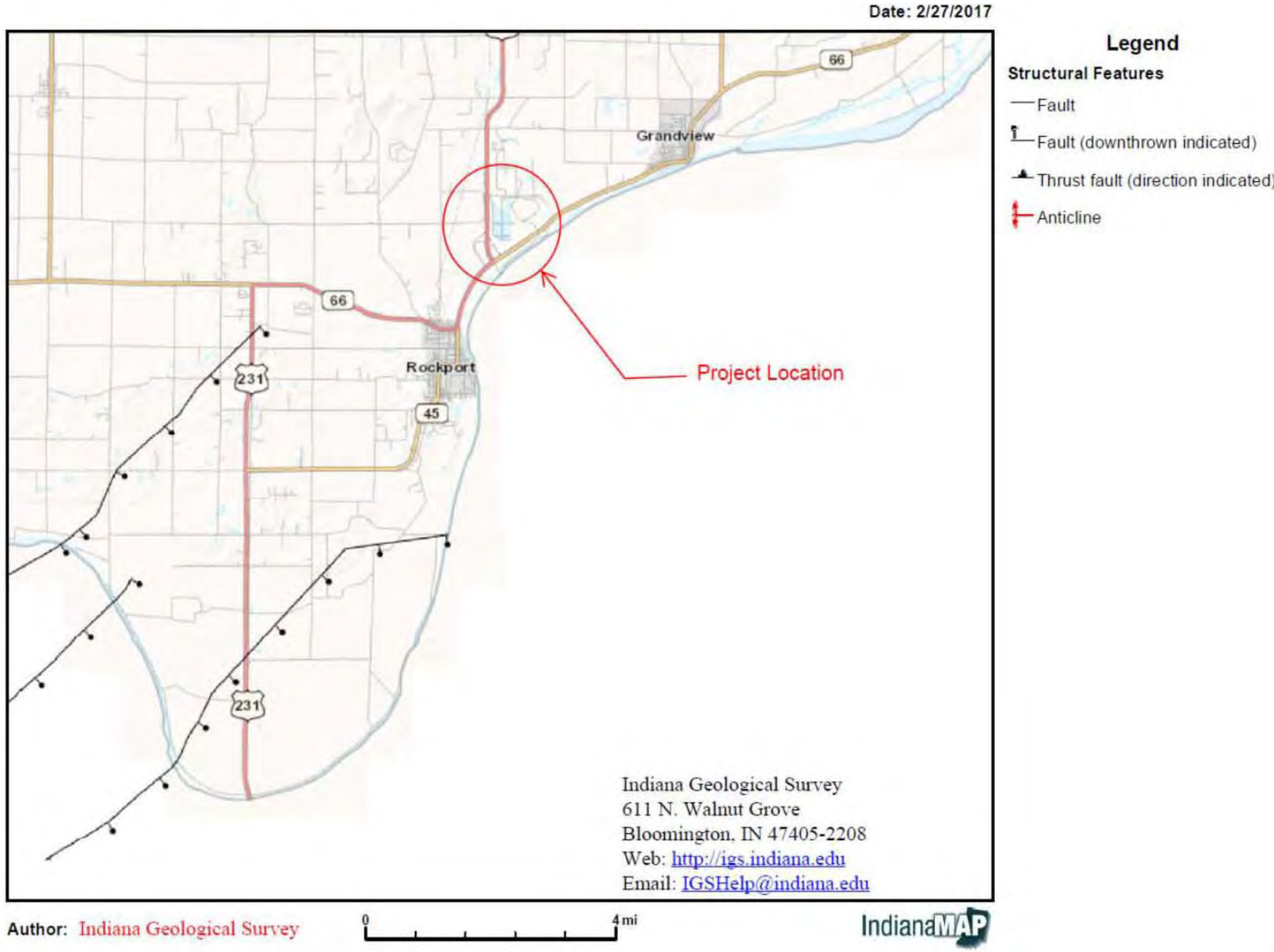

Figure 6 Local Faults Location Map

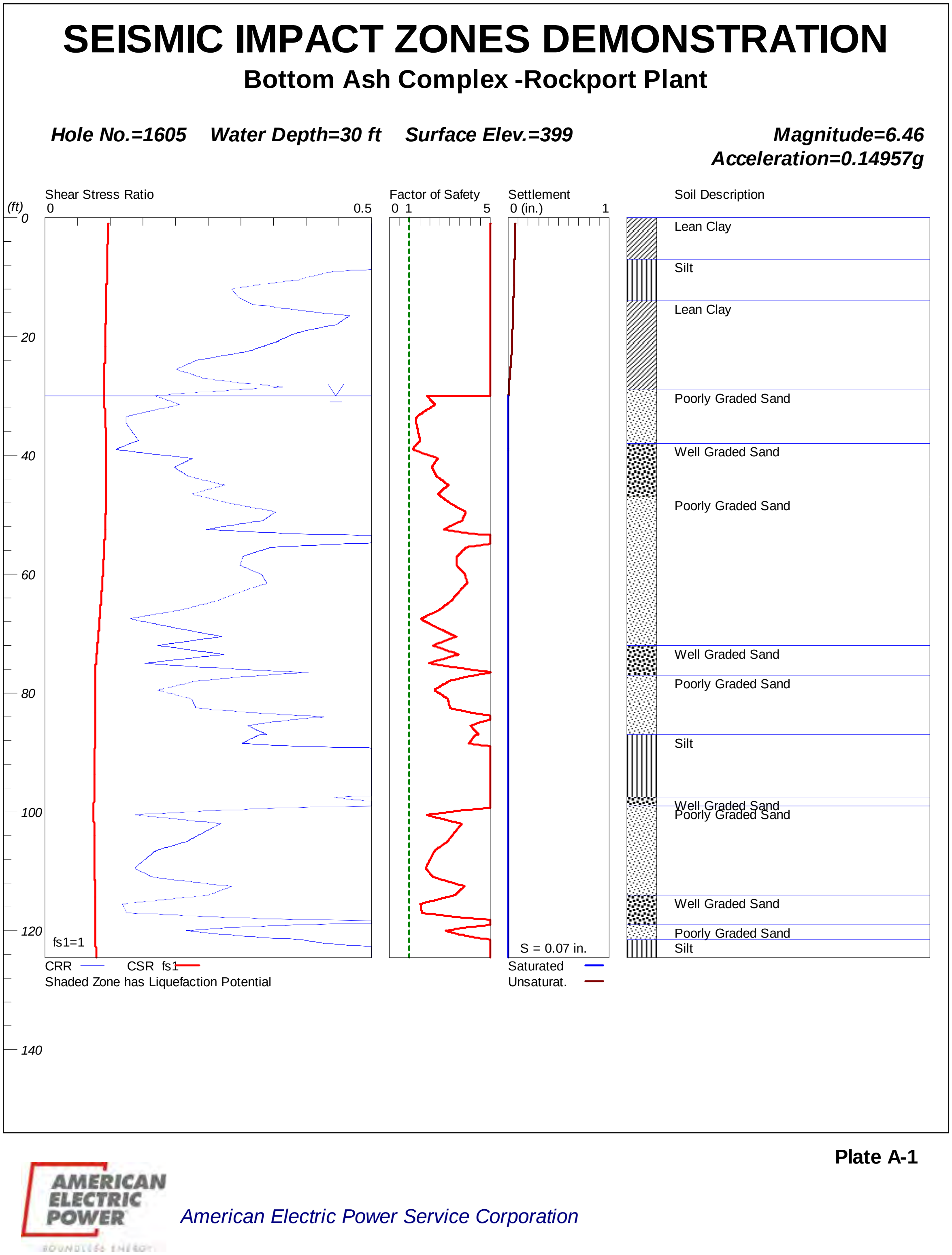

Figure 7 Liquefaction Analysis Results for B-1605Location

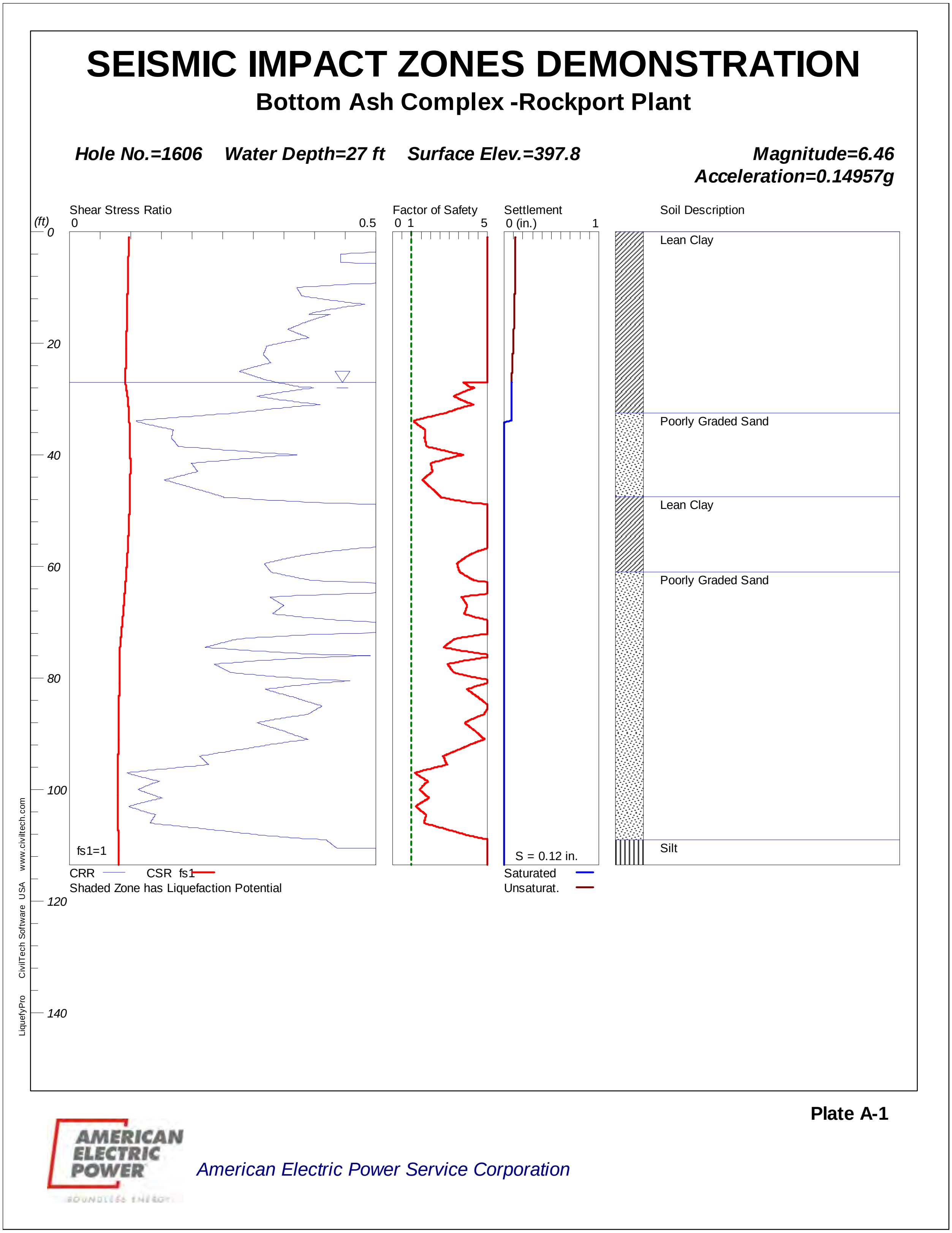

Figure 8 Liquefaction Analysis Results for B-1606Location

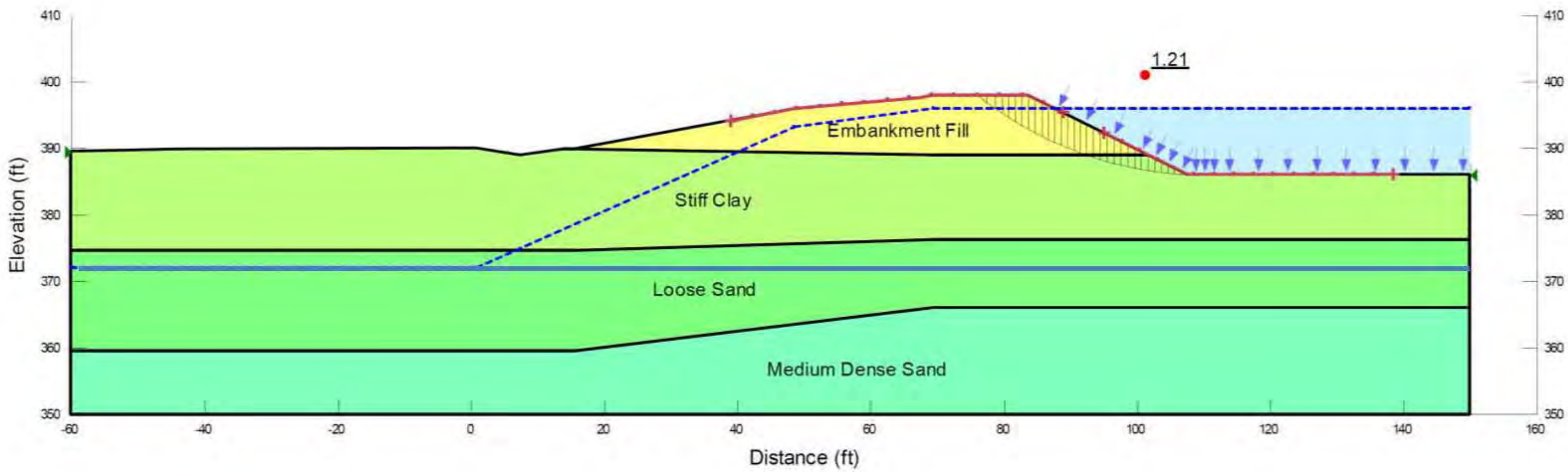

Figure 9 Results of Seismic Stability Analysis (Upstream)

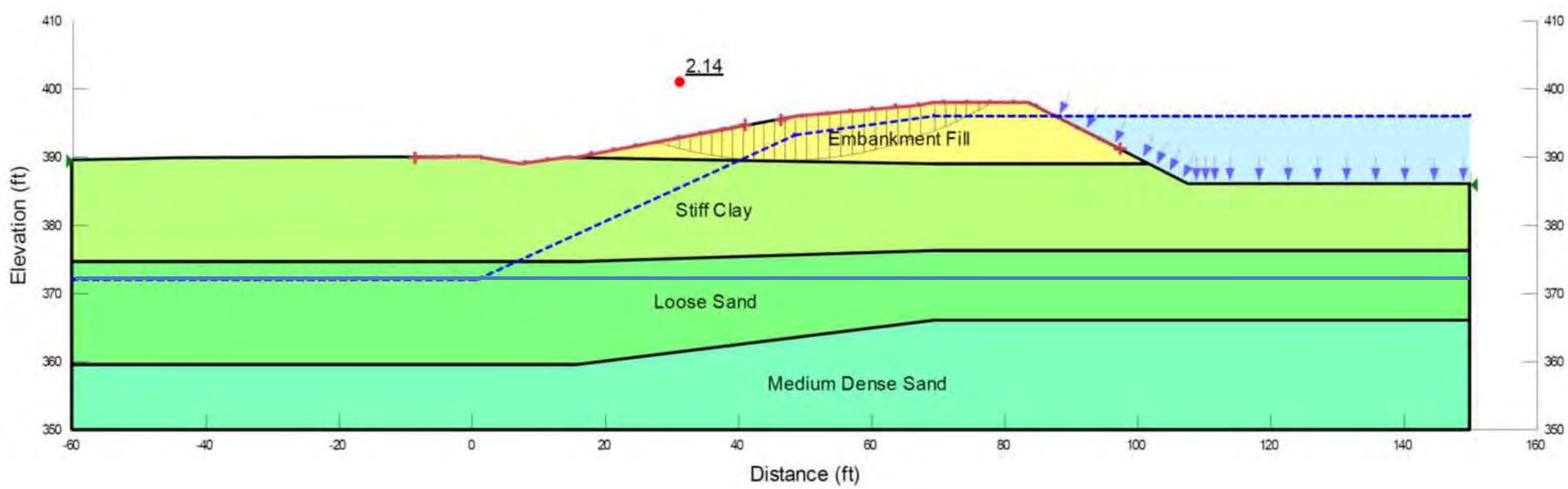

Figure 10 Results of Seismic Stability Analysis (Downstream)

APPENDICIES

APPENDIX A : Excerpts from the SITE-SPECIFIC SEISMIC HAZARD ANALYSIS

REPORT

SITE-SPECIFIC SEISMIC HAZARD ANALYSIS FOR AEP POWER PLANT SITE, ROCKPORT, INDIANA

Submitted to:

America Electric Power
1 Riverside Plaza
Columbus, Ohio 43215-2373

Prepared by:

URS Corporation
1501 4th Avenue, Suite 1400
Seattle, Washington 98101

URS Job No.: 13814835

March 12, 2012

URS performed a site-specific seismic hazard analysis for the American Electric Power (AEP) power plant site in Rockport, Indiana. The coordinates of the site are 37º55'32" N latitude and 87º02'02" W longitude. The objective of the study was to compute the design earthquake response spectrum for the site per the requirements in Chapter 21 of the ASCE 7-05 standard, which is incorporated by reference in the 2006 International Building Code (IBC). The study also meets the requirements of the Indiana State Building Code, which amends certain sections of the IBC.

To obtain the design earthquake response spectrum for the site, URS first conducted a probabilistic seismic hazard analysis (PSHA) to compute the 5% damped, horizontal component response spectrum corresponding to the Maximum Considered Earthquake (MCE). This spectrum pertained to a generic hard rock site condition (Site Class A, as defined in Chapter 20 of ASCE 7-05). The spectrum was then adjusted for the actual Site Class D site condition using the site coefficients in Section 11.4 of ASCE 7-05 and then converted to the design earthquake response spectrum according the provisions in Section 21.3 of the standard.

This report is organized as follows. Section 2.0 provides an overview of the PSHA methodology, while Section 3.0 summarizes the seismotectonic setting and historical seismicity of the site region. Sections 4.0 and 5.0 present, respectively, the inputs and results of the PSHA. Section 6.0 provides the determination of the site-specific design earthquake response spectrum. References are provided in Section 7.0 followed by the tables and figures.

5.1 COMPARISON WITH USGS NATIONAL HAZARD MAPS

In 1996, the USGS released a "landmark" set of National Hazard Maps for earthquake ground shaking, which was a significant improvement from previous maps they had developed (Frankel *et al.*, 1996). These maps were the result of the most comprehensive analyses of seismic sources and ground motion attenuation ever undertaken on a national scale. The maps are the basis for the NEHRP Maximum Considered Earthquake maps, which are used in the International Building Code. The maps are for NEHRP site class B/C (firm rock) and thus are not appropriate for the hard rock site conditions that are generally prevalent in the CEUS. The ground motions

on firm rock, however, can be adjusted to hard rock using adjustment factors developed by David Boore (Frankel *et al*., 1996).

For a 2,475-year return period (2% exceedance probability in 50 years), the updated 2008 National Hazard Maps indicate a firm rock PGA of 0.21 g for the site (Petersen *et al*., 2008). This value adjusted for hard rock is 0.14 g using an adjustment factor of 1.52 for PGA (Frankel *et al*., 1996). The site-specific PGA computed in this study for a 2,475-year return period is 0.13 g, very comparable to the USGS mapped value.

APPENDIX B : Soil boring logs along with soil classification sheets

AMERICAN ELECTRIC POWER SERVICE CORPORATION

AEP CIVIL ENGINEERING LABORATORY

LOG OF BORING

JOB NUMBER 42393125-01

COMPANY	INDIANA MICHIGAN POWER COMPANY	BORING NO.	MW-1605D
		DATE	4/27/16
		SHEET	1 OF 6
PROJECT	ROCKPORT PLANT	BORING START	2/3/16
		BORING FINISH	2/3/16
COORDINATES	N 151,478.9 E 513,537.1	PIEZOMETER TYPE	
		WELL TYPE	OW
GROUND ELEVATION	400.4	HGT. RISER ABOVE GROUND	3.36
SYSTEM	State Plane using NAD27/29	DIA	2.0
		DEPTH TO TOP OF WELL SCREEN	114.6
		BOTTOM	124.22
		WELL DEVELOPMENT	YES
		BACKFILL	
		FIELD PARTY	ZLR / REB
		RIG	D-50

Water Level, ft	▽	▼	▼
TIME			
DATE			

SAMPLE NUMBER	SAMPLE	SAMPLE DEPTH IN FEET FROM	TO	STANDARD PENETRATION RESISTANCE BLOWS / 6"	TOTAL LENGTH RECOVERY	RQD %	DEPTH IN FEET	GRAPHIC LOG	U S C S	SOIL / ROCK IDENTIFICATION	WELL	DRILLER'S NOTES
1	SS	0.0	1.5	20-13-10	1.25					Gravel = 6 inches		
									CL	Silty clay, moderate yellowish brown 10R 5/4 and med l. grey N6 mottled, moist, v. stiff @ 1.5' hard @ 3' v. stiff		
2	SS	1.5	3.0	5-15-18	1.25							
3	SS	3.0	4.5	7-9-15	1.41							
4	SS	4.5	6.0	11-12-14	1.5		5					
5	SS	6.0	7.5	4-8-11	1.41							
6	SS	7.5	9.0	3-6-11	1.33				ML	Clayey silt, medium grey N5, moist, med. dense, w/mod. yellowish brown 10R 5/4 silty clay mottled		
7	SS	9.0	10.5	3-4-7	1.41		10		CL	Silty clay, mod. yellowish brown 10R 5/4, moist, stiff, w/med. grey N5 clayey silt mottled		
8	SS	10.5	12.0	3-4-6	1.5							
9	SS	12.0	13.5	2-2-4	1.5				CH	Fat to lean clay, med. l. grey N6, moist, firm		
10	SS	13.5	15.0	2-2-5	1.41							
							15		CL ML	Silty clay, mod. reddish brown 10R 4/6 w/med. l. grey N6 fat clay heavily mottled, moist, firm @ 15' stiff @ 15.5' l" shale fragment, angular @ 18' very silty @ 20' trace to some pale yellowish brown 10YR 6/2 silt		
11	SS	15.0	16.5	2-4-5	1.5							
12	SS	16.5	18.0	3-5-9	1.5							
13	SS	18.0	19.5	3-6-8	1.41							
14	SS	19.5	21.0	3-5-7	1.41							

TYPE OF CASING USED

	NQ-2 ROCK CORE	
	6" x 3.25 HSA	
	9" x 6.25 HSA	
	HW CASING ADVANCER	4"
	NW CASING	3"
	SW CASING	6"
	AIR HAMMER	8"

Continued Next Page

PIEZOMETER TYPE: PT = OPEN TUBE POROUS TIP, SS = OPEN TUBE SLOTTED SCREEN, G = GEONOR, P = PNEUMATIC

WELL TYPE: OW = OPEN TUBE SLOTTED SCREEN, GM = GEOMON

RECORDER AMEC FOSTER WHEELER

JOB NUMBER 42393125-01

COMPANY INDIANA MICHIGAN POWER COMPANY | BORING NO. MW-1605D | DATE 4/27/16 | SHEET 2 OF 6

PROJECT ROCKPORT PLANT | BORING START 2/3/16 | BORING FINISH 2/3/16

SAMPLE NUMBER	SAMPLE	SAMPLE DEPTH IN FEET FROM	SAMPLE DEPTH IN FEET TO	STANDARD PENETRATION RESISTANCE BLOWS / 6"	TOTAL LENGTH RECOVERY	RQD %
15	SS	21.0	22.5	3-4-7	1.5	
16	SS	22.5	24.0	4-4-5	1.5	
17	SS	24.0	25.5	1-1-3	1.5	
18	SS	25.5	27.0	1-1-1	1.5	
19	SS	27.0	28.5	2-1-4	1.5	
20	SS	28.5	30.0	5-6-7	1.33	
21	SS	30.0	31.5	3-5-7	1.25	
22	SS	31.5	33.0	5-7-8	1.5	
23	SS	33.0	34.5	3-3-6	1.41	
24	SS	34.5	36.0	2-4-5	1.5	
25	SS	36.0	37.5	2-4-6	1.33	
26	SS	37.5	39.0	4-3-8	1.5	
27	SS	39.0	40.5	3-3-5	1.5	
28	SS	40.5	42.0	11-8-10	1.25	
29	SS	42.0	43.5	4-5-11	1.5	
30	SS	43.5	45.0	8-9-9	1.16	
31	SS	45.0	46.5	6-9-14	1.5	

DEPTH IN FEET: 25, 30, 35, 40, 45

USCS	SOIL / ROCK IDENTIFICATION
ML	Clayey silt, pale yellowish brown 10YR 6/2, moist, med. dense, w/silty clay (prev. material), trace sand
SP	Poorly graded sand, v. fine to fine grained, l. brown 5YR 5/6, moist, loose @ 23.2' 2" clayey silt seam (prev. material)
ML	Clayey silt, pale yellowish brown 10YR 6/2, moist to wet, v. loose @ 25' 2" l. brown sand seam (prev. material) @ 26' 2" l. brown sand seam @ 26.4' 15" l. brown sand seam @ 26.8' l" l. brown sand seam @ 27' loose @ 28' 2" l. brown sand seam
SP	Poorly graded sand, fine grained, l. brown 5YR 5/6, moist, med. dense @ 30' d. yellowish orange 10YR 6/6 @ 31' 3" clayey silt seam (prev. material) @ 32.3' trace fine gravel and black silt @ 32.5' no fine gravel or silt @ 33' moist, loose @ 34.1' 2" clayey silt seam (prev. material) @ 34.5' moist to wet, water in spoon @ 34.9' 2.5' clayey silt seam (prev. material)
SW	Well graded sand, fine grained, l. brown 5YR 5/6, moist to wet, med. dense, w/fine gravel
SW	Well graded sand, coarse grained, grayish black N2, moist to wet, med. dense, trace fine gravel
SP	Poorly graded sand, v. fine grained, l. brown 5YR 5/6, moist to wet, med. dense
SW	Well graded sand, fine to med. grained, moderate yellowish brown 10YR 5/4, moist to wet, loose @ 40.5' med. dense @ 41' 1.5" shale seam w/clay
SP	Poorly graded sand, v. fine to fine grained, mod. yellowish brown 10YR 5/4, moist to wet, med. dense
SW	Well graded sand, med. grained, mod. reddish brown 10R 4/6, moist to wet, med. dense @ 44' med. to coarse grained
SP	Poorly graded sand, fine grained, mod. yellowish

WELL | DRILLER'S NOTES

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JOB NUMBER 42393125-01

COMPANY INDIANA MICHIGAN POWER COMPANY BORING NO. MW-1605D DATE 4/27/16 SHEET 3 OF 6

PROJECT ROCKPORT PLANT BORING START 2/3/16 BORING FINISH 2/3/16

SAMPLE NUMBER	SAMPLE	SAMPLE DEPTH IN FEET FROM	TO	STANDARD PENETRATION RESISTANCE BLOWS / 6"	TOTAL LENGTH RECOVERY	RQD %
32	SS	46.5	48.0	6-8-11	1.5	
33	SS	48.0	49.5	6-10-14	1.5	
34	SS	49.5	51.0	8-12-18	1.33	
35	SS	51.0	52.5	8-11-18	1.41	
36	SS	52.5	54.0	8-9-13	.91	
37	SS	54.0	55.5	11-20-26	1.25	
38	SS	55.5	57.0	10-15-16	1.5	
39	SS	57.0	58.5	6-12-16	1.33	
40	SS	58.5	60.0	7-10-18	1.33	
41	SS	60.0	61.5	8-9-12	1.33	
42	SS	61.5	63.0	10-13-19	1.25	
43	SS	63.0	64.5	9-11-18	1.33	
44	SS	64.5	66.0	9-11-15	1.08	
45	SS	66.0	67.5	7-8-13	1.41	
46	SS	67.5	69.0	5-5-8	1.5	
47	SS	69.0	70.5	6-8-12	1.5	
48	SS	70.5	72.0	0-12-16	1.5	

DEPTH IN FEET: 50, 55, 60, 65, 70

USCS	SOIL / ROCK IDENTIFICATION	WELL	DRILLER'S NOTES
SW	brown 10YR 5/4, moist to wet, mod. dense, some fine gravel		
SP	Well graded sand, med. to coarse grained, mod. reddish brown 10R 4/6, moist to wet, med. dense, trace fine gravel Poorly graded sand, fine grained, mod. yellowish brown 10YR 5/4, moist to wet, med. dense, trace fine gravel @ 48' w/fine gravel, trace coarse gravel @ 49.5' no coarse gravel		
SW	Well graded sand, med. to coarse grained, mod. reddish brown 10R 4/6, moist to wet, mod. dense, trace fine gravel		
SP	Poorly graded sand, fine grained, mod. yellowish brown 10YR 5/4, moist to wet, mod. dense, trace fine gravel @ 54' no fine gravel, dense @ 57' wet, mod. dense @ 60' dense @ 63' mod. dense		
SW	Well graded sand, med. to coarse grained, mod. yellowish brown 10YR 5/4, moist to wet, mod. dense, trace black silt		
SP	Poorly graded sand, fine grained, mod. yellowish brown 10YR 5/4, moist to wet, mod. dense @ 68.5' trace fine gravel, trace coal fragments @ 70' no fine gravel, no coal fragments @ 70.9' trace fine gravel @ 71.6' no fine gravel, wet		

