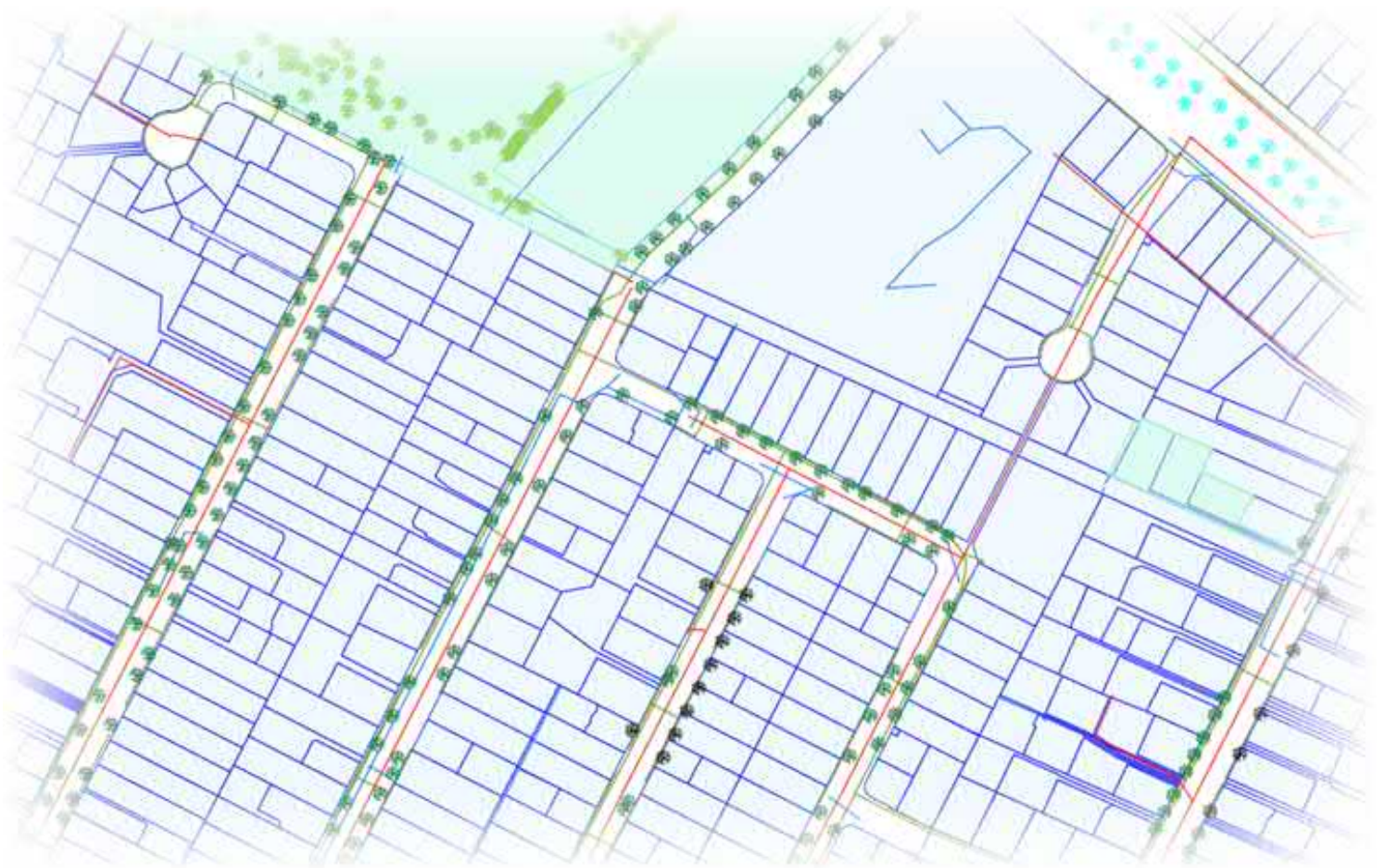


Land Information Memorandum



Property address:
2 Mary Muller Drive

LIM number: 70211569
Page 1

Christchurch City Council
53 Hereford Street, PO Box 73015
Christchurch 8154, New Zealand
Tel 64 3 941 8999
Fax 64 3 941 8984
www.ccc.govt.nz

Application details

Please supply to SOUTH ISLAND COMMERCIAL (2004)
PO BOX 933
CHRISTCHURCH 8140

Client reference 2MARY MULLER

Phone number 365 7887

Fax number (03) 366 0931

Date issued 7 August 2018

Date received 6 August 2018

Property details

Property address 2 Mary Muller Drive

Valuation roll number 22491 40700

Valuation information Capital Value: \$41100000
Land Value: \$8350000
Improvements Value: \$32750000
Please note: these values are intended for Rating purposes

Legal description Lot 2 DP 392999

Existing owner Castle Rock Properties Limited
PO Box 22542
High Street
Christchurch 8142

Council references

Debtor number 3192916

Rate account ID 73125839

LIM number 70211569

Property ID 1157193

Property address:
2 Mary Muller Drive

LIM number: 70211569
Page 2

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Document information

This Land Information Memorandum (LIM) has been prepared for the purpose of section 44A of the Local Government Official Information and Meetings Act 1987 (LGOIMA). It is a summary of the information that we hold on the property. Each heading or "clause" in this LIM corresponds to a part of section 44A.

Sections 1 to 11 contain all of the information known to the Christchurch City Council that must be included under section 44A(2) LGOIMA. Any other information concerning the land as the Council considers, at its discretion, to be relevant is included at section 11 of this LIM (section 44A(3) LGOIMA).

The information included in this LIM is based on a search of Council records only and there may be other information relating to the land which is unknown to the Council. Council records may not show illegal or unauthorised building or works on the property. The applicant is solely responsible for ensuring that the land is suitable for a particular purpose.

If there are no comments or information provided in any section of this LIM this means that the Council does not hold information on the property that corresponds to that part of section 44A.

A LIM is only valid at the date of issue as information is based only upon information the Council held at the time of that LIM request being made.

Property file service

This Land Information Memorandum does not contain all information held on a property file. Customers may request property files by phoning the Council's Customer Call Centre on (03) 941 8999, or visiting any of the Council Service Centres. For further information please visit www.ccc.govt.nz.

To enable the Council to measure the accuracy of this LIM document based on our current records, we would appreciate your response should you find any information contained therein which may be considered to be incorrect or omitted. Please telephone the Customer Call Centre on (03) 941 8999.

A search of records held by the Council has revealed the following information:

1. Special features and characteristics of the land

Section 44A(2)(a) LGOIMA. This is information known to the Council but not apparent from the district scheme under the Town and Country Planning Act 1977 or a district plan under the Resource Management Act 1991. It identifies each (if any) special feature or characteristic of the land concerned, including but not limited to potential erosion, avulsion, falling debris, subsidence, slippage, alluvion, or inundation, or likely presence of hazardous contaminants.

(For enquiries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

1 ECan Liquefaction Assessment

ECan holds indicative information on liquefaction hazard in the Christchurch area. Information on liquefaction can be found on the ECan website at www.ecan.govt.nz/liq or by calling ECan customer services on Ph 03 353 9007. The Christchurch City Council may require site-specific investigations before granting future subdivision or building consent for the property, depending on the liquefaction potential of the area that the property is in.

1 Borelog/Engineer Report Image Available

Borelog/Engineer Report Image Available

1 Potentially Contaminating Activity

Council have a record showing that an activity has taken place on this site which fits within Group A, " Chemical manufacture, application and bulk storage", of the 2011 Hazardous Activities and Industries List (HAIL) .

1 Potentially Contaminating Activity - continued

More detail on the HAIL may be found at the Ministry for the Environment. More detail on this specific site may be available on the Listed Land Use Register (LLUR) maintained by Environment Canterbury. There is a potential for contaminants to be present in the ground. Site specific investigations may be required for any proposed land use.

1 Consultant Report Available

Land Information New Zealand (LINZ) engaged Tonkin and Taylor to provide a Geotechnical Report on Ground Movements that occurred as a result of the Canterbury Earthquake Sequence. The report indicates this property may have been effected by a degree of earthquake induced subsidence. The report obtained by LINZ can be accessed on their website at <https://www.linz.govt.nz/land/surveying/earthquakes/canterbury-earthquakes/information-for-canterbury-surveyors>

1 Coastal Hazard Inundation

The Council has a report, Coastal Hazard Assessment for Christchurch and Banks Peninsula (2017), that indicates this property or part of this property may be susceptible to coastal inundation (flooding by the sea). The 2017 report considers four sea level rise scenarios through to the year 2120. A copy of the 2017 report and other coastal hazard information can be found at www.ccc.govt.nz/coastalhazards.

1 Fill

This property is located in an area known to have been filled. The year the fill occurred is Unknown. The filling was, according to the Councils records carried out in an uncontrolled manner and comprises Mixed Fill with Demolition Material.

┆ Predicted 1 in 50 Year Flood Extent

This property, or parts of this property are predicted to be within the extent of a 1 in 50 year flood event. For new developments a minimum finished floor level may be required for flood limitation purposes under the Building Code. For more information please refer to (<https://ccc.govt.nz/floorlevelmap>) or phone 941 8999.

┆ Mayoral Flooding Taskforce

This property or parts of this property lie within the observed, reported or estimated flood extent of one or more of the flood events between February 2011 and April 2014. For more information please refer to <https://ccc.govt.nz/reports/> or phone council on 941-8999.

┆ Property Affected by Liquified Petroleum Gas pipe line

Council records indicate that the Christchurch Lyttleton LPG pipeline passes through this property. Under the Hazardous Substances and New Organisms Act 1996 minimum separation distances may apply between any excavation or building activities and the pipeline. Early consultation with a CCC hazardous substances enforcement officer is recommended before any work commences.

┆ Softground

Council records show that site contains Soft Ground. Predominant Ground Material: Sand Reason for Assessment: Building Consent Should further buildings be proposed on this site, specific foundation design may be required.

Related information

- ┆ There are attached hazard/special site characteristics supplementary sheet/s
 - 1) Geotechnical Reports - Alan Reay
 - 2) Geotechnical Reports - Eliot Sinclair

2. Private and public stormwater and sewerage drains

Section 44A(2)(b) LGOIMA. This is information about private and public stormwater and sewerage drains as shown in the Council's records.

