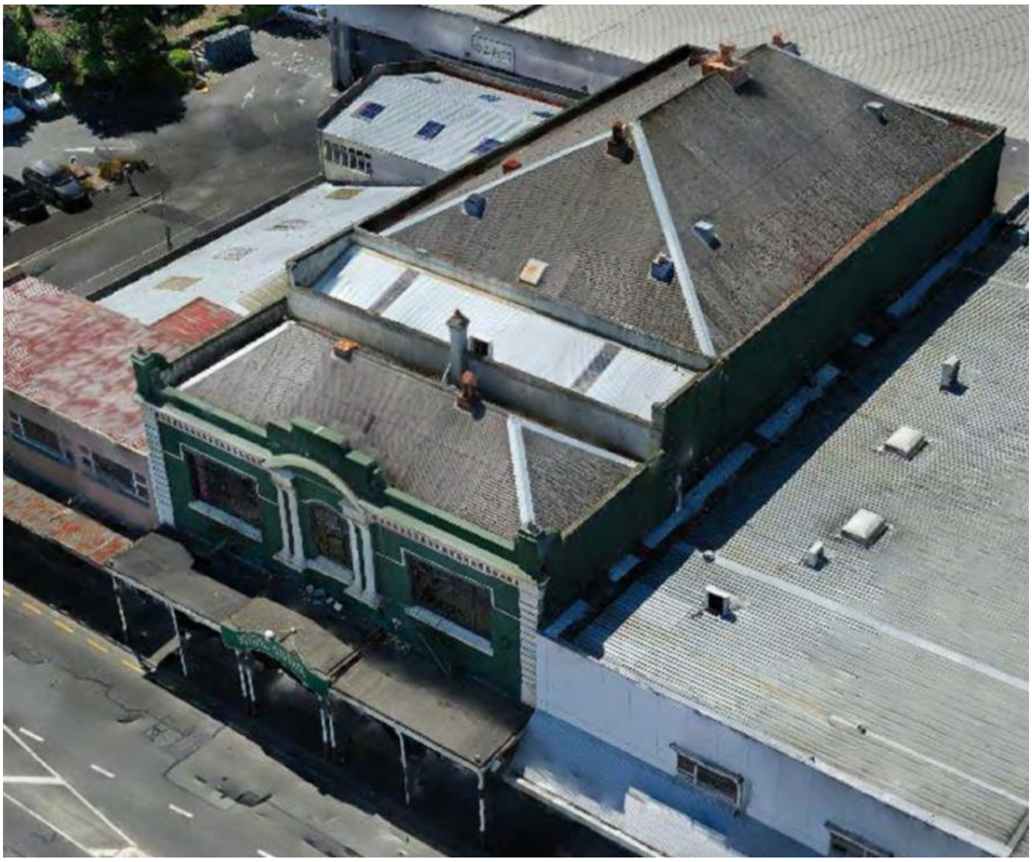
Dunedin Theatre Network

MAYFAIR THEATRE

STRUCTURAL DESIGN FEATURES REPORT PRELIMINARY DESIGN

28 NOVEMBER 2024

CONFIDENTIAL



MAYFAIR THEATRE – SEISMIC STRENGTHENING
STRUCTURAL DESIGN FEATURES REPORT

Mayfair Theatre Charitable Trust

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REV	DATE	DETAILS
2	28/11/2024	Preliminary Design

	NAME	DATE	SIGNATURE
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Approved by:	Kevin Wood	28/11/24	

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ABBREVIATIONS

DSA	Detailed Seismic Assessment
ISA	Initial Seismic Assessment
NBS	New Building Standard
IL	Importance Level
URM	Unreinforced Masonry

EXECUTIVE SUMMARY

The scope of this report covers both the Mayfair theatre building that is located at 100 King Edward Street, and the adjacent separate storage building. The strengthening scheme presented targets at least 70%NBS. To minimise assessment costs, structural assessment was carried out in the strengthened state only for elements that were obviously below 70%NBS. Other elements such as the in plane walls were assessed in both their current and strengthened condition. The more modern storage building was likely to produce a more favourable seismic rating, therefore an initial seismic assessment has been undertaken to confirm this (see Appendix 1). A geotechnical desktop report has also been undertaken to provide geotechnical inputs for the seismic assessment of the theatre (see Appendix 2).

The proposed structural strengthening design for the Mayfair Theatre consists predominantly of 3 main items. In-plane strengthening of some of the transverse walls to be achieved through steel portal frames and a concrete wall. New diaphragms in the form of a new horizontal steel truss over the theatre space (below the roof and above the ornate ceiling), plus relining of the front of house ceilings with ply. Finally, the out-of-plane capacity of the walls is to be strengthened with steel mullions.

The initial seismic assessment of the storage building results indicate a building score of 70%NBS(IL2). Detailed calculations were carried out on the brick infill to supplement the ISA. Calculations also showed that when a bay of the brick infill was removed for the new opening between the buildings, the concrete frame and infill remaining had sufficient strength.

Challenges of the site include both soft ground, and the potential for liquefaction. The preliminary assessment of the probable foundation capacity indicates that the foundations may be operating close to their critical performance state, and therefore a marginal increase in the building loading (eg. new steelwork) could induce intolerable settlements. The quantitative liquefaction assessment from nearby sites indicates that subsidence within liquefied layers would typically be in the range of 20 – 70 mm under a ULS event. Further on-site geotechnical investigation is required to confirm the strengthening scheme.

Once the on-site geotechnical investigations are completed, some refinements to the design for the next stage could be considered:

- The concrete wall could potentially be changed to a steel solution if the opening to create the new foyer can be reduced or eliminated.
- Post-tensioning of the walls could be used to increase the out-of-plane and in-plane strength of the URM walls as a low weight solution instead of the steel UC mullions.

1 GENERAL

1.1 OBJECTIVE

This Design Features Report (DFR) defines the building's structural design criteria and records key structural design decisions. It outlines design loadings, structural modelling assumptions, material properties, foundation requirements and design standards.

1.1.1 SCOPE

The scope is in accordance with the Design Brief and Conditions of Engagement.

In general terms, the scope of works is as follows:

— Design and documentation of the structural elements in the seismic strengthening to 70%NBS(IL3).

1.1.2 MEANS OF COMPLIANCE

The design has been completed on the basis that the new works associated with the strengthening have been designed to meet the requirements of B1/VM1 for ultimate limit states (ULS) design loads equal to 70% ULS (IL3), as defined in NZS 1170.5 and existing structural elements achieve at least 67%NBS(IL3) when assessed after the strengthening against the requirements of the Seismic Assessment of Existing Buildings Technical Guidelines for Engineering Assessment, July 2017.

1.1.2.1 B1 STRUCTURE

Compliance with B1 Structure is achieved through designing to the following cited standards (including any and all amendments) in Verification Method B1/VM1:

— AS/NZS 1170.0:2002, AS/NZS 1170.1:2002,

— NZS 1170.5:2004

— NZS 3101: Part 1:2006

— NZS 3404: Part 1:1997

— NZS 3603:1993

— NZS 3604:2011

1.1.2.2 B2 DURABILITY

Compliance with B2 Durability is achieved through the following means for the listed structural materials:

— Reinforced Concrete. Specified cover to reinforcing meets or exceeds the minimum requirements of NZS 3101: Part 1:2006, which is cited in Acceptable Solution B2/AS1.

— Timber treatment has been selected in accordance with Table 1A of B2/AS1.

- Structural Steel. There is no acceptable solution available for structural steel structures. Corrosion protection is provided through surface treatment in accordance with SNZ TS 3404:2018.

The durability of existing elements has been made no-worse, these elements continue to comply to the same extent as they did before the alteration in accordance with Section 112 of the Building Act.

1.1.2.3 ALTERNATIVE SOLUTIONS

No alternative solutions are proposed.

1.1.3 CHANGES TO THE SEISMIC HAZARD

A re-evaluation of the National Seismic Hazard Model (NSHM) has been carried out by MBIE, EQC and Waka Kotahi. The NSHM is the science behind the building code earthquake loadings. The results indicate that the seismic hazard has increased in many parts of New Zealand and decreased in some other areas. It will take time to confirm how the changes in hazard will be applied to structural design, and how these will be mandated. A draft technical standard TS1170.5 has been released but this has not yet been finalised. MBIE will also review whether there are any changes to the assessment framework for existing buildings, currently it is legislated that all future assessments will use the code current on the 1st July 2017 even if this subsequently changes.

Building owners may wish to consider future proofing the design and seismic performance of a new building using a site-specific hazard study. This is possible to consider in the next stage if desired when the Geotech investigations are carried out, but higher loads would significantly add to the cost of strengthening this building.

2 THE BUILDING

2.1 GENERAL DESCRIPTION

The scope of this report covers both the Mayfair theatre building that is located at 100 King Edward Street, and the nearby separate storage building. The Mayfair Theatre is an unreinforced masonry (URM) structure built on relatively flat ground with two storeys at the front street elevation and a larger auditorium at the rear. The building was originally constructed in the 1910s. The nearby 1960s storage building is structurally separate and is located at 92 King Edward Street. An ISA for the storage building has been undertaken (see Appendix 1).

2.1.1 PROJECT BACKGROUND

As the theatre was very likely to be earthquake prone, it was agreed that to minimise structural assessment costs an innovative solution-based approach where structural assessment is carried out in the strengthened state only. The risk with this approach is that unassessed items may be critical. However, this is seen as a low risk that can be managed by thorough verification of the concept.

The storage building is a concrete block structure with brick infills. As the brick is confined in a concrete frame this is likely to produce a more favourable seismic rating, therefore an initial seismic assessment has been undertaken.

2.1.2 PREVIOUS ASSESSMENT

In 2016 a detailed seismic assessment was carried out that rated the building at 80%NBS*(IL3)*. This assessment pre-dates the current assessment guidelines published in July 2017 by MBIE. Whilst a full assessment has not been completed on the unstrengthened structure in the current analysis, several elements rated less than 34%NBS(IL3) using the latest guidelines, indicating the building is potentially earthquake prone. The previous DSA also used a sophisticated analysis which relied on assumptions around the quality of the brickwork. Recent intrusive investigations on site revealed cracking in critical areas and drilling indicated gaps in the mortar between brick layers, invalidating these assumptions.

2.1.3 GRAVITY STRUCTURE

2.1.3.1 EXISTING THEATRE BUILDING

The gravity structure comprises of unreinforced masonry brick walls around the building with steel roof trusses to span the auditorium. In the front two storey section the intermediate walls are also original brick, below the seating area in the auditorium there is a mixture of concrete block and brick walls. The foundations consist of shallow concrete ground beams.

2.1.3.2 EXISTING STORAGE BUILDING

The gravity structure comprises of light weight timber rafters supported on concrete encased structural steel portal frames that are supported on reinforced concrete ground beams and shallow foundation pads. Reinforced concrete beams span between portal frames at both mid-height and roof level, with cavity brick walls infilled between these beams.

2.1.4 LATERAL LOAD RESISTING STRUCTURE

2.1.4.1 EXISTING THEATRE BUILDING

The existing lateral load resisting structure consists of URM walls as primary elements. In the two storey front of house section loads are currently distributed to the walls through lathe and plaster ceilings and the tongue and groove timber floors. Based on site investigations the brick walls are assumed to be three bricks thick on the upper floor and four bricks thick on the lower floor, with no cavity construction on both levels.

2.1.4.2 EXISTING STORAGE BUILDING

The existing lateral load resisting structure consists of concrete encased steel portal frames in lateral direction and reinforced concrete beams, infilled brick walls and concrete walls in the longitudinal direction.

2.1.5 PROPOSED STRENGTHENING AND ALTERATIONS

The strengthening and alterations for the Mayfair Theatre Building involves the following items:

- Additional concrete wall to roof level
- Structural steel portal frames
- Ply ceiling diaphragms for ground floor of reception area.
- Steel truss roof diaphragm
- Additional ground beams
- Steel mullions in auditorium
- New lift and associated lift pit
- New mezzanine floor to the storage building
- Openings between the two buildings
- Steel angles to provide gravity support to spandrels that fail prematurely under earthquake load.

The new steel frames and the additional concrete wall are to increase the in-plane capacities of the transverse walls. The concrete wall is a significant intervention and is required due to the new opening in the base of the wall to create a larger foyer. This reduces the in-plane shear capacity of the wall that has significant loading on it from half of the theatre. A steel solution may be possible if the wall can be left as is or the opening significantly reduced.

The mullions are required to strengthen the auditorium walls out-of-plane due to their tall unsupported height. They do add significant weight (potential ground issue) and cost. In the next stage it would be worth investigating the possibility of post tensioning the walls instead. This would require new foundations and a specialist contractor. Early contractor involvement would be essential for this option to work. This may also be sufficient tying to mitigate the liquefaction risk.

The steel roof diaphragms are required to distribute seismic load between different lateral load resisting elements. This diaphragm will distribute loads from the face loaded longitudinal walls back to the transverse walls.

Ply diaphragms are being retrofitted to the ceilings on both floors. The diaphragms have been designed for pESA loads for 70%NBS(IL3) with a 1.5 load factor as per NZS 1170.5 in recognition that they are secondary elements.

Due to the results from the storage building ISA, no strengthening is required, as the building overall score is 70%NBS(IL2), which exceeds the minimum threshold of 67%NBS for earthquake risk building. Calculations showed that when a bay of the brick infill was removed, the concrete frame and infill remaining had sufficient strength.

2.1.6 SAFETY BY DESIGN

Safety by Design is a key part of the design process involving all disciplines and influences how the building will be used. In the next design stage safety by design workshops are carried out regularly with the project team and a register of items is updated as a live document to address risks.

The following residual Safety by Design risks have been identified. These may be addressed in the next design stage or by the contractor.

- Excavation adjacent to existing foundations – likely to require staging.
- Working at heights – safe access is required to strengthen the parapets.

Some areas where Safety by Design has been implemented already in the structural design are identified below as examples:

- The opening to the storage building will be formed by removing all the bricks in the bay up to the concrete frame to avoid the need to temporarily support bricks above the opening.
- Steelwork has been used where possible to minimise the use of a wet trade inside the building.

3 SITE CONDITIONS

3.1 SITE DESCRIPTION

3.1.1 SOIL CONDITIONS

From recent nearby Geotech investigations a site subsoil category of 'D-Deep or soft soil sites' (in accordance with NZS1170.5) has been assumed.

3.1.2 SUMMARY OF SOIL CONDITION

The preliminary assessment of the probable foundation capacity indicates that the foundations may be operating close to their critical performance state. This means that a marginal increase in the building loading could induce intolerable settlements to the building. The quantitative liquefaction assessment indicates that subsidence within liquefied layers would typically be in the range of 20 – 70 mm under a ULS event. The available investigation data infers that there is a non-liquefiable crust which may reduce the surface manifestation of this subsidence to negligible levels. Additional on-site investigations will be required to expose the existing foundations and underlying soils, and further testing such as Cone Penetration Tests (CPT's) and dissipation tests are needed to quantify the liquefaction risk and further assess the settlement risk.

3.1.3 SOIL DESIGN VALUES

The soil design values listed in Table 3-1 have been adopted for this building, as outlined in the geotechnical engineering report and Verification Method B1/VM4.

Table 3 1: Soil design parameters.

Parameter	Value
ultimate bearing capacity (pad footings), q_u	60kPa
ultimate bearing capacity (strip footings), q_u	60kPa
shallow foundation bearing strength reduction factor (gravity loading), ϕ	0.5
soil unit weight for soil pressures	$18kN.m^{-3}$

4 MATERIAL PROPERTIES

4.1 REINFORCED CONCRETE

Table 4-1 lists the 28-day concrete strengths specified for this project.

Table 4-1: Specified 28-day concrete strengths.

Element	Specified 28-day strength, f'_c
Foundation Beams	30 MPa
Concrete wall	30 MPa

Table 4-2 lists the reinforcing steel properties used for this project. All reinforcing used in this project is ductility class E to AS/NZS 4671:2001.

Table 4-2: Reinforcing steel properties.

Designation	Deformation	Characteristic Strength, fy	Characteristic Elongation
D	Deformed	300MPa	15%
HD	Deformed	500MPa	10%
R	Round	300MPa	15%

4.1.1 *STRUCTURAL STEEL*

Structural steel elements have the properties shown in Table 4-3. All steel elements have a Young's Modulus of 200 GPa.

Table 4-3: Structural steel strengths.

<table>
<tr><th>Element Type</th><th>Manufacturing Standard</th><th>Grade</th><th>Characteristic Strength, f_y</th><th>Tensile Strength, f_u</th></tr>
<tr><td>Hot-rolled I-sections; t < 11mm.</td><td rowspan="3">AS/NZS 3679.1:2016</td><td rowspan="3">300 SO</td><td>320MPa</td><td rowspan="3">440MPa</td></tr>
<tr><td>Hot-rolled I-sections; 11mm ≤ t < 17mm.</td><td>300MPa</td></tr>
<tr><td>Hot-rolled I-sections; 17mm < t.</td><td>280MPa</td></tr>
<tr><td>Steel plate</td><td>AS/NZS 3678:2011</td><td>C350</td><td>350MPa</td><td>410MPa</td></tr>
<tr><td>Bolts</td><td>AS 1111</td><td>8.8</td><td></td><td></td></tr>
<tr><td>Hot-rolled SHS</td><td>AS 1163</td><td>C450PLUS</td><td>450MPa</td><td>500MPa</td></tr>
<tr><td>Hot-rolled Equal Angles</td><td>AS/NZS 3679.1:2016</td><td>C300PLUS</td><td>320MPa</td><td>440MPa</td></tr>
</table>

4.1.2 URM

The following properties have been assumed for the existing brick.

Table 4-4: Probable strength parameters for URM.

Item	Hardness	Description	Probable compressive Strength (MPa)	Probable tensile Strength, f_{bt}	Probable Cohesion, c (MPa)	Probable coefficient of Friction, μ_f
Brick	Medium	Scratches with copper coin	26MPa	3.1MPa	-	-
Mortar	Medium	Scratches with fingernails	5MPa	-	0.5	0.6

5 DESIGN LOADS

For the purposes of loading, this structure is Importance Level 3 in accordance with AS/NZS 1170.0:2002.

5.1.1 PERMANENT LOADS

Permanent loads from the self-weight of the structure and a superimposed dead load (SDL) have been considered. Structural self-weights are based on the material weights listed in Table 5-1. Permanent loads listed in Table 5-1 were considered in the design of each level of the structure.

Table 5-1: Material unit weights.

Material	Unit Weight
Reinforced Concrete (In-Situ and Precast)	24kN.m^{-3}
Structural Steel	78kN.m^{-3}
Timber	5.0kN.m^{-3}
Reinforced Block Masonry (all cells filled)	22kN.m^{-3}
URM - Brick	18kN.m^{-3}

5.1.2 IMPOSED LOADS

5.1.2.1 FLOORS AND ROOFS

Design floor imposed loads and roof imposed loads meet the minimum magnitudes from Table 3.1 and Table 3.2, respectively, of AS/NZS 1170.1:2002. The imposed loads considered for this project are shown in Table 5-2.

Table 5-2: Imposed loads.

Level	NZS 1170.1 Occupancy Category	Uniformly Distributed Action	Concentrated Action
All	C5 Areas susceptible to overcrowding	5 kPa	3.6kN

5.1.3 WIND LOADS

No wind assessments or design have been carried out. The structure is no-worse in terms of wind performance, and continues to comply to at least the extent it did before the alteration in accordance with Section 112 of the Building Act.

5.2 SEISMIC LOADS

5.2.1 SITE PARAMETERS

Site category D, in terms of NZS 1170.5:2004 definitions, has been assumed for this project.

The building is located further than 20km from a major fault line.

5.2.2 ANALYSIS METHODOLOGY

The seismic analysis has been completed in accordance with AS/NZS 1170.5:2002, using the equivalent static method.

Design Spectra are in accordance with AS/NZS 1170.5:2002 for the site subsoil class.

5.2.3 SEISMIC COEFFICIENT

The seismic coefficient has been determined in accordance with NZS 1170.5:2004.

The parameters in Table 5-3 have been determined for this building.

Table 5-3: Site spectrum seismic design parameters.

Parameter	*x* direction		*y* direction	
	ULS (1/1000)	SLS1 (1/25)	ULS (1/1000)	SLS1 (1/25)
T	<0.5s		<0.5s	
$C_h(T)$	3		3	
Z	0.13			
R	1.3	0.25	1.3	0.25
N(T,D)	1.0		1.0	
$C(T) = C_h(T)ZRN(T,D)$	0.507	0.098	0.507	0.098

The building has been designed for an ultimate limit state ductility demand of 1 in the *x* and the *y* direction.

Based on the chosen design ductility demands and structural performance factors, the horizontal design action coefficients in Table 5-4 have been determined for this building.

Table 5-4: Horizontal design action coefficient.

Parameter	*x* direction		*y* direction	
	ULS (1/500)	SLS (1/25)	ULS (1/500)	SLS (1/25)
μ	1	1	1	1
k_μ	1	1	1	1
S_p	1	1	1	1
$C_d(T) = C(T)S_p/k_\mu$	0.507	0.098	0.507	0.098

5.2.4 POST FIRE STABILITY

Currently no structural elements have been designed or assessed for inherent fire resistance. Fire ratings to be achieved by passive fire protections.

6 DEFLECTION CRITERIA

6.1 SEISMIC DEFLECTIONS AND DRIFTS

Peak seismic deflections and drifts have been determined in accordance with NZS 1170.5:2004 clauses 7.2 and 7.3, respectively.

Drifts limit of height/40 (2.5%) have been imposed for the ultimate limit state. The ULS limit is based on clause 7.5.1 of NZS 1170.5:2004. Where the protection of URM is required, drifts have been limited to 1.1% or 0.003% times the height divided by the length of the worst case pier.

The deflections of elements supporting out-of-plane members have been designed to the maximum usable deflection of the URM walls as defined in C8.

The mullions have been sized based on strength but were checked for deflection which indicated a deflection in the order of 50mm which was considered acceptable given the wall thickness.

6.1.1 GRAVITY DEFLECTIONS

Table 6-1 shows the deflection limits for the listed steel and timber elements under serviceability gravity loading.

Concrete elements are proportioned to the minimum thickness requirements of NZS 3101:2006 Table 2.1 to satisfy the serviceability limit state.

Table 6-1: SLS deflection limit

Load Case	SLS Deflection limit
$G+Q_L$	L/300
$G+Q_S$	L/300
1kN point load	0.5mm

7 DURABILITY PROVISIONS

The design life for durability for the foundations and superstructure is 50 years.

7.1 REINFORCED CONCRETE ELEMENTS

Reinforced concrete elements satisfy the performance requirements of B2 Durability through achieving minimum covers of Section 3 of NZS 3101:2006, which is cited in acceptable solution B2/AS1. The minimum covers of Table 6-1 are required for this building.

Table 7-1: Minimum covers for durability.

Element	Exposure Classification	28 Concrete Strength, f'_c	Minimum Cover(s)
Foundation Beams	A1/A2	30MPa	75mm
Concrete wall	A1/A2	30MPa	30mm

7.1.1 TIMBER ELEMENTS

Timber treatment is to be as per NZS3602:2003

7.1.2 STRUCTURAL STEEL ELEMENTS

Structural steel elements have surface treatments specified to protect against corrosion.

This building's location places Macroclimate Corrosion category C3, as per Figures 1-7 of SNZ TS 3404:2018. Table 7-2 summarises the Surface-specific atmospheric corrosivity of the various steel elements (as defined in Table 2 of SNZ TS 3404:2018) and the proposed coating system for the required time to first major maintenance.

Table 7-2: Proposed Steel Coatings

Element	Exposure	Surface-specific Atmospheric Corrosivity	Time to first major maintenance	Proposed Coating System
Structural steel portal frames	Damp	C3	15 years	Resene Zincilate 11, Armourcote 510, Uracryl 403
Structural steel roof diaphragm	Damp	C3	15 years	Resene Zincilate 11, Armourcote 510, Uracryl 403
Structural steel mullions	Damp	C3	15 years	Resene Zincilate 11, Armourcote 510, Uracryl 403

8 CAPACITIES PRIOR TO STRENGTHENING

The items listed in Table 8-1 show the elements capacities before strengthening and give insight into why the strengthening is required in these locations.

Table 8-1: Element %NBS prior to strengthening

Element	%NBS without strengthening	Comment
In-plane capacity transverse wall 1	25%	Governed by pier failure
In-plane capacity transverse wall 2	85% NBS	
In-plane capacity transverse wall 3	15% NBS with spandrel failure	This %NBS is dictated by the spandrels, if the new opening in this wall was to be lower than 4m this could potentially be improved.
In-plane capacity transverse wall 4	>100% NBS	Rocking failure mode unlikely due to return walls, diagonal tensile failure
Out-of-plane capacity longitudinal walls along sides of theatre	25% NBS	

9 RISKS AND UNKNOWNS

The items listed in Table 8-1 have been identified as potential design risks.

Table 9-1: Design risks and unknowns.

Risk	Status
Soil properties do not match those used for structural design.	Structural design has utilised the soil properties identified in the project geotechnical engineering report which is based on a desktop study from nearby investigations. Given the Geotech study has identified settlement and liquefaction risks, investigations should be carried out before proceeding with any further design work
Existing structure not in line with design assumptions.	Intrusive Investigations have been carried out in places, but until linings are fully removed there is potential for the structure to be different. Assumptions should be verified on site during construction.
Existing elements being weaker than assumed Eg. through deterioration.	To be reviewed during construction once existing linings have been removed.
Temporary instability.	Contractor to provide suitable propping designed by a CPEng Engineer that is well secured.
Liquefaction	WSP recommends further investigations into the risk of liquefaction and potential engineering measures to make the structure more resilient to damage from liquefaction.
Soft ground causing excessive settlement of new elements	Excessive settlement can increase loads on effected elements, and may impact the design assumptions used.
Existing foundation layout and dimensions	The unknown dimensions and layout will need to be confirmed by on-site investigations prior to developed/detailed design.

10 REFINEMENT FOR NEXT DESIGN STAGE

The items listed in Table 8-1 have been identified as areas of potential refinement for the next design stage.

Table 10-1: Design risks and unknowns.

Refinement	Status
On site Geotech investigations	Essential to confirm the design before proceeding with further design work. Adverse ground conditions may add significant additional cost.
Steel chords of roof diaphragm	Currently designed to a conservative loading scenario for preliminary design, so less conservative design forces could be used to result in smaller section sizes. This may also make it possible to use some of the existing members. More investigations will be required to confirm sizes as the original truss drawings were not available.
Opening in theatre rear wall	It may be possible to use a steel solution to strengthen this wall instead of the concrete if the opening to create the new foyer can be reduced or eliminated.
Post-tensioning of all the walls	Further to more geotechnical and foundation investigations, using post-tensioning to increase the out-of-plane and in-plane strength of the URM walls could be a potential low-weight solution to increase the strength of the structure.

11 LIMITATIONS

This report ('Report') has been prepared by WSP New Zealand Limited ('WSP') exclusively for Mayfair Theatre Charitable Trust ('Client') in relation to Seismic Strengthening of the Mayfair Theatre ('Purpose') and in accordance with the contract and variations to it ('Agreement'). The findings in this Report are based on and are subject to the assumptions specified in the Report, shown on the drawings and as outline in the contract. WSP accepts no liability whatsoever for any use or reliance on this Report, in whole or in part, for any purpose other than the Purpose or for any use or reliance on this Report by any third party.

In preparing this Report, WSP has relied upon data, surveys, analyses, designs, plans and other information ('Client Data') provided by or on behalf of the Client. Except as otherwise stated in this Report, WSP has not verified the accuracy or completeness of the Client Data. To the extent that the statements, opinions, facts, information, conclusions and/or recommendations in this Report are based in whole or part on the Client Data, those conclusions are contingent upon the accuracy and completeness of the Client Data. WSP will not be liable for any incorrect conclusions or findings in the Report should any Client Data be incorrect or have been concealed, withheld, misrepresented or otherwise not fully disclosed to WSP.

APPENDIX 1: STORAGE BUILDING ISA

Project Number: 6-CB170.00

Storage Building

Initial Seismic Assessment

13 November 2024

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0	13 November 2024	Shanker Thanapalasingam	Simon Burrough	Kevin Wood	Draft for client comment

Contents

Executive Summary

WSP was asked by the Mayfair Trust to carry out an Initial Seismic Assessment (ISA) of the storage building at 92 King Edward Street, Dunedin.

A site visit was made to the property on 25th September 2024 for the purpose of a visual inspection. The visual inspection showed that the property appears to be constructed in accordance with the drawings. A set of original 1964 Structural plans and sections and details was available.

The Building is a single storey storage building, consists of concrete encased structural steel portal frames with brick masonry infill and light weight roof. Supplementary calculations have been carried out on the brick infill to confirm the rating.

The building has been considered as importance level 2 and the site subsoil class as D – Deep or soft soil. The ISA rates the building at 70%NBS(IL2), which corresponds to a Grade B building, as defined by the NZSEE building grading scheme.

1. Building Information	
Building Name/ Description	Storage Building
Street Address	92 King Edward Street, South Dunedin
Territorial Authority	Dunedin City Council
No. of Storeys	One
Area of Typical Floor (approx.)	168.3m^2
Year of Design (approx.)	1964
NZ Standards designed to	NZSS 95:1955
Structural System including Foundations	Single storey structural steel portal frames encased in concrete with brick masonry infill and light weight roof supported on shallow foundation pads.
Does the building comprise a shared structural form or shares structural elements with any other adjacent titles?	Yes, the building at the southwest shares structural elements with this building.
Key features of ground profile and identified geohazards	Subsoil Class D. Susceptible to liquefaction under large earthquake.
Previous strengthening and/ or significant alteration	None known
Heritage Issues/ Status	No

Other Relevant Information	

2. Assessment Information	
Consulting Practice	WSP New Zealand Limited
CPEng Responsible, including: • Name • CPEng number • A statement of suitable skills and experience in the seismic assessment of existing buildings	Simon Burrough CPEng No. 248690 20+ years of Structural Design and Seismic assessment experience from high rises to single storey structures.
Documentation reviewed, including: • date/ version of drawings/ calculations • previous seismic assessments	Original consent drawings from the Dunedin City Council.
Geotechnical Report(s)	WSP Geotechnical Desktop Report.
Date(s) Building Inspected and extent of inspection	Visual Inspection on 25 September 2024.
Description of any structural testing undertaken and results summary	None.
Previous Assessment Reports	None
Other Relevant Information	An assessment of the brick infill's out-of-plane capacity shows that it is 100% of the required capacity.

3. Summary of Engineering Assessment Methodology and Key Parameters Used	
Occupancy Type(s) and Importance Level	Storage Building. Importance Level 2
Site Subsoil Class	Class D Soil to AS/NZS 1170.5 (Deep or soft soil)
For ISA	

Summary of how Part B was applied, including: • Key parameters such as μ, Sp and F factors • Any supplementary specific calculations	The building consists of concrete encased structural steel portal frames supported on shallow foundation pads. Ductility μ = 1.25 in the longitudinal direction and μ = 2 in the transverse direction Sp = 0.925

4. Assessment Outcomes		
Assessment Status (Draft or Final)	Final	
Assessed *%NBS* Rating	70%	
Seismic Grade and Relative Risk (from Table A3.1)	Grade B and low or medium risk	
<u>For an ISA:</u>		
Describe the Potential Critical Structural Weaknesses	No potential structural weaknesses were identified in this building.	
Does the result reflect the building's expected behaviour, or is more information/ analysis required?	No	
If the results of this ISA are being used for earthquake prone decision purposes, <u>and</u> elements rating <34*%NBS* have been identified:	Engineering Statement of Structural Weaknesses and Location N/A	*Mode of Failure and Physical Consequence* Statement(s) N/A

Introduction

WSP was asked by Mayfair Trust to make an Initial Seismic Assessment of the storage building outlined in red below at 92 King Edward Street, Dunedin. The corner building to the west has not been assessed.

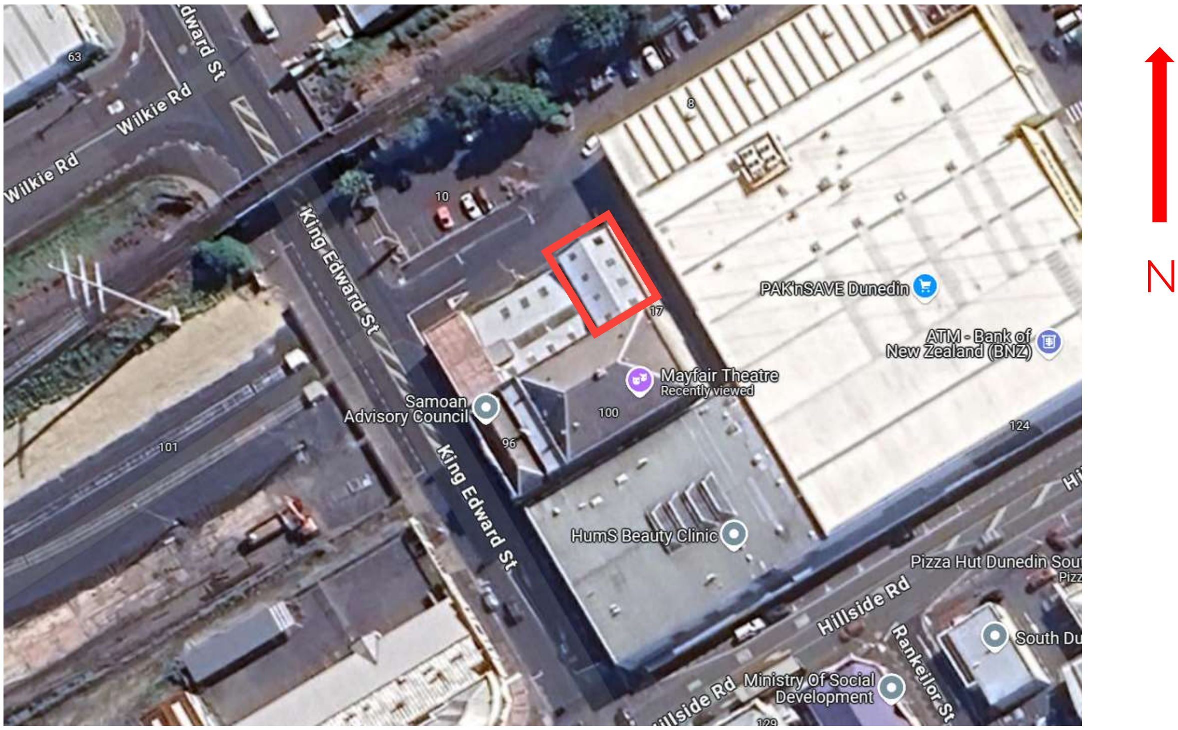


Figure 1: Aerial view of the building source: https://www.google.com/maps

Basis for the assessment

The information we have used for this IEP assessment includes:

- Site visit – interior and exterior walk through;
- Review of existing drawings from Dunedin City Council;
- The IEP spreadsheet tool as described in Part B of the guideline document, The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments, dated July 2017

Building Description

The building consists of light weight roof and timber rafters supported on concrete encased structural steel portal frames that are supported on reinforced concrete ground beams and shallow foundation pads. Reinforced concrete beams span between portal frames at both mid-height and roof level, with cavity brick walls infilled between these beams.

