SAGE
GEOTECHNICAL

Norwest Engineering
Consulting Engineers

March 26, 2025

Transmitted via email to: **William.Johnson@deq.oregon.gov**

Department of Environmental Quality
Agency Headquarters
700 NE Multnomah St, Ste 600
Portland, OR 97232

Attn: William "Ian" Johnson, DEQ Fuel Tank Seismic Stability Inspector

Re: Response to Oregon Department of Environmental Quality's Review
Kinder Morgan Seismic Vulnerability Assessment
Linnton Terminal
Portland, Oregon

Dear Mr. Johnson:

This letter summarizes the response to The Oregon Department of Environmental Quality (ODEQ) comments following review of Kinder Morgan's Seismic Vulnerability Assessment (SVA) for its Linnton terminal. The following documents comprise the official response:

1. This letter and its attachments. Responses specific to the ODEQ checklist are provided in the letter attachments. The rationale for our approach to completing the SVA and answering the checklist is provided in the body of this letter.

2. The updated SVA report for the terminal. The checklist responses are incorporated into the report. The SVA has been stamped and reissued.

Kinder Morgan is committed to compliance with the requirements of the 2022 Oregon Laws Chapter 99 (Law) and Oregon Administrative Rules (OARs) 340-300-0001 through 340-300-0009. Accordingly, Kinder Morgan submitted a facility-wide SVA for its Linnton terminal. Minor issues with the organization of the geotechnical appendix and in-text references to the same appear to have led to confusion during the review of the SVA. Sage Geotechnical LLC (Sage, geotechnical engineer) and Norwest Engineering, Inc. (Norwest, prime engineer) have updated the SVA to address these issues for ease of review and document flow.

OAR 340-300-0003(6)(a) sets forth the requirements for the geotechnical portion of the SVA, and the SVA submitted by Kinder Morgan meets these requirements. The "Seismic Vulnerability Assessment Forms – Form 1: Questions for the Geotechnical Assessment of Each Facility" is inconsistent with OAR 340-300-0003(6). In the assessment team's opinion, the exhaustive questions and requests in the checklist are designed to elicit more information than is needed to assess seismic vulnerability. Where the OAR sets forth clear/fixed requirements for the geotechnical portion of the SVA, the checklist includes additional questions and requests that seem irrelevant to the determination of seismic vulnerability. Nevertheless, Sage and Norwest have attempted to respond to each question to further demonstrate their conclusion. This response is documented in Table 1 and Attachment 1.

The checklist includes arbitrary requirements that cannot be justified by the Law or the OAR. One such requirement is a maximum spacing for subsurface explorations (e.g., boreholes and/or cone penetration test soundings) advanced along berms, which checklist item 4c specifies as 200 feet. This spacing requirement appears to come from the American Association of State Highway and Transportation Officials' (AASHTO) *LRFD Bridge Design Specifications;* however, the AASHTO standard is neither relevant to industrial facilities, like liquid fuel terminals, nor is it referenced in the Law or the OAR.

Kinder Morgan has completed the facility-wide SVA in good faith and it satisfies the requirements in OAR 340-300. The assessment team has concluded that the terminal is vulnerable to damage during an earthquake and additional geotechnical investigation and analyses are unlikely to alter this conclusion. Instead, the results of the existing geotechnical, structural, and safety assessments will be used to prepare and implement Risk Mitigation Implementation Plans (RMIPs). Kinder Morgan will use the SVAs, the Law, the OAR, and its own operational goals to develop the RMIPs with the understanding that the RMIPs could necessitate more detailed geotechnical, structural, or safety investigations and analyses, depending on the selected risk-mitigation strategy.

SAGE GEOTECHNICAL, LLC & NORWEST ENGINEERING, INC.

REGISTERED PROFESSIONAL ENGINEER
97882PE
OREGON
JAN 11, 2022
DANIEL CORDER SIMPSON
03/26/2025

REGISTERED PROFESSIONAL ENGINEER
54567
OREGON
JULY 9, 2001
MICHAEL J. SMITH
3/26/25
Expires: 6/30/26

Daniel Simpson, PE, GE
Principal Geotechnical Engineer
Sage Geotechnical, LLC

Michael Smith, PE
Director of Engineering
Norwest Engineering, Inc.

