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May 23, 2008

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

Subject: Duke Energy Carolinas, LLC
William States Lee III Nuclear Station -- Docket Nos. 52-018 and 52-019
AP1000 Combined License Application for the William States Lee III
Liquefaction Evaluation Results for Seismic Category II and Non-Seismic Power Block Structures

Reference: Dolan to Document Control Desk, *Application for Combined License for William States Lee III Nuclear Station Units 1 and 2*, dated December 12, 2007.

The purpose of this letter is to transmit an evaluation of the potential for seismically induced liquefaction of engineered fill and saprolite soils that provide foundation support to seismic Category II and non-seismic power block structures at the William States Lee III Nuclear Site. The evaluation is provided as Enclosure 1.

The information provided in Enclosure 1 describes the methodologies, results, and conclusions from the analyses of the potential for seismically induced liquefaction of engineered fill and saprolite soils. The evaluation examines the foundation support zones for the Unit 1 non-seismic radwaste building and seismic Category II annex building; and the Unit 2 non-seismic radwaste building, seismic Category II annex building, and non-seismic turbine building. These structures are founded on or over compacted engineered fill over partially weathered/continuous rock, compacted engineered fill over fill concrete and partially weathered/continuous rock, or engineered fill over saprolite soils overlying partially weathered/continuous rock. The non-seismic turbine building for Unit 1 is founded on fill concrete over rock, neither of which is susceptible to liquefaction.

The evaluation provided in Enclosure 1 demonstrates that the liquefaction susceptibility for the engineered fill and saprolite soils beneath the radwaste, annex, and turbine buildings is negligible.

Therefore, adequate protection is provided to ensure that the radwaste building, annex building, and turbine building structures will not adversely affect the function of

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safety-related systems, structures, or components as a result of concerns for seismically induced liquefaction.

If you have any questions or need any additional information, please contact Peter Hastings, Nuclear Plant Development, Licensing Manager, at (980) 373-7820.

Bryan J. Dolan
Vice President
Nuclear Plant Development

Enclosure:
1. Summary of Liquefaction Evaluation Results for Seismic Category II and Non-Seismic Power Block Structures for William States Lee III Nuclear Station Combined Construction Permit and Operating License COL Project

xc (w/out enclosure):
Michael Johnson, Director, Office of New Reactors
Gary Holahan, Deputy Director, Office of New Reactors
David Matthews, Director, Division of New Reactor Licensing
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Stephanie Coffin, Branch Chief, DNRL
Brian Hughes, Senior Project Manager, DNRL

AFFIDAVIT OF BRYAN J. DOLAN

Bryan J. Dolan, being duly sworn, states that he is Vice President, Nuclear Plant Development, Duke Energy Carolinas, LLC, that he is authorized on the part of said Company to sign and file with the U. S. Nuclear Regulatory Commission this supplement to the combined license application for the William States Lee III Nuclear Station and that all the matter and facts set forth herein are true and correct to the best of his knowledge.

Bryan J. Dolan
Bryan J. Dolan

Subscribed and sworn to me: May 23, 2008
Date

Elaine Falcone
Notary Public ELAINE FALCONE

My Commission Expires: February 27, 2011

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ENCLOSURE No. 1

SUMMARY OF LIQUEFACTION EVALUATION RESULTS FOR SEISMIC CATEGORY II AND NON-SEISMIC POWER BLOCK STRUCTURES FOR WILLIAM STATES LEE III NUCLEAR STATION COMBINED CONSTRUCTION PERMIT AND OPERATING LICENSE COL PROJECT

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1.0 Introduction

The information summarized in this enclosure describes the methodologies and results from analyses for seismically induced liquefaction of the soils at the William States Lee III Nuclear Site. The analyses presented herein and resulting factors of safety against liquefaction support evaluations of the foundation support zones for the following power block structures: non-seismic radwaste building and seismic Category II annex building for Units 1 and 2, and the non-seismic turbine building for Unit 2. These structures are founded on or over compacted engineered fill over partially weathered/continuous rock, compacted engineered fill over fill concrete and partially weathered/continuous rock, or engineered fill over saprolite soils overlying partially weathered/continuous rock. The non-seismic turbine building for Unit 1 is founded on fill concrete over rock, neither of which is susceptible to liquefaction. The analyses confirm the absence of liquefaction and demonstrate that the William States Lee III Nuclear Site condition meets the criteria in DCD subsection 2.5.4.6.5. This document supplements the analysis results presented in subsection 2.5.4 of the Final Safety Analysis Report (FSAR).

The evaluations presented herein do not extend to the seismic Category I nuclear island structures as these safety-related structures are founded on continuous rock or fill concrete over continuous rock. Neither fill concrete nor rock are susceptible to seismically induced liquefaction as described in Section 2.5.4 of the FSAR.

The liquefaction results summarized below rely on existing information developed as part of investigations performed as part of the William States Lee III Combined Construction Permit and Operating License (COL) field investigations.

2.0 Background

All seismic Category I safety-related plant foundations for William States Lee III Nuclear Station Units 1 and 2 are founded on rock, or fill concrete over rock; neither fill concrete nor rock is susceptible to liquefaction. Plan maps, cross sections, and summary boring logs presented in FSAR Subsection 2.5.4.3 show the locations and rock foundation conditions of the seismic Category I nuclear island structures that have a design subgrade elevation of 550.5 feet. The design basemat subgrade places most of the foundation for the William States Lee III Unit 1 nuclear island on existing concrete that is placed over a sound and cleaned rock surface remaining from the Cherokee Nuclear Station Unit 1. The foundation for William States Lee III Nuclear Station Unit 2 and for a small area of Unit 1 was placed directly on a newly-excavated and cleaned sound rock surface. Therefore, there is no liquefaction hazard that could affect the foundations of the seismic Category I plant structures and facilities.

Outside the nuclear islands, engineered fill is placed adjacent to the seismic Category I structures over the exposed surfaces after they are cleaned and prepared. The typical thickness of fill is about 30 to 40 feet with a maximum thickness of about 89 feet (near the northwest corner of the Unit 1 radwaste building). The engineered fill completely backfills the historic Cherokee Nuclear Station excavation, surrounding the William States Lee III Nuclear Station nuclear island structures up to yard grade elevation of 589.5 feet. Engineered fill is confined within the excavation perimeter by cuts primarily made in native partly weathered rock and saprolite, providing lateral confinement to the fill. The engineered fill rests on the exposed rock/fill concrete immediately adjacent to the nuclear islands, and on saprolite soils in some locations further away from the nuclear islands and underneath some of the structures named earlier. In accordance with the project criteria, residual soils-saprolite materials with N_{60} blow counts less than 15 are not allowed to remain in-place below structural (Group I) fills or foundations.