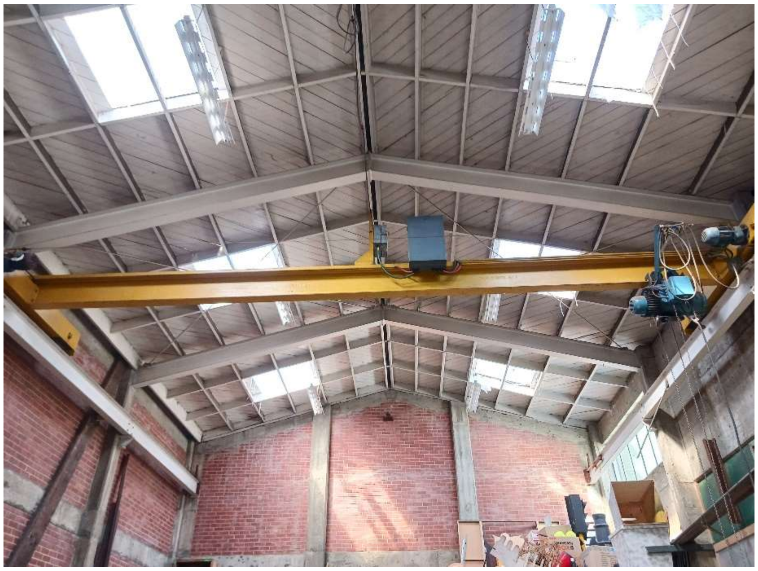


Figure 2.

Overview of the building from inside. Concrete encased portal frames

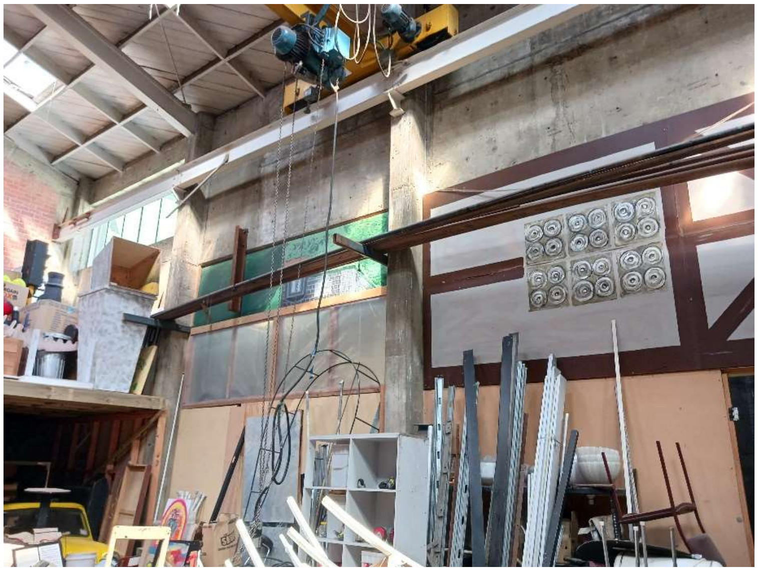


Figure 3

Concrete wall between the portal frames at the south-west end

Figure 4

Roof braces between portals fixed to the top flanges

Figure 5

Reinforced concrete beams between portals at mid-height and roof level, cavity brick wall infilled between these beams

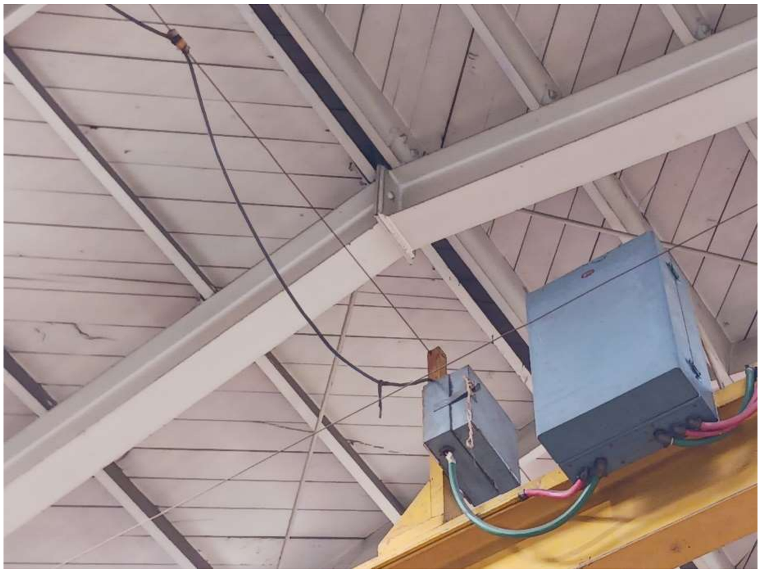


Figure 6

Portal frames connection at the ridge and timber rafters to portal connection

Figure 7

No gap between the external brick walls of this building and the adjacent building

IEP Assessment Results

The key assumptions made during our assessment are shown in Table 2 above. Refer also to the attached IEP assessment in Appendix B.

IEP Grades and Relative Risk

Table 4 taken from the Technical Guidelines, referred to earlier, provides the basis of a proposed grading system for existing buildings, as one way of interpreting the %NBS building score. Occupants in Earthquake Prone buildings (less than 34%NBS) are exposed to more than 10 times the risk that they would be in a similar new building. Broad descriptions of the life-safety risk can be assigned to the building grades as shown in Table 2.

Table 4: Relative Earthquake Risk

Building Grade	Percentage of New Building Strength (%NBS)	Approx. Risk Relative to a New Building	Life Safety Risk Description
A+	>100	<1	low risk
A	80 to 100	1 to 2 times	low risk
B	67 to 79	2 to 5 times	low or medium risk
C	34 to 66	5 to 10 times	medium risk
D	20 to 33	10 to 25 times	high risk
E	<20	more than 25 times	very high risk

This building has been classified by the IEP as a Grade B building and is therefore considered to be a low or medium risk.

The New Zealand Society for Earthquake Engineering (which provides authoritative advice to MBIE and should be considered to represent the consensus view of New Zealand structural engineers) classifies a building achieving greater than 67%NBS as "low or medium risk" and having "acceptable" building structural performance (improvement may be desirable).

Geotechnical Considerations

A geotechnical desktop study has been carried out as part of this assessment.

Based on information from local geological maps and our knowledge of the area the site is likely to classified (in terms of NZS 1170.5:2004) as Class D- Deep or soft Soil.

Result of the Assessment

The ISA assessment indicates an overall score of 70%NBS (IL2). This corresponds to a Grade B building, as defined by the NZSEE building grading scheme. The building was assumed to be Importance Level 2 in accordance with NZS1170.5.

The seismic performance of the building is considered to be a low or medium Earthquake Risk.

Table 5: Summary of Results from IEP Spreadsheet

Structural Component	Result %NBS	Comments
Overall Structure	70% IL2	

Conclusions

The ISA assessment for this building, carried out using the IEP method indicates an overall score of 70%NBS (IL2) which corresponds to a Grade B building, as defined by the NZSEE building grading scheme. The storage building at 92 King Edward Street, Dunedin is therefore considered low or medium Earthquake Risk.

ISAs are an initial look at a building's seismic strength, they tend to be conservative but can also underestimate critical structural weaknesses. If a more accurate result is required we recommend a Detailed Seismic Assessment for this building.

Limitations

This report ('Report') has been prepared by WSP exclusively for the Mayfair Theatre Charitable Trust ('Client') in relation to the Initial Seismic Assessment of the Mayfair Theatre storage building ('Purpose') and in accordance with the Short Form Agreement with the Client dated 17/07/2024. The findings in this Report are based on and are subject to the assumptions specified in the Report and Offer of Service dated 17/07/2024. WSP accepts no liability whatsoever for any reliance on or use of this Report, in whole or in part, for any use or purpose other than the Purpose or any use or reliance on the Report by any third party.
In preparing this Report, WSP has relied upon data, surveys, analyses, designs, plans and other information ('Client Data') provided by or on behalf of the Client. Except as otherwise stated in this Report, WSP has not verified the accuracy or completeness of the Client Data. To the extent that the statements, opinions, facts, information, conclusions and/or recommendations in this Report are based in whole or part on the Client Data, those conclusions are contingent upon the accuracy and completeness of the Client Data. WSP will not be liable for any incorrect conclusions or findings in the Report should any Client Data be incorrect or have been concealed, withheld, misrepresented or otherwise not fully disclosed to WSP.

Appendix A

Background to the IEP and its Limitations

The IEP procedure was developed in 2006 by the New Zealand Society for Earthquake Engineering (NZSEE) and updated in 2013 to reflect experience with its application and as a result of experience in the Canterbury earthquakes. It was always intended to be a screening tool for use by Engineers only to make an assessment as to whether a building required further investigation. This was done by estimating a figure of %NBS – the percentage of seismic capacity of the building compared to a new building built in compliance with the current New Zealand Building Code, on the same site. Since its original conception, it has come into much more common use as a useful measure for owners, tenants, territorial authorities and others. However, it is important to remember that this is only a basic estimate of the building inferred from the information available at the time of assessment. The IEP enables territorial authorities, building owners and managers to review their building stock as part of an overall risk management process.

Characteristics and limitations of the IEP include:

- It may be somewhat conservative, identifying some buildings as earthquake prone, or having a lower %NBS score, which subsequent detailed investigation may indicate is less than actual performance. However, there will be exceptions, particularly when Critical Structural Weaknesses (CSW) are present that have not been recognised from the level of investigation employed.

- It can be undertaken with variable levels of available information, e.g. exterior only inspection, structural drawings available or not, interior inspection, etc. The more information available the more representative the IEP result is likely to be. The IEP records the information that has formed the basis of the assessment and consideration of this is important when determining the likely reliability of the result.

- It is an initial, first-stage review. Buildings or specific issues which the IEP process flags as being problematic or as potentially critical structural weaknesses, need further detailed investigation and evaluation. A Detailed Seismic Assessment is recommended if the seismic status of a building is critical to any decision making.

- The IEP assumes that the buildings have been designed and built in accordance with the building standard and good practice current at the time it was constructed.

- It is a largely qualitative process and should be undertaken or overseen by an experienced Chartered engineer. It involves considerable knowledge of the earthquake behaviour of buildings, and judgement as to key attributes and their effect on building performance. Consequently, it is possible that the %NBS derived for a building by independent experienced engineers may differ.

- An IEP does not consider the seismic performance of non-structural items such as ceiling, plant, services or glazing.

Experience to date shows that the IEP is a useful tool to identify potential issues and expected overall performance of a building in an earthquake. However, the process and the associated %NBS and grade should be considered as only indicative of the building's compliance with current code requirements. A detailed investigation and analysis of the building will typically be required to provide a definitive assessment.

Appendix B

IEP Spreadsheets

Initial Evaluation Procedure (IEP) Assessment - Completed for {Client/TA}

Street Number & Name:	**92 King Edward Street**	**Job No.:**	**6-CB170.00**
AKA:		**By:**	**ST**
Name of building:		**Date:**	**10/10/2024**
City:	**Dunedin**	**Revision No.:**	**1**

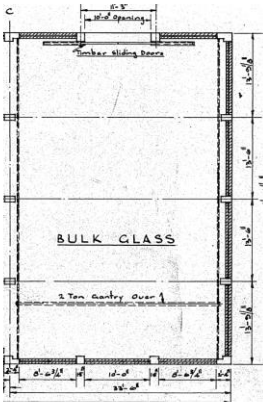
Table IEP-1 Initial Evaluation Procedure Step 1

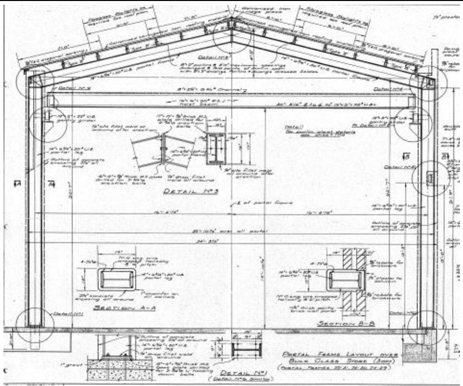
Step 1 - General Information

1.1 Photos (attach sufficient to describe building)

NOTE: THERE ARE MORE PHOTOS ON PAGE 1a ATTACHED

1.2 Sketches (plans etc, show items of interest)





NOTE: THERE ARE MORE SKETCHES ON PAGE 1a ATTACHED

1.3 List relevant features (Note: only 10 lines of text will print in this box. If further text required use Page 1a)

Single storey storage building features double height space and is constructed with structural steel portal frames encased in concrete. It is supported on reinforced concrete shallow footings and has a lightweight roof. Property files and drawings from the Dunedin City Council, dated July 1964, have been reviewed and indicate that the building's primary structural elements and load paths remain consistent with those outlined in the original drawings.

1.4 Note information sources Tick as appropriate

Visual Inspection of Exterior	☑	**Specifications**	☐
Visual Inspection of Interior	☑	**Geotechnical Reports**	☐
Drawings (note type)	☑	**Other (list)**	☐

Existing structural and architectural building layout plans.

Initial Evaluation Procedure (IEP) Assessment - Completed for {Client/TA} Page 2

Street Number & Name:	92 King Edward Street	Job No.:	6-CB170.00
AKA:		By:	ST
Name of building:		Date:	10/10/2024
City:	Dunedin	Revision No.:	1

Table IEP-2 Initial Evaluation Procedure Step 2

Step 2 - Determination of *(%NBS)* $_b$

(Baseline *(%NBS)* for particular building - refer Section B5)

2.1 Determine nominal *(%NBS)* = *(%NBS)* $_{nom}$

	Longitudinal	Transverse
a) Building Strengthening Data		
Tick if building is known to have been strengthened in this direction	☐	☐
If strengthened, enter percentage of code the building has been strengthened to	N/A	N/A
b) Year of Design/Strengthening, Building Type and Seismic Zone		
Pre 1935	○	○
1935-1965	◉	◉
1965-1976	○	○
1976-1984	○	○
1984-1992	○	○
1992-2004	○	○
2004-2011	○	○
Post Aug 2011	○	○
Building Type:	Others	Others
Seismic Zone:	Not applicable	Not applicable
c) Soil Type		
From NZS1170.5:2004, Cl 3.1.3 :	D Soft Soil	D Soft Soil
From NZS4203:1992, Cl 4.6.2.2 : (for 1992 to 2004 and only if known)	Not applicable	Not applicable
d) Estimate Period, *T*		
Comment: h_n =	8.5	8.5 m
A_c =	1.00	1.00 m^2
Moment Resisting Concrete Frames: $T = \max\{0.09h_n^{0.75}, 0.4\}$	○	○
Moment Resisting Steel Frames: $T = \max\{0.14h_n^{0.75}, 0.4\}$	◉	◉
Eccentrically Braced Steel Frames: $T = \max\{0.08h_n^{0.75}, 0.4\}$	○	○
All Other Frame Structures: $T = \max\{0.06h_n^{0.75}, 0.4\}$	○	○
Concrete Shear Walls $T = \max\{0.09h_n^{0.75}/A_c^{0.5}, 0.4\}$	○	○
Masonry Shear Walls: $T \le 0.4$sec	○	○
User Defined (input Period):	○	○
T:	0.70	0.70
e) Factor A: Strengthening factor determined using result from (a) above (set to 1.0 if not strengthened) — Factor A:	1.00	1.00
f) Factor B: Determined from NZSEE Guidelines Figure 3A.1 using results (a) to (e) above — Factor B:	0.03	0.03
g) Factor C: For reinforced concrete buildings designed between 1976-84 Factor C = 1.2, otherwise take as 1.0. — Factor C:	1.00	1.00
h) Factor D: For buildings designed prior to 1935 Factor D = 0.8 except for Wellington and Napier (1931-1935) where Factor D may be taken as 1.0, otherwise take as 1.0. — Factor D:	1.00	1.00
(%NBS) $_{nom}$ **= AxBxCxD** — *(%NBS)* $_{nom}$	3%	3%

Where h_n = height in metres from the base of the structure to the uppermost seismic weight or mass.

WARNING!! *This initial evaluation has been carried out solely as an initial seismic assessment of the building following the procedure set out in "The Seismic Assessment of Existing Buildings" Technical Guidelines for Engineering Assessments, July 2017. This spreadsheet must be read in conjunction with the limitations set out in the accompanying report, and should not be relied on by any party for any other purpose. Detailed inspections and engineering calculations, or engineering judgements based on them, have not been undertaken, and these may lead to a different result or seismic grade.*

Initial Evaluation Procedure (IEP) Assessment - Completed for {Client/TA} Page 3

Street Number & Name:	92 King Edward Street	Job No.:	6-CB170.00
AKA:		By:	ST
Name of building:		Date:	10/10/2024
City:	Dunedin	Revision No.:	1

Table IEP-2 Initial Evaluation Procedure Step 2 continued

2.2 Near Fault Scaling Factor, Factor E

If $T \leq$ 1.5sec, Factor E = 1

		Longitudinal	Transverse
a) Near Fault Factor, *N(T,D)* (from NZS1170.5:2004, Cl 3.1.6)	N(T,D):	1	1
b) Factor E = 1/N(T,D)	Factor E:	1.00	1.00

2.3 Hazard Scaling Factor, Factor F

a) Hazard Factor, *Z*, for site

Location: Dunedin — Refer right for user-defined locations

Z = 0.13 (from NZS1170.5:2004, Table 3.3)

Z_{1992} = 0.6 (NZS4203:1992 Zone Factor from accompanying Figure 3.5(b))

Z_{2004} = 0.13 (from NZS1170.5:2004, Table 3.3)

b) Factor F

For pre 1992 = $1/Z$

For 1992-2011 = Z_{1992}/Z

For post 2011 = Z_{2004}/Z

	Longitudinal	Transverse
Factor F:	7.69	7.69

2.4 Return Period Scaling Factor, Factor G

a) Design Importance Level, I

(Set to 1 if not known. For buildings designed prior to 1965 and known to be designed as a public building set to 1.25. For buildings designed 1965-1976 and known to be designed as a public building set to 1.33 for Zone A or 1.2 for Zone B. For 1976-1984 set I value.)

b) Design Risk Factor, R_o

(set to 1.0 if other than 1976-2004, or not known)

c) Return Period Factor, R

(from NZS1170.0:2004 Building Importance Level) *Choose Importance Level*

d) Factor G = IR_o/R

	Longitudinal	Transverse
I =	1	1
R_o =	1	1
Importance Level	○ 1 ◉ 2 ○ 3 ○ 4	○ 1 ◉ 2 ○ 3 ○ 4
R =	1.0	1.0
Factor G:	1.00	1.00

2.5 Ductility Scaling Factor, Factor H

a) Available Displacement Ductility Within Existing Structure

Comment:

Structural steel portal frames encased in concrete in transverse direction.

b) Factor H

		Longitudinal	Transverse
μ =		1.25	2.00
		k_μ	k_μ
For pre 1976 (maximum of 2)	=	1.25	2.00
For 1976 onwards	=	1	1
Factor H:		1.25	2.00

(where k_μ is NZS1170.5:2004 Inelastic Spectrum Scaling Factor, from accompanying Table 3.3)

2.6 Structural Performance Scaling Factor, Factor I

a) Structural Performance Factor, S_p

(from accompanying Figure 3.4)

Tick if light timber-framed construction in this direction

b) Structural Performance Scaling Factor = $1/S_p$

	Longitudinal	Transverse
Tick if light timber-framed	☐	☐
S_p =	0.93	0.70
Factor I:	1.08	1.43

Note Factor B values for 1992 to 2004 have been multiplied by 0.67 to account for Sp in this period

2.7 Baseline *%NBS* for Building, $(\%NBS)_b$

(equals $(\%NBS)_{nom}$ x E x F x G x H x I)

	Longitudinal	Transverse
$(\%NBS)_b$	35%	74%

Initial Evaluation Procedure (IEP) Assessment - Completed for {Client/TA} Page 4

Street Number & Name:	92 King Edward Street	Job No.:	6-CB170.00
AKA:		By:	ST
Name of building:		Date:	10/10/2024
City:	Dunedin	Revision No.:	1

Table IEP-3 Initial Evaluation Procedure Step 3

Step 3 - Assessment of Performance Achievement Ratio (PAR)

(Refer Appendix B - Section B3.2)

a) Longitudinal Direction

potential CSWs | **Effect on Structural Performance** (Choose a value - Do not interpolate) | **Factors**

3.1 Plan Irregularity

Effect on Structural Performance ○ *Severe* ○ *Significant* ◉ *Insignificant* **Factor A** 1.0

Eccentrically braced concrete encased steel frames

3.2 Vertical Irregularity

Effect on Structural Performance ○ *Severe* ○ *Significant* ◉ *Insignificant* **Factor B** 1.0

High portal columns

3.3 Short Columns

Effect on Structural Performance ○ *Severe* ○ *Significant* ◉ *Insignificant* **Factor C** 1.0

No short columns

3.4 Pounding Potential

(Estimate D1 and D2 and set D = the lower of the two, or 1.0 if no potential for pounding, or consequences are considered to be minimal)

a) Factor D1: - Pounding Effect

Note:
Values given assume the building has a frame structure. For stiff buildings (eg shear walls), the effect of pounding may be reduced by taking the coefficient to the right of the value applicable to frame buildings.

Factor D1 For Longitudinal Direction: 1.0

Table for Selection of Factor D1	Severe	Significant	Insignificant
Separation	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Alignment of Floors within 20% of Storey Height	○ 1	○ 1	◉ 1
Alignment of Floors not within 20% of Storey Height	○ 0.4	○ 0.7	○ 0.8

b) Factor D2: - Height Difference Effect

Factor D2 For Longitudinal Direction: 1.0

Table for Selection of Factor D2	Severe	Significant	Insignificant
	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Height Difference > 4 Storeys	○ 0.4	○ 0.7	○ 1
Height Difference 2 to 4 Storeys	○ 0.7	○ 0.9	◉ 1
Height Difference < 2 Storeys	○ 1	○ 1	○ 1

Factor D 1.0

3.5 Site Characteristics - *Stability, landslide threat, liquefaction etc as it affects the structural performance from a life-safety perspective*

Effect on Structural Performance ○ *Severe* ○ *Significant* ◉ *Insignificant* **Factor E** 1.0

The soil is susceptible to liquefaction under a large earth quake(ULS) event. But the structure is capable of sustaining soil deformations.

3.6 Other Factors - *for allowance of all other relevant characterstics of the building* **For ≤ 3 storeys - Maximum value 2.5 otherwise - Maximum value 1.5. No minimum.** **Factor F** 2.0

Record rationale for choice of Factor F:

The building has adequate in-plane capacity in the longitudinal direction. An assessment of the brick infill's out-of-plane capacity shows that it is sufficient to resist the lateral forces. Based on visual inspections, the building is in relatively good condition considering its age. Also compensation for the pounding penalty which is unlikely to be an issue for structure such as

3.7 Performance Achievement Ratio (PAR)
(equals **A x B x C x D x E x F**)

PAR
Longitudinal **2.00**

WARNING!! *This initial evaluation has been carried out solely as an initial seismic assessment of the building following the procedure set out in "The Seismic Assessment of Existing Buildings" Technical Guidelines for Engineering Assessments, July 2017. This spreadsheet must be read in conjunction with the limitations set out in the accompanying report, and should not be relied on by any party for any other purpose. Detailed inspections and engineering calculations, or engineering judge ments based on them, have not been undertaken, and these may lead to a different result or seismic grade.*

Initial Evaluation Procedure (IEP) Assessment - Completed for {Client/TA}

Street Number & Name:	92 King Edward Street	Job No.:	6-CB170.00
AKA:		By:	ST
Name of building:		Date:	10/10/2024
City:	Dunedin	Revision No.:	1

Table IEP-3 Initial Evaluation Procedure Step 3

Step 3 - Assessment of Performance Achievement Ratio (PAR)

(Refer Appendix B - Section B3.2)

b) Transverse Direction

Factors

potential CSWs **Effect on Structural Performance**
(Choose a value - Do not interpolate)

3.1 Plan Irregularity

Effect on Structural Performance ○ Severe ○ Significant ◉ Insignificant **Factor A** 1.0

Equally spaced portal frames

3.2 Vertical Irregularity

Effect on Structural Performance ○ Severe ○ Significant ◉ Insignificant **Factor B** 1.0

High portal columns

3.3 Short Columns

Effect on Structural Performance ○ Severe ○ Significant ◉ Insignificant **Factor C** 1.0

No short columns

3.4 Pounding Potential

(Estimate D1 and D2 and set D = the lower of the two, or 1.0 if no potential for pounding, or consequences are considered to be minimal)

a) Factor D1: - Pounding Effect

Note:
Values given assume the building has a frame structure. For stiff buildings (eg shear walls), the effect of pounding may be reduced by taking the coefficient to the right of the value applicable to frame buildings.

Factor D1 For Transverse Direction: 1.0

Table for Selection of Factor D1	Severe	Significant	Insignificant
Separation	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Alignment of Floors within 20% of Storey Height	○ 1	○ 1	◉ 1
Alignment of Floors not within 20% of Storey Height	○ 0.4	○ 0.7	○ 0.8

b) Factor D2: - Height Difference Effect

Factor D2 For Transverse Direction: 1.0

Table for Selection of Factor D2	Severe	Significant	Insignificant
	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Height Difference > 4 Storeys	○ 0.4	○ 0.7	○ 1
Height Difference 2 to 4 Storeys	○ 0.7	○ 0.9	◉ 1
Height Difference < 2 Storeys	○ 1	○ 1	○ 1

Factor D 1.0

3.5 Site Characteristics - *Stability, landslide threat, liquefaction etc as it affects the structural performance from a life-safety perspective*

Effect on Structural Performance ○ Severe ○ Significant ◉ Insignificant **Factor E** 1.0

The soil is susceptible to liquefaction under large earth quake(ULS) event. But the structure is capable of sustaining soil deformations.

3.6 Other Factors - *for allowance of all other relevant characterstics of the building* **For ≤ 3 storeys - Maximum value 2.5 otherwise - Maximum value 1.5. No minimum.** **Factor F** 1.00

Record rationale for choice of Factor F:

The structural steel portal frames, located in the transverse direction, provide adequate capacity to resist loads. An assessment of the brick infill's out-of-plane capacity shows that it is sufficient to resist the lateral forces. Based on visual inspections, the building is in relatively good condition considering its age. Also compensation for the pounding penalty which

3.7 Performance Achievement Ratio (PAR)
(equals **A x B x C x D x E x F**)

PAR
Transverse **1.00**

Initial Evaluation Procedure (IEP) Assessment - Completed for {Client/TA} Page 6

Street Number & Name:	92 King Edward Street	Job No.:	6-CB170.00
AKA:		By:	ST
Name of building:		Date:	10/10/2024
City:	Dunedin	Revision No.:	1

Table IEP-4 Initial Evaluation Procedure Steps 4, 5, 6 and 7

Step 4 - Percentage of New Building Standard *(%NBS)*

		Longitudinal	Transverse
4.1	Assessed Baseline *%NBS (%NBS)* $_b$ (from Table IEP - 1)	35%	74%
4.2	Performance Achievement Ratio (PAR) (from Table IEP - 2)	2.00	1.00
4.3	PAR x Baseline *(%NBS)* $_b$	70%	75%
4.4	Percentage New Building Standard *(%NBS)* - Seismic Rating (Use lower of two values from Step 4.3)		70%

Step 5 - Is *%NBS* < 34?

NO

Step 6 - Potentially Earthquake Risk (is *%NBS* < 67)?

NO

Step 7 - Provisional Grading for Seismic Risk based on IEP

Seismic Grade: B

Additional Comments (items of note affecting IEP based seismic rating)

An assessment of the brick infill's out-of-plane capacity shows that it is sufficient to resist the lateral forces. Additionally, visual inspection confirms that the brick infill has adequate in-plane capacity. Overall the building reamins in relatively good condition for its age.

Relationship between Grade and *%NBS* :

WARNING!! *This initial evaluation has been carried out solely as an initial seismic assessment of the building following the procedure set out in "The Seismic Assessment of Existing Buildings" Technical Guidelines for Engineering Assessments, July 2017. This spreadsheet must be read in conjunction with the limitations set out in the accompanying report, and should not be relied on by any party for any other purpose. Detailed inspections and engineering calculations, or engineering judgements based on them, have not been undertaken, and these may lead to a different result or seismic grade.*

Initial Evaluation Procedure (IEP) Assessment - Completed for {Client/TA}

Street Number & Name:	92 King Edward Street	Job No.:	6-CB170.00
AKA:		By:	ST
Name of building:		Date:	10/10/2024
City:	Dunedin	Revision No.:	1

Table IEP-5 Initial Evaluation Procedure Step 8

Step 8 - Identification of potential Severe Structural Weaknesses (SSWs) that could result in significant risk to a significant number of occupants

8.1	**Number of storeys above ground level**	1
8.2	**Presence of heavy concrete floors and/or concrete roof? (Y/N)**	N

Potential Severe Structural Weaknesses (SSWs):

Note: Options that are greyed out are not applicable and need not be considered.

Occupancy not considered to be significant - no further consideration required

Risk not considered to be significant - no further consideration required

The following potential Severe Structural Weaknesses (SSWs) have been identified in the building that could result in significant risk to a significant number of occupants:

1. None identified
2. Weak or soft storey (except top storey)
3. Brittle columns and/or beam-column joints the deformations of which are not constrained by other structural elements
4. Flat slab buildings with lateral capacity reliant on low ductility slab-to-column connections
5. No identifiable connection between primary structure and diaphragms
6. Ledge and gap stairs

IEP Assessment Confirmed by ______ **Signature**

______ **Name**

______ **CPEng. No**

WARNING!! *This initial evaluation has been carried out solely as an initial seismic assessment of the building following the procedure set out in "The Seismic Assessment of Existing Buildings" Technical Guidelines for Engineering Assessments, July 2017. This spreadsheet must be read in conjunction with the limitations set out in the accompanying report, and should not be relied on by any party for any other purpose. Detailed inspections and engineering calculations, or engineering judgements based on them, have not been undertaken, and these may lead to a different result or seismic grade.*

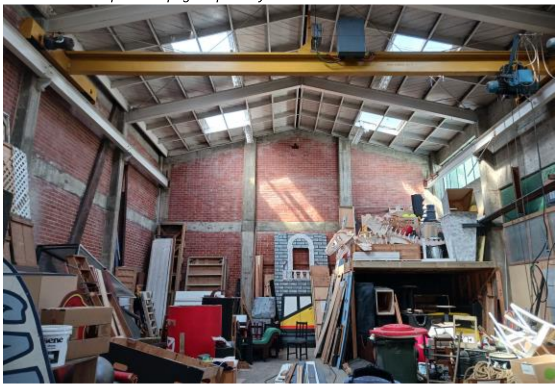
Initial Evaluation Procedure (IEP) Assessment - Completed for {Client/TA} Page 1a

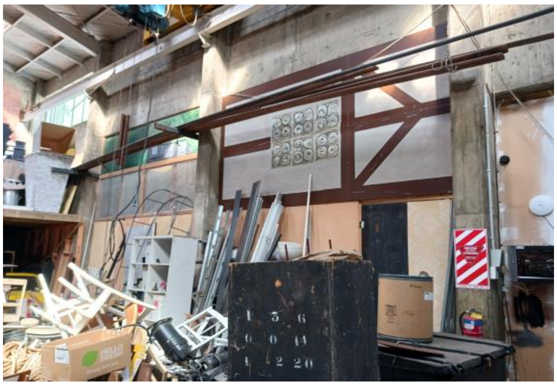
Street Number & Name:	**92 King Edward Street**	**Job No.:**	**6-CB170.00**
AKA:		**By:**	**ST**
Name of building:		**Date:**	**10/10/2024**
City:	**Dunedin**	**Revision No.:**	**1**

Table IEP-1a Additional Photos and Sketches

Add any additional photographs, notes or sketches required below:

Note: print this page separately





WARNING!! *This initial evaluation has been carried out solely as an initial seismic assessment of the building following the procedure set out "The Seismic Assessment of Existing Buildings" Technical Guidelines for Engineering Assessments, July 2017. This spreadsheet must be read in conjunction with the limitations set out in the accompanying report, and should not be relied on by any party for any other purpose. Detailed inspections and engineering calculations, or engineering judgements based on them, have not been undertaken, and these may lead to a different result or seismic grade.*

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APPENDIX 2: GEOTECHNICAL REPORT

Mayfair Theatre Charitable Trust

MAYFAIR THEATRE SEISMIC STRENGTHENING

GEOTECHNICAL DESKTOP REPORT

6-CB170.00

CONFIDENTIAL

MAYFAIR THEATRE SEISMIC STRENGTHENING

GEOTECHNICAL DESKTOP REPORT

Mayfair Theatre Charitable Trust

WSP
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197 Rattray Street
Dunedin 9016
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wsp.com/

REV	DATE	DETAILS
0	11/11/2024	ISSUE FOR CLIENT COMMENTS

	NAME	DATE	SIGNATURE
Prepared by:	Bryce Harrison	11/11/2024	
Reviewed by:	Greg Saul	11/11/2024	
Approved by:	Kevin Wood	11/11/2024	

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TABLE OF CONTENTS

1 INTRODUCTION

WSP has been commissioned by the Mayfair Theatre Charitable Trust (MTCT) to provide geotechnical inputs for the Detailed Seismic Assessment of the Mayfair Theatre building at 100 King Edward Street, South Dunedin ('the site'). The building is a two-storey unreinforced masonry structure, currently utilised as a performing arts facility.

This report presents a summary of the findings the desktop assessment undertaken to inform the ground conditions and provide inputs to the Detailed Seismic Assessment (DSA) of the building.

1.1 SITE DESCRIPTION

The site is located within South Dunedin and is bounded by commercial properties to the north, south and east, and King Edward Street to the west. The site is within a Principal Centre Zone and is designated commercial and mixed use in the Dunedin City Council (DCC) Second Generation District Plan (2GP)

The building is a category 2 heritage structure, originally constructed in 1914.

Figure 1-1: Site Location plan

2 GEOTECHNICAL INFORMATION

2.1 PUBLISHED GEOLOGY

The published geology indicates that the site is underlain by 'Holocene River Deposits' (Unit OIS1) comprised of loose well sorted sandstone, schist, and volcanic-derived gravel and sand (often quartzose) with minor mud and peat. An extract from the GNS QMap is presented below in Figure 2-1.