(For stormwater and sewerage enquiries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

Related information

- | The buildings are shown to be served by a sewer drain and a stormwater drain.
- | The drainage works associated with this property have not been plotted on the Council's drainage plan. A copy of the field Inspectors pickup/approved site plan showing the drains and house outline is attached.
- | The Council's records show a public stormwater pipe passing through the site.
- | Registration to discharge or a consent must be obtained before any trade waste can be discharged to the Council's sewer system. Any consent to discharge trade waste will be issued in accordance with the Council's Trade Waste Bylaw. The Trade Waste Bylaw regulates the transfer of trade waste consents if a change of owner/ discharger occurs.

3. Drinking Water Supply

Section 44A(2)(ba) and (bb) LGOIMA. This is information notified to the Council about whether the land is supplied with drinking water, whether the supplier is the owner of the land or a networked supplier, any conditions that are applicable, and any information the Council has about the supply.

(For water supply queries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

Water Supply

Christchurch City Council is the networked supplier of water to this property. This property is connected to the Christchurch City Council Water Supply. The conditions of supply are set out in the Christchurch City Council Water Supply, Wastewater & Stormwater Bylaw (2014), refer to www.ccc.govt.nz.

4. Rates

Section 44A(2)(c) LGOIMA. This is information on any rates owing in relation to the land.

(For rates enquiries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

(a) Annual rates

Annual rates to 30/06/2019: \$ 307,105.35

	Instalment Amount	Date Due
Instalment 1	\$ 76,776.27	15/09/2018
Instalment 2	\$ 76,776.27	15/12/2018
Instalment 3	\$ 76,776.27	15/03/2019
Instalment 4	\$ 76,776.54	15/06/2019

Rates owing as at 07/08/2018: \$ 76,776.27

(b) Excess water charges

\$ 0.00

(For water charge enquiries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

(c) Final water meter reading required?

Reading is Required

(To arrange a final water meter reading, please phone (03) 941 8999 or visit www.ccc.govt.nz.

5. Consents, certificates, notices, orders, or requisitions affecting the land and buildings

Section 44A(2)(d) LGOIMA. This is information concerning any consent, certificate, notice, order, or requisition, affecting the land or any building on the land, previously issued by the Council.

Section 44A(2)(da) LGOIMA. The information required to be provided to a territorial authority under section 362T(2) of the Building Act 2004. There is currently no information required to be provided by a building contractor to a territorial authority under section 362T(2) of the Building Act 2004. The Building (Residential Consumer Rights and Remedies) Regulations 2014 only prescribed the information that must be given to the clients of a building contractor.

(For building enquiries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

(a) Consents

- | BAA37000664 Applied: 17/09/2013 COMPLETED
Cert. of Acceptance Granted Date: 01/05/2014
Building Act Certificate of Acceptance considered for new roof bracing and additional support to perimeter panels.
- | BCN/1965/4469 Applied: 24/08/1965 Status: Completed
2 Mary Muller Drive Hillsborough
FOUR HORSE STALLS (APPLIED) 2 MARY MULLER DRIVE- Historical Reference PER65200210
- | BCN/1990/8245 Applied: 17/10/1990 Status: Completed
2 Mary Muller Drive Hillsborough
Permit granted 06/12/1990
Permit issued 13/02/1991
FOOTBRIDGE OVER LPG PIPELINE&DRAIN 2 MARY MULLER DRIVE- Historical Reference PER90102620
- | BCN/1998/4452 Applied: 25/06/1998 Status: Cancelled
2 Mary Muller Drive Hillsborough
Accepted for processing 25/06/1998
Application cancelled 01/01/1999
ROAD-SEWER-STORMWATER-WATER SUPPLY MISCELLANEOUS-ROAD-SEWER-STORMWATER-WATER SUPPLY 2 MARY MULLER DRIVE- Historical Reference CON98004903
- | BCN/1999/9562 Applied: 06/12/1999 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 06/12/1999
PIM Granted 15/12/1999
PIM Issued 15/12/1999
Building consent granted 24/12/1999
Building consent issued 24/12/1999
Code Compliance Certificate Granted 21/09/2000
Code Compliance Certificate Issued 21/09/2000
NEW WAREHOUSE BUILDING/NEW OFFICE BUILDING 2 MARY MULLER DRIVE- Historical Reference ABA10000375
- | BCN/2000/2897 Applied: 06/04/2000 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 06/04/2000
PIM Granted 19/04/2000
PIM Issued 19/04/2000
WAREHOUSE & OFFICES 2 MARY MULLER DRIVE- Historical Reference ABA10003520

Property address:
2 Mary Muller Drive

- | BCN/2000/3163 Applied: 14/04/2000 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 14/04/2000
Exemption granted 20/04/2000
Exemption issued 20/04/2000
PIM Granted 20/04/2000
PIM Issued 20/04/2000
SIGN- Historical Reference ABA10003766
- | BCN/2000/3351 Applied: 19/04/2000 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 19/04/2000
PIM Granted 19/04/2000
PIM Issued 19/04/2000
Building consent granted 10/05/2000
Building consent issued 12/05/2000
Code Compliance Certificate Granted 02/04/2001
Code Compliance Certificate Issued 02/04/2001
WAREHOUSE AND ASSOCIATED OFFICES 2 MARY MULLER DRIVE- Historical Reference ABA10003925
- | BCN/2000/6660 Applied: 11/08/2000 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 11/08/2000
PIM Granted 18/08/2000
PIM Issued 18/08/2000
Building consent granted 29/08/2000
Building consent issued 01/09/2000
Code Compliance Certificate Granted 11/04/2003
Code Compliance Certificate Issued 11/04/2003
INTERNAL OFFICE FITOUT 2 MARY MULLER DRIVE- Historical Reference ABA10007212
- | BCN/2000/8532 Applied: 26/10/2000 Status: Completed
2 Mary Muller Drive Hillsborough
PIM Granted 18/08/2000
PIM Issued 18/08/2000
Accepted for processing 26/10/2000
Building consent granted 01/11/2000
Building consent issued 20/11/2000
Code Compliance Certificate Granted 06/04/2001
Code Compliance Certificate Issued 06/04/2001
AMENDMENT-STORAGE AREA INSIDE EXISTING BUILDING 2 MARY MULLER DRIVE- Historical Reference ABA10008994
- | BCN/2000/9152 Applied: 21/11/2000 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 21/11/2000
PIM Granted 30/11/2000
PIM Issued 30/11/2000
WAREHOUSE AND ASSOCIATED OFFICES 2 MARY MULLER DRIVE- Historical Reference ABA10009609
- | BCN/2000/9971 Applied: 22/12/2000 Status: Completed
2 Mary Muller Drive Hillsborough
PIM Granted 30/11/2000
PIM Issued 30/11/2000
Accepted for processing 22/12/2000
Building consent granted 23/03/2001
Building consent issued 27/03/2001
Code Compliance Certificate Granted 28/11/2001
Code Compliance Certificate Issued 28/11/2001
WAREHOUSE AND ASSOCIATED OFFICES 2 MARY MULLER DRIVE- Historical Reference ABA10010404

Property address:
2 Mary Muller Drive

- | BCN/2001/2733 Applied: 20/04/2001 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 20/04/2001
PIM Granted 09/05/2001
PIM Issued 09/05/2001
OFFICE ADDITION 2 MARY MULLER DRIVE- Historical Reference ABA10013061
- | BCN/2001/3120 Applied: 07/05/2001 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 07/05/2001
PIM Granted 21/05/2001
PIM Issued 21/05/2001
Building consent granted 25/05/2001
Building consent issued 18/06/2001
Code Compliance Certificate Granted 22/11/2001
Code Compliance Certificate Issued 22/11/2001
STAGES 1 & 2: FOUNDATIONS, SITEWORKS AND FLOOR SLAB/ WAREHOUSE AND OFFICES- 2 MARY MULLER DRIVE ** CCC Includes project no. 12013483- Historical Reference ABA10013483
- | BCN/2001/3148 Applied: 07/05/2001 Status: Cancelled
2 Mary Muller Drive Hillsborough
Accepted for processing 07/05/2001
Application cancelled 05/11/2008
STAGE 1 OF 2: FOUNDATIONS, SITEWORKS AND FLOOR SLAB 2 MARY MULLER DRIVE- Historical Reference ABA10023483
- | BCN/2001/3214 Applied: 09/05/2001 Status: Completed
2 Mary Muller Drive Hillsborough
PIM Granted 09/05/2001
PIM Issued 09/05/2001
Accepted for processing 09/05/2001
Building consent granted 24/05/2001
Building consent issued 29/05/2001
Code Compliance Certificate Granted 28/11/2001
Code Compliance Certificate Issued 28/11/2001
OFFICE ADDITIONS 2 MARY MULLER DRIVE- Historical Reference ABA10013568
- | BCN/2001/3765 Applied: 31/05/2001 Status: Completed
2 Mary Muller Drive Hillsborough
PIM Granted 21/05/2001
PIM Issued 21/05/2001
Accepted for processing 31/05/2001
Building consent granted 13/06/2001
Building consent issued 18/06/2001
Code Compliance Certificate Granted 22/11/2001
Code Compliance Certificate Issued 22/11/2001
STAGE 2 OF 2: WAREHOUSE AND OFFICES 2 MARY MULLER DRIVE CCC Issued on project no. 10013483 for Stages 1 & 2- Historical Reference ABA12013483
- | BCN/2002/7037 Applied: 03/09/2002 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 03/09/2002
PIM Granted 17/09/2002
PIM Issued 17/09/2002
Building consent granted 12/11/2002
Building consent issued 19/12/2002
Code Compliance Certificate Granted 04/08/2003
Code Compliance Certificate Issued 04/08/2003
STAGES 1 & 2 - FOUNDATIONS/SLAB AND SUPERSTRUCTURE- Historical Reference ABA10027856

Property address:
2 Mary Muller Drive

- | BCN/2002/8834 Applied: 06/11/2002 Status: Completed
2 Mary Muller Drive Hillsborough
PIM Granted 17/09/2002
PIM Issued 17/09/2002
Accepted for processing 06/11/2002
Building consent granted 05/12/2002
Building consent issued 19/12/2002
Code Compliance Certificate Granted 04/08/2003
Code Compliance Certificate Issued 04/08/2003
STAGE 2 OF 2: SUPERSTRUCTURE 2 MARY MULLER DRIVE- Historical Reference ABA12027856

- | BCN/2003/4237 Applied: 05/06/2003 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 05/06/2003
Building consent granted 16/06/2003
PIM Granted 16/06/2003
PIM Issued 16/06/2003
Building consent issued 20/06/2003
Code Compliance Certificate Granted 30/01/2006
Code Compliance Certificate Issued 30/01/2006
SINGLE SIDED PLINTH SIGN/INTERNALLY ILLUMINATED 2 MARY MULLER DRIVE- Historical Reference ABA10035472