Groundwater rises above the bedrock surface within the engineered fill to elevations between about 574 feet to 584 feet. For the liquefaction analysis, groundwater was conservatively set at 584 feet.

FSAR Subsection 2.5.4.5 describes material specifications and compaction for engineered fill (e.g. 95% standard Proctor). Existing Group I fill at the site shows that engineered fills meeting these specifications are stiff to very stiff sandy silt (ML) and medium dense silty sand (SM) materials with fines content generally exceeding 35 percent. The floor of the excavation, and site yard area outside of the excavation perimeter, are relatively flat, and potential free faces or sloping basal surfaces do not exist adjacent to or below the fill that could present a potential lateral spread condition in the event of minor or localized cyclic pore pressure build-up in the engineered fills under seismic loading. Therefore, liquefaction analyses need only consider level site conditions.

No active or potentially active faults or seismic deformation zones occur at the William States Lee III site. COL investigation results presented in FSAR Sections 2.5.1 and 2.5.4 confirm that rock and soil materials at the site have not experienced seismically induced ground failure (e.g. slope failure, liquefaction, lurching, and subsidence) from historic or paleoearthquakes. Therefore, the geologic setting and past performance indicate that liquefaction is not expected within the residual soils-saprolite overlying rock.

The liquefaction evaluations utilize the recommendations of Regulatory Guide 1.198 and use existing information in the references to calculate the safety factors against liquefaction. One evaluation is based on the SPT blow counts from COL borings that encountered existing Group I fill at the William States Lee III Station, and for the residual soil – saprolite materials in COL borings within and adjacent to the power block area of the William States Lee III Nuclear Station. A second evaluation is based on the shear wave velocities for COL locations in existing Group I fill and in the residual soil – saprolite materials.

3.0 Seismic Parameters for Liquefaction

Deterministic median peak acceleration estimates for two earthquake sources were evaluated. The earthquakes reflect controlling sources based on the hazard deaggregation developed for the site. The scenarios evaluated include a local background source, referred to as the 'Nearby' source, reflective of regional background seismicity, and a Charleston, South Carolina source, referred to as the 'Distant' source. The defined earthquakes represent the range of combinations between earthquake magnitudes and distance for earthquake scenarios for the site.

The deterministic Nearby and Distant earthquake moment magnitudes (Mw) and surface accelerations (a_{max}) are listed in Table 1. The surface accelerations are all at Elevation 589.5 ft. The profiles used to calculate the site acceleration values are described below and presented in Table 2.

These profiles capture the range in soil unit thicknesses for the seismic Category II and non-seismic power block structures at the site:

- Group I engineered fill condition – deep fill condition (Profile B1) with 89 feet of Group I engineered fill overlying bedrock such as near the northwest corner of the Unit 1 Radwaste building.
- Group I engineered fill condition – typical fill condition (Profile C1) with 40 feet of Group I engineered fill overlying bedrock such as near the Unit 2 annex building.
- Group I engineered fill and saprolite condition – saprolite condition with 51 feet of saprolite underlying 18.5 feet of Group I engineered fill (Profile F1) such as adjacent to Unit 2 turbine building and general yard area conditions around the perimeter of the historic Cherokee Nuclear Station (CNS) excavation.

The enveloping values of a_{max} in Table 1 are 0.345g and 0.071g for the Nearby and Distant earthquakes, respectively. These values are associated with the saprolite condition (Profile F1) described above.

4.0 Evaluation Methodologies and Results

The evaluations utilize the recommendations of Regulatory Guide 1.198, Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites, and use existing information to determine the liquefaction potential of Group I engineered fill and saprolite materials. Qualitative evaluation was performed using a geologically founded assessment for liquefaction potential (Youd, 1991, and Youd and Perkins, 1978). The primary quantitative evaluation method is based on the Standard Penetration Test (SPT) method (Youd et al, 2001). A secondary quantitative evaluation was performed using the shear wave (Vs) velocity-based method following Andrus and Stokoe (2000) as presented in Youd et al. (2001) and additionally considers updated procedures presented in Andrus et al. (2004).

4.1 Geologic Screening Assessment

The geologic screening process described in Regulatory Guide 1.198 was applied to the Group I engineered fill and saprolite soils at the William States Lee III site. This process is based largely on work by Youd (1991) and Youd and Perkins (1978) that shows most liquefaction risk is associated with saturated, recent Holocene sedimentary deposits of loose sand and silt and uncompacted fills (typically hydraulically-placed sandy fill). The William States Lee III Group I engineered fill and saprolite do not fall within these categories of susceptible soil.

4.1.1 Group I Engineered Fill

Group I engineered fill is derived from on-site borrow sources of native residual soil and saprolite typically consisting of low plasticity sandy silt (ML) to silty sand (SM), with fines content generally greater than 35 percent. The textural composition (comprised of sand and silt) and non-plastic nature of the Group I engineered fill material fall within the range of soils potentially susceptible to liquefaction, but Group I engineered fill placement and compaction specifications (95% standard Proctor) will result in a relatively uniform and medium dense (stiff to very stiff) engineered fill profile that is resistant to liquefaction, as described in

FSAR Subsection 2.5.4.5. Group I engineered fill placed in a controlled manner consistent with the project criteria are not considered susceptible to liquefaction.

4.1.2 Saprolite

Some Group I engineered fill will be placed over medium dense to dense native saprolite. All saprolite exhibiting N_{60} SPT blow counts less than 15 will be removed as described in FSAR Subsection 2.5.4.5. This requirement will make the remaining saprolite resistant to liquefaction. The age of the saprolite suggests additional resistance to liquefaction. Saprolite is formed as a result of in situ weathering of bedrock over a substantially long time period; saprolite generally appears to be of late Pleistocene age. With increased age, soils naturally become more resistant to liquefaction (Youd et al. 2001). This combination of subgrade suitability criteria and substantial geologic age for saprolite indicate significant resistance to liquefaction in any remaining saprolite underlying Group I engineered fill.

In summary, the geologic screening assessment indicates the liquefaction hazard associated with the Group I fill and saprolite soils is low to nonexistent.

4.2 SPT-Based Evaluation

The liquefaction potential is evaluated using the "corrected" (normalized) SPT values, following the procedures of Youd, et. al., 2001. The envelope input ground surface acceleration for Nearby and Distant earthquake accelerations are listed in Table 1.

