Temp Control Mechanical
Building Addition and Temporary Trailers – Channel Avenue
Portland, Oregon

Geotech Solutions Inc.

February 15, 2016

GSI Project: TempControl-03-01-consult2

February 15, 2016 *tempcontrol-3-1-consult2*

Temp Control Mechanical
c/o CIDA
taral@cidainc.com

REPORT OF GEOTECHNICAL ENGINEERING SERVICES
Temp Control Mechanical Facility Building Addition and Temporary Trailers
Channel Avenue, Swan Island - Portland, Oregon

As authorized, we appreciate the opportunity to present this Report of Geotechnical Engineering Services for the proposed new Temp Control Mechanical facility addition and temporary trailers on Channel Avenue on Swan Island in Portland, Oregon. The proposed development will include an approximately 2,500 square foot building addition for office space, as well as interior remodeling. Building loads are expected to be less than 100 kips for columns and 5 kips per foot for walls and 150 psf for floors with less than one foot of fill in the addition area. The purpose of our services was to evaluate geotechnical conditions at the site from our previous explorations and provide geotechnical recommendations for design and construction of the new addition. Specifically, our scope of work for this phase included the following:

- Review geotechnical explorations in the structure areas in our files, including borings and CPT's.
- Review plans provided by CIDA.
- Conduct a site visit to observe present surface conditions.
- Complete liquefaction analyses using previous boring and CPT logs in our database for the site, with estimates of ground deformation and qualitative means to reduce damage, if needed.
- Complete an updated geotechnical report.

Our previous scope of work for the 2003 building design included the following:

- Provide principal level project management including attending one meeting to review preliminary plans and overall project scope, and management of field and subcontracted services, report writing, analyses, and invoicing.
- Review geologic maps and vicinity geotechnical information as indicators of subsurface conditions.
- Explore subsurface conditions by advancing one cone penetrometer test (CPT) probe and three mud-rotary borings.
    - Borings were completed to depths of up to 60 feet in the proposed building areas, and up to 20 feet in the pavement and utility areas to obtain physical samples at depth for laboratory testing.
    - The cone penetrometer probe was completed to a depth of 100 feet.
- Classify the materials encountered in the explorations, and maintain detailed logs. Complete SPT testing and sampling at 2.5 to 5 foot intervals in the borings and complete shear wave velocity and pore pressure dissipation testing in the CPT probe.
- Complete laboratory testing on select samples as necessary, including moisture and fines content as well as consolidation testing of a compressible sample.
- Complete detailed liquefaction analyses of site soils and estimate liquefaction induced deformations, and provide qualitative means to reduce deformations as needed.

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- Evaluate settlement from dock-high fills and aereal loading such as floor slabs and fills and provide estimates of settlement and recommendations for preloading and surcharging if necessary.
- Provide recommendations for earthwork including seasonal material usage, compaction criteria, utility trench backfill, need for subsurface drainage, and reuse of ground pavement materials.
- Provide recommendations for footing foundations, including embedment, bearing pressure, resistance to lateral loads, a seismic coefficient and the need for subsurface drainage.
- Provide recommendations for floor slab support.
- Provide recommended thicknesses for asphalt concrete pavements.
- Provide a written report summarizing the results of our geotechnical evaluation.
- Provide specifications in 3-part CSI format for site preparation, earthwork, and asphalt concrete pavements.

SITE OBSERVATIONS AND CONDITIONS

Surface Conditions

The building addition is located on the southeast part of the existing building in a flat landscaped area, which is roughly 380 feet northwest of the intersection with N. Ballast Street. The existing and proposed structures noted as "possioble future office" and the approximate locations of our previous explorations are shown on the attached ***Site Plan***.

Subsurface Conditions

General* -** The site was explored on February 20, 2003, by advancing three mud rotary borings (B-1 through B-3) to depths of 20, 40, and 60 feet and advancing one CPT probe (P-1) to a depth of 100 feet at the approximate locations shown on the attached ***Site Plan. Soils encountered consisted of dredge sand fill, and soft alluvial silt underlain by sand and gravel to the depth explored.

***Dredge Sand Fill* -** The dredge sand fill generally consisted of poorly graded sand with trace fine gravel and occasional organics. The fill was medium dense with standard penetration test (SPT) blow counts (N_{60}) of 11 to 23.

***Silt* -** Silt was encountered at depths of 18 and 19 feet in B-1 and B-2, respectively, and ranged from 12 to 14 feet thick. Samples obtained during drilling generally contained trace to some organics (wood) and were soft to medium stiff with SPT blow counts (N_{60}) of 3 to 5.

***Sand* -** The silt unit was underlain by sand to the depths explored. The underlying sand layer contained thin lenses of silt and occasional fine gravel and organics. The consistency of the sand was medium dense to very dense, with SPT blow counts (N_{60}) of 18 to 59.

***Groundwater* -** The depth to groundwater during our explorations was approximately 28 feet (based on pore pressure dissipation testing). Groundwater was not directly measured in our borings due to the mud-rotary methods used. However, our samples generally became wet below a depth of 20 to 25 feet.

CONCLUSIONS AND RECOMMENDATIONS

General

Based on our observations and analyses, the proposed building addition to current building codes would require ground improvement or possibly special pile supported mat foundations due to large liquefaction induced lateral spreading and settlement.

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Earthwork

Preparation - Prior to completing earthwork, the site should be prepared by removing existing structures and utilities. The AC surfacing should be stripped from building and pavement areas and a 5-foot perimeter around those areas. Roots greater than one inch in diameter should be removed where trees are currently present. Resulting excavations should be brought back to grade with structural fill.

Stabilization and Soft Areas - After stripping, we should be contacted to evaluate the exposed subgrade for soft silty areas. This evaluation can be done by proof rolling in dry conditions or probing during wet conditions. Soft silty areas will require overexcavation and backfilling with well graded, angular crushed rock compacted as structural fill. A separation geosynthetic may also be required such as a Propex Geotex 601 or equivalent.

Working Blankets and Haul Roads – Much of the site for access to the addition is currently paved and will not require the use of haul roads or working blankets during construction if the pavement section is left intact. Construction equipment operating directly on the sand subgrade may result in localized disturbance and require confinement or replacement with clean crushed rock.

Select Fill – The on-site soils consist predominantly of sand and gravel with trace to no silt. These materials are suitable for use as structural fill provided the material is properly moisture conditioned. Samples containing silt may require drying prior to compaction. Once moisture contents are within 3 percent of optimum the material should be compacted to 95 percent relative to ASTM D-1557 (modified proctor) using a vibratory compactor, such as a smooth drum-roller, hoe pack, or jumping jack. Fill should be placed in lifts no greater than 12 inches in loose thickness. In addition to meeting density specifications, fill will also need to pass a proof roll using a loaded dump truck, water truck, or similar size equipment.

Granular Fill - Imported granular fill, such as clean sand or rock, should have a maximum particle size of 6-inches, be well graded, and have less than 5 percent passing the #200 sieve. This material should be compacted to 95 percent relative to ASTM D 1557.

Recycled Asphalt and Base Rock - Provided the recycled asphalt and base rock meet the requirements for granular fill, the on-site material can be used as structural fill. This material should also be compacted to 95 percent relative to ASTM D 1557.

Trenches – Slight to moderate caving of the medium dense sand and gravel should be anticipated during excavation. Shoring of utility trenches will be required for depths greater than 4 feet. Pipe bedding should be installed in accordance with the pipe manufacturers' recommendations. Trench backfill above the pipe zone should consist of well graded, angular crushed rock or on-site dredge sand or gravel fill with no more than 7 percent passing a #200 sieve. Trench backfill should be compacted to 92 percent relative to ASTM D-1557, and paving should not occur within one week of backfilling.

Liquefaction – Liquefaction and cyclic failure can occur in non-plastic saturated soils subjected to strong earthquake motions, particularly in sand and sandy soils. Design level earthquakes for the site were evaluated from crustal sources in the Portland area, as well as more distant but larger and longer duration sources of the Cascadia Subduction Zone (CSZ) interface compatible with the State of Oregon Structural Specialty Code.

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Design level earthquakes for liquefaction at this site (2% chance of exceedence in 50 years) are controlled by CSZ interface earthquakes, of estimated magnitudes (Mw) of 8.3 to 9. Liquefaction and lateral spreading of some of the sand and sandy layers at depths below ground water levels (roughly below 25 feet) will likely result in settlement and ground movement toward the Willamette River or Swan Island Basin in the current design level earthquakes. Features such as spreading grabens/ground cracks, vertical deformation, and liquefaction vents are likely. Total lateral spreading was calculated for the CPT and boring profiles using the program *Cliq* and methods developed by Robertson and NCEER to be within a range of 2-3 feet (attached). Differential lateral spreading from one side of the new building to the other is expected to be at least one foot. In addition, we analyzed vertical settlement to be 6 to 8 inches.

Foundations

Building Addition - For the building addition, foundation choices are a function of movement and building deformation tolerance as determined by the structural engineer. As 2-3 feet of lateral movement and 6-8 inches of vertical settlement is expected, it is unlikely that conventional shallow foundations will be suitable. The existing building will also undergoe these deformations associated with present code level design earthquakes. At the time of design of the existing building in 2003, code criteria were for a 10% chance of being exceeded in 50 years, and less than an inch of deformation was/is calculated for those ground motions.

With current code level ground motions for new permanent occupied structures , the ground deformations likely require deep ground improvement. For the size of the planned addition, this may be cost prohibitive. Pile foundations and rigid surface mat foundations may not be feasible to resist the deep seated movements and high moments on embedded pile elements, unless the structural engineer can develop a system to tolerate large remaining movements and total building loss is allowed/accepted.

Ground improvement could include a grid of stone columns constructed in a footprint extending at least 30 feet past the building edges and to depths of 60 feet, with typical area replacement ratios of 10-15%. Such improvement would likely require the structure to be moved away from the existing building as undue settlement and damage would occur to the existing building unless it was underpinned with deep foundations. Alternatively, the typically more expensive process of deep soil mixing could be used. In either case, these procedures are typically valued in a design-build manner by a specialty contractor. If ground improvement is chosen, we should be contacted for additional ground improvement design criteria.

Pile supported mat systems should first be evaluated in concept by the structural engineer. Piles could likely reduce vertical differential settlement to several inches across the building when coupled with a reinforced mat, but total lateral movement of a foot or more is still expected due to the very high bending moments in pile elements at the deep liquefied interface. This would require a heavily reinforced mat to better resist tensional forces. If this approach is deemed viable by the structural engineer, we should be contacted for additional pile and mat design criteria.

Temporary Structures - For temporary structures not subject to seismic design for liquefaction, a bearing pressure of 4,000 psf, with an increase to 8,000 psf for temporary loads, can be used for design. Resistance to conventional lateral wind and earthquake loads can be obtained by a passive equivalent fluid pressure of 350 pcf against the edge of the footings (ignoring the top one foot unless covered by

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pavement or a slab) and by a friction coefficient of 0.40 on such elements. The seismic site class is F, but in accordance with code for short period buildings, site class E can be used for design.

Settlement From Fill and Slab Loads - Based on our discussions with the architect, we understand that floor slab loads will be less than 150 psf with a slab elevation no more than one-foot above existing average addition area grades. For this scenario, post slab construction settlement is not expected to exceed one inch. Settlement of up to ½ inch is expected on the adjacent structure. If finished floor elevations are to be one-foot higher than this, post slab construction settlement of one to two inches is expected. Fills of 2-feet or more may result in intolerable slab distortion and damage to the existing building, and alternatives such as light weight fill must be used under our consultation.

Surveying / Monitoring - Survey monitoring of slab fill settlement is required if the finished floor elevation is greater than one-foot above existing grades. A surveyor should complete daily monitoring of hubs or steel rods placed at the center, two sides, and two corners of the pad. Elevations should be read to the nearest 0.01 feet, with data provided to us for evaluation of suitable slab and foundation construction timing. Surveying reference bench marks should be at least 100 feet from the edge of the pad fill or other new fills and loads.

Floor Slabs - Floor slabs should be designed using a modulus of subgrade reaction equal to 150 pci. A minimum of six inches of clean, angular crushed rock with no more than 6 percent passing a #200 sieve is recommended for underslab rock. Prior to slab placement the subgrade will need to be evaluated by us by probing, or will need to pass a proof roll with a fully loaded truck and meet 92 percent compaction relative to ASTM D-1557. In addition, any areas contaminated with fines must be removed and replaced with clean rock. If the base rock is saturated or trapping water, this water must be removed prior to slab placement.

Retaining/Stem Walls

General - The following recommendations are based on the assumptions that (1) Wall backfill consists of level, well-drained, angular, granular material and (2) Walls are less than 5 feet in height.

Walls restrained against rotation should be designed using an equivalent fluid pressure of 55 pcf. Walls not restrained against rotation should be design using an equivalent fluid pressure of 32 pcf. These forces can be resisted by passive pressure at the toe of the wall using an equivalent fluid pressure of 350 pcf (this should exclude the top 12 inches of embedment) and friction along the base using a friction coefficient of 0.40.

Backfill - Retaining walls should be backfilled with clean, well-graded granular soil with less than 6 percent fines, such as clean sand or rock. This material should also be compacted to a minimum of 92 percent relative to ASTM D-1557 (modified proctor). Within 3 feet of the wall, backfill should be compacted to not more than 90 percent relative to ASTM D-1557 using hand-operated equipment.