- | BCN/2003/4496 Applied: 12/06/2003 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 12/06/2003
PIM Granted 23/06/2003
PIM Issued 23/06/2003
FACTORY ADDITION- Historical Reference ABA10035690

- | BCN/2003/5651 Applied: 17/07/2003 Status: Completed
2 Mary Muller Drive Hillsborough
PIM Granted 23/06/2003
PIM Issued 23/06/2003
Accepted for processing 17/07/2003
Building consent granted 08/08/2003
Building consent issued 11/08/2003
Code Compliance Certificate Granted 29/03/2004
Code Compliance Certificate Issued 29/03/2004
EXTENSION TO EXISTING FACTORY- Historical Reference ABA10036809

- | BCN/2003/8102 Applied: 01/10/2003 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 01/10/2003
PIM Granted 16/10/2003
PIM Issued 16/10/2003
Building consent granted 17/10/2003
Building consent issued 21/10/2003
Code Compliance Certificate Granted 05/03/2004
Code Compliance Certificate Issued 05/03/2004
PRECAST CONCRETE SIGN- Historical Reference ABA10039230

- | BCN/2004/8547 Applied: 27/10/2004 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 27/10/2004
PIM Granted 13/12/2004
PIM Issued 13/12/2004
WAREHOUSE AND ASSOCIATED OFFICES WITH CAFE 15 MARY MULLER DRIVE- Historical Reference ABA10050226

Property address:
2 Mary Muller Drive

- | BCN/2004/9821 Applied: 10/12/2004 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 10/12/2004
PIM Granted 13/12/2004
PIM Issued 13/12/2004
Building consent granted 28/01/2005
Building consent issued 03/02/2005
Code Compliance Certificate Granted 22/08/2005
Code Compliance Certificate Issued 22/08/2005
WAREHOUSE & ASSOCIATED OFFICES WITH MEZZANINE CAFE INCL. AMMENDED DRGS 3/05. 15 MARY MULLER DRIVE- Historical Reference ABA10051435
- | BCN/2005/10655 Applied: 21/01/2005 Status: Completed
2 Mary Muller Drive Hillsborough
Exemption from building consent declined 21/01/2005
Application for exemption from Building Consent for internal alterations to Kathmandu Offices
- Historical Reference BAE35000725
- | BCN/2005/760 Applied: 07/02/2005 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 07/02/2005
Building consent granted 17/03/2005
PIM Granted 17/03/2005
PIM Issued 17/03/2005
Building consent issued 22/03/2005
Code Compliance Certificate Granted 20/04/2006
Code Compliance Certificate Issued 20/04/2006
INTERNAL PARTITIONS REVISIONS- Historical Reference ABA10052563
- | BCN/2005/1231 Applied: 24/02/2005 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 24/02/2005
PIM Granted 06/04/2005
PIM Issued 06/04/2005
NEW OFFICE BUILDING 15 MARY MULLER DRIVE- Historical Reference ABA10053030
- | BCN/2005/2325 Applied: 01/04/2005 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 01/04/2005
PIM Granted 13/05/2005
PIM Issued 13/05/2005
Building consent granted 17/05/2005
Building consent issued 18/05/2005
Code Compliance Certificate Granted 24/11/2005
Code Compliance Certificate Issued 24/11/2005
NEW OFFICE BUILDING 15 MARY MULLER DRIVE- Historical Reference ABA10054080
- | BCN/2005/3595 Applied: 12/05/2005 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 12/05/2005
PIM Granted 07/06/2005
PIM Issued 07/06/2005
EXTENSION TO EXISTING FACTORY- Historical Reference ABA10055298

Property address:
2 Mary Muller Drive

- | BCN/2005/4834 Applied: 22/06/2005 Status: Completed
2 Mary Muller Drive Hillsborough
PIM Granted 02/06/2005
PIM Issued 02/06/2005
Accepted for processing 22/06/2005
Building consent granted 10/08/2005
Building consent issued 18/08/2005
Code Compliance Certificate Granted 17/02/2006
Code Compliance Certificate Issued 17/02/2006
EXTENSION TO EXISTING FACTORY- Historical Reference ABA10056499

- | BCN/2005/5002 Applied: 29/06/2005 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 29/06/2005
PIM Granted 14/07/2005
PIM Issued 14/07/2005
Building consent granted 01/08/2005
Building consent issued 05/08/2005
Code Compliance Certificate Granted 17/05/2006
Code Compliance Certificate Issued 17/05/2006
MEZZANINE FLOOR EXTENSION- Historical Reference ABA10056648

- | BCN/2005/5928 Applied: 29/07/2005 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 29/07/2005
PIM Granted 19/08/2005
PIM Issued 22/08/2005
Building consent granted 01/09/2005
Building consent issued 08/09/2005
Code Compliance Certificate Granted 30/08/2007
Code Compliance Certificate Issued 30/08/2007
FREE STANDING SIGN/PAINTED SIGN ON BUILDING KNOWN AS 11 MARY MULLER DRIVE- Historical Reference ABA10057565

- | BCN/2005/8333 Applied: 14/10/2005 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 14/10/2005
PIM Granted 04/11/2005
PIM Issued 04/11/2005
COMMERCIAL BUILDING FOR USE AS CAFE 30/3/5 amended plans with P.I.M check-linda return to bruce when finished thanks 15 MARY MULLER DRIVE- Historical Reference ABA10059890

- | BCN/2005/10030 Applied: 07/12/2005 Status: Code Compliance Certificate refused S93
2 Mary Muller Drive Hillsborough
Accepted for processing 07/12/2005
PIM Granted 22/12/2005
PIM Issued 23/12/2005
Building consent granted 10/01/2006
Building consent issued 16/02/2006
Council refused to issue a Code Compliance Certificate, s93 Building Act 2004 19/02/2013
ERECT NON ILLUMINATED PLINTH SIGN FOR TYCO SAFETY PRODUCTS PREMISES 19 MARY MULLER DRIVE- Historical Reference ABA10061524

Property address:
2 Mary Muller Drive

- | BCN/2006/229 Applied: 17/01/2006 Status: Completed
2 Mary Muller Drive Hillsborough
PIM Granted 07/11/2005
PIM Issued 07/11/2005
Accepted for processing 17/01/2006
Building consent granted 06/04/2006
Building consent issued 07/04/2006
Code Compliance Certificate Granted 10/07/2006
Code Compliance Certificate Issued 10/07/2006
CAFE BUILDING 15 MARY MULLER DRIVE- Historical Reference ABA10062360
- | BCN/2006/7840 Applied: 28/09/2006 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 28/09/2006
PIM Granted 01/11/2006
PIM Issued 06/11/2006
WAREHOUSE WITH ASSOCIATED OFFICE/SHOWROOM AND STAFF CAFETERIA- Historical Reference ABA10070365
- | BCN/2007/3139 Applied: 30/04/2007 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 30/04/2007
PIM Granted 14/05/2007
PIM Issued 15/05/2007
Building consent granted 28/05/2007
Building consent issued 01/06/2007
Code Compliance Certificate Granted 21/01/2008
Code Compliance Certificate Issued 21/01/2008
SEWER STORMWATER DRAINAGE FOR NEW ROAD- Historical Reference ABA10076086
- | BCN/2007/6215 Applied: 07/08/2007 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 07/08/2007
PIM Granted 04/09/2007
PIM Issued 04/09/2007
SINGLE LEVEL OFFICE BUILDING- Historical Reference ABA10079208
- | BCN/2007/7977 Applied: 12/10/2007 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 12/10/2007
PIM Granted 07/11/2007
PIM Issued 07/11/2007
Building consent granted 04/12/2007
Building consent issued 04/12/2007
Code Compliance Certificate Granted 25/07/2008
Code Compliance Certificate Issued 25/07/2008
OFFICE BUILDING- Historical Reference ABA10080970
- | BCN/2008/2586 Applied: 23/04/2008 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 23/04/2008
Exemption granted 28/04/2008
Exemption issued 28/04/2008
PIM Granted 28/04/2008
PIM Issued 28/04/2008
ONE FREESTANDING SIGN AND ONE WALL SIGN- Historical Reference ABA10085347

Property address:
2 Mary Muller Drive

- | BCN/2008/4432 Applied: 03/07/2008 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 03/07/2008
PIM Granted 18/07/2008
PIM Issued 25/07/2008
Building consent granted 08/08/2008
Building consent issued 13/08/2008
Code Compliance Certificate Granted 27/04/2009
Code Compliance Certificate Issued 27/04/2009
ADD MEZZANINE FLOOR TO EXISTING BUILDING- Historical Reference ABA10087249
- | BCN/2008/4540 Applied: 09/07/2008 Status: Cancelled
2 Mary Muller Drive Hillsborough
Accepted for processing 09/07/2008
PIM Granted 30/07/2008
Application cancelled 13/08/2008
ADD MEZZANINE FLOOR FOR LIGHT STORAGE- Historical Reference ABA10087360
- | BCN/2008/5276 Applied: 14/08/2008 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 14/08/2008
PIM Granted 28/08/2008
PIM Issued 09/09/2008
WAREHOUSE AND 2 LEVEL OFFICES NTF 27 NOVEMBER 2008 BL- Historical Reference ABA10088095
- | BCN/2008/5330 Applied: 18/08/2008 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 18/08/2008
PIM Granted 09/09/2008
PIM Issued 09/09/2008
Building consent granted 01/10/2008
Building consent issued 02/10/2008
Code Compliance Certificate Granted 02/09/2009
Code Compliance Certificate Issued 02/09/2009
STAGE 1 OF 2 - FOUNDATIONS ONLY- Historical Reference ABA10088170
- | BCN/2008/5554 Applied: 28/08/2008 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 28/08/2008
PIM Granted 24/09/2008
PIM Issued 24/09/2008
Building consent granted 05/12/2008
Building consent issued 08/12/2008
Code Compliance Certificate Granted 02/09/2009
Code Compliance Certificate Issued 02/09/2009
STAGE 2 OF 2 - SUPERSTRUCTURE (FLOOR-SLAB AND ABOVE), PLUS SERVICES- Historical Reference ABA12088170
- | BCN/2008/5699 Applied: 04/09/2008 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 04/09/2008
Building consent issued 01/10/2008
Amended plan granted 01/10/2008
AMENDED PLANS SET 1 ABA 10087249 SLIDING GATE ADDED TO MEZZANINE FLOOR AND STAIRS
MOVED 100 MM- Historical Reference ABA10088561