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AMERICAN ELECTRIC POWER SERVICE CORPORATION
AEP CIVIL ENGINEERING LABORATORY
LOG OF BORING

JOB NUMBER **42393125-01**

COMPANY **INDIANA MICHIGAN POWER COMPANY** BORING NO. **MW-1605D** DATE **4/27/16** SHEET **4** OF **6**

PROJECT **ROCKPORT PLANT** BORING START **2/3/16** BORING FINISH **2/3/16**

SAMPLE NUMBER	SAMPLE	SAMPLE DEPTH IN FEET FROM	TO	STANDARD PENETRATION RESISTANCE BLOWS / 6"	TOTAL LENGTH RECOVERY	RQD %	DEPTH IN FEET	GRAPHIC LOG	U S C S	SOIL / ROCK IDENTIFICATION	WELL	DRILLER'S NOTES
49	SS	72.0	73.5	8-8-10	1.25				SW	Well graded sand, fine grained d. yellowish brown 10YR 4/2, moist to wet, mod. dense, trace fine gravel		
50	SS	73.5	75.0	9-12-17	1.41					@ 73.5' w/fine gravel, trace coarse gravel		
51	SS	75.0	76.5	8-7-9	1.5		75					
									SW	Well graded sand, coarse grained, brownish grey 5YR 4/1, moist to wet, mod. dense, w/fine gravel, trace coarse gravel		
52	SS	76.5	78.0	10-15-25	1.5							
									SP	Poorly graded sand, fine grained, pale yellowish brown 10YR 6/2, wet, dense, trace fine gravel @ 78' mod. dense @ 81' v. fine to fine grained @ 82.5' no fine gravel @ 84' dense @ 85' 2" shale fragment @ 85.2' v. fine grained @ 85.5' 3.5" shale fragment @ 87' fine grained, d. yellowish brown 10YR 4/2 @ 88.5' v. fine grained, mod. dense		
53	SS	78.0	79.5	7-13-12	1.33							
54	SS	79.5	81.0	5-7-12	1.5		80					
55	SS	81.0	82.5	6-12-13	1.5							
56	SS	82.5	84.0	8-10-16	1.41							
57	SS	84.0	85.5	10-21-22	1.41		85					
58	SS	85.5	87.0	14-21-14	.5							
59	SS	87.0	88.5	6-13-25	1.41							
60	SS	88.5	90.0	8-9-9	1.16							
									ML	Clayey silt, med. l. grey N6, moist to wet, mod. dense		
61	SS	90.0	91.5	15-24-7	1.41		90					
									SP	Poorly graded sand, fine grained, d. yellowish brown 10YR 4/2, moist, dense		
62	SS	91.5	93.0	7-21-28	1.5				ML	Clayey silt, med. l. grey N6, moist to wet, dense		
									SW	Well graded sand, coarse grained, med. grey N5, w/fine gravel, some coarse gravel		
63	SS	93.0	94.5	14-18-21	1.5				ML	Clayey silt, med. l. grey N6, moist to wet, dense		
									SW	Well graded sand, fine grained, med. grey N5, moist to wet, dense, w/fine gravel		
64	SS	94.5	96.0	12-17-25	1.5		95		ML	Clayey silt, med. l. grey N6, moist to wet, dense		
									SW	Well graded sand, coarse grained, med. grey N5, moist to wet, dense, w/fine gravel @ 98.7' coal fragments		
65	SS	96.0	97.5	20-21-19	1.33							
66	SS	97.5	99.0	13-11-18	1.41							

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AMERICAN ELECTRIC POWER SERVICE CORPORATION
AEP CIVIL ENGINEERING LABORATORY
LOG OF BORING

JOB NUMBER 42393125-01

COMPANY INDIANA MICHIGAN POWER COMPANY BORING NO. MW-1605D DATE 4/27/16 SHEET 5 OF 6

PROJECT ROCKPORT PLANT BORING START 2/3/16 BORING FINISH 2/3/16

SAMPLE NUMBER	SAMPLE	SAMPLE DEPTH IN FEET FROM	TO	STANDARD PENETRATION RESISTANCE BLOWS / 6"	TOTAL LENGTH RECOVERY	RQD %	DEPTH IN FEET	GRAPHIC LOG	U S C S	SOIL / ROCK IDENTIFICATION	WELL	DRILLER'S NOTES
67	SS	99.0	100.5	15-22-28	1.5				SP	Poorly graded sand, v. fine to fine grained, pale yellowish brown 10YR 6/2, moist to wet, dense, w/fine gravel @ 100.5' no fine gravel, mod. dense @ 102' v. fine, dense @ 105' mod. dense @ 106' trace coal fragments @ 106.3' no coal fragments @ 109.5' moist @ 111' v. moist to wet @ 112.5' moist to wet, dense @ 113' trace fine gravel, trace coarse gravel @ 113.5' no fine gravel, no coarse gravel		
68	SS	100.5	102.0	8-8-9	1.5		100					
69	SS	102.0	103.5	10-16-18	1.5							
70	SS	103.5	105.0	9-13-18	1.41							
71	SS	105.0	106.5	8-12-16	1.5		105					
72	SS	106.5	108.0	6-9-13	1.5							
73	SS	108.0	109.5	7-8-12	1.25							
74	SS	109.5	111.0	6-8-10	1.41		110					
75	SS	111.0	112.5	5-10-12	1.25							
76	SS	112.5	114.0	6-11-27	1.33							
77	SS	114.0	115.5	13-21-13	1.25				SW	Well graded sand, med. to coarse grained, med. grey N5, moist to wet, dense, w/fine gravel, some coarse gavel @ 115.5' coarse grained, mod. dense, trace coarse gravel @ 118.5' v. dense		
78	SS	115.5	117.0	7-7-9	1.33		115					
79	SS	117.0	118.5	9-9-8	1.16							
80	SS	118.5	120.0	12-36-22	1.5				SP	Poorly graded sand, v. fine grained, med. l. grey N6, moist to wet, v. dense @ 120' med. dense, sl. moist @ 122' fine grained, w/fine gravel, dense @ 124.5' trace coarse gravel		
81	SS	120.0	121.5	10-11-19	1.41		120					
82	SS	121.5	123.0	12-20-29	1.5							
83	SS	123.0	124.5	14-16-19	1.5							

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AMERICAN ELECTRIC POWER SERVICE CORPORATION

AEP CIVIL ENGINEERING LABORATORY

LOG OF BORING

JOB NUMBER **42393125-01**

COMPANY **INDIANA MICHIGAN POWER COMPANY** BORING NO. **MW-1605D** DATE **4/27/16** SHEET **6** OF **6**

PROJECT **ROCKPORT PLANT** BORING START **2/3/16** BORING FINISH **2/3/16**

SAMPLE NUMBER	SAMPLE	SAMPLE DEPTH IN FEET FROM	TO	STANDARD PENETRATION RESISTANCE BLOWS / 6"	TOTAL LENGTH RECOVERY	RQD %	DEPTH IN FEET	GRAPHIC LOG	U S C S	SOIL / ROCK IDENTIFICATION	WELL	DRILLER'S NOTES
84	SS	124.5	126.0	18-12-25	1.5		125					
85	SS	126.0	127.5	17-28-50/5	1.5				ML	Clayey silt, l. grey N7, moist, hard, non-durable shale @ 126' flaky, dry to moist Spoon refusal @ 127.4' Auger refusal @127.5' (shale)		
86	SS	127.5	129.0	27-50/2	.66							

AMERICAN ELECTRIC POWER SERVICE CORPORATION

AEP CIVIL ENGINEERING LABORATORY

LOG OF BORING

JOB NUMBER **42393125-01**

COMPANY **INDIANA MICHIGAN POWER COMPANY** BORING NO. **MW-1606D** DATE **4/27/16** SHEET **1** OF **5**

PROJECT **ROCKPORT PLANT** BORING START **2/12/16** BORING FINISH **2/12/16**

COORDINATES **N 151,502.1 E 512,881.5** PIEZOMETER TYPE ____ WELL TYPE **OW**

GROUND ELEVATION **397.8** SYSTEM **State Plane using NAD27/29** HGT. RISER ABOVE GROUND **2.91** DIA **2.0**

DEPTH TO TOP OF WELL SCREEN **100.2** BOTTOM **109.82**

WELL DEVELOPMENT **YES** BACKFILL ____

FIELD PARTY **ZLR / REB** RIG **D-120**

Water Level, ft	▽	▼	▼
TIME			
DATE			

SAMPLE NUMBER	SAMPLE	SAMPLE DEPTH IN FEET FROM	TO	STANDARD PENETRATION RESISTANCE BLOWS / 6"	TOTAL LENGTH RECOVERY	RQD %	DEPTH IN FEET	GRAPHIC LOG	USCS	SOIL / ROCK IDENTIFICATION	WELL	DRILLER'S NOTES
1	SS	0.0	1.5	3-5-9	1.5					Crushed stone gravel (limestone)		
									CL	Lean clay, moderate yellowish brown 10YR 5/4, moist, trace fine grained sand, stiff		
2	SS	1.5	3.0	4-7-9	1.5					@ 1.5' as above, trace coarse grain sand and black decomposed organic staining @ 3' trace fine gravel		
3	SS	3.0	4.5	3-4-6	1.3							
4	SS	4.5	6.0	1-2-8	1.3		5					
5	SS	6.0	7.5	5-9-10	1.5				CL	Lean clay, pale yellow brown 10YR 6/2, moist, some light brown oxide staining @ 6.0' yellow brown and brown 10YR 5/4 @ 7.5' pale yellow brown 10YR 6/2, trace fine roots, trace fine grained sand		
6	SS	7.5	9.0	3-6-9	1.5				CL	Lean clay w/sand, dark yellow brown 10YR 4/2, moist, little fine grained sand		
7	SS	9.0	10.5	2-4-5	1.5		10		CL	Lean clay, light bluish gray 5B 7/1, moist, some brown oxide staining, trace coarse grained sand @ 12.5' as above, becomes moderate brown in color 5YR 4/4 @ 13.5' moderate yellow brown 10YR 5/4 and pale yellow brown 10YR 6/2) mottled @ 13.5' - 15' trace fine grained sand, trace fine gravel @ 19.5' mostly 10YR 6/2 in color		
8	SS	10.5	12.0	3-4-6	1.5							
9	SS	12.0	13.5	3-5-9	1.5							
10	SS	13.5	15.0	4-5-7	1.5		15					
11	SS	15.0	16.5	3-5-6	1.5							
12	SS	16.5	18.0	3-4-6	1.5							
13	SS	18.0	19.5	2-5-7	1.5							
14	SS	19.5	21.0	3-3-6	1.5							

TYPE OF CASING USED

	NQ-2 ROCK CORE	
	6" x 3.25 HSA	
	9" x 6.25 HSA	
	HW CASING ADVANCER	4"
	NW CASING	3"
	SW CASING	6"
	AIR HAMMER	8"

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PIEZOMETER TYPE: PT = OPEN TUBE POROUS TIP, SS = OPEN TUBE SLOTTED SCREEN, G = GEONOR, P = PNEUMATIC

WELL TYPE: OW = OPEN TUBE SLOTTED SCREEN, GM = GEOMON

RECORDER **AMEC FOSTER WHEELER**

AMERICAN ELECTRIC POWER SERVICE CORPORATION

AEP CIVIL ENGINEERING LABORATORY

LOG OF BORING

JOB NUMBER **42393125-01**

COMPANY **INDIANA MICHIGAN POWER COMPANY** BORING NO. **MW-1606D** DATE **4/27/16**

PROJECT **ROCKPORT PLANT** BORING START **2/12/16** BORING FINISH **2/12/16**

SAMPLE NUMBER	SAMPLE	SAMPLE DEPTH IN FEET FROM	TO	STANDARD PENETRATION RESISTANCE BLOWS / 6"	TOTAL LENGTH RECOVERY	RQD %	DEPTH IN FEET	GRAPHIC LOG	U S C S	SOIL / ROCK IDENTIFICATION	WELL	DRILLER'S NOTES
15	SS	21.0	22.5	3-4-5	1.5							
16	SS	22.5	24.0	2-4-6	1.5				CL ML	Silty clay, pale yellow brown 10YR 6/2, moist, trace to little fine grained sand		
17	SS	24.0	25.5	1-2-5	1.2		25		SP SM	Poorly graded sand w/silt, pale yellow brown 10YR 6/2, moist, fine to medium grained sand @ 24.9' 3" silt layer		
18	SS	25.5	27.0	2-4-6	1.5				CL	Lean clay, moderate yellowish brown 10YR 5/4, moist, few sandy layers <1" thick @ 28.3' SP-SM layer (~3" thick)		
19	SS	27.0	28.5	1-5-9	1.3							
20	SS	28.5	30.0	4-4-5	1.3				SP SM	Poorly graded sand w/silt, dark yellowish orange 10YR 6/6, wet, fine to medium grained sand, little coarse grained sand @ 31.5' trace fine gravel @ 34.5' trace fine gravel		
21	SS	30.0	31.5	5-7-8	1.5		30					
22	SS	31.5	33.0	3-3-4	1.1							
23	SS	33.0	34.5	1-2-5	0							
24	SS	34.5	36.0	3-4-8	.8		35					
25	SS	36.0	37.5	3-5-7	1.0							
26	SS	37.5	39.0	5-6-7	.9				SP	Poorly graded sand, dark yellowish orange 10YR 6/6, wet, fine to medium grained sand, trace to little coarse grained sand @ 37.5' trace gravel		
27	SS	39.0	40.5	4-7-20	1.2		40		SP SM	Poorly graded sand w/silt, dark yellowish orange 10YR 6/6, wet, fine to medium grained sand, trace coarse grained sand		
28	SS	40.5	42.0	7-7-8	1.1				SC	Clayey sand, moderate brown 5YR 3/4, wet, fine to medium grained sand		
29	SS	42.0	43.5	4-6-10	1.0				SP	Poorly graded sand, dark yellowish orange 10YR 6/6, wet, fine to medium grained sand, trace coarse grained sand & fine gravel @ 42.0' - 43.5' increase in coarse grained sand @ 45.2' - 45.5' color change to moderate brown 5YR 4/4 @ 46.5' increase in coarse grained sand, trace wood fragments (tree bark) @ 48' color change to pale yellowish brown 10YR		
30	SS	43.5	45.0	4-5-7	1.0							
31	SS	45.0	46.5	4-6-10	1.2		45					

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AMERICAN ELECTRIC POWER SERVICE CORPORATION

AEP CIVIL ENGINEERING LABORATORY

LOG OF BORING

JOB NUMBER 42393125-01

COMPANY INDIANA MICHIGAN POWER COMPANY BORING NO. MW-1606D DATE 4/27/16

PROJECT ROCKPORT PLANT BORING START 2/12/16 BORING FINISH 2/12/16

SAMPLE NUMBER	SAMPLE	SAMPLE DEPTH IN FEET FROM	TO	STANDARD PENETRATION RESISTANCE BLOWS / 6"	TOTAL LENGTH RECOVERY	RQD %	DEPTH IN FEET	GRAPHIC LOG	U S C S	SOIL / ROCK IDENTIFICATION	WELL	DRILLER'S NOTES
32	SS	46.5	48.0	8-9-11	1.1					6/2, few black decomposed organic layers		
33	SS	48.0	49.5	6-10-13	1.1							
34	SS	49.5	51.0	18-13-13	.9		50		SW SM	Well graded sand w/silt & gravel, wet, pale yellowish brown 10YR 6/2, fine to coarse grained sand, little to some fine gravel, trace coarse gravel		
35	SS	51.0	52.5	7-14-16	1.1				SP SM	Poorly graded sand w/silt, moderate yellowish brown 10YR 5/4, wet, fine to medium grained sand, trace coarse grained sand, few layers of decomposed organics (from 51' - 52.5') @ 54' trace coarse gravel, fines between 5 - 10% @ 55.5' trace fine gravel		
36	SS	52.5	54.0	7-9-15	1.0							
37	SS	54.0	55.5	10-10-14	1.2		55					
38	SS	55.5	57.0	8-10-13	1.2							
39	SS	57.0	58.5	7-9-9	1.3				SW	Well graded sand, med. to coarse grained, dark yellowish brown 10YR 4/2), wet, med. dense, trace fine gravel @ 59' trace coarse gravel		
40	SS	58.5	60.0	4-5-9	1.2							
41	SS	60.0	61.5	6-6-9	1.5		60		SP	Poorly graded sand, fine grained, dusky yellowish brown 10YR 2/2, wet, med. dense, w/fine gravel @ 60.5' 2" shale fragment @ 61.5' dark yellowish brown 10YR 4/2, dense @ 61.8' 2" shale fragment @ 62' some lean clay, pale yellowish brown (prev. material) @ 62.5' no clay, trace fine gravel @ 63' no fine gravel @ 64.5' med. dense @ 65.8' 15" coarse sand seam (prev. material) @ 66' dense @ 67.2' 3" shale seam, med. l. grey N6 @ 67.7' med. grained		
42	SS	61.5	63.0	6-13-21	1.5							
43	SS	63.0	64.5	10-17-31	1.3							
44	SS	64.5	66.0	13-13-17	1.4		65					
45	SS	66.0	67.5	6-14-18	1.5							
46	SS	67.5	69.0	9-14-17	1.5				SP	Poorly graded sand, fine gravel, pale yellowish brown 10YR 6.2, wet, dense @ 69' moist to v. moist @ 72' med. dense, fine grained @ 75' dense, d. yellowish brown 10YR 4.2 @ 76.5' med. dense, trace black silt @ 80.6 3" shale plug (responsible for increase in N value (same material)) @ 81.3' 1.5" shale plug, dense		
47	SS	69.0	70.5	10-20-20	1.1							
48	SS	70.5	72.0	10-19-26	1.4		70					

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AMERICAN ELECTRIC POWER SERVICE CORPORATION
AEP CIVIL ENGINEERING LABORATORY
LOG OF BORING

JOB NUMBER **42393125-01**

COMPANY **INDIANA MICHIGAN POWER COMPANY** BORING NO. **MW-1606D** DATE **4/27/16**

PROJECT **ROCKPORT PLANT** BORING START **2/12/16** BORING FINISH **2/12/16**

SAMPLE NUMBER	SAMPLE	SAMPLE DEPTH IN FEET FROM	SAMPLE DEPTH IN FEET TO	STANDARD PENETRATION RESISTANCE BLOWS / 6"	TOTAL LENGTH RECOVERY	RQD %	DEPTH IN FEET	GRAPHIC LOG	U S C S	SOIL / ROCK IDENTIFICATION	WELL	DRILLER'S NOTES
49	SS	72.0	73.5	7-10-17	1.3					@ 81.5' no recovery, potential cobble blocking during sampling		
50	SS	73.5	75.0	8-9-13	1.2							
51	SS	75.0	76.5	10-16-25	1.4		75					
52	SS	76.5	78.0	9-10-14	1.4							
53	SS	78.0	79.5	6-9-18	1.5							
54	SS	79.5	81.0	10-17-34	1.5		80					
55	SS	81.0	82.5	31-19-14	1.3							
56	SS	82.5	84.0	10-16-21	1.5				CH	Fat clay, med. l. grey N6, moist, firm		
									SW	Well graded sand, med. grained, dark yellowish brown 10YR 4/2, wet, dense, w/fine gravel @ 83' coal fragment (2" diam., 1" thick) @ 83.6' coal fragment (2" diam, 1" thick)		
57	SS	84.0	85.5	9-19-21	1.5		85					
58	SS	85.5	87.0	7-15-24	1.3				SP	Poorly graded sand, fine grained, pale yellowish brown 10YR 6/2, wet, dense @ 88.5' trace fine gravel @ 91.5' with fine gravel		
59	SS	87.0	88.5	10-13-20	1.2							
60	SS	88.5	90.0	8-14-23	1.4		90					
61	SS	90.0	91.5	8-13-27	1.3							
62	SS	91.5	93.0	8-7-16	1.5							
63	SS	93.0	94.5	7-9-15	1.5							
									SW	Well graded sand, med. to coarse grained, dark yellowish brown 10YR 4/2, wet, med. dense, w/fine gravel		
64	SS	94.5	96.0	12-12-14	1.5		95		SP	Poorly graded sand, coarse grained, greyish red 5R 4/2, wet, med. dense, trace fine gravel		
									SW	Well graded sand, med. to coarse grained, dark yellowish brown 10YR 4/2, wet, med. dense, w/fine gravel		
65	SS	96.0	97.5	3-5-5	1.5				SP			
66	SS	97.5	99.0	5-5-6	1.4				SP			

Continued Next Page

AMERICAN ELECTRIC POWER SERVICE CORPORATION

AEP CIVIL ENGINEERING LABORATORY

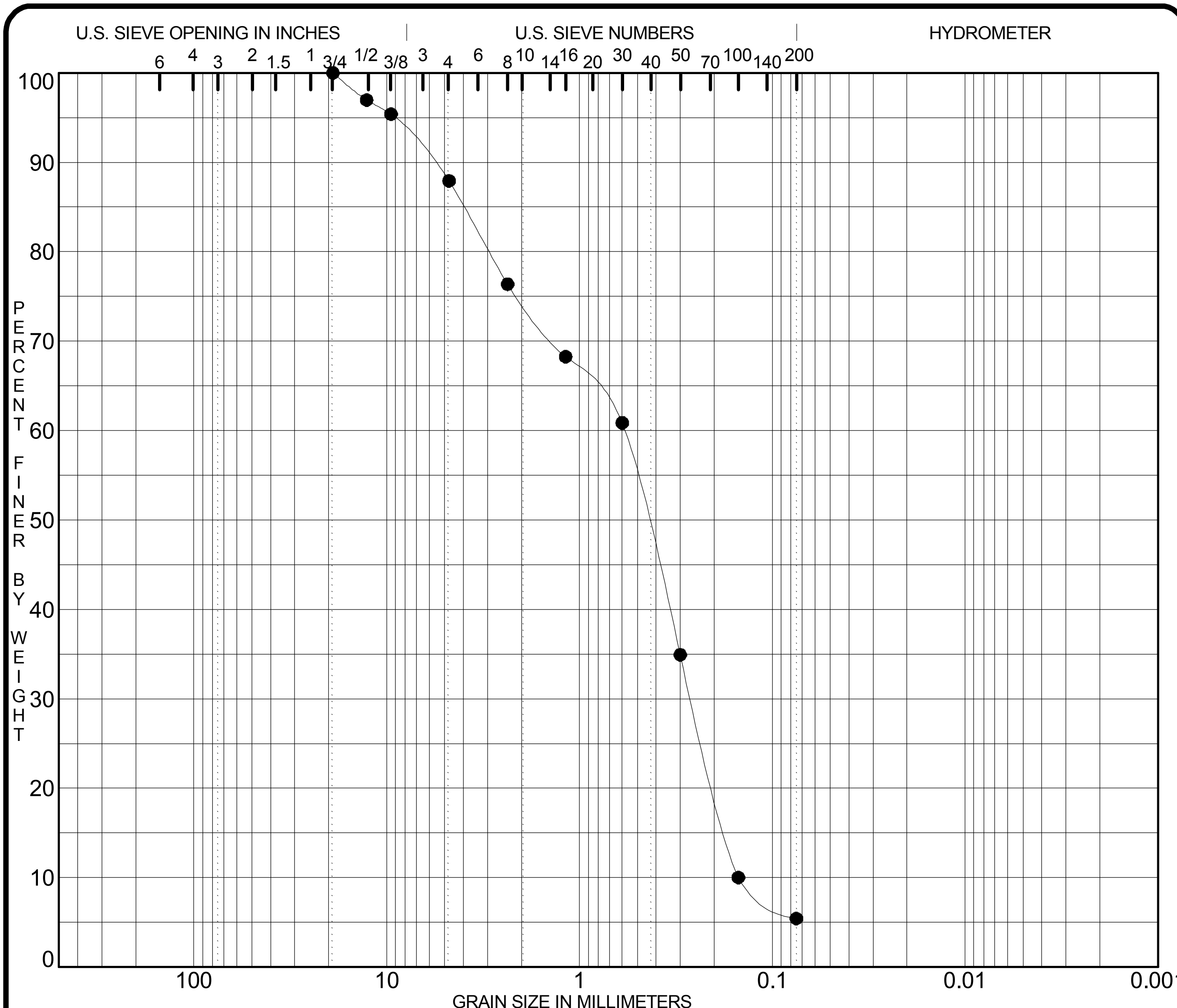
LOG OF BORING

JOB NUMBER **42393125-01**

COMPANY **INDIANA MICHIGAN POWER COMPANY** BORING NO. **MW-1606D** DATE **4/27/16** SHEET **5** OF **5**

PROJECT **ROCKPORT PLANT** BORING START **2/12/16** BORING FINISH **2/12/16**

SAMPLE NUMBER	SAMPLE	SAMPLE DEPTH IN FEET FROM	TO	STANDARD PENETRATION RESISTANCE BLOWS / 6"	TOTAL LENGTH RECOVERY	RQD %	DEPTH IN FEET	GRAPHIC LOG	U S C S	SOIL / ROCK IDENTIFICATION	WELL	DRILLER'S NOTES
										Poorly graded sand, coarse grained, greyish red 5R 4/2, wet, med. dense to loose, trace fine gravel		
67	SS	99.0	100.5	4-5-7	1.5					Poorly graded sand, fine grained, pale yellowish brown 10YR 6/2, wet, loose		
							100			@ 97.5' med. dense, fine grained		
68	SS	100.5	102.0	7-7-10	1.4				SP	Poorly graded sand, fine to fine grained, dusky red 5R 3/4, wet, med. dense		
69	SS	102.0	103.5	4-4-6	1.5					@ 102' loose, fine grained, moist @ 103.5' med. dense		
70	SS	103.5	105.0	5-6-10	1.3					@ 105' fine grained @ 106.5' dense @ 108' med. dense, trace fine gravel @ 109' no fine gravel @110.6' siltstone fragments to 2.5", moderate		
71	SS	105.0	106.5	4-6-9	1.5		105			brown 5YR 4/4, shiny, angular		
72	SS	106.5	108.0	7-11-20	1.4							
73	SS	108.0	109.5	8-13-15	1.5							
74	SS	109.5	111.0	10-18-11	1.3		110					
75	SS	111.0	112.5	14-50/3					ML	Silt, l. grey N7, moist, med. dense, non-durable shale @ 111' clayey silt, hard		
76	SS	112.5	114.0	50/4						Spoon refusal @ 111.7' Auger refusal @ 112.9 BT @ 112.9'		



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification	Classification	MC%	LL	PL	PI	Sp.Gr.	
● MW-1605D 115.0-124.6ft		14.5					
	SS-78,79,80,81,82,83 (Composite Sample)						
	N 151,478.9 E 513,537.1						
	ELEVATION 400.4						

Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Fines	%<.002
● MW-1605D 115.0-124.6ft	19.000	0.586	0.262	0.150	12.1	82.5	5.4	

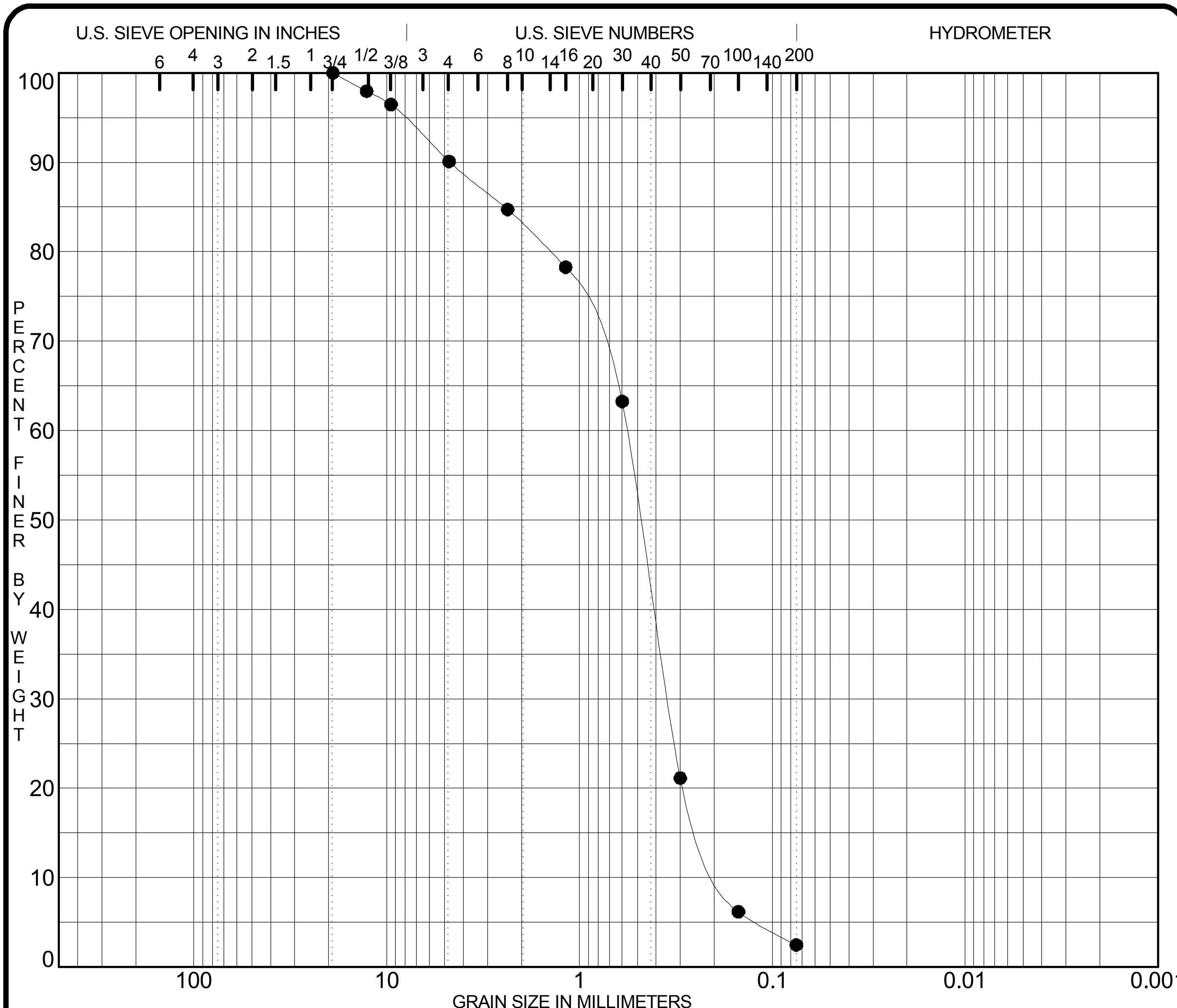
PROJECT **ROCKPORT PLANT -**

JOB NO. **42393125-01**
DATE **3/22/16**

GRADATION CURVES

American Electric Power Service Corp.