Pavement

We have developed pavement thickness at the site for 5, 10, 25, and 50 trucks per day (with a truck factor 0.6) and a 20-year design life. These volumes can be revised if specific traffic data is available. Our analyses are based on AASHTO methods and subgrade of structural fill or undisturbed medium dense sand or better, having a resilient modulus of 7,500 psi. We have also assumed that pavement

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construction will be completed during an extended period of dry weather. The results of our analyses based on these parameters are provided in the table below.

Trucks / Day	ESAL's	Asphalt Concrete inches	Crushed Rock inches
5	32542	3	6
10	65084	3	8
25	162711	3	10
50	325422	4	9

The thicknesses listed in the table above are intended to the minimum acceptable. Crushed rock should conform to ODOT base rock standards and have less than 6 percent passing the #200 sieve. Asphalt concrete should be compacted in one lift to 91 percent of a Rice Density, or to 98 percent of the maximum density from a test strip.

LIMITATIONS AND OBSERVATION DURING CONSTRUCTION

We have prepared this report for use by Temp Control Mechanical and CIDA, and their design and construction teams for this project only. The information herein could be used for bidding or estimating purposes but should not be construed as a warranty of subsurface conditions. We have made observations only at the aforementioned locations and only to the stated depths. These observations do not reflect soil types, strata thicknesses, water levels or seepage that may exist between observations. We should be consulted to observe all foundation bearing surfaces, proof rolling of slab and pavement subgrades, installation of structural fill, and any cut slopes. We should be consulted to review final design and specifications in order to see that our recommendations are suitably followed. If any changes are made to the anticipated locations, loads, configurations, or construction timing, our recommendations may not be applicable, and we should be consulted. The preceding recommendations should be considered preliminary, as actual soil conditions may vary. In order for our recommendations to be final, we must be retained to observe actual subsurface conditions encountered. Our observations will allow us to interpret actual conditions and adapt our recommendations if needed.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared. No warranty, expressed or implied, is given.

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We appreciate the opportunity to work with you on this project and look forward to our continued involvement. If you have any questions, please do not hesitate to call.

Sincerely,

Don Rondema, MS, PE, GE
Principal

Attachments

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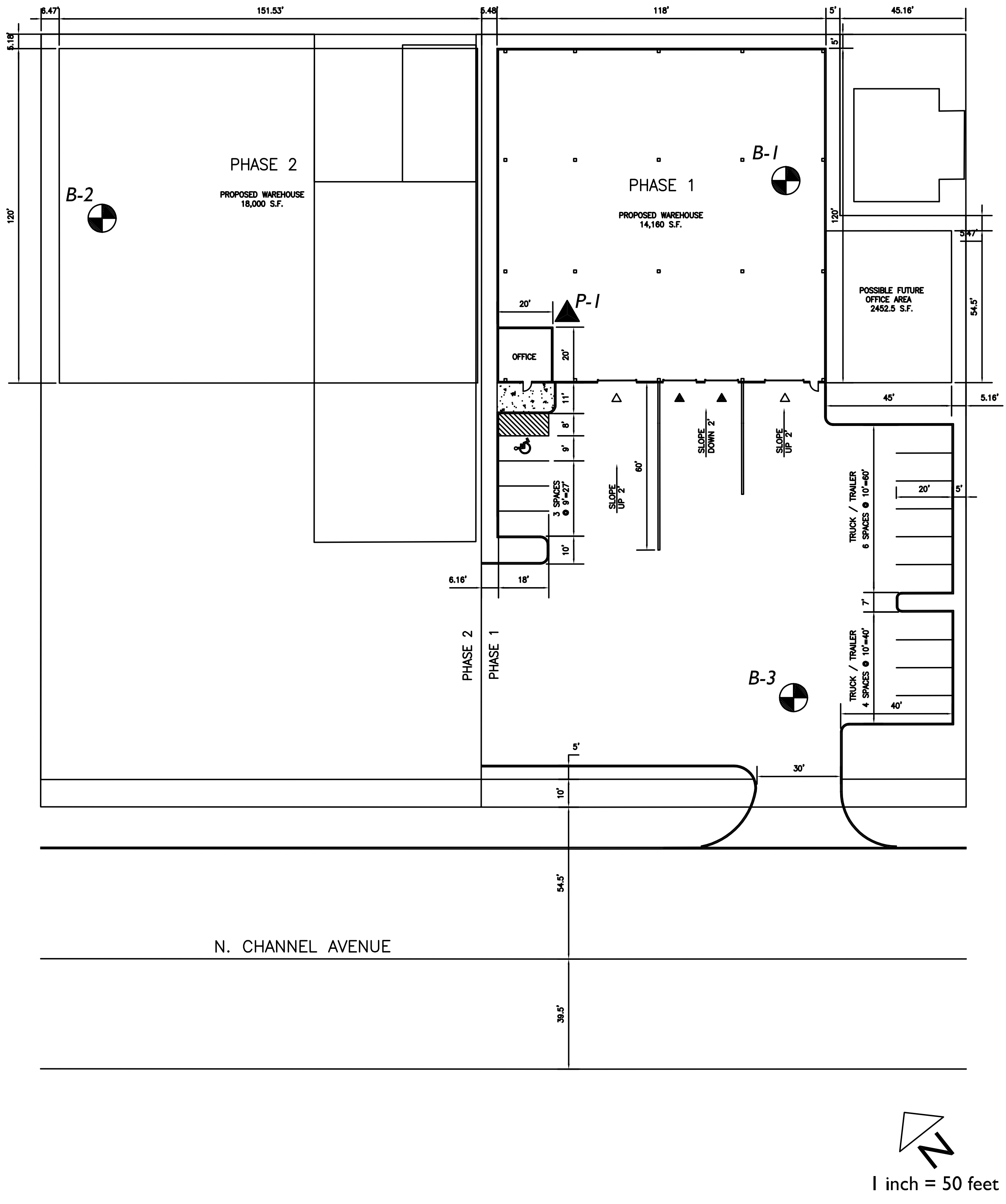

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SITE PLAN

TempControl-02-01-gi

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil	
Relative Density	**Standard Penetration Resistance (N-values) blows per foot**
very loose	0 - 4
loose	4 - 10
medium dense	10 - 30
dense	30 - 50
very dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils		
Consistency	**Standard Penetration Resistance (N-values) blows per foot**	**Torvane Undrained Shear Strength, tsf**
very soft	0 - 2	less than 0.125
soft	2 - 4	0.125 - 0.25
medium stiff	4 - 8	0.25 - 0.50
stiff	8 - 15	0.50 - 1.0
very stiff	15 - 30	1.0 - 2.0
hard	over 30	over 2.0

Grain-Size Classification	
Description	**Size**
Boulders	12 - 36 in.
Cobbles	3 - 12 in.
Gravel	¼ - ¾ in. (fine)
	¾ - 3 in. (coarse)
Sand	No. 200 - No. 40 Sieve (fine)
	No. 40 - No. 10 sieve (medium)
	No. 10 - No. 4 sieve (coarse)
Silt/Clay	Pass No. 200 sieve

Modifier for Subclassification	
Adjective	**Percentage of Other Material In Total Sample**
Clean/Occasional	0 - 2
Trace	2 - 10
Some	10 - 30
Sandy, Silty, Clayey, etc.	30 - 50

w = moisture content
N = SPT blowcount
* = No recovery

Depth (ft)	Soil and Rock Description	Samples and Data (N)	
0 ft	AC - 5 inches CRB - 12 inches		
	Medium dense, gray-brown, SAND FILL (dredge fill); moist.	15	w = 15%
		13	w = 16%
		13	w = 20%
10	with trace fine, sub-rounded gravel at 10 feet.	23	w = 17%
		17	w = 22%
20	Soft, gray SILT with some organics (wood); moist to wet.	3	w = 50%
		P	CONSOL
		3	w = 42%
30	with some sand at 30 feet.	3	w = 42%
	Dense, well-graded, gray-brown, SAND; moist.	15	w = 15%
	2-foot thick silt lens at 37.5 feet.		
40			

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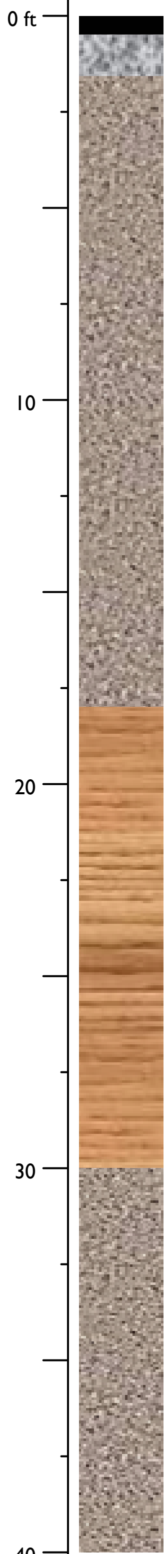
BORING B-1

TempControl-03-01-gi

w = moisture content
N = SPT blowcount
* = No recovery

Depth	Soil and Rock Description	Samples and Data
40 ft	Medium dense, gray, fine SAND; wet.	11 w = 31%
50		
60		
70		
80		

w = moisture content
N = SPT blowcount
* = No recovery



Soil and Rock Description

AC - 5 inches
CRB - 8 inches (6-inch minus "pit run")

Medium dense, gray, medium SAND FILL (dredge fill); moist

with trace fine, sub-rounded gravel.

no gravel at 15 feet.

Soft to medium stiff, gray SILT with trace sand; moist.

Loose, gray SAND with trace to some silt; wet.

with some fine gravel at 33.5 feet.

becomes dense at 35 feet.

Samples and Data

N	w
21	w = 12%
11	w = 14%
22	w = 9%
18	w = 15%
16	w = 20%
4	w = 36%
5	w = 36%
P	
5	w = 35%
48	w = 13%

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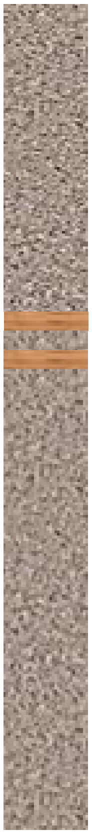
BORING B-2
TempControl-03-01-gi

w = moisture content
N = SPT blowcount
* = No recovery

Depth		Soil and Rock Description	Samples and Data	
40 ft		Medium dense, gray, fine SAND; wet.	18	w = 31%
			24	w = 26%
		6-inch silt lenses at 48 and 49 feet.		
50			20	w = 32%
		with trace fine gravel at 53.5 feet.		
		with some organics (wood) at 55 feet.	27	w = 29%
60		becomes very dense at 60 feet.	59*	
70				
80				

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BORING B-2 cont.

TempControl-03-01-gi

w = moisture content
N = SPT blowcount
* = No recovery

Soil and Rock Description | **Samples and Data**

0 ft

AC - 5 inches
CRB - 13 inches

Medium dense, brown, sandy, fine, sub-rounded GRAVEL FILL; moist.

N	w
18	w = 11%

becomes very loose at 8 feet. Lose drilling mud.

10

Medium dense, brown, SAND FILL (dredge fill); with trace, fine, sub-rounded gravel; moist.

N	w
19	w = 17%

no gravel at 15 feet.

N	w
23	w = 24%

organics (wood) in cuttings at 19 to 20 feet and in sample at 20-feet.

20

N	w
8	w = 27%

30

40

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BORING B-3
TempControl-03-01-gi

T A B L E O F C O N T E N T S

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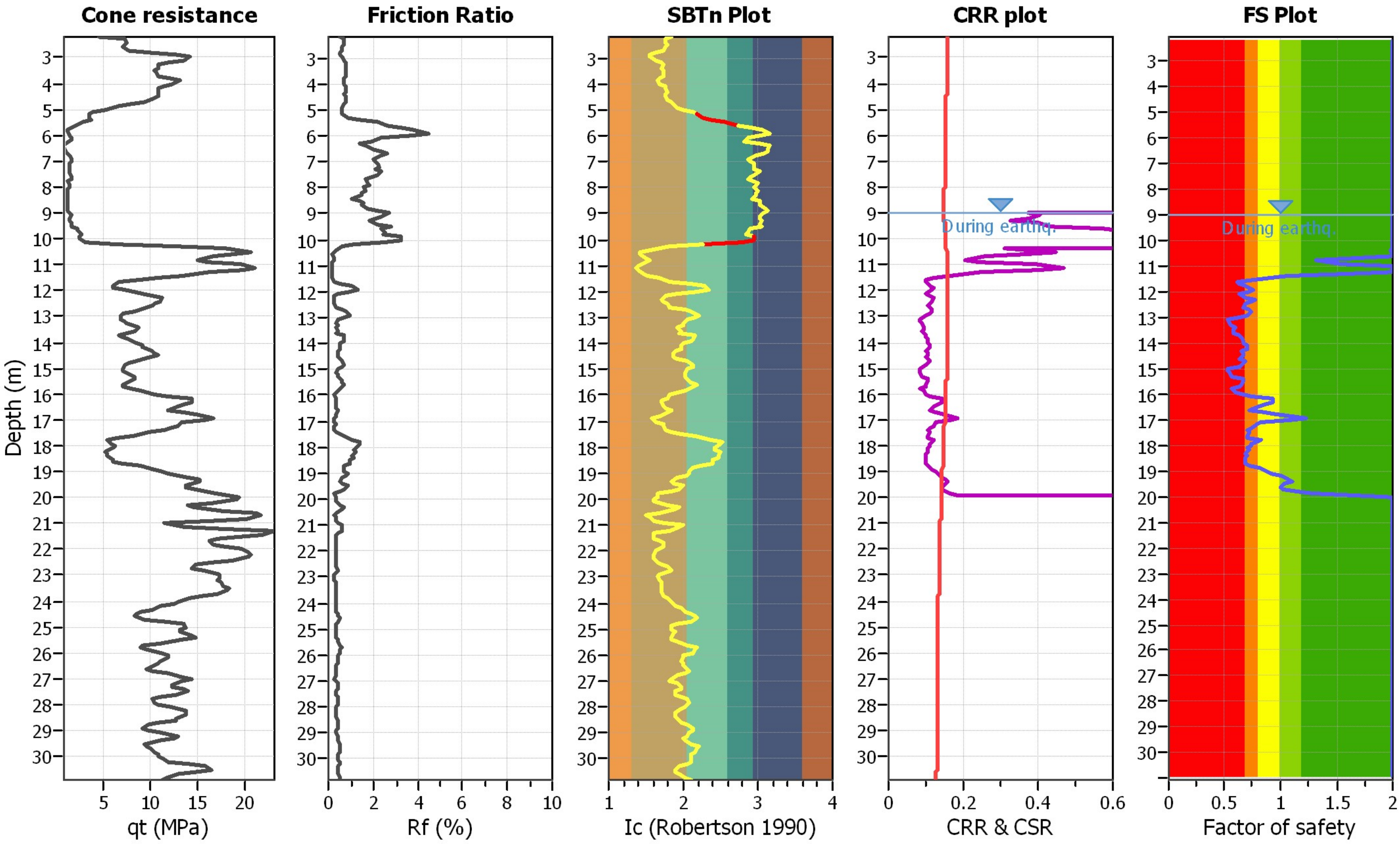
LIQUEFACTION ANALYSIS REPORT