Property address:
2 Mary Muller Drive

- | BCN/2008/6144 Applied: 26/09/2008 Status: Code Compliance Certificate refused S93
2 Mary Muller Drive Hillsborough
Accepted for processing 26/09/2008
PIM Granted 03/11/2008
PIM Issued 03/11/2008
Building consent granted 10/11/2008
Building consent issued 12/11/2008
Council refused to issue a Code Compliance Certificate, s93 Building Act 2004 19/02/2013
INTERIOR FITOUT TO GROUND AND FIRST FLOOR- Historical Reference ABA10088997
- | BCN/2008/7959 Applied: 31/10/2008 Status: Exemption from building consent approved
2 Mary Muller Drive Hillsborough
Exemption from building consent approved 31/10/2008
Application for exemption from Building Consent for NEW DOORWAY INTO
STOREROOM AT PDNZ OFFICE
- Historical Reference BAE35001526
- | BCN/2009/184 Applied: 21/01/2009 Status: Completed
2 Mary Muller Drive Hillsborough
PIM Granted 29/01/2008
PIM Issued 29/01/2008
Accepted for processing 21/01/2009
Building consent granted 18/02/2009
Building consent issued 18/02/2009
Code Compliance Certificate Granted 21/09/2009
Code Compliance Certificate Issued 21/09/2009
DEMOLITION OF TWO INTERNAL OFFICE & STORAGE AREAS (INCL MEZZANINE FLOORS)- Historical
Reference ABA10090870
- | BCN/2009/223 Applied: 22/01/2009 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 22/01/2009
Building consent issued 20/02/2009
Amended plan granted 20/02/2009
AMENDED PLANS SET 1 FOR ABA 10088170 SPRINKLER VALVE ROOM AMENDED- Historical Reference
ABA10090911
- | BCN/2009/224 Applied: 22/01/2009 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 22/01/2009
Amended plan granted 10/06/2009
AMENDED PLANS SET 1 FOR ABA 12088170 SPRINKLER VALVE ROOM/SWALE AND PARKING/AMENDED
OFFICE FACADE- Historical Reference ABA10090912
- | BCN/2011/1764 Applied: 19/05/2011 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 19/05/2011
PIM Granted 23/05/2011
PIM Issued 23/05/2011
Building consent granted 30/08/2011
Building consent issued 30/08/2011
Code Compliance Certificate Granted 10/12/2012
Code Compliance Certificate Issued 10/12/2012
EARTHQUAKE REMEDIAL TEMPORARY CONSTRUCTION ROOF & PERMANENT WALL BRACING UNIT-
Historical Reference ABA10110258

Property address:
2 Mary Muller Drive

- | BCN/2011/3652 Applied: 05/10/2011 Status: Lapsed
2 Mary Muller Drive Hillsborough
Accepted for processing 05/10/2011
PIM Granted 14/10/2011
PIM Issued 14/10/2011
Building consent granted 07/11/2011
Building consent issued 07/11/2011
Building consent lapsed 18/12/2012
INSTALLATION OF PALLET RACKING- Historical Reference ABA10112217
- | BCN/2012/1455 Applied: 27/03/2012 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 27/03/2012
PIM Granted 18/04/2012
PIM Issued 20/04/2012
MANUFACTURING FACILITY- Historical Reference ABA10115190
- | BCN/2012/3532 Applied: 22/06/2012 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 22/06/2012
PIM Granted 04/07/2012
PIM Issued 04/07/2012
Building consent granted 29/08/2012
Building consent issued 30/08/2012
Code Compliance Certificate Issued 26/06/2013
MANUFACTURING FACILITY- Historical Reference ABA10117386
- | BCN/2012/4590 Applied: 30/07/2012 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 30/07/2012
PIM Granted 08/08/2012
PIM Issued 08/08/2012
Building consent granted 29/08/2012
Building consent issued 03/09/2012
Code Compliance Certificate Issued 01/07/2013
COMMERCIAL WAREHOUSE WITH ASSOCIATED OFFICES- Historical Reference ABA10118267
- | BCN/2012/6371 Applied: 05/10/2012 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 05/10/2012
Building consent issued 02/11/2012
Amended plan granted 02/11/2012
AMENDMENT 1 ABA10118267 AMENDED FOUNDATIONS AND GROUND FLOOR SLAB- Historical Reference ABA10119986
- | BCN/2013/2267 Applied: 09/04/2013 Status: Withdrawn
2 Mary Muller Drive Hillsborough
Accepted for processing 09/04/2013
Application withdrawn 29/07/2013
VOLUNTARY UPGRADE FOR FIRE & ACCESSIBILITY AFTER EMERGENCY POST EARTHQUAKE STRENGTHENING.- Historical Reference ABA10124489
- | BCN/2013/3732 Applied: 17/05/2013 Status: Cancelled
2 Mary Muller Drive Hillsborough
Accepted for processing 17/05/2013
Application cancelled 22/05/2013
MARQUEE FOR 19 APRIL FUNCTION- Historical Reference ABA10125686

Property address:
2 Mary Muller Drive

- | BCN/2013/9036 Applied: 01/10/2013 Status: Completed
14 Mary Muller Drive Hillsborough
Accepted for processing 02/10/2013
PIM Granted 14/10/2013
PIM Issued 22/10/2013
PIM Application for commercial building.
- | BCN/2013/12216 Applied: 24/10/2013 Status: Completed
2 Mary Muller Drive Hillsborough
Exemption from building consent approved 28/11/2013
Application for exemption from Building Consent for demolition of warehouse/office.
- Historical Reference BAE35007129
- | BCN/2013/11525 Applied: 20/12/2013 Status: Completed
2 Mary Muller Drive Hillsborough
Accepted for processing 31/01/2014
Building consent granted 02/07/2014
Building consent issued 08/07/2014
Code Compliance Certificate Issued 21/01/2016
New Building Office and Warehouse
- | BCN/2013/12489 Applied: 23/12/2013 Status: Completed
2 Mary Muller Drive Hillsborough
Exemption from building consent approved 24/01/2014
Application for exemption from Building Consent for seismic strengthening to 67% NBS.
- Historical Reference BAE35007427
- | BCN/2013/11525 Applied: 28/07/2014 Status: Completed
14 Mary Muller Drive Hillsborough
Accepted for processing 13/08/2014
Building consent granted 03/10/2014
Building consent issued 06/10/2014
Amendment 1 - Alterations to the interior layout of factory floor, minor adjustment to office block, and the inclusion of an additional mezzanine storage area, replace 2 wc's with squat wc's
- | BCN/2013/11525 Applied: 11/11/2014 Status: Completed
14 Mary Muller Drive Hillsborough
Accepted for processing 12/11/2014
Building consent granted 25/11/2014
Building consent issued 01/12/2014
Commercial: Warehouse extension to Northern Boundary / minor plumbing & drainage changes
- | BCN/2013/11525 Applied: 03/03/2015 Status: Completed
14 Mary Muller Drive Hillsborough
Accepted for processing 06/03/2015
Building consent granted 01/04/2015
Building consent issued 02/04/2015
Amendment 3 - new location for value room, bike shed added
- | BCN/2015/6834 Applied: 09/07/2015 Status: Completed
21 Mary Muller Drive Hillsborough
Exemption from building consent approved 20/07/2015
Steel framed external canopy to outdoor area of existing cafe, including louvre system
- | BCN/2015/7662 Applied: 03/08/2015 Status: Completed
2 Mary Muller Drive Hillsborough
Exemption from building consent approved 11/08/2015
New facade.

Property address:
2 Mary Muller Drive

- | BCN/2015/8662 Applied: 31/08/2015 Status: Completed
14 Mary Muller Drive Hillsborough
Accepted for processing 01/09/2015
Building consent granted 26/01/2016
Building consent issued 29/03/2016
Code Compliance Certificate Issued 23/02/2018
Warehouse Alterations - Installation of pallet and shelving racking system.
- | BCN/2016/7711 Applied: 09/09/2016 Status: Completed
11 Mary Muller Drive Hillsborough
Accepted for processing 12/09/2016
Building consent granted 21/10/2016
Building consent issued 27/10/2016
Code Compliance Certificate Issued 13/11/2017
Alterations to a Warehouse Building - Interior Fit-Out with Structural Strengthening of the Building to 100% NBS
- | BCN/2016/9425 Applied: 07/11/2016 Status: Completed
11 Mary Muller Drive Hillsborough
Exemption from building consent approved 14/11/2016
Installation of Storage Racking System.
- | BCN/2017/2618 Applied: 10/04/2017 Status: Completed
15 Mary Muller Drive Hillsborough
Exemption from building consent approved 12/04/2017
Rainscreen to west facade
- | BCN/2017/3602 Applied: 10/05/2017 Status: Completed
15 Mary Muller Drive Hillsborough
Exemption from building consent approved 25/05/2017
Modify opening for new roller door install.
- | BCN/2016/7711 Applied: 23/05/2017 Status: Completed
11 Mary Muller Drive Hillsborough
Accepted for processing 24/05/2017
Building consent granted 13/06/2017
Building consent issued 19/06/2017
Amendment 1 - Removal of internal lift
- | BCN/2018/1084 Applied: 23/02/2018 Status: Completed
6 Mary Muller Drive Hillsborough
Exemption from building consent approved 04/04/2018
Minor upgrades to an existing office and warehouse.
- | BCN/2018/2001 Applied: 03/04/2018 Status: Completed
2 Mary Muller Drive Hillsborough
Exemption from building consent approved 03/05/2018
Minor modification of internal layout to split tenancy including provision of new entry door.
- | BCN/2018/2167 Applied: 09/04/2018 Status: Completed
6 Mary Muller Drive Hillsborough
Exemption from building consent approved 13/04/2018
Strengthening

(b) Certificates

Note: Code Compliance Certificates were only issued by the Christchurch City Council since January 1993.

(c) Notices

| Development Constraint Conditions

Council records show there is a specific condition on the use of this site: Specific Foundation Design Required

Property address:
2 Mary Muller Drive

Ministry of Business, Innovation & Employment Foundation Design

Some properties have experienced land damage and considerable settlement during the sequence of Canterbury earthquakes. While land in the green zone is still generally considered suitable for residential construction, houses in some areas will need more robust foundations or site foundation design where foundation repairs or rebuilding are required. Most properties have been assigned a technical category. Details of the MBIE guidance can be found at www.building.govt.nz/

- Placards issued under the Civil Defence Emergency Management Act 2002 as a result of the 4 September 2010 and 22 February 2011 earthquakes have now expired (by 12 July 2011 if not before). Some civil defence placards were replaced with dangerous building notices issued under section 124 Building Act 2004, and where this has happened the section 124 notice is separately recorded. Many other buildings, although not issued with a section 124 notice, may require structural work or other repairs before they can be occupied again. It is the building owners responsibility to make sure the building is safe for any occupier or visitor. Detailed structural engineering assessments may still be required to be carried out.