The safety factors for Group I fill for Nearby and Distant earthquakes are shown in Figures 1 and 2, respectively. The safety factors for residual soil – saprolite for Nearby and Distant earthquakes are shown in Figures 3 and 4, respectively. The lowest factors of safety are summarized in Table 3. The Nearby earthquake safety factors govern in the analyses of both the Group I fill and the saprolite. The safety factors are described and discussed later herein.

4.3 Shear Wave (Vs) Velocity-Based Liquefaction Potential Evaluation

Analyses are conducted based on varying conditions that span the range of conditions at the seismic Category II and non-seismic power block structures for Group I engineered fills and saprolite soils. The envelope input ground surface accelerations for Nearby and Distant earthquake listed in Table 1 are used.

The results of the Vs-based liquefaction analyses show that nearly all results are not liquefiable as the equivalent clean sand stress corrected shear-wave velocity (V_{S1cs}) exceeds the limiting upper shear-wave velocity (V^*_{S1}). In the few instances where safety factors are calculated, the resulting value exceeds 10 for saprolite materials with SPT $N_{60} \geq 15$ and Group I engineered fills for both the Nearby and Distant earthquakes. For this reason, safety factor verses depth plots are not practical. Comparative plots of Cyclic Stress Ratio (CSR) and the available Cyclic Resistance Ratio (CRR) of equivalent clean sand for Nearby and Distant earthquakes are shown in Figures 5 and 6, respectively. The available CRR, depicted by the curving line, separates the zone of "no liquefaction" and "liquefaction". The CSR points are in the "no liquefaction" zone of the plots for both figures. The lowest factors of safety are summarized in Table 3.

5.0 Summary of Results

The results of screening and empirical evaluations using the SPT and shear-wave velocity data gathered during the COL exploration for saprolite and Group I engineered fills are summarized in Table 3.

For evaluating the SPT and shear-wave velocity results for safety factor, Regulatory Guide 1.198, Section 3.2, indicates that a safety factor of 1.1 is generally considered as a "trigger" value, whereas safety factors between 1.1 and 1.4 are considered intermediate, and safety factor values greater than or equal to 1.4 are considered high.

The Nearby earthquake safety factors govern in the analyses of both the Group I fill and the saprolite.

The SPT safety factors representative of the Group I fill are in the intermediate to high range, with 96 percent of the individual values in the high range. Results of the SPT liquefaction safety factor for the Group I fill and the Nearby earthquake are plotted in Figure 1. One of 192 samples, or about 0.5 percent, has a safety factor equal to 1.00 of the SPT tests analyzed and it occurred in a soil sample that is not typical for the Group I fill soils. The next lowest safety factor is 1.20, and occurs in soil that is more typical of the Group I fill materials. There are six, or about three percent, of the SPT tests in Group I fill that indicate safety factors in the intermediate range (1.1 to 1.4). The remaining 185, or about 96 percent, of the SPT tests indicate high safety factors greater than 1.4.

The SPT safety factors in the residual soil – saprolite that remains in-place beneath the Group I fill are in the high range. Results of the SPT liquefaction safety factor for the residual soil – saprolite materials for the Nearby earthquake plotted in Figure 3 show that these materials exhibit generally intermediate to high safety factors, with the lowest equal to about 1.2. This lowest value occurs in soil that has a N_{60} value less than 15 blows per foot; under the project criteria these N_{60} less than 15 soils are removed and replaced with compacted fill. The lowest liquefaction safety factor in residual soil – saprolite which exhibits N_{60} equal to 15 or higher and which therefore remains in place is greater than 2, and therefore in the high safety factor range.

Consistent with the SPT-based evaluation, the results from shear-wave velocity based evaluation show factors of safety in the high range for instances where safety factors can be calculated. In most cases, the equivalent clean sand stress corrected shear-wave velocity (V_{S1cs}) exceeds the limiting upper shear-wave velocity (V^*_{S1}) for both Group I engineered fill (98 percent of values) and saprolite (80 percent for values for N_{60} ≥15) for the Nearby earthquake and by definition are not liquefiable.

6.0 Conclusions

Based on the earthquake scenarios and material profiles evaluated, the liquefaction susceptibility is negligible as the SPT liquefaction safety factors are dominantly in the high range (SF ≥1.4) for both the Group I fill and the saprolite to remain beneath the fill. The analysis performed for William States Lee III Units 1 and 2 seismic Category II and non-seismic structures conform to the liquefaction criteria in DCD subsection 2.5.4.6.5.

The liquefaction potential for Group I engineered fill and saprolite for each of William States Lee III Units 1 and 2 seismic Category II and non-seismic structures for the nearby and distant earthquake scenarios are summarized in Table 4a and 4b, respectively. The results summarized in Tables 4a and 4b support the findings presented in William States Lee III FSAR subsection 2.5.4.8.

7.0 References

Andrus, R.D., Stokoe II, K.H., 2000, "Liquefaction Resistance of Soils from Shear-Wave Velocity", Journal of Geotechnical and Geoenvironmental Engineering, November 2000, p. 1015-1025.

Andrus, R.D., Stokoe II, K.H., Juang, C.H., 2004, "Guide for Shear-Wave-Based Liquefaction Potential Evaluation," Earthquake Spectra, Vol. 20, No. 2, p. 285-308.

Regulatory Guide 1.198, "Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plants," USNRC, November 2003.

Youd, T.L., and Perkins, D.M., 1978, Mapping of liquefaction induced ground failure potential: Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol., 104, No. GT4, pp. 433-446.

Youd, T.L., 1991, "Mapping of Earthquake-Induced Liquefaction for Seismic Zonation," Fourth International Conference on Seismic Zonation, pp. 111-138.

Youd, T. L., et al., 2001, "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2003, pp. 817-883.

Also see the Discussions and Closures from March 2003:

Pyke, R., 2003, Discussion of "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils" by Youd, T.L., Idriss, I.M. Andrus, R.D. Arango, I., Castro, G., Christian, J.T., Dobry, R., Liam Finn, W.D.L., Harder, L.F., Jr., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson, W.F., III, Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., Stokoe, K.H., II": Journal of Geotechnical and Geoenvironmental Engineering, Vol. 129, No. 3, American Society of Civil Engineers, March, 2003, pp. 283 to 284.

Youd, T. L., et al, 2003, Errata in *Closure to "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils"*, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 129, No. 3, American Society of Civil Engineers, March, 2003, pp. 284 to 286.