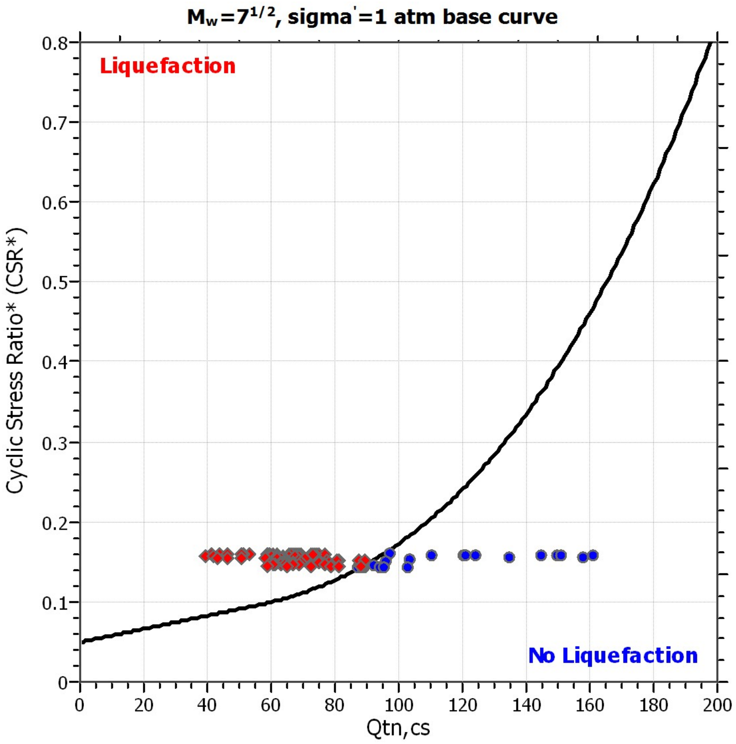
Project title : **Location :**

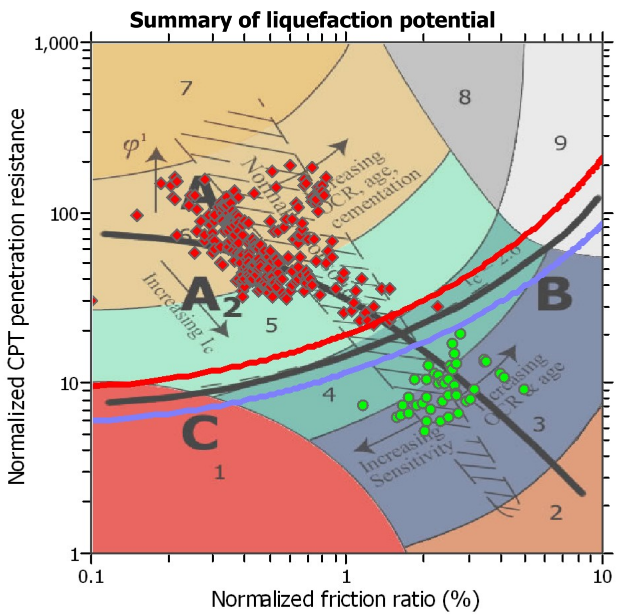
CPT file : cpt1

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	9.00 m	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	9.00 m	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	20.00 m
Earthquake magnitude M_w:	8.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	No		

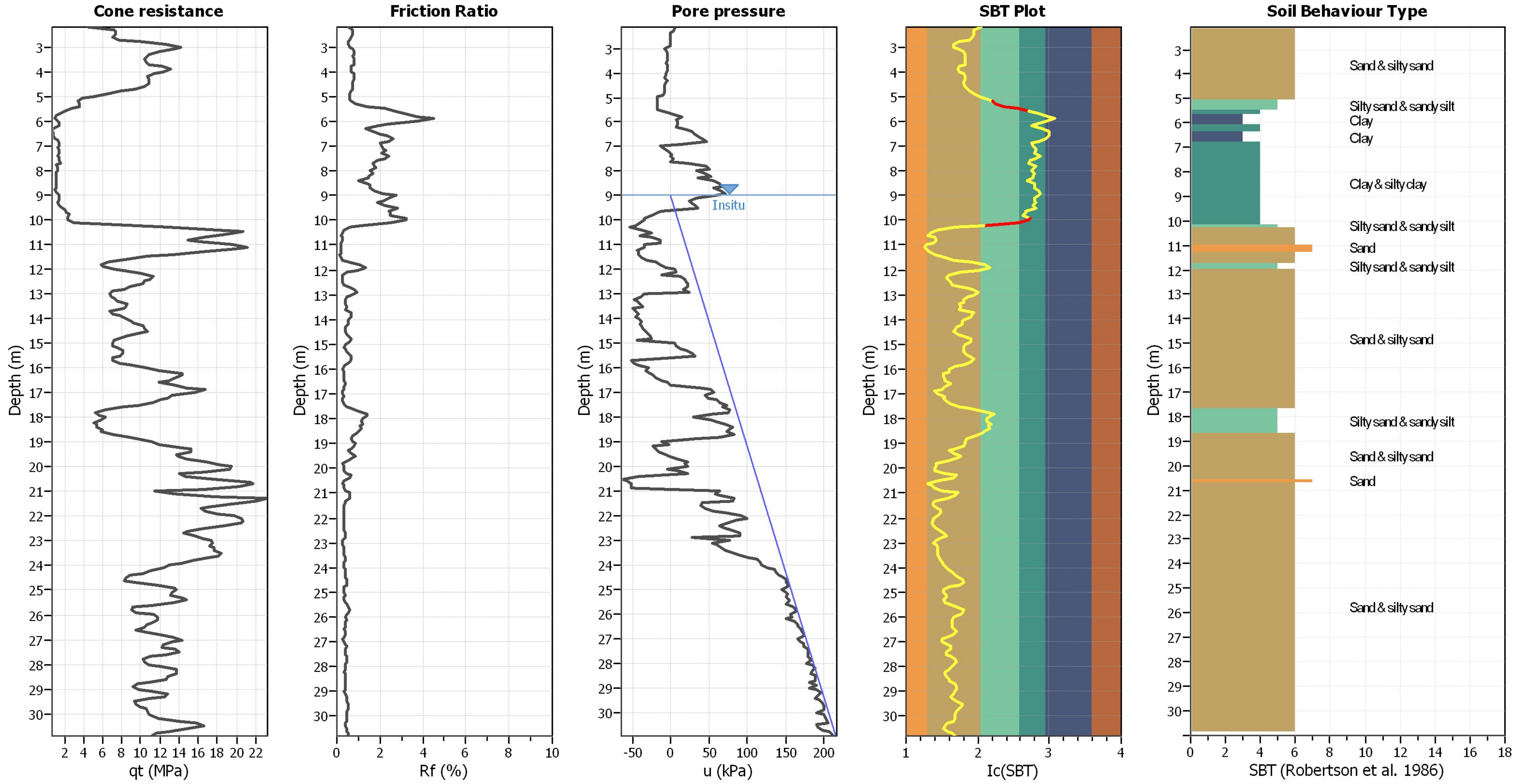






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



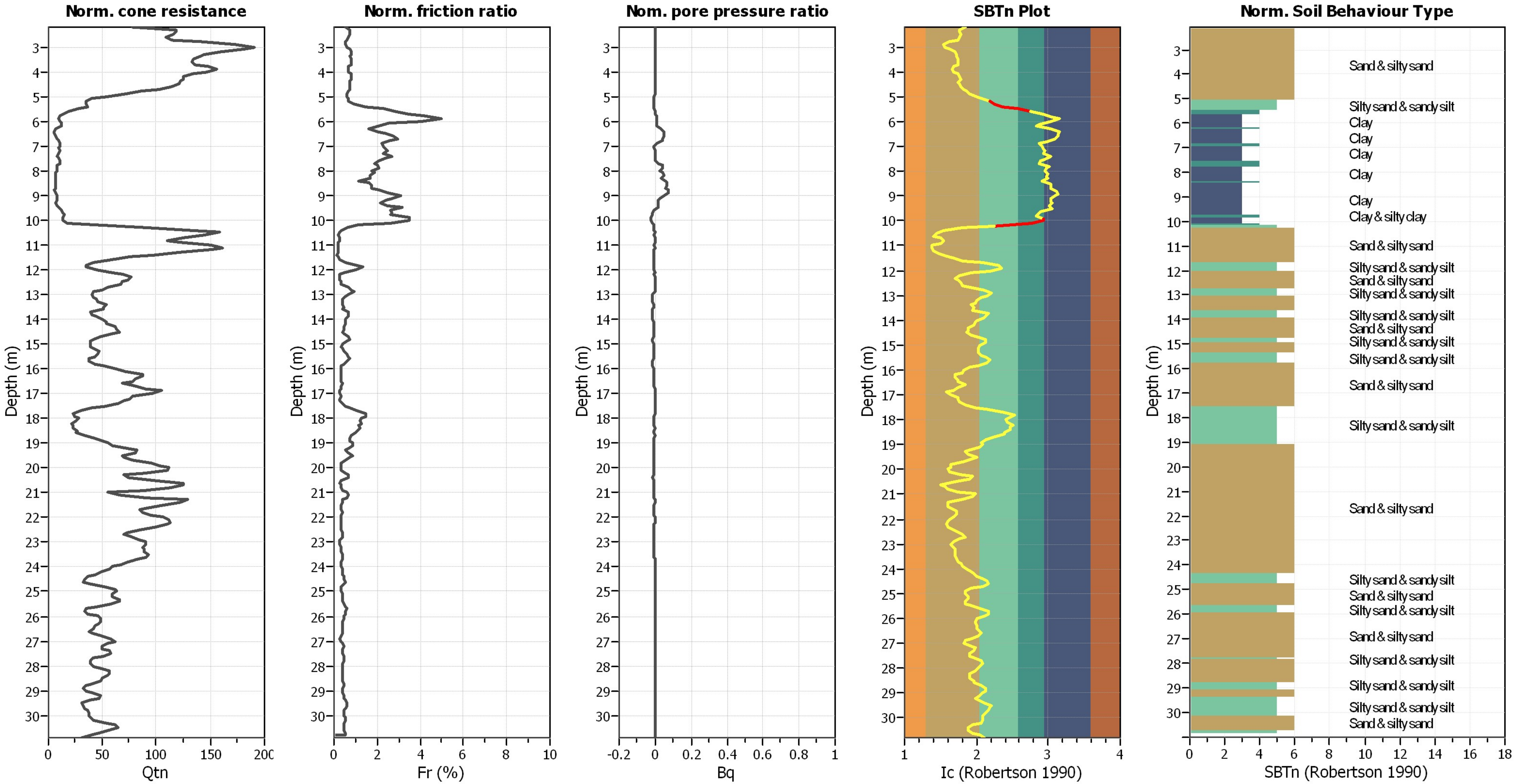
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	9.00 m	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	8.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.00 m	Fill height:	N/A	Limit depth:	20.00 m

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

CPT basic interpretation plots (normalized)