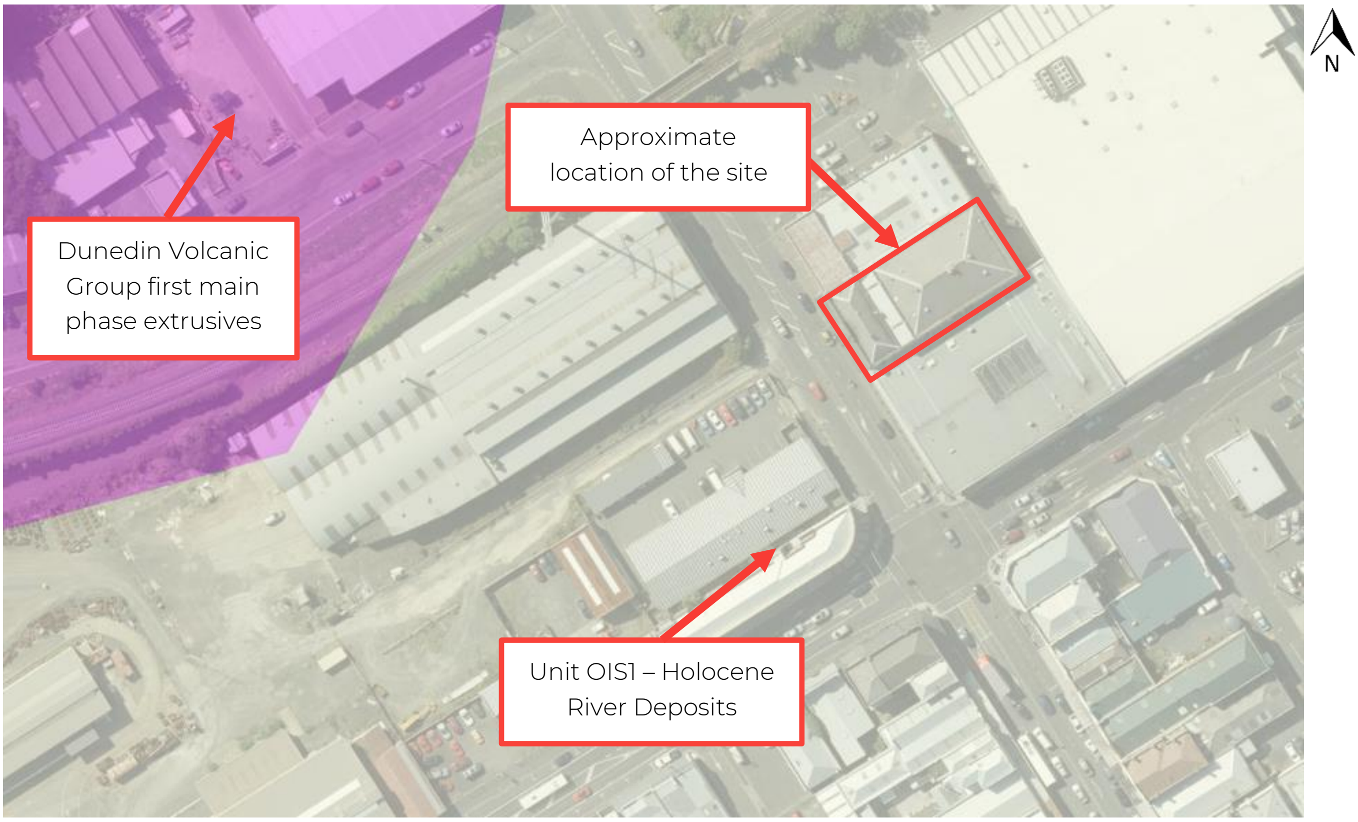

Figure 2-1: Published geology (geological basemap courtesy of (Heron, 2020))

2.2 AVAILABLE GEOTECHNICAL INFORMATION

A review of the publicly available geotechnical information has been undertaken using the New Zealand Geotechnical Database (NZGD) to form a general understanding of the underlying ground conditions and the liquefaction potential of the site. An indicative site plan of the investigation locations used in the assessment has been provided below in Figure 2-2. The investigations presented below represent a sample of what is available on NZGD and have been selected based on termination depth, site proximity, and orientation to allow interpolation of the expected site conditions. The investigations considered in this assessment comprise six Cone Penetration Tests (CPTs) performed to depths between 12.8 – 21.4 m bgl.

These investigations provide a high-level understanding of the underlying ground conditions. The investigations are between 250 – 350 m away from the site which may mean that the available investigation data is not completely representative of the site. Additional site-specific investigations are therefore recommended to confirm the assessment provided in this report.

Figure 2-2: NZGD Investigation locations

2.3 INDICATIVE GROUND PROFILE

An indicative ground profile has been summarised below based on the available geotechnical information. This is indented to act as a preliminary description of the site conditions and further site-specific investigations should be undertaken.

Fill: The upper 0.5 – 1.0 m of the site is expected to be underlain by fill material imported to form a construction platform. Fill material measured in adjacent locations appear to have a CPT cone tip resistance (q_c) between 1 – 5 MPa (typically 2 MPa). However, as this unit is not controlled by geological processes, it is not possible to accurately infer the likely composition of this material without site-specific investigations.

Marine/Estuarine Deposits: The fill material is directly underlain by marine/estuarine deposits to a depth of approximately 20 m bgl. There are possibly rare inclusions of cobbles or boulders within this unit, transported from the adjacent hillside northwest of the site. This layer can be divided within three separate sub-layers:

- 0.5 – 5 m bgl – soil behaviour type typically clay like with some layers of organic soils up to 0.5 m thick. The q_c is between 0.1 – 0.5 MPa (typ. 0.25 MPa).
- 5 – 10 m bgl – soil behaviour type indicates silt/clay mixtures interbedded with sand and silt/sand mixtures. The q_c is between 0.2 – 0.7 MPa (typ. 0.5 MPa) in the clay and silt/clay mixtures and between 4 – 14 MPa (typ. 6 MPa) in the interbedded sand and silt/sand mixtures.

- 10 – 20 m bgl – soil behaviour type indicates sand and silt/sand mixtures interbedded with silt/clay mixtures. The qc is between 4 – 14 MPa (typ. 6 MPa) in the interbedded sand and silt/sand mixtures and between 0.2 – 0.7 MPa (typ. 0.5 MPa) in the clay and silt/clay mixtures.

Sandstone/Siltstone: Sandstone/siltstone is found at depths below 20 m bgl and is the typical refusal depth of the CPT investigations.

2.4 GROUNDWATER

A study of the groundwater in the low-lying coastal areas of Dunedin was completed by Cox et al. (2023) on behalf of GNS Science. An extract from the report is presented below in Figure 2-3 which indicates the depth to groundwater at the site. Based on Figure 2-3, the depth to groundwater at the site is inferred between 0.5 -1.0 m bgl.

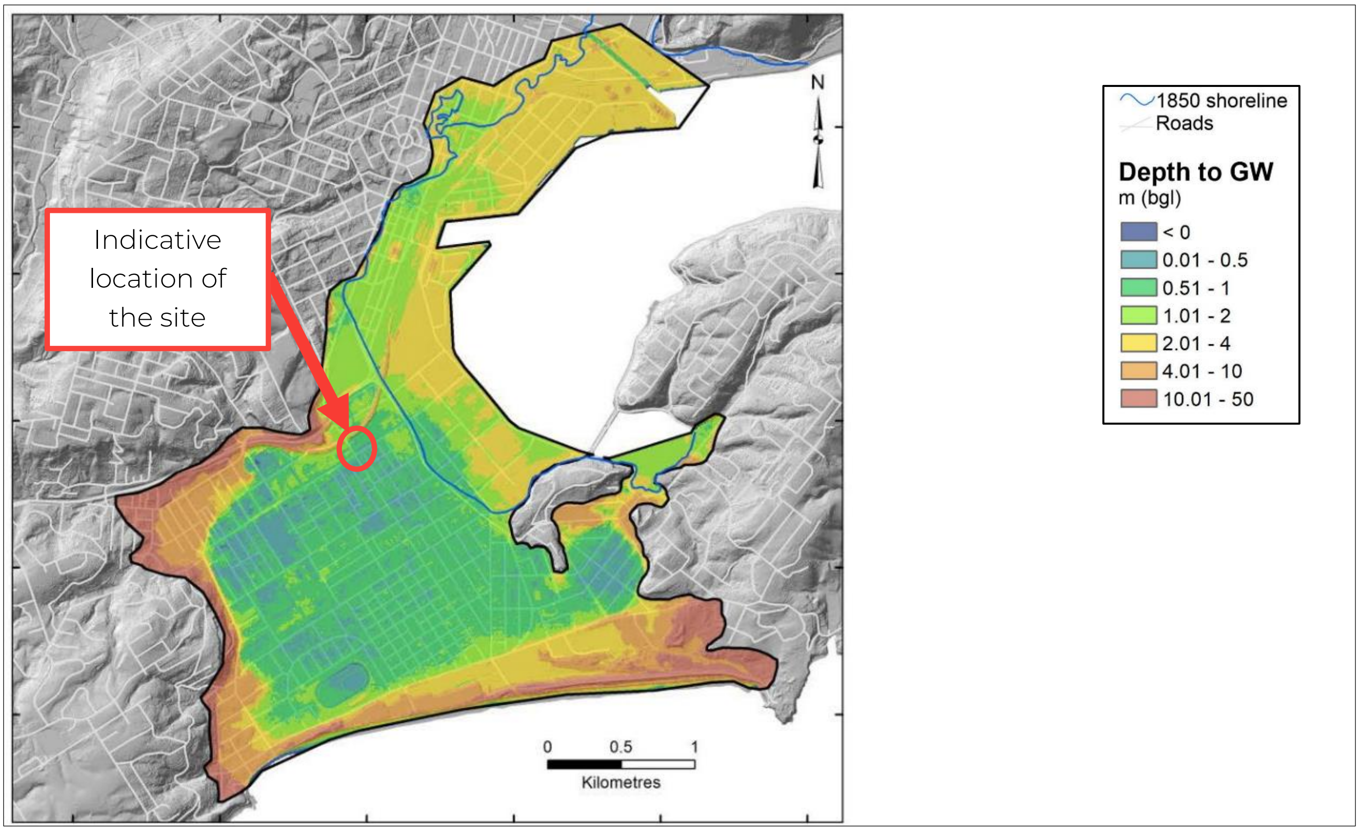

Figure 2-3: Groundwater of low-lying Dunedin suburbs (image courtesy of Cox et al. (2023))

3 SEISMICITY

3.1 FAULT MAP

Barrell (2021) indicated there are 26 active or potentially active faults identified within the Clutha and Dunedin City Districts. Of these faults, only two have a recurrence interval (RI) of surface rupture of <2,000 years (RI Class I after (Kerr, et al., 2003)). The remaining 24 faults have RIs >10,000 years (> RI Class V). The two RI Class 1 faults within the Clutha and Dunedin City Districts are the Akatore fault and the Settlement fault, with recurrence intervals of 1700 years and 1800 years, respectively (Barrell, 2021).

The Akatore fault is the closest RI Class 1 fault and is approximately 24 km south of the site. The closest of all the mapped faults is the Kaikorai Fault (<1 km from the site). However, the Kaikorai fault is only considered potentially active and has an inferred recurrence interval of >20,000 years (Barrell, 2021).

An indicative map of the active faults in proximity of Dunedin City There are no faults listed within NZS1170.5 that are within 20 km of the site for determination of structural design actions.

Figure 3-1: Fault map (base image courtesy of ORC hazard maps)

3.2 HISTORICAL EARTHQUAKE PERFORMANCE

Dunedin has experienced minimal local seismicity since European settlement. The largest earthquake that has been experienced within the Dunedin City District was the 9 April 1974 Dunedin Earthquake which had a reported local magnitude of 5.0 and a maximum ground acceleration of 0.27g recorded in St Clair.

The 1974 earthquake had an epicentre approximately 10 km south of central Dunedin and a focal depth of 20 km (Barrell, 2021). Bishop (1974) noted that the concentration of damage claims from the 1974 earthquake was greatest in the southern Dunedin area. Most of the reported damage was to residential housing, primarily in the form of damage/collapse of chimneys and there were no reports of liquefaction from this event.

3.3 DESIGN LOADING

Peak ground accelerations (PGAs) have been derived for liquefaction and lateral spreading assessments in accordance with Section 6.2 of the Waka Kotahi Bridge Manual (2014) using Equation 1 below:

$$C_o g\ (or\ PGA) = C_{0,1000} \frac{R_u}{1.3} f g \qquad \text{(Equation 1)}$$

PGAs have been derived based on the following parameters and presented in Table 3-1 below.

- Importance Level 3 (IL3) structure with a 50-year design life.
- Site subsoil class 'D or E'(in accordance with NZS1170.5).
- f, site subsoil class factor = 1.00
- $C_{0,1000}$, 1000-year return period PGA coefficient = 0.25 (Dunedin).

'TS1170.5 – Structural Design Actions – Part 5 Earthquake Actions' was released for comment on 15 February 2024 by Standards New Zealand. Within TS 1170.5, there is revised seismic design criteria based off the updated National Seismic Hazard Model (NSHM) that was released in October 2022. The current issue of TS 1170.5 indicates that the DCLS seismic criteria for this site could increase by approximately 44% in terms of PGA ($PGA_{Mw=6.5}$=0.36g).

Table 3-1:Seismic design criteria

<table>
<tr><th>Seismic Case</th><th>Annual Probability of Exceedance [1]</th><th>Return Period Factor (R) [2]</th><th>PGA (g)</th><th>Effective Magnitude</th></tr>
<tr><td>Serviceability Limit State (SLS)</td><td>~1/50</td><td>0.33</td><td>0.06</td><td rowspan="3">6.0</td></tr>
<tr><td>Damage Control Limit State (DCLS)/Ultimate Limit State (ULS)</td><td>1/1000</td><td>1.5</td><td>0.25</td></tr>
<tr><td>Collapse Avoidance Limit State (CALS)</td><td>>1/2500</td><td>2.0</td><td>0.38</td></tr>
</table>

[1] Annual probabilities of exceedance (APE) are based on Table 3.3 NZS 1170.0, Table 3.5 NZS 1170.5 and Table 5.3 of the Bridge Manual.

[2] Return period factors are based on Table 3.5 of NZS1170.5 and Table 5.1 of the Bridge Manual

4 LIQUEFACTION AND CYCLIC SOFTENING

4.1 DEFINITION AND CONSEQUENCE

Liquefaction is a phenomenon where saturated loose to medium-dense sands and low plasticity silts experience a rise in porewater pressures during strong shaking. This results in the loss of strength and stiffness of liquefied soils and consequent large deformations due to the development and subsequent dissipation of excess pore water pressures.

Cyclic softening is a behaviour that is exhibited in cohesive materials, characterised by a small (typically 20 to 30%) reduction in the shear strength due to successive cyclic loading. The susceptibility of a soil to cyclically soften is governed primarily by the stress history and peak undrained shear strength. The magnitude of deformation or instability that is experienced due to cyclic softening is dependent on the sensitivity of the soil but is significantly less than that expected with liquefaction.

4.2 MAPPED HAZARD

The mapped liquefaction hazard of the site on the Otago Regional Council (ORC) online hazards portal has been classified as 'C Domain'. C domain is defined as 'ground predominantly underlain by poorly consolidated marine or estuarine sediments with shallow groundwater' which provides a liquefaction susceptibility of 'moderate to high'. A snippet of the ORC hazards map has been presented below in Figure 4-1.

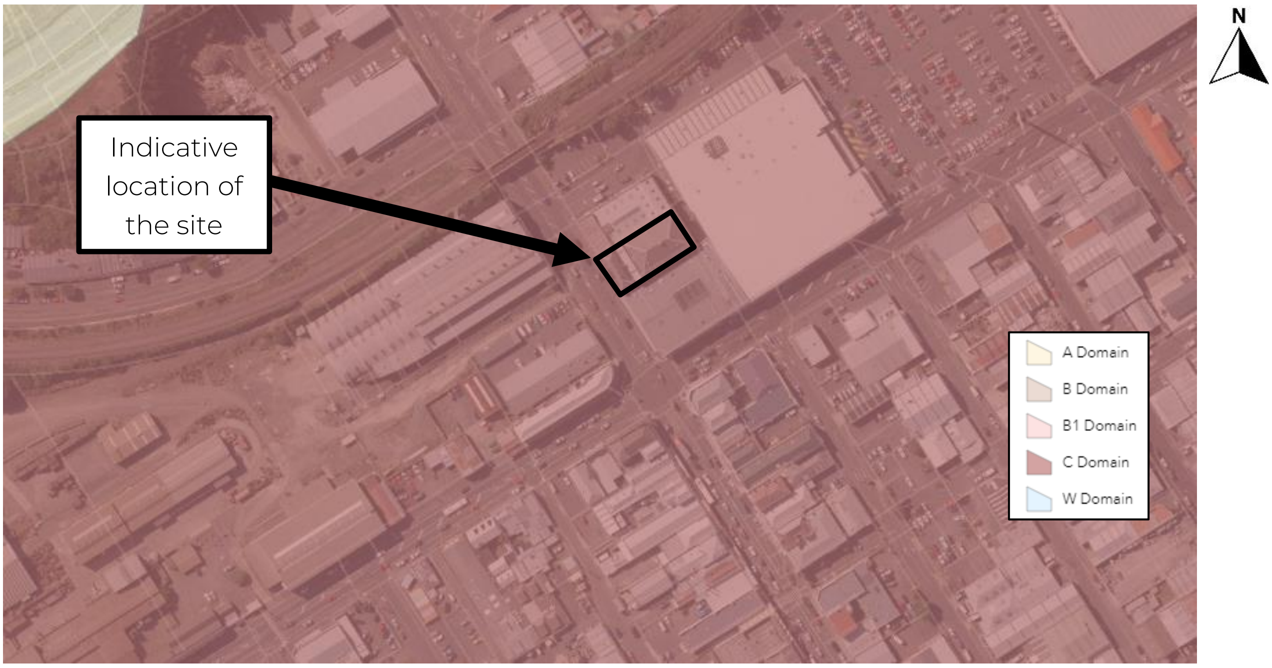

Figure 4-1: Liquefaction hazard map (image courtesy of ORC hazard portal)

4.3 LIQUEFACTION ASSESSMENT

4.3.1 TRIGGERING ASSESSMENT

A liquefaction triggering assessment has been undertaken based on the CPT data described in Section 2.2.

The assessment has been undertaken using the CLiq software (Version 3.0.2.4) based on the methodology outlined by Idriss and Boulanger (2014) method. An I_c cut-off value of 2.6 and probability of liquefaction of 15% have been adopted. Free-field liquefaction-induced settlements have been estimated using the methodology outlined by Zhang et al (2004). The fines content values have been adopted using the Idriss and Boulanger (2014) method with a default fines content correction (CFC) value of 0.

The Transition Zone function in Cliq has been adopted to correct the overestimation of liquefaction by the CPT probe in highly interbedded soils.

An earthquake groundwater level of 0.5 m bgl has been adopted for the assessment.

4.3.2 LIQUEFACTION ANALYSIS

The approximate extent of liquefaction under the ULS shaking event at the building site is presented in Table 4-1 below. Selected CLiq outputs under ULS shaking event are presented in Appendix A of this report.

Table 4-1: Summary of liquefaction assessment (CPTs > 250m from Site)

CPT ID[1]	Approximate Depth of Liquefaction (m bgl)	Approximate Depth of Cyclic Softening (m bgl)	ULS Approximate Free-field Settlements (mm)	Liquefaction Potential Index (LPI)	Liquefaction Severity Number (LSN)
CPT 216279 (20.2 m)	-	6.5 - 13	<20 mm	5 - 10	0 - 5
CPT 216278 (21.4 m)	-	7 – 12.5 (FoS ~1)	< 20 mm	0 - 5	0 - 5
CPT 185213 (20.7 m)	4.5 – 12 as 0.1 – 0.5 m thick interbeds at >1 m spacing except a 1 m thick layer between 5.7 and 6.7 m	-	40 mm	0 - 5	5 - 10
CPT 185470 (12.8 m)	3.5 – 4.7 and from 4.7 – 10.0 as 0.1 – 0.5m thick interbeds at ~1 m spacing	2.2 – 2.7	70 mm	10 - 15	10 – 15

CPT ID[1]	Approximate Depth of Liquefaction (m bgl)	Approximate Depth of Cyclic Softening (m bgl)	ULS Approximate Free-field Settlements (mm)	Liquefaction Potential Index (LPI)	Liquefaction Severity Number (LSN)
CPT 125217 (21.2 m)	8.0 -21 (0.1 – 0.5 m thick interbeds at >1 m spacing)	2.5 – 8	50 mm	15 - 20	5 -10
CPT 125219 (15.0 m)	10.5 – 12.0	2.5 – 10	40 mm	10 – 15	1 – 5

[1] Approximate termination depth reported in brackets as m bgl.

The main findings from the liquefaction and cyclic softening assessment of CPTs > 250m from the site are as follows:

- Liquefaction triggering appears to occur around 0.1 - 0.15g. Therefore, liquefaction is not expected under a SLS shaking event (i.e. 1 in 25-year event).
- Liquefaction of very thin sand like deposits is expected under a ULS shaking event (i.e. 1 in 500-year event), although, the combined thickness of liquefiable layers is typically less than 1.0 m – 1.5 m and occurs at depths >5 m bgl in the available CPTs.
- There is no to slight potential for liquefaction in half of the CPTs and a non-liquefiable crust at least 3.5 m thick in the other CPTs.
- Cyclic softening of the underlying cohesive soil deposits could be expected under a ULS shaking event from about 2.5 m bgl.
- The Liquefaction Potential Index (LPI) indices for the ULS event are in the range of 0 to 15 and Liquefaction Severity Number (LSN) are in the range of 0 – 20.
- The Ishihara (1985) chart for evaluating effect of crust thickness on liquefaction induced ground damage has been replicated below in Figure 4-2. With the CPTs used in the liquefaction assessment plotted on this chart, it appears that there is generally a sufficient non-liquefiable crust to prevent ground damage.

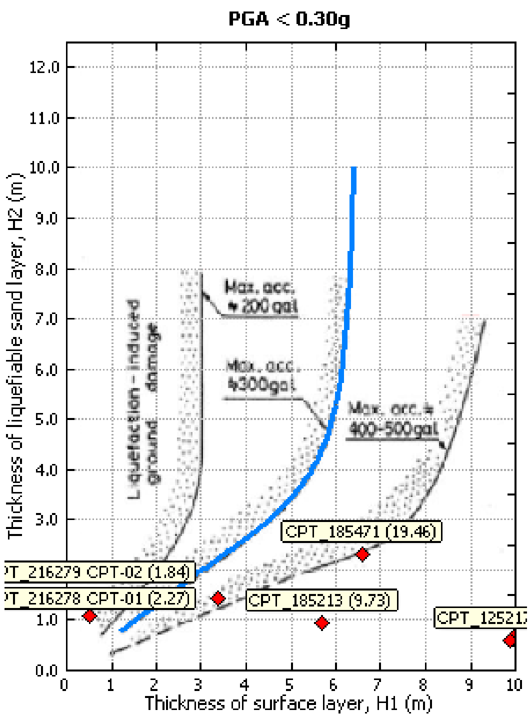

Figure 4-2: Ishihara method of assessment of liquefaction induced ground damage

4.3.3 DISCUSSION OF LIQUEFACTION ANALYSIS

- CPTs are more than 250 m away from the site and may not be representative.
- The available CPTs indicate total subsidence of liquefiable layers at > 3.5m depth are indicated to be between 20 – 70 mm. The free field settlement of the ground excluding any contribution from the building load is influence by the depth of the liquefiable layer
- The CLiq analysis indicated that there could possibly be cyclic softening 2.5 m bgl. The reduction in shear strength within the founding soils may result in an appreciable reduction in bearing capacity which could induce additional building settlement, but this is unlikely to result in settlement of shallow foundations.
- The calculated for ULS LSN and LPI range indicates the risk of ground surface damage due to liquefaction across the site is 'Mild to Moderate', in accordance with Table 5.1 of MBIE (2021b).
- The thickness of non-liquefied crust is such that free field settlement is likely to be less than that indicated from subsidence alone.
- A detailed assessment of shear induced ground displacement is required once a site-specific investigation has been undertaken to confirm the foundation size, composition and characteristics of the founding soils.

5 FOUNDATION ASSESSMENT

5.1 EXISTING FOUNDATION FORM

There are no records of the existing foundation form within the as-built drawing set. Site specific investigations will have to be undertaken to determine the foundation form to inform a detailed assessment of the available capacity. Based on our experience with structures of similar age and composition, it is probable that the existing foundations are shallow foundations consisting of strip footings beneath the brick walls and square pad footings beneath the columns.

5.2 PROVISIONAL FOUNDATION CAPACITIES

The actual composition/characteristics of the founding soils and foundation composition will need to be confirmed by a site-specific investigation before a detailed assessment of the foundation capacities can be made. However, a generalised assessment based on the available information is provided below.

The available investigation information indicates that the upper 1 - 2 m is generally comprised of a 'crust' of soils with marginally improved strength. The crust has either been formed by some degree of ground improvement through prior site loading and/or through importation of fill material. The crust is subsequently underlain by very soft to soft cohesive soils with an undrained strength of <15 kPa.

The available CPTs (>250m from the site) indicate very soft to soft normally consolidated cohesive soils at 0.5 to 1.0m depth with an undrained strength of 15 kPa. Shallow foundations on these soft soils have an estimated ultimate bearing capacity of approximately 60 kPa (allowable bearing capacity of 20 – 30 kPa for a settlement tolerance of 25 mm). The building does not appear to have settled significantly. However, relevelling works and/or masonry repairs may have been undertaken to obscure evidence of prior settlement.

Therefore, it is possible the building footings are narrow (<0.5m in width) and bear on the 0.5 to 1.0m thick layer of fill. This would reduce the bearing loads on the underlying soft cohesive soils.

Overall, we consider the existing foundations are likely to be applying loading which is close to the allowable bearing capacity.

Strengthening schemes involving even a marginal increase in the existing loading (provisionally of greater than approximately 10%) could cause intolerable settlements. Also, increasing the width of existing footings to compensate for increased loading may increase the influence depth of the foundation which would also induce additional settlement.

6 CONCLUSIONS AND RECOMMENDATIONS

- A review of the publicly available geotechnical investigation data and published geology indicates that the site is underlain by an estimated 0.5 – 1 m thick layer of **Fill** which is subsequently underlain by **Marine and Estuarine Sediments** to a depth of 20 m bgl. **Sandstone** is typically encountered at depths below 20 m bgl.
- The preliminary assessment of the probable foundation capacity indicates that the foundations may be operating close to the critical state performance. This means that a marginal increase in the building loading could induce intolerable settlements to the building.
- The quantitative liquefaction assessment indicates that subsidence within liquefied layers would typically be in the range of 20 – 70 mm under a ULS event. The available investigation data infers that there is a non-liquefiable crust which may reduce the surface manifestation of this subsidence to negligible levels. A site-specific investigation will be required to accurately assess the site liquefaction performance.
- The CLiq analysis indicated cyclic softening could occur from about 2.5 m bgl. This is possibly within the zone of influence of the building foundations which could result in additional settlement due to a loss in bearing capacity.
- The size and composition of the building foundations is unknown. Additional investigations are required to expose the existing foundations and underlying soils. This will require localised excavations to depths of 0.5 – 1.0 m bgl both inside and outside the building footprint.
- The CPTs that have been adopted for the assessment in this report range from 250 – 350 m away from the building footprint and may not present an accurate representation of the site itself. Therefore, 3 – 4 site specific CPTs are recommended to be undertaken to a depth of 15 – 20 m bgl both adjacent to and within the building footprint. It is also advised that 2 – 4 dissipation tests are undertaken within the very soft to soft cohesive soils as part of the CPT investigations.
- To inform an estimate of potential settlement, samples of soft soils should be taken with push tubes in either shallow boreholes or test pits for oedometer testing in a soils laboratory. Alternatively, a localised load trial should be undertaken to assess the sensitivity of the site to vertical settlement due to additional loading. This could be completed by placement of a rubbish skip loaded with aggregate near the building footprint, but outside of the influence zone. Survey would then need to be taken periodically to measure vertical settlement. A loading period of 6 months is recommended, or until the rate of change in vertical settlement appears negligible. This will be used to confirm predicted settlement of footings due to additional loading.
- The effects of liquefaction induced settlement and reduction in bearing capacity should be carefully considered in the DSA. Given the building is comprised of unreinforced masonry, the building will be relatively intolerant to differential settlement.

7 Bibliography

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MBIE (Nov 2021) Earthquake geotechnical engineering practice Module 3. Identification, assessment and mitigation of liquefaction hazards, Rev 1

8 LIMITATIONS

This report ('Report') has been prepared by WSP New Zealand Limited ('WSP') exclusively for Mayfair Theatre Charitable Trust ('Client') in relation to the preliminary geotechnical desktop report for the Mayfair Theatre ('Purpose') and in accordance with the Short Form Agreement dated 17/07/2024 ('Agreement'). The findings in this Report are based on and are subject to the assumptions specified in the Report. WSP accepts no liability whatsoever for any use or reliance on this Report, in whole or in part, for any purpose other than the Purpose or for any use or reliance on this Report by any third party.

In preparing this Report, WSP has relied upon data, surveys, analyses, designs, plans and other information ('Client Data') provided by or on behalf of the Client. Except as otherwise stated in this Report, WSP has not verified the accuracy or completeness of the Client Data. To the extent that the statements, opinions, facts, information, conclusions and/or recommendations in this Report are based in whole or part on the Client Data, those conclusions are contingent upon the accuracy and completeness of the Client Data. WSP will not be liable for any incorrect conclusions or findings in the Report should any Client Data be incorrect or have been concealed, withheld, misrepresented or otherwise not fully disclosed to WSP.

