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Project No. TET 16-93E
June 16, 2017

Mr. Luis Valenzuela
M&D Properties
3100 E Imperial Highway
Lynwood, CA 90262

Subject: **ADDENDUM No.1 − GEOTECHNICAL INVESTIGATION REPORT**
PLAZA MEXICO RESIDENCES
3000 E Imperial Highway
Lynwood, California

Reference: Geotechnical Report, Plaza Mexico Residences, 3000 Imperial Highway, Lynwood, California by Tetra Tech, dated January 13, 2017

Dear Mr. Valenzuela:

Presented herein is Tetra Tech's Addendum No.1 to the above-referenced geotechnical report for the proposed Plaza Mexico residential complex (Plaza Mexico Residences) dated January 13, 2017. This addendum presents revised liquefaction analyses considering recently updated methods for evaluating liquefaction triggering and post-liquefaction settlement. In addition, the latest version of CLiq v.2.1.6.11 released on April 6, 2017 was used. The revised results of the post-liquefaction settlement evaluation indicate smaller, albeit still significant, settlements than those reported in our original report. Therefore, the pile foundation system recommended in our original Geotechnical Report remains valid. However, due to the reduction in the magnitude of these settlements, this addendum introduces recommendations for a foundation alternative including ground improvement. The appendix of this addendum includes the details of the results of the liquefaction analyses.

We appreciate the opportunity to provide our professional services on this project. If you have any questions regarding this addendum or if we can be of further service, please do not hesitate to contact the undersigned.

Respectfully submitted,
Tetra Tech, Inc.

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INTRODUCTION

This addendum presents the results of the revised liquefaction analyses considering the recently updated methods for liquefaction evaluation which are based on a larger data base including data from new liquefaction cases and a revised database of existing liquefaction cases. This addendum also includes the liquefaction analyses for the Cone Penetration Tests (CPTs) using the latest version of CLiq v.2.1.6.11 released on April, 2017.

The results of the updated liquefaction analysis continue to validate the recommendation of the driven pile foundation system included in our original Geotechnical Report. However, this addendum also presents additional recommendations for a foundation alternative with ground improvement in combination with shallow foundations.

The provisions and recommendations contained in our original Geotechnical Report remain valid unless specifically superseded as stated in this addendum. The following sections in the original Geotechnical Report dated January 16, 2017 are superseded by those noted in this Addendum:

- Sections 7.4.4 Evaluation of Liquefaction Potential and Sensitivity Analyses
- Sections 7.4.5 Dynamic Settlement
- Section 8.1 General
- Sections 8.6 B must be added to the original report.
- Section Selected References
- Appendix D − Liquefaction Analyses

This addendum can only be read and understood in conjunction with the original Geotechnical Report.

Replacement of Sections 7.4.4 and 7.4.5
in Chapter 7. SEISMIC AND GEOLOGIC HAZARDS

7.4.4 Evaluation of Liquefaction Potential and Sensitivity Analyses

Liquefaction potential of on-site soils was evaluated from SPT blowcounts as well as from CPT probes. Generally, significant liquefaction potential was identified in all borings and probes. Liquefiable intervals were identified in various thickness and elevations at depths between 8 and 60 feet and composed about 30 to 45 percent of the soil profile within this interval.

The liquefaction potential of cohesionless (sandy) soils was evaluated based on the SPT blowcounts and laboratory test results utilizing procedure published by Boulanger and Idriss (2014) and generally as recommended in the County of Los Angeles Administrative Manual, Liquefaction/Lateral Spreading/GS045.0 dated October 6, 2014. Cohesive soils with a Plasticity Index less than 7 were considered to be susceptible to liquefaction and considered to behave as cohesionless materials for analysis purposes.

The analyses based on Standard Penetration Tests (SPTs) considered an energy ratio correction factor C_E of 1.25. This ratio is based on Table 5.2 of the Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California (SCEC, 1999). For an automatic trip hammer the table suggests the energy ratio correction factor range from 0.9 to 1.6 (modified from Youd and Idriss, 1997). Consequently, the selected design energy ratio correction factor of 1.25 is an average and reflects a hammer efficiency of approximately 75 percent, which is consistent with our experience with similar equipment. The blowcounts recorded for soils driven with the 3-inch O.D. California Sampler with brass rings were converted to an equivalent SPT blowcounts using a reduction factor of 0.67. Borehole diameter correction factor C_B of 1 based on the internal diameter of the hollow stem auger system used for the drilling was utilized in our liquefaction evaluation.

Results of SPT-based liquefaction analyses of granular soils are summarized in Table 1 in the next section of this addendum and individually presented in Appendix D.

The liquefaction potential of the subsurface materials was also evaluated from the CPT data using the computer software CLiq v.2.1.6.11 by Geologismiki released in April 2017. The liquefaction potential and the liquefaction induced settlements were evaluated using the Boulanger and Idriss method (2014). Results of the liquefaction analyses using CPT data are summarized in Table 1 in the next section of this addendum and presented in detail in Appendix D.

Seismic sensitivity of fine-grained soils (clays and silts) was further evaluated per County of Los Angeles Administrative Manual GS045.0 with modifications proposed by Idriss and Boulanger (2008) and the fine-grained soils were classified in the following 3 categories:

1. Soils with Plasticity Index < 7 and below groundwater are classified as fine-grained soils susceptible to liquefaction (typically includes silts);

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2. Soils with Plasticity Index > 18 and a degree of sensitivity S_t > 6 are classified as fine-grained soils potentially susceptible to significant loss of strength during seismic shaking and require additional evaluation. The sensitivity of the on-site fine-grained soils is evaluated based on the water content, Atterberg limits, and effective vertical stresses using the procedures suggested by Holtz and Kovacs (1981) and Terzaghi, Peck and Mesri (1996).

3. Fine-grained soils falling outside the two categories above are considered to behave like clays and are not considered susceptible to liquefaction or seismic sensitivity.

Analyses of the sensitivity of the saturated fine-grained soils indicated low sensitivity based on the estimated sensitivity ratios of 1.9 to 4.5 as evaluated from Terzagi, Peck and Mesri (1996). The soil sensitivity was also estimated from the CPT data which indicated that the fine-grained soils at the site ranges in sensitivity between 0.9 and 4 with most values on the order of 1.1 to 2, i.e., significantly less than the critical sensitivity threshold value of 6. Therefore, the onsite cohesive soils are not considered to be susceptible to undergo seismically induced loss of strength. Consequently, the potential for significant loss of strength of fine-grained materials and ensuing bearing failure or significant deformations during seismic shaking is considered low. The results of the sensitivity analyses for the soil borings are included in Appendix D.

7.4.5 Dynamic Settlement

Dynamic settlement can occur in both dry and saturated sands when loose to medium-dense granular soils undergo volumetric changes during ground shaking. Dynamic settlement can occur in saturated sands due to liquefaction or in dry sands due to densification of the soil matrix. The anticipated dynamic settlement of the saturated soils at the site was evaluated using SPT data from the current exploration using procedures outlined by Yoshimine et al (2006). The estimated settlements by Yoshimine et al (2006) were further adjusted by a calibration factor of 0.9 recommended by Cetin (2009) to match the observed settlements. The potential for dry dynamic settlement using SPT data was calculated according to the procedure outlined in Pradel (1998a and 1998b).

The anticipated dynamic settlement of the saturated soils at the site was evaluated from the CPT data from the current exploration using the computer software CLiq v. 2.1.6.11. The settlement of saturated soils was computed using the procedure outlined by Boulanger and Idriss (2014). The potential for dry dynamic settlement was evaluated using the CPT data according to the procedure outlined by Robertson and Shao (2010). The transition layer detection option, which excludes from the liquefaction settlement calculation data points within transition layers and thus eliminates some of the conservatism inherent in CPT predicted settlement, was activated in CLiq as recommended by Robertson (2008). The details of dynamic settlement analyses are presented in the updated Appendix D.

Table 1 presents the results of the liquefaction analyses and includes both post-liquefaction and the seismically-induced settlement of materials above groundwater table. As shown in Table 1, the combined dynamic settlement at the ground surface of the on-site soils was estimated from the SPT data to range between 2.8 and 7.7 inches and from the CPT data to range between 2.1 and 4.9 inches. These results obtained by different methods match relatively well and therefore increase confidence in their reliability. For total seismically-induced settlements exceeding 4 inches, structural mitigation of the settlement effects is usually not practical and ground improvement and/or pile-supported foundations to mitigate the effects of settlement is recommended.

Table 1
Results of Post-liquefaction and Dry Dynamic Settlement Analyses

Boring No.	Assumed Groundwater Depth (feet)	Liquefiable Zone Depth Interval [1] (feet)	FS_{liq}	Post-liquefaction Settlement (inches)	Settlement of Dry Sands (inches)	Combined Dynamic Settlement (inches)	Dynamic Settlement After Ground Improvement to 20 feet (inches)
B-105	8	15-20 40-45 50-57	0.26 0.24 0.23	7.7	negligible	**7.7**	**5.9**
B-109		15-20 30-35 35-40 45-50	0.49 0.51 0.58 0.41	3.5	negligible	**3.5**	**2.3**
B-110		10-13 20-25 25-30 40-45 55-60	0.52 0.74 0.53 0.46 0.49	5.4	0.9	**6.3**	**4.3**
B-111		10-13 20-25 40-50	0.73 0.51 0.95	2.5	0.3	**2.8**	**1.9**
C-101		17-18 23-26 35-50 52-57	0.3 0.3 0.5 0.6	2.0	0.1	**2.1**	**1.3**
C-102		8-14 22-28 31-38 41-48 53-57	0.3 0.3 0.3 0.6 0.7	2.6	0.1	**2.7**	**1.5**
C-103		15-19 21-25 29-39 42-46 53-57	0.3 0.3 0.3 0.6 0.7	4.2	0.1	**4.3**	**2.8**
C-104		11-21 26-36 40-48 55-59	0.3 0.3 0.5 0.6	4.8	0.1	**4.9**	**3.0**

Note: [1] Depth below existing grade

Assuming that total settlements on the order of 4 inches can be mitigated by the structural design, ground improvement to mitigate the anticipated seismically-induced settlements within the range of 2.1 to 7.7 inches to less than about 4 inches is recommended. This can be accomplished by elimination of the seismically-induced settlement in the upper 20 feet of the site soils as illustrated in the last column of Table 1. Deeper ground improvement depths may be considered, if only partial mitigation of liquefaction settlement is achieved within the upper 20 foot zone, to reduce the overall seismically-induced settlement to less than 4 inches. Additional consideration for the ground improvement method may be to provide enough support for conventional floor slabs so that structural floor slabs could be eliminated. This would likely require a denser grid of ground improvement points.