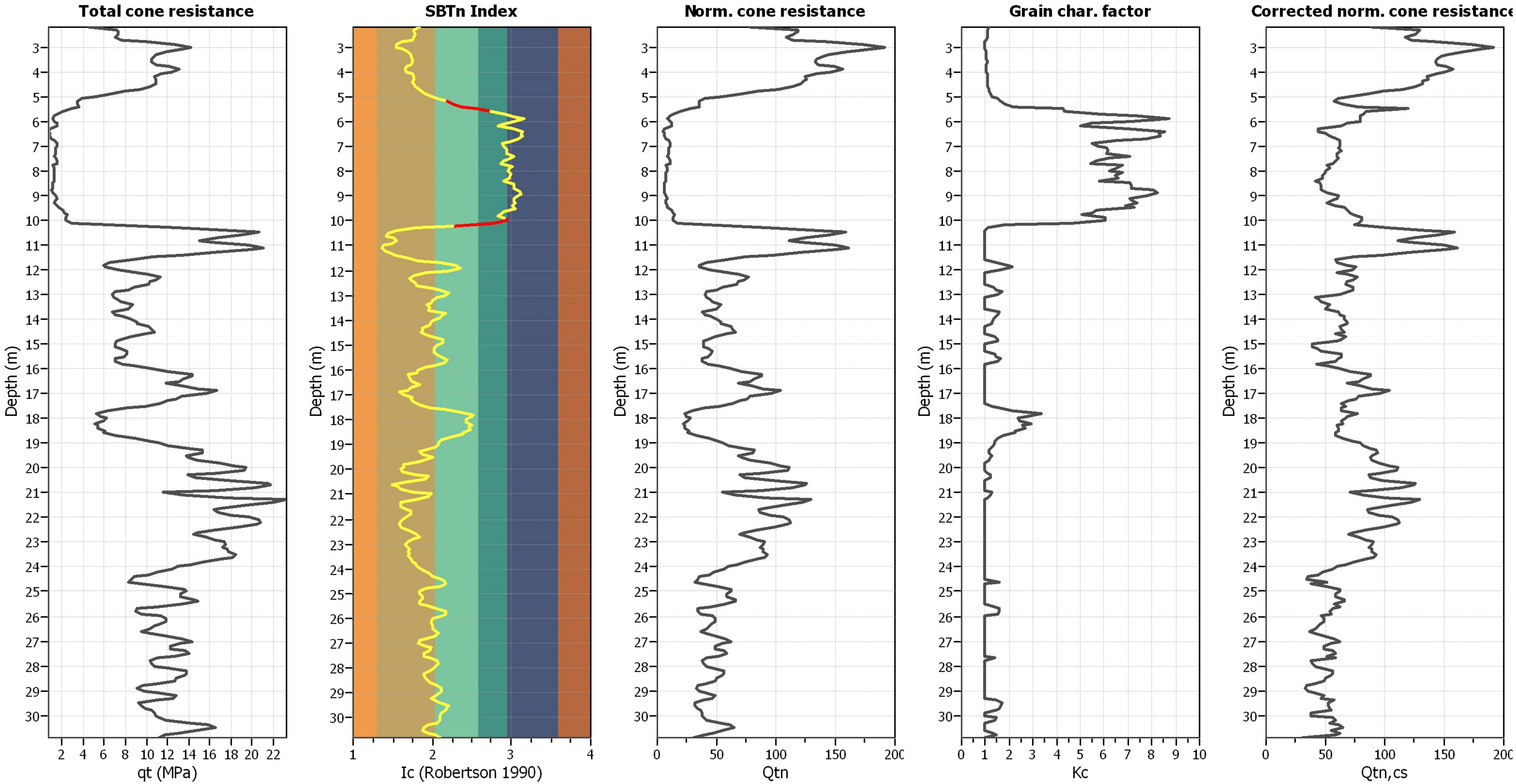
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	9.00 m	Fill weight:	N/A		
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes		
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No		
Earthquake magnitude M_w:	8.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils		
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	Yes		
Depth to water table (insitu):	9.00 m	Fill height:	N/A	Limit depth:	20.00 m		

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

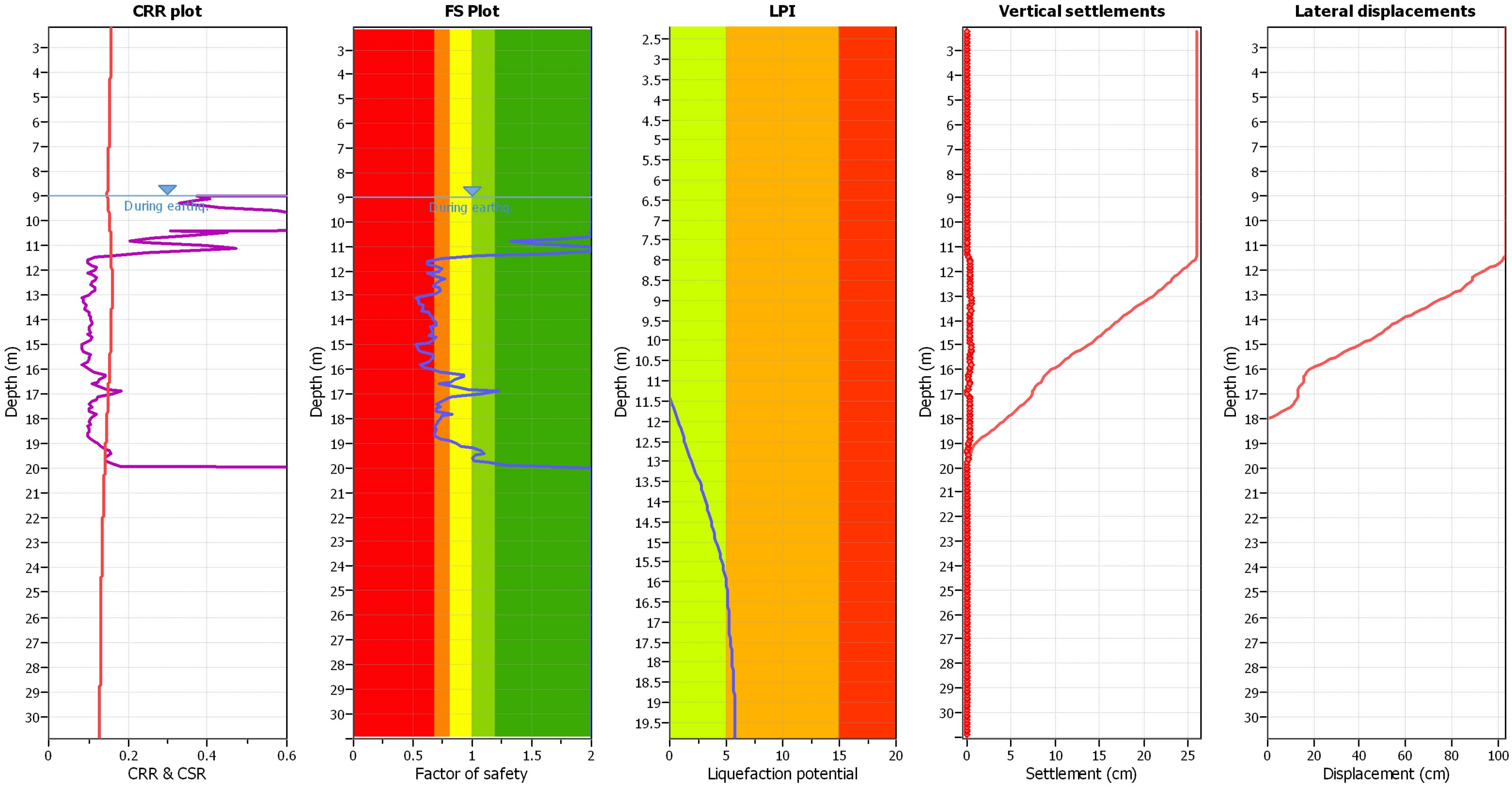
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	9.00 m	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	No
Earthquake magnitude M_w:	8.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.00 m	Fill height:	N/A	Limit depth:	20.00 m

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	9.00 m	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	8.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.00 m	Fill height:	N/A	Limit depth:	20.00 m

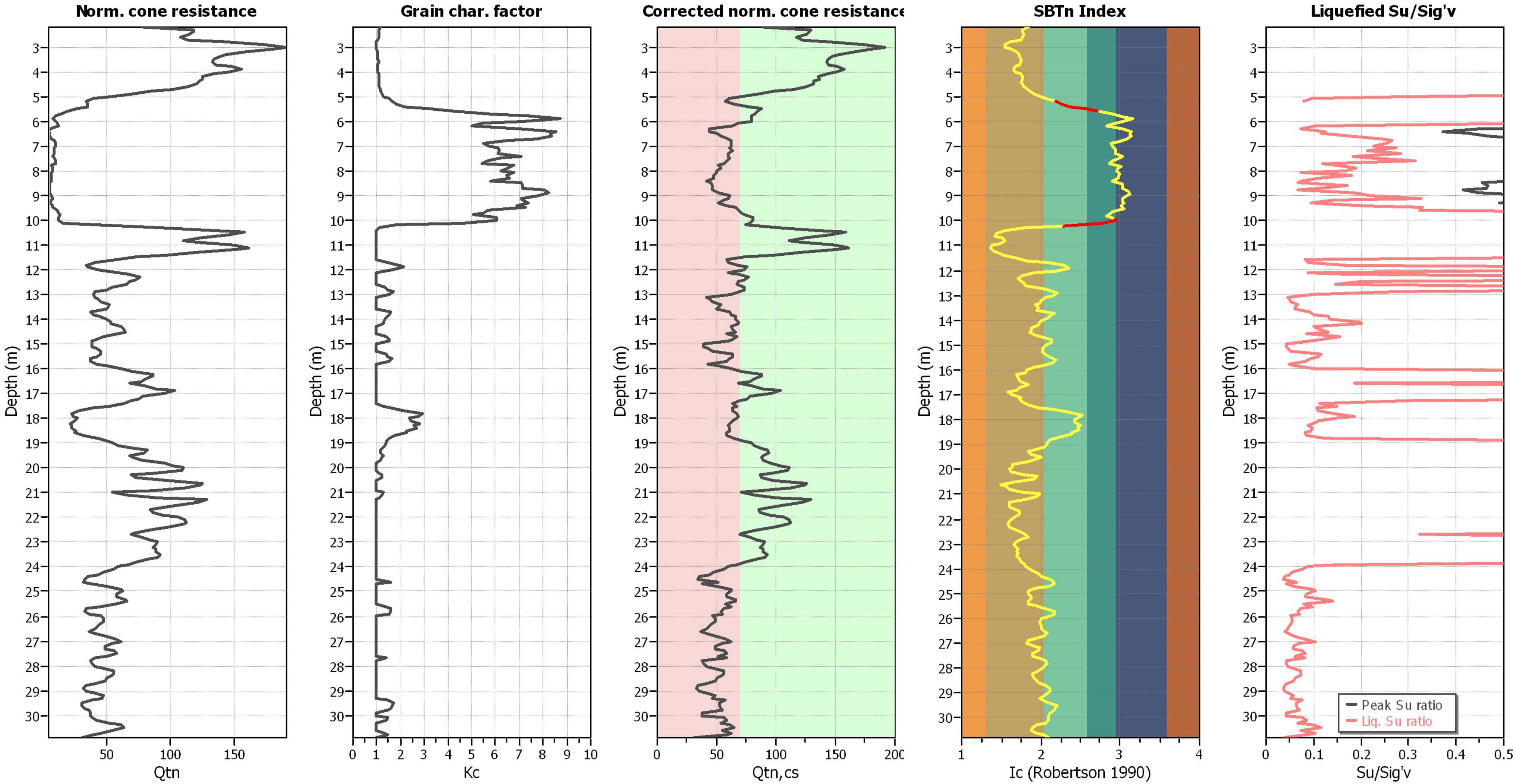
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

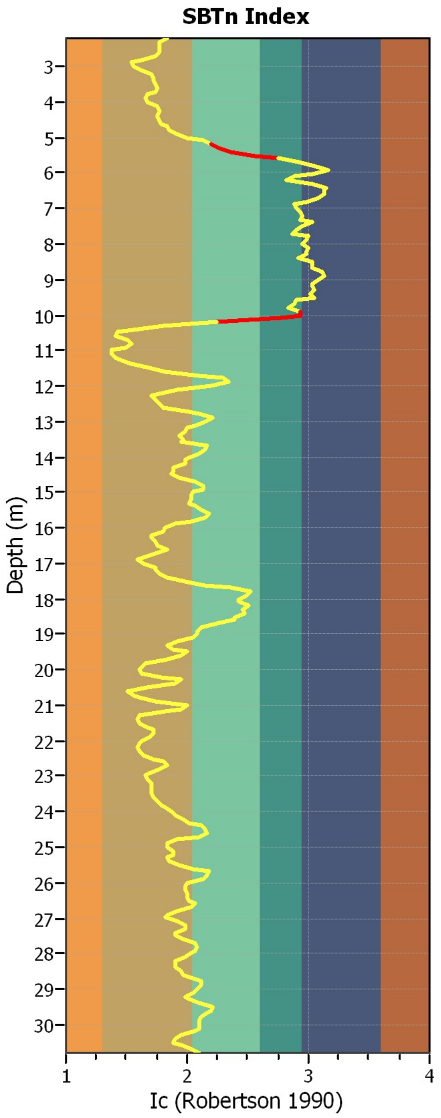
Analysis method:	Robertson (2009)	Depth to water table (erthq.):	9.00 m	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	8.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.00 m	Fill height:	N/A	Limit depth:	20.00 m