AEP



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

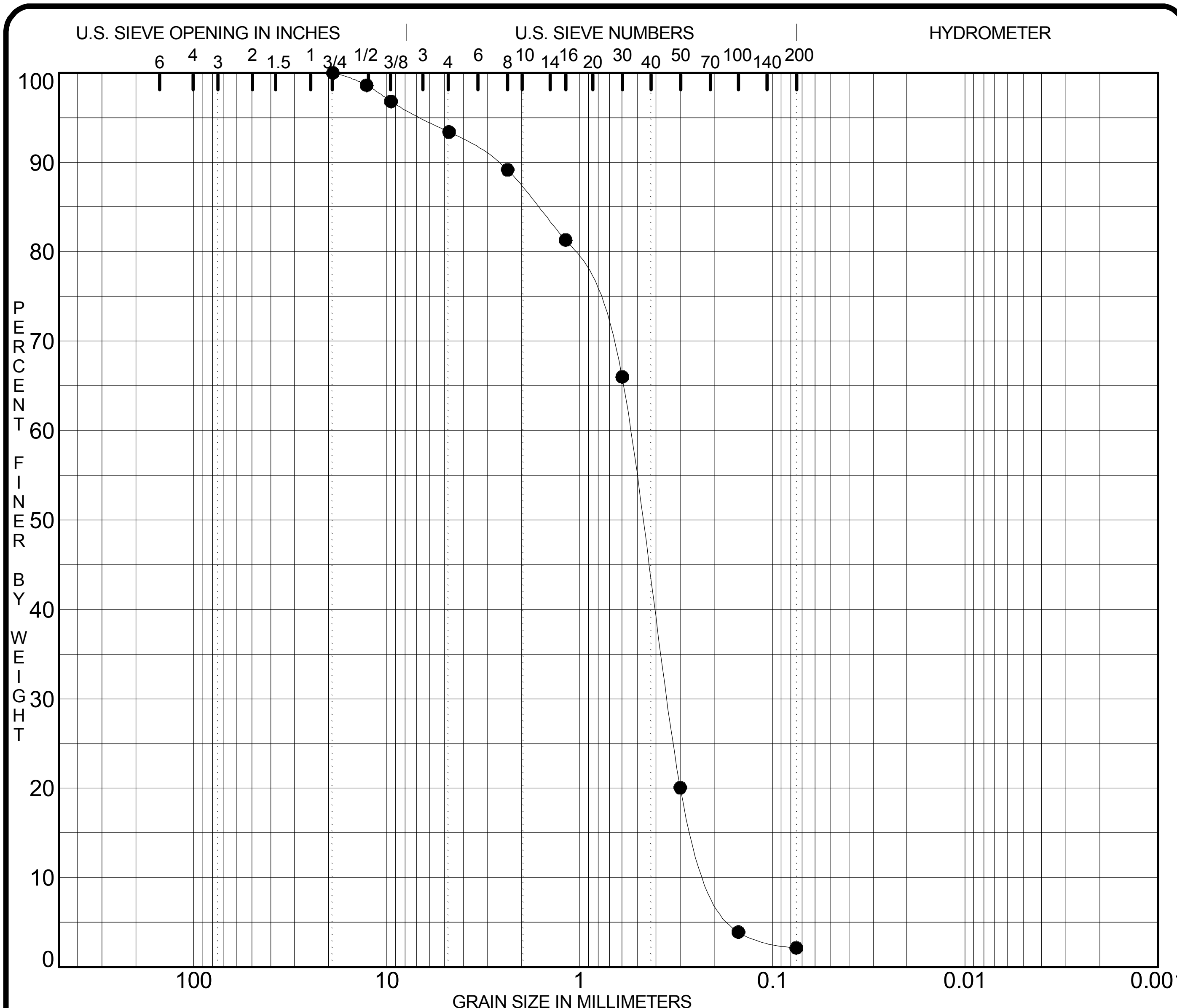
Specimen Identification	Classification	MC%	LL	PL	PI	Sp.Gr.
● **MW-1605I 68.6-78.2ft**		**19.7**				
	POORLY GRADED SAND SP					
	SS-48,49,50,51,52 (Composite Sample)					
	N 151,478.9 E 513,532.6					
	ELEVATION 400.6					

Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Fines	%<.002
● **MW-1605I 68.6-78.2ft**	**19.000**	**0.569**	**0.347**	**0.179**	**9.9**	**87.6**	**2.5**	

PROJECT **ROCKPORT PLANT -**

JOB NO. **42393125-01**
DATE **3/22/16**

GRADATION CURVES
American Electric Power Service Corp.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification	Classification	MC%	LL	PL	PI	Sp.Gr.	
● MW-1605S 37.6-47.2ft		16.0					
	POORLY GRADED SAND SP						
	SS-27,28,29,30,31 (Composite Sample)						
	N 151,478.8 E 513,528.4						
	ELEVATION 400.3						

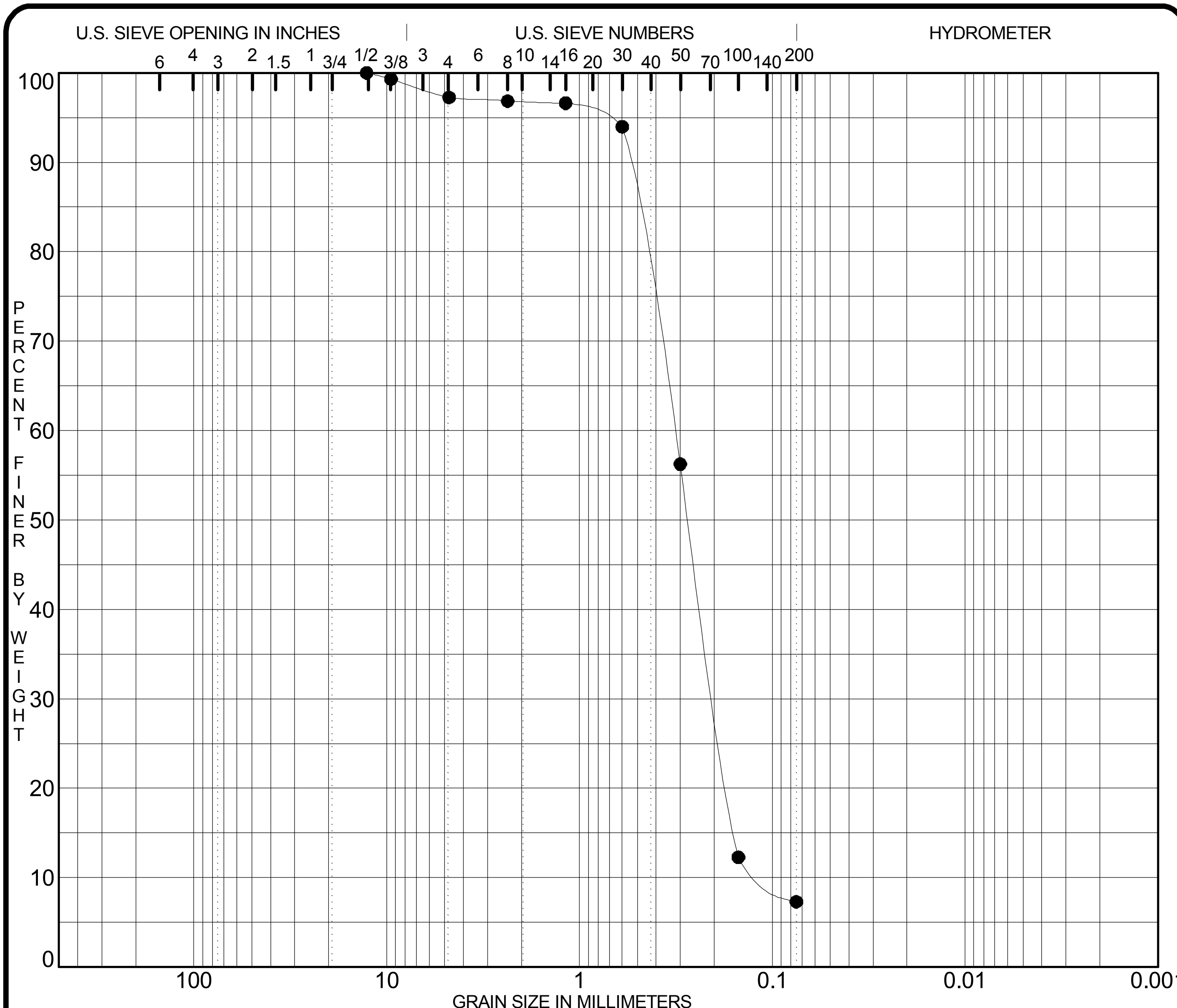
Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Fines	%<.002
● **MW-1605S 37.6-47.2ft**	**19.000**	**0.548**	**0.349**	**0.195**	**6.6**	**91.3**	**2.1**	

PROJECT **ROCKPORT PLANT -**

JOB NO. **42393125-01**
DATE **3/22/16**

GRADATION CURVES

American Electric Power Service Corp.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification	Classification	MC%	LL	PL	PI	Sp.Gr.	
●	MW-1606D 100.0-109.6ft		28.6					
		SS-68,69,70,71,72,73 (Composite Sample)						
		N 151,502.4 E 512,881.5						
		ELEVATION 397.8						

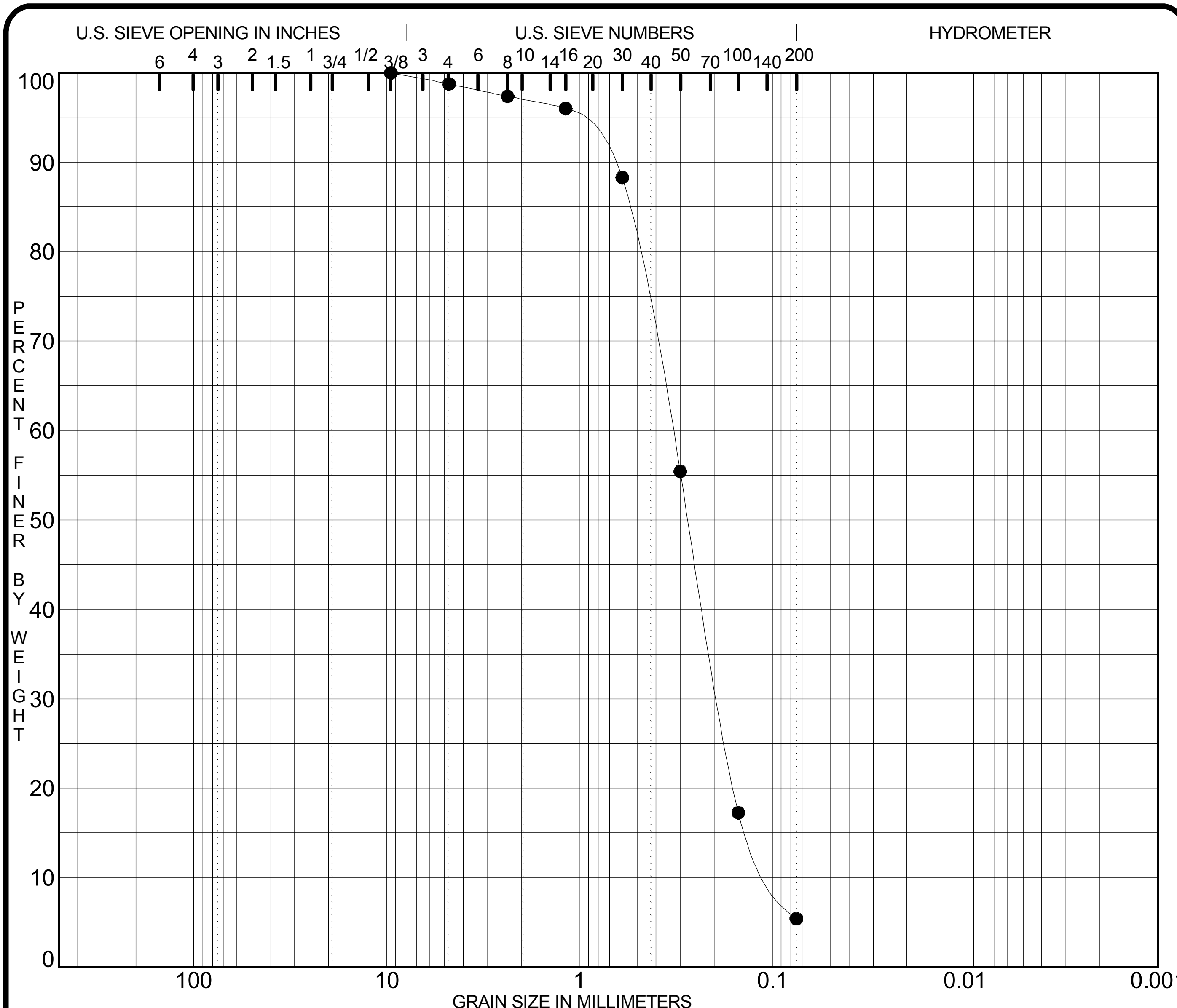
	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Fines	%<.002
●	MW-1606D 100.0-109.6ft	12.700	0.321	0.198	0.109	2.7	90.0	7.3	

PROJECT **ROCKPORT PLANT -**

JOB NO. **42393125-01**
DATE **3/22/16**

GRADATION CURVES

American Electric Power Service Corp.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification	Classification	MC%	LL	PL	PI	Sp.Gr.	
●	MW-1606I 65.7-75.3ft		18.9					
		SS-45,46,47,48,49,50 (Composite Sample)						
		N 151,508.4 E 512,885.5						
		ELEVATION 397.8						

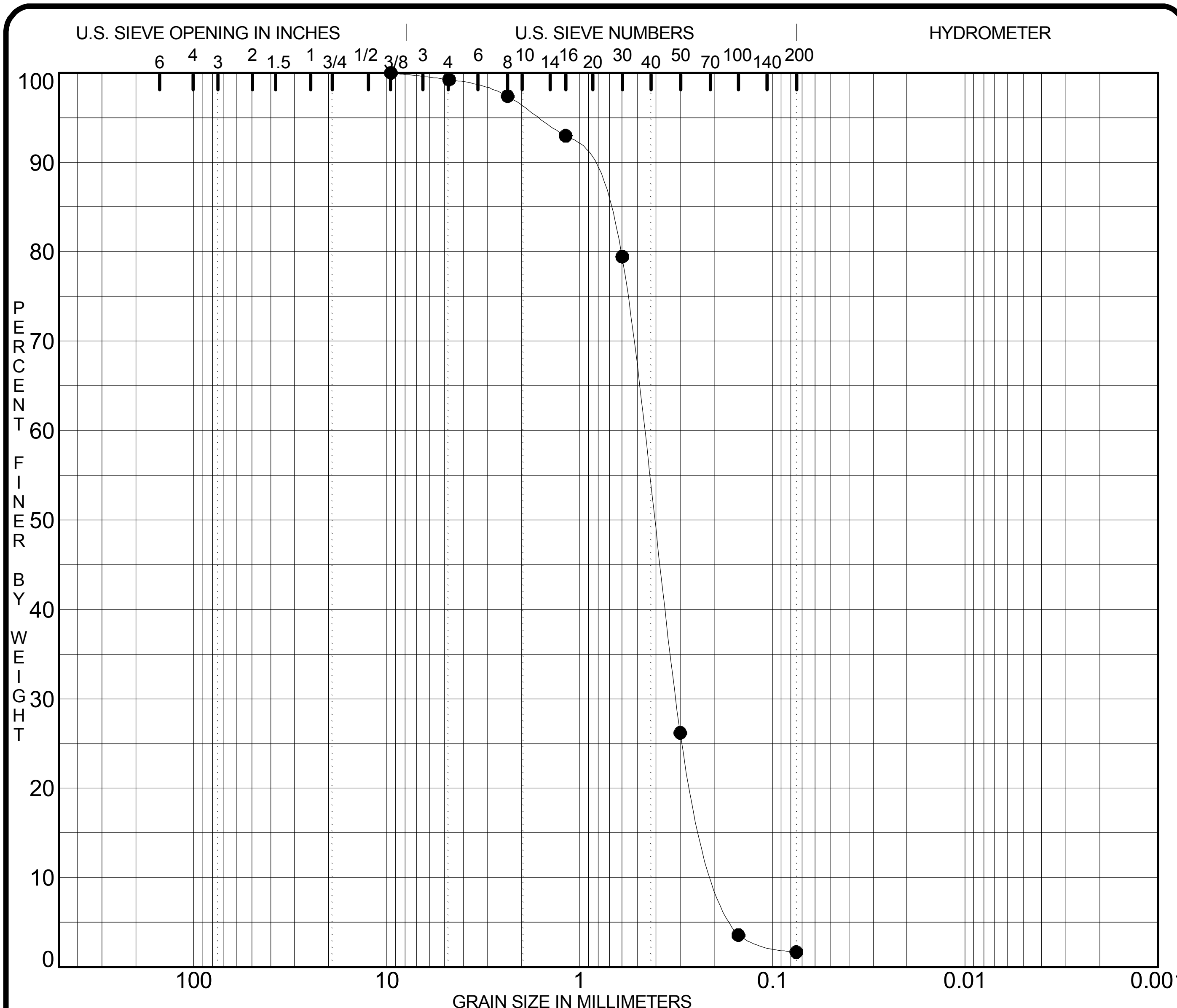
	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Fines	%<.002
●	**MW-1606I 65.7-75.3ft**	**9.500**	**0.330**	**0.189**	**0.098**	**1.2**	**93.4**	**5.4**	

PROJECT **ROCKPORT PLANT -**

JOB NO. **42393125-01**
DATE **3/22/16**

GRADATION CURVES

American Electric Power Service Corp.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification	Classification	MC%	LL	PL	PI	Sp.Gr.
●	MW-1606S 34.7-44.3ft		20.9				
		POORLY GRADED SAND SP					
		SS-25,26,27,28,29 (Composite Sample)					
		N 151,498.9 E 512,889.4					
		ELEVATION 397.6					

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Fines	%<.002
●	MW-1606S 34.7-44.3ft	9.500	0.466	0.315	0.183	0.8	97.6	1.7	

PROJECT **ROCKPORT PLANT -**

JOB NO. **42393125-01**
DATE **3/22/16**

GRADATION CURVES

American Electric Power Service Corp.

APPENDIX C : LiquefyPro Analysis Input and Output

SEISMIC IMPACT ZONES DEMONSTRATION

Bottom Ash Complex -Rockport Plant

Hole No.=1605 Water Depth=30 ft Surface Elev.=399 ***Magnitude=6.46***
Acceleration=0.14957g

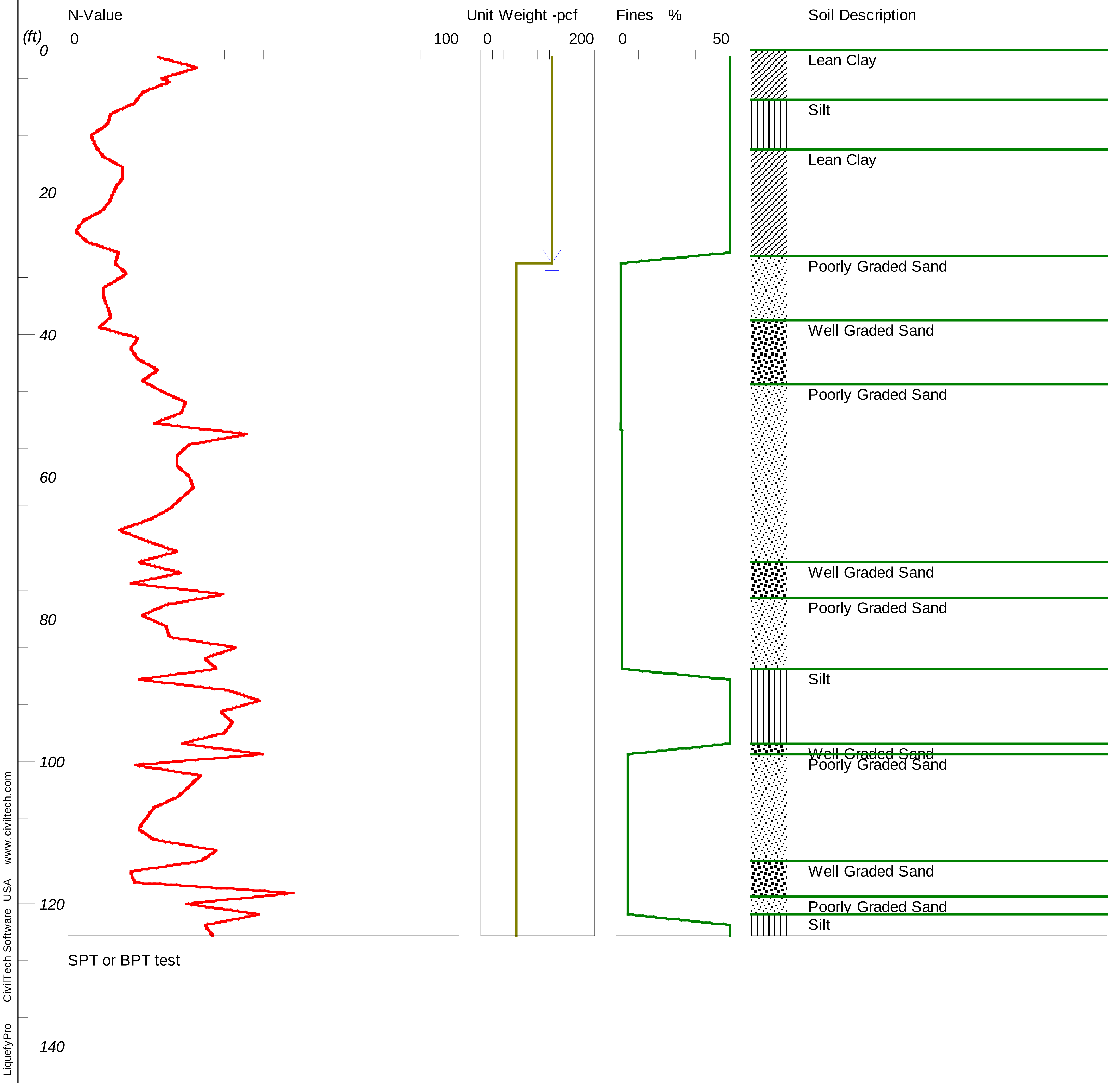


Plate A-1

American Electric Power Service Corporation

SEISMIC IMPACT ZONES DEMONSTRATION

Bottom Ash Complex -Rockport Plant

Hole No.=1605 Water Depth=30 ft Surface Elev.=399 ***Magnitude=6.46 Acceleration=0.14957g***