The ground improvement methods may include replacement, partial replacement, strengthening, or densification of the soil. The selection of the ground improvement method usually depends on the economics of the available methods and on the structural design of the proposed building and its foundations. A key consideration in selecting an effective ground improvement method will be the ability to improve zones consisting mostly of interbedded layers of fine-grained soils (i.e., sandy silts and silts) and silty sands. The effectiveness of the method will need to be verified by a field test as discussed in Section 8.6 B. Given the presence of contaminated soils at the site, methods that minimize removal and handling of the contaminated soils are likely to be preferred.

Replacement of Section 8.1 in Chapter 8. DESIGN RECOMMENDATIONS

8.1 General

Based on the results of the field exploration and engineering analyses, it is Tetra Tech's opinion that the proposed construction is feasible from a geotechnical standpoint, provided that the recommendations contained in this addendum and in the referenced Geotechnical Report are incorporated into the design plans and implemented during construction. To support the proposed development the following two foundation systems may be considered:

1. A deep foundation system consisting of driven precast concrete piles connected by grade beams supporting a structural concrete slab.

2. A shallow foundation system in combination with ground improvement.

The recommended deep foundation system consisting of driven precast concrete pile foundation is expected to be installed relatively easily using conventional construction methods. An important benefit is that the installation does not generate any soil cuttings and thus minimizes the need to handle contaminated soil. Because of the large magnitude of the liquefaction induced settlement, a structurally supported slab spanning between pile caps and supporting the ground floor loads is recommended. Within the parking structures an alternative of flor slab-on-grade could be considered since damage to the slab due to liquefaction-induced settlement following a design seismic event could be relatively readily repaired. Subgrade overexcavation and recompaction possibly in combination with geogrid reinforcement of the subgrade would still be required. This approach would result in need to potentially handle contaminated soil

Cast-in-drilled-hole (CIDH) piers were also considered for the deep foundation system, but because of the significant volume of excavated soil and the resulting need to handle contaminated material, this system is not expected to be cost effective. However, drilled displacement piles, also called auger pressure grouted displacement piles, although relatively costly, may be considered because they generate relatively minor drill cuttings volume and could provide higher load capacities, if needed, than the herein recommended driven precast concrete piles.

The ground improvement with the shallow foundation system will reduce the seismically induced settlement and the compression settlement due to the structural loads, to levels manageable by the structural design of the superstructure. The ground improvement method should minimize the excavation and handling of the onsite contaminated soils. Given these considerations, it is expected that Rammed Aggregate Piers (e.g. Geopier Rammed Aggregate Piers (RAPs) built with the Impact System, or Hayward Baker Vibro Aggregate Piers, or similar equivalent product) could be considered. A key consideration in selecting an effective ground improvement method will be the ability to improve the interbedded layers of fine-grained soils (i.e., sandy silts and silts) and silty sands. It should be noted that a RAP ground improvement will generally consist of an overall ground improvement of the project site to mitigate the seismically-induced settlement and an augmented ground improvement to increase the bearing capacity and provide support to the foundation itself.

The conventional shallow foundation resting on the improved ground could consist of either a mat foundation or footings with a structural concrete slab. If the ground is improved sufficiently in between the footings, then a conventional slab-on-grade instead of structural slab may be considered.

The shallow foundations with ground improvement should be designed to accommodate 4 inches of total settlement (combination of seismically-induced and static settlement) and up to 2 inches of differential settlement over a distance of 30 feet. The bearing capacity and the subgrade modulus for the design of the mat foundation should be determined based on the method of the ground improvement. For reference purposes the bearing capacity and subgrade modulus for the existing unimproved ground between the RAP piers and for the conventionally overexcavated and recompacted on-site soils to a depth of 3 feet is shown in the following table.

Design Parameters for Soil between RAPs for Mat Foundation and for Slab On-Grade

Mat foundation or Floor Slab-on-Grade	On existing, unimproved ground	On conventionally overexcavated and recompacted on-site soils to a depth of 3 feet
Allowable bearing capacity (psf)	1,500	1,700
Subgrade Modulus k_1 for 1-foot-square plate (pci)	70	115
Scaling of subgrade modulus (pci) (Where *B* and *L* are the width and length of the element in feet, respectively, while *B* is no more than 14 times the thickness of the element, i.e., mat)	$k = k_1 \dfrac{1 + 0.5 * \frac{B}{L}}{1.5 * B}$	

Alternatively pad and continuous footings shallow foundation system supported on the improved ground and a floor slab may be considered The floor slab may be a structural slab spanning between the footings or also supported on RAPs or on overexcavated and recompacted on-site soils to mitigate the effects of the relatively weak near-surface soils. For reference purposes for the design of the conventional floor slab-on-grade the subgrade modulus for the existing, unimproved ground between the RAP piers and for the conventionally overexcavated and recompacted on-site soils to a depth of 3 feet is shown in the table above.

It is recommended that the designer of the RAP ground improvement works in conjunction with the Structural Engineer to optimize the costs structural mitigation and the costs of ground improvement mitigation.

Observations and laboratory tests indicate that the on-site soils have negligible levels of water-soluble sulfates, therefore, the soils are not expected to cause injurious sulfate attack on concrete with a minimum 28-day compressive strength of 2,500 psi.

Observations and laboratory tests indicated that the on-site soils have a low expansion potential and therefore expansion of the near surface materials is not likely to be a problem at the site.
The design recommendations presented below reflect these considerations.

The design recommendations presented below are based on Tetra Tech's current understanding of the project. Once the project configuration is finalized and the design is complete, Tetra Tech should review the plans and specifications to evaluate if the geotechnical design recommendations have been incorporated as intended.

Addition of Section 8.6 B in Chapter 8. DESIGN RECOMMENDATIONS

8.6 B Rammed Aggregate Piers

As discussed above, RAP ground improvement in combination with a shallow foundation system may be considered to mitigate the adverse effects of seismically induced settlement and the generally weak on-site soils. The RAP ground improvement should be installed under the entire development to mitigate the effects of seismically induced settlement and augmented, as appropriate, under the shallow foundations and, if so selected, also under the floor slabs.

The installation of RAP groups under the shallow footings will substantially increase the allowable bearing capacity by reinforcing the soil mass and enhancing strength and consolidation characteristics of the foundation subgrade.

RAPs are constructed by compaction of a suitable aggregate gradation. A displacement 12-inch-diameter mandrel is driven into the soil to the required depth with no casing and no soil spoils making this system particularly appealing for sites with contaminated soils. The RAP elements are then constructed by slowly withdrawing the mandrel while feeding the aggregate material to the bottom of the hole and applying direct vertical ramming energy to densely compact successive lifts of the aggregate to form engineered high stiffness columns about 18 to 24 inches in diameter. The vertical ramming action also improves the soil surrounding the hole, which results in settlement control and greater bearing pressures for design. Typical aggregates used for the pier construction include crushed rock or aggregate base.

For preliminary consideration, it is anticipated that RAPs will be installed to a depth of about 20 feet below foundation subgrade. The RAP's are typically 24 to 30 inches in diameter. Although not expected to be encountered during construction, this RAP system can be effectively used for installation below groundwater.

Based on our experience with such systems and based on review of the subsurface soils encountered at the site, the allowable bearing capacity using the RAP system may increase from approximately 1,500 to 2,000 psf to up to approximately 5,000 to 6,000 psf. The coefficient of sliding friction between the bottom of footing and the aggregate pier reinforced subgrade may be taken as approximately 0.5. The foundation design parameters are provided by the RAP specialty contractor, as they depend on the type of the aggregate and the layout and size of the RAP elements. The RAP foundation system shall be designed to limit the anticipated foundation load-induced total settlements to 1 inch or less, and anticipated differential settlement between adjacent footings to approximately 0.5 inch or less. The footing subgrade should be at least 24 inches below the lowest adjacent grade or top of the slab, whichever is lower.

It is recommended that a qualified specialty contractor be retained to design and install the RAP system. Prior to initiating the installation, we recommend that a test pier cell be installed on site to observe the performance of the RAP in the in-situ material and measure the degree of improvement of the soil located in between the RAPs. The field tests should include:

1. Load test of an individual RAP under axial load to observe the settlement-stress and settlement-time response of the pier.

2. Advance 2 CPTs before and 2 CPTs after installation of the RAP cell to quantify the ground improvement and verify that the liquefaction performance after improvement conforms to the design parameters.

Installation of the RAPs should be carried out under the observation of a representative of the Geotechnical Engineer of Record.

CLOSURE

This document is intended to be used only in its entirety and in conjunction with the original Geotechnical Report for this project dated January 13, 2017. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Tetra Tech should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. Reliance by others on the data presented herein or for purposes other than those stated in the text is authorized only if so permitted in writing by Tetra Tech. It should be understood that such an authorization may incur additional expenses and charges.

Tetra Tech has endeavored to perform its evaluation using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area in similar soil conditions. No other warranty, either expressed or implied, is made as to the conclusions and recommendations contained in this addendum.