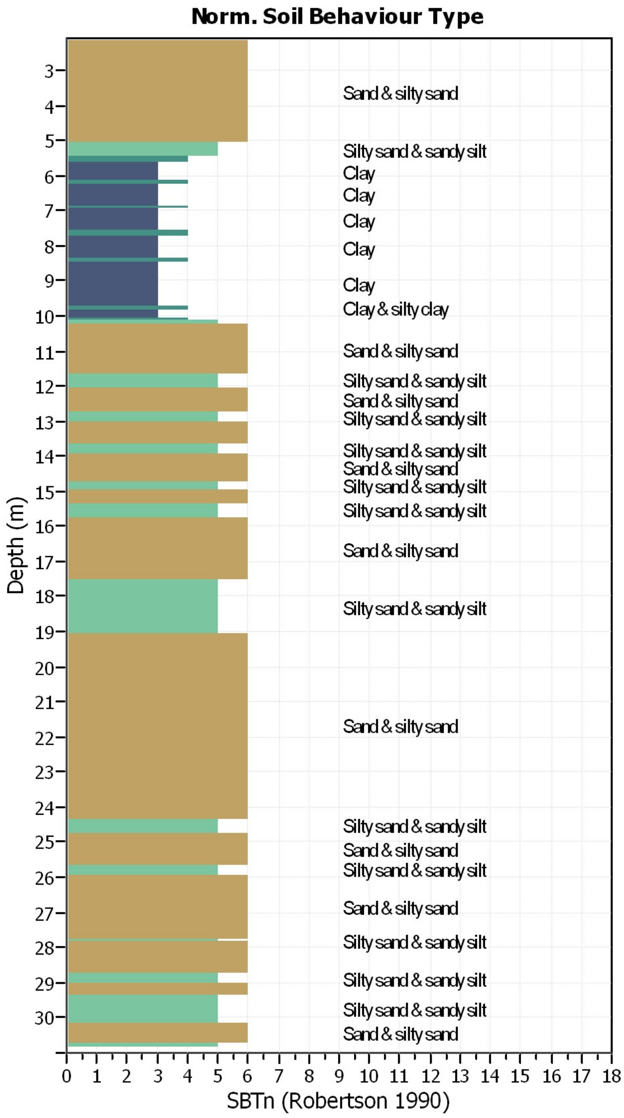
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





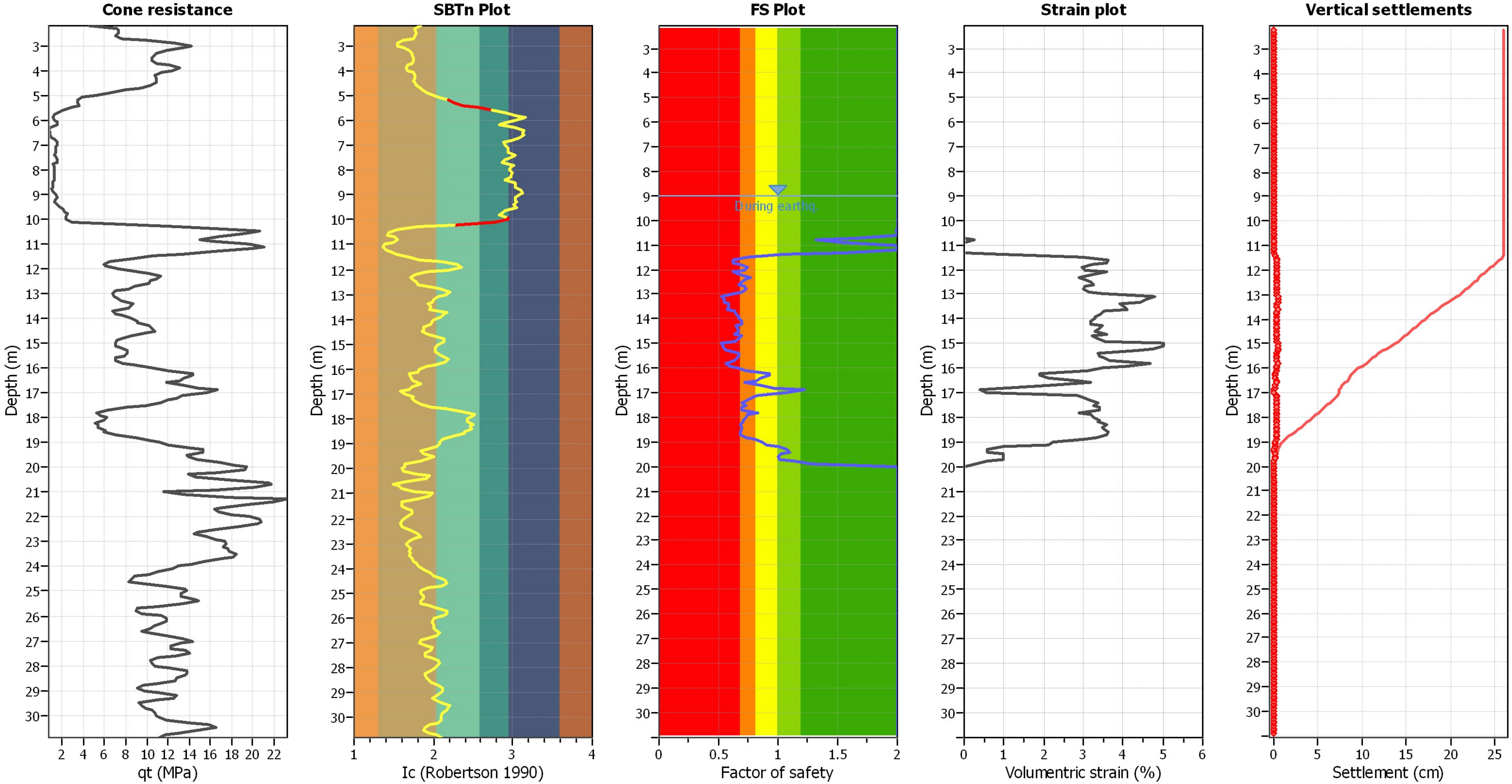
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0100
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	288
Total points excluded:	9
Exclusion percentage:	3.13%
Number of layers detected:	2

Estimation of post-earthquake settlements

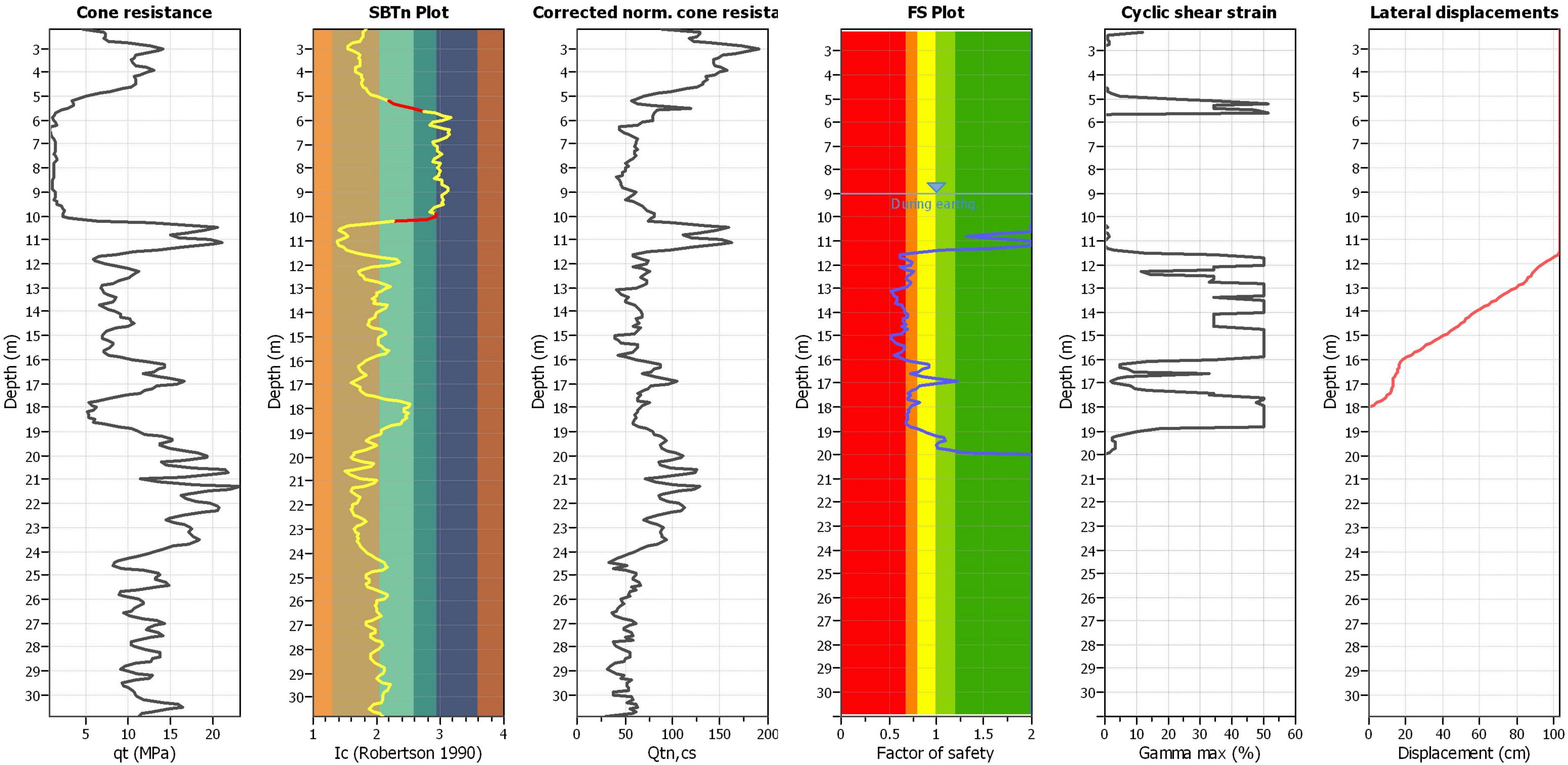


Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements

Geometric parameters: Level ground (or gently sloping) with free face (L: 237.00 m - H: 9.00 m)

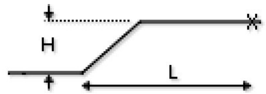


Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$Q_{tn,cs}$: Equivalent clean sand normalized CPT total cone resistance

F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Surface condition



:: Strength loss calculation (Robertson (2009) ::

Depth (m)	q_t (MPa)	Q_{tn}	K_c	$Q_{tn,cs}$	I_c	$S_{u(liq)}/\sigma'_v$	$S_{u(peak)}/\sigma'_v$
2.20	4.56	78.46	1.14	89.17	1.84	0.79	0.79
2.30	7.19	118.27	1.09	128.71	1.77	0.85	0.85
2.40	7.35	118.16	1.09	128.61	1.77	0.85	0.85
2.50	7.33	115.24	1.09	125.77	1.78	0.85	0.85
2.60	7.06	108.12	1.08	117.06	1.77	0.84	0.84
2.70	7.64	113.89	1.07	121.49	1.74	0.85	0.85
2.80	9.97	140.95	1.00	140.95	1.62	0.88	0.88
2.90	12.69	172.95	1.00	172.95	1.54	0.91	0.91
3.00	14.20	191.07	1.00	191.07	1.55	0.92	0.92
3.10	13.53	181.99	1.00	181.99	1.63	0.92	0.92
3.20	11.87	159.51	1.04	165.89	1.70	0.90	0.90
3.30	10.85	143.92	1.06	151.93	1.73	0.88	0.88
3.40	10.76	140.30	1.05	147.79	1.72	0.88	0.88
3.50	10.44	134.44	1.07	143.31	1.74	0.87	0.87
3.60	10.50	133.15	1.07	142.35	1.75	0.87	0.87
3.70	10.87	135.59	1.07	144.43	1.74	0.87	0.87
3.80	12.18	148.28	1.02	151.54	1.68	0.88	0.88
3.90	13.06	156.30	1.01	157.47	1.66	0.89	0.89
4.00	12.36	146.24	1.02	149.50	1.68	0.88	0.88
4.10	11.39	134.13	1.07	143.96	1.75	0.87	0.87
4.20	10.74	124.87	1.09	135.70	1.77	0.86	0.86
4.30	10.92	125.22	1.09	136.55	1.78	0.86	0.86
4.40	10.87	122.73	1.07	131.88	1.75	0.86	0.86
4.50	10.82	120.82	1.09	131.27	1.77	0.85	0.85
4.60	10.33	113.91	1.10	125.28	1.79	0.85	0.85
4.70	9.32	101.68	1.13	115.07	1.83	0.83	0.83
4.80	7.95	85.56	1.15	98.67	1.86	0.81	0.81
4.90	6.63	70.52	1.20	84.76	1.91	0.78	0.78
5.00	5.07	53.23	1.31	69.52	2.00	0.28	0.74
5.10	3.94	40.73	1.50	61.18	2.12	0.10	0.71
5.20	3.42	34.91	1.65	57.71	2.19	0.08	0.69
5.30	3.52	35.41	1.81	64.15	2.25	0.12	0.69
5.40	3.62	35.94	2.18	78.30	2.37	0.69	0.69
5.50	2.89	28.11	3.11	87.39	2.56	0.66	0.66
5.60	2.06	19.42	4.35	84.46	2.75	1.39	1.39
5.70	1.51	13.74	5.93	81.45	2.92	0.98	0.98
5.80	1.23	10.75	7.35	78.95	3.05	0.77	0.77
5.90	1.07	9.11	8.69	79.21	3.16	0.65	0.65
6.00	1.33	11.28	7.05	79.56	3.03	0.81	0.81
6.10	1.47	12.43	5.50	68.36	2.88	0.63	0.89
6.20	1.52	12.66	4.99	63.23	2.82	0.10	0.90
6.30	0.82	6.30	7.04	44.31	3.03	0.07	0.45
6.40	0.71	5.21	8.54	44.45	3.15	0.13	0.37
6.50	0.81	5.95	8.30	49.41	3.13	0.12	0.43
6.60	0.89	6.52	8.33	54.25	3.13	0.18	0.47
6.70	1.04	7.73	7.76	59.96	3.09	0.24	0.55
6.80	1.29	9.62	6.52	62.71	2.98	0.27	0.69
6.90	1.50	11.23	5.51	61.87	2.88	0.25	0.80