8.0 Tables and Figures

Tables

Table 1 Deterministic Seismic Parameters by Profile

Earthquake (Seismic Loading)	Magnitude (Mw)(1)	Surface Acceleration at Site by Profile, a_{max} (g)		
		B1	C1	F1
Nearby	5.1	0.229	0.272	0.345(2)
Distant	7.1	0.062	0.064	0.071(2)

Notes:

(1) Mw = Moment Magnitude.

(2) Values selected for analysis (site acceleration envelope).

Table 2 Summary of Profile Properties

Profile	Analysis Desctiption	Profile Properties			
		Geologic Unit	Thickness (ft)	Depth (ft)	Elevation (ft)
B1	Unit 1 Deep Fill Condition	Group I Fill	89	0.0 - 89.0	589.5 - 500.5
C1	Unit 1 & 2 Typical Fill Condition	Group I Fill	40	0.0 - 40.0	589.5 - 549.5
F1	Group I Engineered Fill and Saprolite Condition	Group I Fill	18.5	0.0 - 18.5	589.5 - 571.0
		Saprolite	51	18.5 - 69.5	571.0 - 520.0

Table 3 Summary of Liquefaction Evaluation Results

Seismic Loading	**Screening Analysis**		**Empirical Analysis**			
	Geologic Screening		Corrected SPT Method		Shear-Wave (V_S) Velocity-Based Method	
	Group I Fill	Saprolite	Group I Fill	Saprolite	Group I Fill	Saprolite
Nearby Earthquake	Nil to Low	Nil to Low	Minimum SF(1) = 1.00 Second to Minimum SF(2) = 1.20	Minimum SF(3) = 1.17 Minimum SF(4) ($N_{60} \geq 15$) = 2.18	Minimum SF = 17	Minimum SF(3) = 2.11 Minimum SF(4) ($N_{60} \geq 15$) = 10
Distant Earthquake			Minimum SF(1) = 2.09 Second to Minimum SF(2) = 2.50	Minimum SF(3) = 2.44 Minimum SF(4) ($N_{60} \geq 15$) = 4.58	Minimum SF = 35	Minimum SF(3) = 4.40 Minimum SF(4) ($N_{60} \geq 15$) = 21

SF = Safety Factor

Notes:

(1) Minimum Safety Factor (SF) occurred in the clean sand (SP) found in Boring B-1068 at 13.5 to 15.0 ft. Similar soil with higher SPT value was found at comparable depth in nearby Boring B-1069. None of the other 190 SPT samples were similar to the clean sand. This clean sand is not typical of the Group I fill soil which is mostly sandy silt (ML) and lesser amounts of silty sand (SM) materials.

(2) The "Second to Minimum SF" value is more representative of Group I fill minimum. The typical SF in Group I Fill is above 1.4 (> 96% of individual values for Nearby Earthquake; 100% of values for Distant Earthquake).

(3) Minimum Safety Factor (SF) in saprolite occurred in materials with $N_{60} < 15$, which will be removed and replaced with Group I Engineered Fill in accordance with project criteria.

(4) Minimum SF $N_{60} \geq 15$ is representative of the saprolite that will remain in place.

Table 4a. Summary of Liquefaction Evaluation Results for Nearby Earthquake

Unit / Structure		Seismic Design Criteria	Structure Foundation Support Condition	Safety Factor[1]	
				Corrected SPT Method	Shear Wave (Vs)-Based Method
Unit 1	Annex Building	Category II	Group I Engineered Fill over Fill Concrete and Partially Weathered/Continuous Rock	$SF^{(2)} \geq 1.4$	$SF \geq 17$
	Radwaste Building	Non-seismic	Group I Engineered Fill over Fill Concrete and Partially Weathered/Continuous Rock	$SF^{(2)} \geq 1.4$	$SF \geq 17$
	Turbine Building	Non-seismic	Fill Concrete over Continuous Rock	No Liquefiable Material	No Liquefiable Material
Unit 2	Annex Building	Category II	Group I Engineered Fill over Partially Weathered/Continuous Rock	$SF^{(2)} \geq 1.4$	$SF \geq 17$
	Radwaste Building	Non-seismic	Group I Engineered Fill over Saprolite and Partially Weathered/Continuous Rock	$SF^{(3)} = 2.18$	$SF \geq 10$
	Turbine Building	Non-seismic	Group I Engineered Fill over Saprolite and Partially Weathered/Continuous Rock	$SF^{(3)} = 2.18$	$SF \geq 10$

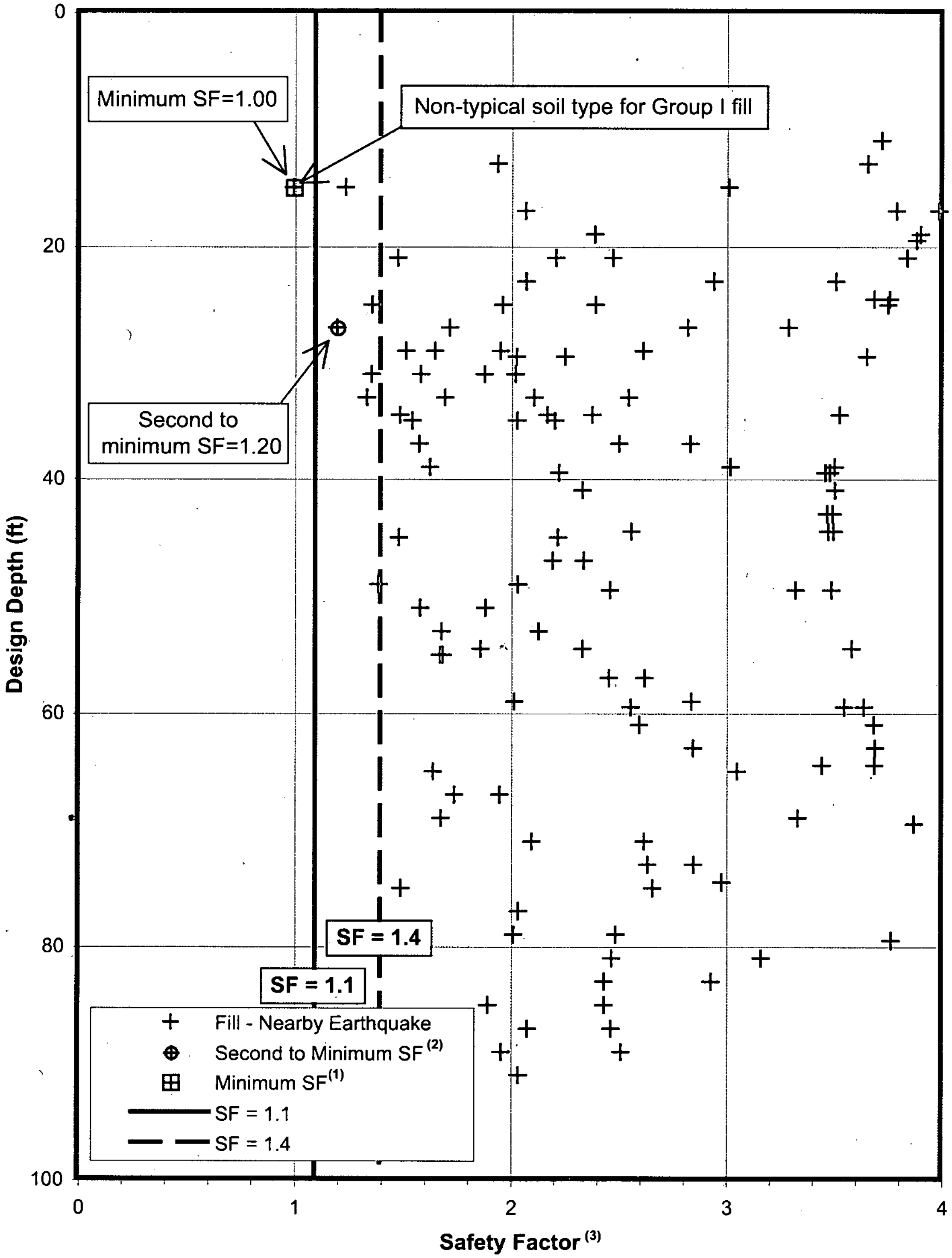
Notes
(1) SF = Safety Factor, represents minimum SF based on material profile and constructed condition.
(2) Minimum SF of typical Group I fill, representing 96 percent of SPT values as depicted on Figure 1.
(3) SF is representative minimum for the saprolite that will remain in place after construction ($N_{60} \geq 15$) as depicted on Figure 3. Group I Fill will have a $SF \geq 1.4$.