CDB75084892 12/04/2011

Castle Rock Cafe - 21 Mary Muller Drive : Building Inspected Under Civil Defence Emergency , Green Placard Issued (a deemed Building Act notice)

CDB75084893 12/04/2011

GION - 6 Mary Muller Drive : Building Inspected Under Civil Defence Emergency , Green Placard Issued (a deemed Building Act notice)

WOF50100 Expires: 01/02/2019

Compliance Schedule Warrant of Fitness/Statement of Fitness/Compliance Schedule Statement

WOF50463 Expires: 01/09/2018

Compliance Schedule Warrant of Fitness/Statement of Fitness/Compliance Schedule Statement

WOF51588 Expires: 01/05/2018

Compliance Schedule Warrant of Fitness/Statement of Fitness/Compliance Schedule Statement

WOF52805 Expires: 01/09/2018

Compliance Schedule Warrant of Fitness/Statement of Fitness/Compliance Schedule Statement

WOF52830 Expires: 01/09/2017

Compliance Schedule Warrant of Fitness/Statement of Fitness/Compliance Schedule Statement

WOF53535 Expires: 01/06/2019

Compliance Schedule Warrant of Fitness/Statement of Fitness/Compliance Schedule Statement

WOF53601 Expires: 01/06/2018

Compliance Schedule Warrant of Fitness/Statement of Fitness/Compliance Schedule Statement

WOF54560 Expires: 01/09/2018

Compliance Schedule Warrant of Fitness/Statement of Fitness/Compliance Schedule Statement

WOF55586 Expires: 01/06/2018

Compliance Schedule Warrant of Fitness/Statement of Fitness/Compliance Schedule Statement

WOF55589 Expires: 01/07/2019

Compliance Schedule Warrant of Fitness/Statement of Fitness/Compliance Schedule Statement

(d) Orders

(e) Requisitions

Related information

- There are numerous Building Consent amendments. They are an approved change to a building consent . Please note a code compliance certificate is not issued for an amendment.

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Fax 64 3 941 8984
www.ccc.govt.nz

- | Attached is a copy of the form 9 for a certificate of acceptance (BAA37000664) that has been issued for building work at this property. There will be items that have been accepted by Council and some that may have been excluded acceptance.
- | In the property file there is either an electrical & or gas fitters certificate relating to works that have been carried on the current building/dwelling out at this address. If you require a copy of the certificate/s please order a property file through the Council website www.ccc.govt.nz or phone 03 941-8999.

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6. Certificates issued by a building certifier

Section 44A(2)(e) LGOIMA. This is information notified to the Council concerning any certificate issued by a building certifier pursuant to the Building Act 1991 or the Building Act 2004.

(For building enquiries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

7. Weathertightness

Section 44A(2)(ea) LGOIMA. This is information notified to the Council under section 124 of the Weathertight Homes Resolution Services Act 2006.

(For weathertight homes enquiries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

If there is no information below this means Council is unaware of any formal Weathertight Homes Resolution Services claim lodged against this property.

8. Land use and conditions

Section 44A(2)(f) LGOIMA. This is information relating to the use to which the land may be put and conditions attached to that use. The planning information provided is not exhaustive and reference to the District Plan(s) is recommended. There have been Proposed Christchurch Replacement District Plan provisions notified. The Proposed Christchurch Replacement District Plan may include changes that affect this property. The Proposed Christchurch Replacement District Plan includes provisions relating to protected historic heritage and protected areas of ecological significance that have immediate legal effect. Decisions have also been made on some of the provisions in the Proposed Christchurch Replacement District Plan which also have legal effect, or may be operative or have to be treated as operative. Proposed Replacement District Plan provisions which are operative, or have to be treated as operative, supersede the relevant provisions in the Christchurch City Plan or the Banks Peninsula District Plan. Some decisions on provisions of the Christchurch Replacement District Plan may be subject to changes as a result of further decisions. To find out more about the Proposed Replacement District Plan and what this might mean for this property, please visit <https://ccc.govt.nz/the-council/plans-strategies-policies-and-bylaws/plans/districtplans> for more information.

(For planning queries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

! **Regional plan or bylaw**

There may be objectives, policies or rules in a regional plan or a regional bylaw that regulate land use and activities on this site. Please direct enquiries to Canterbury Regional Council (Environment Canterbury).

! **Waterway Provisions for Other Councils**

A resource consent or permit may also be required from the Canterbury Regional Council or other territorial authority, particularly with respect to water bodies managed by those authorities. Please refer to the relevant regional plan and any relevant bylaws, and contact the Christchurch City Council if you are uncertain which authority manages the water body in question.

(a)(i) Operative Christchurch City Plan & Banks Peninsula District Plan

1. Special Amenity Area	No
2. Community Footprint	No
3. Opposite Important Open Space	No
4. Designations on Site	No
5. Road Widening Designations	No
6. Historic or Protection Building	No
7. Other Heritage Protection Items	No
8. Protected Trees	
Heritage/Notable Tree	No
Other; eg Category A, B, C Street Plantings; Subdivision trees	No
9. Noise Control	No
10. Coastal Protection	No
11. Landscape Protection	No

(ii) Proposed Christchurch Replacement District Plan/Christchurch District Plan

† **Christchurch International Airport Protection Surfaces**

Property or part of property within the Christchurch International Airport Protection Surfaces overlay which is operative.

† **Liquefaction Management Area (LMA)**

Property or part of property within the Liquefaction Management Area (LMA) Overlay which is operative.

† **Waterway Provisions**

This property or part of this property is close to at least one waterway with a setback within which District Plan rules apply to activities including buildings, earthworks, fences and impervious surfacing. Any part of the property within the setback will be affected by those rules.

† **Waterway Provisions**

This property or part of this property is close to at least one waterway. It may be within the setback for an Environmental Asset Waterway. Within that setback, District Plan rules apply to activities including buildings, earthworks, fences and impervious surfacing. Any part of the property within the setback will be affected by those rules.

† **Flood Management Area**

Property or part of property within the Flood Management Area (FMA) Overlay which is operative.

† **Fixed Minimum Floor Overlay**

This property or parts of the property are located within the Fixed Minimum Floor Overlay level in the Christchurch District Plan. Under this plan pre-set minimum floor level requirements apply to new buildings and additions to existing buildings. The fixed minimum floor level can be searched at <http://ccc.govt.nz/floorlevelmap>. For more information please contact a CCC duty planner on 941 8999.

† **District Plan Zone**

Property or part of property within the Industrial General Zone which is operative.

(iii) Notice of Requirement for a Designation

(b) Resource consents

- † RMA/1999/4877 - Land Use Consent
2 Mary Muller Drive Hillsborough
Industrial development comprising a single storey warehouse and detached administration office and amenities. -
Historical Reference RMA20000427
Status: Processing complete
Applied 22/12/1999
Decision issued 10/01/2000
Granted 10/01/2000

- † RMA/2000/855 - Land Use Consent
2 Mary Muller Drive Hillsborough
Shortfall in carparks. - Historical Reference RMA20001532
Status: Processing complete
Applied 28/03/2000
Granted 03/05/2000
Decision issued 05/05/2000

- † RMA/2000/2777 - Certificate of compliance
2 Mary Muller Drive Hillsborough
Certificate of Compliance for printing and stationery manufacture. - Historical Reference RMA20003537
Status: Processing complete
Applied 07/11/2000
Certificate issued 13/11/2000

- I RMA/2001/348 - Land Use Consent
2 Mary Muller Drive Hillsborough
Application for reduction in carparking and bicycle spaces. - Historical Reference RMA20004200
Status: Processing complete
Applied 08/02/2001
Granted 07/03/2001
Decision issued 09/03/2001

- I RMA/2001/1043 - Land Use Consent
2 Mary Muller Drive Hillsborough
Warehouse development which does no car parking (Transitional), high traffic generator (Proposed Plan). -
Historical Reference RMA20004917
Status: Processing complete
Applied 02/05/2001
Granted 23/05/2001
Decision issued 25/05/2001

- I RMA/2002/2194 - Land Use Consent
2 Mary Muller Drive Hillsborough
Warehouse and office that fail to comply with car parks 7m waterway setback, cycle parks, heavy goods bays and
high traffic generator. - Historical Reference RMA20011227
Status: Processing complete
Applied 04/09/2002
Granted 18/10/2002
Decision issued 21/10/2002

- I RMA/2003/1844 - Land Use Consent
2 Mary Muller Drive Hillsborough
Commercial development and development fee application. - Historical Reference RMA20014149
Status: Processing complete
Applied 17/07/2003
Granted 26/08/2003
Decision issued 27/08/2003

- I RMA/2003/2552 - Land Use Consent
2 Mary Muller Drive Hillsborough
Application for signage - Historical Reference RMA20014877
Status: Withdrawn
Applied 29/09/2003

- I RMA/2005/202 - Land Use Consent
2 Mary Muller Drive Hillsborough
New industrial building which requires consent for land scaping, cycle parking, queue space and high traffic generation. - Historical Reference RMA20019024
Status: Processing complete
Applied 20/01/2005
Granted 09/02/2005
Decision issued 10/02/2005

- I RMA/2005/1572 - Land Use Consent
2 Mary Muller Drive Hillsborough
Modify an industrial complex - Historical Reference RMA20020437
Status: Processing complete
Applied 11/07/2005
Granted 29/07/2005
Decision issued 01/08/2005

- I RMA/2008/166 - Land Use Consent
2 Mary Muller Drive Hillsborough
Develop business 4 zone and store gas with various non compliances - Historical Reference RMA92010947
Status: Processing complete
Applied 30/01/2008
Decision issued 15/05/2008
Granted 15/05/2008

- I RMA/2008/315 - Land Use Consent
2 Mary Muller Drive Hillsborough
additional carparks not complying with queue distance setback - Historical Reference RMA92011107
Status: Processing complete
Applied 19/02/2008
Decision issued 19/03/2008
Granted 19/03/2008

- I RMA/2008/1719 - Land Use Consent
2 Mary Muller Drive Hillsborough
Construct industrial warehouse with associated ancillar y office - Historical Reference RMA92012591
Status: Processing complete
Applied 13/08/2008
Decision issued 12/09/2008
Granted 12/09/2008

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- | RMA/2012/774 - Land Use Consent
2 Mary Muller Drive Hillsborough
Construction of an industrial warehouse/factory - Historical Reference RMA92020130
Status: Processing complete
Applied 25/05/2012
Granted 03/07/2012
Decision issued 04/07/2012