Attachments: Table 1. Geotechnical Checklist Comments and Responses
Attachment 1. Supplemental Information

Table 1

Kinder Morgan Linnton Terminal

No.	DEQ Comment	Response
1	Provide a scale plan or plot drawing of the entire facility, including all tanks, berms, marine terminals, loading racks, pipelines, etc. [GEO1]	See Appendix A Figure A-1, Figure A-2, and Appendix B.
2	Provide all available soil data, boring logs, and geotechnical reports developed for the site since the original design and as-built properties of he	See Appendix A-1 and A-2.
3	Provide locations of all existing boreholes or CPTs on the plan or plot drawings. [GEO3]	See Appendix A Figure A-2.
4a	Boring or CPT depth shall be a minimum of 100ft (Appendix E, API 650, MOTEMS Section 3106F.2.2, ASCE 7, Section 20.1. [GEO4]	Explorations were advanced to refusal (approximately 45 to 60 feet below grade). Refusal is due to a very dense layer of Troutdale formation or basalt. Exploring these layers that cause exploration refusal is unnecessary for a seismic vulnerability assessment.
4b.	Borings are to be onshore and offshore (if any marine structures are present). [GEO4]	Offshore borings are unnecessary to establish seismic vulnerability. The marine structures are vulnerable. Offshore borings may be performed during the risk mitigation analyses for the marine structure.
4c.	Spacing of boreholes or CPTs along the berms shall not be more than 200 ft. (AASHTO, Table 10.4.2-1). For the perimeter of tank farms, there must be a minimum of one record at each corner. If there are minimal or no differences, this may be adequate. If not, a spacing of 200 ft along the berm or perimeter is necessary of there are erratic subsurface conditions encountered (AASHTO, Table 10.4.2-1). [GEO4]	We assume the reference here is to AASHTO's *LRFD Bridge Design Specifications* , which is a guide specification for vehicle bridge design and state highway transportation projects used by the general public. Nowhere in Table 10.4.2-1 is there a reference to tank farms, secondary containment, above ground storage tanks, or pipe supports. Furthermore, nowhere in OAR 340-300 (the "Rules") is there a minimum number, depth, or extent of exploration required. In our opinion, the historic subsurface data coupled with the data collected for this SVA are sufficient for assessing seismic vulnerability of an industrial facility. Additional exploration may be performed during the risk mitigation and implementation phase.
4d.	If CPTs are used, a few cases of verification of results should be compared to those from adjacent borings. Relationships between the SPTs, CPTs and full borings should be provided, using the latest geotechnical references and	The data includes CPTs and borings (see Figure A-2).

Table 1

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4e	Provide geologic cross sections (color) of the facility to provide stratigraphy of the site, and to establish the site classification. [GEO4]	A geologic cross section (color) is provided on Figure A-3. We are not aware of how the site classification is determined from a geologic cross section. A seismic site classification calculation is provided in Appendix A-3-1.
4f	If any other geotechnical data (other than COT, SPT, or borings) was available, provide details	None other available.
4g	Employ contemporary standards of practice for all new soil investigations. [GEO4]	Noted. CPT testing was completed in general accordance with ASTM D5778.
4h	Verify compliance with items (i) through (v) of OAR 340-300-0003(6)(a). [GEO4]	We are in compliance. Section 5.1.1 is laid out in the same format as the rules in OAR 340-300-0003(6) for easy comparison of our content to the rules. See SVA section 5.1.1, the subsection titles in the SVA match the subheadings under OAR 340-300-0003(6)(a).
5	The following considerations must be addressed in the geotechnical design report. [GEO5]	The SVA should not be considered a design report; nothing in the Rules indicate a design should be a result of the SVA. The future risk mitigation implementation plan may have elements of design. The SVA has been prepared to meet the requirements of the Rules and should be evaluated accordingly.