Table 4b. Summary of Liquefaction Evaluation Results for Distant Earthquake

Unit / Structure		Seismic Design Criteria	Structure Foundation Support Condition	Safety Factor[1] Corrected SPT Method	Safety Factor[1] Shear Wave (Vs)-Based Method
Unit 1	Annex Building	Category II	Group I Engineered Fill over Fill Concrete and Partially Weathered/Continuous Rock	SF[2] = 2.50	SF ≥ 35
	Radwaste Building	Non-seismic	Group I Engineered Fill over Fill Concrete and Partially Weathered/Continuous Rock	SF[2] = 2.50	SF ≥ 35
	Turbine Building	Non-seismic	Fill Concrete over Continuous Rock	No Liquefiable Material	No Liquefiable Material
Unit 2	Annex Building	Category II	Group I Engineered Fill over Partially Weathered/Continuous Rock	SF[2] = 2.50	SF ≥ 35
	Radwaste Building	Non-seismic	Group I Engineered Fill over Saprolite and Partially Weathered/Continuous Rock	SF[3] > 4	SF ≥ 21
	Turbine Building	Non-seismic	Group I Engineered Fill over Saprolite and Partially Weathered/Continuous Rock	SF[3] > 4	SF ≥ 21

Notes
(1) SF = Safety Factor, represents minimum SF based on material profile and constructed condition.
(2) Minimum SF of typical Group I fill as depicted on Figure 2.
(3) SF is representative minimum for the saprolite that will remain in place after construction ($N_{60} \geq 15$) as depicted on Figure 3. Group I Fill will have a SF ≥ 1.4.

Figures



(1) Minimum Safety Factor (SF) occurred in the clean sand (SP) found in Boring B-1068 at 13.5 to 15.0 ft. Similar soil with higher SPT value was found at comparable depth in nearby Boring B-1069. None of the other 190 SPT samples were similar to this clean sand. This clean sand is not typical of the Group I fill soil which is mostly sandy silt (ML) and silty sand (SM) materials.
(2) The "Second to Minimum SF" (1.2) value is more representative of Group I fill minimum. The typical SF in Group I fill is above 1.4 (>96 percent of individual values for Nearby Earthquake).
(3) Safety factor values > 4 are not shown.

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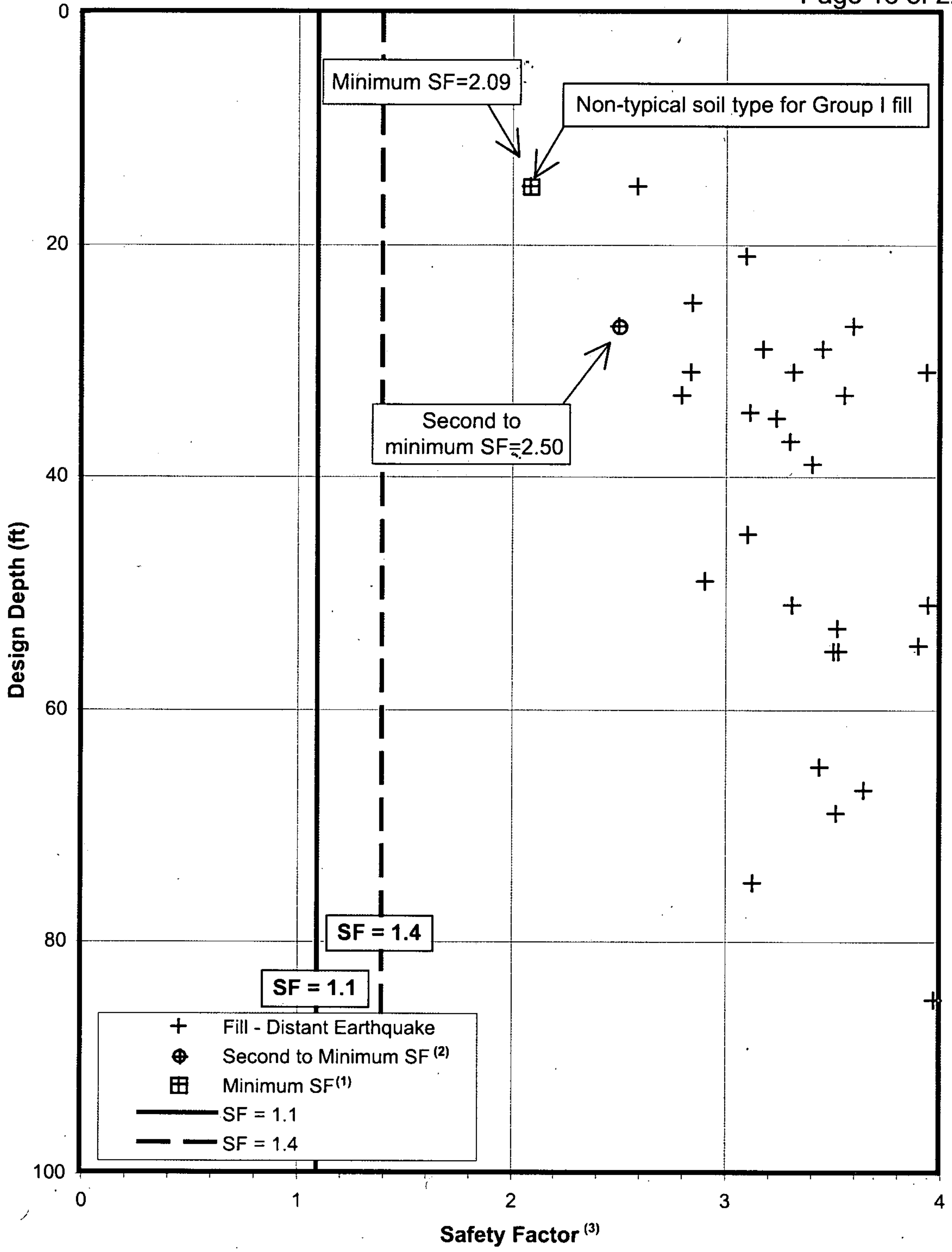
NUCLEAR STATION UNITS 1 & 2