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APPENDIX A

SELECTED CLIQ OUTPUTS

GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

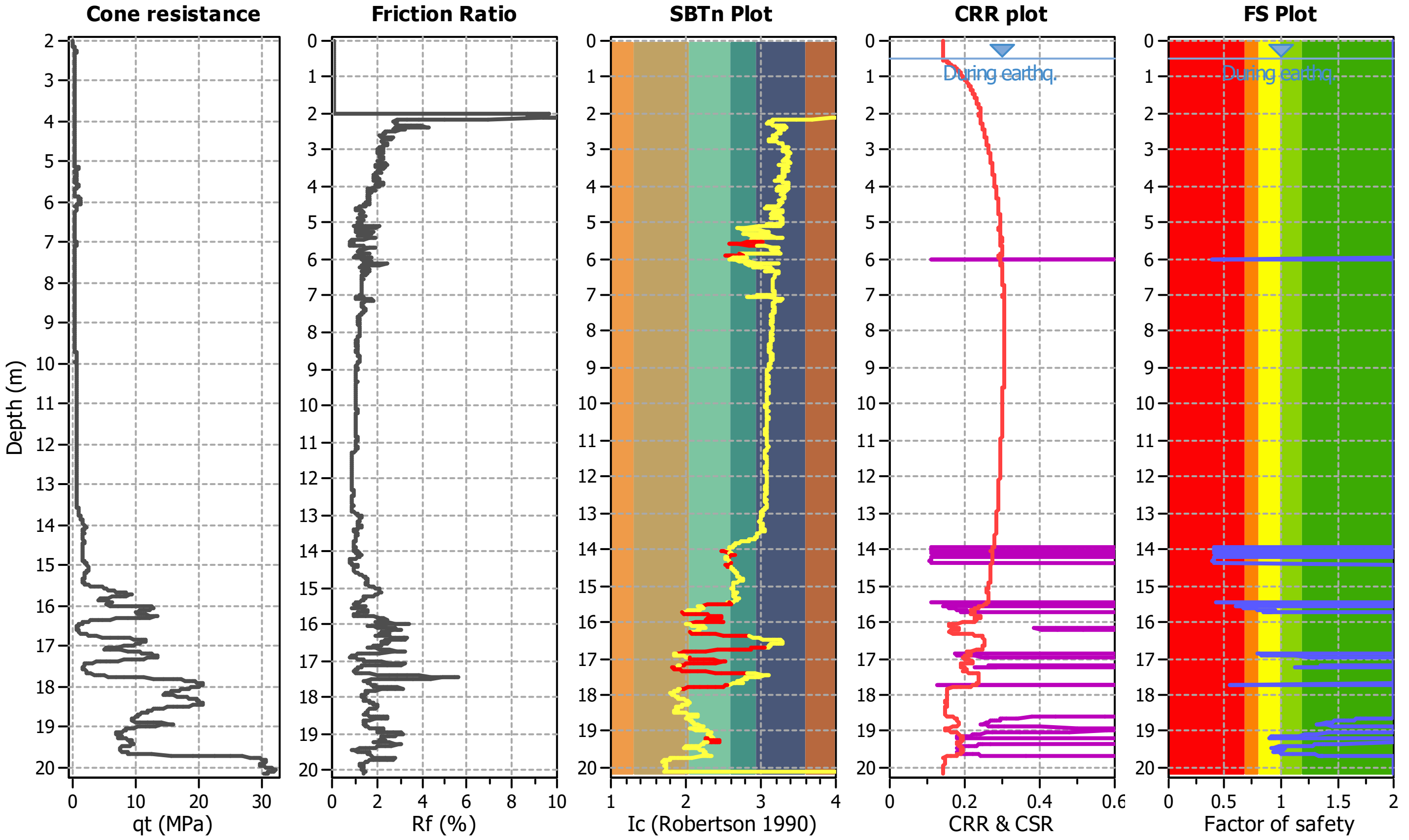
LIQUEFACTION ANALYSIS REPORT

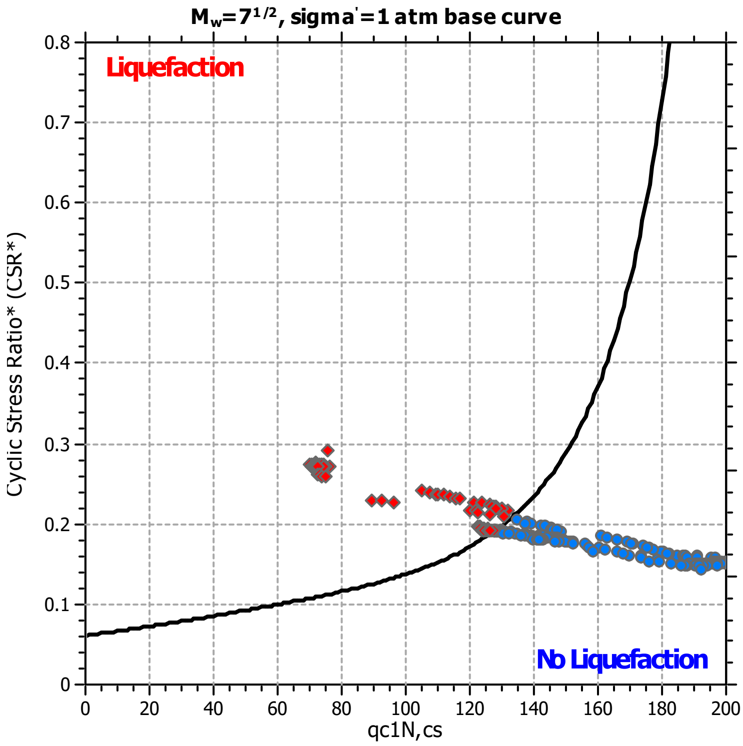
Project title : **Location :**

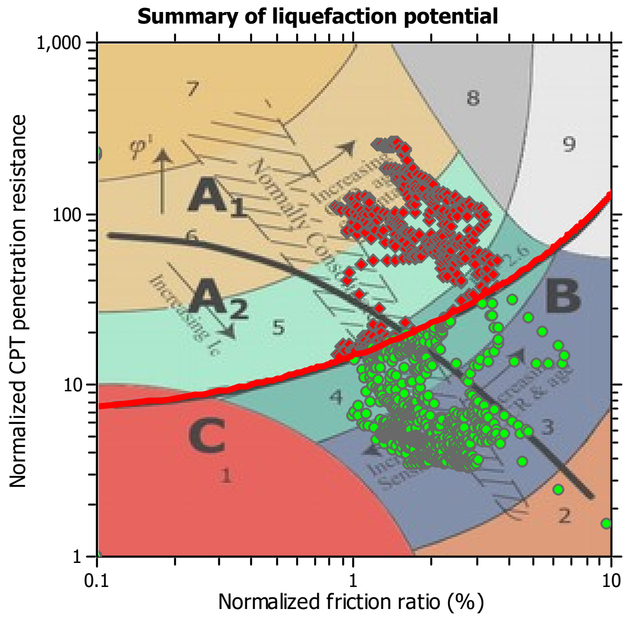
CPT file : CPT_216279

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.25	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

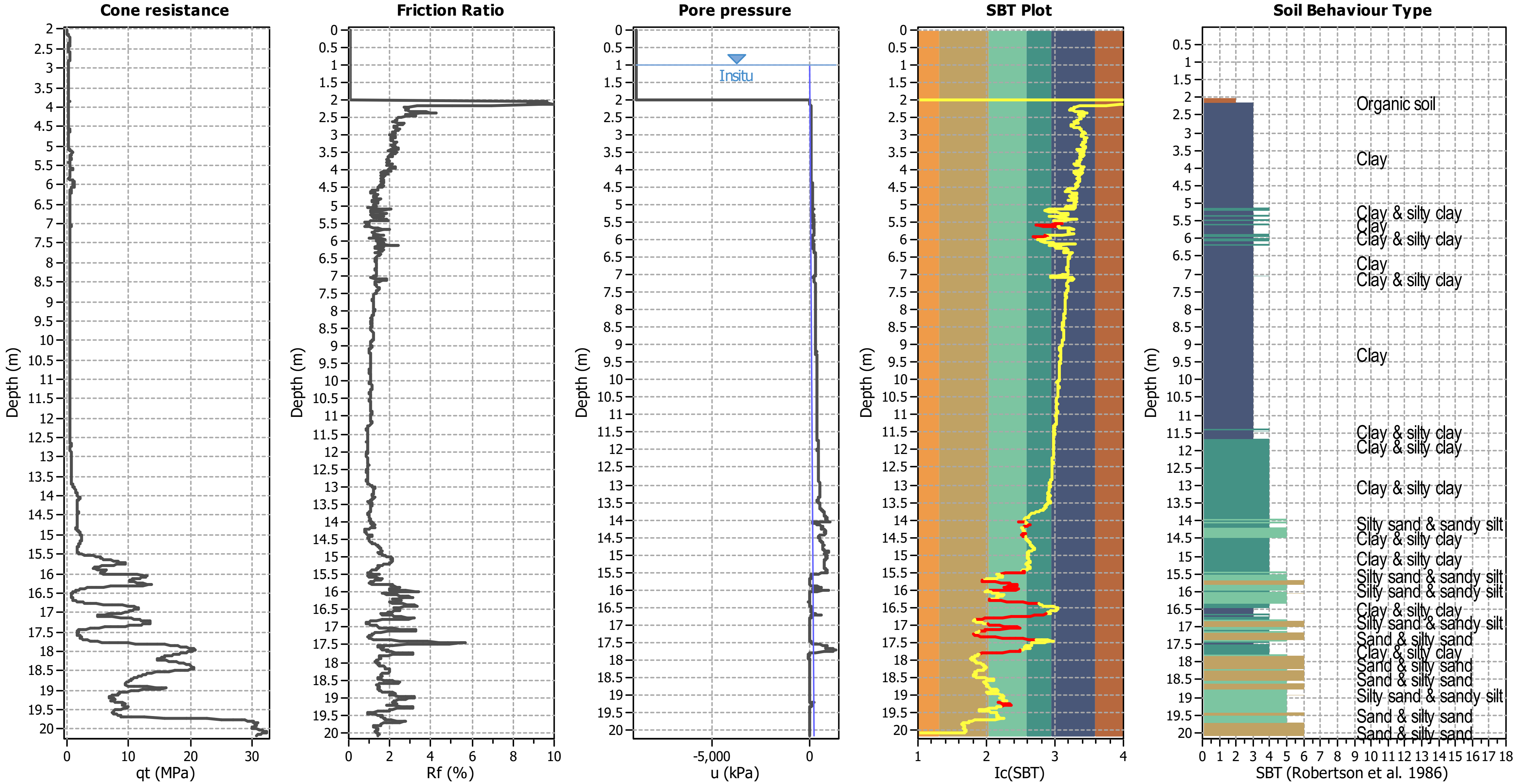






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



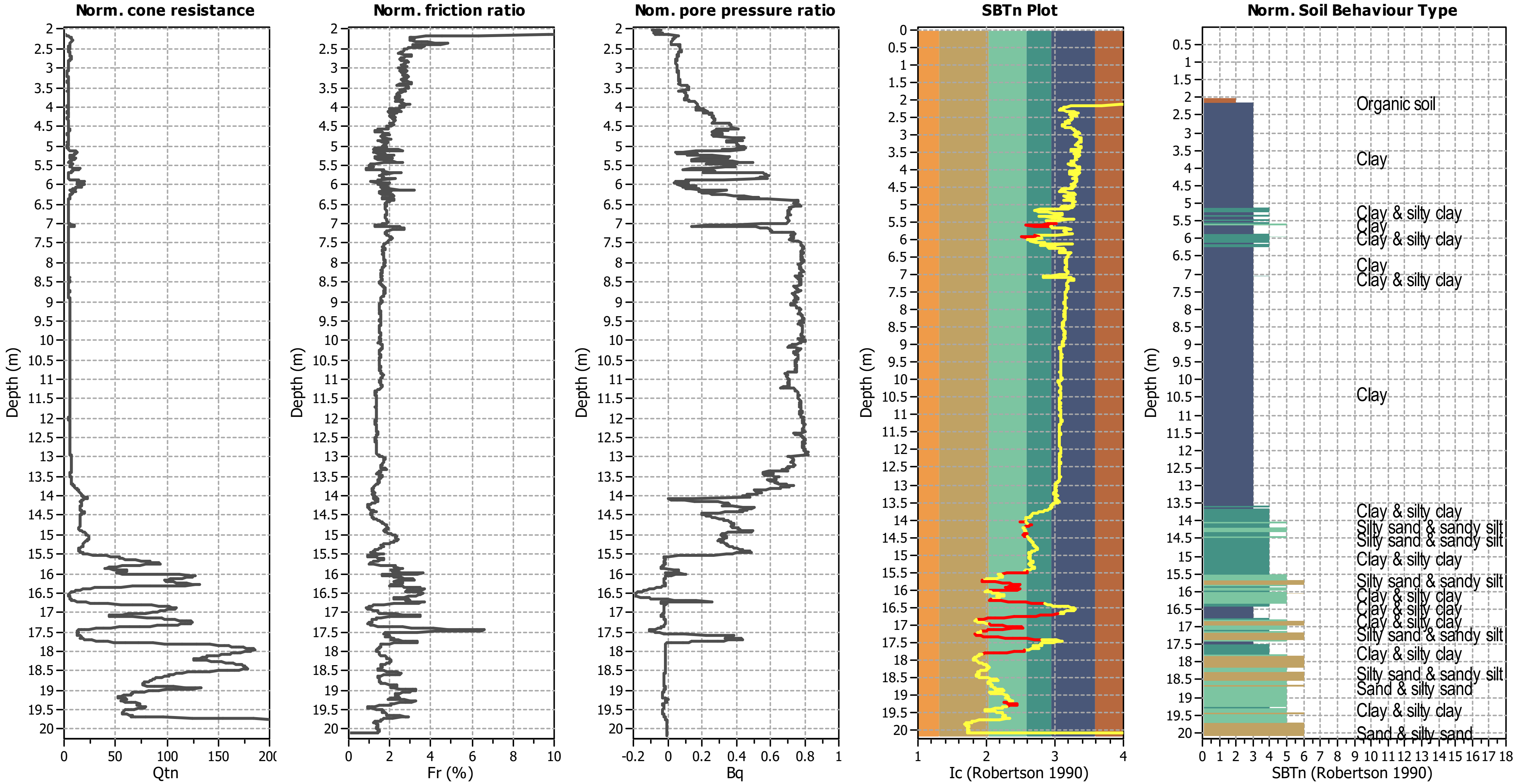
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



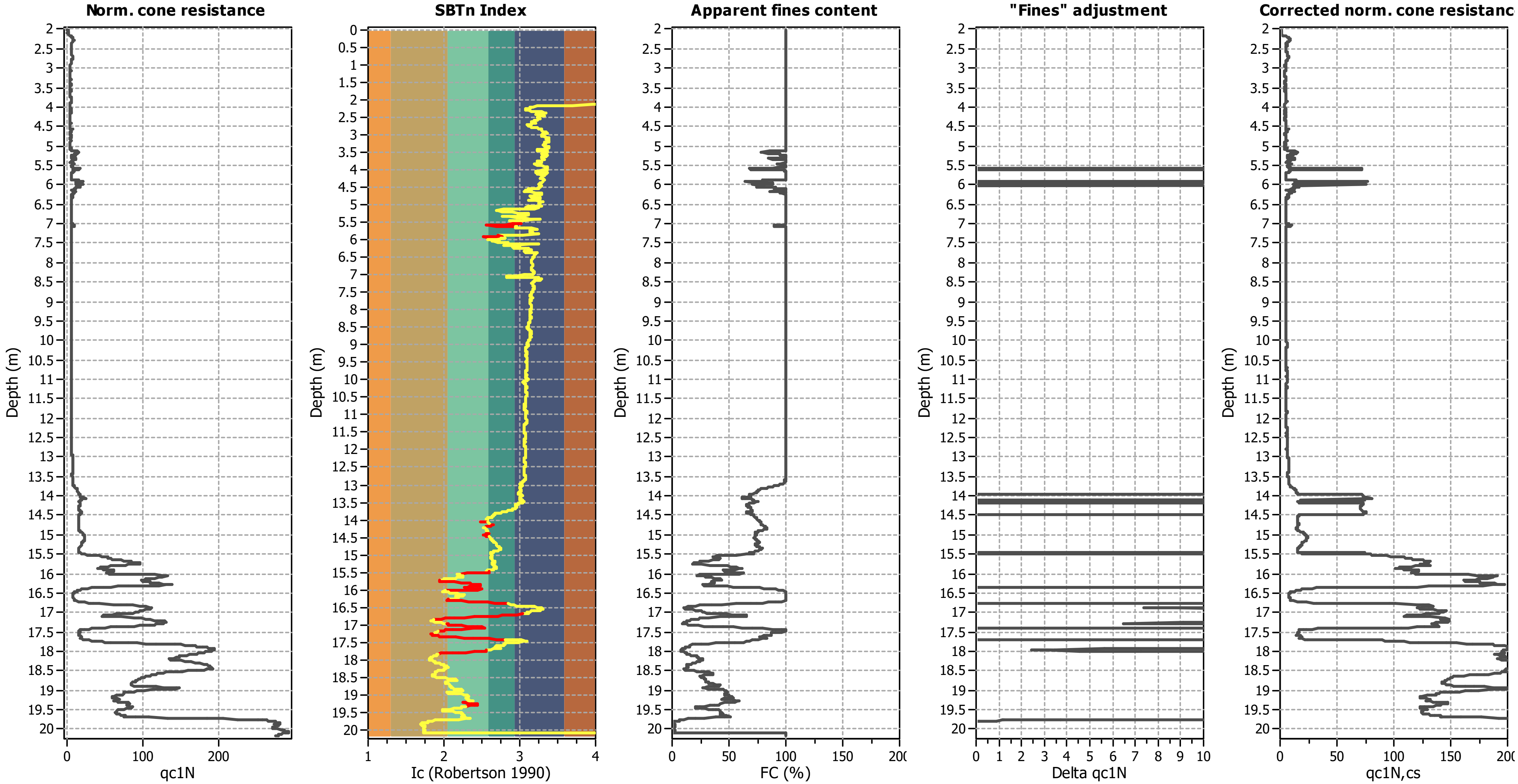
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

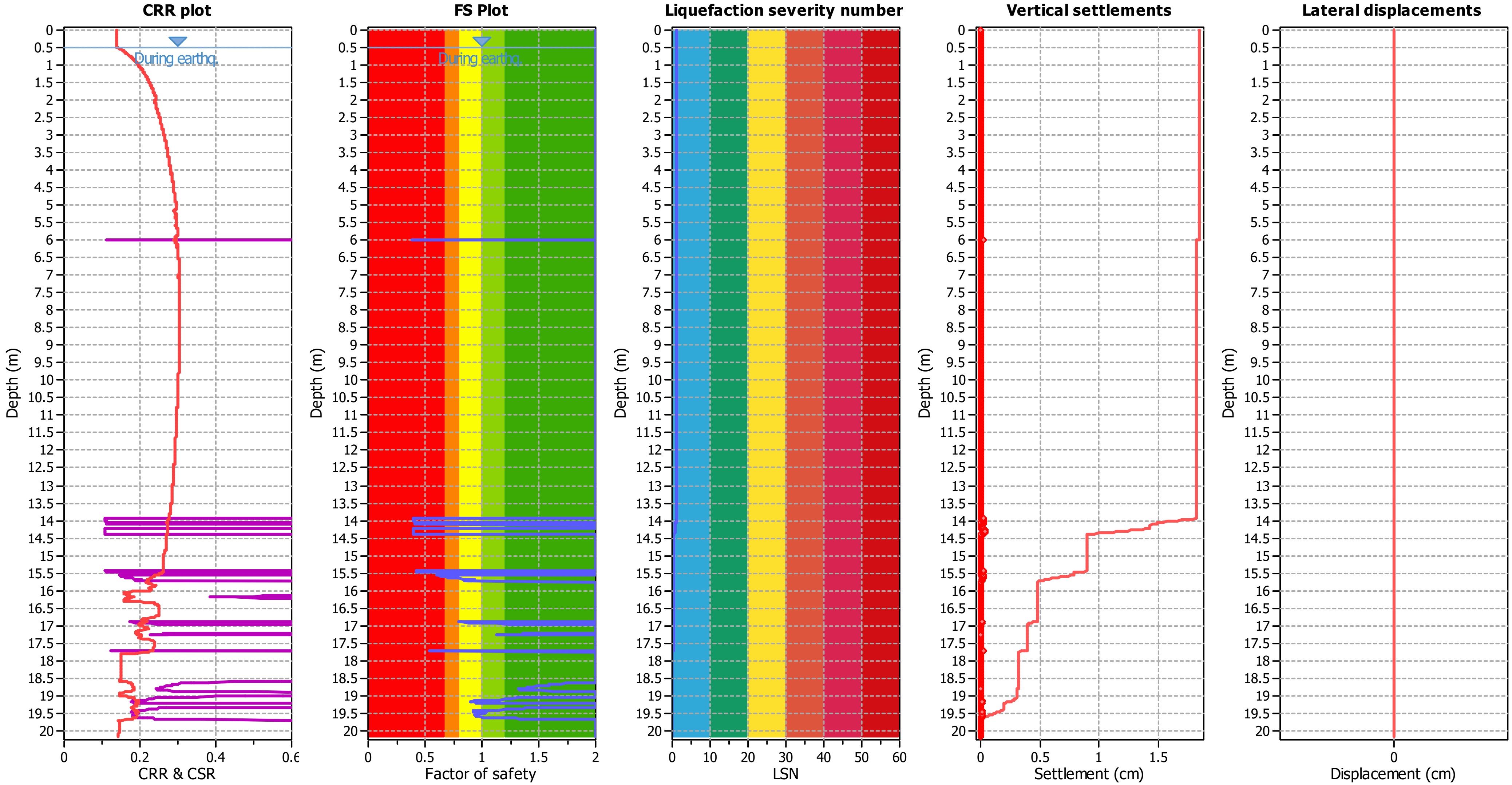
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

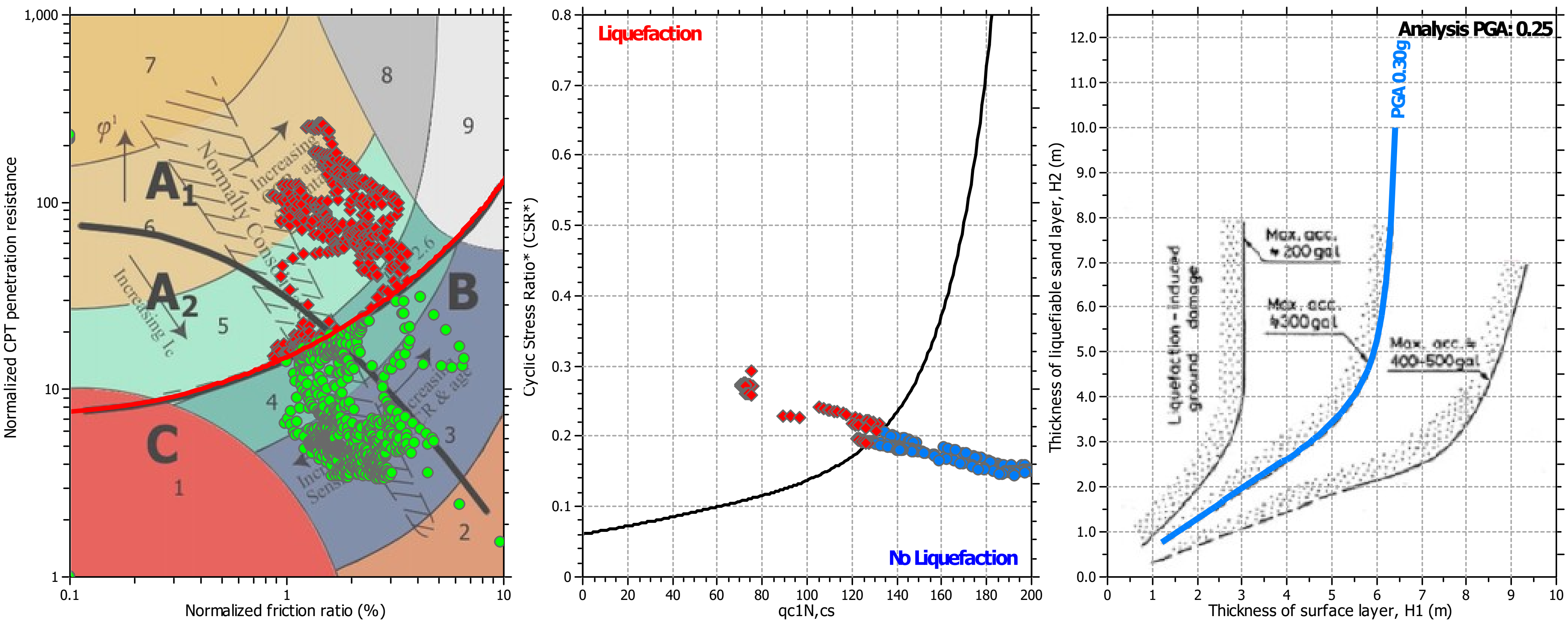
F.S. color scheme

- 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

LSN color scheme

- Severe damage
- Major expression of liquefaction
- Moderate to severe exp. of liquefaction
- Moderate expression of liquefaction
- Minor expression of liquefaction
- Little to no expression of liquefaction

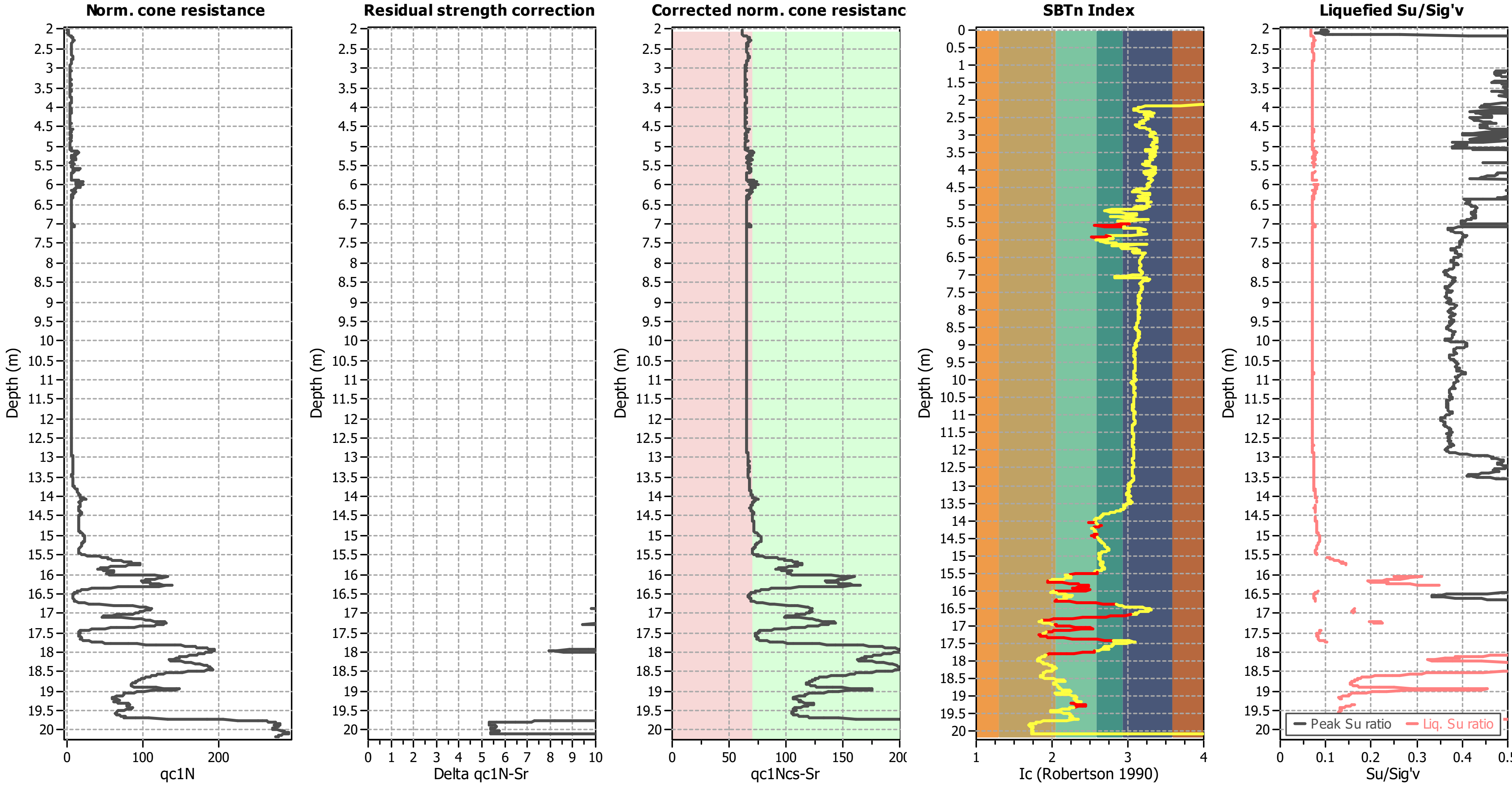
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

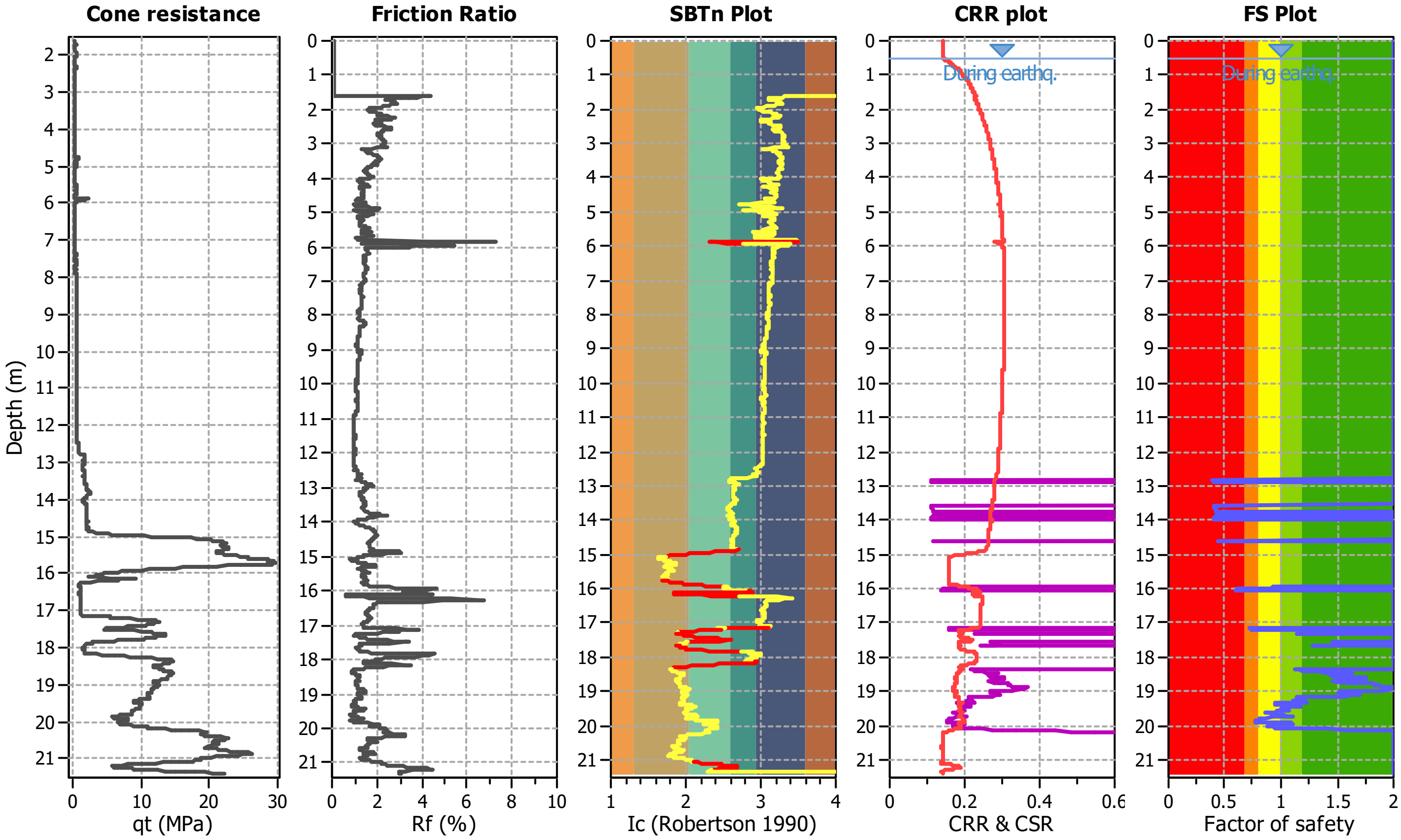
LIQUEFACTION ANALYSIS REPORT

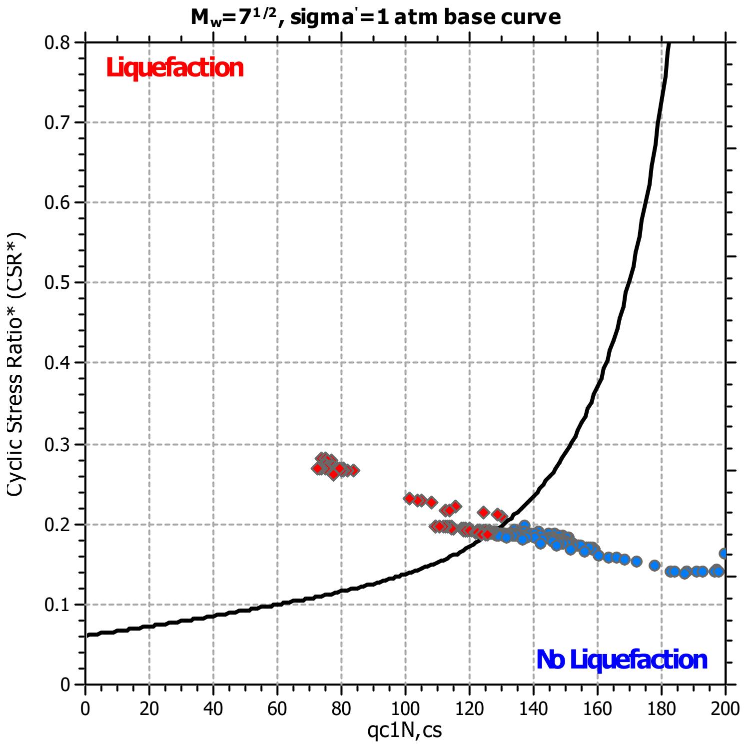
Project title : **Location :**

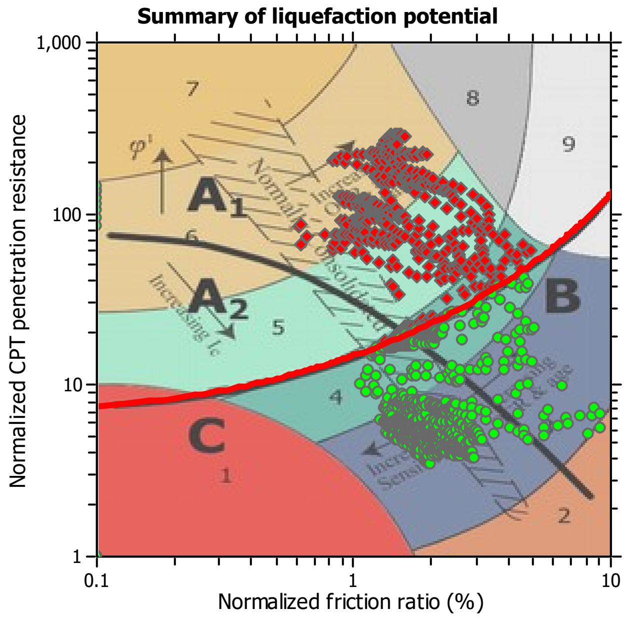
CPT file : CPT_216278

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.25	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