5a	Liquefaction potential in "Sand-Like" Soil and Cyclic Degradation in "Clay-Like" Soil How was cyclic resistance ratio evaluated (simplified or site-specific)? [GEO5]	Simplified. This is addressed in great detail in SVA section 5.1.1.2.2 and Appendix A-3. Supplemental information is provided below: The average Su,r discussed in SVA section 5.1.1.2.2 was computed based on liquefaction FS with a FS cutoff of 1.0 (see comment/response to question 5f for additional discussion on using FS=1.0). This means that all soil layers with FS<1.0 were assigned a residual shear strength based on their CRR (i.e., the sand-like approach), regardless of their Ic. In other words, layers that are clay-like and may only degrade 85 percent or so of their static strength were still assigned a sand-like residual shear strength if they were triggered (i.e., FS <1.0). This is a conservative approximation/simplification. As discussed in section 5.1.1.2.2, CPT soil behavior type Ic was used to determine susceptibility to liquefaction. Specifically, we selected Ic > 3.0 as not susceptible to liquefaction. Triggering calculations are performed on all soil layers (i.e., the liquefaction FS is computed for all soil layers) and is not automatically set larger than 2 if not susceptible. Typically, an Ic of 2.6 is used to determine susceptibility (i.e. Ic < 2.6 is not susceptible per Robertson and Wride 1998). Our experience with Willamette Silts/Willamette River Alluvium from several sites along NW St. Helens Blvd and Yeon Ave indicates that an Ic of 2.6 is typically an unconservative cutoff. However, we have not observed a strong correlation between plasticity index and Ic for these soils. Therefore, we have selected Ic of 3.0 as the cutoff based on the following justification. First, it represents a more fines-dominant soil behavior type grouping. Second, by using Ic > 3.0 as the susceptibility cutoff, we filter out all data with a normalized friction ratio F less than about 2 percent, indicating low likelihood of liquefaction because F is large (Robertson and Wride 1998). Finally, as shown by the cumulative frequency of Ic (see Attachment 1), using Ic of 3.0 excludes less than 1 percent of data from being susceptible to liquefaction. These three points indicate a conservative approach to determining susceptibility and sand-like vs. clay-like behavior. Our opinion is that this approach is suitable for a vulnerability assessment.
5b	If a site-specific response analysis has been performed, was it one or two dimensional?	Not applicable.
5c	What ground motion parameters were used?	See SVA section 5.1.1.2.1 and Appendix A-3.

5d	What methodology was used to calculate residual shear strength? [GEO5]	See SVA section 5.1.1.2.2.
5e	What safety factor for liquefaction in sand (CRR/CSR). [GEO5]	Not sure what is the question here? A safety factor of 1.0 was used to distinguish between triggered and not triggered susceptible soils.
5f	If using a simplified procedure, what current methodology has been used? Is the safety factor less than 1.4, what reduction factor has been applied to the initial shear strength of the soil? [GEO5]	See SVA Section 5.1.1.2.2. As noted above, a safety factor of 1.0 was used to create a simple binary condition of either fully liquefied or non-liquefied and no distinction was made for "partially liquefied" soil. This approach was selected for simplicity and is suitable for the vulnerability assessment. Attachment 1 provides additional analysis of the CPT data. Consider the plot of soil behavior type (represented with Ic) versus Liquefaction FS. The software program Cliq was used to complete liquefaction calculations. As shown with this plot by the data points with Ic > 3.0 (our susceptibility threshold - discussed previously on comment/response no. 5a) and FS<1.0, Cliq completes triggering calculations (i.e., Liquefaction FS) regardless of whether a soil is susceptible (i.e., whether Ic is > 3.0). This plot demonstrates that in our calculations, the quantity of data points triggered (i.e. liquefaction FS) and specifically FS> 1.4, is not artificially inflated due to susceptibility or non-susceptibility. Attachment 1 also shows cumulative frequency plots of liquefaction FS. As shown by the cumulative frequency, 1 percent of all individual CPT measurements resulted in a computed FS between 1.0 and 1.4. This corresponds to less than 1 ft of soil that is in a "partially liquefied" liquefied state. These two plots demonstrates that distinguishing a partially liquefied state for this site is not significant, especially for a study intended to determine general seismic vulnerability.
5g	If the Safety Factor is 1.0 < SF < 1.2, how have the seismically induced ground movements	See above.
5h	If the Safety Factor SF<1.0, what is the residual shear strength. [GEO5]	See SVA section 5.1.1.2.2.
6	Provide evaluations for other geotechnical hazards, if applicable: Slope movement, Lateral Spreading, Ground settlement, other surface	This is addressed in SVA Section 5.1.1.2. As this is a seismic vulnerability assessment, only seismic hazards were considered.