- | RMA/2012/888 - Land Use Consent
2 Mary Muller Drive Hillsborough
Construct new warehouse/factory building on contaminated land - Historical Reference RMA92020252
Status: Processing complete
Applied 15/06/2012
Decision issued 26/07/2012
Granted 26/07/2012

- | RMA/2014/889 - Land Use Consent
2 Mary Muller Drive Hillsborough
Excavation - Historical Reference RMA92025522
Status: Processing complete
Applied 14/04/2014
Decision issued 05/05/2014
Granted 05/05/2014

- | RMA/2018/1850 - Land Use Consent
2 Mary Muller Drive Hillsborough
Reinstatement of offices
Status: Processing
Applied 01/08/2018

- | RMA/2018/1530 - Land Use Consent
15 Mary Muller Drive Hillsborough
Ancillary retail for warehouse and distribution activity including ability to return retail use to existing ancillary office
Status: Processing complete
Applied 28/06/2018
Decision issued 27/07/2018
Granted 27/07/2018

- | RMA/1998/3851 - Subdivision Consent
Right Of Way SUBDIVISION - Historical Reference RMA6560
Status: Processing complete
Applied 20/08/1998

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- I RMA/1999/4284 - Subdivision Consent
Bdy Adj SUBDIVISION 223 RECEIVED 04/12/00 Certified 14/12/00 224 Requested 20/12/2000 224 Issued 10/4/2001
83338 - Historical Reference RMA11812
Status: Processing complete
Applied 23/09/1999
Granted 25/11/1999
Decision issued 25/11/1999

- I RMA/1999/5175 - Subdivision Consent
Bdy Adj SUBDIVISION 223 received 18/2/00 Certified 22/2/00 224 REQUESTED 02/05/2000 224 Issued 9/5/00 DP
82375 - Historical Reference RMA4531
Status: Processing complete
Applied 23/09/1999
Granted 11/10/1999
Decision issued 11/10/1999

- I RMA/2000/187 - Subdivision Consent
BOUNDARY ADJUSTMENT 223 RECEIVED 15/07/02 CERTIFIED 09/08/02 224 Requested 26/09/02 224 Issued
03/11/04 307167 - Historical Reference RMA20000591
Status: Processing complete
Applied 14/01/2000
Granted 25/07/2000
Decision issued 25/07/2000

- I RMA/2007/174 - Subdivision Consent
Boundary Adjustment 223 Requested 30/10/2007 Certified 31/10/2007 DP 392999 224 Requested 30/11/2007
Issued 21/01/2008 - Historical Reference RMA92007381
Status: Processing complete
Applied 29/01/2007
Granted 15/02/2007
Decision issued 15/02/2007

Related information

- I Council records show that there is a current/on hold monitoring job in our system for RMA/2018/1530. This monitoring is to ensure that the resource conditions have been met. For further information you can contact the Compliance & Investigation team on 941 8999.

9. Other land and building classifications

Section 44A(2)(g) LGOIMA. This is information notified to the Council by any statutory organisation having the power to classify land or buildings for any purpose.

(For land and building enquiries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

Please refer to Section 1 for details

10. Network utility information

Section 44A(2)(h) LGOIMA. This is information notified to the Council by any network utility operator pursuant to the Building Act 1991 or the Building Act 2004.

(For network enquiries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

! **None recorded for this property**

11. Other information

Section 44A(3) LGOIMA. This is information concerning the land that the Council has the discretion to include if it considers it to be relevant.

(For any enquiries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

(a) Kerbside waste collection

- | Your recycling is collected Fortnightly on the Week 1 collection cycle on a Monday. Please leave your recycling at the Kerbside by 6:00 a.m. Your nearest recycling depot is the Metro Place Refuse Station.
- | Your refuse is collected Fortnightly on the Week 1 collection cycle on a Monday. Please leave your rubbish at the Kerbside by 6:00 a.m. Your nearest rubbish depot is the Metro Place Refuse Station.
- | Your organics are collected Weekly on Monday. Please leave your organics at the Kerbside by 6:00 a.m.

(b) Other

| **Community Board**

Property located in Linwood-Central-Heathcote Community Board

| **Electoral Ward**

Property located in Heathcote Electoral Ward

| **Listed Land Use Register**

Hazardous activities and industries involve the use, storage or disposal of hazardous substances. These substances can sometimes contaminate the soil. Environment Canterbury identifies land that is used or has been used for hazardous activities and industries. This information is held on a publically available database called the Listed Land Use Register (LLUR). The Christchurch City Council may not hold information that is held on the LLUR Therefore, it is recommended that you check Environment Canterbury's online database at www.llur.ecan.govt.nz

| **Spatial Query Report**

A copy of the spatial query report is attached at the end of this LIM. The spatial query report lists land use resource consents that have been granted within 100 metres of this property.

| **Health Licence**

FSH/2016/290
CCC000304/1
Castle Rock Cafe
Castle Rock Cafe
Food Control Plan
Current

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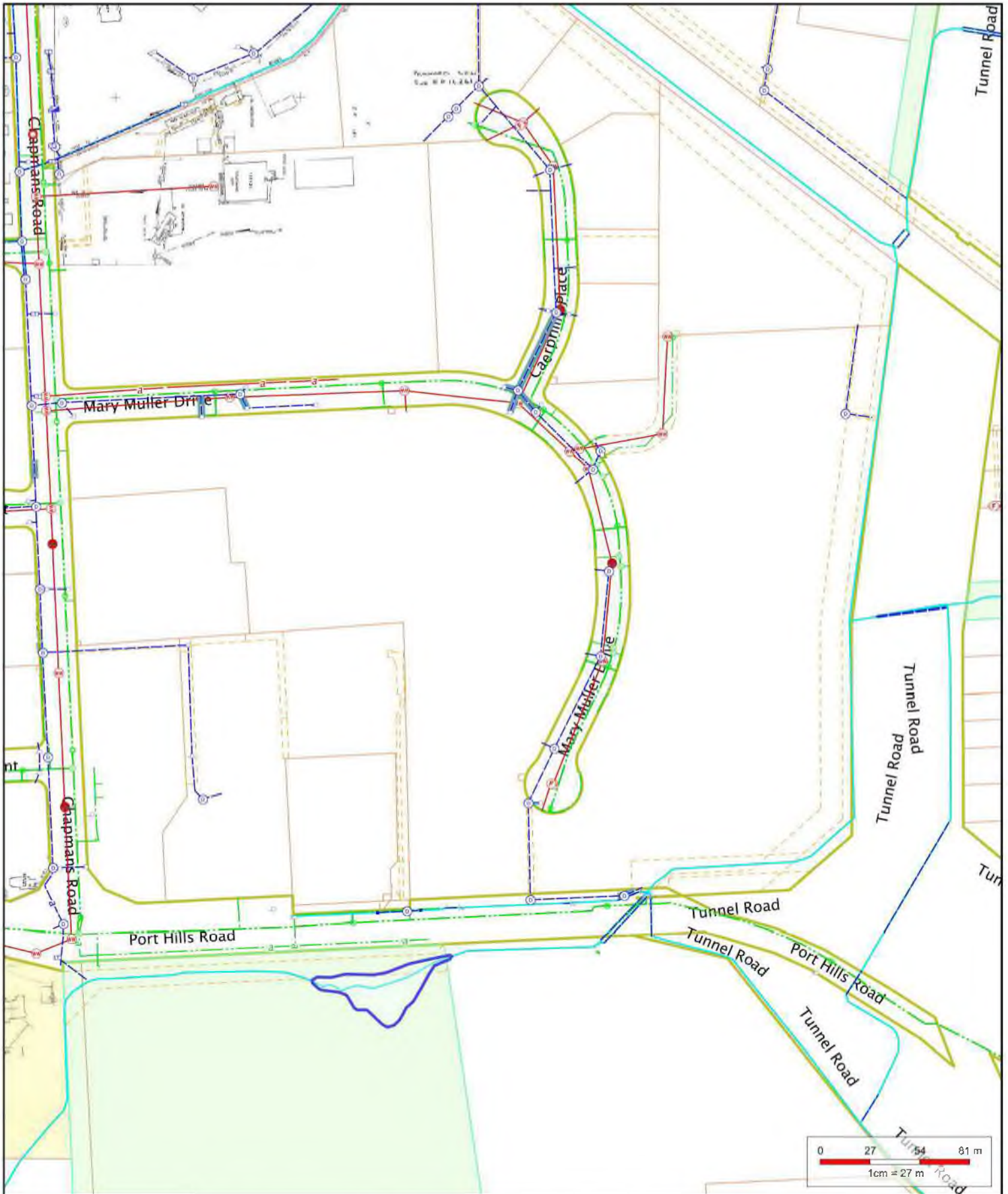
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- | The Council has received a third party work completion report/information relating to the building exemption application on this property (BCN/2016/9425) . It has been placed on the property file. The Council does not accept any liability for the contents, or representations, made within the report/information. The report/information is not included in the Land Information Memorandum (LIM) because the Council has not verify the information/report supplied. If a copy is required you can request a property file by contacting Council on (03) 941 8999 or visiting a Council Service Centre.
- | Please see attached a copy of the Detailed Engineering Evaluation (DEE) assessment report - aka 4 Mary Muller Drive.
- | Dangerous Goods Licences have been replaced with Location Test Certificates/ Location Compliance Certificates administered by Worksafe. You can contact a local Test Certifier to advise you or to issue the type of test certificate you need.
- | Attached council drainage plan shows that a LPG/Petrol pipe passes through this site.

Property address:
2 Mary Muller Drive

LIM number: 70211569
Page 35

Christchurch City Council
53 Hereford Street, PO Box 73015
Christchurch 8154, New Zealand
Tel 64 3 941 8999
Fax 64 3 941 8984
www.ccc.govt.nz































































































1 : 2,700 on A4
 Aug 7, 2018 11:00:49 AM



 ph: 941-8300 fax: 941-8385


Accuracy not guaranteed. Onsite verification required. Display of data scale dependent, full detail available at 1:500.