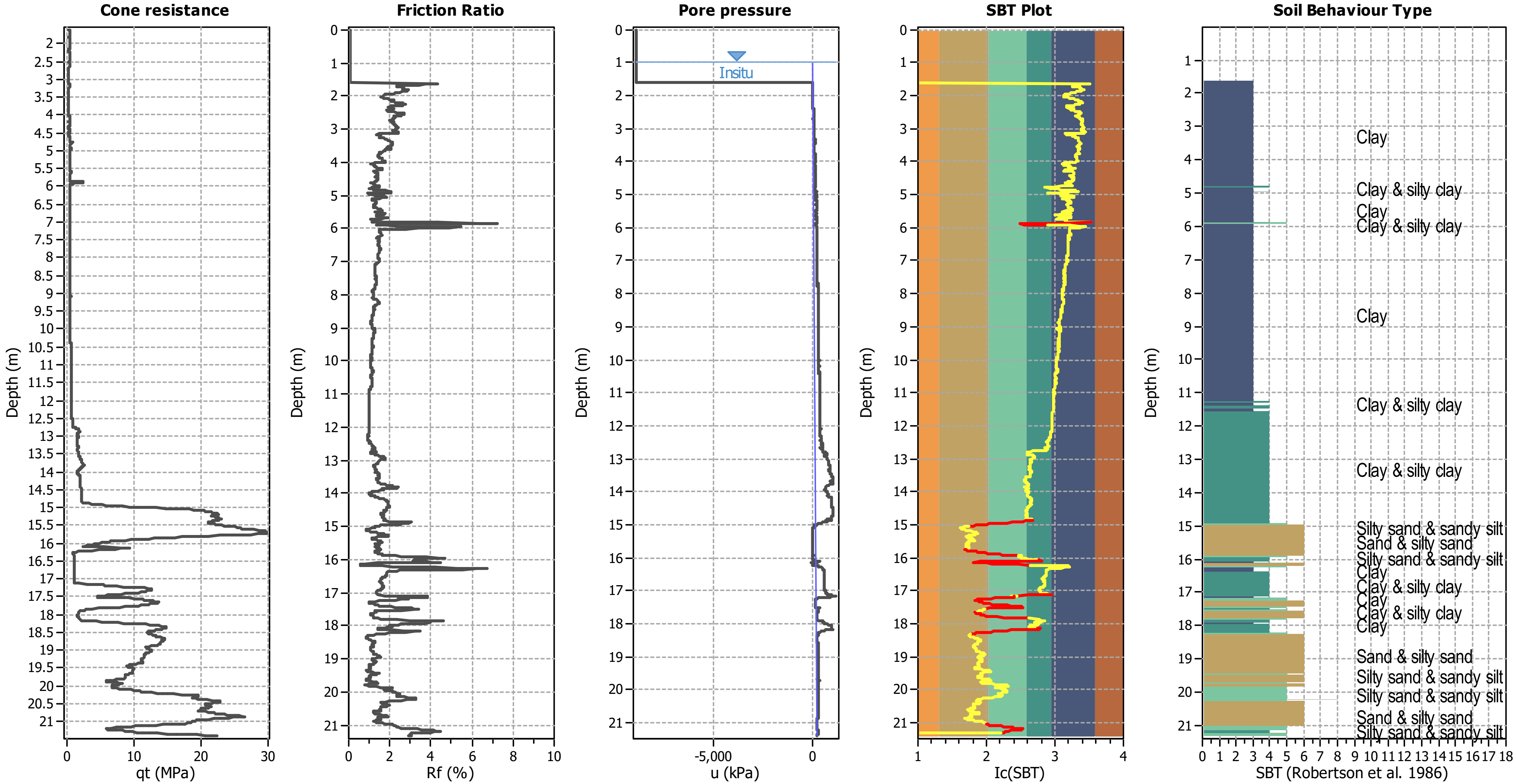






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



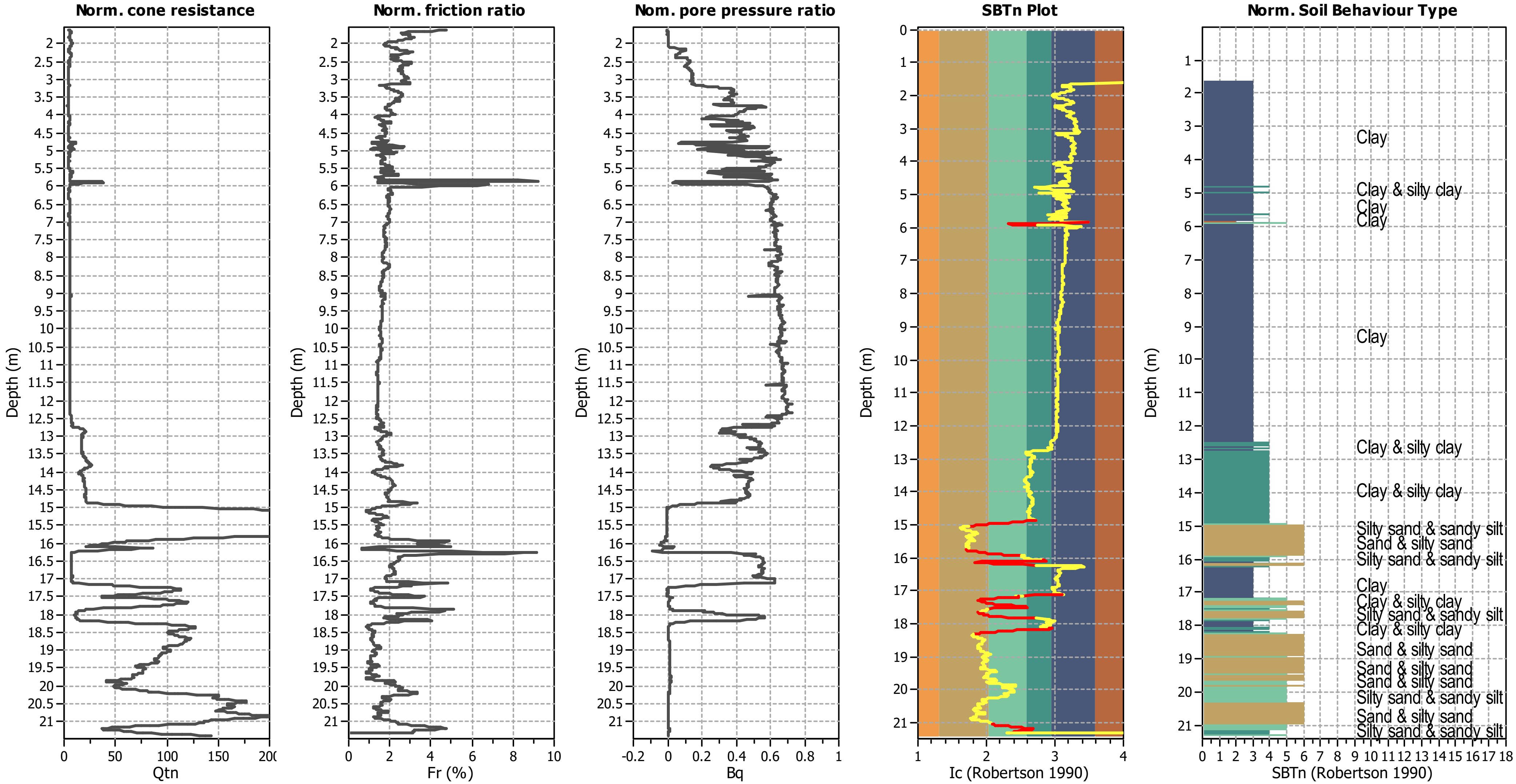
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



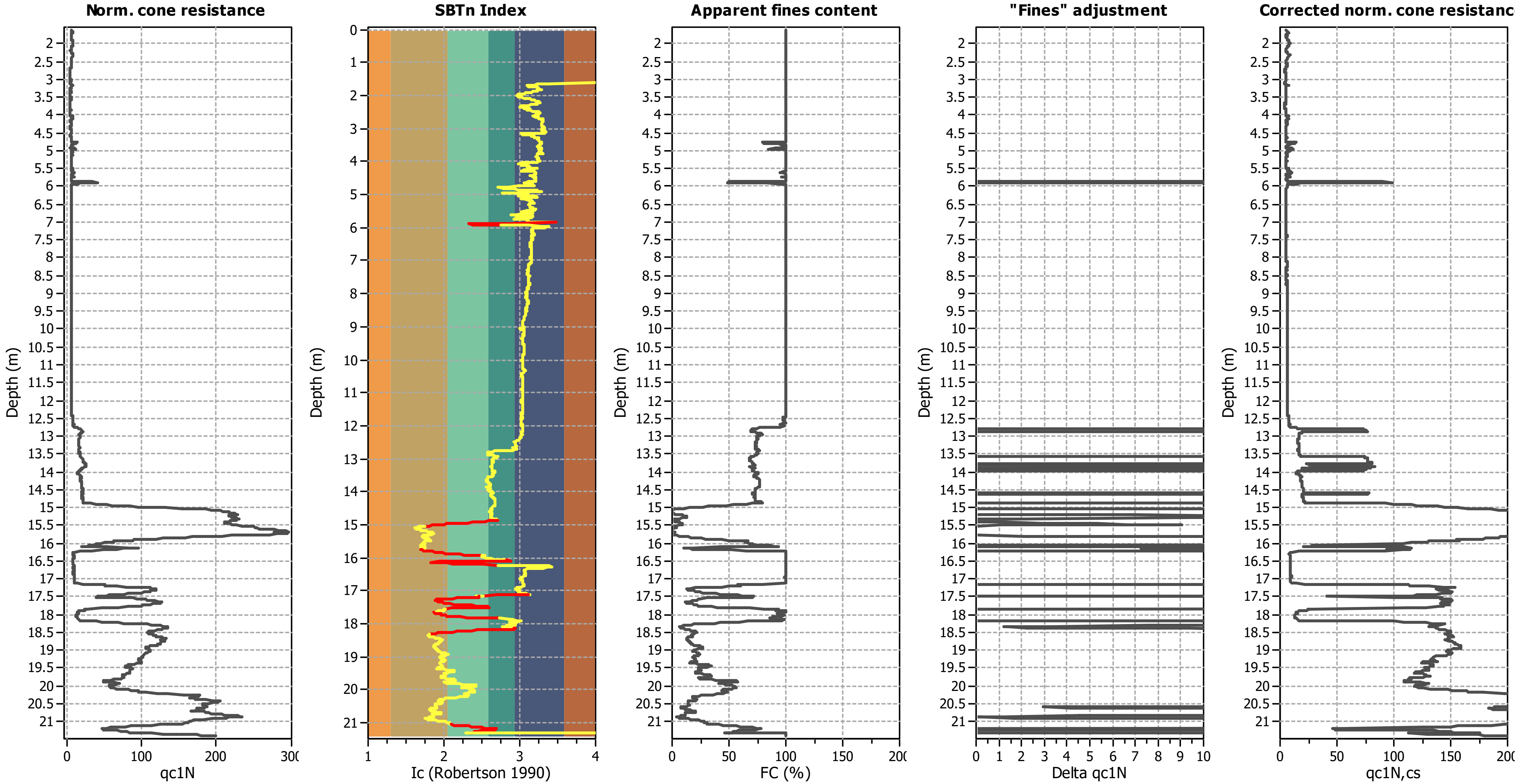
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

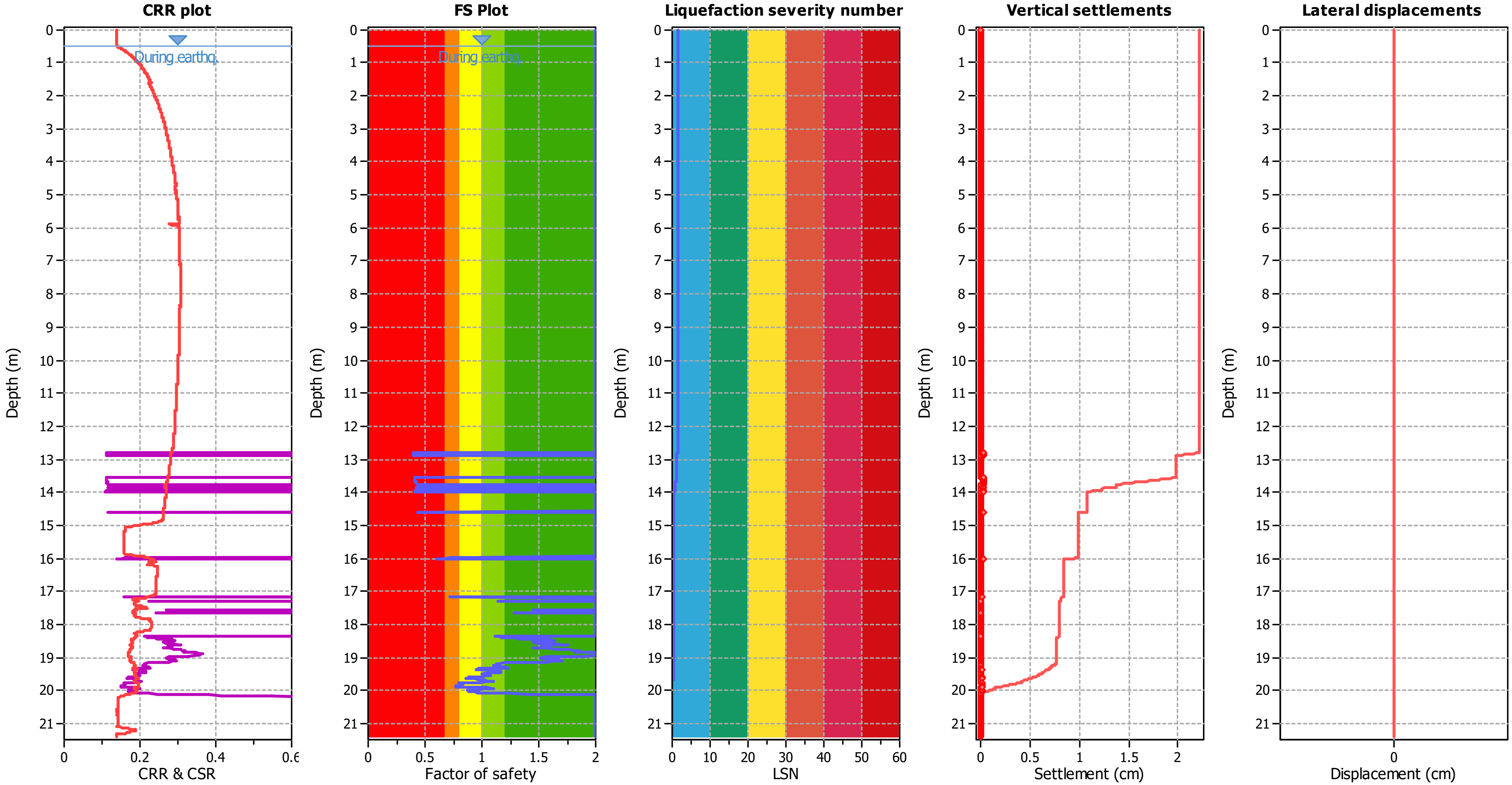
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

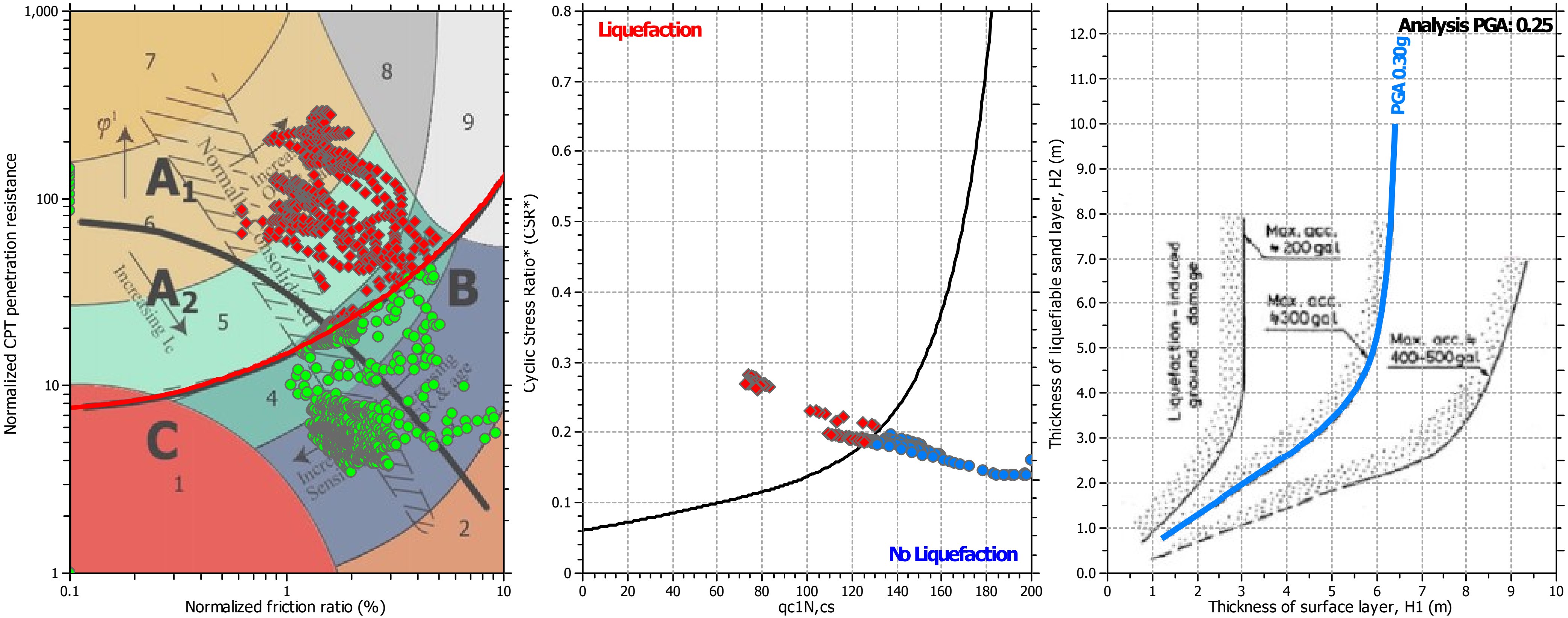
F.S. color scheme

- 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

LSN color scheme

- Severe damage
- Major expression of liquefaction
- Moderate to severe exp. of liquefaction
- Moderate expression of liquefaction
- Minor expression of liquefaction
- Little to no expression of liquefaction

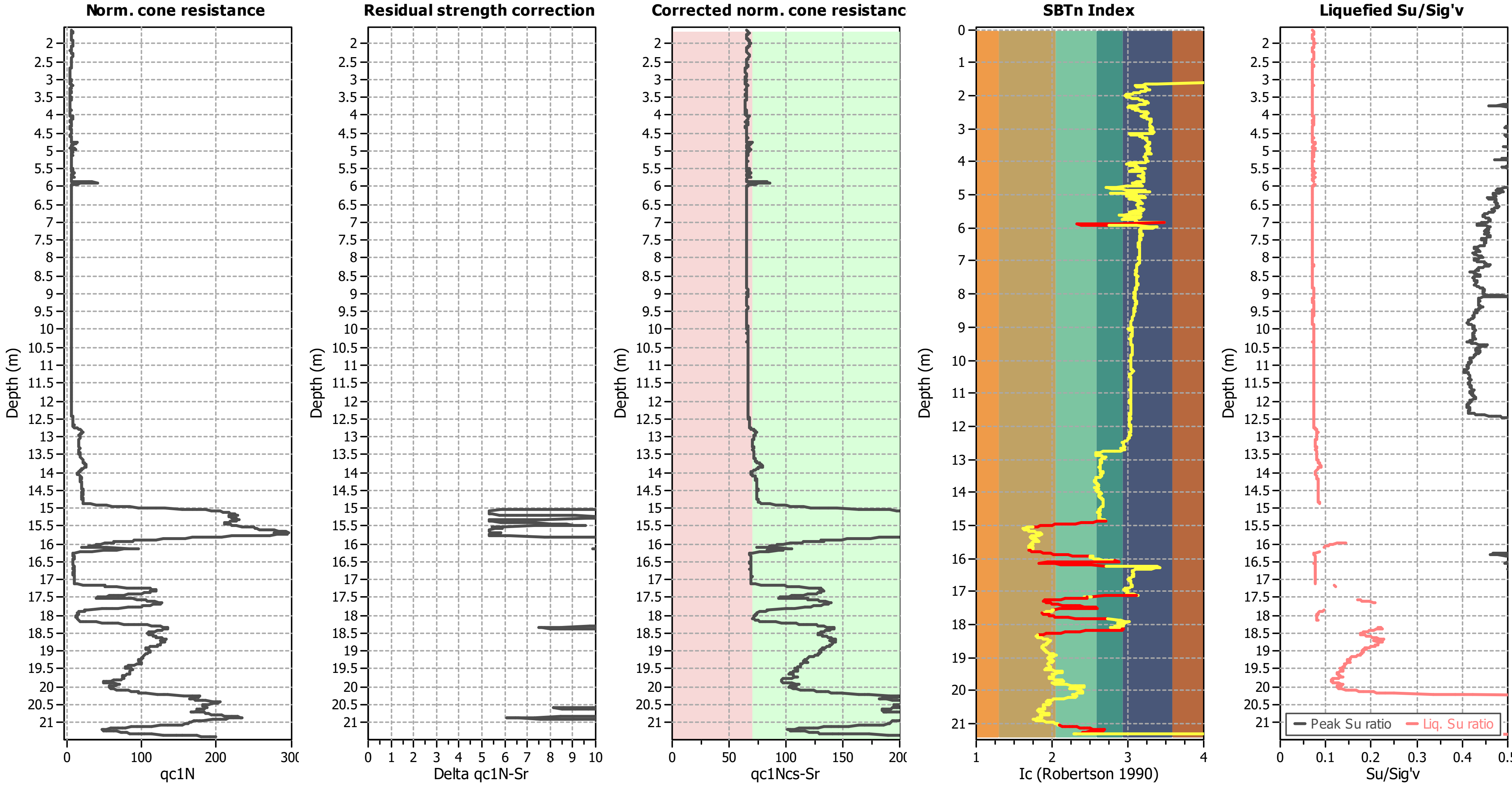
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

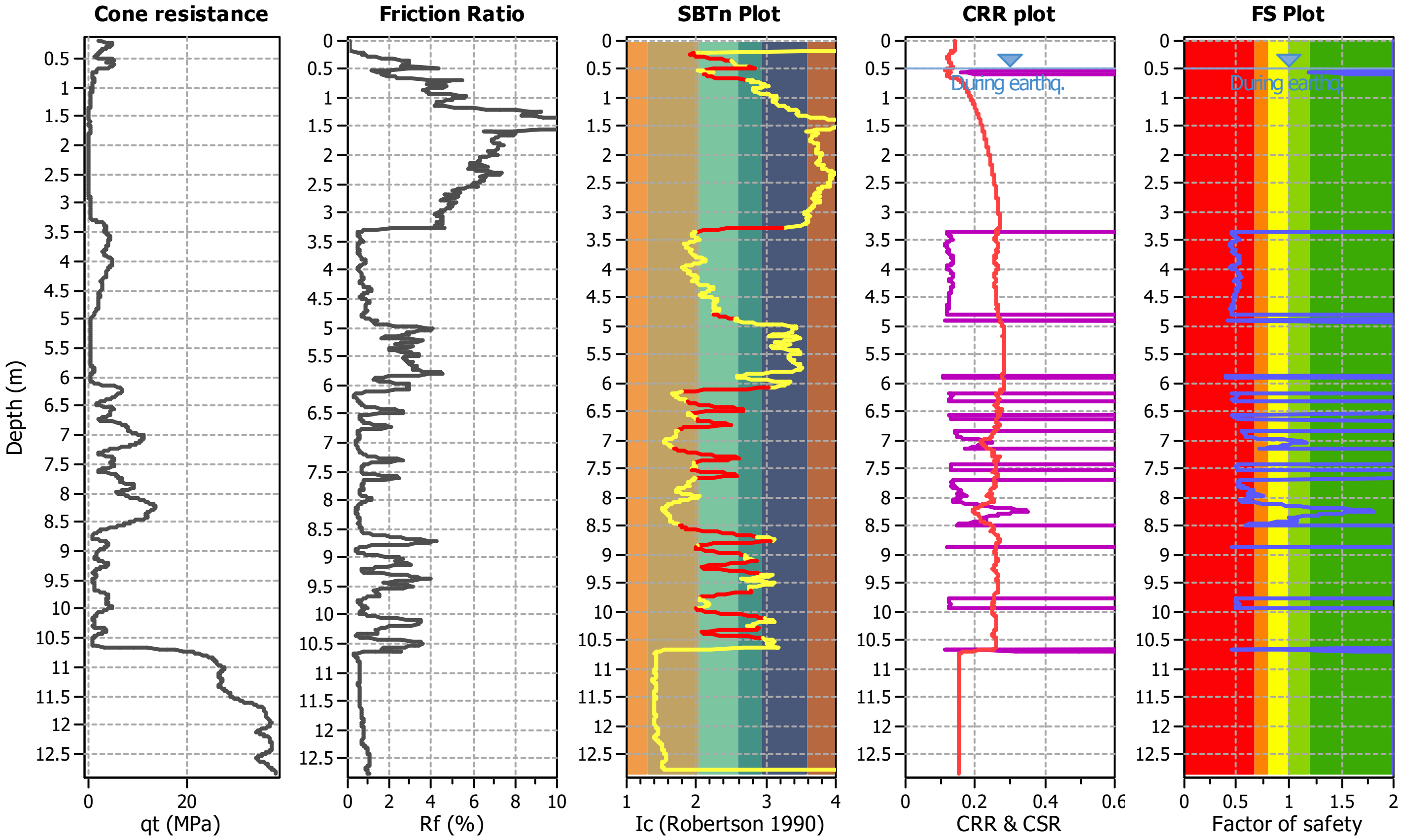
LIQUEFACTION ANALYSIS REPORT

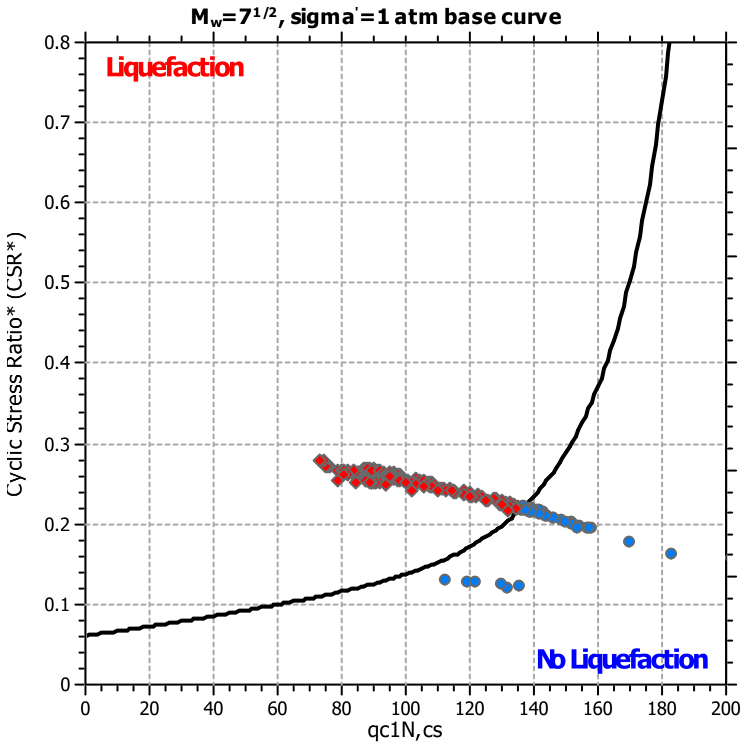
Project title : **Location :**

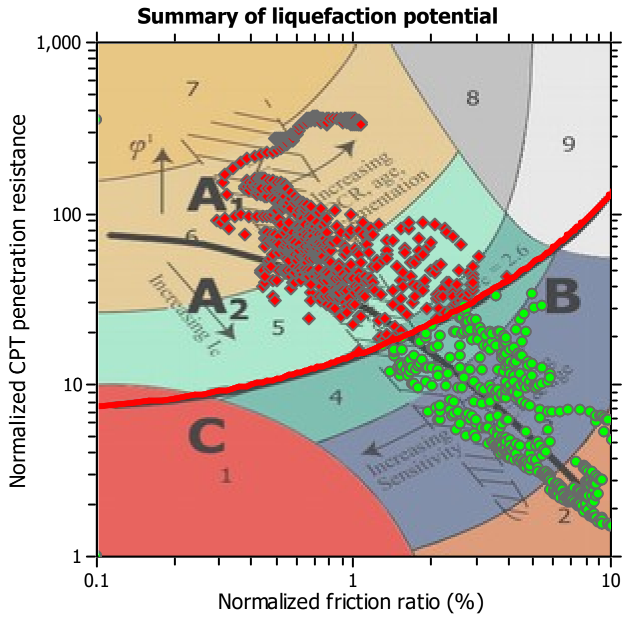
CPT file : CPT_185470

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.25	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

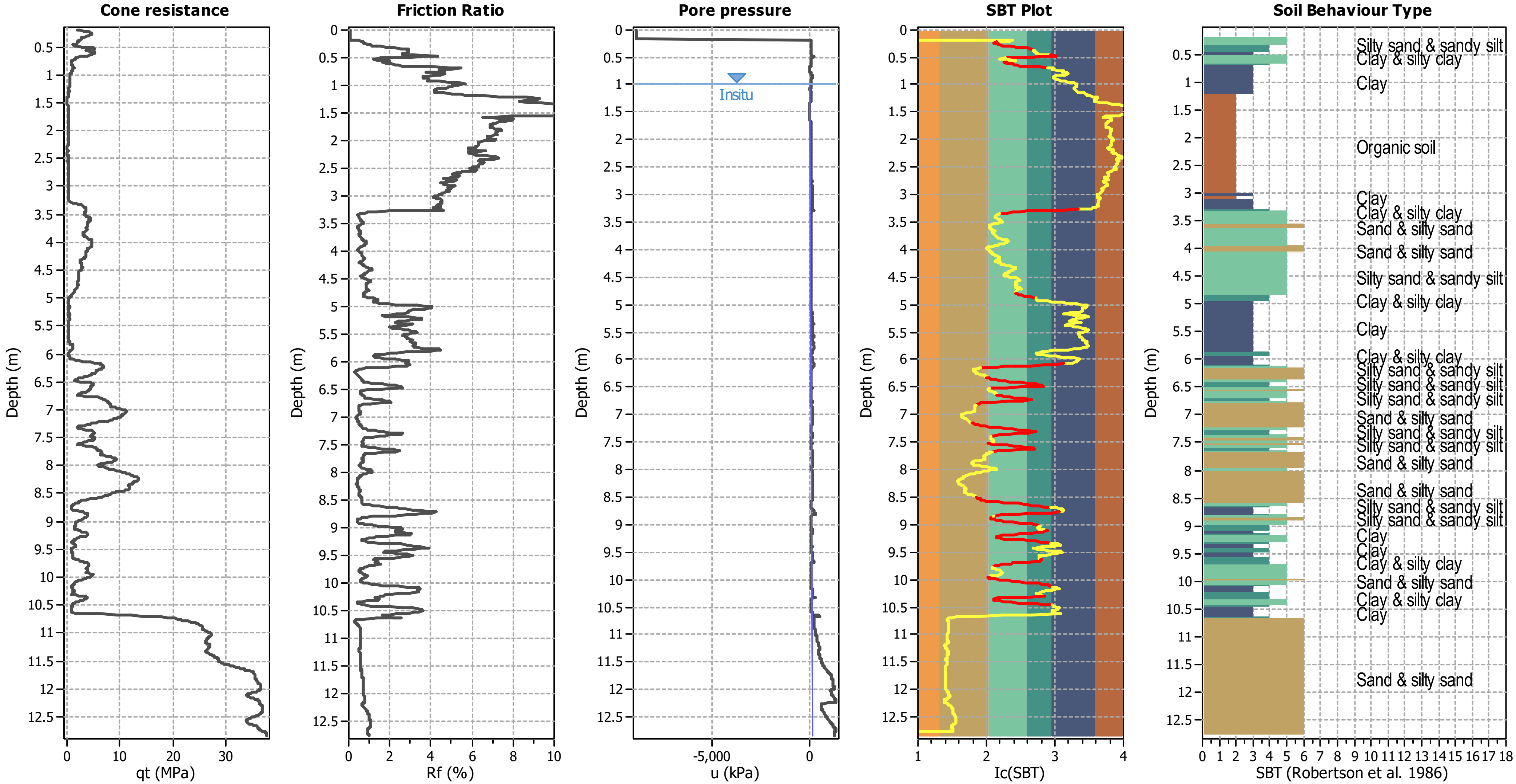






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



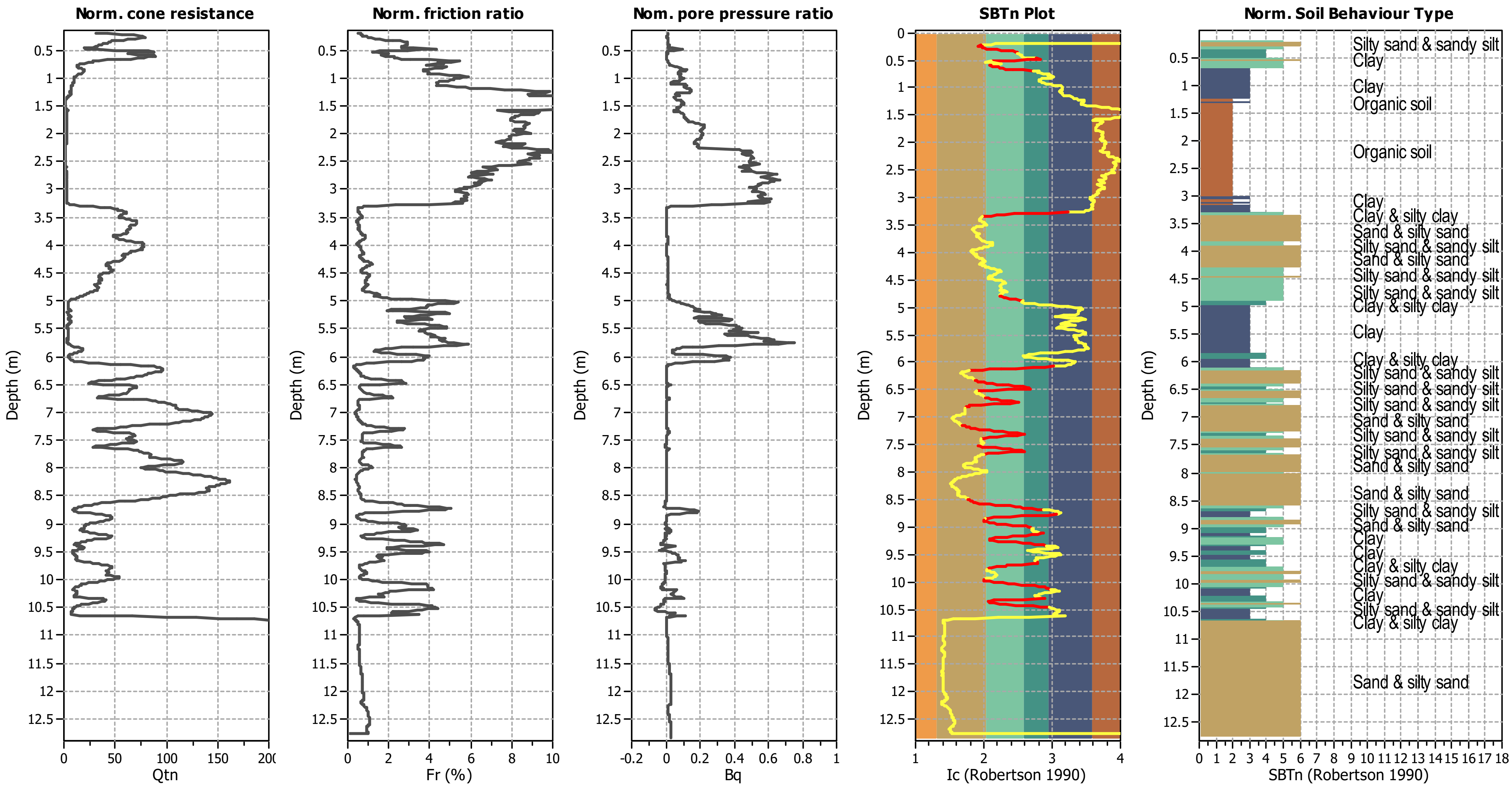
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