Table 1

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7a.	Is there a possibility that a slope failure could affect any component of the facility? [GEO7]	See SVA Section 5.1.2. Slope instability (i.e., seismically-induced landsliding due to inertial loading and not a result of liquefaction) is not a risk at this site. Lateral ground movement due to liquefaction-induced lateral spreading and flow failure is a risk.
7b.	If slope failure is possible, has a stability analysis been performed?	No, see above and below responses.
7c.	Are seismically induced ground movements considered?	Yes, the SVA considered liquefaction-induced settlement, lateral spreading, and flow failure.
7d.	If there are ground movements considered, what methods have been used to analyze them?	Liquefaction-induced settlement was evaluated using empirical procedures proposed by Idriss and Boulanger (2008) and (2014). Lateral spreading and flow failure were evaluated using limit-equilibrium methods and a Newmark sliding block analysis. This is all well documented in the SVA.
7e.	Is the expected seismic (DE) displacement	Yes.
8a.	What aspects of dynamic SSI have been evaluated (e.g., piles, pipelines, tanks, earth retention systems, or other)?	Rigorous soil-structure-interaction (SSI) analyses were not completed for a myriad of reasons. Chiefly, detailed SSI analysis is not useful for this vulnerability assessment. We have concluded that due to the potential for large liquefaction induced ground movements, it is highly likely that the performance objective [OAR 340-300-0002(18)] will not be met. SSI analyses may be appropriate during the risk mitigation implementation phase. SSI analyses are not required to arrive at the vulnerable conclusion.
8b.	What assumptions and procedures have been used to assess SSI?	See above.
9a.	Describe the local geologic and geomorphologic setting of the facility.	See section 5.1.1.1.2 of the SVA.
9b.	Include any and all historical geotechnical data, reports, or boring information.	See section 5.1.1.1.3, Appendix A Figures, Appendix A-1, and Appendix A-2 of the SVA.
9c.	Present the subsurface profiles in graphical	See Figure A-3 of the SVA.
9d.	Describe groundwater levels and possible artesian or sub-artesian conditions.	See section 5.1.1.1.4 of the SVA.
9e.	Identify main subsurface units, based on material type, strength, and deformability.	See section 5.1.1.1.4 of the SVA.
9f.	Assess lateral variability of subsurface units.	See discussion bullet "liquefaction/seismic strength loss" in section 5.1.1.2.2 of the SVA.