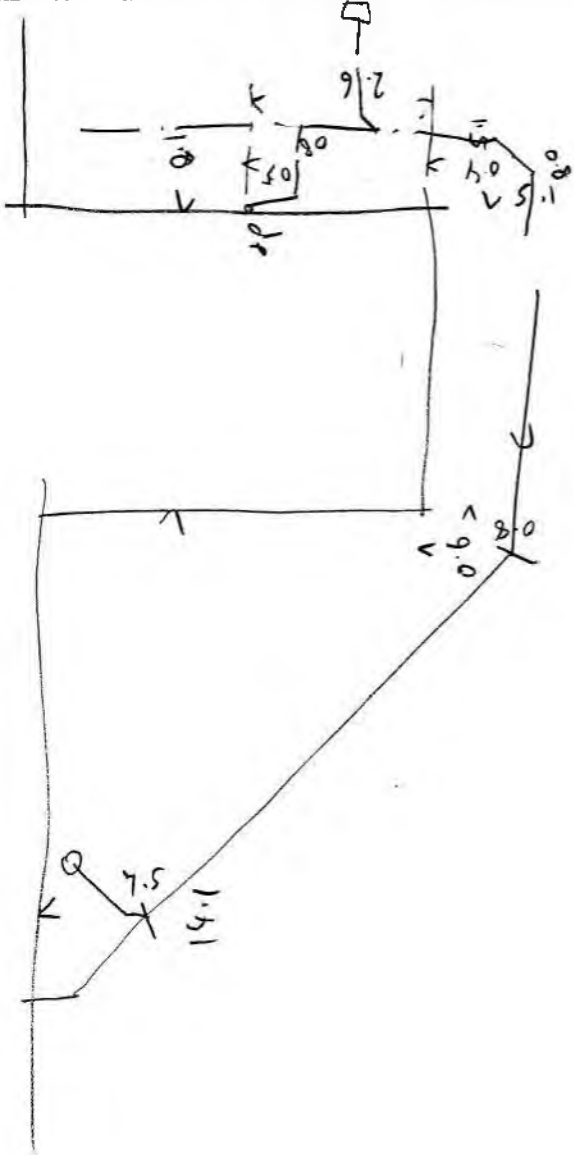
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Private Drainage		Water Intake/Supply	Wastewater	Wastewater	Stormwater	Stormwater
Standard Infrastructure  Bio Gas  Condensate Trap  End Cap  Inlet  Outlet  Valve  Main  Cable	Water Intake/Supply  Connector  Bellows  Connector  Hydrant	Water Intake/Supply  Inlet  Meter  Outlet  Pump  Restrictor  Valve  Air Release  Butterfly  Flow restriction  Gate  Pressure Activated  Sluice  Valve  Reservoir  Structure  Lateral  Main  Sub Main	Wastewater  End Cap  Valve  Air Gap Separator  Vent  Eye  Eye (Vertical)  Outfall  Pump  Junction  Access  Flush Manhole  Inspection Point  Standard Manhole  Trap  Vented Manhole  Lateral  Main  Pressure Main	Wastewater  Lateral Fitting  Local Pressure  Control Panel  Boundary Kit  Tank System  Site  Vacuum Chamber  Vacuum Breather Stormwater  Bend  Change  Eye  Flow Restriction  Inlet  Dome Sump  Double Sump  Gross Debris Trap	Stormwater  Inlet  Inlet Headwall  Pipe End  Silt Trap  Single Sump  Soak Pit  Triple Sump  Junction  Standard Manhole  Outlet  Pump  Structure  Basin  Lateral  Main  Lateral Fitting  Double Sump	Stormwater  Lateral Fitting  Single Sump  Soak Pit  Inspection point  Manhole All services  Pipe Protection  Abandoned  Proposed  Out of service Landbase  Easement

32002

CHRISTCHURCH CITY COUNCIL - DRAINAGE PICK UP

 <p>CHRISTCHURCH THE GARDEN CITY <i>the city that shines</i></p>	ADDRESS: 212 Port Hills Rd	OWNER:	RECEIVED:	CONNECTION NUMBER
	LEGAL DESCRIPTION:	DRAINLAYER: K & T Drainage	BLOCK PLAN:	
	PROJECT No.: 1227856	PLUMBER:	PLOTTED: / /	
	DATE: 27.3.3	FIELD OFFICER: K.J. Walsh	EYE BOOK:	



32184 1 of 2

CHRISTCHURCH CITY COUNCIL - DRAINAGE PICK UP

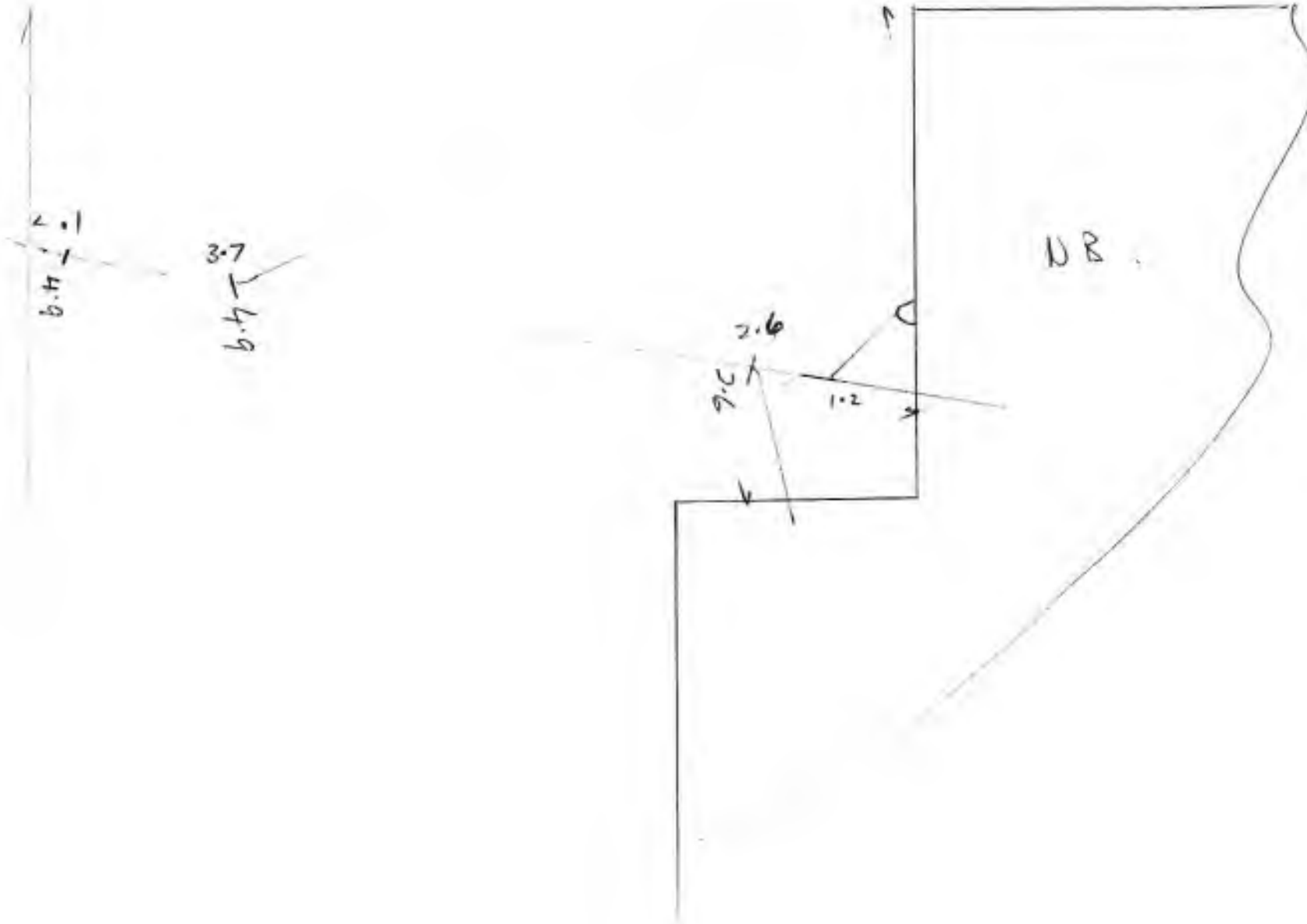



ADDRESS: 212 PORT HILLS RD
LEGAL DESCRIPTION:
PROJECT No.: 1207856
DATE: 25.3.03

OWNER:
DRAINLAYER: KAT
PLUMBER:
FIELD OFFICER: C TRAYLOR

RECEIVED:
BLOCK PLAN:
PLOTTED: / /
EYE BOOK:

CONNECTION NUMBER



	ADDRESS: 212 PORT HILLS Ave	OWNER:	RECEIVED:	CONNECTION NUMBER
	LEGAL DESCRIPTION:	DRAINLAYER: KAT	BLOCK PLAN:	
	PROJECT No.: 1207856	PLUMBER:	PLOTTED: / /	
	DATE: 25/3/03	FIELD OFFICER: C Taylor	EYE BOOK:	