Replacement of Chapter 12. SELECTED REFERENCES

12. SELECTED REFERENCES

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Replacement of Appendix D – Liquefaction Analyses

Appendix D

Liquefaction Analyses

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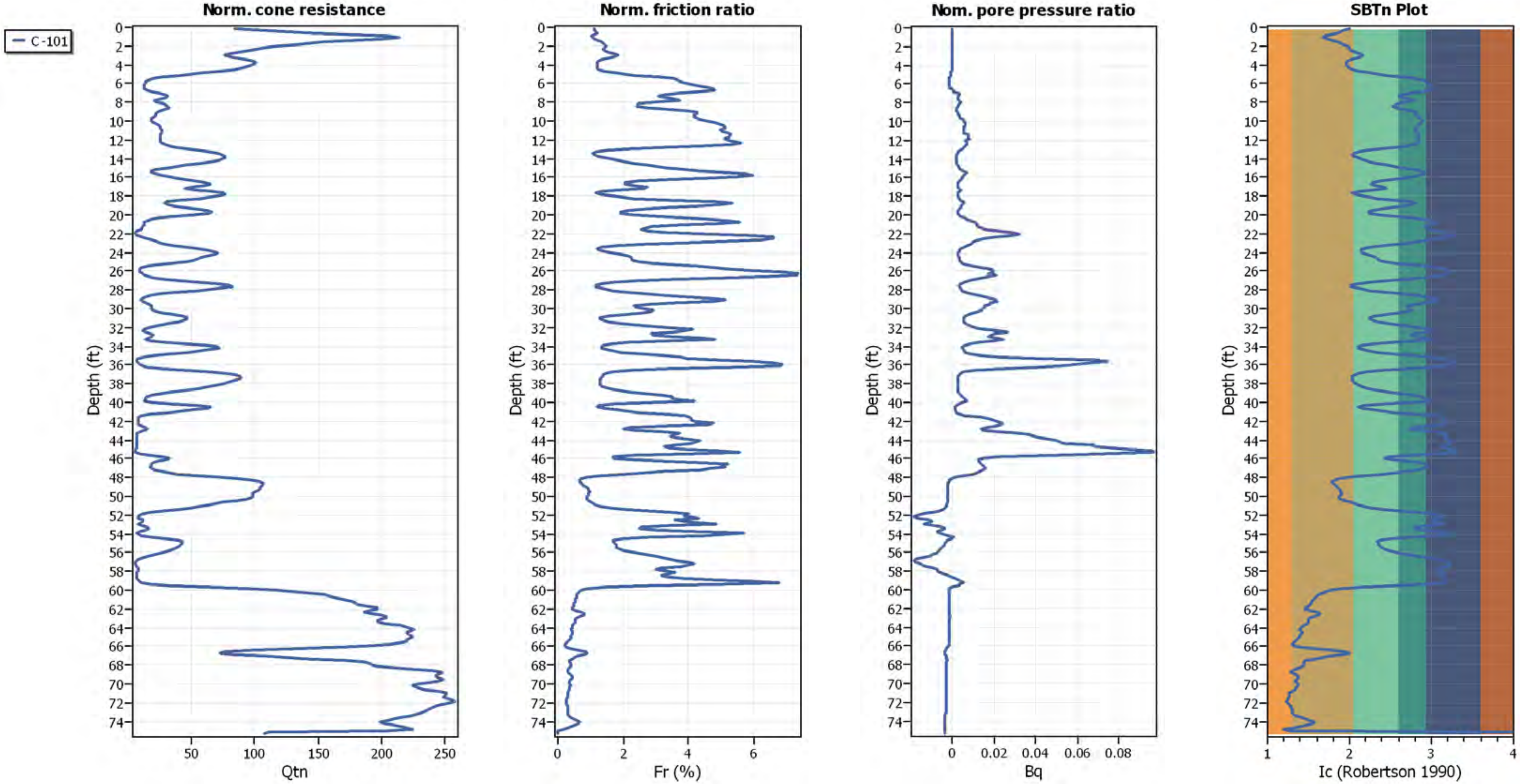
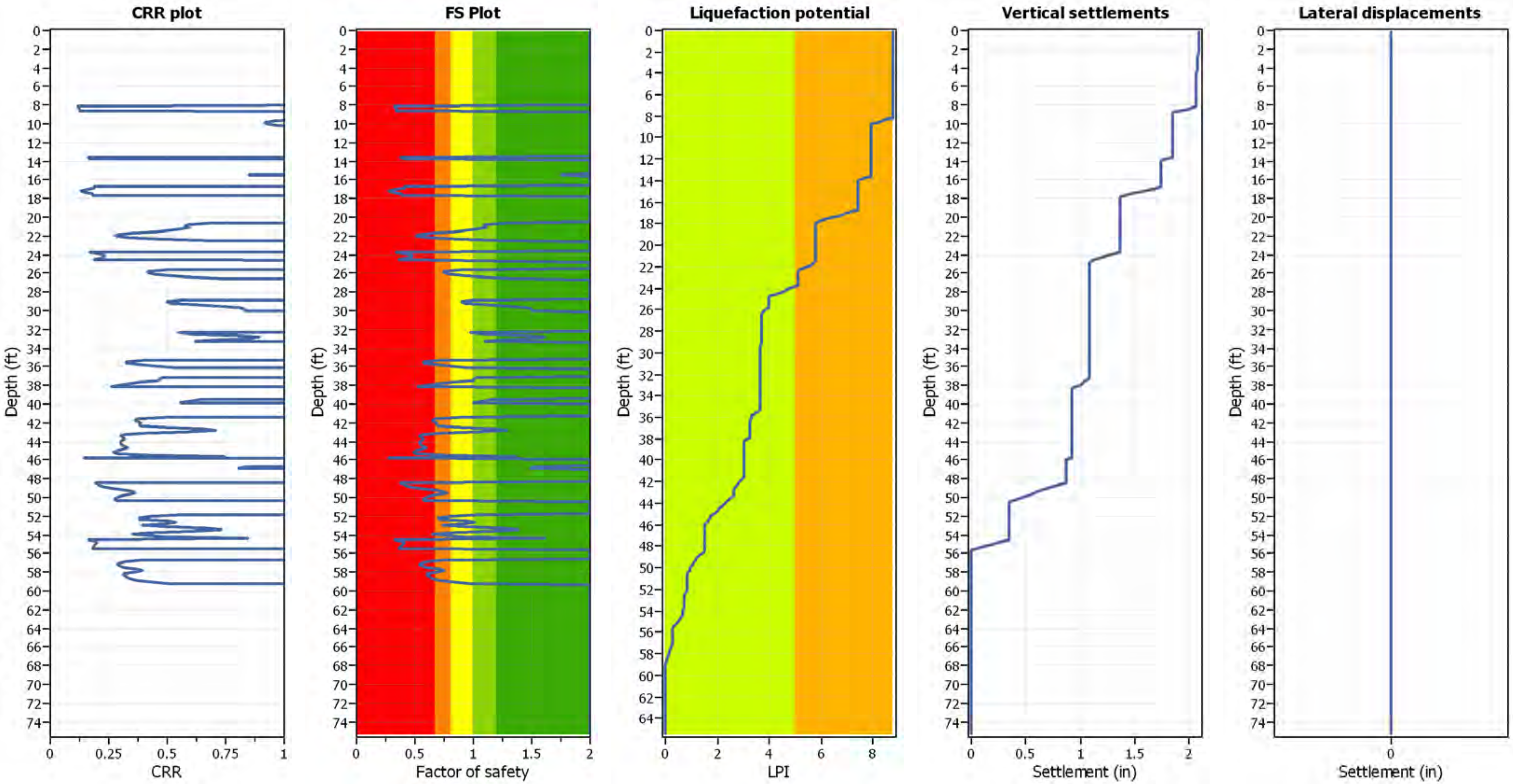
Project: 3000 Imperial Highway Revised