:: Strength loss calculation (Robertson (2009) :: (continued)

Depth (m)	q_t (MPa)	Q_{tn}	K_c	$Q_{tn,cs}$	I_c	$S_{u(liq)}/\sigma'_v$	$S_{u(peak)}/\sigma'_v$
7.00	1.48	10.85	5.73	62.22	2.90	0.22	0.78
7.10	1.39	9.98	6.12	61.05	2.94	0.27	0.71
7.20	1.44	10.27	6.14	63.04	2.94	0.21	0.73
7.30	1.40	9.80	6.11	59.93	2.94	0.28	0.70
7.40	1.24	8.45	7.10	59.97	3.03	0.18	0.60
7.50	1.44	9.81	6.23	61.16	2.95	0.22	0.70
7.60	1.54	10.43	5.64	58.84	2.89	0.32	0.74
7.70	1.55	10.37	5.42	56.22	2.87	0.12	0.74
7.80	1.19	7.58	6.80	51.53	3.00	0.15	0.54
7.90	1.29	8.19	6.53	53.48	2.98	0.19	0.59
8.00	1.29	8.13	6.20	50.38	2.95	0.17	0.58
8.10	1.20	7.38	6.78	50.00	3.00	0.07	0.53
8.20	1.20	7.31	6.49	47.42	2.98	0.18	0.52
8.30	1.20	7.22	6.57	47.43	2.98	0.11	0.52
8.40	1.22	7.22	5.80	41.86	2.91	0.07	0.52
8.50	1.10	6.36	7.06	44.91	3.03	0.07	0.45
8.60	1.14	6.53	7.13	46.56	3.03	0.17	0.47
8.70	1.15	6.49	7.14	46.38	3.03	0.11	0.46
8.80	1.05	5.82	8.05	46.83	3.11	0.07	0.42
8.90	1.13	6.26	8.25	51.60	3.13	0.18	0.45
9.00	1.40	7.86	7.80	61.34	3.09	0.22	0.56
9.10	1.51	8.52	7.06	60.13	3.03	0.33	0.61
9.20	1.40	7.79	7.13	55.59	3.03	0.14	0.56
9.30	1.26	6.87	7.40	50.84	3.06	0.09	0.49
9.40	1.52	8.42	6.92	58.24	3.02	0.21	0.60
9.50	1.63	9.07	7.26	65.82	3.05	0.33	0.65
9.60	2.12	12.01	5.68	68.21	2.90	0.32	0.86
9.70	2.28	12.92	5.51	71.20	2.88	0.92	0.92
9.80	2.55	14.51	5.07	73.60	2.83	1.04	1.04
9.90	2.36	13.23	6.06	80.19	2.94	0.95	0.95
10.00	2.38	13.32	6.05	80.64	2.94	0.95	0.95
10.10	2.97	16.83	4.60	77.47	2.78	1.20	1.20
10.20	6.37	41.11	1.81	74.36	2.25	0.71	0.71
10.30	12.74	90.86	1.11	100.79	1.80	0.81	0.81
10.40	17.97	134.85	1.00	134.85	1.55	0.87	0.87
10.50	20.64	158.29	1.00	158.29	1.43	0.89	0.89
10.60	18.92	145.26	1.00	145.26	1.41	0.88	0.88
10.70	16.63	124.62	1.00	124.62	1.51	0.86	0.86
10.80	14.95	110.73	1.00	110.73	1.55	0.84	0.84
10.90	16.11	120.43	1.00	120.43	1.49	0.85	0.85
11.00	19.64	150.14	1.00	150.14	1.38	0.89	0.89
11.10	21.10	161.34	1.00	161.34	1.37	0.90	0.90
11.20	19.95	151.03	1.00	151.03	1.40	0.89	0.89
11.30	16.39	121.34	1.00	121.34	1.49	0.85	0.85
11.40	13.34	97.22	1.00	97.22	1.54	0.82	0.82
11.50	10.65	74.51	1.00	74.51	1.69	0.79	0.79
11.60	8.65	58.44	1.00	58.44	1.82	0.08	0.75
11.70	6.54	41.04	1.45	59.32	2.09	0.09	0.71

:: Strength loss calculation (Robertson (2009) :: (continued)

Depth (m)	q_t (MPa)	Q_{tn}	K_c	$Q_{tn,cs}$	I_c	$S_{u(liq)}/\sigma'_v$	$S_{u(peak)}/\sigma'_v$
11.80	5.88	34.95	1.95	68.10	2.30	0.17	0.69
11.90	6.04	35.38	2.12	75.06	2.35	0.69	0.69
12.00	7.26	44.14	1.64	72.58	2.19	0.72	0.72
12.10	9.15	59.32	1.00	59.32	1.92	0.09	0.76
12.20	10.10	67.80	1.00	67.80	1.77	0.16	0.77
12.30	11.30	76.96	1.00	76.96	1.71	0.79	0.79
12.40	10.93	73.63	1.00	73.63	1.74	0.79	0.79
12.50	10.37	68.86	1.00	68.86	1.78	0.20	0.78
12.60	10.19	66.87	1.00	66.87	1.81	0.15	0.77
12.70	8.87	55.03	1.32	72.75	2.02	0.75	0.75
12.80	8.29	50.07	1.46	73.21	2.10	0.73	0.73
12.90	7.05	41.17	1.69	69.53	2.21	0.28	0.71
13.00	6.76	39.81	1.55	61.66	2.15	0.10	0.71
13.10	6.91	41.47	1.00	41.47	2.06	0.05	0.71
13.20	7.22	43.90	1.00	43.90	2.00	0.05	0.72
13.30	7.55	45.96	1.00	45.96	1.99	0.05	0.72
13.40	8.62	53.32	1.00	53.32	1.93	0.07	0.74
13.50	8.40	51.26	1.00	51.26	1.96	0.06	0.74
13.60	8.24	50.35	1.00	50.35	1.94	0.06	0.74
13.70	6.69	38.27	1.58	60.53	2.16	0.09	0.70
13.80	7.04	40.37	1.54	62.08	2.14	0.10	0.71
13.90	7.76	44.93	1.46	65.70	2.10	0.13	0.72
14.00	8.48	50.31	1.31	66.03	2.01	0.14	0.74
14.10	9.05	54.01	1.27	68.76	1.98	0.20	0.74
14.20	9.11	54.23	1.27	68.87	1.98	0.20	0.74
14.30	10.17	62.00	1.00	62.00	1.89	0.10	0.76
14.40	10.44	63.60	1.00	63.60	1.88	0.11	0.77
14.50	10.76	65.67	1.00	65.67	1.86	0.13	0.77
14.60	9.92	59.13	1.00	59.13	1.93	0.09	0.76
14.70	8.51	48.59	1.39	67.41	2.06	0.16	0.73
14.80	7.59	42.17	1.52	63.99	2.13	0.11	0.71
14.90	7.16	39.49	1.52	60.02	2.13	0.09	0.70
15.00	7.03	39.53	1.00	39.53	2.05	0.04	0.70
15.10	7.01	39.52	1.00	39.52	2.03	0.04	0.70
15.20	7.48	42.18	1.00	42.18	2.03	0.05	0.71
15.30	8.19	46.34	1.00	46.34	2.02	0.05	0.72
15.40	8.19	45.45	1.41	63.94	2.07	0.11	0.72
15.50	7.82	42.57	1.49	63.35	2.12	0.11	0.71
15.60	7.07	37.52	1.62	60.94	2.18	0.09	0.70
15.70	7.07	37.78	1.54	58.22	2.14	0.08	0.70
15.80	7.77	42.97	1.00	42.97	2.03	0.05	0.71
15.90	8.86	50.77	1.00	50.77	1.91	0.06	0.74
16.00	10.50	61.91	1.00	61.91	1.82	0.10	0.76
16.10	11.92	70.81	1.00	70.81	1.80	0.78	0.78
16.20	14.30	87.37	1.00	87.37	1.70	0.81	0.81
16.30	14.33	87.22	1.00	87.22	1.71	0.81	0.81
16.40	13.51	81.14	1.00	81.14	1.74	0.80	0.80
16.50	13.02	77.60	1.00	77.60	1.75	0.79	0.79

:: Strength loss calculation (Robertson (2009) :: (continued)

Depth (m)	q_t (MPa)	Q_{tn}	K_c	$Q_{tn,cs}$	I_c	$S_{u(liq)}/\sigma'_v$	$S_{u(peak)}/\sigma'_v$
16.60	11.87	68.56	1.00	68.56	1.84	0.19	0.78
16.70	13.55	80.71	1.00	80.71	1.74	0.80	0.80
16.80	14.80	89.42	1.00	89.42	1.69	0.81	0.81
16.90	16.69	104.03	1.00	104.03	1.58	0.83	0.83
17.00	15.70	95.99	1.00	95.99	1.63	0.82	0.82
17.10	13.34	78.39	1.00	78.39	1.74	0.79	0.79
17.20	12.82	75.17	1.00	75.17	1.74	0.79	0.79
17.30	12.05	69.62	1.00	69.62	1.77	0.31	0.78
17.40	11.36	63.90	1.00	63.90	1.84	0.11	0.77
17.50	9.85	52.69	1.27	67.08	1.98	0.15	0.74
17.60	8.06	40.40	1.56	63.03	2.15	0.11	0.71
17.70	6.34	29.34	2.16	63.25	2.36	0.11	0.67
17.80	5.31	22.92	2.91	66.61	2.53	0.14	0.64
17.90	5.70	24.82	2.76	68.54	2.50	0.19	0.65
18.00	6.26	28.03	2.39	66.95	2.42	0.15	0.66
18.10	5.89	26.13	2.43	63.42	2.43	0.11	0.65
18.20	5.13	21.87	2.80	61.32	2.51	0.10	0.63
18.30	5.33	23.05	2.59	59.60	2.46	0.09	0.64
18.40	5.39	23.07	2.66	61.41	2.48	0.10	0.64
18.50	6.01	26.55	2.29	60.68	2.39	0.09	0.66
18.60	5.89	25.98	2.25	58.56	2.39	0.08	0.65
18.70	7.09	32.94	1.79	58.97	2.25	0.09	0.68
18.80	8.90	43.39	1.50	64.95	2.12	0.12	0.72
18.90	10.39	51.54	1.41	72.70	2.07	0.74	0.74
19.00	11.19	55.28	1.42	78.74	2.08	0.75	0.75
19.10	11.94	59.64	1.37	81.50	2.05	0.76	0.76
19.20	14.00	73.26	1.20	87.85	1.91	0.78	0.78
19.30	15.25	81.80	1.13	92.74	1.84	0.80	0.80
19.40	15.27	80.37	1.17	94.15	1.88	0.80	0.80
19.50	13.70	68.75	1.30	89.57	2.00	0.78	0.78
19.60	13.85	70.44	1.25	87.77	1.96	0.78	0.78
19.70	14.77	78.24	1.13	88.49	1.83	0.79	0.79
19.80	16.92	95.57	1.00	95.57	1.65	0.82	0.82
19.90	18.28	103.15	1.00	103.15	1.65	0.83	0.83
20.00	19.39	110.97	1.00	110.97	1.60	0.84	0.84
20.10	19.28	109.54	1.00	109.54	1.62	0.84	0.84
20.20	16.58	88.38	1.09	96.60	1.78	0.81	0.81
20.30	13.97	69.73	1.23	86.08	1.95	0.78	0.78
20.40	14.56	73.52	1.20	87.89	1.91	0.78	0.78
20.50	17.78	97.69	1.00	97.69	1.68	0.82	0.82
20.60	21.43	125.41	1.00	125.41	1.50	0.86	0.86
20.70	21.80	124.39	1.00	124.39	1.57	0.86	0.86
20.80	20.22	113.09	1.00	113.09	1.61	0.84	0.84
20.90	16.37	86.61	1.00	86.61	1.74	0.81	0.81
21.00	11.50	54.83	1.29	70.89	1.99	0.75	0.75
21.10	13.55	65.97	1.23	81.31	1.94	0.77	0.77
21.20	17.79	91.18	1.12	101.77	1.81	0.81	0.81
21.30	23.24	128.71	1.00	128.71	1.61	0.86	0.86

:: Strength loss calculation (Robertson (2009) :: (continued)