Liquefaction Safety Factors of
Group I Fill for Nearby Earthquake

FIGURE 1



(1) Minimum Safety Factor (SF) occurred in the clean sand (SP) found in Boring B-1068 at 13.5 to 15.0 ft. Similar soil with higher SPT value was found at comparable depth in nearby Boring B-1069. None of the other 190 SPT samples were similar to this clean sand. This clean sand is not typical of the Group I fill soil which is mostly sandy silt (ML) and silty sand (SM) materials.

(2) The "Second to Minimum SF" value is more representative of Group I fill minimum. The typical SF in Group I fill is above 2.5. (191 of 192 individual values for distant earthquake).

(3) Safety factor values > 4 are not shown.

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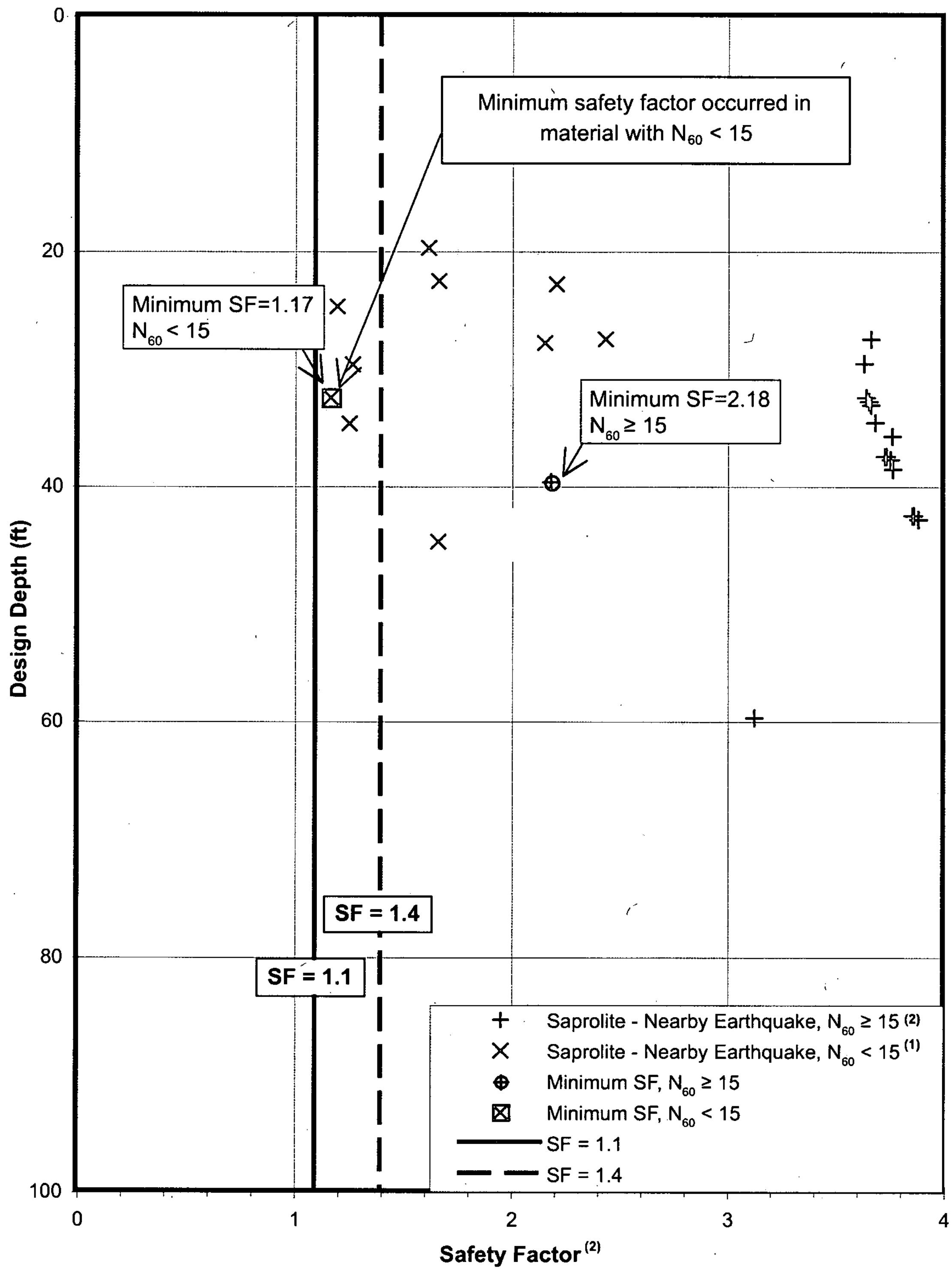
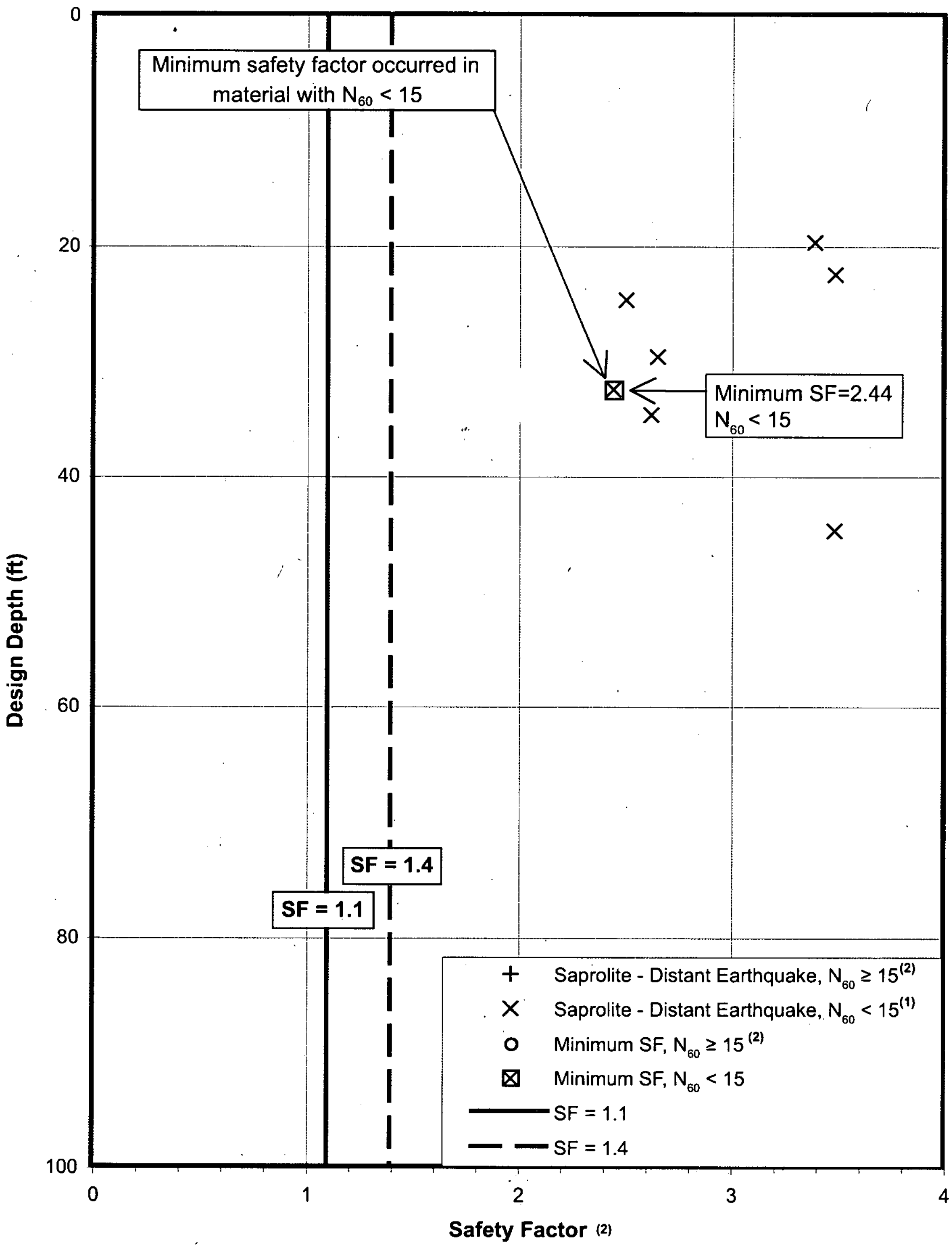
Liquefaction Safety Factors of Group I Fill for Distant Earthquake