Overlay Normalized Plots



Project: 3000 Imperial Highway Revised

Overlay Cyclic Liquefaction Plots



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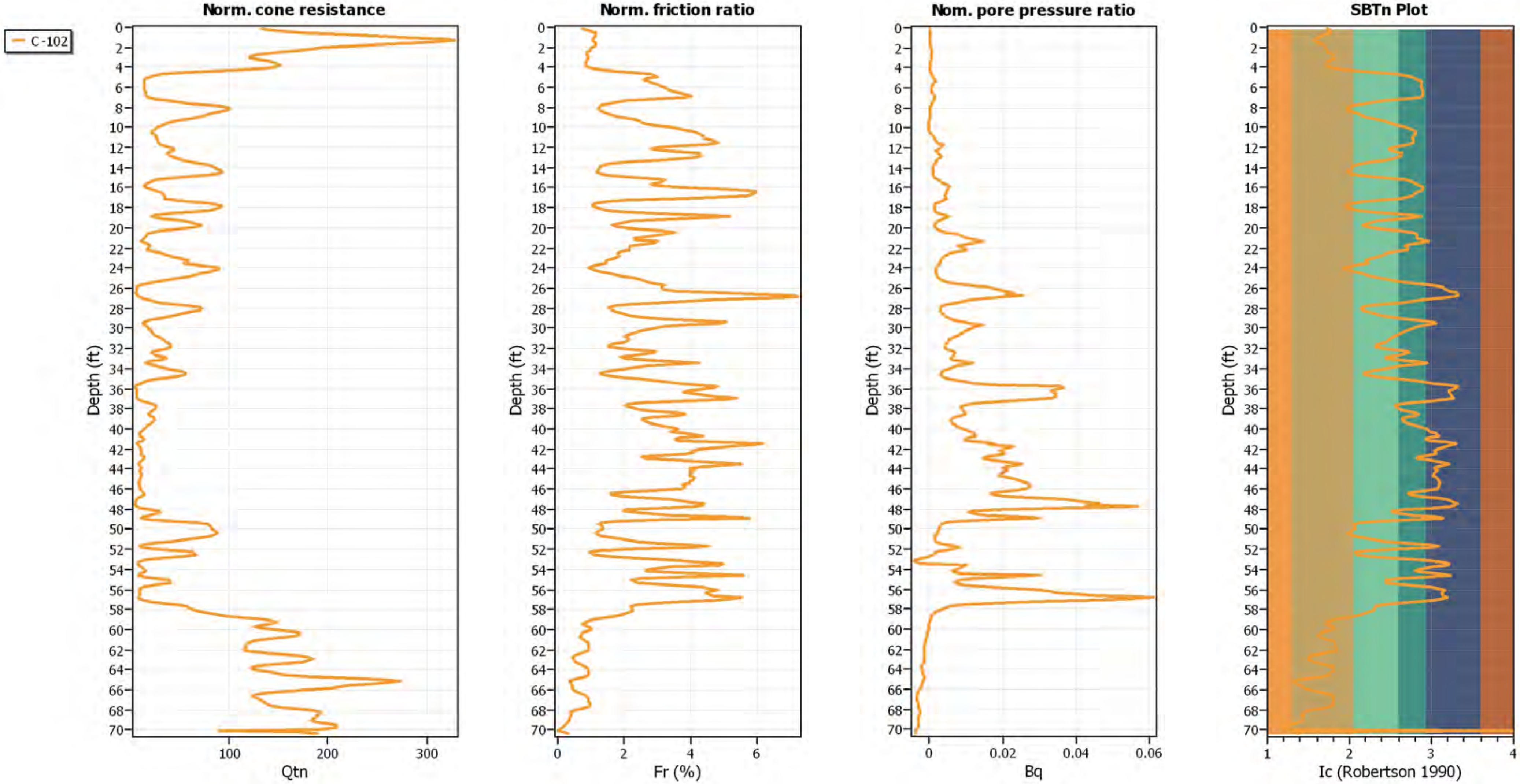
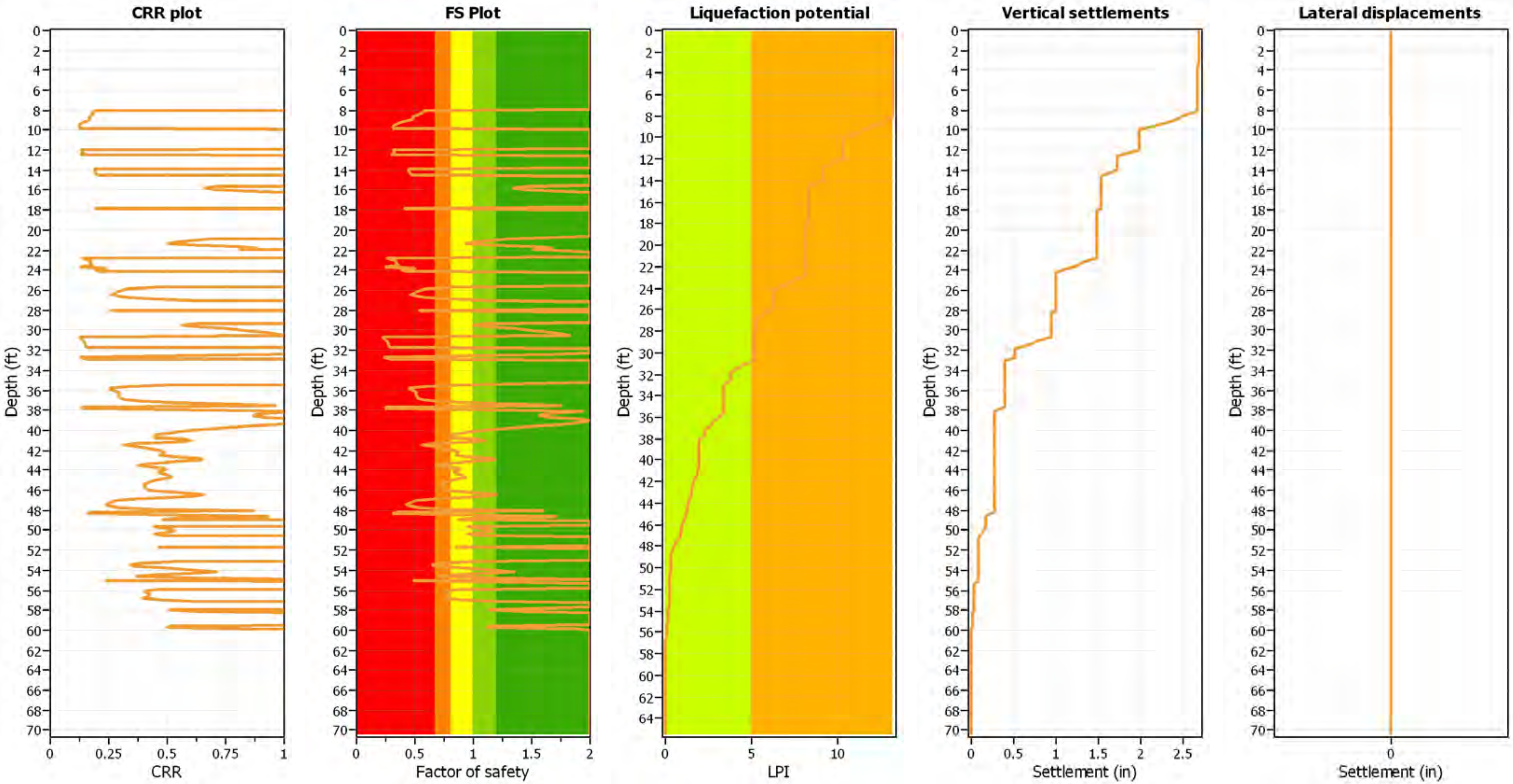
Project: 3000 Imperial Highway Revised