Depth (m)	q_t (MPa)	Q_{tn}	K_c	$Q_{tn,cs}$	I_c	$S_{u(liq)}/\sigma'_v$	$S_{u(peak)}/\sigma'_v$
21.40	21.99	121.76	1.00	121.76	1.60	0.86	0.86
21.50	19.42	106.35	1.00	106.35	1.61	0.84	0.84
21.60	17.16	91.22	1.00	91.22	1.68	0.81	0.81
21.70	16.35	85.04	1.00	85.04	1.73	0.80	0.80
21.80	16.77	87.19	1.00	87.19	1.72	0.81	0.81
21.90	18.07	95.23	1.00	95.23	1.68	0.82	0.82
22.00	19.70	106.26	1.00	106.26	1.62	0.84	0.84
22.10	20.57	111.20	1.00	111.20	1.61	0.84	0.84
22.20	20.74	112.57	1.00	112.57	1.59	0.84	0.84
22.30	20.50	110.27	1.00	110.27	1.61	0.84	0.84
22.40	18.74	98.94	1.00	98.94	1.65	0.83	0.83
22.50	16.51	84.19	1.00	84.19	1.72	0.80	0.80
22.60	14.78	72.77	1.00	72.77	1.80	0.78	0.78
22.70	14.40	69.66	1.00	69.66	1.84	0.32	0.78
22.80	15.63	77.48	1.00	77.48	1.77	0.79	0.79
22.90	16.54	84.52	1.00	84.52	1.69	0.80	0.80
23.00	17.34	89.90	1.00	89.90	1.65	0.81	0.81
23.10	17.45	89.47	1.00	89.47	1.67	0.81	0.81
23.20	17.23	86.78	1.00	86.78	1.71	0.81	0.81
23.30	17.59	88.78	1.00	88.78	1.70	0.81	0.81
23.40	17.70	88.77	1.00	88.77	1.71	0.81	0.81
23.50	18.48	92.89	1.00	92.89	1.70	0.82	0.82
23.60	18.12	89.95	1.00	89.95	1.72	0.81	0.81
23.70	17.08	83.63	1.00	83.63	1.75	0.80	0.80
23.80	15.41	74.06	1.00	74.06	1.78	0.79	0.79
23.90	14.23	67.18	1.00	67.18	1.81	0.15	0.77
24.00	12.97	59.68	1.00	59.68	1.86	0.09	0.76
24.10	12.43	56.10	1.00	56.10	1.90	0.07	0.75
24.20	11.05	48.16	1.00	48.16	1.97	0.06	0.73
24.30	10.19	43.36	1.00	43.36	2.01	0.05	0.72
24.40	8.82	35.73	1.00	35.73	2.11	0.04	0.69
24.50	8.48	33.65	1.00	33.65	2.15	0.04	0.68
24.60	8.25	32.29	1.60	51.62	2.17	0.06	0.68
24.70	9.31	37.75	1.00	37.75	2.09	0.04	0.70
24.80	11.74	51.40	1.00	51.40	1.92	0.06	0.74
24.90	13.62	61.86	1.00	61.86	1.84	0.10	0.76
25.00	13.79	62.56	1.00	62.56	1.84	0.10	0.76
25.10	13.21	58.61	1.00	58.61	1.88	0.08	0.75
25.20	13.18	58.39	1.00	58.39	1.88	0.08	0.75
25.30	14.50	65.42	1.00	65.42	1.84	0.13	0.77
25.40	14.81	66.34	1.00	66.34	1.85	0.14	0.77
25.50	13.40	57.64	1.00	57.64	1.93	0.08	0.75
25.60	11.08	44.85	1.37	61.62	2.05	0.10	0.72
25.70	9.12	34.65	1.60	55.59	2.17	0.07	0.69
25.80	8.96	34.01	1.59	53.96	2.17	0.07	0.69
25.90	9.47	36.54	1.51	55.07	2.13	0.07	0.69
26.00	11.30	46.17	1.00	46.17	2.00	0.05	0.72
26.10	11.78	48.38	1.00	48.38	1.99	0.06	0.73

:: Strength loss calculation (Robertson (2009) :: (continued)

Depth (m)	q_t (MPa)	Q_{tn}	K_c	$Q_{tn,cs}$	I_c	$S_{u(liq)}/\sigma'_v$	$S_{u(peak)}/\sigma'_v$
26.20	11.78	48.45	1.00	48.45	1.98	0.06	0.73
26.30	11.13	45.07	1.00	45.07	2.00	0.05	0.72
26.40	10.76	43.35	1.00	43.35	2.01	0.05	0.72
26.50	10.33	41.16	1.00	41.16	2.02	0.05	0.71
26.60	9.54	36.97	1.00	36.97	2.07	0.04	0.70
26.70	10.18	40.09	1.00	40.09	2.04	0.04	0.71
26.80	11.85	49.14	1.00	49.14	1.92	0.06	0.73
26.90	13.42	57.76	1.00	57.76	1.84	0.08	0.75
27.00	14.37	62.29	1.00	62.29	1.83	0.10	0.76
27.10	13.26	54.90	1.00	54.90	1.92	0.07	0.75
27.20	12.30	49.28	1.00	49.28	1.98	0.06	0.73
27.30	12.24	49.04	1.00	49.04	1.98	0.06	0.73
27.40	13.64	56.53	1.00	56.53	1.90	0.08	0.75
27.50	14.06	58.33	1.00	58.33	1.90	0.08	0.75
27.60	12.72	50.67	1.00	50.67	1.97	0.06	0.74
27.70	11.05	41.90	1.39	58.42	2.06	0.08	0.71
27.80	10.29	38.36	1.00	38.36	2.09	0.04	0.70
27.90	10.43	39.29	1.00	39.29	2.06	0.04	0.70
28.00	10.78	41.11	1.00	41.11	2.03	0.05	0.71
28.10	12.00	46.95	1.00	46.95	1.98	0.05	0.73
28.20	13.73	55.76	1.00	55.76	1.90	0.07	0.75
28.30	13.79	55.83	1.00	55.83	1.90	0.07	0.75
28.40	13.68	55.10	1.00	55.10	1.91	0.07	0.75
28.50	12.81	50.17	1.00	50.17	1.96	0.06	0.73
28.60	12.61	48.90	1.00	48.90	1.97	0.06	0.73
28.70	10.84	39.93	1.00	39.93	2.07	0.04	0.71
28.80	9.69	34.51	1.00	34.51	2.12	0.04	0.69
28.90	9.15	32.37	1.00	32.37	2.12	0.04	0.68
29.00	9.75	35.04	1.00	35.04	2.09	0.04	0.69
29.10	11.03	40.52	1.00	40.52	2.05	0.04	0.71
29.20	12.82	48.57	1.00	48.57	1.99	0.06	0.73
29.30	12.54	46.70	1.00	46.70	2.02	0.05	0.73
29.40	10.99	39.16	1.45	56.81	2.10	0.08	0.70
29.50	9.29	31.11	1.68	52.40	2.21	0.06	0.68
29.60	9.48	31.81	1.66	52.80	2.20	0.07	0.68
29.70	9.96	33.88	1.59	53.94	2.17	0.07	0.69
29.80	10.48	36.51	1.49	54.30	2.12	0.07	0.69
29.90	10.75	37.82	1.00	37.82	2.09	0.04	0.70
30.00	10.87	38.14	1.00	38.14	2.09	0.04	0.70
30.10	11.17	39.09	1.45	56.52	2.10	0.08	0.70
30.20	11.85	42.07	1.39	58.66	2.06	0.08	0.71
30.30	14.13	53.12	1.00	53.12	1.95	0.07	0.74
30.40	15.85	61.32	1.00	61.32	1.89	0.10	0.76
30.50	16.53	64.26	1.00	64.26	1.88	0.12	0.77
30.60	14.79	55.14	1.00	55.14	1.96	0.07	0.75
30.70	12.92	45.96	1.35	62.20	2.04	0.10	0.72
30.80	11.67	40.06	1.45	58.17	2.10	0.08	0.71
30.90	11.21	30.74	1.00	30.74	-1.00	0.04	0.67

:: Strength loss calculation (Robertson (2009) :: (continued)

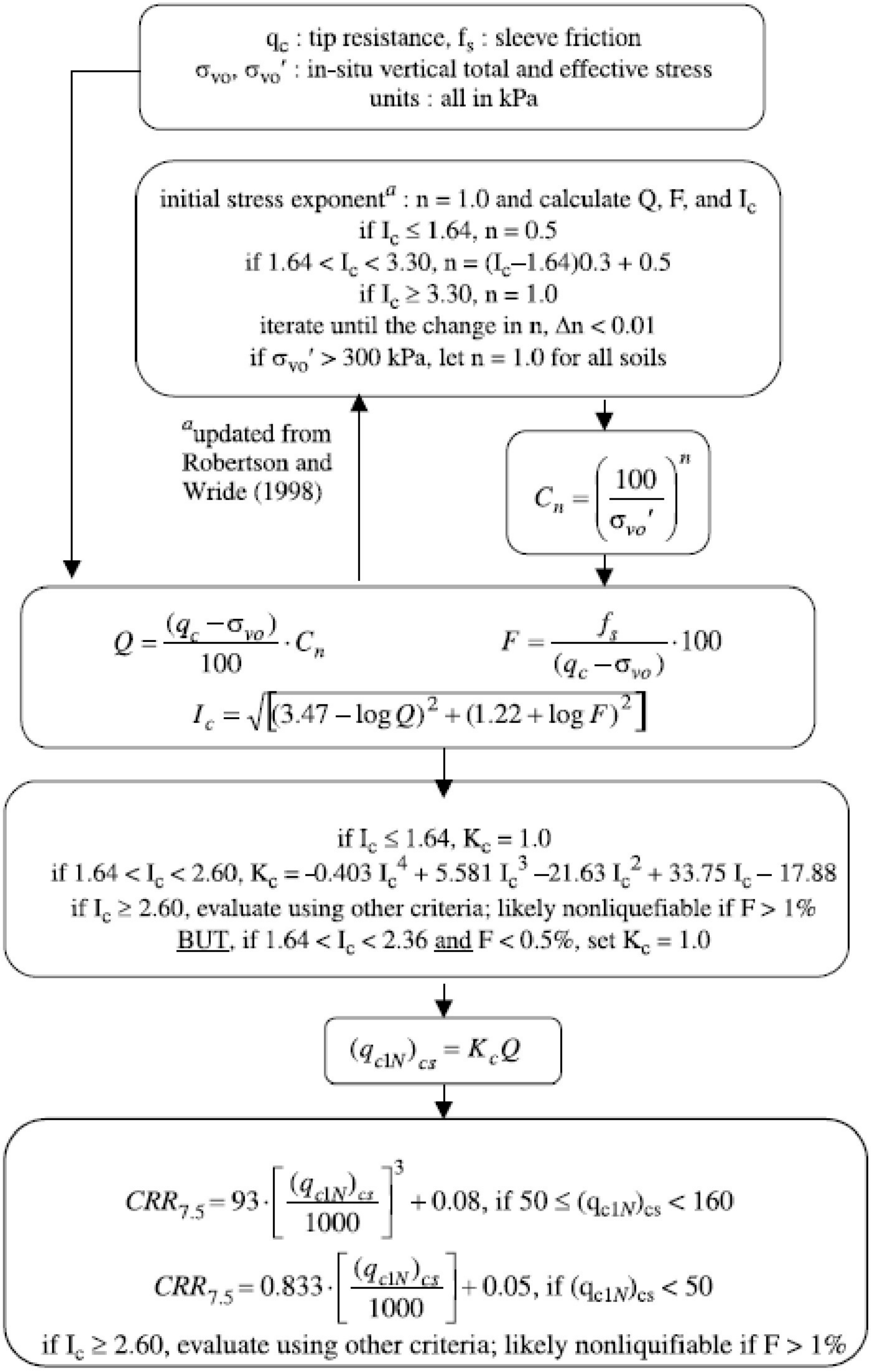
Depth (m)	q_t (MPa)	Q_{tn}	K_c	$Q_{tn,cs}$	I_c	$S_{u(liq)}/\sigma'_v$	$S_{u(peak)}/\sigma'_v$

Abbreviations

q_t: Total cone resistance
K_c: Cone resistance correction factor due to fines
$Q_{tn,cs}$: Adjusted and corrected cone resistance due to fines
I_c: Soil behavior type index
$S_{u(liq)}/\sigma'_v$: Calculated liquefied undrained strength ratio
$S_{u(peak)}/\sigma'_v$: Calculated peak undrained strength ratio

Procedure for the evaluation of soil liquefaction resistance, NCEER (1998)

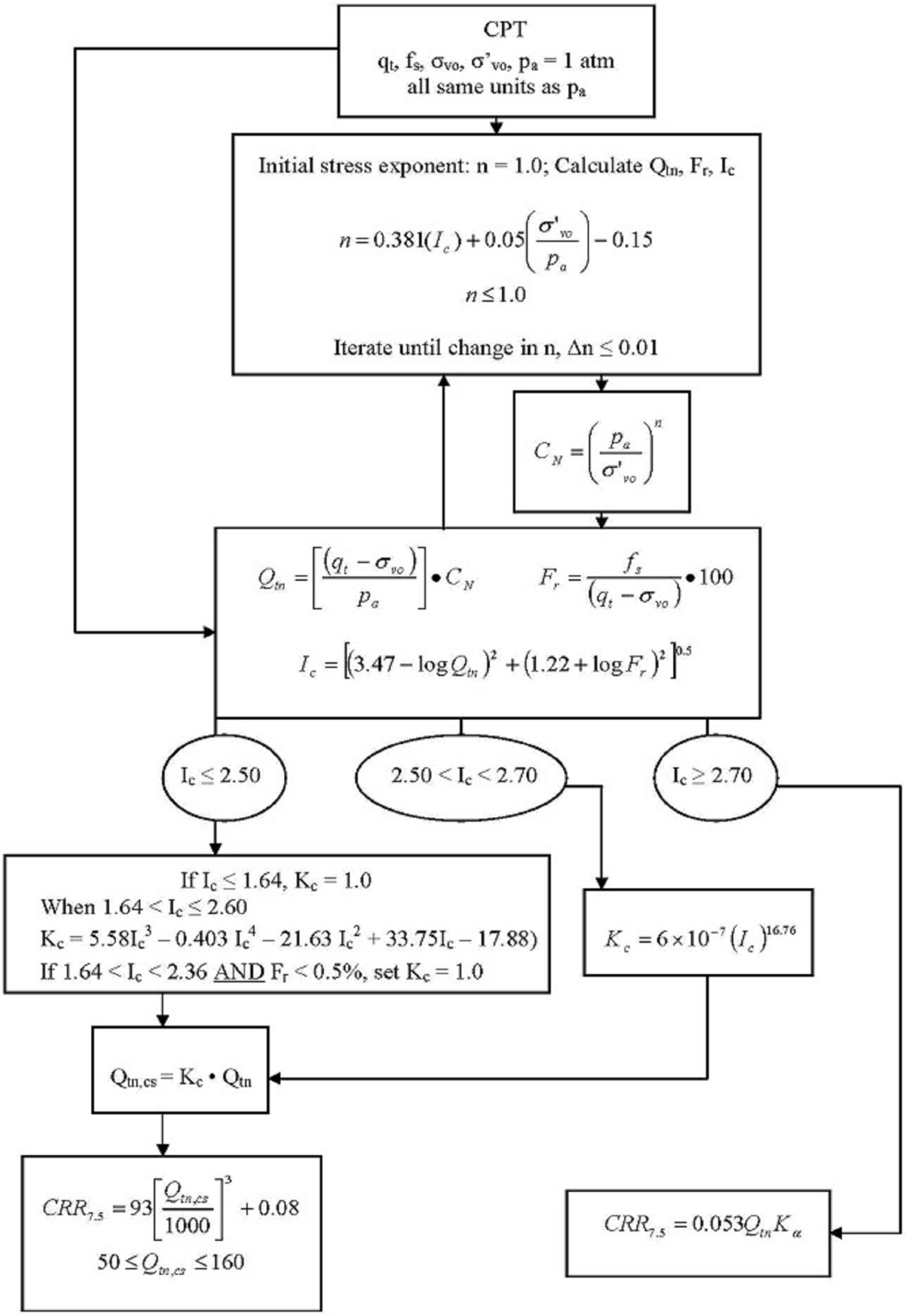
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)

Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. This procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



[1] P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009

Procedure for the evaluation of soil liquefaction resistance, Idriss & Boulanger (2008)

q_c: tip resistance, f_s: sleeve friction
σ_{vo}, σ_{vo}': in situ vertical total and effective stress

$m = 1.338 - 0.249 \times (q_{c1N})^{0.264}$
iterate until change in m, $\Delta m < 0.01$

$$C_N = \left(\frac{P_a}{\sigma'_{vo}}\right)^m \leq 1.7$$

$$q_{c1N} = \frac{C_N \times q_c}{P_a}$$

$I_c < 2.60$

$I_c > 2.60$

$q_{c1Ncs} = q_{c1N} + \Delta q_{c1N}$
where :

$$\Delta q_{c1N} = \left(5.4 + \frac{q_{c1N}}{16}\right) \times e^{\left(1.63 + \frac{9.7}{FC+0.01} - \left(\frac{15.7}{FC+0.01}\right)^2\right)}$$

$$CRR_{M=7.5,\sigma'_{vo}=1} = 0.80 \times \frac{s_u}{\sigma'_{vo}} \times K_a$$

$$CRR_{M=7.5,\sigma'_{vo}=1} = e^{\left(\frac{q_{c1Ncs}}{540} + \left(\frac{q_{c1Ncs}}{67}\right)^2 - \left(\frac{q_{c1Ncs}}{80}\right)^3 + \left(\frac{q_{c1Ncs}}{114}\right)^4 - 3\right)}$$

Procedure for the evaluation of soil liquefaction resistance (sandy soils), Moss et al. (2006)

CPT

q_t, f_s, I_c

$I_c < I_c$ cut-off

Initial estimate using raw tip measurements, friction ratio. Calculate $q_{t,1}$. Repeat until an acceptable convergence tolerance is achieved.

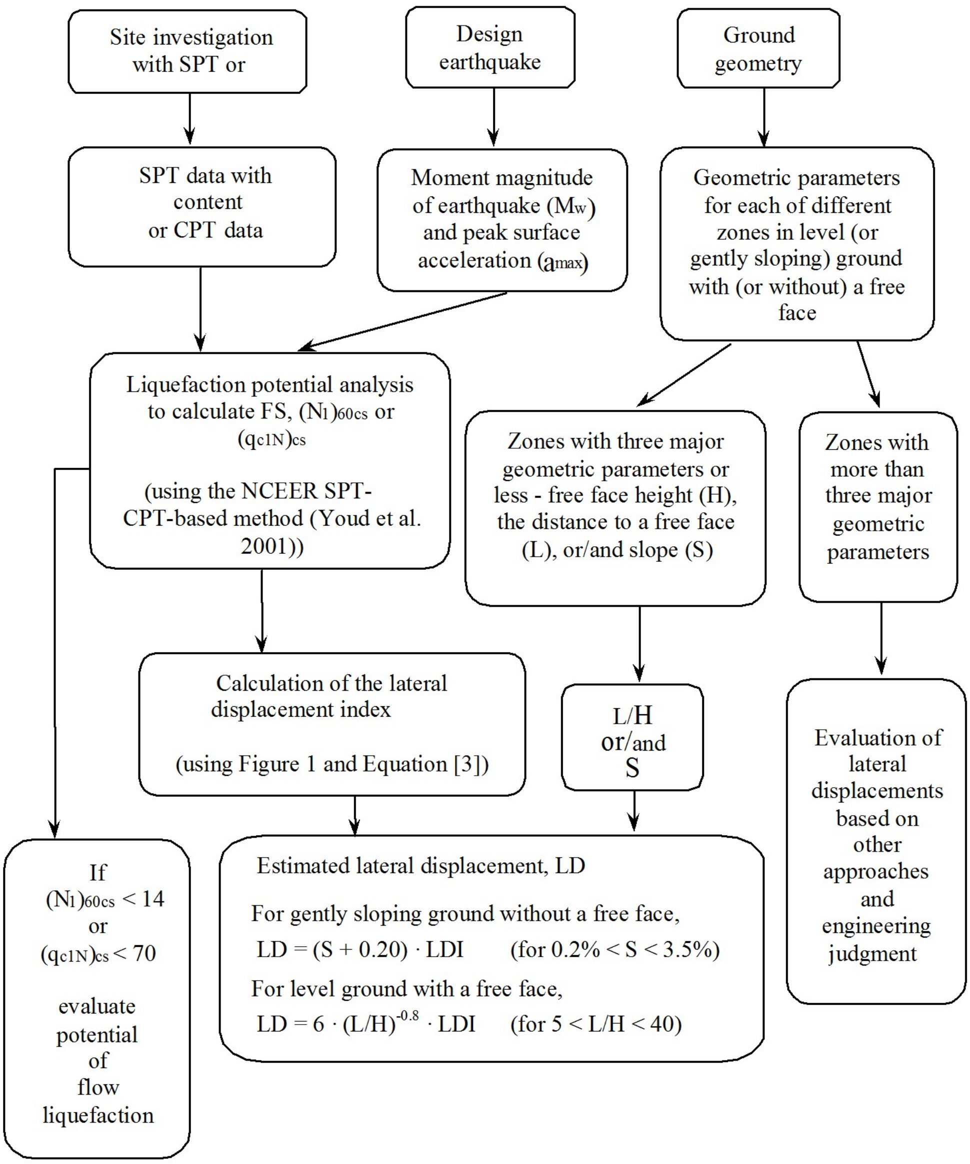
$$c = f_1 \cdot \left(\frac{R_f}{f_3}\right)^{f_2}$$

$$C_q = \left(\frac{P_a}{\sigma'_v}\right)^{c}$$

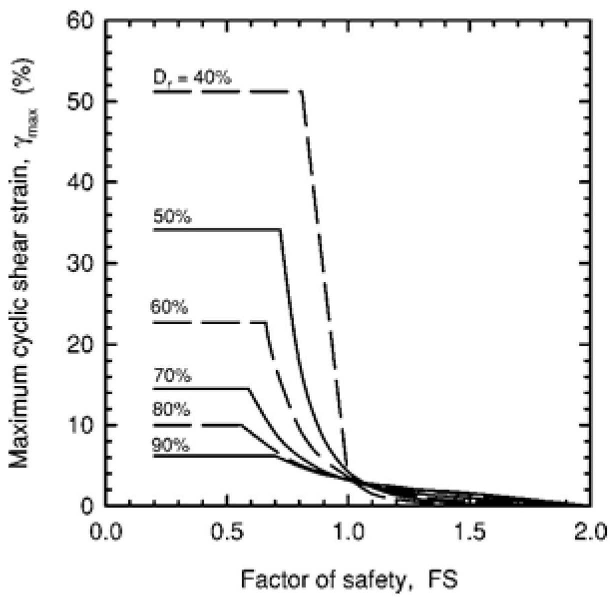
$$q_{t,1} = C_q \cdot q_t$$

$$CRR = \exp\left[\frac{q_{t,1}^{1.045} + q_{t,1} \cdot (0.110 \cdot R_f) + (0.001 \cdot R_f) + c \cdot (1 + 0.850 \cdot R_f) - 0.848 \cdot \ln(M_w) - 0.002 \cdot \ln(\sigma'_v) - 20.923 + 1.632 \cdot \Phi^{-1}(P_L)}{7.177}\right]$$

Procedure for the evaluation of liquefaction-induced lateral spreading displacements



[1] Flow chart illustrating major steps in estimating liquefaction-induced lateral spreading displacements using the proposed approach



[1] **Figure 1**

$$LDI = \int_0^{Z_{max}} \gamma_{max} dz$$

[1] **Equation [3]**

[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

Procedure for the estimation of seismic induced settlements in dry sands

Average shear stress, τ_{av}

$$\tau_{av} = CSR^{*} \cdot \sigma_{v0}' = 0.65 \cdot \frac{a_{max}}{g} \cdot \sigma_{v0} \cdot r_d$$

↓

Estimate small shear strain modulus, G_0

$$G_0 = 0.0188 \cdot \left[10^{(0.55 I_c - 1.68)}\right] \cdot (q_t - \sigma_v)$$

↓

Estimate shear strain amplitude, γ (based on Pradel (1998))

$$\gamma = \left[\frac{1 + \alpha \cdot e^{bR}}{1 + \alpha}\right] \cdot R \cdot 100\ (\%)$$

$$R = \frac{\tau_{av}}{G_0} \text{ (Note } \tau_{av} \text{ and } G_0 \text{ same units)}$$

$$\alpha = 0.0389 \cdot \left(\frac{\sigma_v}{p_a}\right) + 0.124$$

$$b = 6400 \cdot \left(\frac{\sigma_v}{p_a}\right)$$

↓

Estimate volumetric strain in 15 cycles

$$\varepsilon_{vol(15)} = \gamma \cdot \left[\frac{(N_1)_{60,cs}}{20}\right]^{-1.20}$$

$$(N_1)_{60,cs} = \frac{Q_{tn,cs}}{8.5 \cdot \left[1 - \frac{I_c}{4.6}\right]}$$

↓

Volumetric strain in design earthquake

$$\Sigma_{vol} = \varepsilon_{vol(15)} \cdot \left[\frac{N_c}{15}\right]^{0.45}$$

$$N_c = (M - 4)^{2.17}$$

↓

Seismic settlement, s

$$s = 2 \cdot \int_0^{GWT} \Sigma_{vol} \cdot dz$$

Robertson, P.K. and Lisheng, S., 2010, "Estimation of seismic compression in dry soils using the CPT" FIFTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS, Symposium in honor of professor I. M. Idriss, San Diego, CA

Liquefaction Potential Index (LPI) calculation procedure

Calculation of the Liquefaction Potential Index (LPI) is used to interpret the liquefaction assessment calculations in terms of severity over depth. The calculation procedure is based on the methology developed by Iwasaki (1982) and is adopted by AFPS.

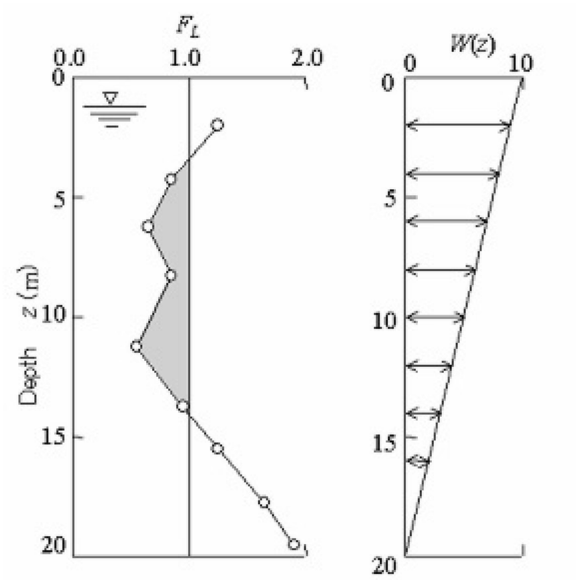
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

$$\textbf{LPI} = \int_0^{20} (10 - 0{,}5_z) \times F_I \times d_z$$

where:
F_L = 1 - F.S. when F.S. less than 1
F_L = 0 when F.S. greater than 1
z depth of measurment in meters

Values of LPI range between zero (0) when no test point is characterized as liquefiable and 100 when all points are characterized as susceptible to liquefaction. Iwasaki proposed four (4) discrete categories based on the numeric value of LPI:

- LPI = 0 : Liquefaction risk is very low
- 0 < LPI <= 5 : Liquefaction risk is low
- 5 < LPI <= 15 : Liquefaction risk is high
- LPI > 15 : Liquefaction risk is very high



Graphical presentation of the LPI calculation procedure

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