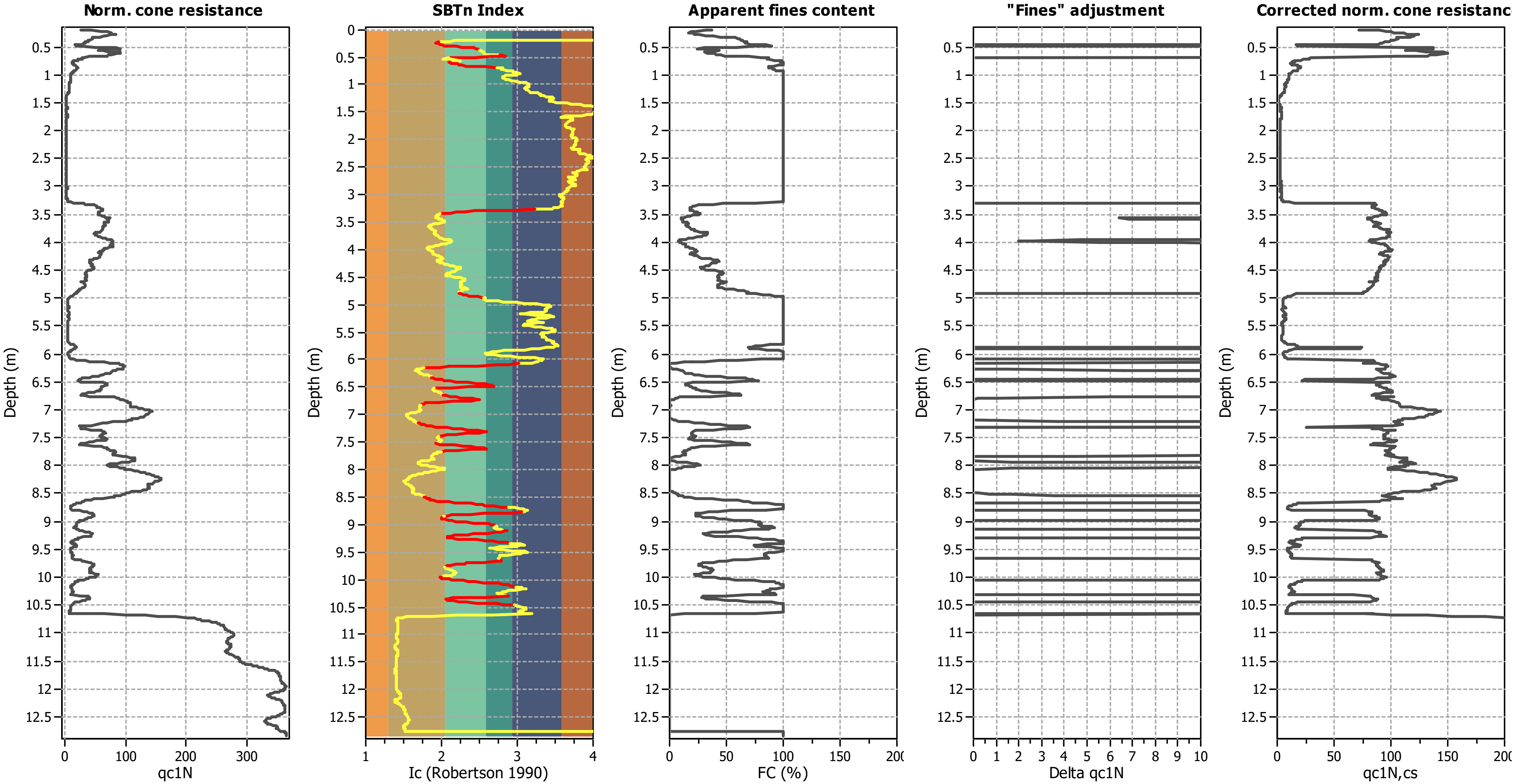
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

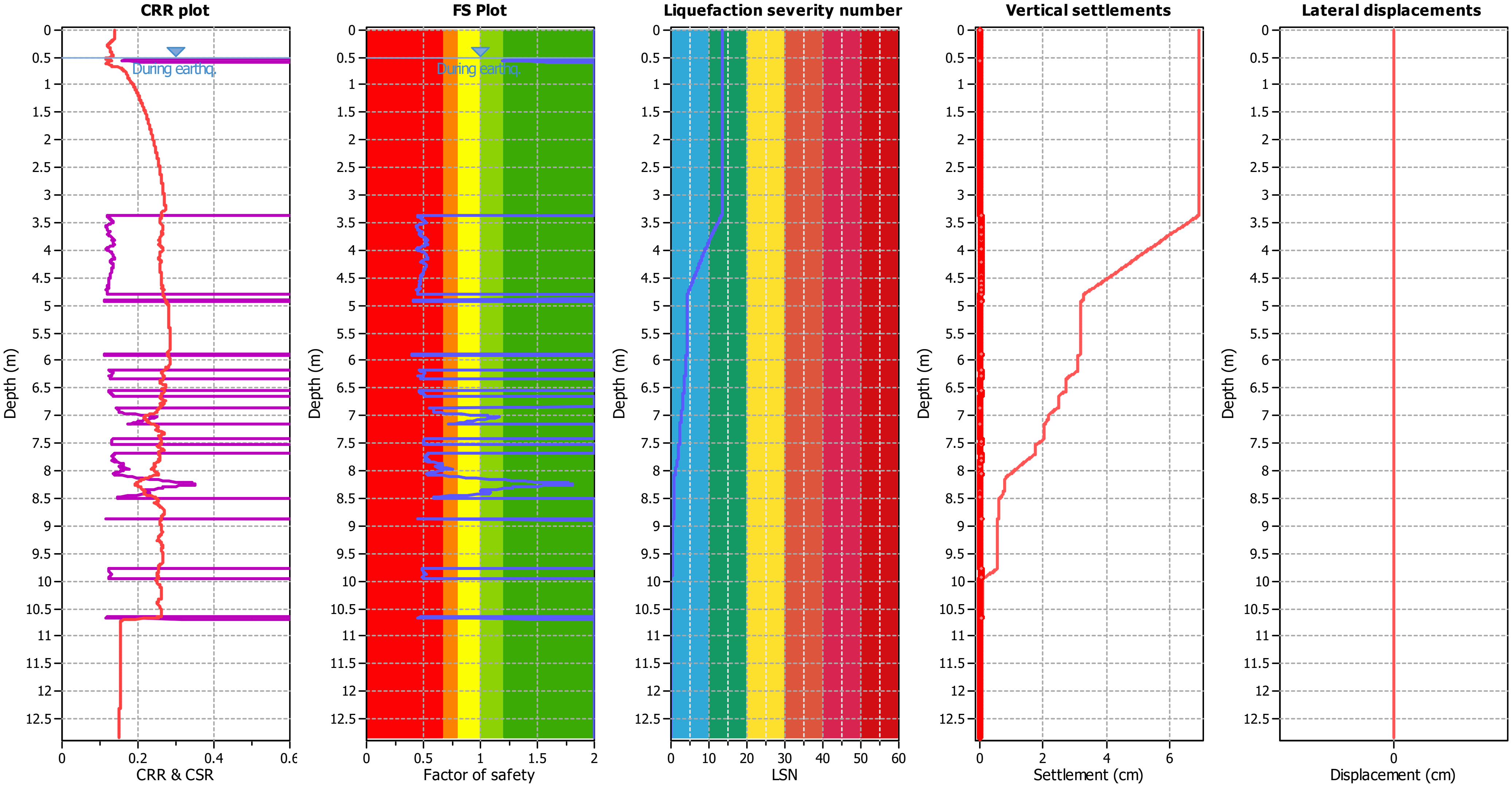
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

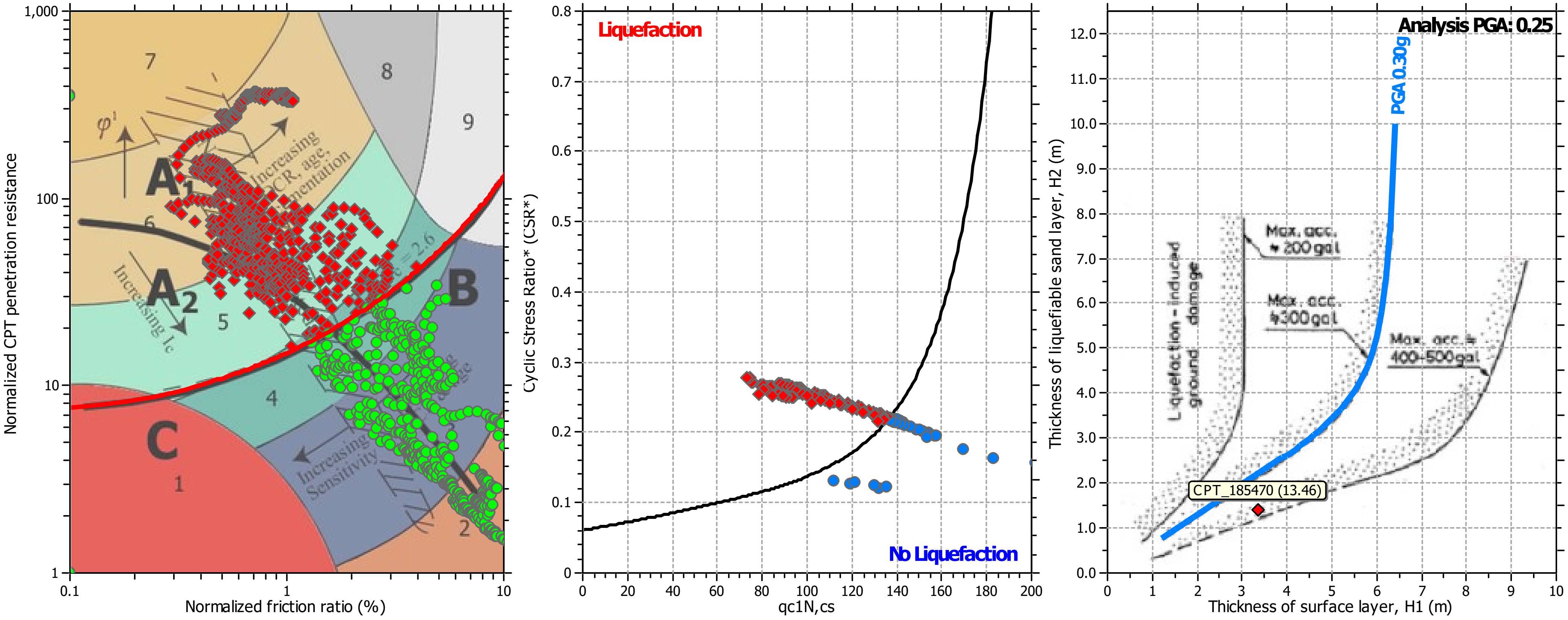
F.S. color scheme

- 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

LSN color scheme

- Severe damage
- Major expression of liquefaction
- Moderate to severe exp. of liquefaction
- Moderate expression of liquefaction
- Minor expression of liquefaction
- Little to no expression of liquefaction

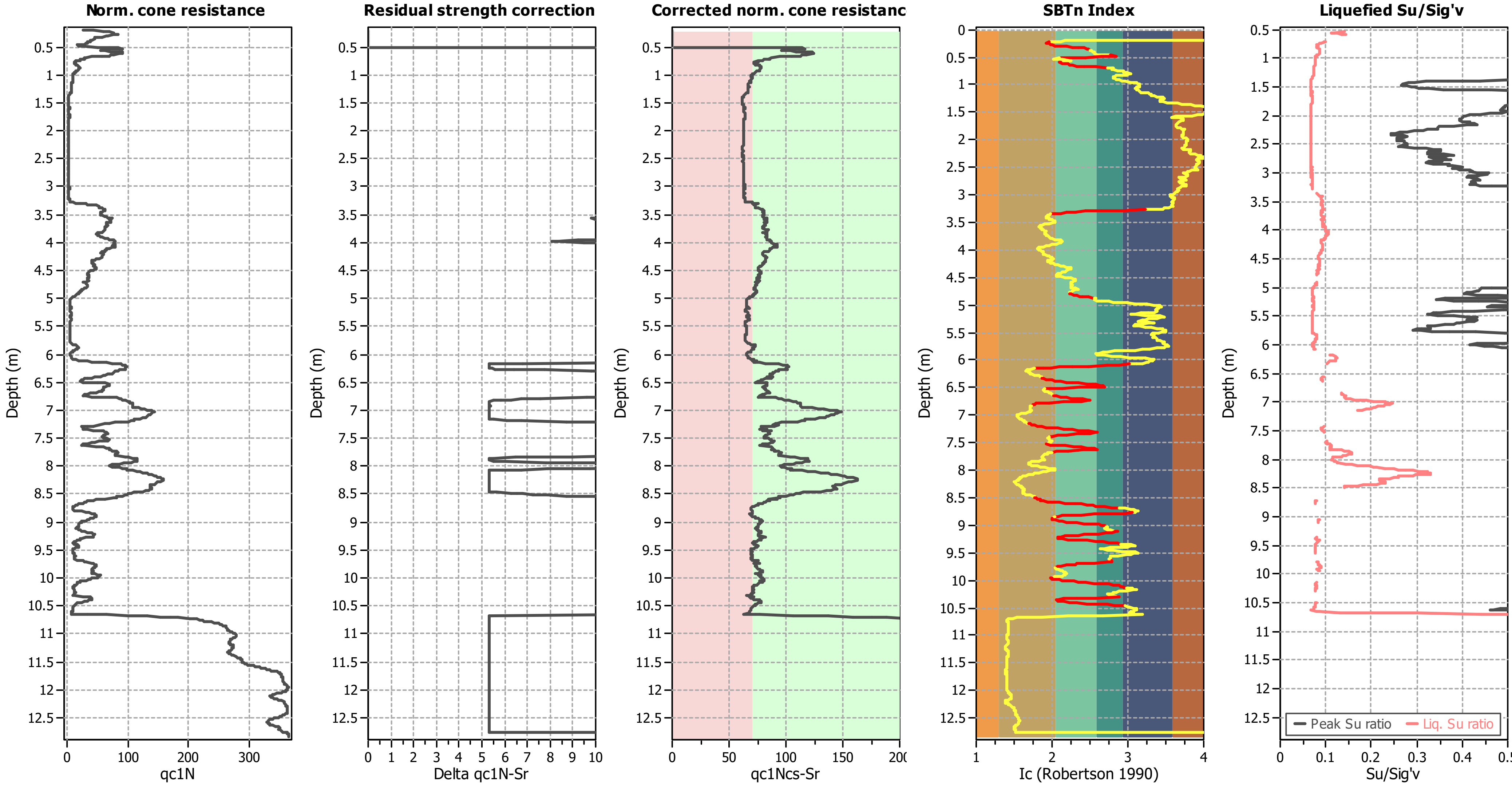
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

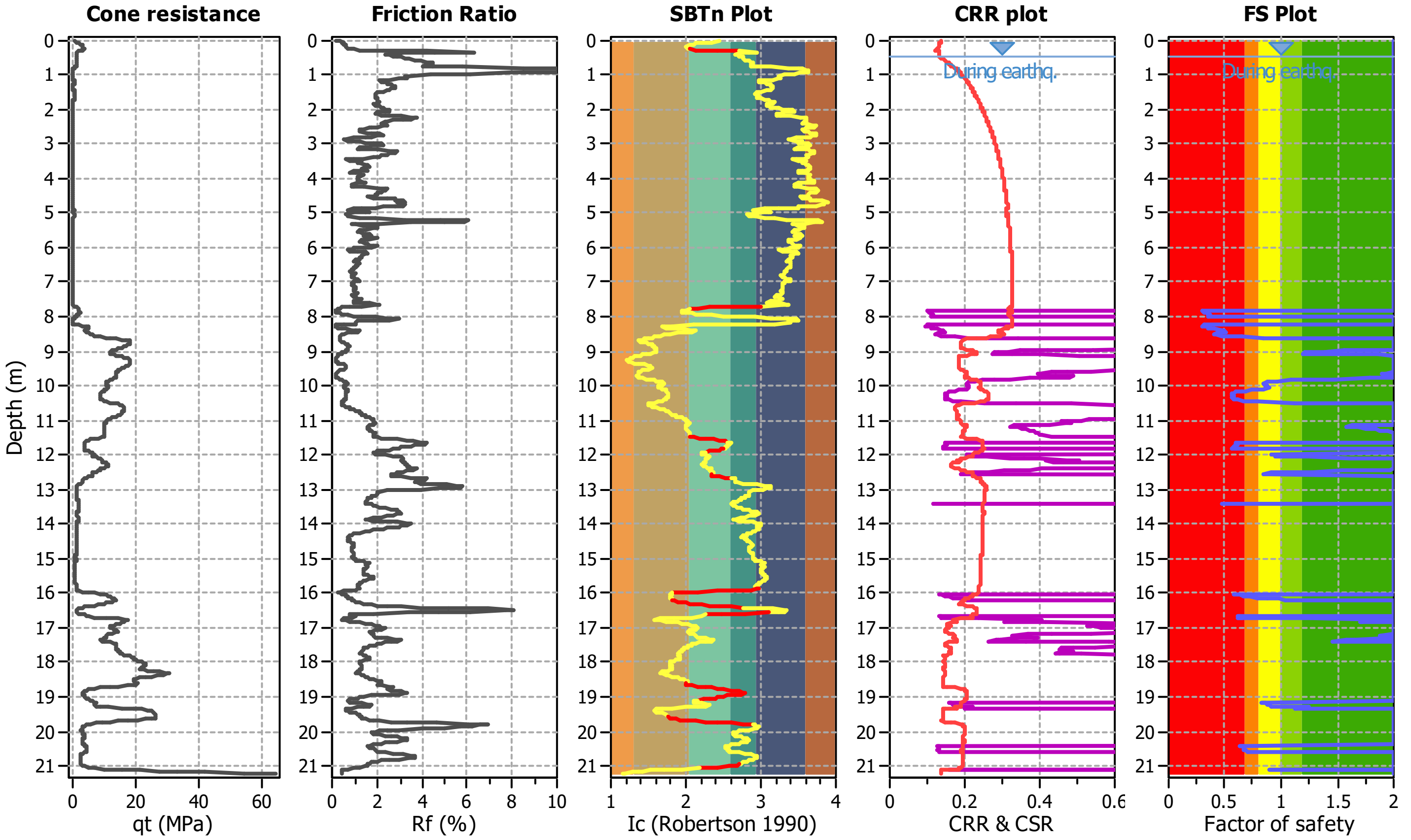
LIQUEFACTION ANALYSIS REPORT

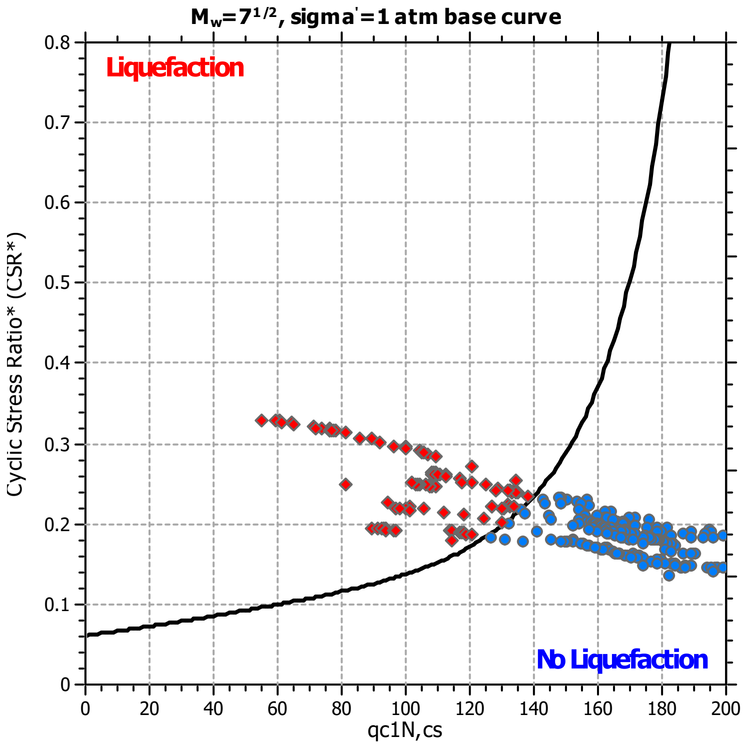
Project title : **Location :**

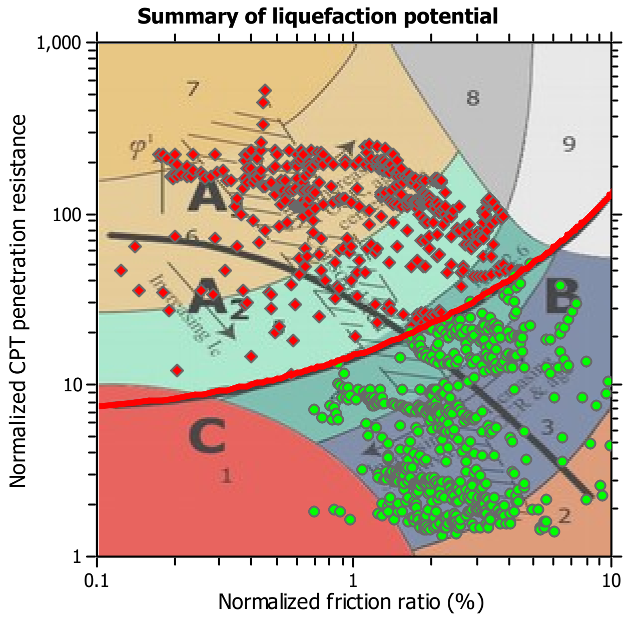
CPT file : CPT_125217

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.25	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

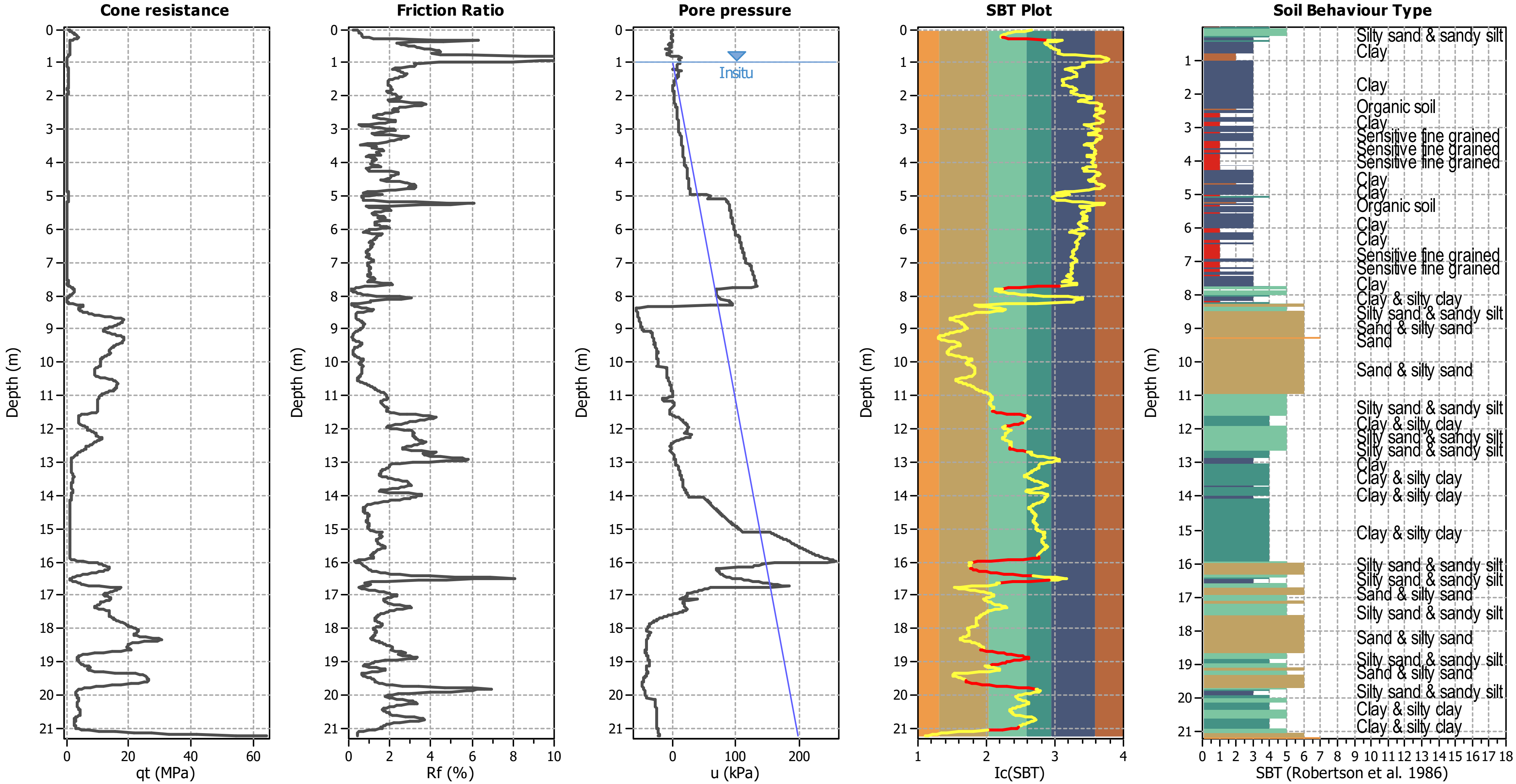






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



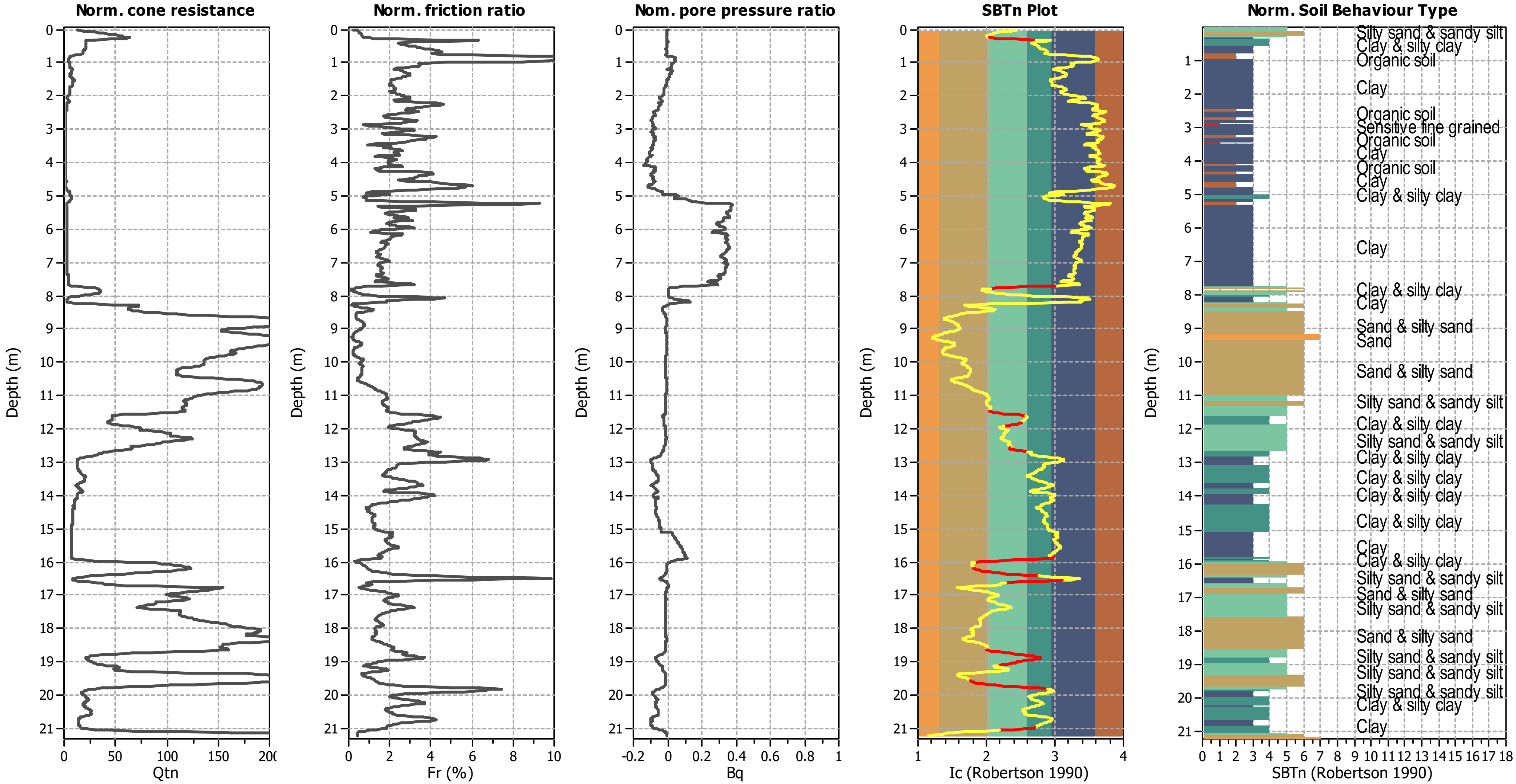
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



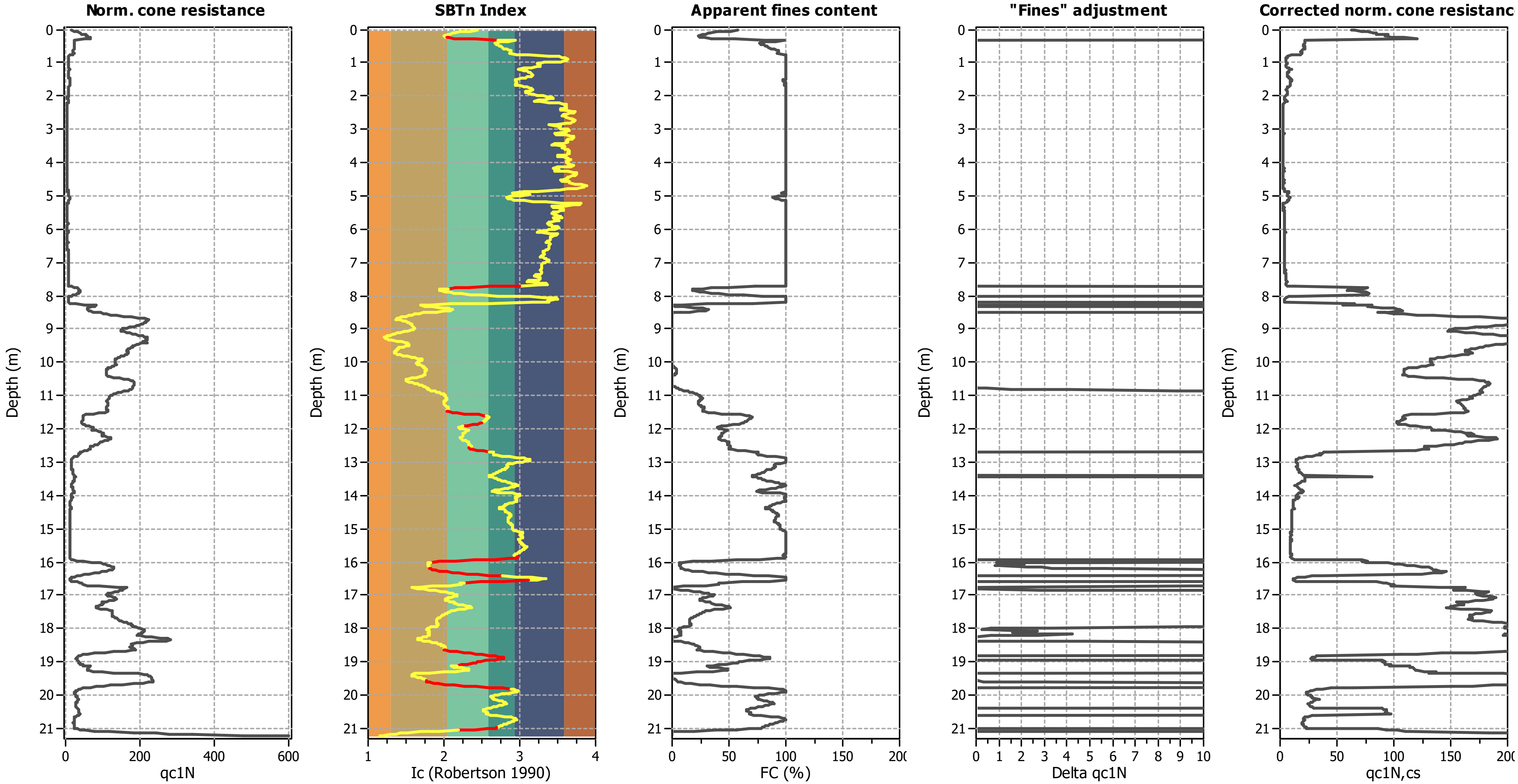
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

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

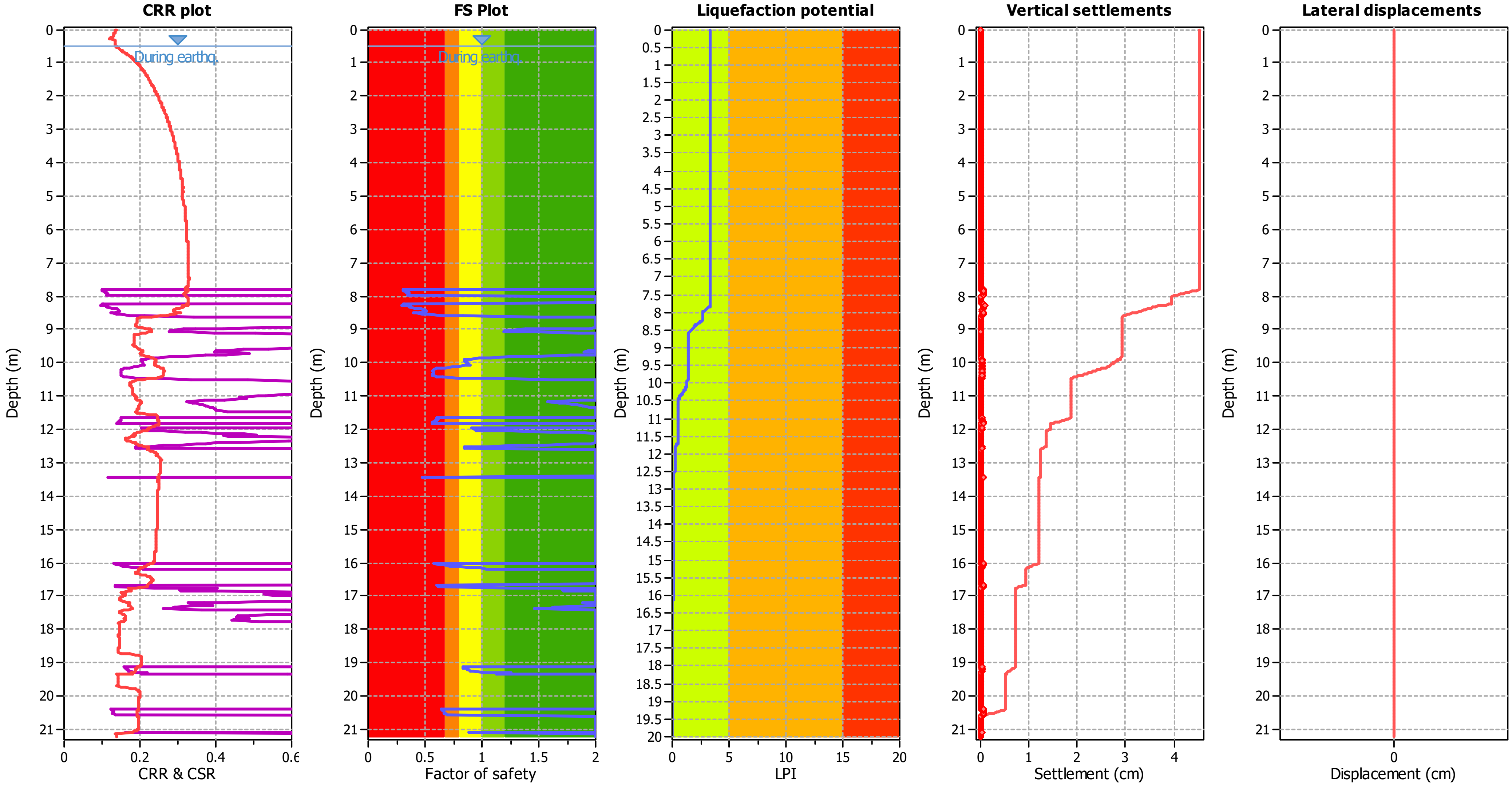
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