Overlay Normalized Plots



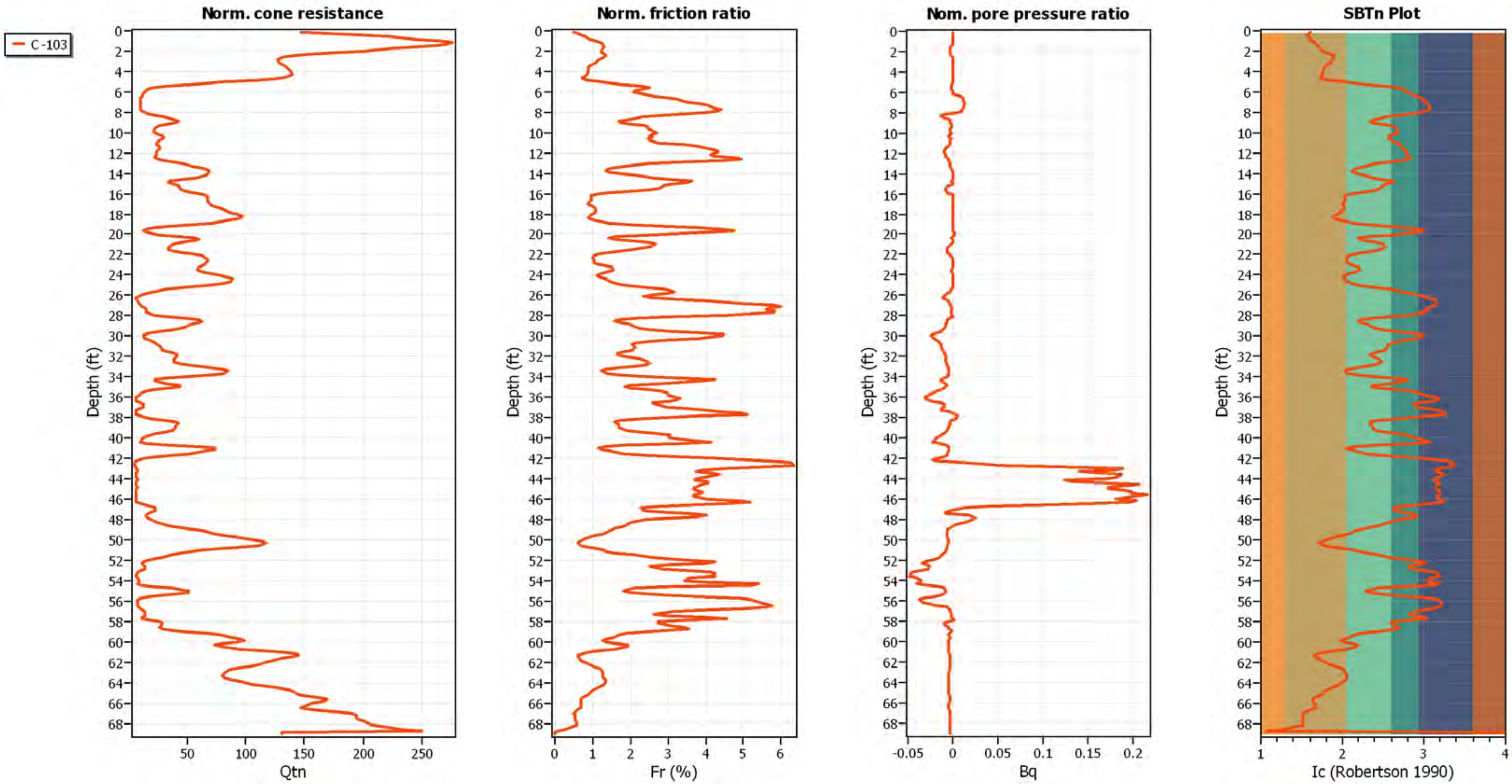
Project: 3000 Imperial Highway Revised

Overlay Cyclic Liquefaction Plots



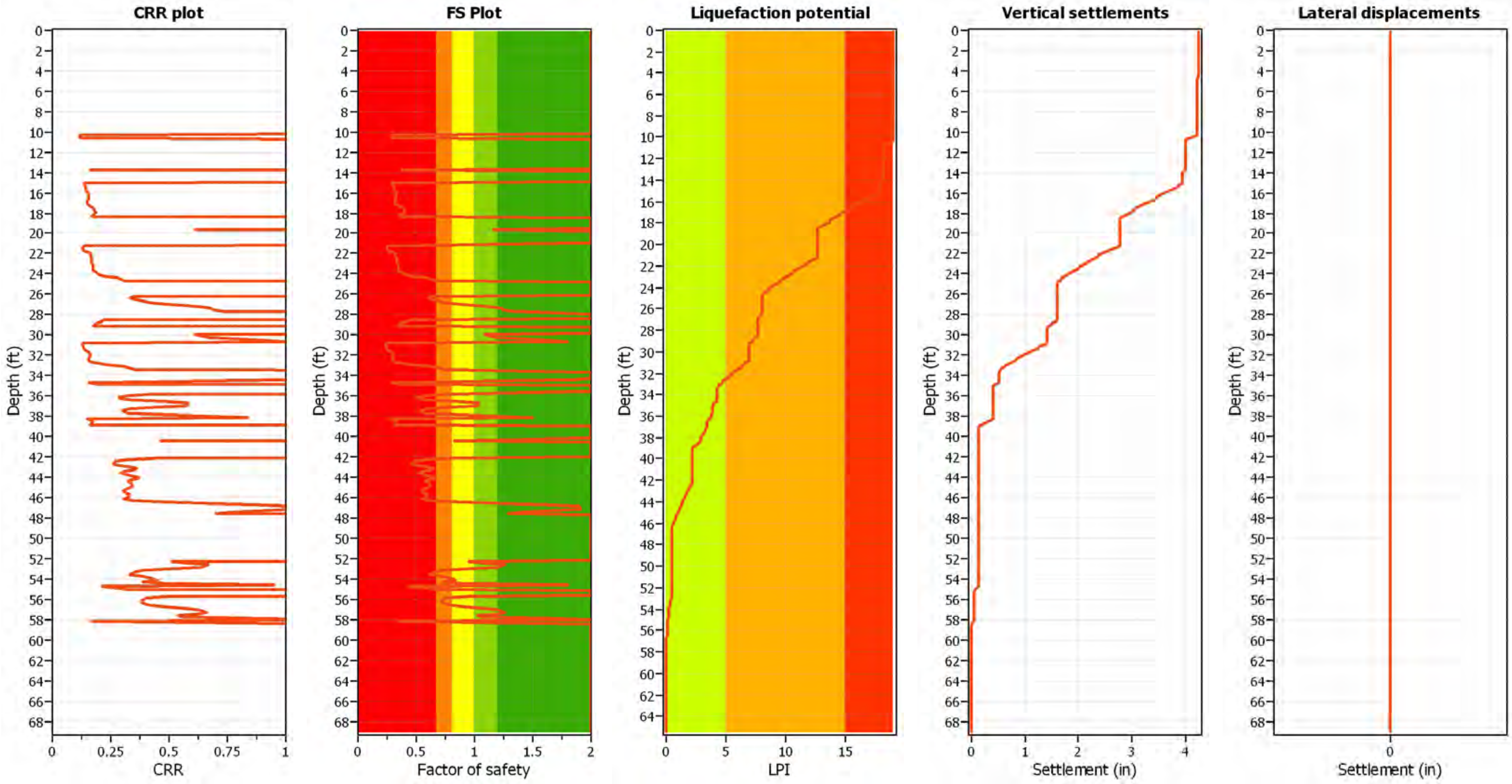
Project: 3000 Imperial Highway Revised

Overlay Normalized Plots



Project: 3000 Imperial Highway Revised

Overlay Cyclic Liquefaction Plots



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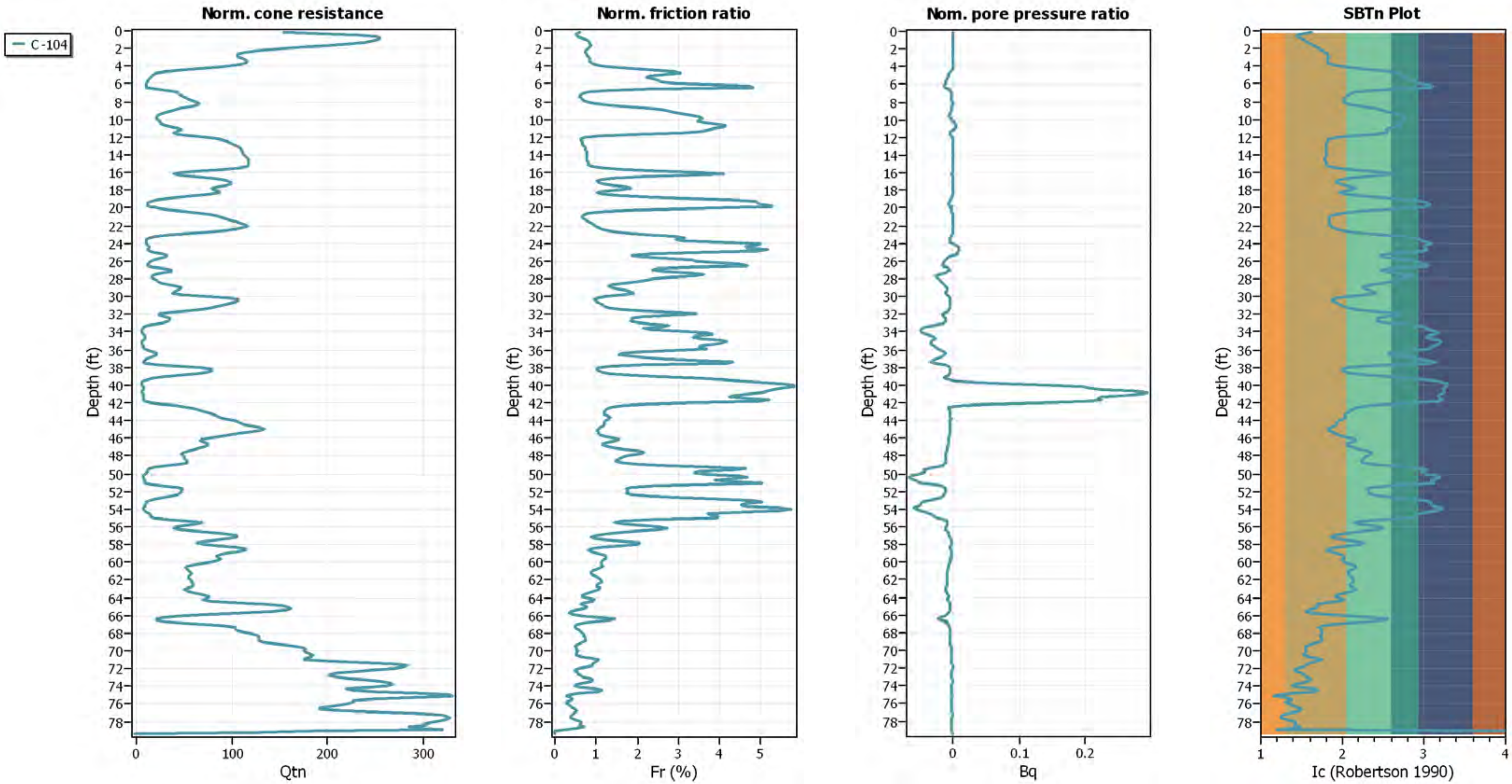
Project: 3000 Imperial Highway Revised

Overlay Normalized Plots



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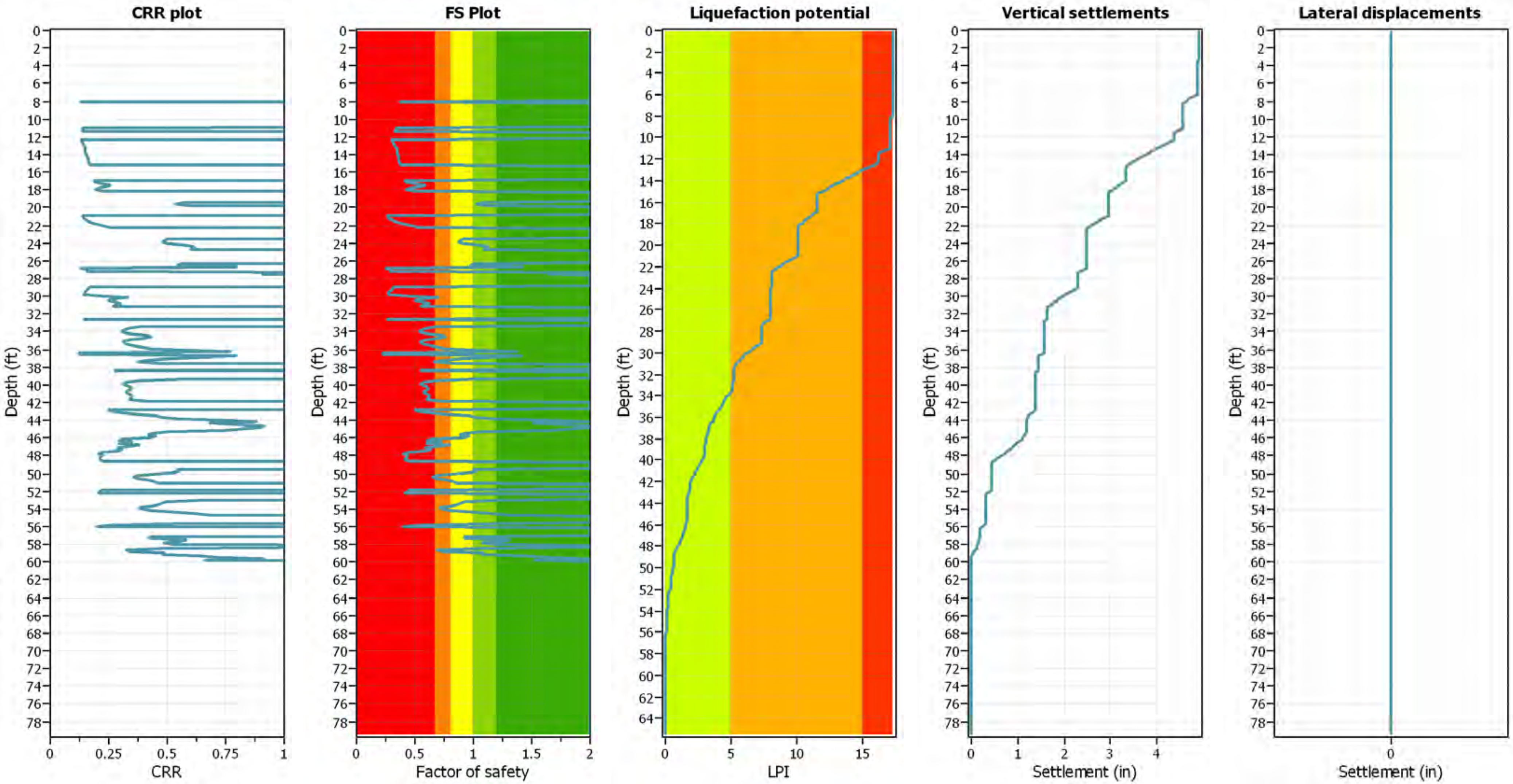
Project: 3000 Imperial Highway Revised

Overlay Cyclic Liquefaction Plots



Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	TET 16-XXXE 3000 Imperial Hwy., Lynnwood	Boring:	**B-105**	Engineer:	FC	Date:	6/12/2017

Liquefaction Evaluation Method

Correction for fines content	Idriss & Boulang. 2008, 2014
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014
Magnitude scaling factor MSF	Idriss & Boulang. 2014
Liquefaction settlement	Yoshimine et al., 2006 – no adjustment
Dry settlement	Pradel, 1998a,b

Liquefaction Analysis Statistics

Total thickness of evaluated profile	60 feet
Profile thickness susceptible to liquefaction	20 feet
Number of evaluated intervals	12
Number of potentially liquefiable intervals	4
Average Factor of Safety of sandy intervals	**0.4**
Dry sand settlement	0.00 inches
Liquefaction settlement	8.64 inches
Total earthquake-induced settlement	**8.64 inches**