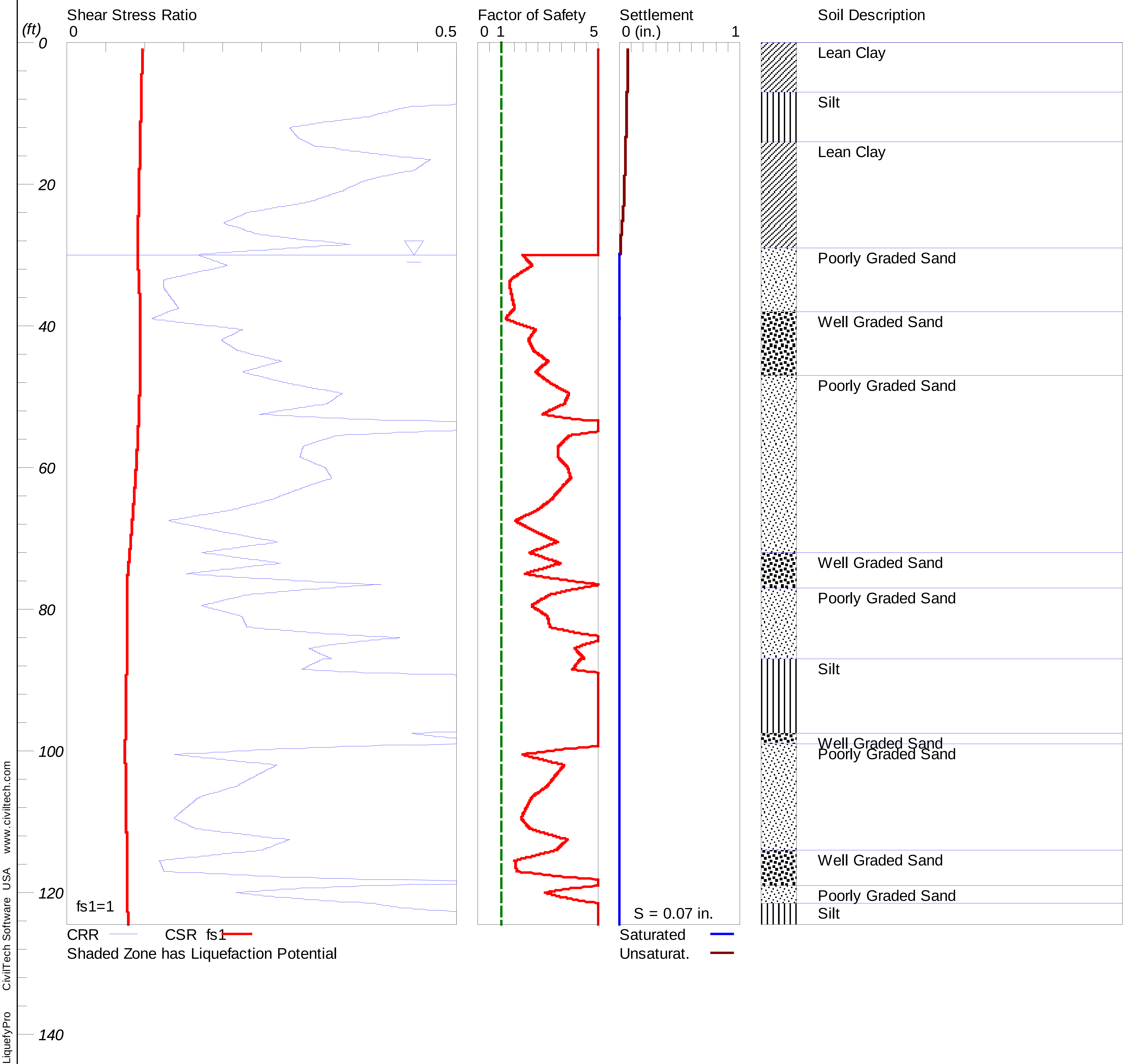


Plate A-1

AMERICAN ELECTRIC POWER
BOUNDLESS ENERGY

American Electric Power Service Corporation

SEISMIC IMPACT ZONES DEMONSTRATION

Bottom Ash Complex -Rockport Plant

Hole No.=1606 Water Depth=27 ft Surface Elev.=397.8

Magnitude=6.46
Acceleration=0.14957g

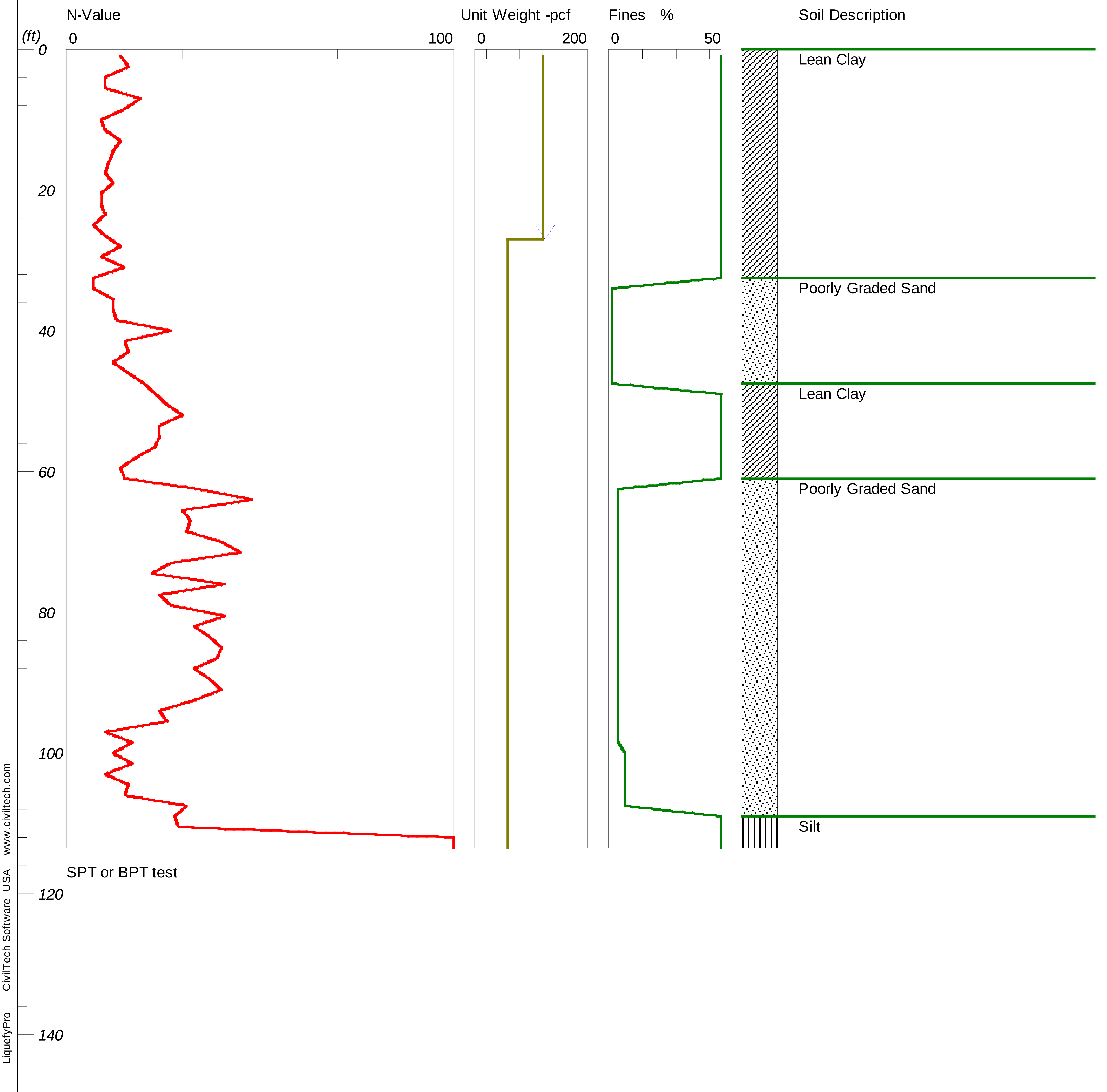


Plate A-1

AMERICAN ELECTRIC POWER
BOUNDLESS ENERGY

American Electric Power Service Corporation

SEISMIC IMPACT ZONES DEMONSTRATION

Bottom Ash Complex -Rockport Plant

Hole No.=1606 Water Depth=27 ft Surface Elev.=397.8

Magnitude=6.46
Acceleration=0.14957g

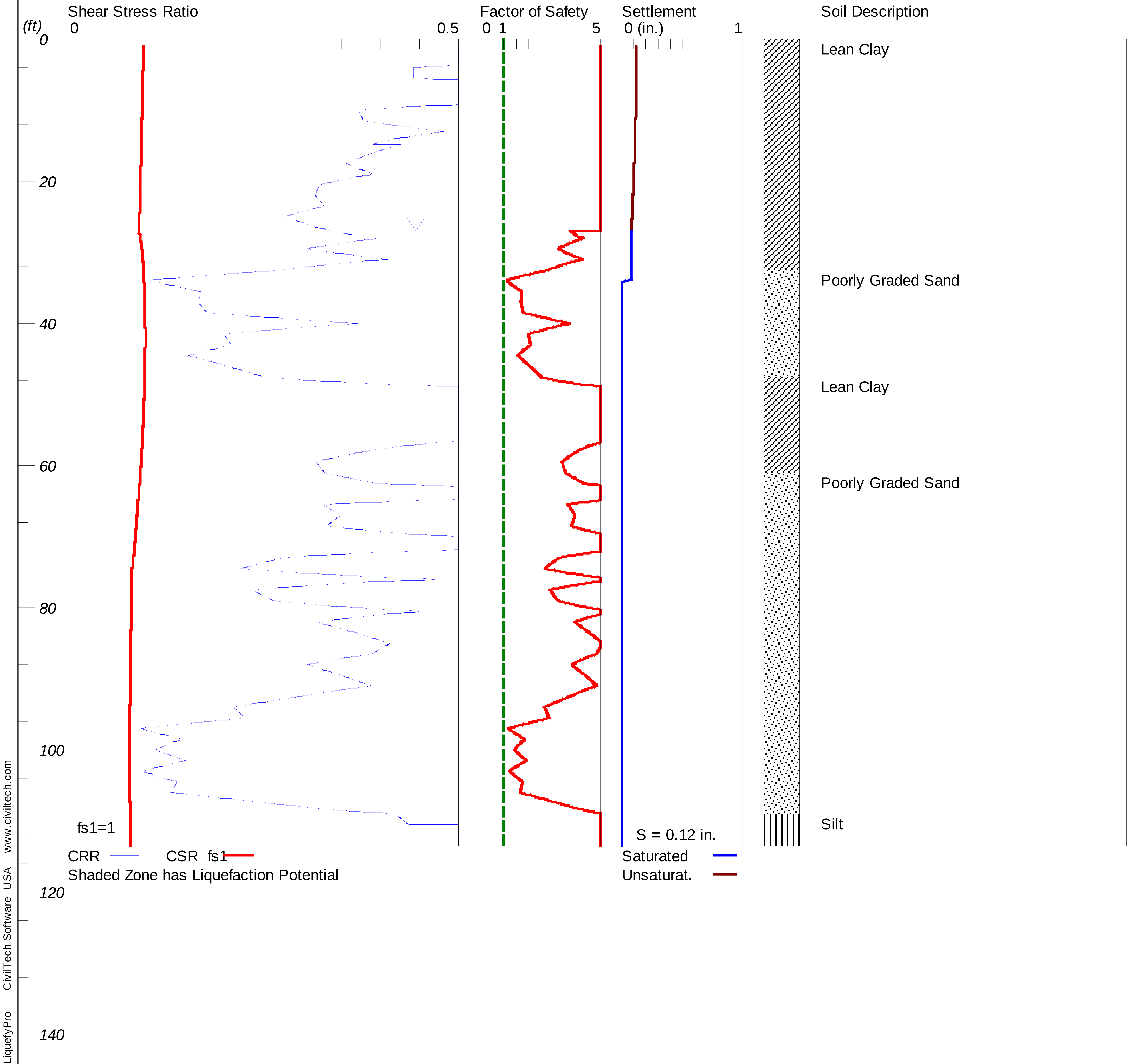


Plate A-1

American Electric Power Service Corporation

APPENDIX D : Structural Calculation SES-CALC-02391

AEP AMERICAN ELECTRIC POWER

1/38

CALCULATION COVER SHEET

PLANT:	Rockport	**TITLE:**	CCR Compliance of Seismic Impact Zone Review of Structures Located on Bottom Ash Ponds
UNIT:	0		

CALCULATION NUMBER: SES-CALC- 02391 **REV. NUMBER:** 1

STRUCTURE/SYSTEM/COMPONENT: West and East Bottom ash Ponds

ASSOCIATED DRAWING NUMBERS: 12-30013

PURPOSE OF CALCULATION: to
Validate Compliance with CCR Rules
Rev. 1 provides clarification to corrective action. See page 37.

ALTERNATE CALCULATION PERFORMED TO VALIDATE THIS CALCULATION: (Y/N) _Y____
IF YES, THE ALTERNATE CALCULATION SHALL BE INCLUDED IN THIS PACKAGE.

PREPARER: Satyananda Chakrabarti **Date:** 9/22/2018

CHECKER: J. Reiniger **Date:** 9/26/2018

ENGINEERING SECTION MANAGER: [signature] **Date:** 10-5-18

Total No. of Pages Including Coversheet and Checklist: 38

38

Table of Contents Page No.

Calculation No. SES-CALC- 02391

THIS IS A SUPPORTING CALC FOR GEC-16-007

Units used in this calculation:

Mass: Length: Pressure/Stress: Angularity:

$kip \equiv 1000 \cdot lbf$ $lb \equiv \frac{kip}{1000}$ $ft \equiv 1 \cdot L$ $psi \equiv \frac{lbf}{in^2}$ $Rad \equiv 1$

$kips \equiv kip$ $lbs \equiv lb$ $ft \equiv 12 \cdot in$ $ksi \equiv \frac{kip}{in^2}$ $deg \equiv 2 \cdot \pi \cdot \frac{rad}{360}$

$pcf \equiv \frac{lbf}{ft^3}$ $acc := 32.2 \cdot \frac{ft}{sec^2}$ $psf \equiv \frac{lbf}{ft^2}$

$plinf := \frac{lbf}{ft}$

$ksf \equiv \frac{kip}{ft^2}$

SUMMARY OF CALCULATION

Objective of the calculation: Demonstrate that the structural components of the CCR units West and East Bottom Ash ponds are designed to meet the maximum horizontal acceleration in lithified earth material for the site. This calculation evaluates the seismic impact on the surface water control systems.

ASSUMPTION: This calculation assumes that the stability of the dikes which are currently being investigated for earthquakes will be found to be stable.

Background

1. The CCR rule requires:
§ 257.63 Seismic impact zones.
(a) New CCR landfills, existing and new CCR surface impoundments, and all lateral expansions of CCR units must not be located in seismic impact zones unless the owner or operator demonstrates by the dates specified in paragraph (c) of this section that all structural components including liners, leachate collection and removal systems, and surface water control systems, are designed to resist the maximum horizontal acceleration in lithified earth material for the site. (b) The owner or operator of the CCR unit must obtain a certification from a qualified professional engineer stating that the demonstration meets the requirements of paragraph (a) of this section.

2. The structures were constructed in the late 70s and early 80s for a 2-unit operating plant with

a total capacity of approximately 2,600 MW.

3. The structures reviewed in this evaluation are all surface water control units facilitating water flow from the Bottom Ash Ponds to the Waste water ponds.

4. The units included in the total population can be classified into two groups:

- Group 1: Units subjected to lateral loading due to the quakes used for transfering water from Bottom Ash ponds to waste water ponds including units used to dewater the BA ponds. The units are:

1. Energy Dissipator structure (EDS - 2 nos.) - approximately 8 plant pipes of 8 - 10 inch dia pipes discharging into this structure and then transported into the BA pond through the Energy Dissipator troughs/Pond Discharge Inlet Chutes. EDSs are of concrete with steel dissipation flaps.

2.Energy Dissipator troughs/Pond Discharge Inlet Chutes (EDT) - These are concrete structures partially open at the top and partially covered by yellow steel boxes called Discharge Chute Covers.

3. Skimmers (SKM)- Timber structures surrounding the waste water discharge chute

4. Waste water Discharge shaft (WWDS)- a steel and concrete prismoidal structure for routing waste water into the waste water discharge pipe.

- Group 2: Waste water discharge pipe (WWDP) - Two buried 48 inch (one fiberglass and the other HDPE) pipes that transfer water under the dikes. Because they are buried they are affected by seismic waves and ground displacements.

Two sets of analyses have been performed for the two groups.

5. Presentation of the Calculation

A. Part I - Relevant Cross-sections.
CCR Rule list

B. Part II - A - Maximum Considered Earthquake Spectral Acceleration
S_s = Mapped maximum considered earthquake spectral acceleration at 0.2 seconds
S_1 = Mapped maximum considered earthquake spectral acceleration at 1.0 seconds

Get the *"Latitude"* & *"Longitude"* for the site and input as shown below.

Use the Tool

Documentation & Help

Recent Changes

Worldwide Seismic Design Tool

Use the Tool

Documentation & Help

Application | Batch Mode | Help

Design Code Reference Document

Consult your local design official if you need help selecting this

2010 ASCE 7 (w/March 2013 errata)

Report Title (Optional)

This will appear at the top of the generated report

Report 1

Site Soil Classification

This is **not** automatically selected based on site location

Site Class C - "Very Dense Soil and Soft Rock

Risk Category

Used to compute the seismic design category

I or II or III

Site Latitude

Decimal degrees for the site location

37.92556

Site Longitude

Decimal degrees for the site location

-87.03389

Compute Values

Powered by Leaflet — Content may not reflect National Geographic's current map policy

Part II - B. Apply site coefficients Fa and Fv - these are site amplification factors

Adjust MCE spectral response acceleration S_{MS} and S_{M1}

Derive S_{DS} and S_{D1} 5% damped design spectral response acceleration at 0.2 second period - ASCE 7-05

$S_{DS} = 2/3\ S_{MS}$ $S_{D1} = 2/3\ S_{M1}$

Plot Elastic Design Response Spectra

Part II - C Determine Seismic Design Category and Equivalent Lateral Load without SSI

Determine Risk and seismic category

Value of S_{DS}	Occupancy Category		
	I or II	III	IV
$S_{DS} < 0.16g$	A	A	A
$0.16g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$S_{DS} \ge 0.50g$	D	D	D

Value of S_{D1}	Occupancy Category		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$S_{D1} \ge 0.20g$	D	D	D
$S_{D1} \ge 0.75g$	E	E	F

This section calculates the initial lateral load without soil-structure interaction.

Part II-D Consider Soil-structure Interaction ASCE 7 Chapter 19

Soil Density = 120 pcf Vs = 1,200 ft/sec

Vertical and lateral spring calculated per Hall.

The reduction is minimal and neglected.

Finally vertical force due to vertical earthquake is calculated per ASCE 7 eqn. 12.4.2.2

Part II-E Apply hydrodynamic force

Because the skimmer is constructed of timber materials and its structural condition is deteriorated, it is assumed to fail during earthquake. The subject unit may then be subjected to hydrodynamic forces generated by waves,

The dynamic and static water pressure are then added to other forces for equilibrium.

Part II-F Final Check for equilibrium

Check safety against sliding and overturning

High safety factor obtained and no further check is performed.

This structure was analyzed as a typical structure subject to shear loading. Based on high safety margins, no other shear-susceptible structure was analyzed because by judgment they will be OK,

Part III - Analyses for underground piping

FEMA -ASCE, American Lifelines Alliance, Guidelines for the Design of Buried Steel Pipe, Jul 200'

The pipelines (2 nos) one fiberglass and the other HDPE are not specifically addressed by the reference but the the treatment of strains and stresses can be transferred from steel properties to non-steel properties. If high safety factors are obtained, then specific analyses with the specific properties are not needed to evaluate.

11.1 Seismic Wave Propagation

Wave propagation provisions are presented in terms of longitudinal axial strain, that is, strain parallel to the pipe axis induced by ground strain. Flexural strains due to ground curvature are neglected since they are small for typical pipeline diameters.

The axial strain, ε_a, induced in a buried pipe by wave propagation can be approximated using the following equation:

$$\varepsilon_a = \frac{V_g}{\alpha C_s} \quad (11\text{-}1)$$

where:

V_g = peak ground velocity generated by ground shaking

C_s = apparent propagation velocity for seismic waves (conservatively assumed to be 2 kilometers per second)

α = 2.0 for C_s associated with shear waves, 1.0 otherwise

The axial strains produced by Equation (11-1) can be assumed to be transferred to the pipeline but need not be taken as larger than the axial strain induced by friction at the soil pipe interface:

$$\varepsilon_a \le \frac{T_u \lambda}{4AE} \quad (11\text{-}2)$$

where:

T_u = peak friction force per unit length at soil-pipe interface (see Appendix A)

λ = apparent wavelength of seismic waves at ground surface, sometimes assumed to be 1.0 kilometers without further information

A = pipe cross-sectional area

E = steel modulus of elasticity

Tu is calculated by the eqn. below.

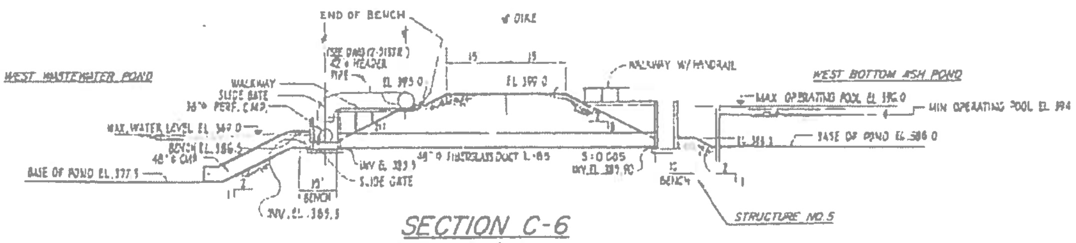
For our case, with cohesionless backfill, the peak force per unit length of the soil-pipe interface (from Appendix B) is:

$$T_u = \frac{\pi}{2} D H \bar{\gamma} \left(1 + K_o\right) \tan\delta$$

PART I

Relevant Cross Sections

Reference drawings:
12-30013, -27



SECTION C-6
SCALE: 1"=20'

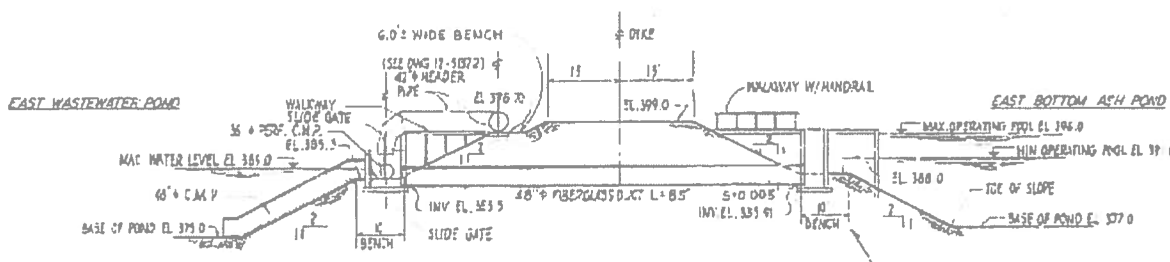
Structures/structural components at Section C-6:

West Bottom Ash Pond Side

C-6-1: 48" Fiberglass Duct
C-6-2: Concrete Inflow Box
C-6-3: Timber box for skimmer
C-6-4: Walkway with handrails

West Wastewater Pond Side

C-6-5: 48" CMP Discharge Line
C-6-6: 36" Perforated CMP
C-6-7: Concrete Inflow Box
C-6-8: Walkway with handrails



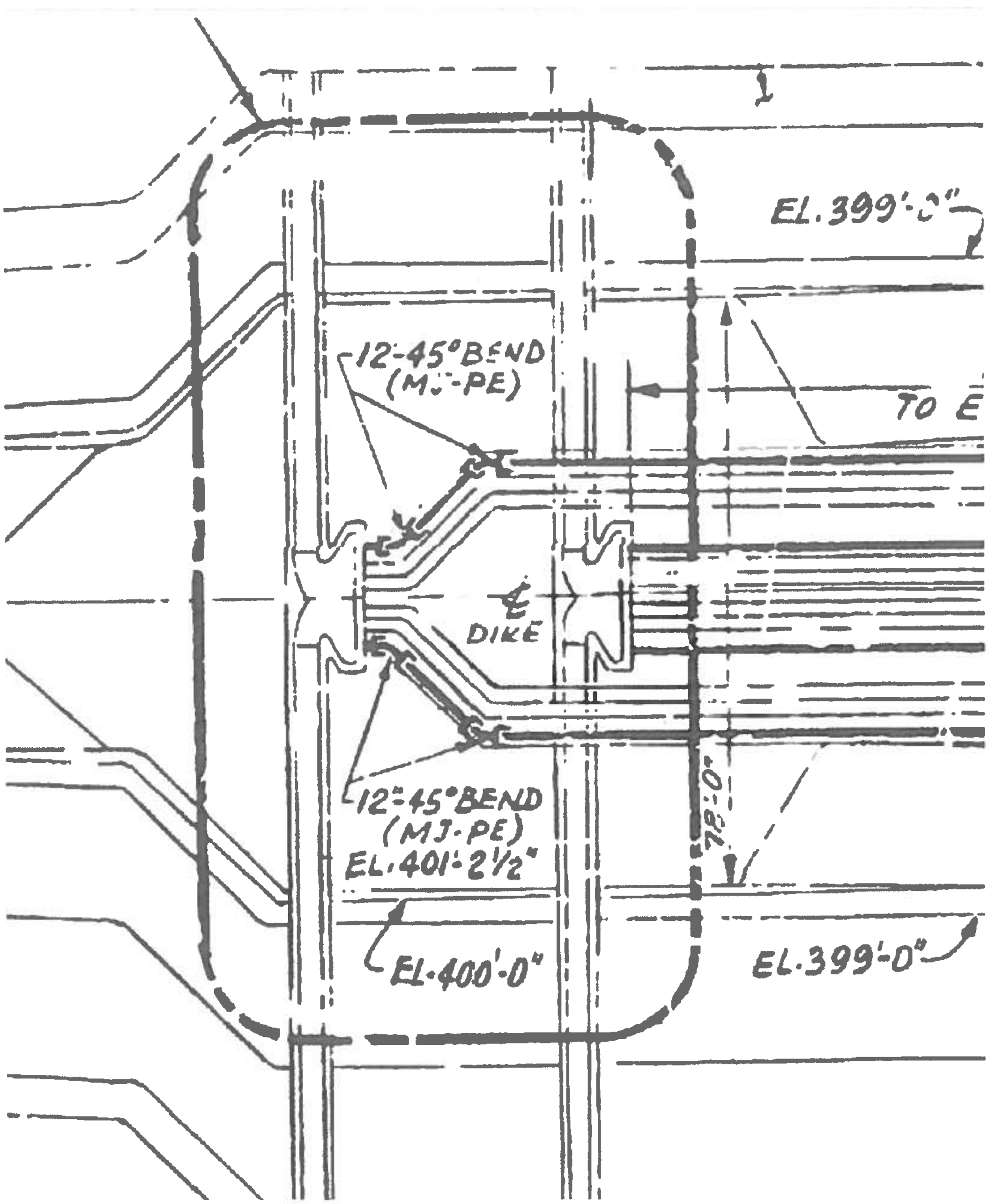
SECTION C-4
SCALE: 1"=20'

East Wastewater Pond Side

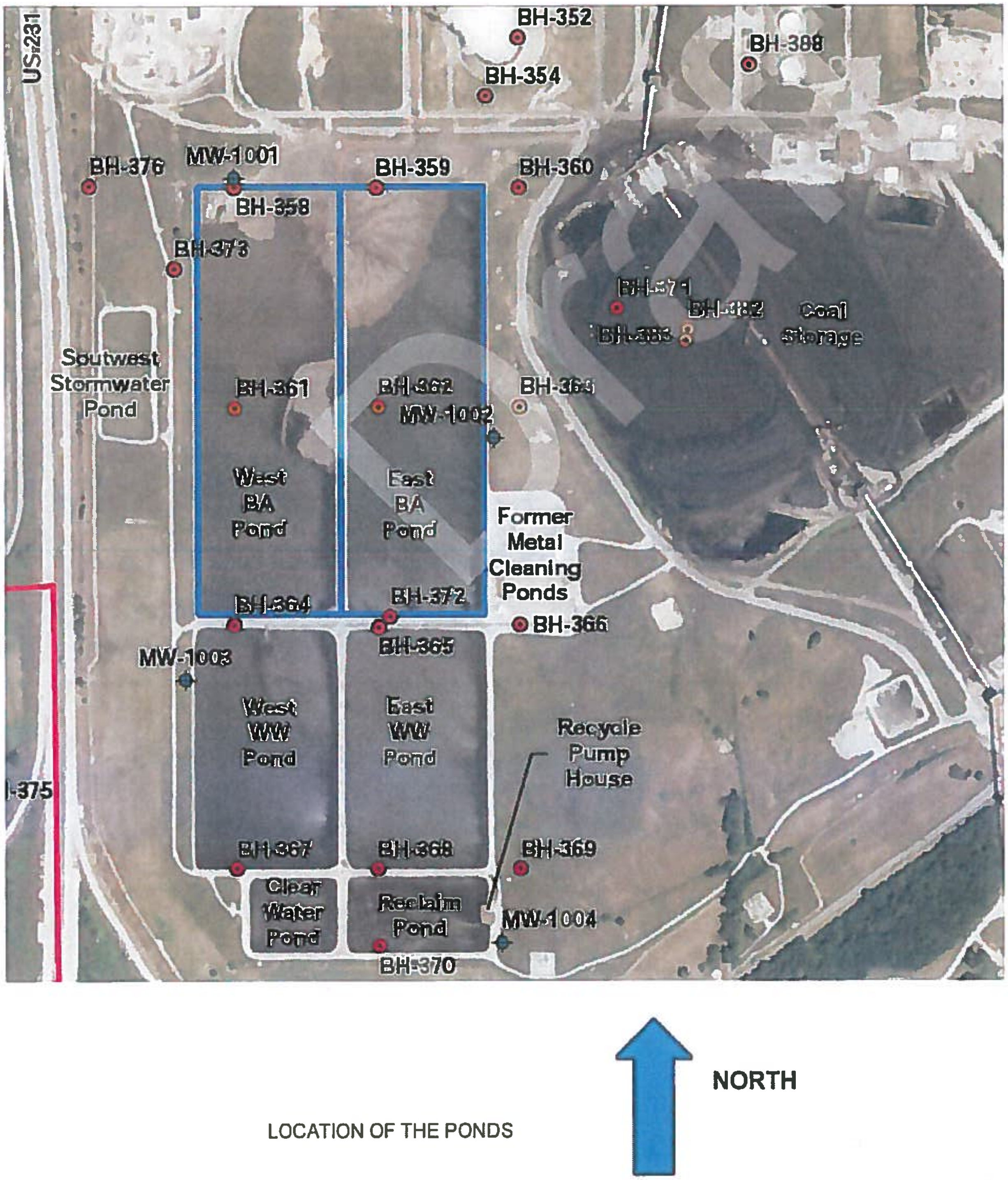
C-4-5: 48" CMP Discharge Line
C-4-6: 36" Perforated CMP
C-4-7: Concrete Inflow Box
C-4-8: Walkway with handrails

East Bottom Ash Pond Side

C-4-1: 48" Fiberglass Duct
C-4-2: Concrete Inflow Box
C-4-3: Timber box for skimmer
C-4-4: Walkway with handrails



Dike and Flow Diverters



LOCATION OF THE PONDS

PART II - A
Maximum Considered Earthquake Spectral Acceleration

CCR Rule

Requirement	Existing Surface Impoundments				New Surface Impoundments and Lateral Expansions			
	Five feet high AND 20 acre-feet, or 20 feet high				Five feet high AND 20 acre-feet, or 20 feet high			
	Yes		No		Yes		No	
	Required?	Rule Section	Required?	Rule Section	Required?	Rule Section	Required?	Rule Section
Location Restrictions:	√	§257.60 - §257.64	√	§257.60 - §257.64	√	§257.60 - §257.64	√	§257.60 - §257.64
Placement Above the Uppermost Aquifer	√	§257.60	√	§257.60	√	§257.60	√	§257.60
Wetlands	√	§257.61	√	§257.61	√	§257.61	√	§257.61
Fault Areas	√	§257.62	√	§257.62	√	§257.62	√	§257.62
Seismic Impact Zones	√	§257.63	√	§257.63	√	§257.63	√	§257.63
Unstable Areas	√	§257.64	√	§257.64	√	§257.64	√	§257.64

With respect to seismic reviews two issues are relevant:

1. Fault Areas 2. Seismic Impact Zones

Reference : **EVALUATION OF LOCATION RESTRICTIONS, Bottom Ash Ponds, Rockport Plant, Draft Final, 25 sep 2015. p.15:**

"Based on available information, it is our opinion that the site meets the criterion of being located more than 200 feet from the outermost damage zone of a fault with displacement in Holocene time, as set forth in 40 CFR §257.62." End of Fault contributing to earthquake.