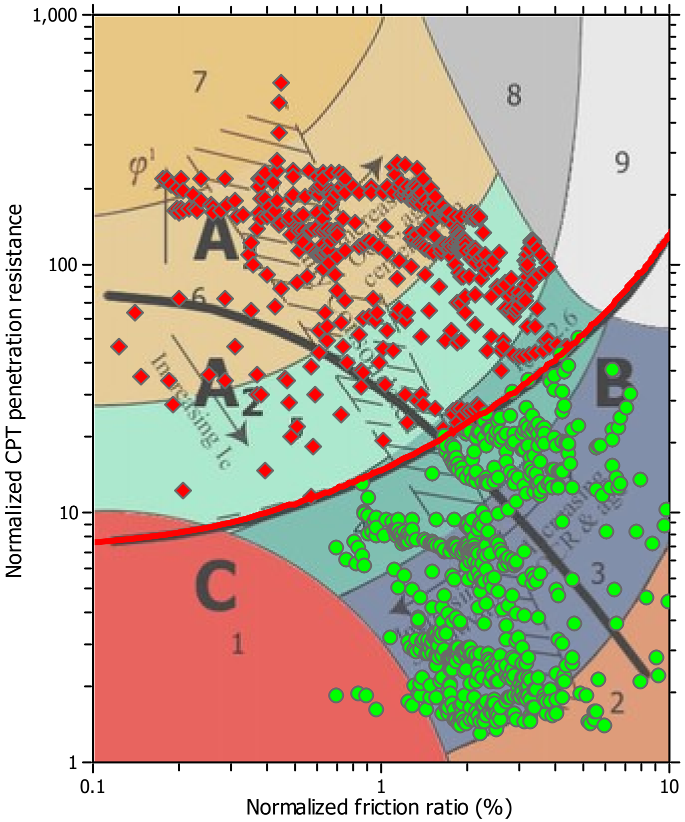
F.S. color scheme

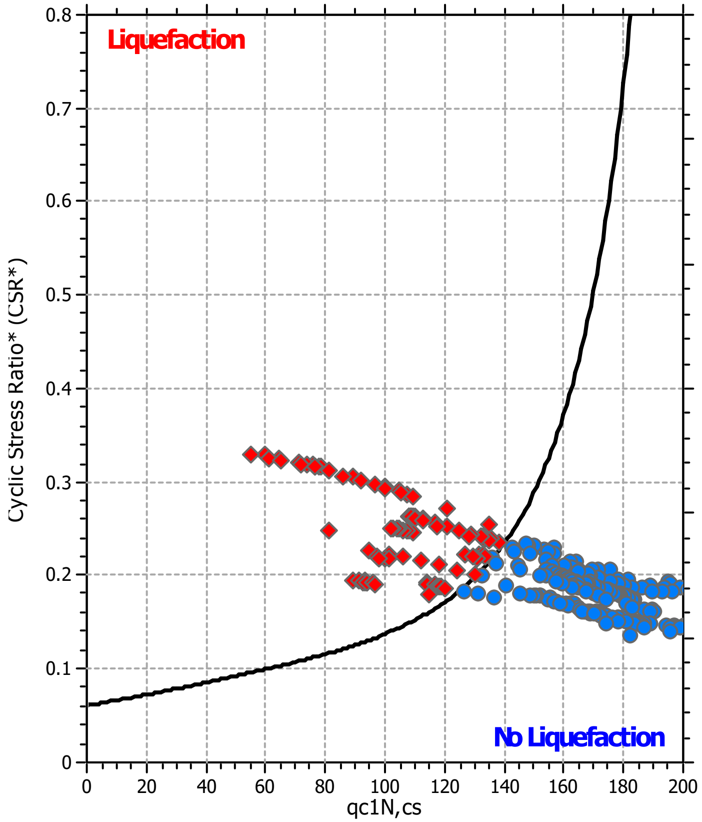
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

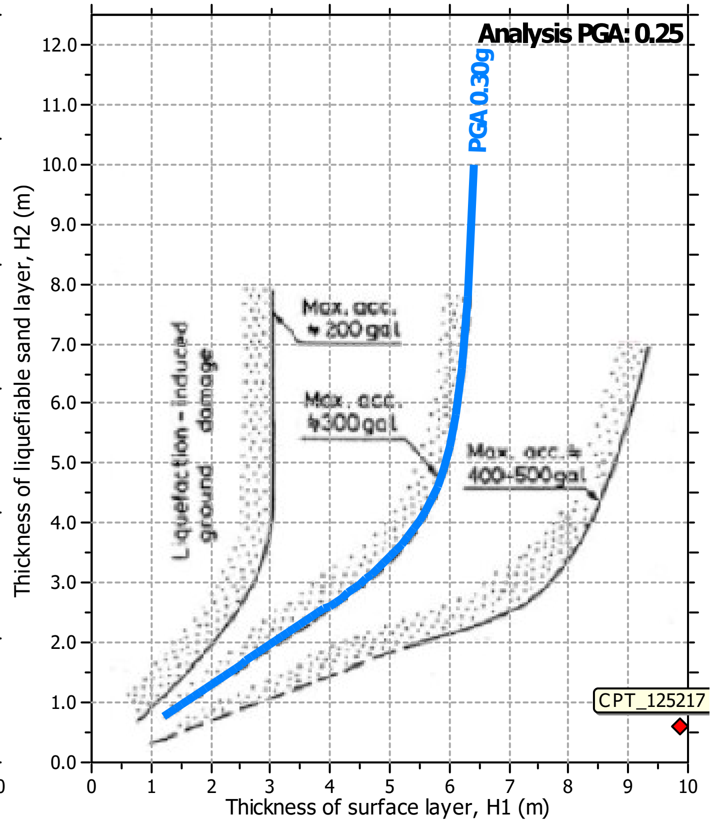
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



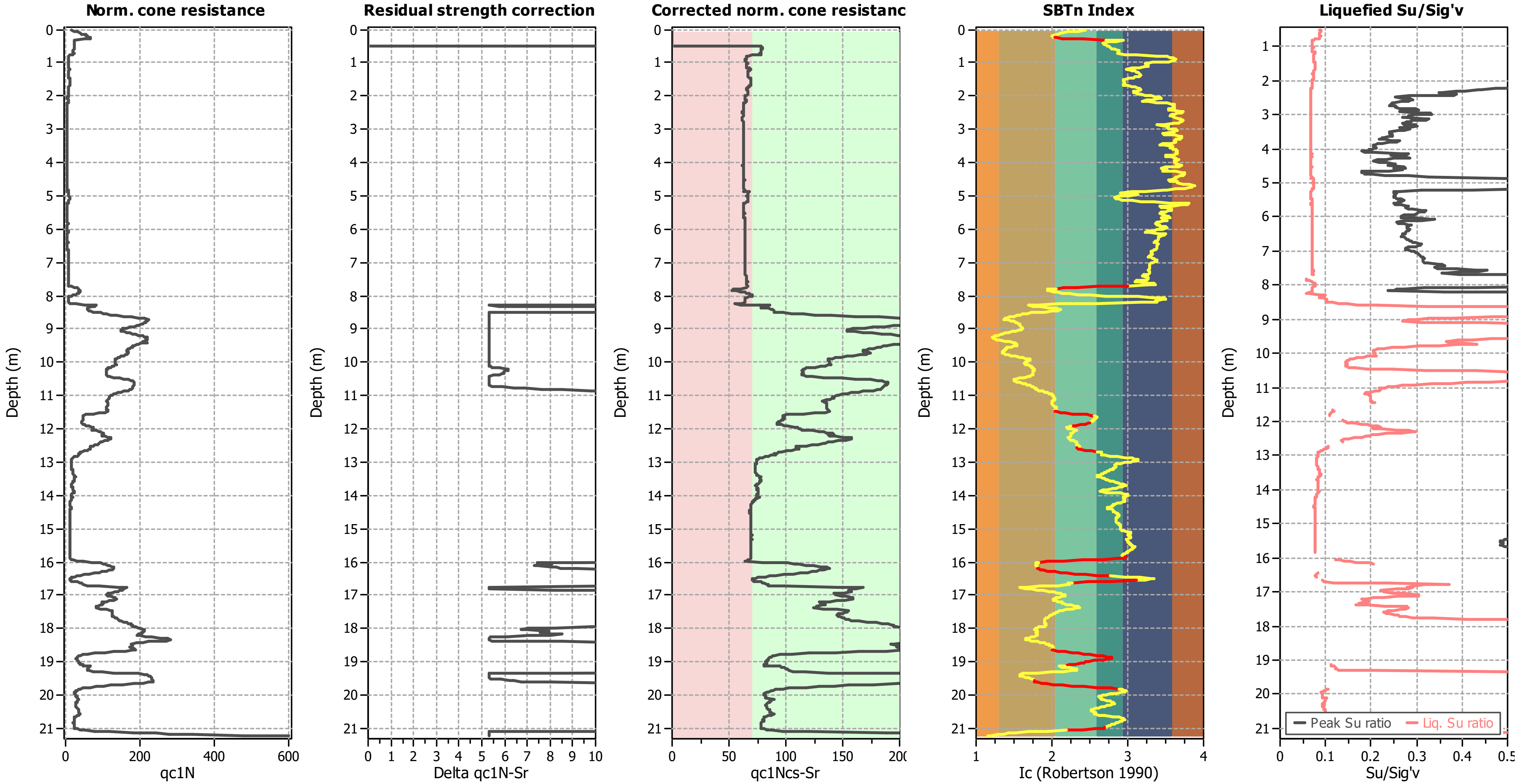




Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

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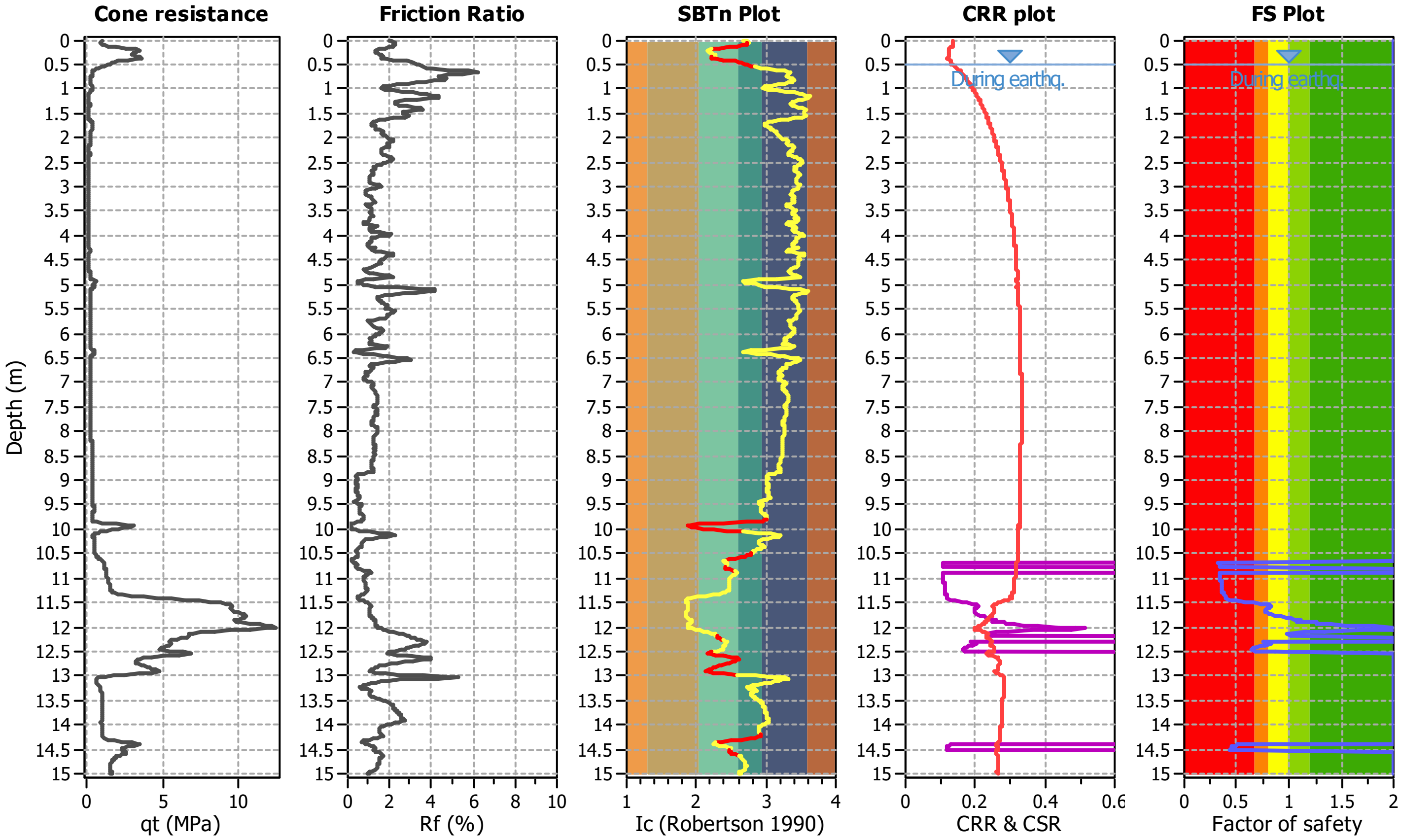
LIQUEFACTION ANALYSIS REPORT

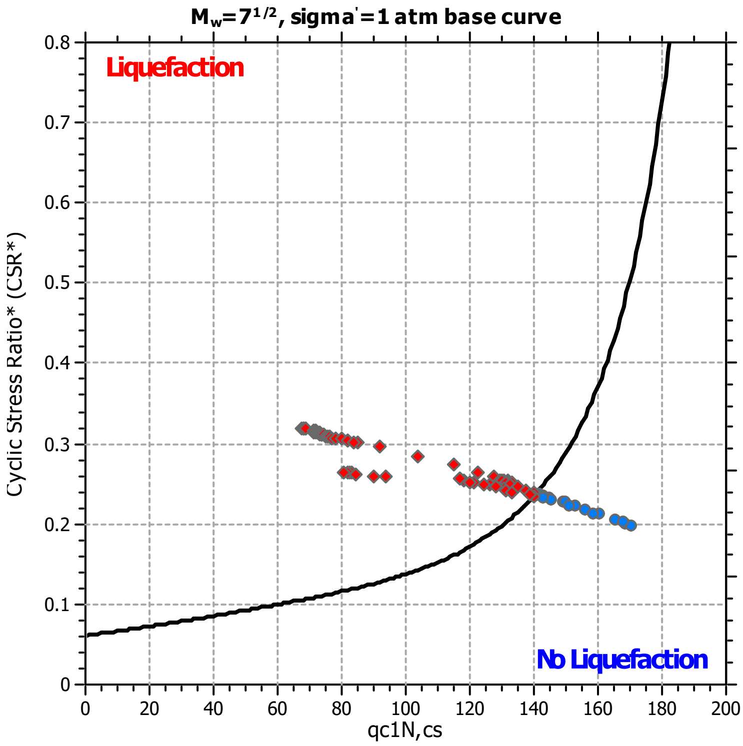
Project title : **Location :**

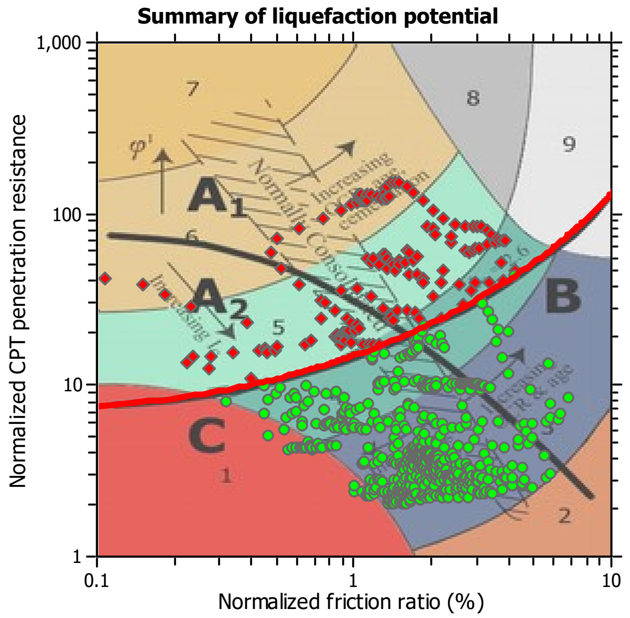
CPT file : CPT_125219

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.25	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

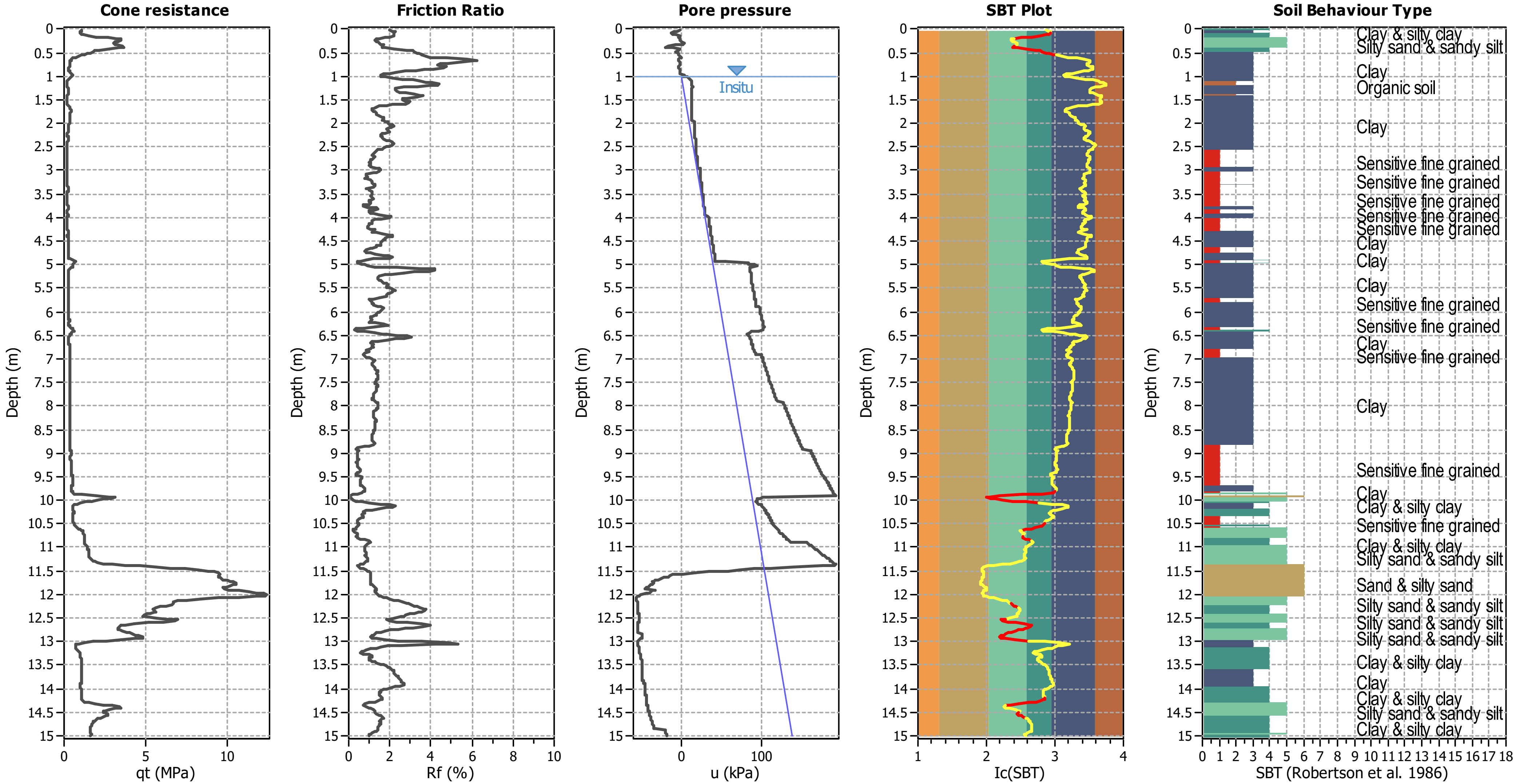






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



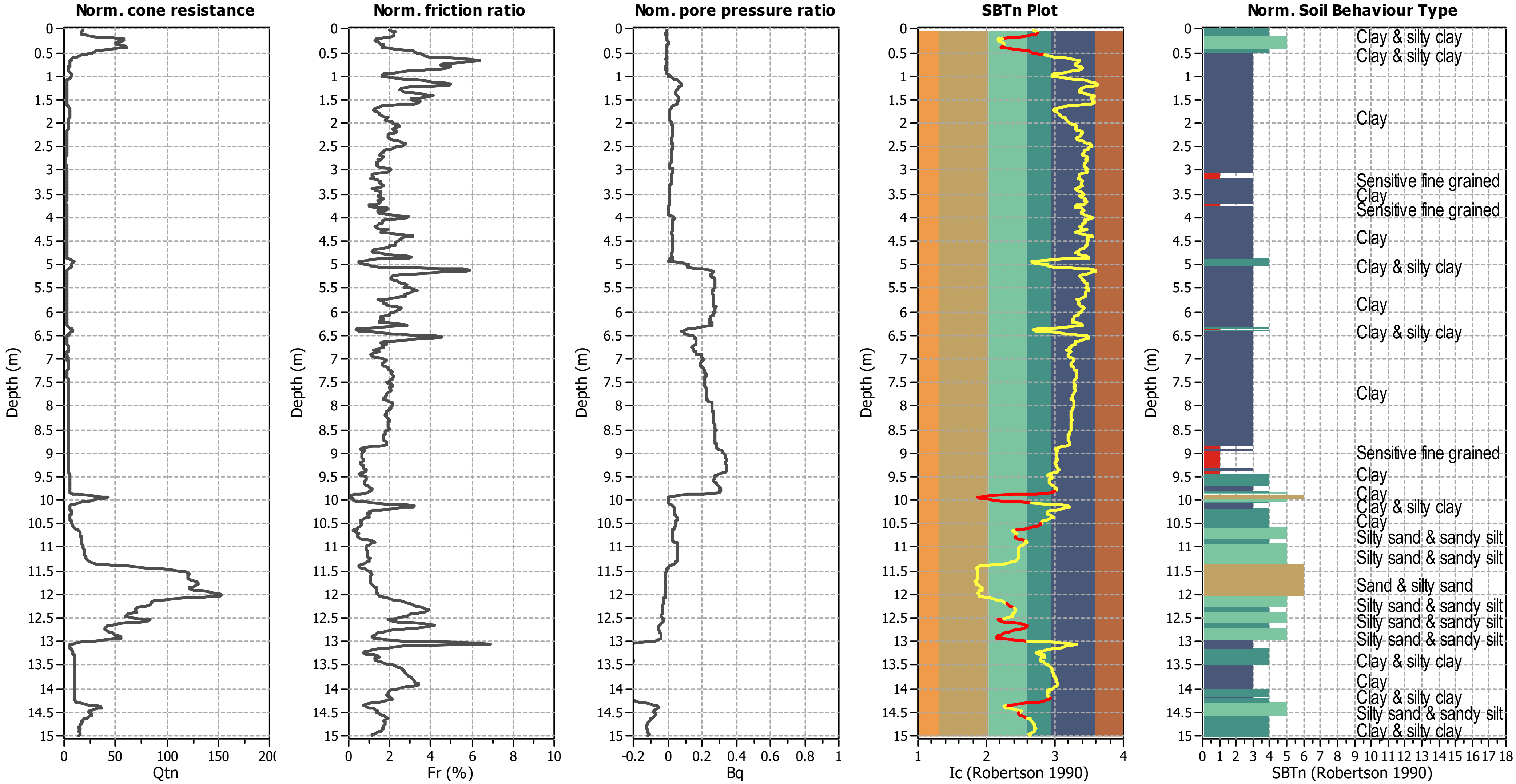
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



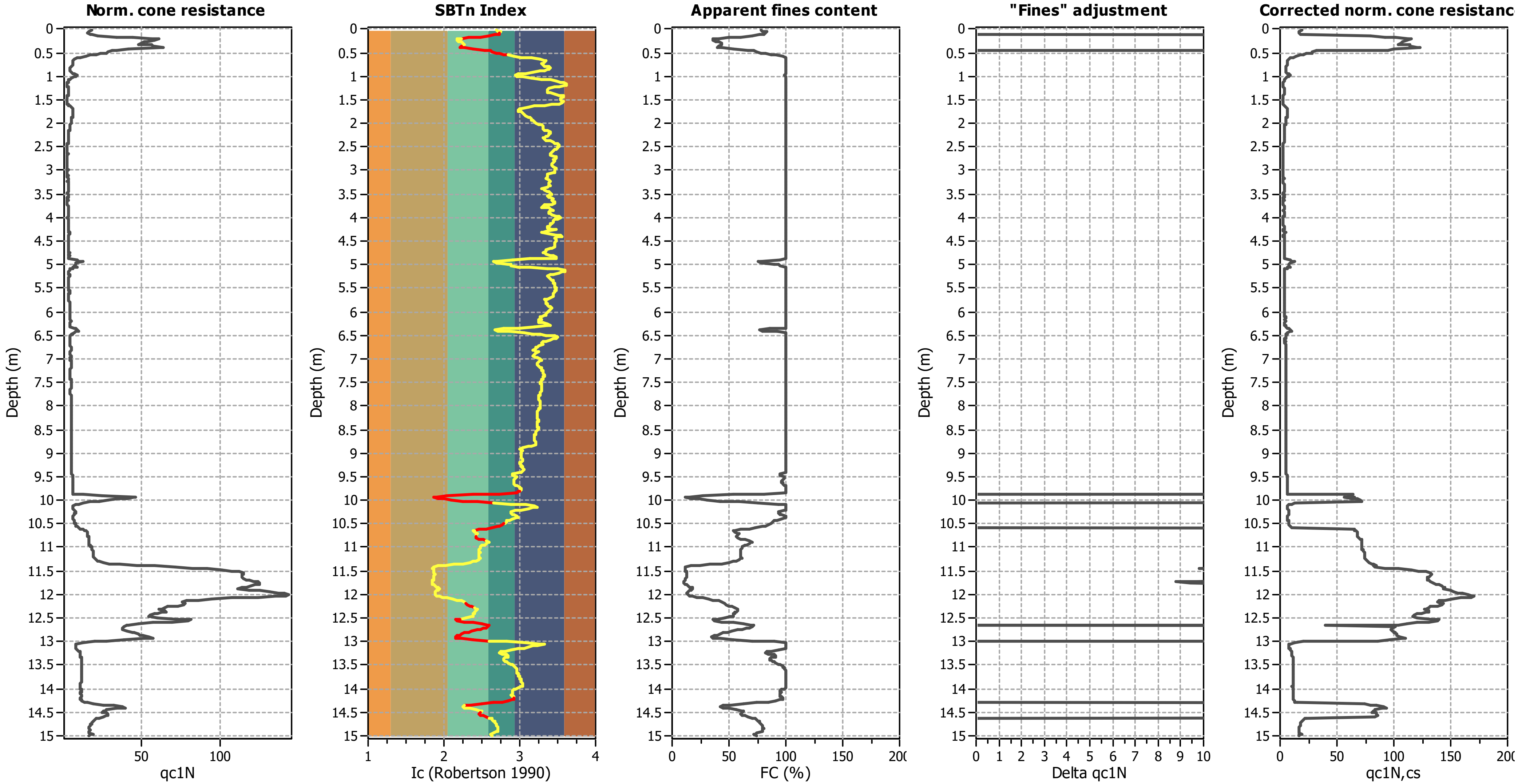
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

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

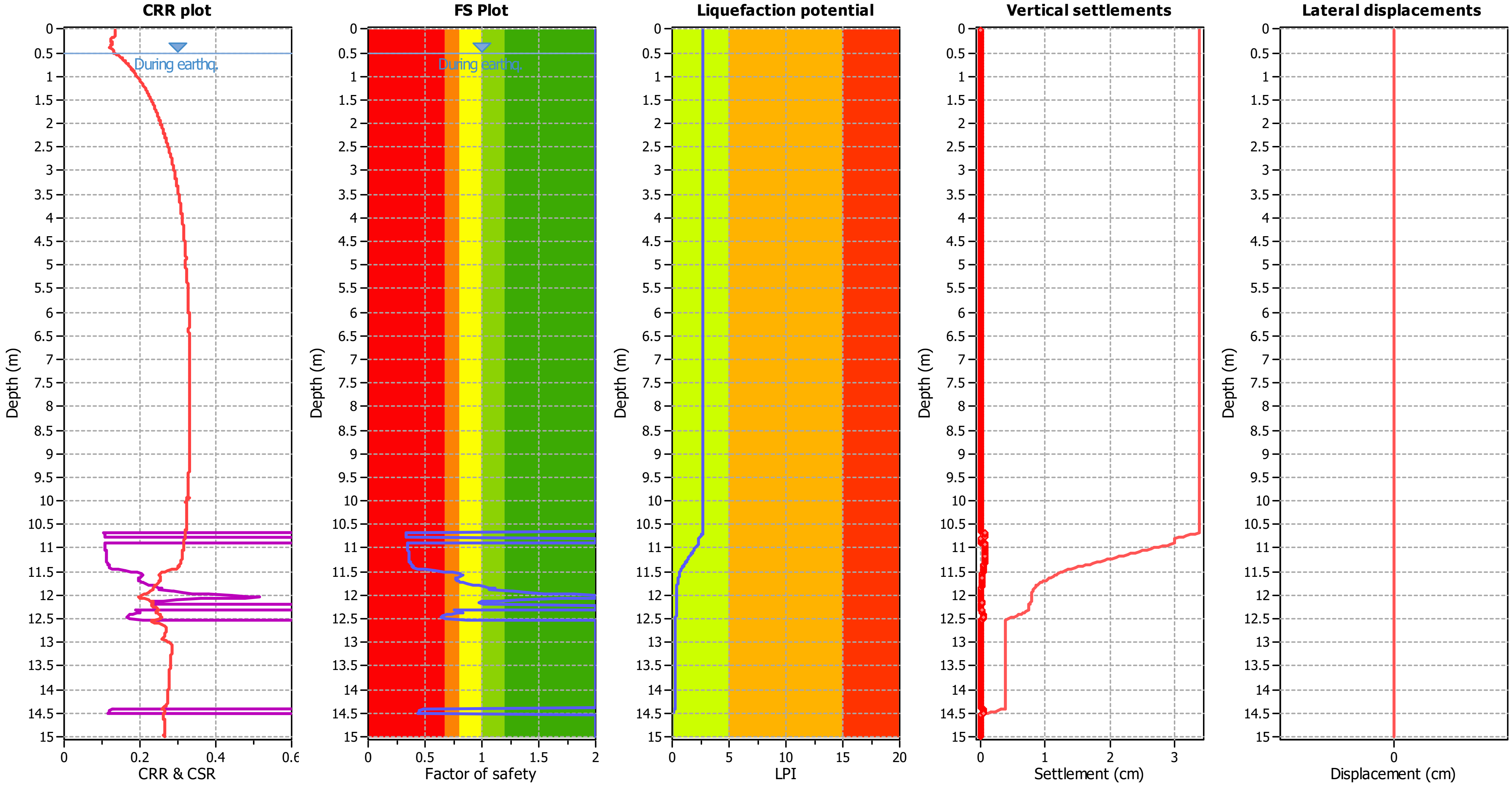
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

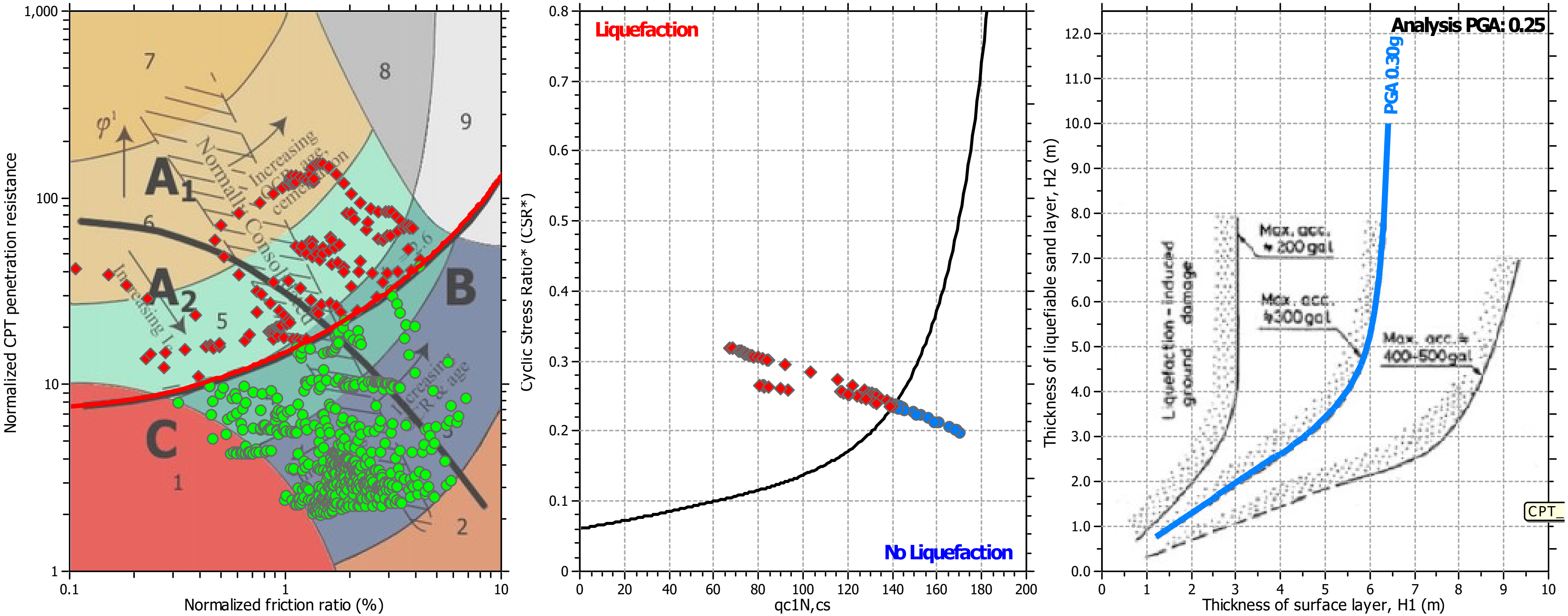
F.S. color scheme

- 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

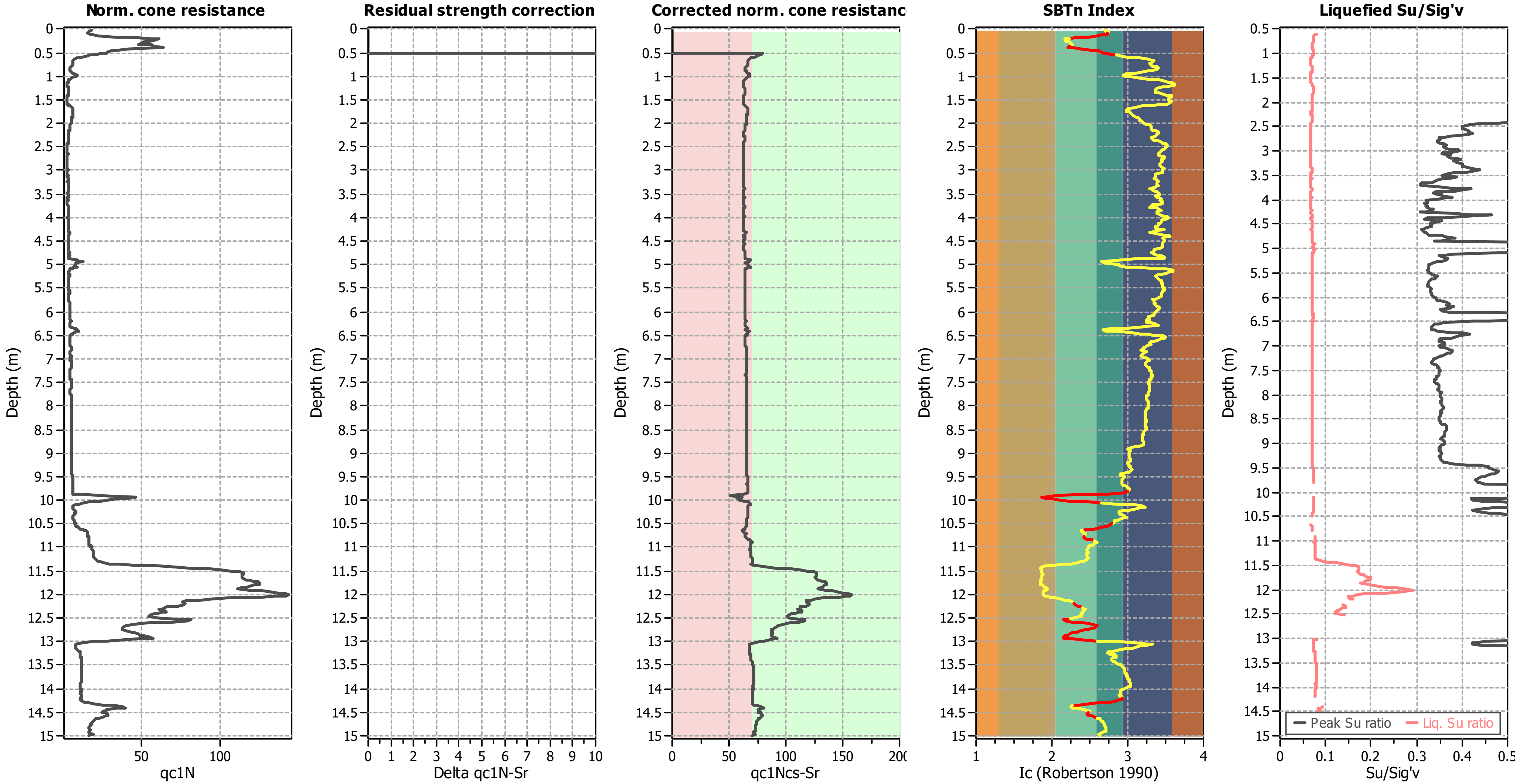
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