Plasticity Index sand behavior threshold 7

Calculate settlement only for layers with less than 50 % of fines

Depth to Layer Top	Layer Thickness	Total Unit Weight In-situ	Total Unit Weight Design	Fines %	Plasticity Index	Considered Blowcounts SPT-N	N1,60	N1,60,cs	Liquefaction Factor of Safety	Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf	–	–	in	in
0	8	120.75	121.0	60	0	4.0	7.7	13.3	Not liquefiable	- no groundwater	0.00	8.64
8	2	120.75	125.0	60	11	20.0	28.1	33.7	Not liquefiable	- clay-like behaviour	0.00	8.64
10	3	120.75	125.0	60	11	3.0	4.8	10.4	Not liquefiable	- clay-like behaviour	0.00	8.64
13	2	120.75	125.0	60	11	6.0	8.5	14.1	Not liquefiable	- clay-like behaviour	0.00	8.64
15	5	120.75	125.0	45	0	4.0	5.7	11.3	0.26	- liquefieable - FS < 1.3	2.09	8.64
20	5	120.75	125.0	60	11	16.0	20.6	26.2	Not liquefiable	- clay-like behaviour	0.00	6.55
25	10	120.75	125.0	60	11	5.0	5.6	11.2	Not liquefiable	- clay-like behaviour	0.00	6.55
35	5	120.75	125.0	60	11	7.0	7.2	12.8	Not liquefiable	- clay-like behaviour	0.00	6.55
40	5	120.75	125.0	42	0	6.0	5.9	11.5	0.24	- liquefieable - FS < 1.3	2.06	6.55
45	5	120.75	125.0	60	11	3.0	2.8	8.4	Not liquefiable	- clay-like behaviour	0.00	4.49
50	5	120.75	125.0	10	0	9.0	8.1	9.3	0.22	- liquefieable - FS < 1.3	2.34	4.49
55	5	120.75	125.0	20	0	7.0	6.2	10.7	0.24	- liquefieable - FS < 1.3	2.15	2.15
316.00	60.00	1449.00	1496.00	597.00	77.00	90.00	111.09	172.73	0.96		8.64	80.51

Profile

In-Situ Groundwater depth	35.00 feet
DESIGN Groundwater depth	8.00 feet
DESIGN Excavation depth	0.00 feet
DESIGN Surcharge (fill)	0.00 feet

Earthquake loading

M	6.6
PGA	0.654

Checks

Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

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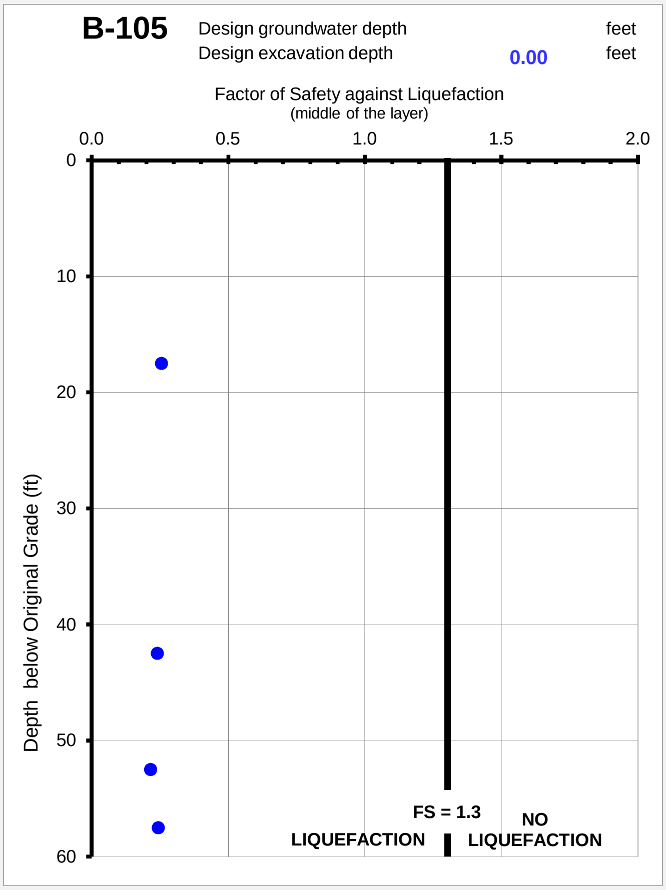
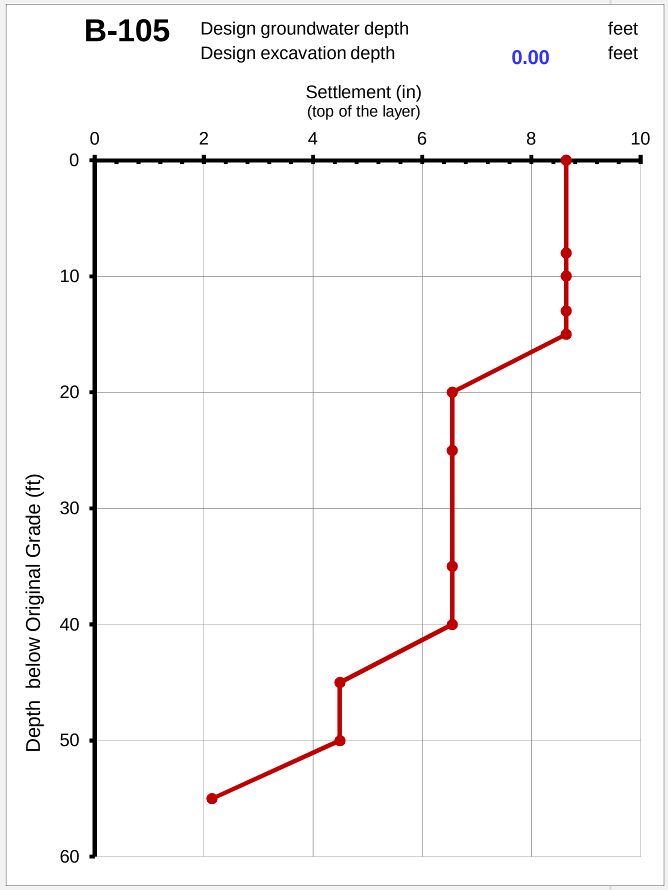

B-105
Design groundwater depth
Design excavation depth
0.00
feet
feet
Factor of Safety against Liquefaction
(middle of the layer)
0.0
0.5
1.0
1.5
2.0
0
10
20
30
40
50
60
Depth below Original Grade (ft)
FS = 1.3
LIQUEFACTION
NO
LIQUEFACTION


B-105
Design groundwater depth
Design excavation depth
0.00
feet
feet
Settlement (in)
(top of the layer)
0
2
4
6
8
10
0
10
20
30
40
50
60
Depth below Original Grade (ft)

Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	TET 16-XXXE 3000 Imperial Hwy., Lynnwood	Boring:	**B-109**	Engineer:	FC	Date:	6/12/2017

Liquefaction Evaluation Method

Correction for fines content	Idriss & Boulang. 2008, 2014
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014
Magnitude scaling factor MSF	Idriss & Boulang. 2014
Liquefaction settlement	Yoshimine et al., 2006 – no adjustment
Dry settlement	Pradel, 1998a,b

Liquefaction Analysis Statistics

Total thickness of evaluated profile	50	feet
Profile thickness susceptible to liquefaction	20	feet
Number of evaluated intervals	11	
Number of potentially liquefiable intervals	6	
Average Factor of Safety of sandy intervals	**2.4**	
Dry sand settlement	0.00	inches
Liquefaction settlement	3.88	inches
Total earthquake-induced settlement	**3.88**	**inches**

Plasticity Index sand behavior threshold 7

Calculate settlement only for layers with less than 50 % of fines

Depth to Layer Top	Layer Thickness	Total Unit Weight In-situ	Total Unit Weight Design	Fines %	Plasticity Index	Considered Blowcounts SPT-N	N1,60	N1,60,cs	Liquefaction Factor of Safety	Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf	–	–	in	in
0	8	120.75	121.0	60	11	4.0	7.7	13.3	Not liquefiable	- no groundwater	0.00	3.88
8	2	120.75	125.0	60	11	9.0	13.6	19.2	Not liquefiable	- clay-like behaviour	0.00	3.88
10	3	120.75	125.0	60	11	9.0	13.8	19.4	Not liquefiable	- clay-like behaviour	0.00	3.88
13	2	120.75	125.0	60	11	10.0	14.0	19.6	Not liquefiable	- clay-like behaviour	0.00	3.88
15	5	120.75	125.0	35	0	11.0	15.6	21.1	0.49	- liquefieable - FS < 1.3	1.32	3.88
20	5	120.75	125.0	20	0	22.0	28.7	33.1	1.95	- too dense – (N1)60,CS > 32	0.01	2.56
25	5	120.75	125.0	60	11	10.0	11.5	17.1	Not liquefiable	- clay-like behaviour	0.00	2.55
30	5	120.75	125.0	60	0	15.0	17.3	22.9	0.51	- liquefieable - FS < 1.3	0.00	2.55
35	5	120.75	125.0	49	0	17.0	18.9	24.5	0.58	- liquefieable - FS < 1.3	1.16	2.55
40	5	120.75	125.0	10	0	41.0	51.2	52.3	4.43	- too dense – (N1)60,CS > 32	0.00	1.39
45	5	120.75	125.0	20	0	15.0	15.4	19.8	0.41	- liquefieable - FS < 1.3	1.39	1.39
241.00	50.00	1328.25	1371.00	494.00	55.00	163.00	207.49	262.33	8.38		3.88	32.39

Profile

In-Situ Groundwater depth	35.00	feet
DESIGN Groundwater depth	8.00	feet
DESIGN Excavation depth	0.00	feet
DESIGN Surcharge (fill)	0.00	feet