SEISMIC IMPACT ZONES **37.925560 Lat. -87.033890 Long.**

The same reference also states that: http://earthquake.usgs.gov/designmaps/us/application.php

"The 2014 USGS National Seismic Hazard Maps (NSHM) display earthquake ground motions for various probability levels across the United States. We have reviewed the USGS National Seismic Hazard Map showing a 2% probability that the maximum expected horizontal acceleration, expressed as a percentage of the earth''s gravitational pull (g), will be exceeded in years (2% exceedance in 50 years, Peak Ground Acceleration (PGA)). The USGS NSHM map is provided as **Figure 9**. Based on the NSHM map for a 2% exceedance in 50 years, we have determined the PGA for this site is 0.2 g."
(Figure 9 not attached).

From CCR Rules: " A Seismic impact zone means an area having a 2% or greater probability that the maximum expected horizontal acceleration, expressed as a percentage of the earth's gravitational pull (g), will exceed 0.10 g in 50 years. Seismic zones, which represent areas of the United States with the greatest seismic risk, are mapped by the U.S. Geological Survey and readily available for all the U.S. (http://earthquake.usgs.gov/hazards/apps/)".

References : The following references are used:

1. CCR Publication: Federal Register, April 17, 2015.
2. IBC Code, 2012/ASCE 7
3. FERC, Evaluation of Earthquake Ground Motions, Draft 06.5, FERC Feb 2007

USGS Design Maps Summary Report

User-Specified Input

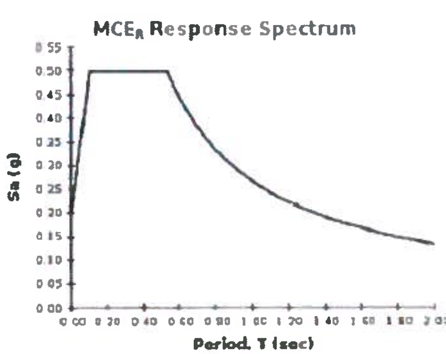
Report Title	USGS Data Sat August 27, 2016 14:53:43 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	37.92556°N, 87.03389°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Risk Category	I/II/III

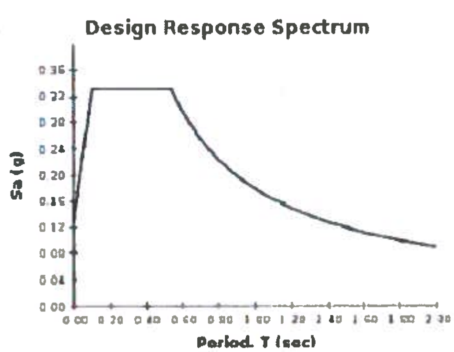
USGS-Provided Output

$S_S =$ 0.415 g $S_{MS} =$ 0.498 g $S_{DS} =$ 0.332 g

$S_1 =$ 0.163 g $S_{M1} =$ 0.267 g $S_{D1} =$ 0.178 g

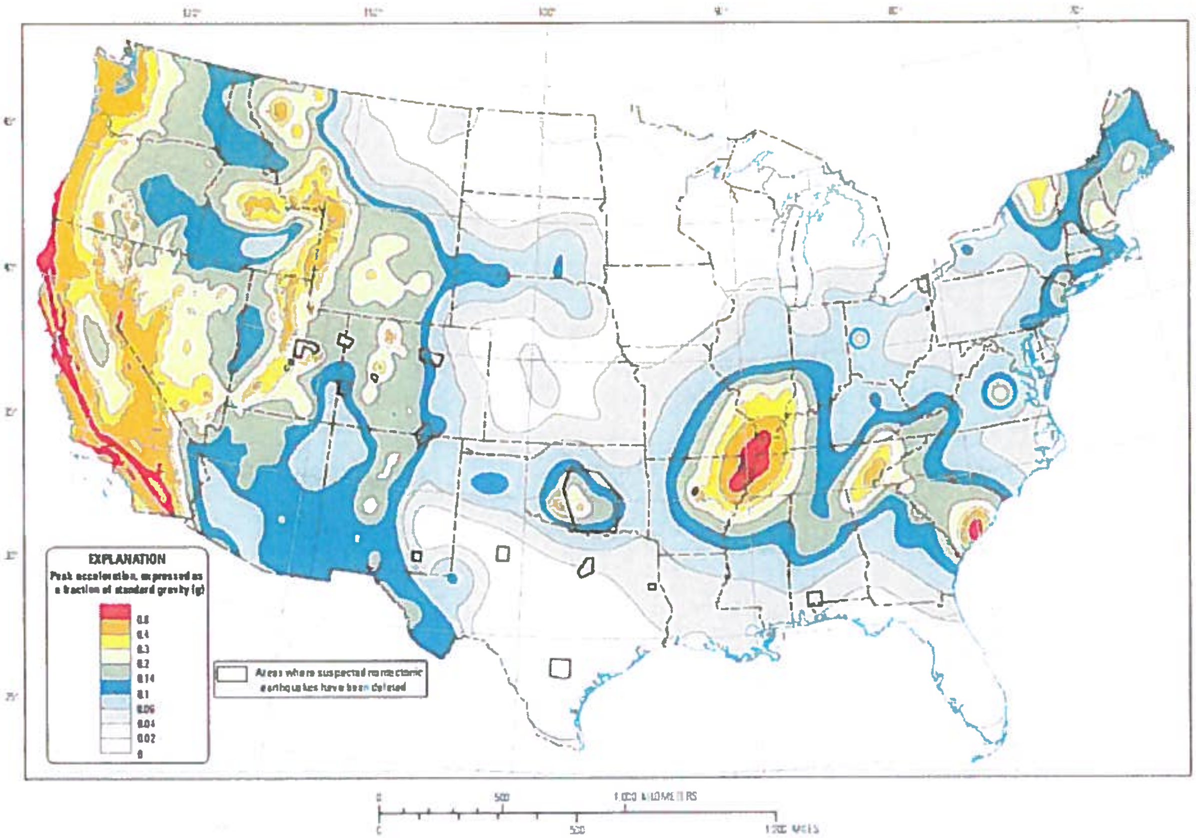
For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA$_M$, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.



Two-percent probability of exceedance in 50 years map of peak ground acceleration

The map indicates PGA = 0.2 g

Determine Building Occupancy: from IBC Section 312, Group U

Determine basic ground motion parameters

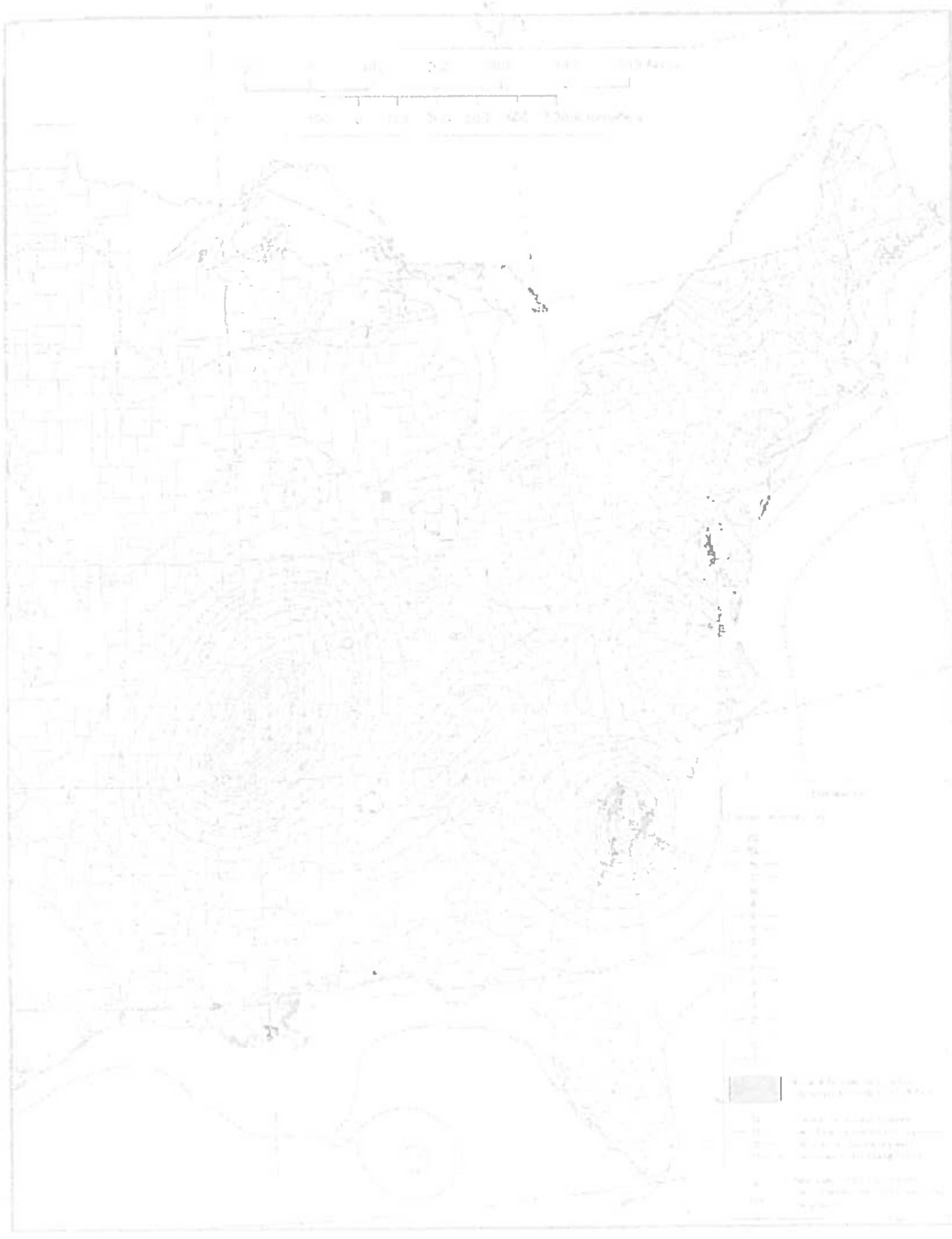


FIGURE 1613.3.1(1)—continued
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

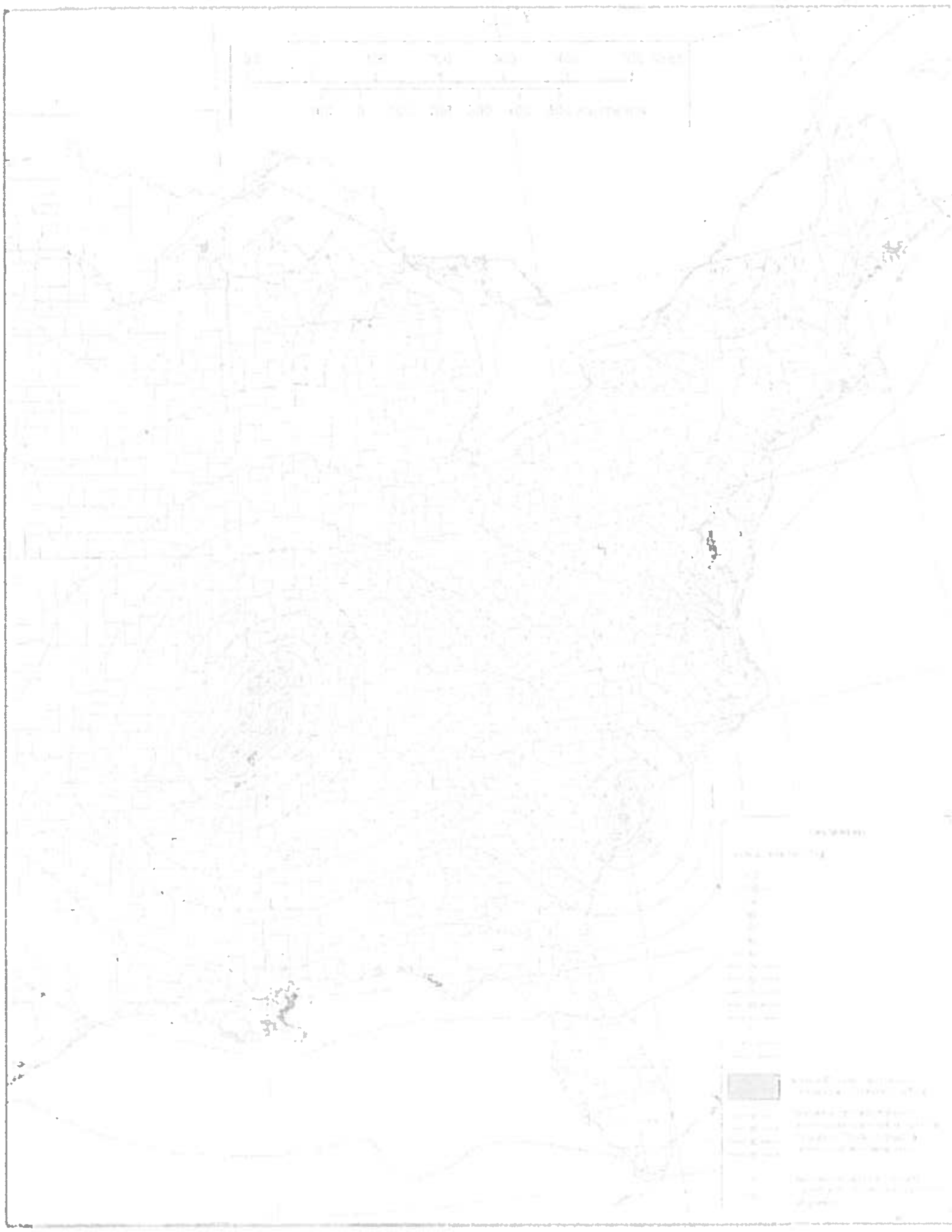


FIGURE 1613.3.1(2)—continued
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

S_s = Spectral acceleration for 0.2s

$S_s := 0.415 \cdot g$

S_1 = Spectral acceleration for 1s

$S_1 := 0.163 \cdot g$

Part II - B
Apply Site Amplification Factors

Calculate S_{DS} and S_{D1}

S_{DS} and S_{D1} = spectral values from Tables 1613.3.3(1) and 1613.3.3(2) respectively.

$S_{DS} = 2/3*F_a*S_s$ $S_{D1} = 2/3*F_v*S_1$

Fa and Fv are site coefficients.

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a ᵃ

TE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIOD				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	Note b	Note b	Note b	Note b	Note b

aight-line interpolation for intermediate values of mapped spectral response acceleration at short period, S_s.
, shall be determined in accordance with Section 11 4.7 of ASCE 7.

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_V ᵃ

TE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT 1-SECOND PERIOD				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	Note b	Note b	Note b	Note b	Note b

aight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, S_1.
. shall be determined in accordance with Section 11 4.7 of ASCE 7.

Table 20.3-1 of ASCE 7 provides definition of Site Class.

Table 20.3-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the following characteristics: —Plasticity index $PI > 20$, —Moisture content $w \geq 40\%$, —Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1 ft/s = 0.3048 m/s; 1 lb/ft^2 = 0.0479 kN/m^2.

Reference: Engineering Report Minor Permit Modification, Mar 2012 provides a geotechnical evaluation of subsurface soi in the area. It estimates that the shear wave velocity is 1200 ft/sec. That makes it a site class C.

For S_s= 0.415g $\quad F_a := 1.2$

For S_1 =0.16 $\quad F_v := 1.7 - (1.7 - 1.6)\cdot\frac{.06}{0.1} = 1.64$

$$S_{DS} := \frac{2}{3}\cdot F_a\cdot S_s = 0.332\cdot g \qquad S_{D1} := \frac{2}{3}\cdot F_v\cdot S_1 = 0.178\cdot g$$

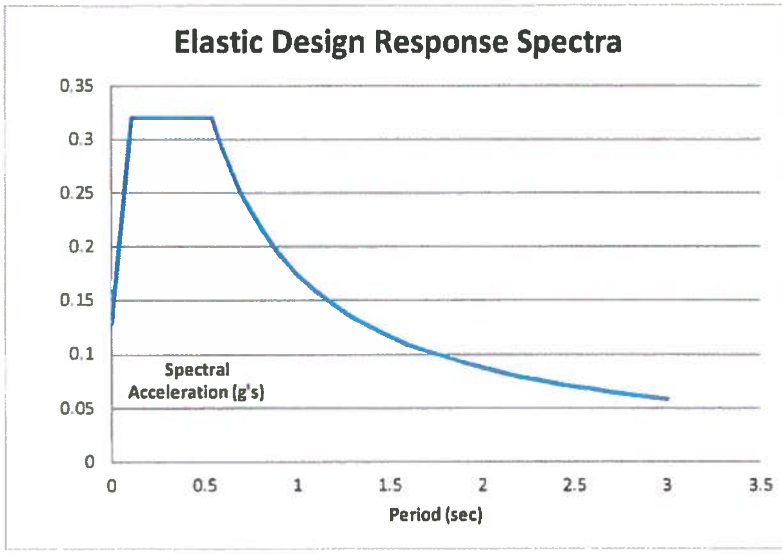
$$T_0 := 0.2\cdot\frac{S_{D1}}{S_{DS}}\cdot sec = 0.107 \quad T_s := \frac{S_{D1}}{S_{DS}}\cdot sec = 0.537\,s$$

For T< T_0 $\qquad$ Sa =(0.4+0.6*T/T_0)S_{DS}

For T< T_0 $\qquad$ $S_{a1}\left[S_{DS}\cdot\left(0.4 + 0.6\cdot\frac{1\cdot sec}{T_0}\right) = \blacksquare\right]$

T	S_{DS}	S_{D1}	T_0	T_S	S
	0.32	0.175	0.109	0.547	

T	Sa	
0	0.128	
0.109	0.32	9.17
0.547	0.32	1.83
0.6	0.29	1.67
0.7	0.25	1.43
0.8	0.22	1.25
0.9	0.19	1.11
1	0.18	1.00
1.1	0.16	0.91
1.2	0.15	0.83
1.3	0.13	0.77
1.4	0.13	0.71
1.5	0.12	0.67
1.6	0.11	0.63
1.7	0.10	0.59
1.8	0.10	0.56
1.9	0.09	0.53
2	0.09	0.50
2.1	0.08	0.48
2.2	0.08	0.45
2.3	0.08	0.43
2.4	0.07	0.42
2.5	0.07	0.40
2.6	0.07	0.38
2.7	0.06	0.37
2.8	0.06	0.36
2.9	0.06	0.34
3	0.06	0.33



Part II - C
Determine Seismic Design Category and Equivalent Lateral Load without SSI

RISK CATEGORY

Select Risk Category 1 per Table 1.5-1. below and Seismic Importance Factor =1.00

$Risk_{cat} := 1$

$I_e := 1$

Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

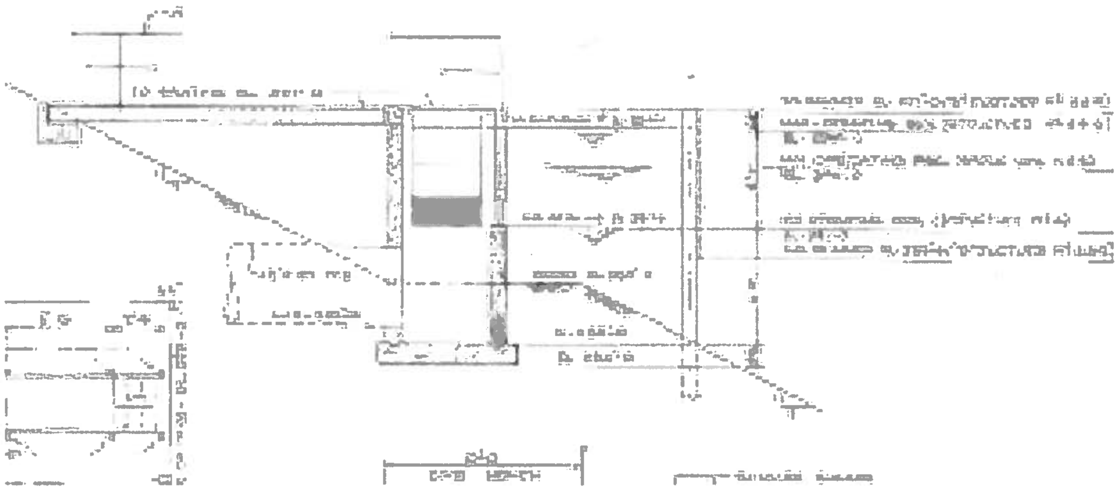
Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent a low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where their quantity exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.	III
Buildings and other structures designated as essential facilities. Buildings and other structures, the failure of which could pose a substantial hazard to the community. Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity exceeds a threshold quantity established by the authority having jurisdiction to be dangerous to the public if released and is sufficient to pose a threat to the public if released.[a] Buildings and other structures required to maintain the functionality of other Risk Category IV structures.	IV

[a]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the substances is commensurate with the risk associated with that Risk Category.

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads[a]

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.25	1.00	1.25
IV	1.20	1.25	1.00	1.50

[a]The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it is dependent on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.



Response Modification Factors, R_{xf}

ASCE 7-10, Table 12.2-1

Ordinary Reinforced Concrete Shear wall

$R_{cf} := 4$

$\Omega_{c0} := 2.5$ Overstrength Factor

$C_{cd} := 2.5$ Deflection Amplification Factor

Steel Ordinary Moment Frames

$R_{sf} := 3.5$

$\Omega_{s0} := 3$

$C_{sd} := 3$

Light -framed wood walls (by Judgment)

$R_{wf} := 3.5$

$\Omega_{w0} := 3$

$C_{wd} := 3$

Determine Seismic Response Coefficients - Concrete

Seismic Base shear

ASCE 7, Section 12.8.1.1 Concrete

$$C_{s\_conc1} := \frac{S_{DS}}{\left(\frac{R_{cf}}{I_e}\right)} = 0.083 \cdot g$$

ASCE 7, Eqn. 12.8-5, C_s shall not be less than

$$C_{s\_min} := \max\left(0.044 \cdot S_{DS} \cdot I_e, 0.01 \cdot g\right) = 0.015 \cdot g$$

Satisfy ASCE 7 eqn. 12.8-3 and 12.8-4 after calculating T_a

Calculate C_t $\quad C_{t\_conc} := 0.02$ ASCE 7 Table 12.8-2

$$x_{ct\_conc} := .75$$

ASCE 7 Page 225 Fig 22-12 $\quad T_{L\_conc} := 12 \cdot sec$

Calculate T_{a_conc}

Method 1: Use ASCE 7, eqn. 12.8-7 $\quad h_n := 13$ Dwg No. 12-3453

$$T_{a\_conc1} := C_{t\_conc} \cdot \left(h_n\right)^{x_{ct\_conc}} \cdot 1 \cdot sec = 0.137\,s$$

$$freq_{a\_conc1} := \frac{1}{T_{a\_conc1}} = 7.303 \cdot Hz$$

Method 2: Use ASCE 7 eqn. 12.8-7

$$N_{conc2} := 1$$

$$T_{a\_conc2} := 0.1 \cdot N_{conc2} \cdot 1 \cdot sec = 0.1\,s$$

$$freq_{a\_conc2} := \frac{1}{T_{a\_conc2}} = 10 \cdot Hz$$

Use average T.a

$$T_{a\_ave} := \frac{\left(T_{a\_conc1} + T_{a\_conc2}\right)}{2} = 0.118\,s$$

$S_{D1} = 0.178 \cdot g$

Determine C_s for $T_{a\_conc} < T_{L\_conc}$

$$C_{s\_conc2} := \frac{S_{D1}}{T_{a\_ave} \cdot \frac{1}{sec} \left(\frac{R_{cf}}{I_e}\right)} = 0.376 \cdot g$$

$$C_{s\_conc1} = 0.083 \cdot g$$

$$C_{s\_conc} := \max\left(\min\left(C_{s\_conc1}, C_{s\_conc2}\right), C_{s\_min}\right) = 0.083 \cdot g$$

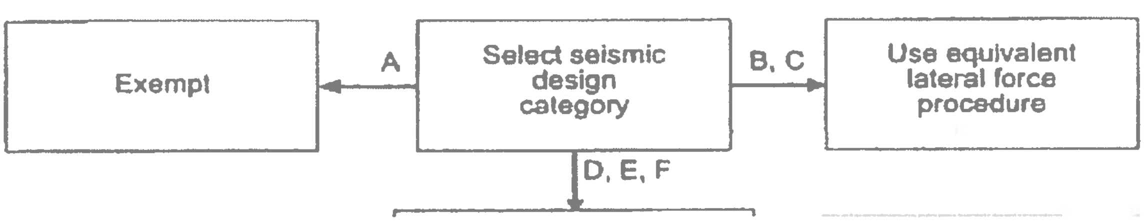
SUMMARY WITHOUT SOIL STRUCTURE INTERACTION

$T_{a\_wossi} := T_{a\_ave} = 0.118\,s$ Period

$C_{s\_conc\_wossi} := C_{s\_conc} = 0.083 \cdot g$

$S_{DS} = 0.332 \cdot g$

$S_{D1} = 0.178 \cdot g$



Part II-D
Consider Soil-structure Interaction ASCE 7 Chapter 19

Consider Soil-Structure Interaction per ASCE 7 Chapter 19

Size of found - 7'-8" by 7'-2"

$$A_{fdn} := 7.67 \cdot ft \cdot 7.17 \cdot ft = 54.994\,ft^2$$

Equivalent Radius $\quad r_{equiv} := \sqrt{\dfrac{A_{fdn}}{\pi}} = 4.184\,ft$

Reference: Engineering Report Minor Permit Modification, Mar 2012 provides a geotechnical evaluation of subsurface soils in th
area. It estimates that the shear wave velocity is 1200 ft/sec.

Because of the small size and the approximate nature of the shear wave velocity, no correction will be made due to G/G_{max} and D/D_{max} for being less than 1.

$$V_s := 1200 \cdot \frac{ft}{sec} = 1.2 \times 10^3 \frac{ft}{s} \qquad \rho := \frac{120 \cdot \dfrac{lb}{ft^3}}{32.2 \cdot \dfrac{ft}{sec^2}} = 3.727 \frac{s^2}{ft} \cdot \frac{lb}{ft^3}$$

$$\nu := 0.3 \qquad G_{ssi} := \rho \cdot {V_s}^2 = 5.366 \times 10^6 \cdot psf$$

Vibrations of Soils and Foundations, Richart, Hall and Woods, 1970

Sliding stiffness

$$K_{yssi} := 32 \cdot \frac{(1-\nu)}{(7-8 \cdot \nu)} \cdot G_{ssi} \cdot r_{equiv} = 1.093 \times 10^8 \cdot \frac{lb}{ft}$$

Rocking

$$K_{\theta ssi} := \frac{8 \cdot \left(\dfrac{1}{rad}\right) \cdot G_{ssi} \cdot {r_{equiv}}^3}{3 \cdot (1-\nu)} = 1.497 \times 10^6\,ft \cdot \frac{1}{rad} \cdot kip$$

Calculate W_{bar}

Weight of walls

$$W_{tot1} := \left[\left(5+\frac{4}{12}\right)\cdot\frac{10}{12}\cdot 2+\left(5+\frac{10}{12}\right)\cdot\frac{10}{12}\cdot 2\right]\cdot 13\cdot 150\cdot \mathbf{lbs} = 3.629\times 10^{4}\,\mathbf{lbf}$$

$$W_{tot2} := \left(7+\frac{8}{12}\right)\cdot\left(7+\frac{2}{12}\right)\cdot 1\cdot 150\cdot \mathbf{lbs} = 8.242\times 10^{3}\,\mathbf{lbf}$$ Slab weight

$$W_{tot} := W_{tot1} + W_{tot2} = 4.453\times 10^{4}\,\mathbf{lbf}$$ Weight of wall approximate due to stubby structure

$$T_{a\_ave} = 0.118\,\mathbf{s}$$

$$W_{bar} := .7\cdot W_{tot} = 3.117\times 10^{4}\,\mathbf{lbf}$$

$$k_{bar} := 4\cdot\pi^{2}\cdot\frac{W_{bar}}{32.2\cdot\frac{\mathbf{ft}}{\mathbf{sec}^{2}}\cdot T_{a\_ave}^{\;2}} = 2.723\times 10^{6}\,\frac{1}{\mathbf{ft}}\cdot\mathbf{lbf}$$

$$T_{bar} := T_{a\_ave}\cdot\sqrt{1+\frac{k_{bar}}{K_{yssi}}\cdot\left[1+\frac{K_{yssi}\cdot(.7\cdot 14\cdot\mathbf{ft})^{2}}{K_{\theta ssi}}\right]} = 0.13\,\mathbf{s}$$

$$freq_{bar} := \frac{1}{T_{bar}} = 7.707\cdot\mathbf{Hz}$$

Parametric Variation + or - 50% in G value or 123% or 70% respectively for V_s

Plugging the limiting shear wave velocities, the periods and freq values are:

$T_{bar150} := .125\cdot\mathbf{sec}$ $T_{bar50} := 0.136\cdot\mathbf{sec}$ $T_{bar} = 0.13\,\mathbf{s}$

$$freq_{bar150} := \frac{1}{T_{bar150}} = 8\cdot\mathbf{Hz} \qquad freq_{bar50} := \frac{1}{T_{bar50}} = 7.353\cdot\mathbf{Hz}$$

Calculate base shear reduction ASCE 7 Section 19.2

$r_{equiv} = 4.184\,\mathbf{ft}$ $\qquad h_{bar} := 0.7 \cdot 12.\mathbf{ft} = 8.4\,\mathbf{ft}$ $\qquad L_0 := 7.67 \cdot \mathbf{ft}$

$r_{radgyr} := .289 \cdot 7.167 \cdot \mathbf{ft} = 2.071\,\mathbf{ft}$

$b_0 := 7.167$ $\qquad d_0 := 7.67$ $\qquad I_0 := \dfrac{b_0 \cdot d_0^{\,3}}{12} = 269.49$

$r_m := 4 \cdot \sqrt{4 \cdot \dfrac{I_0}{\pi}} \cdot \mathbf{ft} = 74.095\,\mathbf{ft}$ $\qquad \dfrac{h_{bar}}{r_m} = 0.113$ $\qquad$ Use h/r=0.5 curve

Calculate period lenghening

$\dfrac{T_{bar}}{T_{a\_wossi}} = 1.095$ $\qquad \beta_0 := 0.04$

ASCE 7 eqn 19.2-9

$$\beta_{bar} := \frac{(\beta_0 \cdot 0.05) \cdot 100}{\left(\dfrac{T_{bar}}{T_{a\_wossi}}\right)^3} = 0.152$$

No further SSI consideration Conservative

$C_s := C_{s\_conc} = 0.083 \cdot \mathbf{g}$ $\qquad$ FINAL LATERAL LOAD FROM EARTHQUAKE

VERTICAL EARTHQUAKE

$E_{vert} := .2 \cdot S_{DS} = 0.066 \cdot \mathbf{g}$ $\qquad$ ASCE 7 Eqn. 12.4.2.2

Part II-E
Apply Hydrodynamic Force

Reference:Hydrodynamic Pressure on Culvert Gates during an Earthquake

Ali Rasekh *2012 SIMULIA Community Conference Attachment 2*

It is accepted in civil engineering to estimate the hydrodynamic pressure on the rigid reservoir dams by the Westergaard hydrodynamic pressure equation, which is

$$p_d = 0.875\rho_w g k\sqrt{H.h}$$

where is the hydrodynamic pressure, g= acceleration due to gravity, and k is the design seismic coefficient. The value of is two third of the peak ground acceleration in terms of *g (i.e.k=(2/3))* PGA/g. H is the total depth of the water reservoir and is the depth from the reservoir water surface to the point of action of hydrodynamic pressure.