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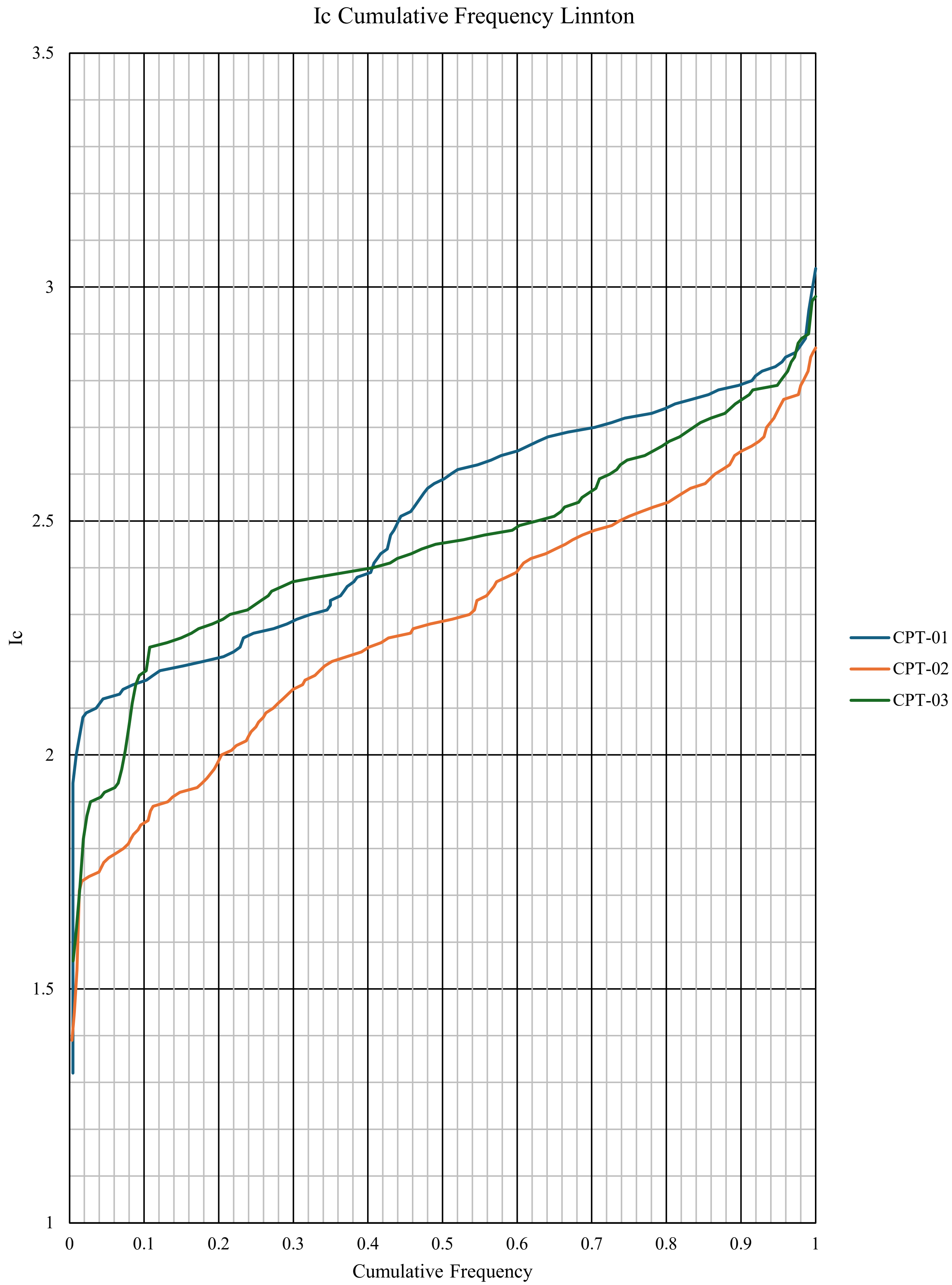
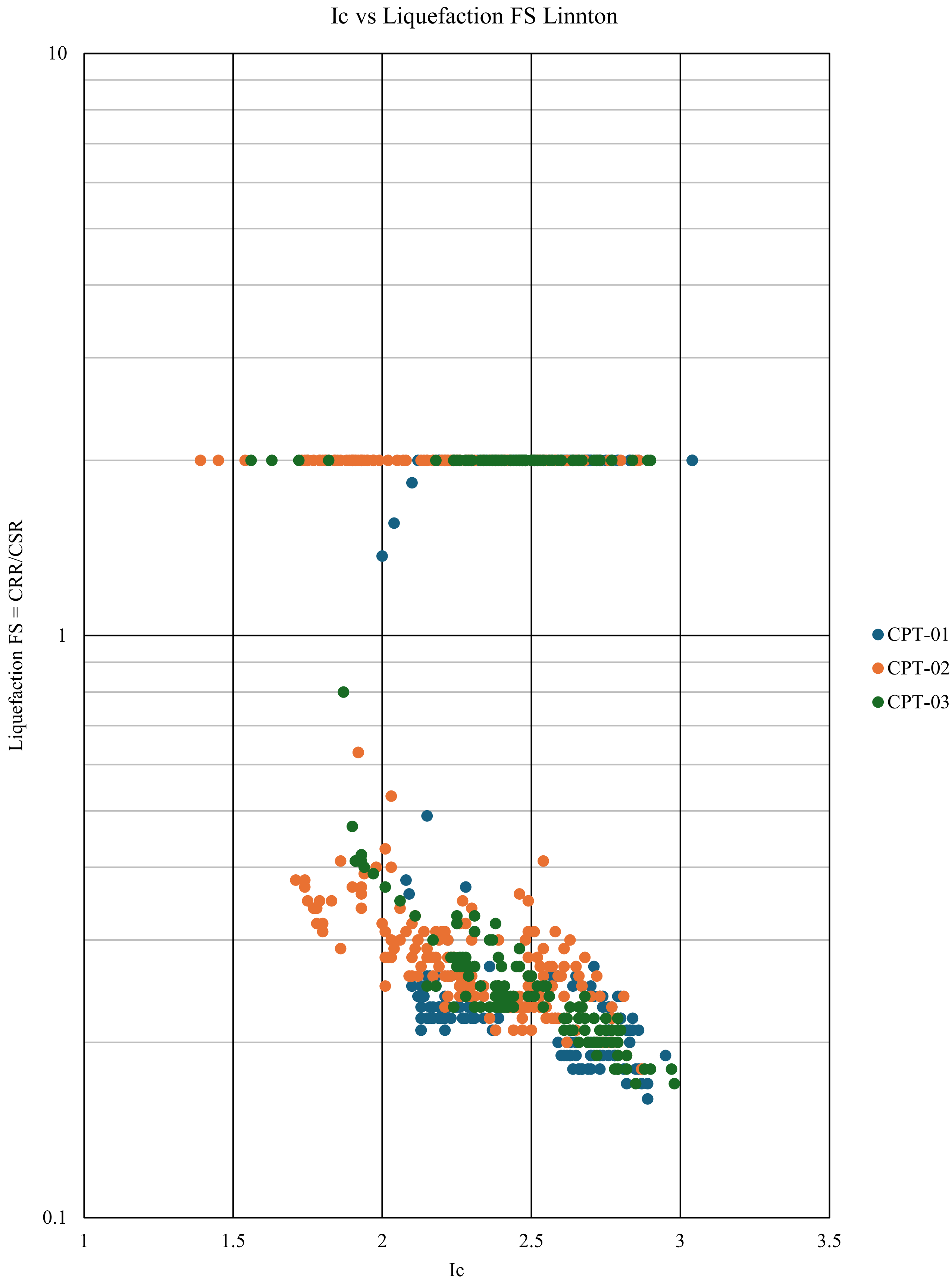
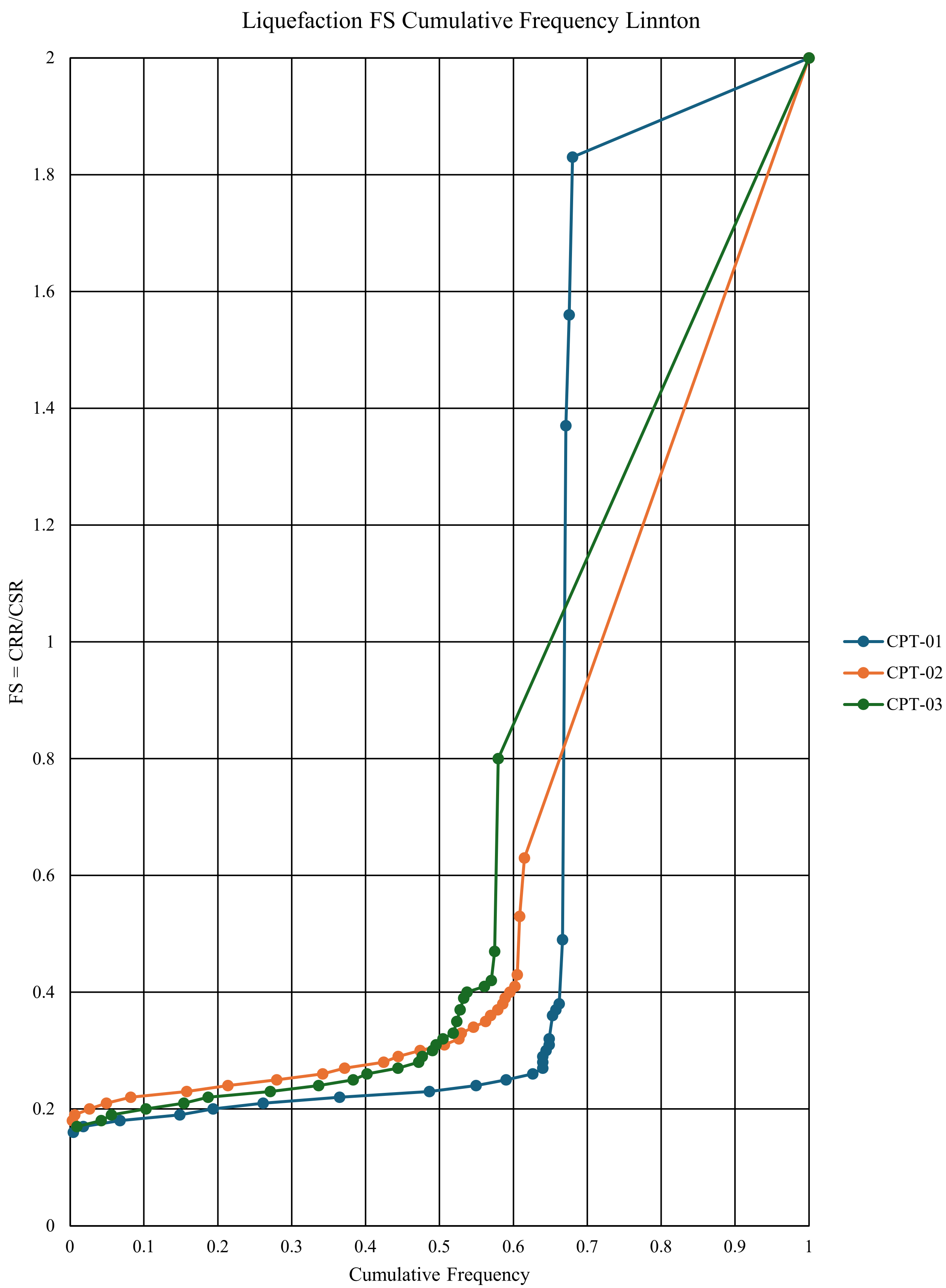
9g.	Summarize main soil and rock parameters, for each of the identified subsurface units.	See section 5.1.1.1.4 and Figure A-3 of the SVA.
9h.	Describe the lateral variability to the top of rock, where the rock is present within the depth of	See Figure A-3 of the SVA.
9i.	What is the likelihood of encountering rock or cobbles that might be present within the soil	This is addressed by Section 5.1.1.1.4 of the SVA.
9j.	Provide justification for the "site classification" (A-F) for this facility.	See discussion in section 5.1.1.2.1 of the SVA and also calculation in Appendix A-3.
9k.	Any additional requirements per Oregon Specialty Code, Section 1803.6.	Noted.

Attachment 1
Supplemental Information


Ic Cumulative Frequency Linnton
3.5
3
2.5
2
1.5
1
Ic
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
Cumulative Frequency
CPT-01
CPT-02
CPT-03


Ic vs Liquefaction FS Linnton
Liquefaction FS = CRR/CSR
10
1
0.1
1
1.5
2
2.5
3
3.5
Ic
CPT-01
CPT-02
CPT-03



Liquefaction FS Cumulative Frequency Linnton
FS = CRR/CSR
Cumulative Frequency
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
CPT-01
CPT-02
CPT-03