Earthquake loading

M	6.6
PGA	0.654

Checks

Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

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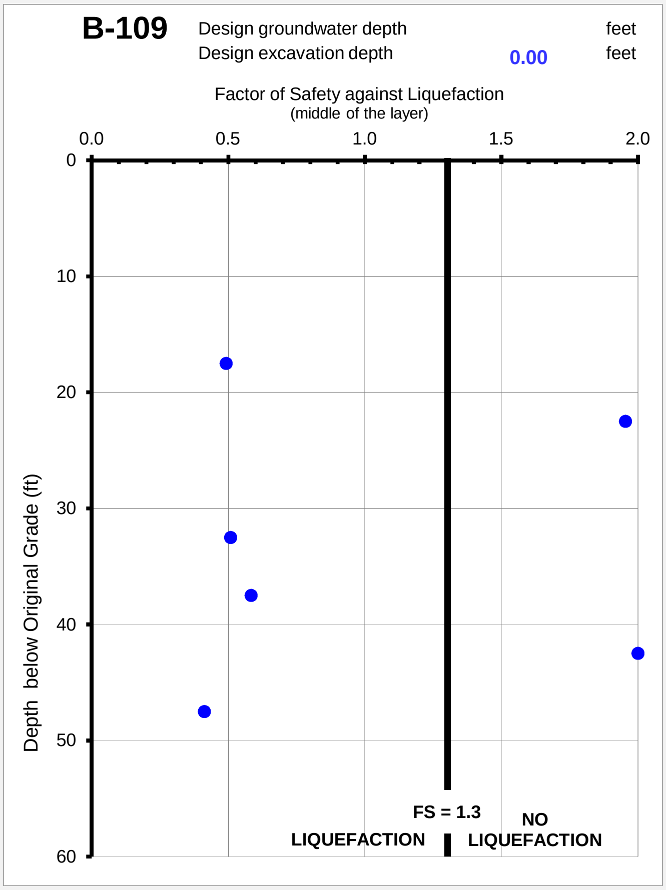
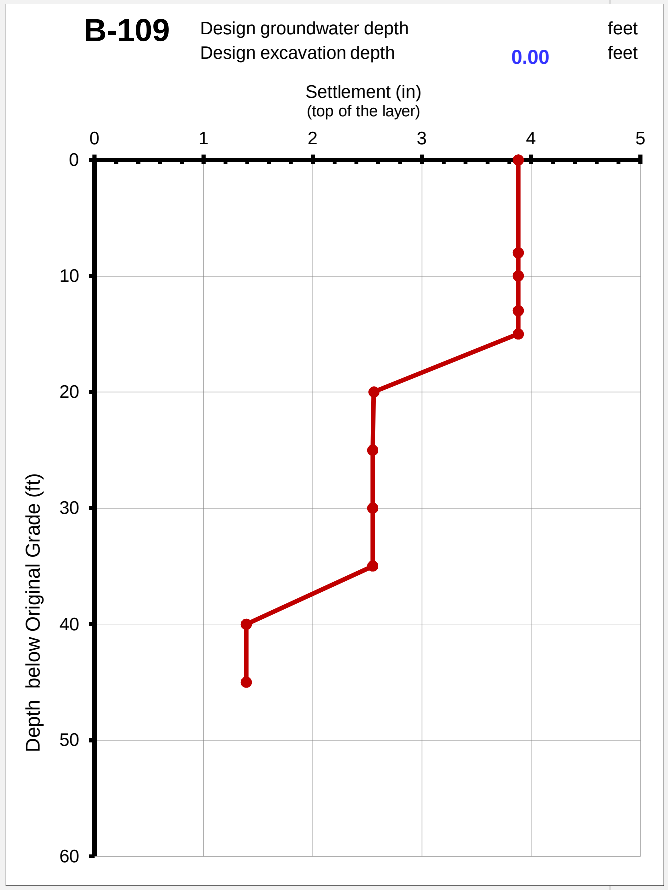

B-109
Design groundwater depth
Design excavation depth
0.00
feet
feet
Factor of Safety against Liquefaction
(middle of the layer)
0.0
0.5
1.0
1.5
2.0
Depth below Original Grade (ft)
FS = 1.3
LIQUEFACTION
NO LIQUEFACTION


B-109
Design groundwater depth
Design excavation depth
0.00
feet
feet
Settlement (in)
(top of the layer)
Depth below Original Grade (ft)

Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	TET 16-XXXE 3000 Imperial Hwy., Lynnwood	Boring:	**B-110**	Engineer:	FC	Date:	6/12/2017

Liquefaction Evaluation Method

Correction for fines content	Idriss & Boulang. 2008, 2014
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014
Magnitude scaling factor MSF	Idriss & Boulang. 2014
Liquefaction settlement	Yoshimine et al., 2006 – no adjustment
Dry settlement	Pradel, 1998a,b

Liquefaction Analysis Statistics

Total thickness of evaluated profile	65	feet
Profile thickness susceptible to liquefaction	28	feet
Number of evaluated intervals	13	
Number of potentially liquefiable intervals	8	
Average Factor of Safety of sandy intervals	**1.9**	
Dry sand settlement	0.92	inches
Liquefaction settlement	5.98	inches
Total earthquake-induced settlement	**6.90**	**inches**

Plasticity Index sand behavior threshold 7

Calculate settlement only for layers with less than 50 % of fines

Depth to Layer Top	Layer Thickness	Total Unit Weight In-situ	Total Unit Weight Design	Fines %	Plasticity Index	Considered Blowcounts SPT-N	N1,60	N1,60,cs	Liquefaction Factor of Safety	Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf	–	–	in	in
0	8	120.75	121.0	20	0	7.0	13.4	17.9	Not liquefiable	- no groundwater	0.92	6.90
8	2	120.75	125.0	20	0	21.0	29.4	33.9	3.15	- too dense – (N1)60,CS > 32	0.00	5.98
10	3	120.75	125.0	20	0	10.0	15.3	19.8	0.52	- liquefieable - FS < 1.3	0.84	5.98
13	2	120.75	125.0	60	11	9.0	12.6	18.2	Not liquefiable	- clay-like behaviour	0.00	5.14
15	5	120.75	125.0	20	0	17.0	24.1	28.6	1.03	- liquefieable - FS < 1.3	0.42	5.14
20	5	120.75	125.0	20	0	17.0	21.9	26.3	0.74	- liquefieable - FS < 1.3	0.87	4.73
25	5	120.75	125.0	20	0	16.0	18.8	23.3	0.53	- liquefieable - FS < 1.3	1.21	3.86
30	5	120.75	125.0	60	11	19.0	22.5	28.1	Not liquefiable	- clay-like behaviour	0.00	2.65
35	5	120.75	125.0	60	11	15.0	16.5	22.1	Not liquefiable	- clay-like behaviour	0.00	2.65
40	5	120.75	125.0	20	0	16.0	17.0	21.5	0.46	- liquefieable - FS < 1.3	1.30	2.65
45	10	120.75	125.0	60	11	20.0	21.3	26.9	Not liquefiable	- clay-like behaviour	0.00	1.35
55	5	120.75	125.0	10	0	21.0	20.8	22.0	0.49	- liquefieable - FS < 1.3	1.28	1.35
60	5	120.75	125.0	10	0	30.0	31.6	32.7	1.71	- too dense – (N1)60,CS > 32	0.07	0.07
356.00	65.00	1569.75	1621.00	400.00	44.00	218.00	265.16	321.22	8.62		6.90	48.46

Profile

In-Situ Groundwater depth	35.00 feet
DESIGN Groundwater depth	8.00 feet
DESIGN Excavation depth	0.00 feet
DESIGN Surcharge (fill)	0.00 feet

Earthquake loading

M	6.6
PGA	0.654

Checks

Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

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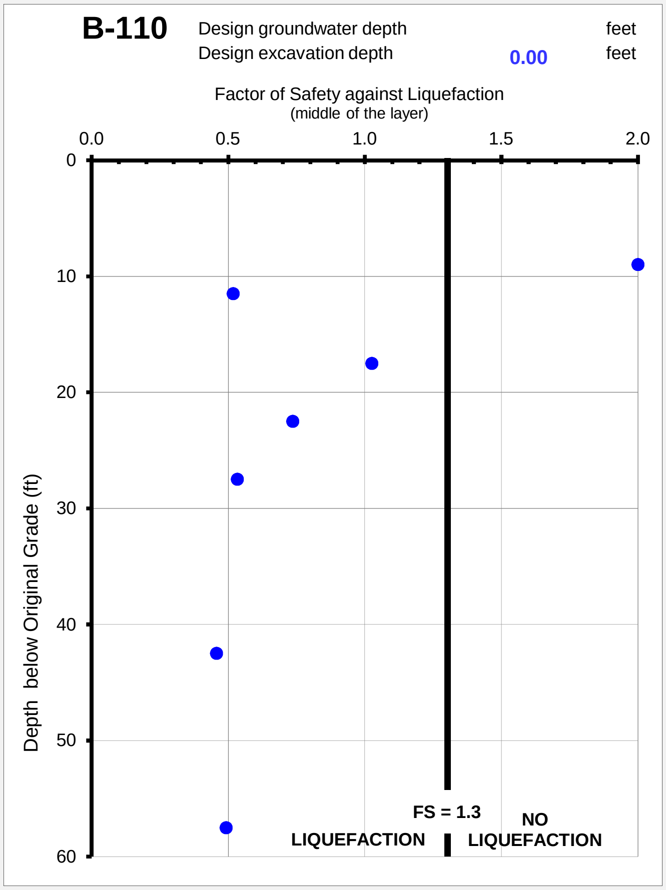
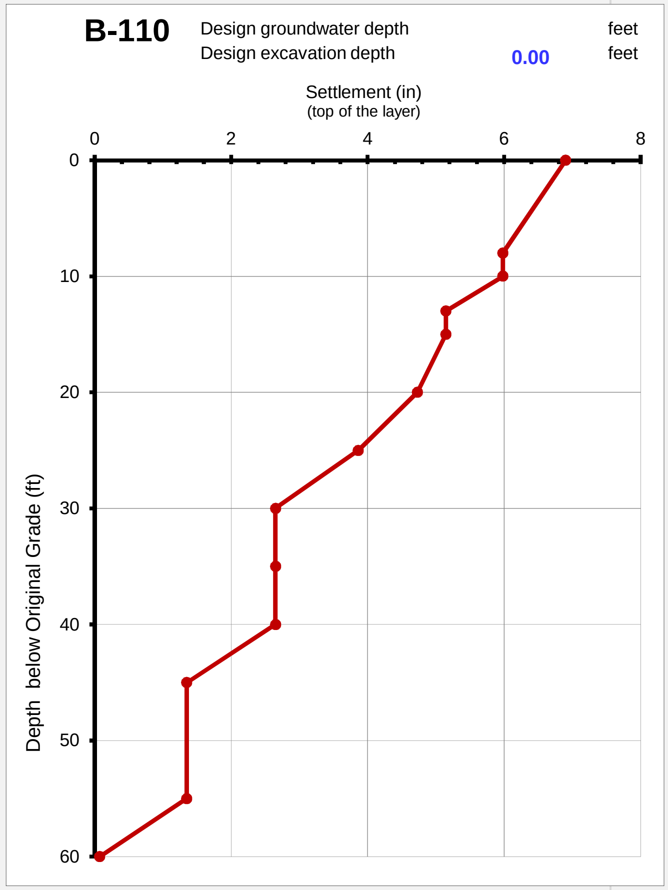