FIGURE 2



(1) Saprolite material with N_{60} < 15 to be removed in accordance with excavation criteria.
(2) Safety factor values > 4 are not shown.

WILLIAM STATES LEE III NUCLEAR STATION UNITS 1 & 2
Liquefaction Safety Factors of Residual Soil – Saprolite for Nearby Earthquake FIGURE 3

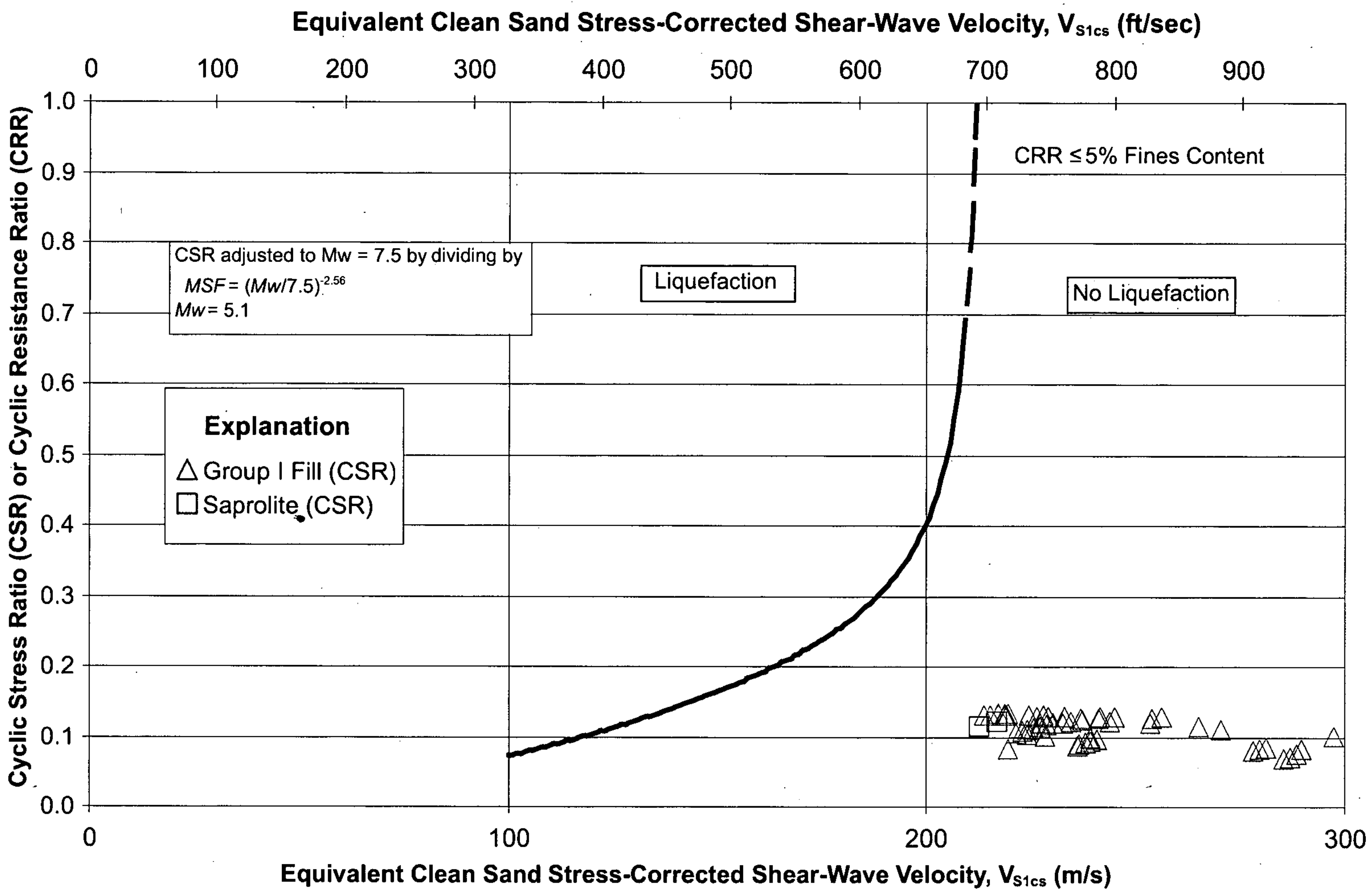


(1) Saprolite material with N_{60} < 15 to be removed in accordance with excavation criteria.
(2) Safety factor values > 4 are not shown. All saprolite samples with $N_{60} \geq 15$ have SF > 4.

WILLIAM STATES LEE III
NUCLEAR STATION UNITS 1 & 2

Liquefaction Safety Factors of Residual Soil – Saprolite for Distant Earthquake

FIGURE 4



Notes: (1) 78 total points analyzed.

(2) Data points with V_{S1cs} greater than 300 m/s (984 ft/sec) not shown.