$H_{East} := (394 - 377)\cdot ft = 17 ft$ $h_{EAST} := (394 - 388)\cdot ft = 6 ft$

$PGA := 0.213\cdot g$ $k_{East} := 2\cdot\frac{PGA}{3.0\cdot g} = 0.142$

$H_{East} := 17 ft$ $h_{West} := 6.0\cdot ft$ $H_{West} := 6. ft$

$p_d := .875\cdot 62.4\cdot pcf\cdot k_{East}\cdot\sqrt{H_{West}\cdot h_{West}} = 46.519\cdot psf$

Total Hydrodynamic force

$P_d := \frac{2\cdot h_{West}\cdot p_d}{3} = 186.077\frac{1}{ft}\cdot lbf$

Static water pressure

$$p_{static} := 62.4pcf \cdot H_{West} = 2.6\,psi$$

$$P_{static} := .5 \cdot p_{static} \cdot H_{West} = 1.123 \times 10^{3}\,\frac{1}{ft} \cdot lbf$$

Total water pressure

$$PW_{total} := P_{static} + P_{d} = 1.309 \times 10^{3}\,\frac{1}{ft} \cdot lbf$$

$$W_{tot} = 4.453 \times 10^{4}\,lbf$$

Part II-F
Final Check for Equilibrium

CHECK AGAINST SLIDING

$$SEISMIC_{hor} := \frac{C_s \cdot W_{tot}}{1.g} = 3.696 \times 10^{3} \cdot lbf$$

$$Seismic_{ver} := \frac{E_{vert} \cdot W_{tot}}{1.g} = 2.957 \times 10^{3}\,lbf$$

$$Hydro_{stdyn} := PW_{total} \cdot 6.67 \cdot ft = 8.733 \times 10^{3}\,lbf$$

Driving force $P_{dr} := SEISMIC_{hor} + Hydro_{stdyn} = 1.243 \times 10^{4}\,lbf$

active pressure coefficient

Assume $PHI := 30.deg$

$$PHI_{rad} := 30 \cdot \frac{\pi}{180} = 0.524 \cdot rad$$

$K_a := .333$ $\quad K_p := 3$

$$C_c := \cos\left(PHI_{rad}\right) = 0.866$$

For seismic

$$C_{sin} := \sin(PHI_{rad}) = 0.5$$

$$K_{ae} := \frac{\left(1 + \frac{E_{vert}}{1 \cdot g}\right) \cdot C_c{}^2}{\left(1 + C_{sin}\right)^2} = 0.355$$

$$K_{pe} := \frac{\left(1 + \frac{E_{vert}}{1 \cdot g}\right) \cdot C_c{}^2}{\left(1 - C_{sin}\right)^2} = 3.199$$

Assume both active/passive pressure has the inverted pressure distribution. Conservative.
Net pressure coefficient

$$Net_{aepe} := K_{pe} - K_{ae} = 2.844$$

Net Resisting active/passive force

$$Net_{aepeforce} := .5 \cdot Net_{aepe} \cdot 67.6 \cdot pcf \cdot (3 \cdot ft)^2 \cdot 6.67 \cdot ft = 5.77 \times 10^3 \cdot lbf$$

$$Fric_{coef} := 0.5$$

Resisting force

$$Res_{force} := Fric_{coef} \cdot W_{tot} + Net_{aepeforce} = 2.804 \times 10^4 \, lbf$$

$$Res_{force1} := Fric_{coef} \cdot W_{tot} = 2.227 \times 10^4 \, lbf$$

$$\boxed{SF_{sliding} := \frac{Res_{force}}{P_{dr}} = 2.256} \qquad \boxed{SF_{sliding1} := \frac{Res_{force1}}{P_{dr}} = 1.791}$$

Minimum

CHECK OVERTURNING

$$SEISMIC_{hor} = 3.696 \times 10^{3}\,lbf \qquad Hydro_{stdyn} = 8.733 \times 10^{3}\,lbf$$

$$MOMARM_{S} := 7.ft \qquad MOMARM_{H} := H_{West} - h_{West} + 5.ft = 5\,ft$$

Driving Mpment

$$MOM_{DRsei} := SEISMIC_{hor} \cdot MOMARM_{S} + Hydro_{stdyn} \cdot MOMARM_{H} = 6.954 \times 10^{4}\,ft \cdot lbf$$

Restring Moment

$$RESTORING_{M} := W_{tot} \cdot \frac{7.167ft}{2} = 5.135 \times 10^{6}\,\frac{ft^{2} \cdot lb}{s^{2}}$$

$$\boxed{SF_{Mom} := \frac{RESTORING_{M}}{MOM_{DRsei}} = 2.295}$$

CHECK BASE SHEAR

$P_{dr} = 1.243 \times 10^{4}\,lbf$ Horizontal Base shear

$$Area_{tot} := \left[\left(5 + \frac{4}{12}\right) \cdot ft \cdot \frac{10}{12} \cdot ft \cdot 2 + \left(5 + \frac{10}{12}\right) \cdot ft \cdot \frac{10}{12} \cdot ft \cdot 2\right] = 18.611\,ft^{2}$$

$$Base_{Shear} := \frac{P_{dr}}{Area_{tot}} = 4.638\,psi$$

$$f_{cprime} := 3000 \cdot psi$$

$$Shear_{allow} := 1.1 \cdot \frac{\sqrt{f_{cprime}}}{1.\sqrt{psi}} \cdot 1.psi = 60.249\,psi$$

$$\boxed{SF_{shear} := \frac{Shear_{allow}}{Base_{Shear}} = 12.991}$$

Because of the stubbiness of the structure, moment check is not necessary by judgment.

Part III - Analyses for Underground Piping

CHECK BURIED PIPING 48" DIAMETER

The thickness of pipe is not available. Unit wt is available. Derive thickness of pipe

$$R_{pipe} := \frac{48}{2} \cdot in = 2\,ft$$

The density of HDPE can range from 0.93 to 0.97 g/cm3 or 970 kg/m3.

$$HDPE_{den} := 0.95 \cdot \frac{gm}{cm^3} = 59.307\,\frac{lb}{ft^3}$$

$$t := \frac{90 \cdot \frac{lbm}{ft}}{2 \cdot \pi \cdot R_{pipe} \cdot HDPE_{den}} = 0.121 \cdot ft \qquad t = 1.449 \cdot in$$

$$\varepsilon_c := 0.175 \cdot \frac{t}{R_{pipe}} = 0.011$$

This

Stiff Soil

Table 11.1-1 ASCE

Assume Moment Magnitude =6.5

$$Ratio_{PGVPGA} := 109 \cdot \frac{\frac{cm}{sec}}{g}$$

$$PGA = 0.213 \cdot g$$

$$PGV := Ratio_{PGVPGA} \cdot PGA = 23.217 \cdot \frac{cm}{sec}$$

$$C_{sasce} := 2 \cdot \frac{km}{sec} = 6.562 \times 10^3\,\frac{ft}{s}$$

Apparent wavelength of seismic waves at ground surface approx 2 km/sec

$$\alpha := 1$$

equal to 2 for shear waves and 1 otherwise assumed 1 for maximum

$$\varepsilon_{asce} := \frac{PGV}{\alpha \cdot C_{sasce}} = 0.00012$$

The axial strains produced by Equation (11-1) can be assumed to be transferred to the pipeline but need not be taken as larger than the axial strain induced by friction at the soil pipe interface:

$$\varepsilon_a = T_u \lambda/(4AE)$$

where

T_u = peak friction force per unit length at soil-pipe interface

λ = apparent wavelength of seismic waves at ground surface, sometimes assumed to be 1.0 kilometers without further information

A = Pipe cs area

E = Modulus elastic

For our case, with cohesionless backfill, the peak force per unit length of the soil-pipe interface (from Appendix B) is:

$$T_u = \frac{\pi}{2} D H \bar{\gamma} (1 + K_o) \tan\delta$$

$$D_{pipe} := 48 \cdot in$$

$$H_{BurDEP} := (399 - 385.5) \cdot ft + \frac{D_{pipe}}{2} = 15.5\, ft$$

$$\gamma_{soil} := 120 \cdot pcf \qquad K_0 := 1$$

$$\delta_{wall} := .8 \cdot 30 \cdot deg = 24 \cdot deg \qquad \tan(\delta_{wall}) = 0.445$$

$$\lambda := 1 \cdot km = 3.281 \times 10^3\, ft$$

$$T_u := \frac{\pi}{2} \cdot D_{pipe} \cdot H_{BurDEP} \cdot \gamma_{soil} \cdot (1 + K_0) \cdot \tan(\delta_{wall}) = 1.041 \times 10^4 \cdot \frac{lb}{ft}$$

$$E_{pipe} := 2 \cdot 10^6 \cdot psi \qquad Area_{pipe} := \left[D_{pipe}^{\ 2} - (D_{pipe} - 2 \cdot t)^2\right] \cdot \frac{\pi}{4} = 1.472\, ft^2$$

$$\varepsilon_a := \frac{T_u \cdot \lambda}{4 \cdot Area_{pipe} \cdot E_{pipe}} = 0.02 \qquad \varepsilon_a > \varepsilon_{asce} \qquad \textbf{OK}$$

Alternate evaluation using "Design Guideline for seismic Resistant Water pipeline Insrtallations", John Eidinger

The Guidelines provide three approaches can be used in the design of buried pipelines:

•Chart method. The simplest approach. Avoids all mathematical models, and allows the designed to pick a style of pipe installation based on parameters such as regional maps for PGV and PGD hazards, and the pipeline function class.

• Equivalent static method. Uses simple quantifiable models to predict the amount of stress, strain and displacement on a pipe for a particular level of earthquake loading.

• Finite element method. This method uses finite element models to examine the seismic loads (whether PGA, PGV or PGD) over the length of the pipeline, and then uses beam on inelastic foundation finite element models (or sometimes use two- or threedimensional mesh models) to examine the state of stress and strain and displacement within the pipeline and pipeline joints.

We select Chart method.

Conditions to meet:

"• Deliver water at serviceable pressure to 65% to 90% of all hydrants within the first hours after the earthquake, as long as there are adequate supply sources; and

• Deliver water via the pipe network to at least 90% of all customers within 3 days following an earthquake;"

These conditions can be met.

Define function classification:

Function	Seismic Importance	Description
I	Very Low to None	Pipelines that represent very low hazard to human life in the event of failure. Not needed for post earthquake system performance. response. or recovery. Widespread damage resulting in long restoration times (weeks or longer) will not materially harm the economic well being of the community.
II	Ordinary. Normal	Normal and ordinary pipeline use. common pipelines in most water systems. All pipes not identified as Function I. III. or IV.
III	Critical	Critical pipelines and appurtenances serving large numbers of customers and present significant economic impact to the community or a substantial hazard to human life and property in the event of failure.
IV	Essential	Essential pipelines required for post-earthquake response and recovery and intended to remain functional and operational during and following a design earthquake.

Table 1. Pipe Function Classifications

We select Class I because the probability of human impact is very low.

The seismic geotechnical analyses indicate there will not be any permanent displacement along the slope of the dike. There is also indication that the dike does not transverse a known fault and will not liquefy. The only load is ground shaking due to seismic waves. A single Design Category defines the earthquake loading.

Inch/sec	Function I	Function II	Function III, IV
$0 < PGV \leq 10$	A	A	A
$10 < PGV \leq 20$	A	A	A
$20 < PGV \leq 30$	A	A	A (with additional valves)
$30 < PGV$	A	A (with additional valves)	B

Table 6. Distribution Pipelines – Ground Shaking

$$\mathbf{PGV} = 9.141 \cdot \frac{\mathbf{in}}{\mathbf{sec}}$$

Design Category = A

Design Category	Cost Effective Design Approach	Notes
A	Standard	
B	Butt Fusion Joints	
C	Butt Fusion Joints	
D	Butt Fusion Joints	
E	Butt Fusion Joints	

Table 17. HDPE Pipe

A standard design approach is in sync with the earlier determination.

Part IV - Condition of the Units and Actions Needed

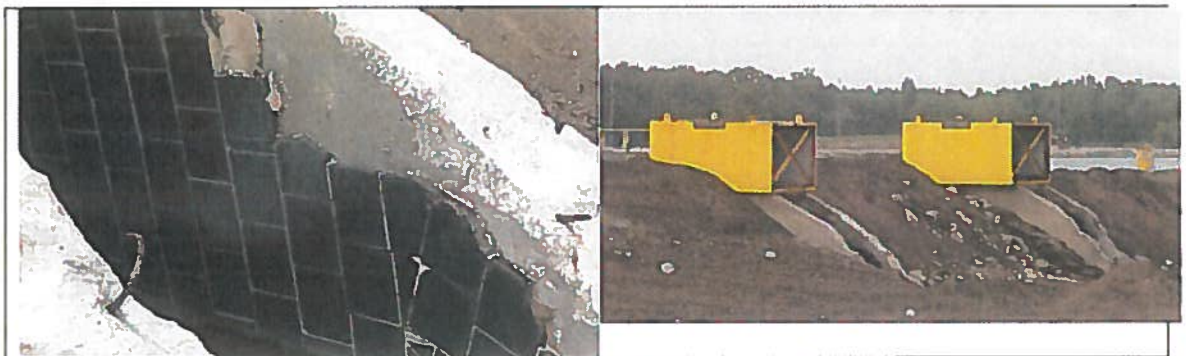
The study reported above is not a new construction. Therefore, a visual inspection of the site was also conducted to ascertain that the structure has not become visibly distressed. The results of that inspection are summarized below.

In general, most units were found to be in decent condition except the following:

- Energy Dissipator Troughs and Covers
- Wooden Skimmer

The actions for maintenance and immediate corrective measures are discussed below.

Institute an inspection program to monitor all structural units included in this list and any other items that, in the event of its failure, would crucially affect plant operation during and/or after an earthquak

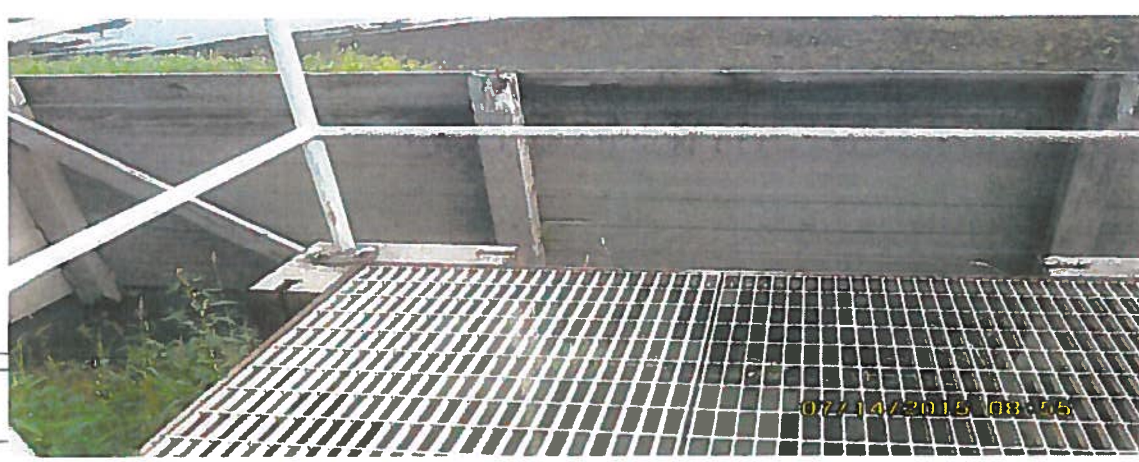


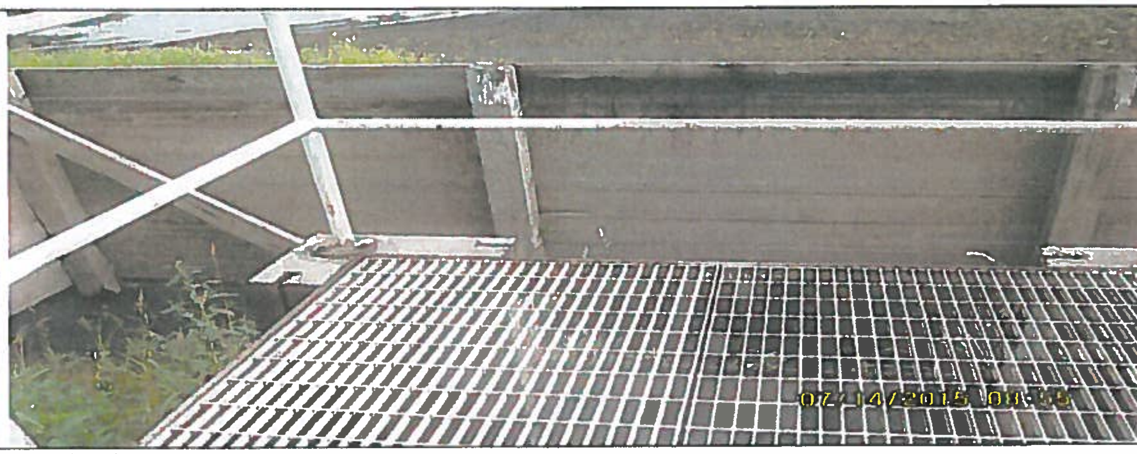
These actions may be implemented within the next 2-3 years from the date of this revision. SL 9/22/18
REV. 1

ACTION 1

The yellow boxes on top of the structure are not anchored and must be provided with anchors or replaced with a different anchored structure.

Also, the concrete inflow box is badly deteriorated and should be replaced with a like structure but with a better corrosion protection cladding.





ACTION 2
The skimmer is a wooden wall that presently is deemed non-effective. Either a more sturdy wall or a maintenance program needs to be initiated.

ACTION 3

Inspect the 48-inch diameter fiberglass/HDPE pipes to verify that the pipes are not distressed inside and out.

ACTION 4

Finally, institute a maintenance prigram that will periodically inspect the structural units against any degeneration.

CONCLUSION

1. Based on a typical configuration, the seismic analyses of the structures are judged to meet local seismic requirements with the following exception.

2. Some of the units are found to be deteriorated and should be remediated by either returning them to original configurations or replaced by new units.

ATTACHMENT 1

 ATTACHMENT 1

USGS Design Maps Summary Report

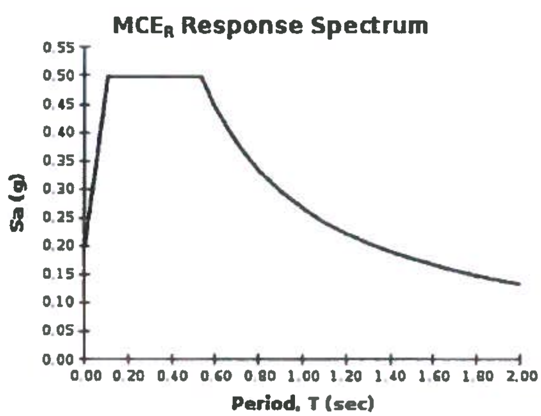
User–Specified Input

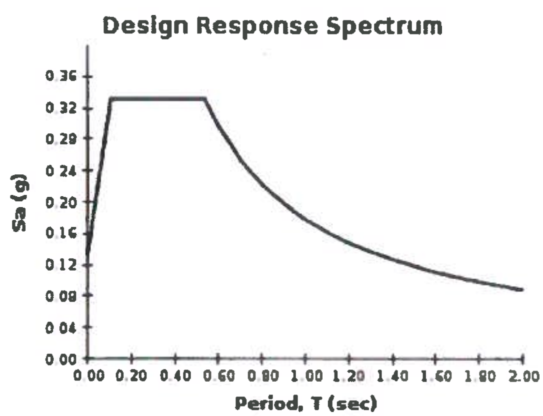
Report Title	USGS Data Sat August 27, 2016 14:53:43 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	37.92556°N, 87.03389°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Risk Category	I/II/III

USGS–Provided Output

$S_S =$ 0.415 g	$S_{MS} =$ 0.498 g	$S_{DS} =$ 0.332 g
$S_1 =$ 0.163 g	$S_{M1} =$ 0.267 g	$S_{D1} =$ 0.178 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (37.92556°N, 87.03389°W)

Site Class C – "Very Dense Soil and Soft Rock", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	S_S = 0.415 g
From Figure 22-2 [2]	S_1 = 0.163 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class C, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index PI > 20, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = C and S_S = 0.415 g, F_a = 1.200

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = C and S_1 = 0.163 g, F_v = 1.637

Equation (11.4-1): $S_{MS} = F_aS_S = 1.200 \times 0.415 = 0.498$ g

Equation (11.4-2): $S_{M1} = F_vS_1 = 1.637 \times 0.163 = 0.267$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

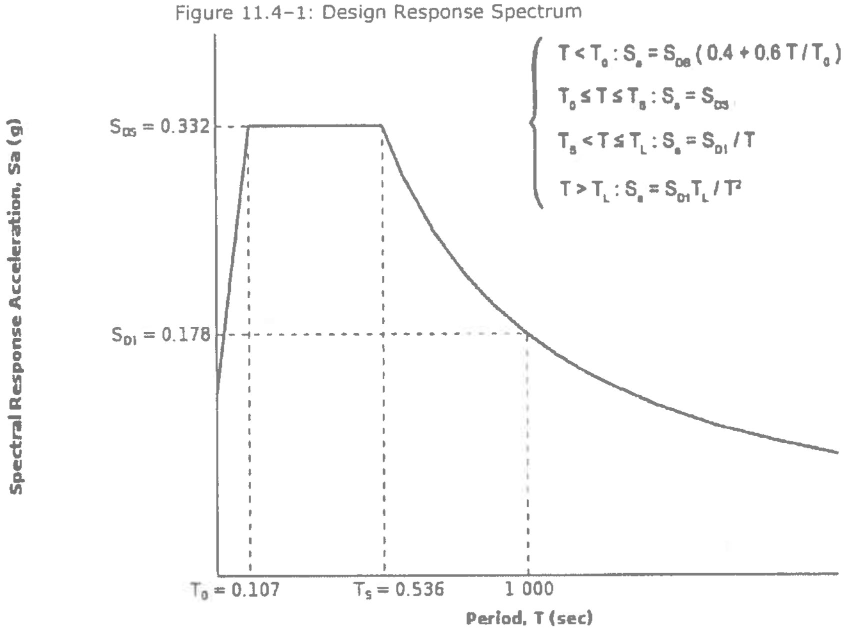
Equation (11.4-3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 0.498 = 0.332$ g

Equation (11.4-4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 0.267 = 0.178$ g

Section 11.4.5 — Design Response Spectrum

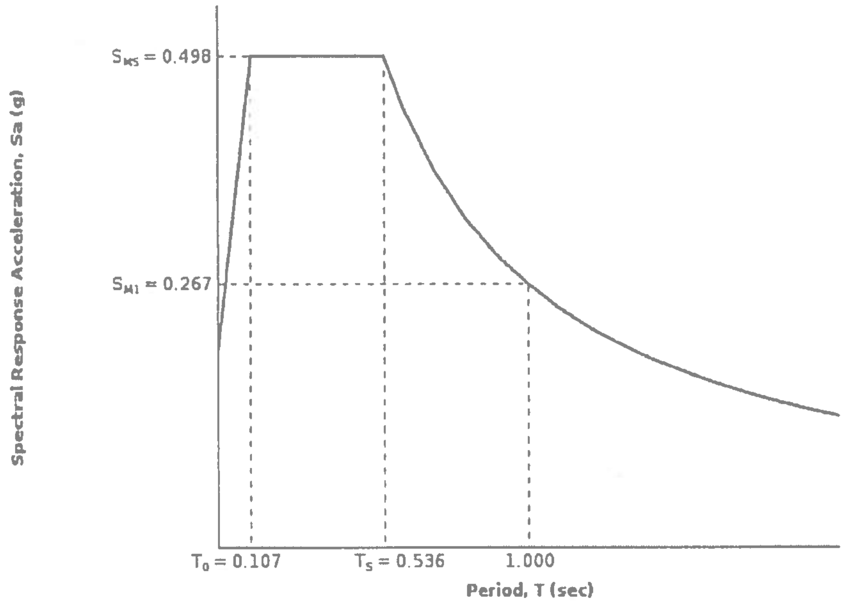
From Figure 22-12 [3] T_L = 12 seconds

Figure 11.4-1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.213

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.187 \times 0.213 = 0.253$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = C and PGA = 0.213 g, F_{PGA} = 1.187

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.876

From Figure 22-18 [6] C_{R1} = 0.842

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.332 g, Seismic Design Category = C

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.178 g, Seismic Design Category = C