B-110
Design groundwater depth
Design excavation depth
0.00
feet
feet
Factor of Safety against Liquefaction
(middle of the layer)
0.0
0.5
1.0
1.5
2.0
0
10
20
30
40
50
60
Depth below Original Grade (ft)
FS = 1.3
LIQUEFACTION
NO
LIQUEFACTION


B-110
Design groundwater depth
Design excavation depth
0.00
feet
feet
Settlement (in)
(top of the layer)
0
2
4
6
8
0
10
20
30
40
50
60
Depth below Original Grade (ft)

Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	TET 16-XXXE 3000 Imperial Hwy., Lynnwood	Boring:	**B-111**	Engineer:	FC	Date:	6/12/2017

Liquefaction Evaluation Method

Correction for fines content	Idriss & Boulang. 2008, 2014
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014
Magnitude scaling factor MSF	Idriss & Boulang. 2014
Liquefaction settlement	Yoshimine et al., 2006 – no adjustment
Dry settlement	Pradel, 1998a,b

Liquefaction Analysis Statistics

Total thickness of evaluated profile	65 feet
Profile thickness susceptible to liquefaction	18 feet
Number of evaluated intervals	13
Number of potentially liquefiable intervals	5
Average Factor of Safety of sandy intervals	**2.5**
Dry sand settlement	0.32 inches
Liquefaction settlement	2.76 inches
Total earthquake-induced settlement	**3.08 inches**

Plasticity Index sand behavior threshold 7

Calculate settlement only for layers with less than 50 % of fines

Depth to Layer Top	Layer Thickness	Total Unit Weight In-situ	Total Unit Weight Design	Fines %	Plasticity Index	Considered Blowcounts SPT-N	N1,60	N1,60,cs	Liquefaction Factor of Safety	Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf	–	–	in	in
0	8	120.75	121.0	25	0	10.0	19.1	24.2	Not liquefiable	- no groundwater	0.32	3.08
8	2	120.75	125.0	60	11	7.0	10.7	16.3	Not liquefiable	- clay-like behaviour	0.00	2.76
10	3	120.75	125.0	20	0	13.0	19.6	24.1	0.73	- liquefieable - FS < 1.3	0.63	2.76
13	2	120.75	125.0	60	11	10.0	14.0	19.6	Not liquefiable	- clay-like behaviour	0.00	2.13
15	5	120.75	125.0	60	11	20.0	28.4	34.0	Not liquefiable	- clay-like behaviour	0.00	2.13
20	5	120.75	125.0	20	0	14.0	17.9	22.3	0.51	- liquefieable - FS < 1.3	1.26	2.13
25	5	120.75	125.0	60	11	24.0	29.3	35.0	Not liquefiable	- clay-like behaviour	0.00	0.87
30	5	120.75	125.0	60	11	9.0	10.0	15.6	Not liquefiable	- clay-like behaviour	0.00	0.87
35	5	120.75	125.0	60	11	19.0	21.5	27.1	Not liquefiable	- clay-like behaviour	0.00	0.87
40	5	120.75	125.0	20	0	22.0	24.5	29.0	0.95	- liquefieable - FS < 1.3	0.48	0.87
45	5	120.75	125.0	20	0	23.0	25.1	29.6	1.04	- liquefieable - FS < 1.3	0.39	0.39
50	10	120.75	125.0	60	11	18.0	18.3	23.9	Not liquefiable	- clay-like behaviour	0.00	0.00
60	5	120.75	125.0	20	0	43.0	52.2	56.7	4.40	- too dense – (N1)60,CS > 32	0.00	0.00
351.00	65.00	1569.75	1621.00	545.00	77.00	232.00	290.65	357.32	7.62		3.08	18.89

Profile

In-Situ Groundwater depth	35.00 feet
DESIGN Groundwater depth	8.00 feet
DESIGN Excavation depth	0.00 feet
DESIGN Surcharge (fill)	0.00 feet

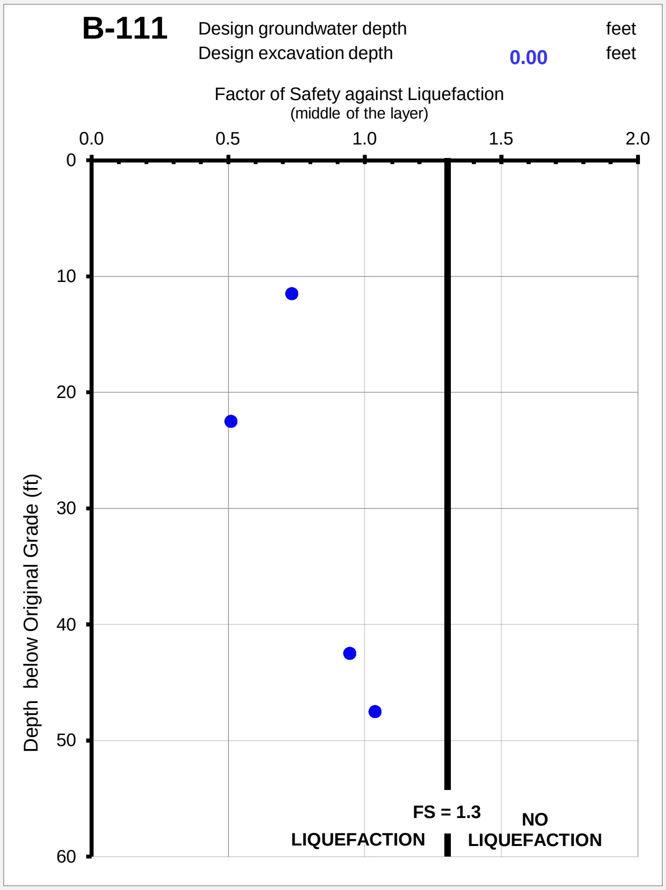
Earthquake loading

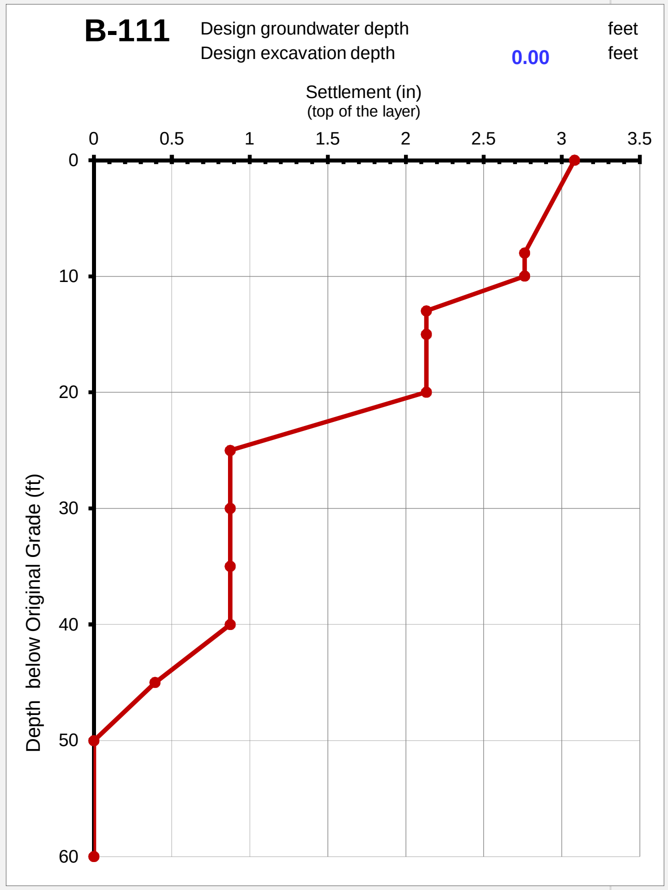
M	6.6
PGA	0.654

Checks

Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

Version v1b 2017-01


B-111
Design groundwater depth
Design excavation depth
0.00
feet
feet
Factor of Safety against Liquefaction
(middle of the layer)
0.0
0.5
1.0
1.5
2.0
0
10
20
30
40
50
60
Depth below Original Grade (ft)
FS = 1.3
LIQUEFACTION
NO LIQUEFACTION


B-111
Design groundwater depth
Design excavation depth
0.00
feet
feet
Settlement (in)
(top of the layer)
0
0.5
1
1.5
2
2.5
3
3.5
0
10
20
30
40
50
60
Depth below Original Grade (ft)

Sensitivity Analyses of Fine-Grained Materials

Sample No.	Sample Depth (ft)	Groundwater Depth (ft)	Assumed Total Unit Weight above GWT (pcf)	assumed Total Unit Weight below GWT (pcf)	USCS Classification	Sample Moisture Content (%)	Dry Unit Weight at this depth (pcf)	Liquid Limit	Plastic Limit	Assumed Specific Gravity	Plasticity Index	Total Unit Weight (pcf)	Saturated Moisture Content (%)	Liquidity Index	Approximate Effective Vertical Stress (atm)	Sensitivity from Peck, Mesri (1996)	Sensitivity from Mitchell and Soga (2005)
B-1 SPT-9	25	8	120	125	ML	32	90	37	29	2.65	8	118.8	31.6	0.32	0.96	2.67	2.08
B-2 SPT-7	20	8	120	125	CL	32	93	34	21	2.65	13	122.8	29.4	0.64	0.81	4.47	4.33
B-104 SPT-5	16	8	120	125	CL/ML	20	101	33	23	2.65	10	121.2	24.0	0.10	0.69	1.87	0.46
B-107 R-7	20	8	120	125	ML	21	93	36	25	2.65	11	112.5	29.4	0.40	0.81	3.00	2.40
B-110 SPT-10	35	8	120	125	CL/ML	20	100	31	23	2.65	8	120.0	24.7	0.21	1.25	2.21	1.58