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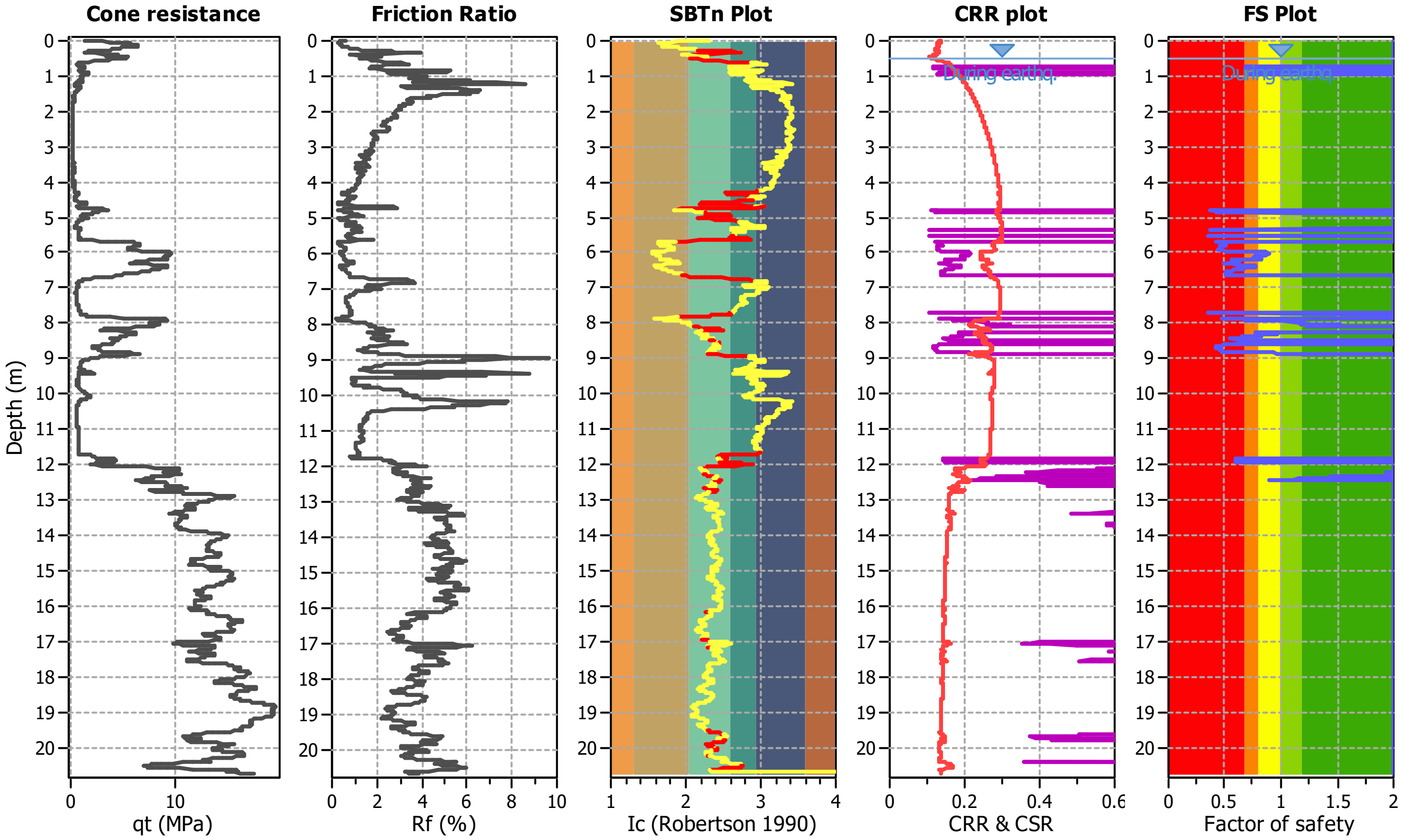
LIQUEFACTION ANALYSIS REPORT

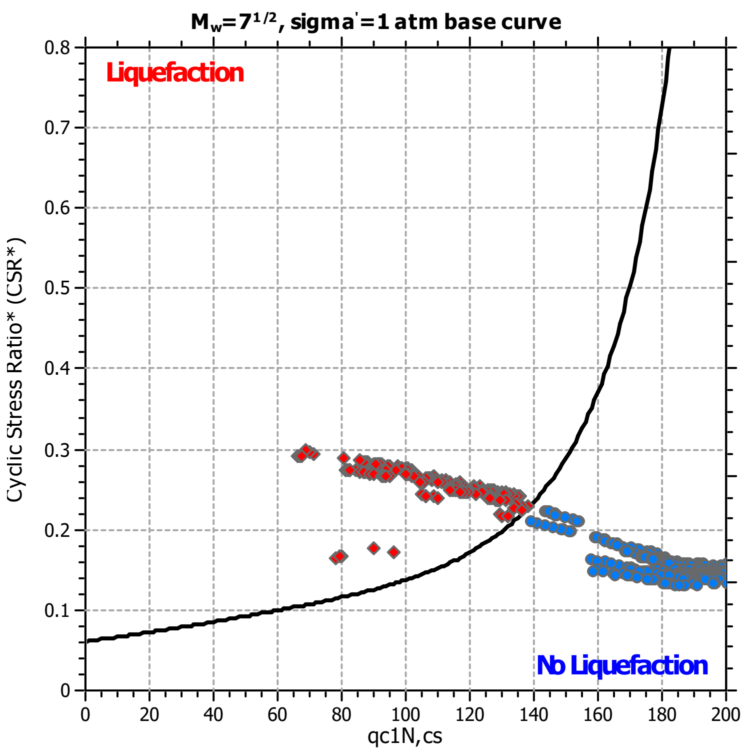
Project title : **Location :**

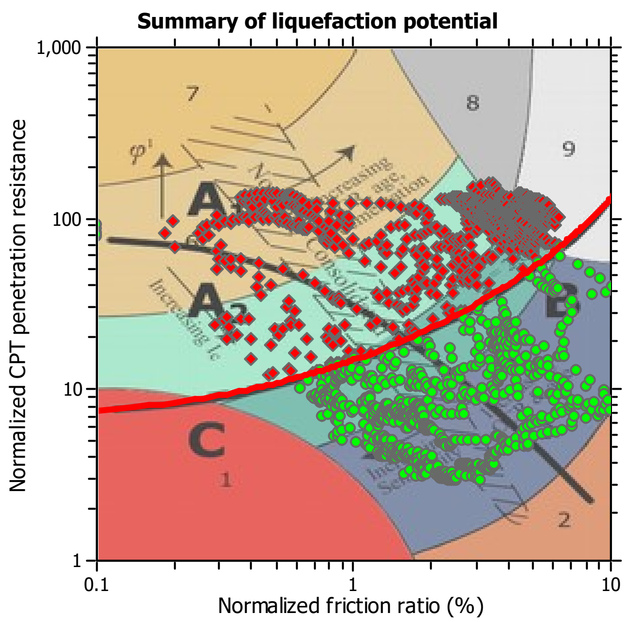
CPT file : CPT_185213

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.25	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

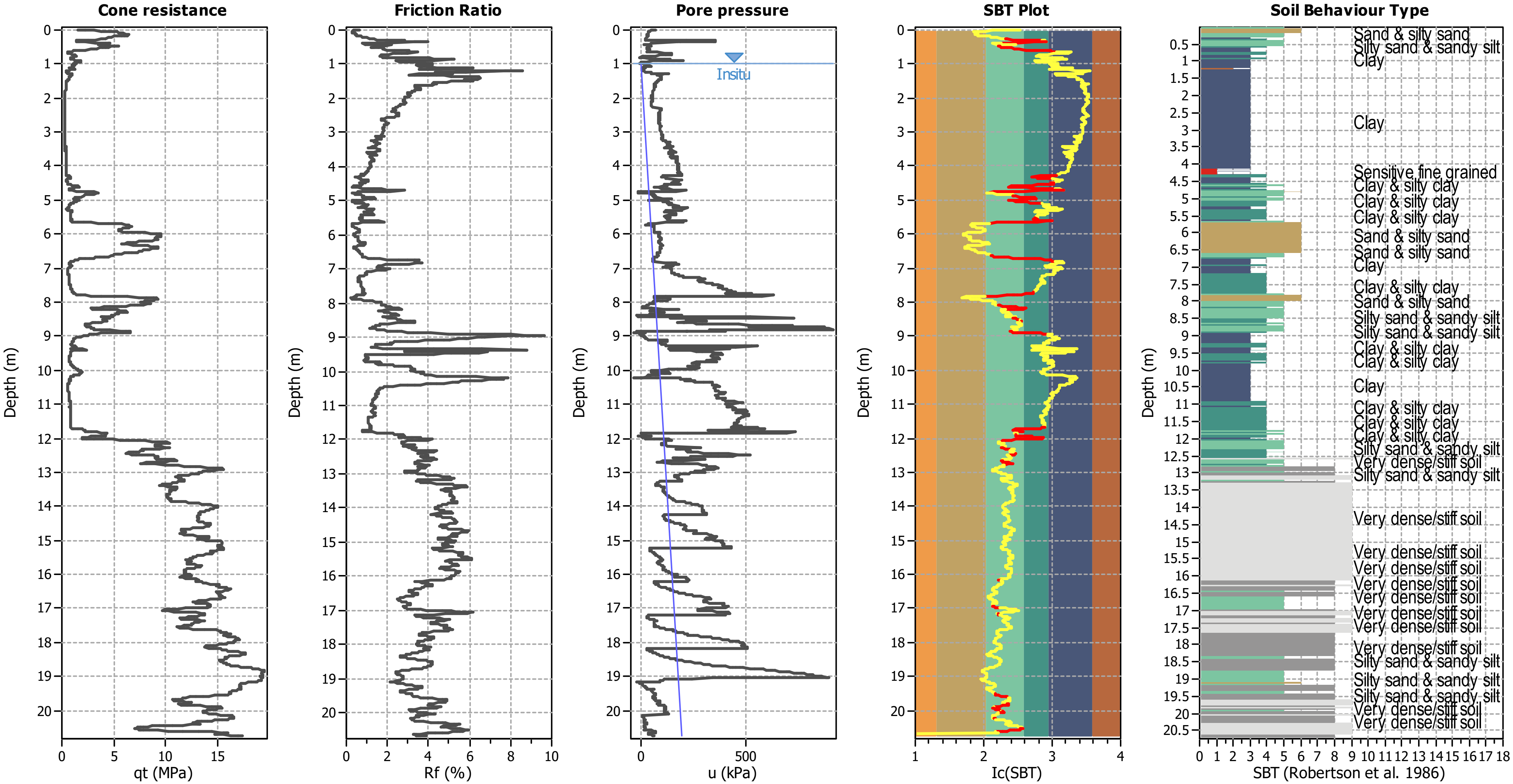






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



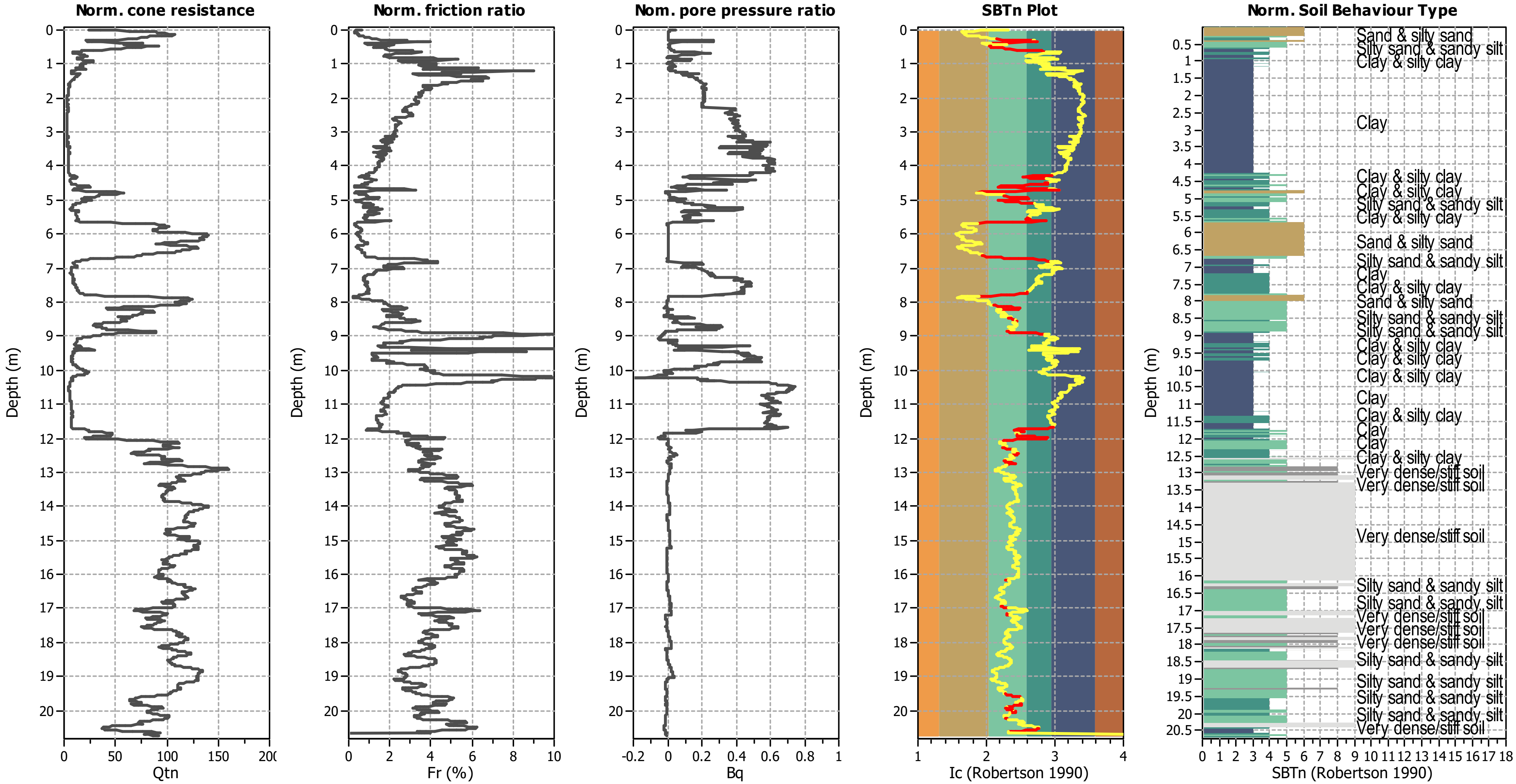
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



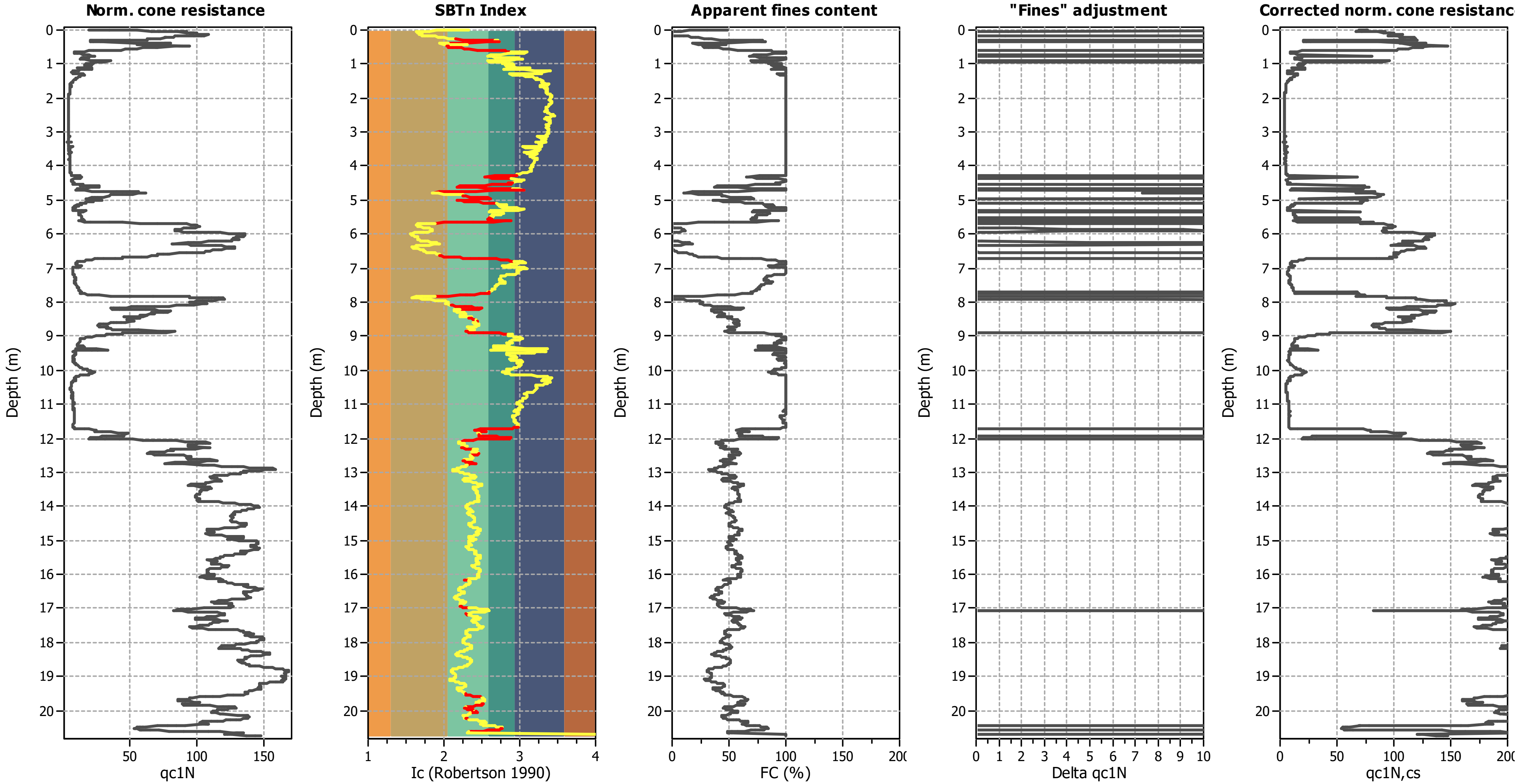
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

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

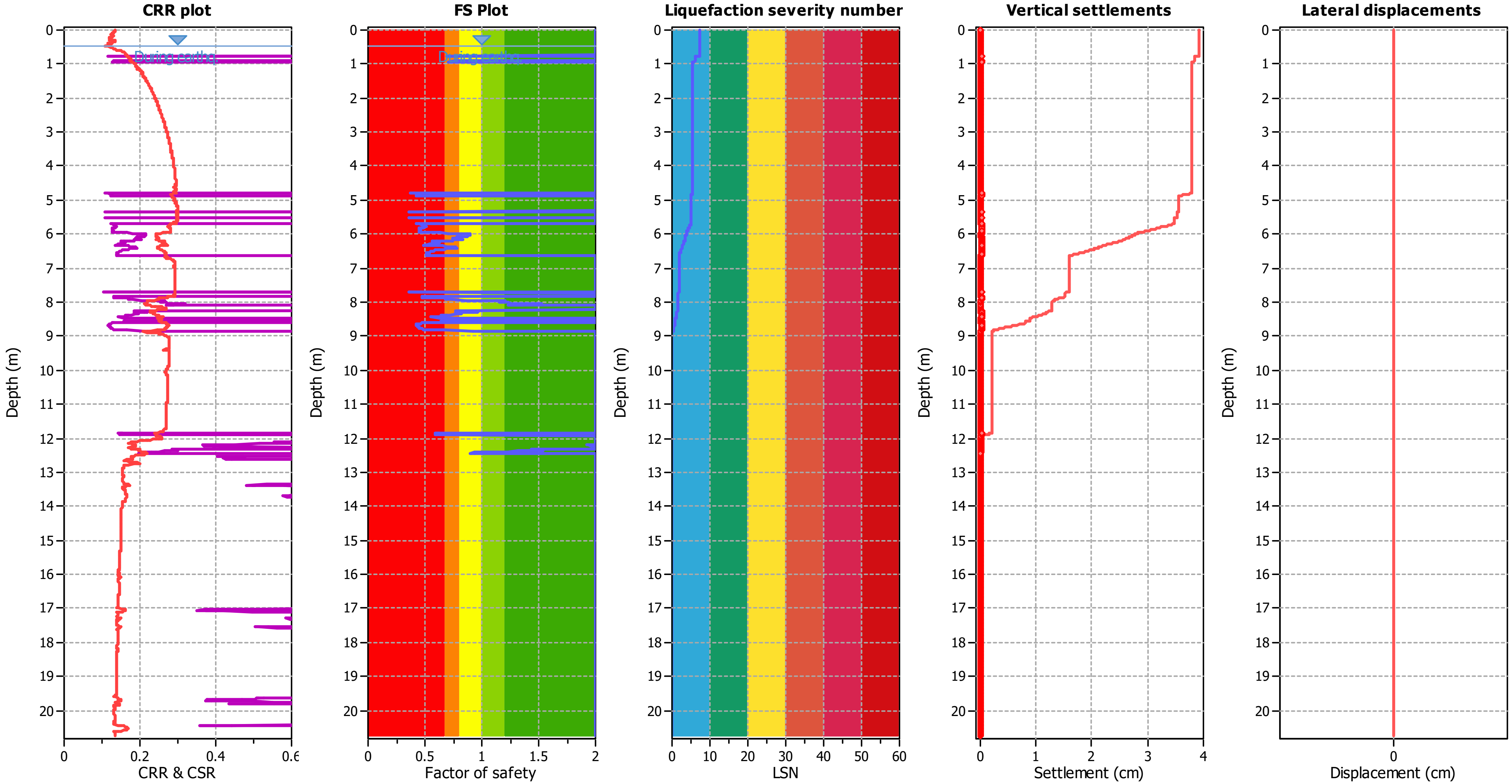
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

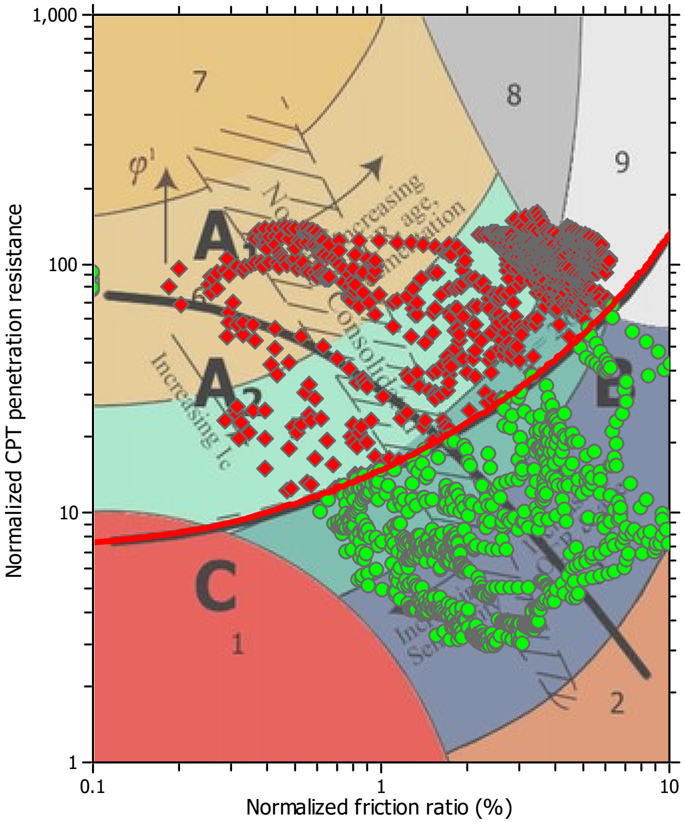
F.S. color scheme

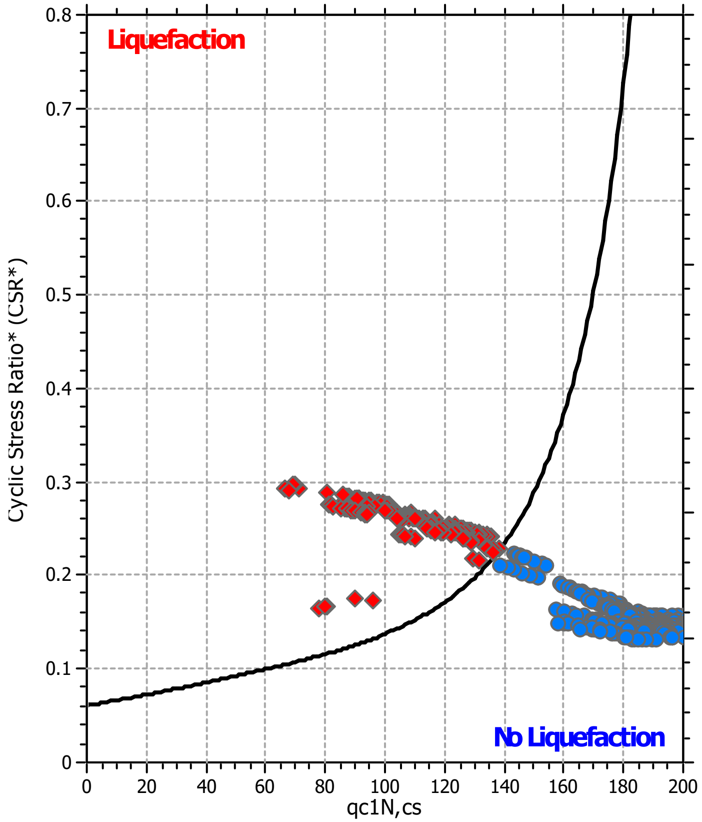
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

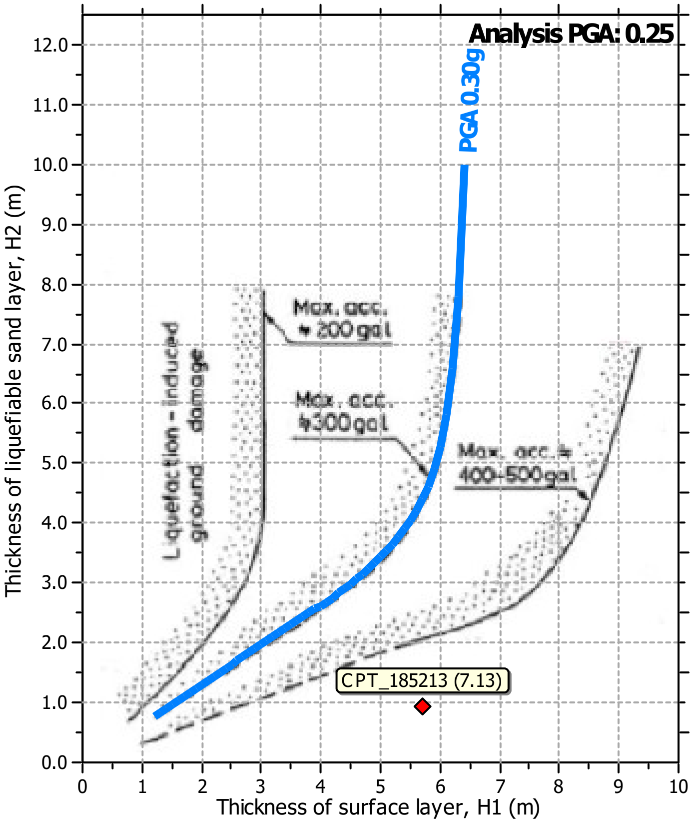
LSN color scheme

- Severe damage
- Major expression of liquefaction
- Moderate to severe exp. of liquefaction
- Moderate expression of liquefaction
- Minor expression of liquefaction
- Little to no expression of liquefaction

Liquefaction analysis summary plots



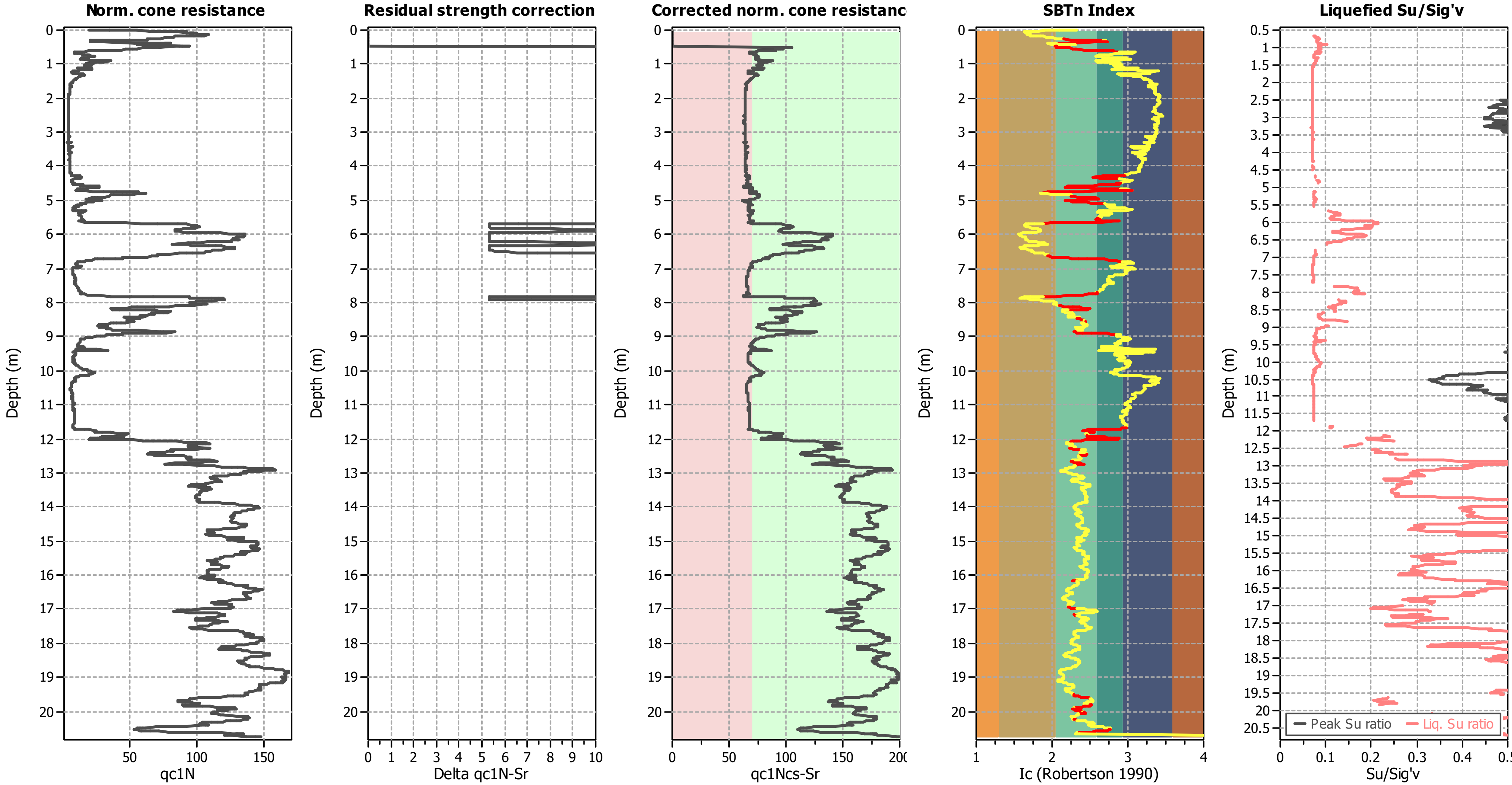




Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A