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = C

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

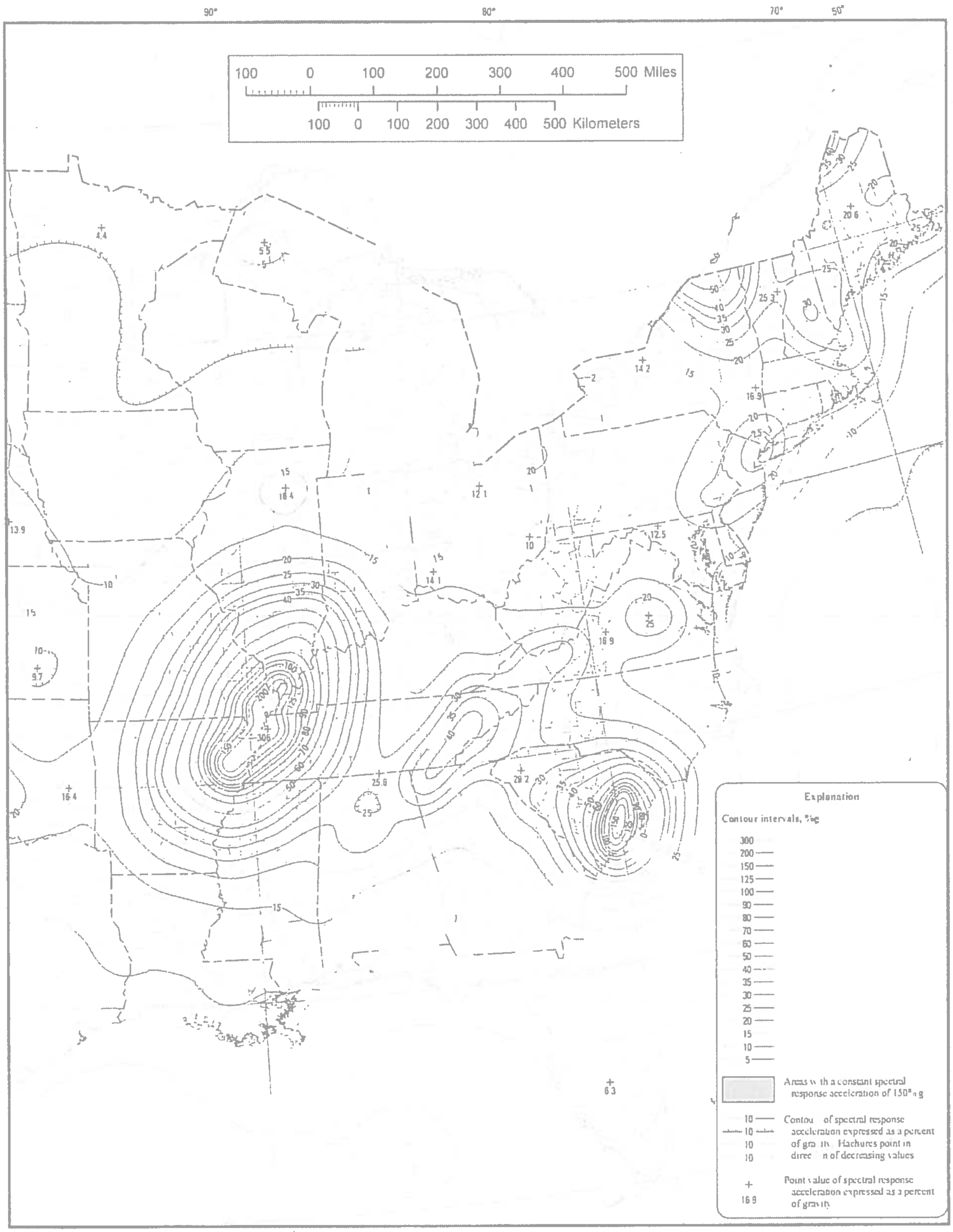
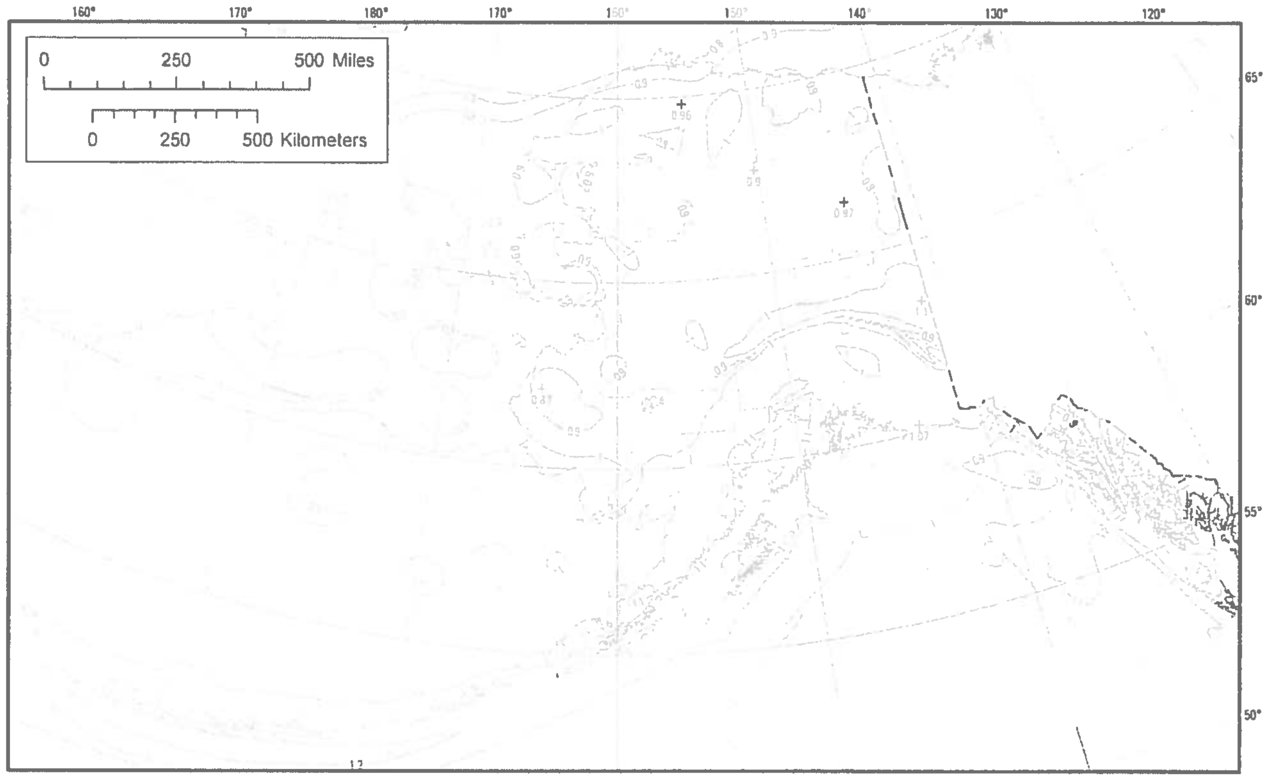
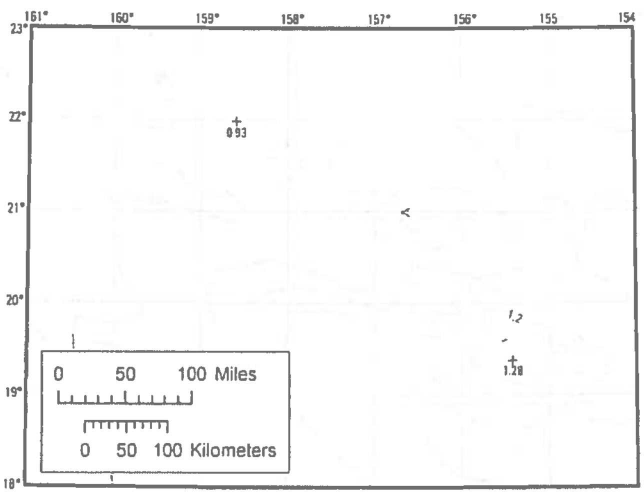


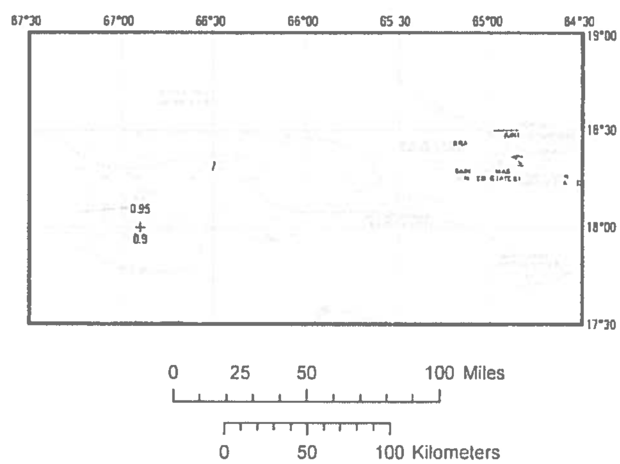
Figure 22-1 *(continued)* S_s Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for 0.2 s Spectral Response Acceleration (5% of Critical Damping), Site Class B.

Figure 22-12 (*continued*) Mapped Long-Period Transition Period, T_L (s), for the Conterminous United States.

Figure 22-17 Mapped Risk Coefficient at 0.2 s Spectral Response Period, C_{RS}.



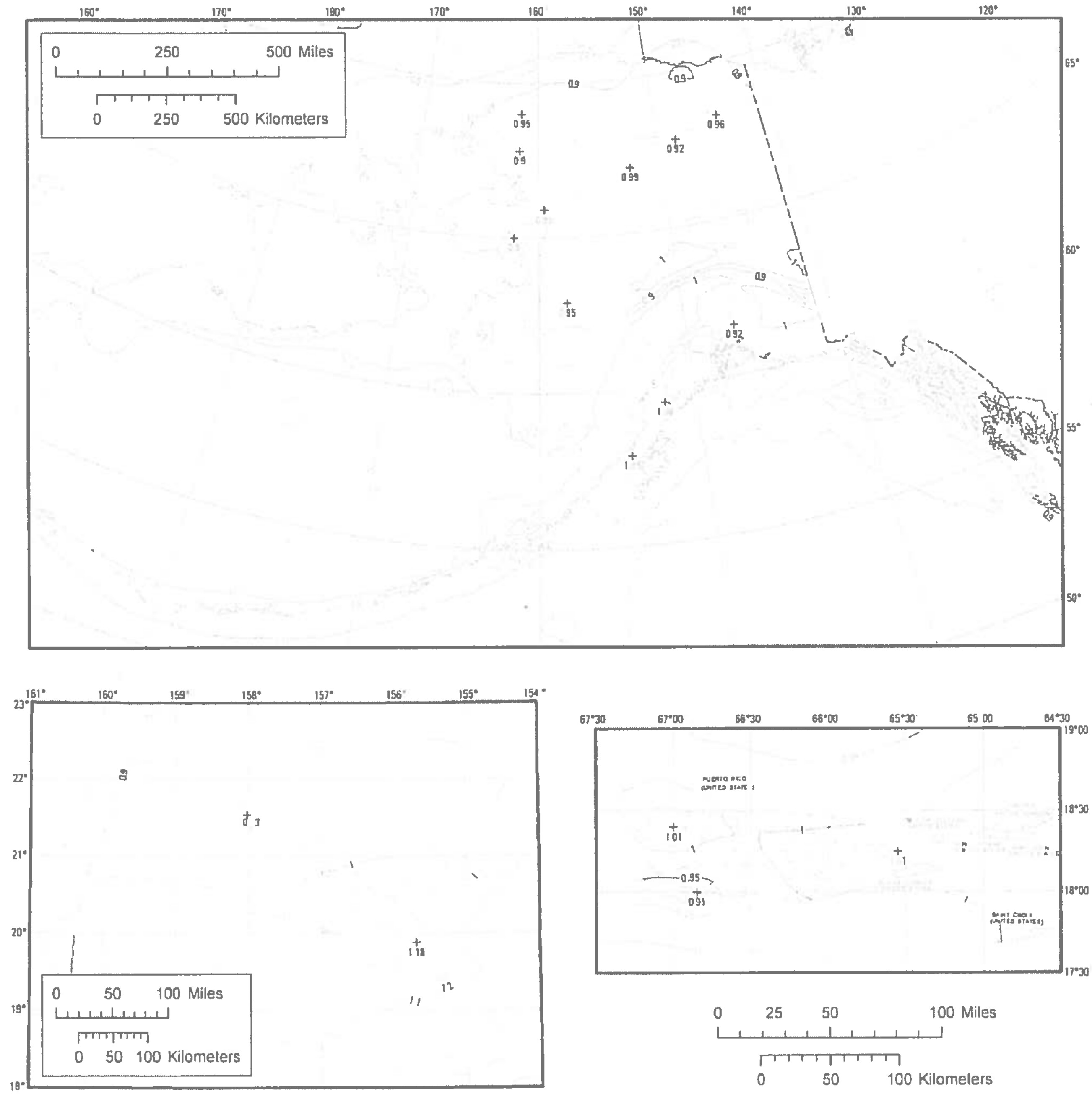




Notes:
- Maps prepared by United States Geological Survey (USGS).
- Larger, more detailed versions of these maps are not included because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/designmaps/) be used to determine the mapped value for a specified location.

Figure 22-17 (*continued*) Mapped Risk Coefficient at 0.2 s Spectral Response Period, C_{RS}.

Figure 22-18 Mapped Risk Coefficient at 1.0 s Spectral Response Period, C_{R1}.



Notes:
- Maps prepared by United States Geological Survey (USGS).
- Larger, more detailed versions of these maps are not included because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/designmaps/) be used to determine the mapped value for a specified location.

Figure 22-18 (*continued*) Risk Coefficient at 1.0 s Spectral Response Period, C_{R1}.

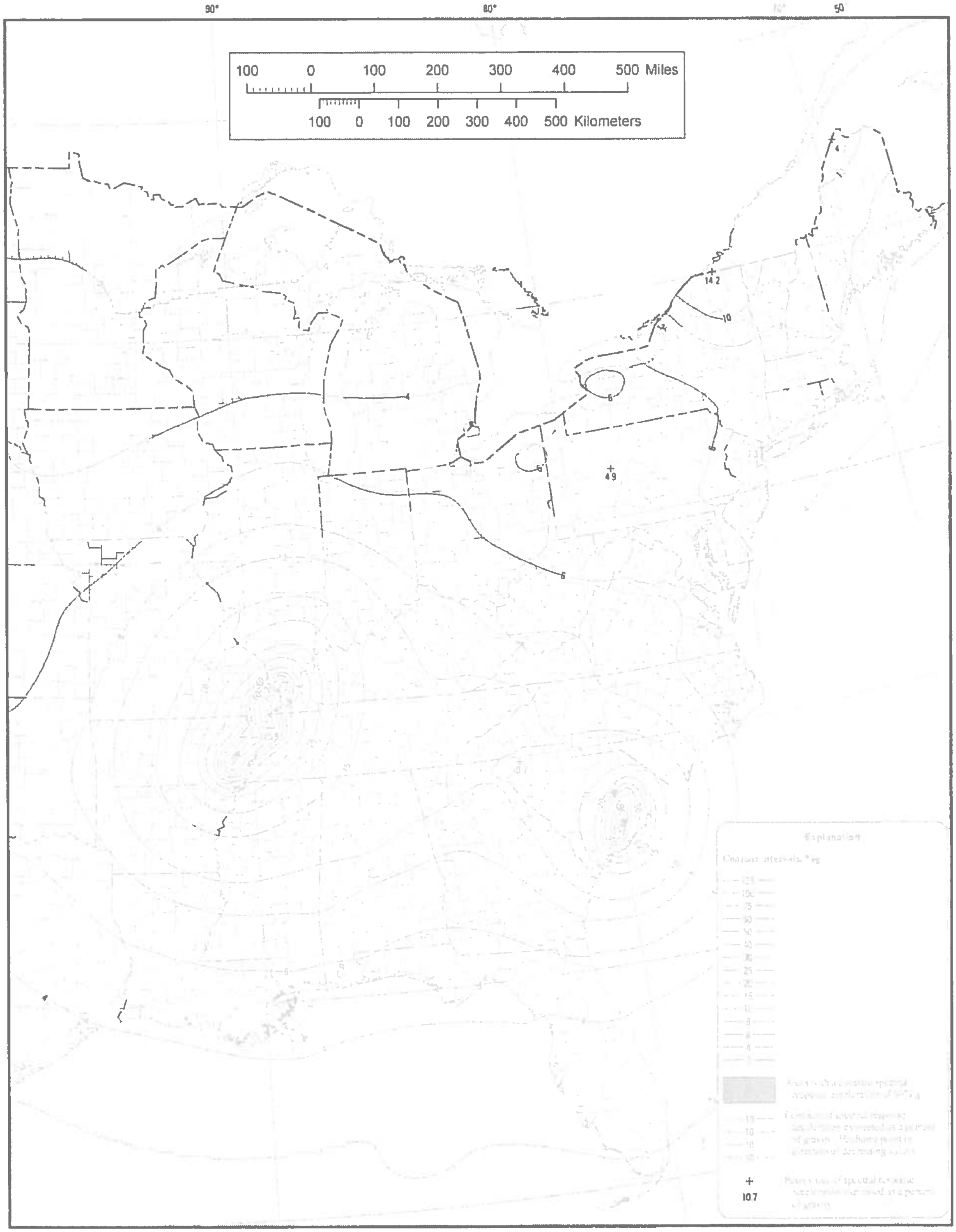


Figure 22-2 (*Continued*) S_1 Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for 1.0 s Spectral Response Acceleration (5% of Critical Damping), Site Class B.

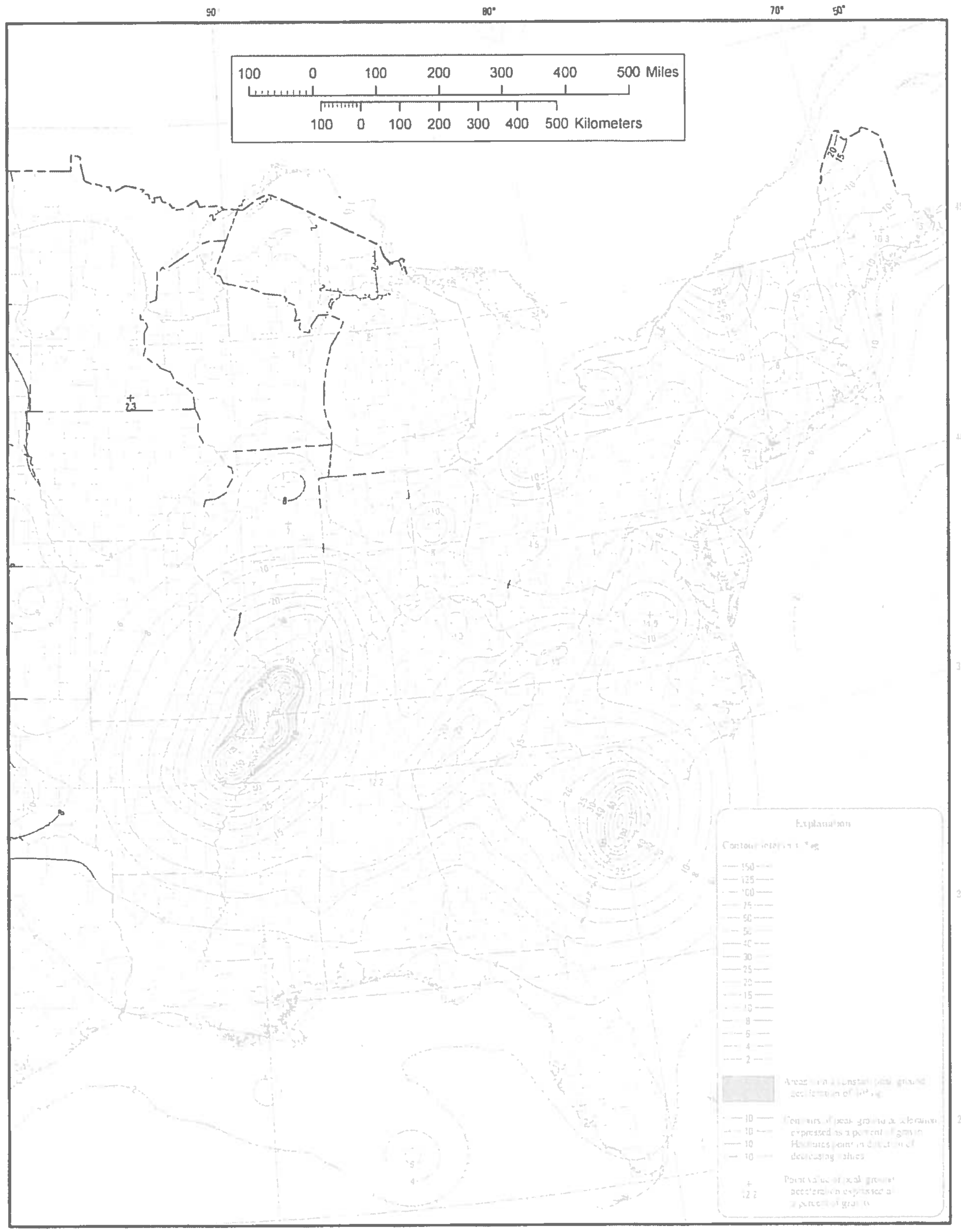


Figure 22-7 (*continued*) Maximum Considered Earthquake Geometric Mean (MCE_G) PGA, %g, Site Class B for the Conterminous United States.

ATTACHMENT 2

ENGINEERING DEPT.
AMERICAN ELECTRIC POWER SERVICE CORP.
1 RIVERSIDE PLAZA
COLUMBUS, OHIO

DATE 11/14/16 SHEET 30 OF ___
BY JAR CK. ___
COMPANY AEP G.O. ___
PLANT ROCKPORT

SUBJECT CCR COMPLIANCE CHECK

CALCULATE HYDRODYNAMIC FORCES & CHECK STABILITY OF DISCHARGE STRUCTURE

$H_{EAST} = 394 - 377 = 17'$ $PGA = 0.213g$ PG 27

$h_{EAST} = 394 - 388 = 6'$ $K_{EAST} = \frac{2PGA}{3.0} = \frac{2}{3}(.213) = 0.142$

PG 27 $p_d = 0.875\,\rho_w g k \sqrt{Hh}$

$= 0.875 \times 62.4 \times .142 \sqrt{17 \times 6} = 78.3\,psf$

$P_d = \frac{2\,h_{EAST}\,p_d}{3} = \frac{2(6.0)}{3}(78.3) = 313\ lbf/ft$

$DL\ WALLS = \left[6.17 \times \frac{10}{12} \times 2 \times 12.0 + 5.0 \times 6.0 \times \frac{10}{12} + 2(.75) \times 6.0 \times \frac{10}{12} + (5.0 \times 12 - 2.0^2\pi)\frac{10}{12}\right]150$

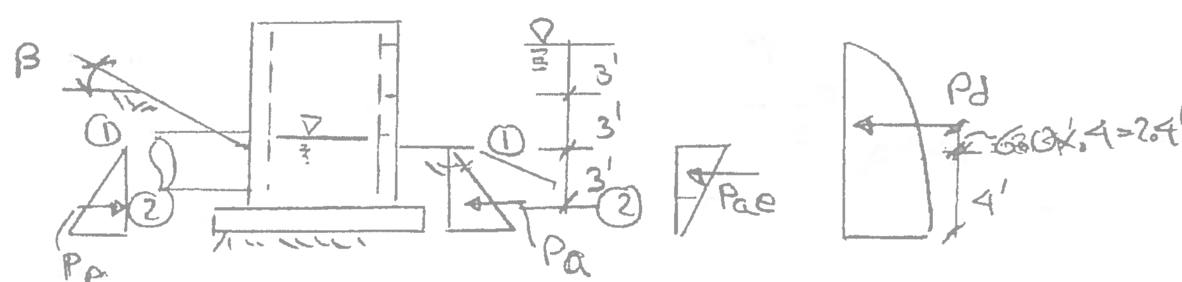
$= 29315\ \#$

$DL\ SLAB = 7.167 \times 7.067 \times 1.0 \times 150 = 8242\ \#$

$W_{TOT} = 29315 + 8242 = 37557\ lb$

$SEISMIC_{HOR} = C_S W_{TOT} = .083(37557) = 3117\ \#$

$SEISMIC\ VERT = E_{VERT} W_{TOT} = 0.066(37557) = 2479\ \#$



72223C1000

ENGINEERING DEPT.
AMERICAN ELECTRIC POWER SERVICE CORP.
1 RIVERSIDE PLAZA
COLUMBUS, OHIO

SHEET 39 OF
DATE 11/14/16 BY JAR CK.
COMPANY AEP G.O.
PLANT ROCKPORT

SUBJECT CCR COMPLIANCE CHECK

$\phi = 30^\circ$

$\beta = 0^\circ$ CONSERVATIVE

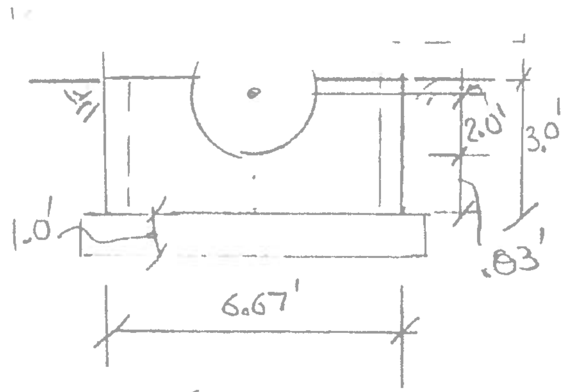
$\delta = 0^\circ$

$\gamma = 120pcf$

$\gamma_w = 62.4pcf$

SEE PG 42 FOR

K_a, K_{ae}, K_p, K_{pe}



$A_g = 4.0(6.67) = 26.68SF$

$A_{OUT} = 12.56 - 5.62 = 6.94SF$

$P_{ae} = (.478 - 0.355)(4 \times .0676) \times 1/2 \times 4.0 \times 6.67 = 0.42^K$

$P_a = 0.355(4 \times .0676) \times 1/2 \times 4.0 \times 6.67 = 1.28$

$P_p = 2.526 \times 4^2 \times .0676/2 \times \frac{26.68 - 6.94}{26.68} \times 6.67 = 6.74^K$

TOTAL SLIDING FORCE $= 1.28 + 0.313(6.67) + 3.12 + 0.42 = 6.91^K$

$P_{BOUYANT} = [7.17 \times 7.67 \times 1.0 + 6.67 \times 6.17 \times 9.0] \frac{62.4}{1000} = 26.5^K$

WATER IN SUMP: $5.0 \times 4.5 \times 4.0 \times .0624 = 5.62^K$

$\mu = 0.5$

SLIDING RESISTANCE $= (37.56 + 5.62 - 26.5)(.9)(1 - .066).5 + 6.74(.9)$
$= 13.08^K$

SAFETY FACTOR $= \frac{13.08}{6.91} = 1.89$

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$$M_{OV} = .313(6.67)(6.4) + 1.28\left(\frac{4.0}{3}\right) + .42\left(\frac{2 \times 4.0}{3}\right) + 25.59$$

EQ ON STRUCTURE

$= 41.78'^{-K}$

DL STRUCTURE

WATER IN STRUCTURE

P_P

$$M_R = (37.56 + 5.62 - 26.5).9(1 - .066)\left(\frac{7.17}{2}\right) + 6.79\left(\frac{4.0}{3}\right)$$

$= 59.25'^{-K}$

$$\text{SAFETY FACTOR} = \frac{59.25}{41.78} = 1.42$$

EAST BOTTOM ASH POND IS WORST CASE, DEPTH OF POND & DEPTH OF WATER ABOVE DISCHARGE WEIR IS LARGEST. CONSERVATIVELY WATER LEVEL IN STRUCTURE IS ASSUMED TO MATCH WASTEWATER POND ELEVATION OF 389 ft.

V_U = SHEAR LOAD ON WALLS $= 6.91^K$

$$\phi V_C = \frac{0.75 \times 2\sqrt{3000}\,(74)(10) \times 2}{1000} = 121.6^K >> V_U \text{ OK.}$$

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LATERAL PRESSURE ON WALLS @ TOP OF FILL

1-1 $= \dfrac{78.3 + 6.0(62.4)}{1000} = 0.453\,KSF$

ASSUME WALLS HAVE MINIMUM REINFORCEMENT

$A_s = \dfrac{0.002(12)(10)}{2} = 0.12\,in^2/ft$

$M_u = 5.83^2(.453)/8 = 1.93^{k\text{-}ft}/ft$ $\quad d = 10 - 2 - .625 - \dfrac{.625}{2} = 7.06''$

$\phi M_n = \dfrac{0.9(.12)(60)}{12}\left(7.06 - \dfrac{.59(.12)(60)}{3.0(12)}\right) = 3.75^{k\text{-}ft}/ft > M_u$

O.K.

2-2 $9.0 \times .0624 + .355(4.0 \times .0676) = 0.658\,KSF$

$M_u = 5.83^2(.658)/8 = 2.80^{k\text{-}ft} < 3.75^{k\text{-}ft}$ O.K.

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$$K_a = \frac{(1+K_v)\cos^2\phi}{(1+\sin\phi)^2} = 0.355$$

$$\phi = 30^\circ$$

$$K_p = \frac{(1-K_v)\cos^2\phi}{(1-\sin\phi)^2} = 2.802$$

$$\theta' = \tan^{-1}\left(\frac{K_h}{1 \pm K_v} \cdot \frac{\gamma}{\gamma - \gamma_w}\right)$$

$$K_h = 0.083$$
$$K_v = 0.066$$

$$\theta' = 9.214^\circ$$
$$10.489^\circ$$

$$K_{ae} = \frac{(1+K_v)\cos^2(\phi-\theta-\alpha)}{\cos\theta\cos^2\alpha\cos(\alpha+\delta+\theta)\left(1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta-\theta)}{\cos(\alpha+\delta+\theta)\cos(\alpha-\beta)}}\right)^2}$$

$$= \frac{(1+.066)\cos^2(30-9.214)}{\cos 9.214\cos^2 0\cos(0+0+9.214)\left(1+\sqrt{\frac{\sin(30)\sin(20.786)}{\cos(9.214)\cos 0}}\right)^2}$$

$$K_{ae} = \frac{0.932}{1.976} = 0.472$$

$$K_{pe} = \frac{(1-K_v)\cos^2(\phi+\alpha-\theta)}{\cos\theta\cos^2\alpha\cos(\alpha-\delta-\theta)\left(1-\sqrt{\frac{\sin(\phi+\delta)\sin(\phi+\beta-\theta)}{\cos(\alpha-\delta+\theta)\cos(\alpha-\beta)}}\right)^2}$$

$$= \frac{(1-.066)\cos^2(20.786)}{\cos 9.214\cos^2 0\cos(-9.214)\left(1-\sqrt{\frac{\sin 30\sin(20.786)}{\cos(-9.214)\cos 0}}\right)^2}$$

$$K_{pe} = \frac{0.816}{0.323} = 2.526$$

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SUBJECT CCR COMPLIANCE CHECK

CHECK UNDERGROUND PIPING

PIPE $\sigma_y = 3300$ psi

$L = 41.5'$

$$\sigma_{DESIGN} = \frac{T_u L}{2\pi D t} = \frac{10410 \times 465}{2\pi 48 \times 1.45} = 900 \text{ psi}$$

O.K.

$$\epsilon_a = \frac{T_u L}{\pi D t E}\left(1 + \cancel{\frac{N}{(1+r)}\left(\frac{T_u L}{2\pi D t \sigma_y}\right)^r}^{\,1.0}\right)$$

N = 10
r = 100

$$\epsilon_a = \frac{10410 \times 41.5}{2\pi(48)(1.45)(2\times10^6)} = 4.94\times10^{-4} < 0.02 \text{ O.K.}$$

(PG 32)

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ATTACHMENT 3

DESIGN GUIDELINE FOR SEISMIC RESISTANT WATER PIPELINE INSTALLATIONS

John Eidinger[1]

ABSTRACT

Seismic design for water pipelines is not explicitly included in current AWWA standards. Compounding this problem, standard water pipeline materials and installation techniques are prone to high damage rates whenever there is significant permanent ground deformations or excessively high levels of ground shaking.