(3) Data points exhibiting SPT N_{60} blowcounts < 15 not shown as this material is removed in accordance with project criteria.

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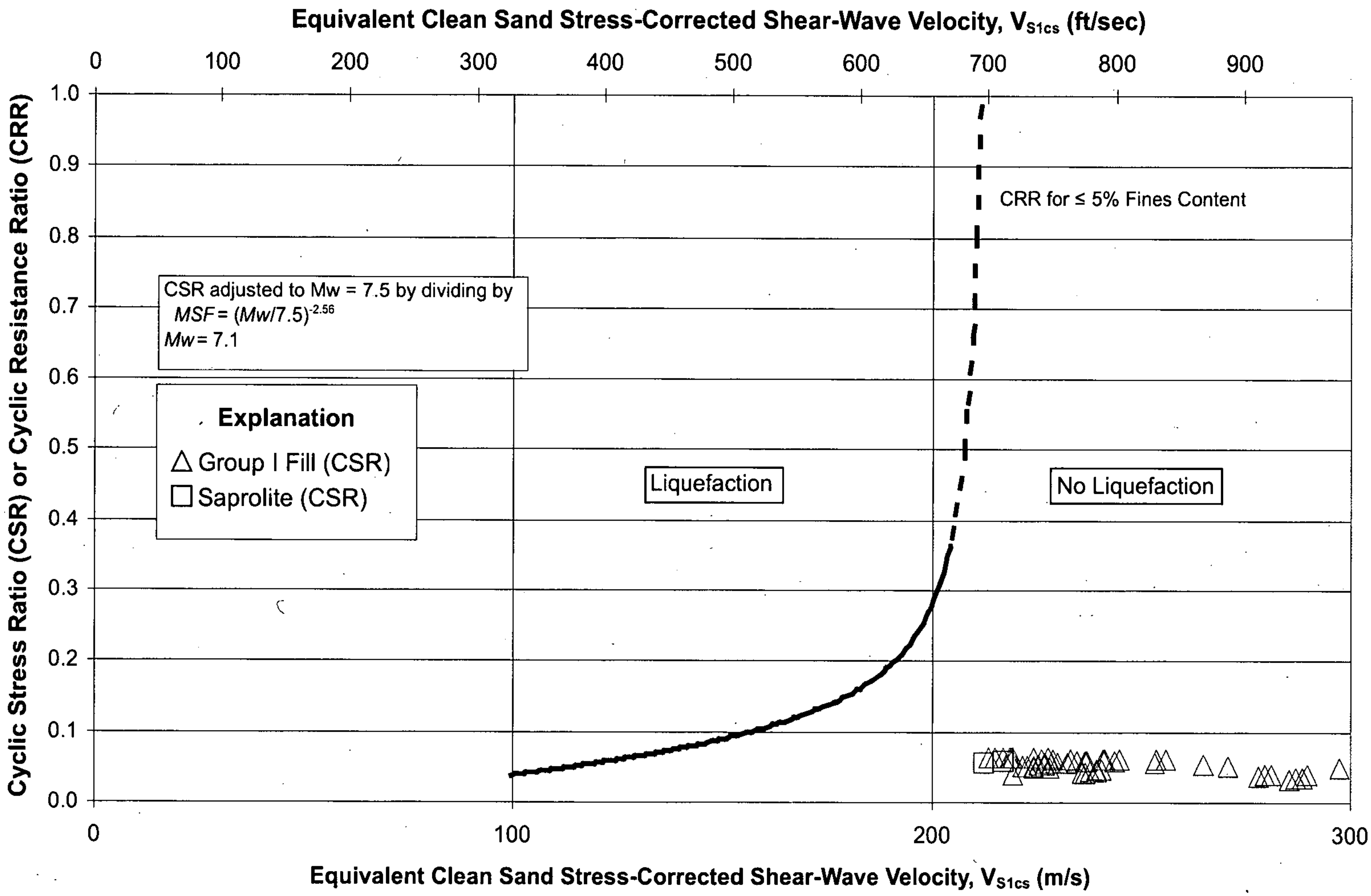
Comparison of CSR to Equivalent Clean Sand Stress-Corrected Shear-Wave Velocity for Nearby Earthquake
FIGURE 5



Notes: (1) 78 total points analyzed.
(2) Data points with V_{S1cs} greater than 300 m/s (984 ft/sec) not shown.
(3) Data points exhibiting SPT N_{60} blowcounts < 15 not shown as this material is removed in accordance with project criteria.

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Comparison of CSR to Equivalent Clean Sand Stress-Corrected Shear-Wave Velocity for Distant Earthquake

FIGURE 6