To help improve this situation, a new Design Guideline for Seismic Resistant Water Pipeline Installations (the Guidelines) has been developed. It is intended that the Guidelines be issued in March 2005. For the period from November 2004 through January 2005, the Guidelines are available in draft form for public comment. Comments from U.S., Japanese, Canadian and all other water utilities, pipeline manufacturers, AWWA, JWWA and other interested parties are welcomed.

The Guidelines provide direction for three situations:

- When the pipeline engineer has just rough estimates of the earthquake hazard, does not have the resources to do design by analysis, and wishes to rely on standardized pipeline components. The Guidelines provide the Chart Method. This is the preferred approach for common pipeline installations like 6-inch to 8-inch diameter pipes, fire hydrants and service laterals.

- When the pipeline engineer wishes to perform a limited design by analysis. The Guidelines provide the Equivalent Static Method. This is the preferred approach for medium important pipelines like 12-inch to 24-inch installations, or as a preliminary approach for major transmission pipelines.

- When the pipeline engineer has the resources to perform detailed subsurface investigations, geotechnical engineering and pipe stress analyses. The Guidelines provide the Finite Element Method. This is the preferred approach for essential non-redundant installations, like 36-inch to 120-inch pipelines.

INTRODUCTION

In most every severe earthquake, the largest negative impact to water utilities has been the damage to buried water pipelines. At the past three JWWA-AWWARF workshops (Oakland

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2000, Tokyo 2001, Los Angeles 2003), a great emphasis was placed by many participants on the rate of pipe damage, the causes of pipe damage, and the improved earthquake performance of new types of pipe.

After the Los Angeles workshop, many US participants got together and decided something ought to be done about this. Accordingly, in concert with FEMA, NIBS and the ALA, a team of engineers was assembled to put together the first ever US seismic design guideline for buried water pipelines. The American Lifelines Alliance (ALA) was formed by the Federal Emergency Management Agency (FEMA) in 1998 as a public-private partnership whose goal is to reduce risk to utility and transportation systems from natural hazards and manmade threats. In 2002, FEMA contracted with the National Institute of Building Sciences (NIBS) through its Multihazard Mitigation Council (MMC) to, among other things, assist FEMA in developing these Guidelines. The ALA sponsors this work through funding from NIBS and FEMA.

AmericanLifelinesAlliance

U.S. DEPARTMENT OF HOMELAND SECURITY FEMA National Institute of BUILDING SCIENCES

AUTHORS

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Mr. Alex Tang	Nortel Networks, Retired
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The Guidelines would not have been possible without the contributions from numerous staff of the San Francisco Public Utilities Commission, East Bay Municipal Utilities District, City of San Diego Water Department, the Los Angeles Department of Water and Power, and many other participating agencies.

OUTLINE OF THE GUIDELINES

The Guidelines describe the various steps in seismic water pipeline design, with commentary. The main topics included are: Goals; Performance Objectives; Earthquake Hazards; Subsurface Investigations; General Pipeline Design; Analytical Models; Transmission Pipelines; Bypass Pipelines; Distribution Pipelines; Service and Hydrant Laterals; Distribution Pipelines; and Other Components. The Guidelines are meant to be a self-standing document that can be used by pipeline designers in water utilities; as such, it is geared to provide simple procedures to achieve the overall goal. The Guidelines always allow for more detailed procedures to be used by geologists, geotechnical engineers and pipeline engineers when suitable. A link to obtain the entire draft Guidelines is listed in the Conclusions.

For the 4th AWWARF-JWWA workshop, four papers cover the major topic areas of the Guidelines. This paper describes performance goals and the design-by-chart method. The paper by Dr. Craig Davis covers reliability goals and definition of geotechnical hazards. The paper by Mr. Luke Cheng covers design issues for transmission pipelines. The paper by Mr. Bruce Maison covers the two design-by-analysis models and design issues for service laterals.

GOAL OF SEISMIC DESIGN FOR WATER PIPELINES

The goal of the Guidelines is to improve the capability of water pipelines to function and operate during and following design earthquakes for life safety and economic reasons. This is accomplished using a performance based design methodology that provides cost-effective solutions and alternatives to problems resulting from seismic hazards. Improved water pipeline performance will help create a more resilient community for post-earthquake recovery; therefore portions of the Guidelines inherently consider the community impacts if pipeline damage were to occur. The Guidelines do not intend to prevent all pipelines from being damaged.

To achieve this goal, the fundamental intent of the Guidelines is to assure a reasonably low rate of water pipeline damage throughout a water utility system, such that about 90% of customers in a system can be restored with piped water service within about three days after a design basis earthquake.

To achieve this level of performance, an acceptable damage rate will be about 0.03 to 0.06 breaks per 1,000 feet (0.1 to 0.2 breaks per kilometer) of equivalent 6-inch diameter pipe. The commentary of the Guidelines provides a calculation to convert a network of pipes of different diameters that may suffer both breaks and leaks, in conjunction with network redundancy, into a single equivalent break rate per equivalent 6-inch diameter pipe. By minimizing pipeline damage after earthquakes to this level of damage, a typical water utility serving a population of 150,000 people could expect to:

- Deliver water at serviceable pressure to 65% to 90% of all hydrants within the first hours after the earthquake, as long as there are adequate supply sources; and

- Deliver water via the pipe network to at least 90% of all customers within 3 days following an earthquake;

as long as the utility can isolate most of the leaking and broken pipes within one day or so, and repair equivalent 6-inch diameter pipes at a rate of about 20 within the first three days after the earthquake, and 20 per day thereafter.

For water utilities with limited post-earthquake repair capability, or serving pipe networks with limited or no redundancy, it is important to limit the damage rate to the lower range. For water utilities with much greater post-earthquake repair capability, it might be acceptable to sustain damage to the higher range.

NEW INSTALLATIONS AND REPLACEMENT / RETROFIT

It is the intent of the Guidelines that they be used for all new pipeline installations. Over a period of many years, a sufficiently high percentage of pipelines in a network will eventually have been designed per these Guidelines. Thus, it may take decades for some utilities to ultimately achieve the goals, unless a pipeline replacement / retrofit program is also adopted.

The decision to replace older pipes is a complex one. In many networks, many existing pipelines (such as cast iron pipe with caulked joints) will not meet the seismic design capability recommended by the Guidelines. Still, the Guidelines do not recommend replacing older 4-inch to 10-inch diameter cast iron pipes solely on the basis of earthquake improvement, since this is not thought to be cost effective. However, as old pipeline are thought to need replacement because they no longer provide adequate fire flows, or have been observed to require repair at a rate of more than once every 5 years, then the added benefit of improved seismic performance may justify pipe replacement. When replaced, the new pipes should be designed per the Guidelines.

Replacement of larger diameter pipelines (12-inch diameter and upwards) may be cost effective just from a seismic point of view, in areas prone to PGDs.

PIPELINE FUNCTION CLASSES

A pipeline's function within the system identifies its importance in achieving the system performance goal. Table 1 provides the 4 function classes. A pipe function identifies a performance objective of an individual pipe, but not that of an entire system.

Function	Seismic Importance	Description
I	Very Low to None	Pipelines that represent very low hazard to human life in the event of failure. Not needed for post earthquake system performance, response, or recovery. Widespread damage resulting in long restoration times (weeks or longer) will not materially harm the economic well being of the community.
II	Ordinary, Normal	Normal and ordinary pipeline use, common pipelines in most water systems. All pipes not identified as Function I, III, or IV.
III	Critical	Critical pipelines and appurtenances serving large numbers of customers and present significant economic impact to the community or a substantial hazard to human life and property in the event of failure.
IV	Essential	Essential pipelines required for post-earthquake response and recovery and intended to remain functional and operational during and following a design earthquake.

Table 1. Pipe Function Classifications

THREE DESIGN APPROACHES

The Guidelines provide three approaches can be used in the design of buried pipelines.

- Chart method. The simplest approach. Avoids all mathematical models, and allows the designed to pick a style of pipe installation based on parameters such as regional maps for PGV and PGD hazards, and the pipeline function class.

- Equivalent static method. Uses simple quantifiable models to predict the amount of stress, strain and displacement on a pipe for a particular level of earthquake loading. The pipeline can then be designed to meet these quantified values, or pipe styles can be selected that presumably meet these quantified values without a formal capacity to demand check. Pipe selection is usually made by specification from available manufacturer's catalogs.

- Finite element method. This method uses finite element models to examine the seismic loads (whether PGA, PGV or PGD) over the length of the pipeline, and then uses beam on inelastic foundation finite element models (or sometimes use two- or three-dimensional mesh models) to examine the state of stress and strain and displacement within the pipeline and pipeline joints. Pipe design is often shown on contract drawings, covering material selection, joint preparation, trench design and other factors.

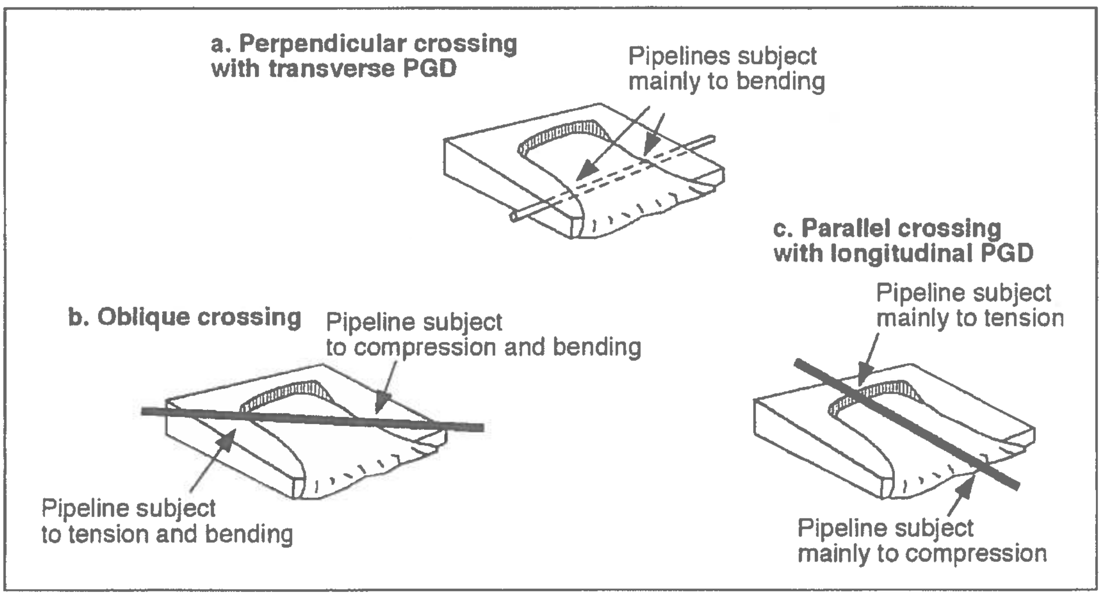


Figure 1. Direction of Permanent Ground Deformation (PGD)

CHART METHOD

Transmission Pipelines

Transmission pipelines may carry raw or treated water. Due to their importance to a great number of people, Function Class I is generally to be avoided except for those pipes whose failure would not impact any customer for 30 days or more.

Tables 2 to 5 set the pipeline design category (A, B, C, D or E). Figure 1 shows the meaning of perpendicular (transverse) and parallel (along the axis) orientations. If a portion of a pipeline has two or more categories for the various hazards (ground shaking, transverse PGDs, parallel PGDs, fault offset PGDs), then the highest category controls.

Inch/sec	Function I	Function II	Function III	Function IV
$0 < PGV \leq 10$	A	A	A	A
$10 < PGV \leq 20$	A	A	A	B
$20 < PGV \leq 30$	A	A	B	C
$30 < PGV$	A	B	C	D

Table 2. Transmission Pipelines – Ground Shaking

Inches	Function I	Function II	Function III	Function IV
$0 < PGD \leq 2$	A	A	A	A – welded steel B - segmented
$2 < PGD \leq 6$	A	A	A	B
$6 < PGD \leq 12$	A	A	B	C
$12 < PGD$	A	B	C	D

Table 3. Transmission Pipelines – Liquefaction and Landslide Transverse to Pipeline Alignment

Inches	Function I	Function II	Function III	Function IV
$0 < PGD \leq 2$	A	A	B	B
$2 < PGD \leq 6$	A	B	B	C
$6 < PGD \leq 12$	C	C	C	D
$12 < PGD$	D	D	D	E

Table 4. Transmission Pipelines -- Liquefaction (Lateral Spread) and Landslide Along Axis of Pipeline

Inches	Function I	Function II	Function III	Function IV
$0 < PGD \leq 2$	A	A	B	B
$2 < PGD \leq 6$	A	B	B	C
$6 < PGD \leq 12$	A	C	C	D
$12 < PGD \leq 24$	A	D	D	E
$24 < PGD$	A	D	E	E

Table 5. Transmission Pipelines – Fault Offset

Distribution Pipelines, Service Laterals and Fire Hydrant Laterals

In most cases, distribution pipelines are in networks. Failure of a single distribution pipeline will not fail the entire network (once the broken pipe is valved out), but the customers on the broken distribution pipeline will have no piped water service until the pipe is repaired. The engineer can assume that distribution pipelines are Function Class II, except in the following cases:

- The pipeline is the only pipe between lower elevation pump station and upper elevation pump station / reservoir in a pressure zone, and the failure of that pipeline will lead to complete loss of supply to the pump station serving a higher zone, or loss of the water in the reservoir for fire fighting purposes. For example, a 12-inch diameter pipe from lower elevation pump station that delivers water to a higher elevation tank within a pressure zone, and that also serves water to higher elevation pump stations.

- The pipeline is the only pipe delivering water to particularly important customers, such as critical care hospitals. For example, an 8-inch diameter pipe that has a service connection to a 200 bed hospital.

Past earthquakes have shown that there can be great quantity of damage to distribution pipelines, especially in areas prone to PGDs or high velocity pulses. While no single distribution pipeline is as important as a transmission pipeline, the large quantity of distribution pipe damage can lead to rapid system-wide depressurization, loss of fire fighting capability, and long outage times due to the great amount of repair work needed. Accordingly, we recommend that most distribution pipes be classified as Function Class II and very few as Function Class I (under ~5% of total pipeline inventory). A few distribution pipes serving essential facilities could be classified as Function III or IV; or they could be designated in suitable emergency response plans as prioritized for prioritized and rapid repair (generally under one day or two days at most). Once the Function Class is set, Tables 6 to 11 define the Design Category.

Inch/sec	Function I	Function II	Function III, IV
$0 < PGV \le 10$	A	A	A
$10 < PGV \le 20$	A	A	A
$20 < PGV \le 30$	A	A	A (with additional valves)
$30 < PGV$	A	A (with additional valves)	B

Table 6. Distribution Pipelines – Ground Shaking

Inches	Function I	Function II	Function III, IV
$0 < PGD \le 2$	A	A	A (with additional valves)
$2 < PGD \le 6$	A	A (with additional valves)	B
$6 < PGD \le 12$	A	B	C
$12 < PGD$	A	C	C

Table 7. Distribution Pipelines – Liquefaction and Landslide Transverse to Pipeline Alignment

Inches	Function I	Function II	Function III, IV
0 < PGD ≤ 2	A	A	B (with additional valves)
2 < PGD ≤ 6	A	B	C
6 < PGD ≤ 12	A	C	D
12 < PGD	A	D	D

Table 8. Distribution Pipelines – Lateral Spread and Landslide Along Axis of Pipeline

Inches	Function I	Function II	Function III, IV
0 < PGD ≤ 2	A	B	B
2 < PGD ≤ 6	A	B	C
6 < PGD ≤ 12	A	C	D
12 < PGD ≤ 24	A	D	E
24 < PGD	A	E	E

Table 9. Distribution Pipelines – Fault Offset

Service Laterals and Hydrant Laterals

Inch/sec	Any Lateral
0 < PGV ≤ 10	A
10 < PGV ≤ 30	A
30 < PGV	B

Table 10. Laterals – Ground Shaking

Inches	Any Lateral
0 < PGD ≤ 2	A
2 < PGD ≤ 12	B
12 < PGD	C

Table 11. Laterals – Liquefaction, Landslide and Surface Faulting

Design Categories

There are five design categories. Category A denotes standard (non-seismic) design. The following summarizes the general design approach for Categories B, C, D and E:

- B = restrained with extra valves
- C = B + better pipe materials
- D = C + quantified seismic design; or provide bypass system.
- E = D + peer review (it is strongly recommended that FEM method be used for any pipe with Classification E)

Tables 12 to 19 provide guidance for seismic pipe design using the chart method based on the categories A through E. Note. This guidance is based on commonly available pipe and joinery as of 2004. As new pipe products become available, they can be used in the chart method as long as suitable justification (FEM, test, etc.) is provided to show that the pipe meets the intended reliability of the pipe and performance of the pipe network as a whole.

Design Category	Cost Effective Design Approach	Notes
A	Standard	
B	Extended Joints	
C	Restrained Joints	
D	Extended and Restrained Joints	Standard with bypass
E	Special Joints	Standard with bypass

Table 12. Ductile Iron Pipe

Design Category	Cost Effective Design Approach	Notes
A	Standard	
B	Standard with extra insertion	
C	Restrained Joints	
D	Extended and Restrained Joints	Standard with bypass
E	Not recommended	Standard with bypass

Table 13. PVC Pipe

Design Category	Cost Effective Design Approach	Notes
A	Single Lap Weld	
B	Single Lap Weld	Weld t = pipe t
C	Double Lap Weld	Weld t = pipe t
D	Double Lap Weld / Butt Weld	D/t max 110 in PGD zones
E	Butt Weld	D/t max 95 in PGD zones

Table 14. Welded Steel Pipe

Design Category	Cost Effective Design Approach	Notes
A	Standard	
B	Extended Joints	
C	Restrained Joints	
D	Extended and Restrained Joints	Standard with bypass
E	Not recommended	Standard with bypass

Table 15. Gasketed Steel Pipe

Design Category	Cost Effective Design Approach	Notes
A	Gasketed or Single Lap weld	
B	Single Lap Weld	Weld t = pipe t
C	Double Lap Weld	Weld t = pipe t
D	Not recommended	Standard with bypass
E	Not recommended	Standard with bypass

Table 16. CCP & RCCP Pipe

Design Category	Cost Effective Design Approach	Notes
A	Standard	
B	Butt Fusion Joints	
C	Butt Fusion Joints	
D	Butt Fusion Joints	
E	Butt Fusion Joints	

Table 17. HDPE Pipe

Design Category	Cost Effective Design Approach	Notes
A	Standard	
B	Soldered joints	
C	Soldered joints	Expansion loop / Christie box / Other box

Table 18. Copper Pipe

Design Category	Cost Effective Design Approach	Notes
A	Standard	
B	Dresser-type coupling	
C	Multiple dresser couplings	
D	EBAA flextend type couplings	
E	Not recommended	Relocate hydrant

Table 19. Segmented Pipelines Used as Hydrant Laterals

Design Category	Cost Effective Design Approach	Notes
A	Bolted, Single Lap Weld, Fusion Weld	
B	Bolted, Single Lap Weld, Fusion Weld	Weld t = pipe t
C	Bolted, Double Lap Weld, Single Lap Weld with fiber wrap, Fusion Weld	Weld t = pipe t
D	Bolted, Double Lap Weld, Single Lap Weld with fiber wrap, Butt Weld, Fusion Weld	Bolted, Double Lap Weld, Single Lap Weld with fiber wrap, Fusion Weld
E	Bolted, Double Lap Weld, Single Lap Weld with fiber wrap, Butt Weld, Fusion Weld	Bolted, Double Lap Weld, Single Lap Weld with fiber wrap, Fusion Weld

Table 20. Continuous Pipelines Used as Hydrant Laterals

In addition to the design categories in Tables 12 to 20, the following additional requirements are made. These recommendations are cumulative (For C, include B and C recommendations).

- B. Add isolation valves on all pipes within 50 feet of every intersection, for example, four valves on a four-way cross.
- C. Maximum pipe length between connections for segmented pipe is 16 feet, or as otherwise justified by ESM or FEM.
- D. Maximum pipe length between connections for segmented pipe is 12 feet, or as otherwise justified by ESM or FEM.

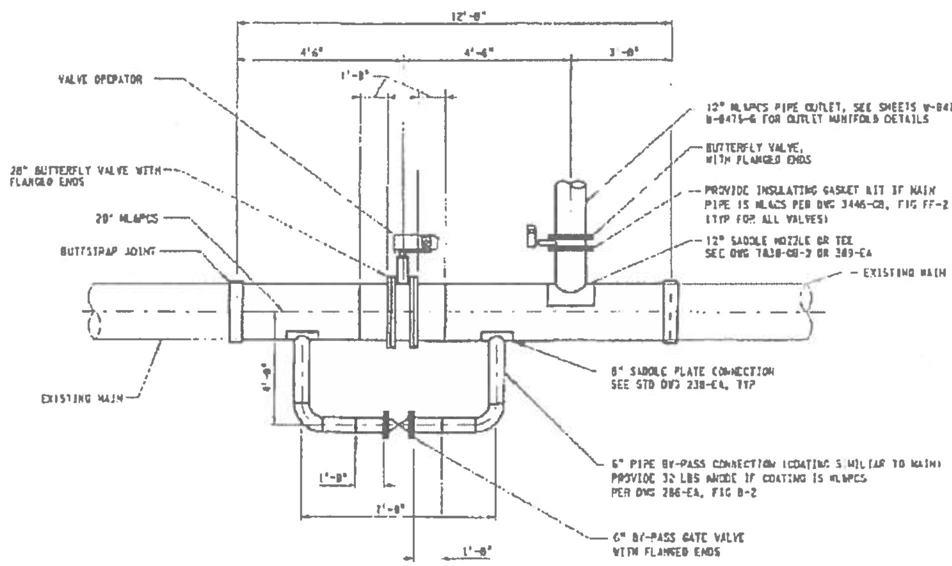
Bypass Pipelines

During design of a pipeline, it is typical to perform some preliminary seismic and hazard investigation. A geotechnical engineer can perform literature search of available publications and assess the seismic setting of the pipeline and identify potential hazards such as fault crossings, landslides, and zones of potential liquefaction.

With this information, the pipeline design engineer can often times route the pipeline to avoid well-defined hazards. This is the most cost-effective approach for minimizing seismic-related damage to a pipeline. However, sometimes there is no feasible way to avoid a hazard and the pipeline must be routed through the hazard.

Instead of using a higher Category Design (such as D or E), the owner can elect to provide a bypass capability, as long as the owner has the ability to install the bypass within about 1 day after the earthquake, and in consideration of the entire post-earthquake response. Bypass capability might be the most cost effective approach to mitigate many fault and landslide crossings for Function Class III pipelines. Bypasses can be used in retrofitting existing pipelines or for new construction where loss of service cannot be tolerated for more than one day.

A typical bypass is illustrated in Figure 2, consisting of a line isolation valve, if none previously existed, and a 12-inch diameter connection and manifold assembly on either side of the defined hazard. In order for the bypass to be used effectively, the hazard must be relatively well defined. Each of the manifolds is configured to accept one or multiple large diameter hose connections. In the event of a seismic event that results in a pipeline failure within the bounds of the hazard, the hazard isolation valves are closed, thereby stopping leakage at the point of failure. The hose is then deployed across the ground between the two manifold assemblies and serves as a temporary pipe bypass, allowing restoration of flows through the system. Figure 3 shows a deployed bypass system at a fault crossing where deployment of three flex hoses was possible.



Typical Isolation Valve with Bypass

Figure 2. Bypass Manifold Assembly

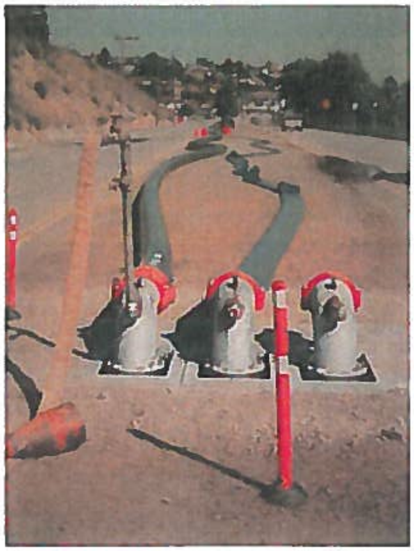


Figure 3. Flex Hose Attached to Manifold Outlets

The criteria for the bypass system components are included in Table 21. So called "large diameter flex hose" (diameter ~5-inches) will generally not provide sufficient flow rate at a reasonable pressure drop, for distances on the order of 1,000 feet between manifolds. So called "ultra large diameter flex hose" (diameter ~12-inches) can provide high flow rates at separation distances of 1,000 feet (or more). There are pros and cons with using either 5-inch or 12-inch hose, including: flow rate and pressure drop; cost; storage life; deployment effort and time; hose breakage and resultant pipe whip; etc.

Description	**Criteria**
Pipe Materials	Mortar-lined and mortar- or tape/epoxy-coated steel pipe Field joints shall be flanged, welded, or mechanically coupled with suitable restraint Design for anticipated internal, external, and transient loading conditions Provide cathodic protection as needed
Manifold Pit	Precast reinforced concrete with seismic design factors suitable for site Traffic rated steel plate cover Sized for easy hose deployment
12-inch Valves and Smaller	Butterfly or Gate
Flexible Hose	12 -inch flex hose, burst pressure ~ 400 psi, operating pressure ~150 psi. Distances up to 1,000 feet or more at flow rates of up to 5,000 gpm 5-inch fire hose from local Fire Department. Distances up to 1,000 feet at flow rates of up to 500 gpm Connections to be coordinated with manifold configuration

Table 21. Bypass System Components Criteria

CONCLUSIONS

It is the intent of these Guidelines to provide a unified, comprehensive and simple approach that can be readily adopted by water utilities for the design of new pipeline installations. The draft Guidelines are available for public comment through January 2005. They may be obtained via the Internet at: http://homepage.mac.com/eidinger/ (follow the link to downloads, and then download Seismic Guidelines.doc.) Comments should be sent to any of the authors.

The Guidelines may result in changes in pipeline installations in moderate and high seismic areas throughout the United States. Given the large economic consequences of widespread pipeline damage, the authors believe that the extra reliability afforded by these changes is worthwhile and cost effective. We hope that the Guidelines will spur water utilities to procure better pipelines in high hazard locations; in turn, the pipeline manufacturers will manufacture and supply better products. This is, in part, a "chicken and egg" process, since prior to the current moment (late 2004 – early 2005) we have not had the Guidelines for water utilities; nor have we always had suitable cost effective pipelines provided by manufacturers to meet the Guidelines.

ABBREVIATIONS AND UNITS

Customary US units (inches, pounds, gallons) are used in this paper. Conversions to SI units are provided below. All pipe sizes are in customary US units; conversion of a customary pipe size (such as 12-inch diameter) to SI units has no precision, as a 12-inch pipe may often have outside diameter anywhere from ~12-inches to ~13-inches.

ALA	American Lifelines Alliance
AWWA	American Water Works Association
AWWARF	American Water Works Association Research Foundation
ESM	Equivalent Static Method
FEM	Finite Element Method
FEMA	Federal Emergency Management Agency
JWWA	Japan Water Works Association
MMC	Multihazard Mitigation Council
NIBS	National Institute of Building Sciences
PGA	Peak Ground Acceleration (g)
PGD	Permanent Ground Deformation (1 inch = 2.54 cm)
PGV	Peak Ground Velocity (1 inch/sec = 2.54 cm/sec)
inch	inch (1 inch = 2.54 cm)
feet	feet (1 foot = 12 inches = 30.48 cm)
g	gravity constant (1g = 386.4 inch/sec^2 = 981 cm/sec^2)
gpm	gallons per minute (1 gpm = 3.785 liters per minute)
psi	pounds per square inch (1 psi = 6.895 kilopascals)
sec	second