32501

CHRISTCHURCH CITY COUNCIL - DRAINAGE PICK UP

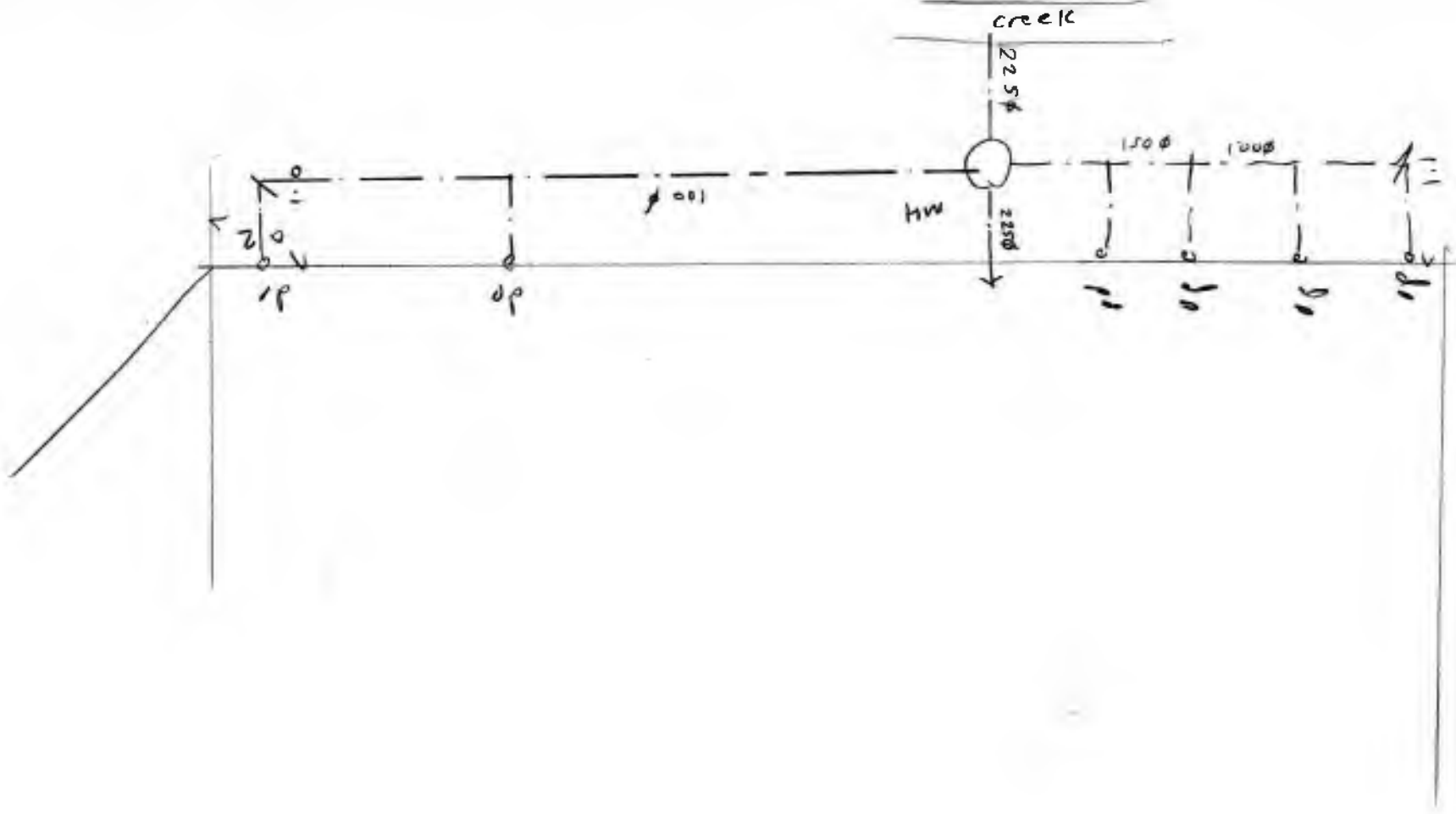


ADDRESS: 212 Port Hills Rd
 LEGAL DESCRIPTION:
 PROJECT No: 2097856
 DATE: 15-5-7

OWNER:
 DRAINLAYER: Gordon Keating
 PLUMBER:
 FIELD OFFICER: K.S. Walsh

RECEIVED:
 BLOCK PLAN:
 PLOTTED: / /
 EYE BOOK:

CONNECTION NUMBER





CHRISTCHURCH
CITY COUNCIL - YOUR PEOPLE - YOUR CITY

DRAINAGE PICKUP

CONSENT NUMBER

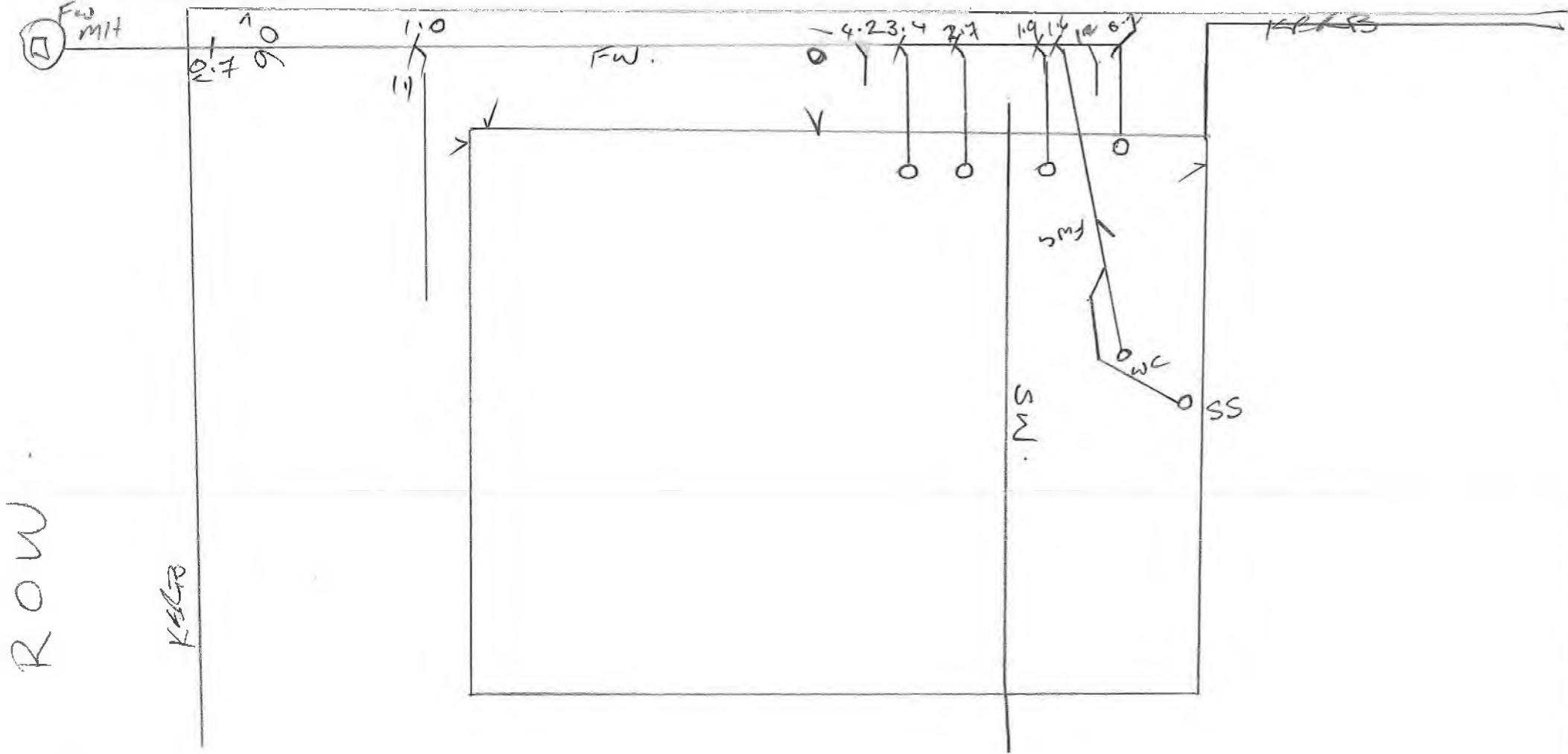
Address: 2 MARY MULLAR DU

Drainlayer: D. WICKIE

10088170

Date: 10-10-08

Inspector: FISIT





DRAINAGE PICKUP

CONNECTION NUMBER

Address: 15 Mary Muller Drive

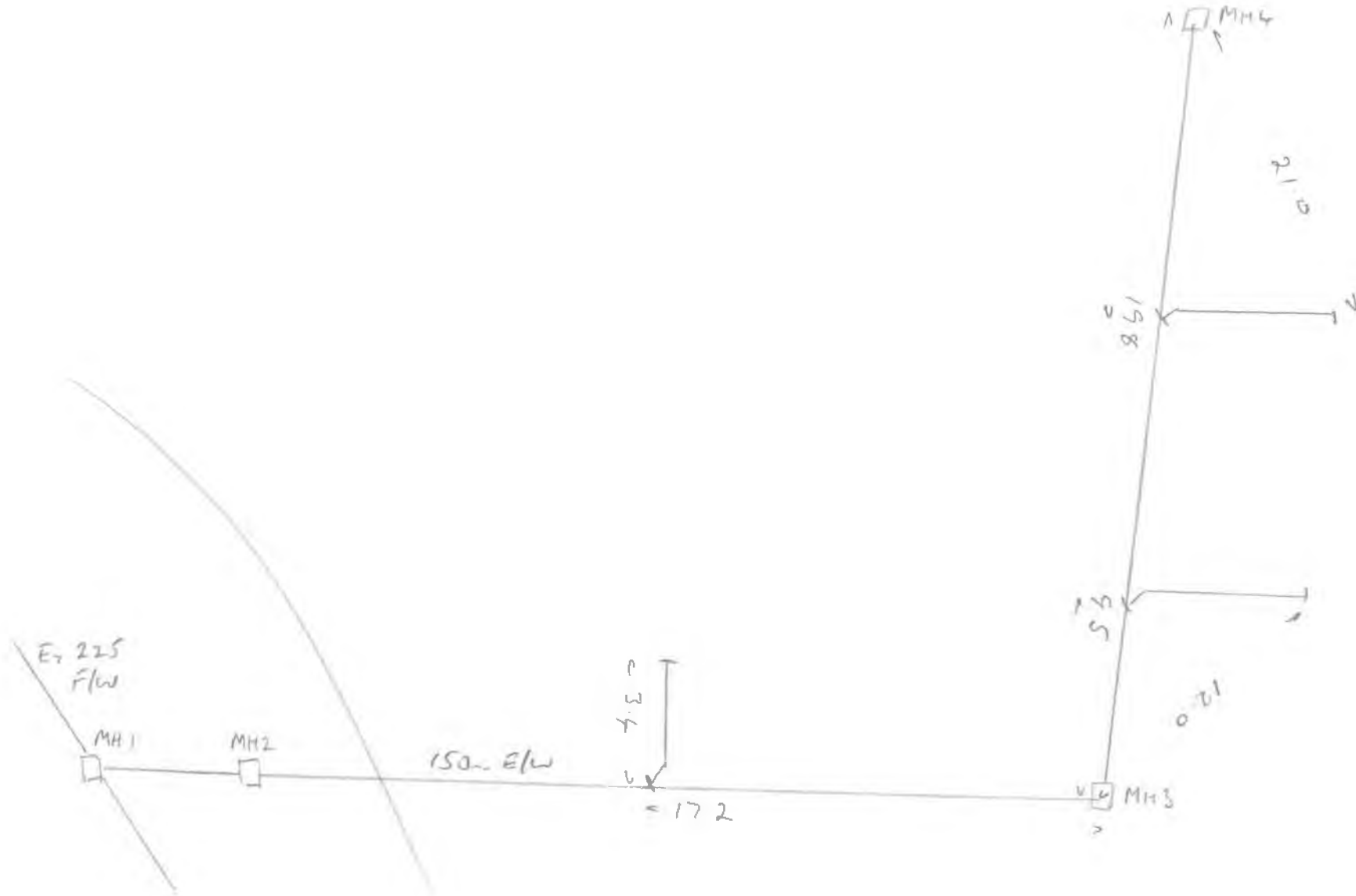
Owner:

Project No: 100-76086

Drainlayer: Terao

Date: 3.7.07

Inspector: Morlock





DRAINAGE PICKUP

CONNECTION NUMBER

Address: 15 Mary Muller Dr

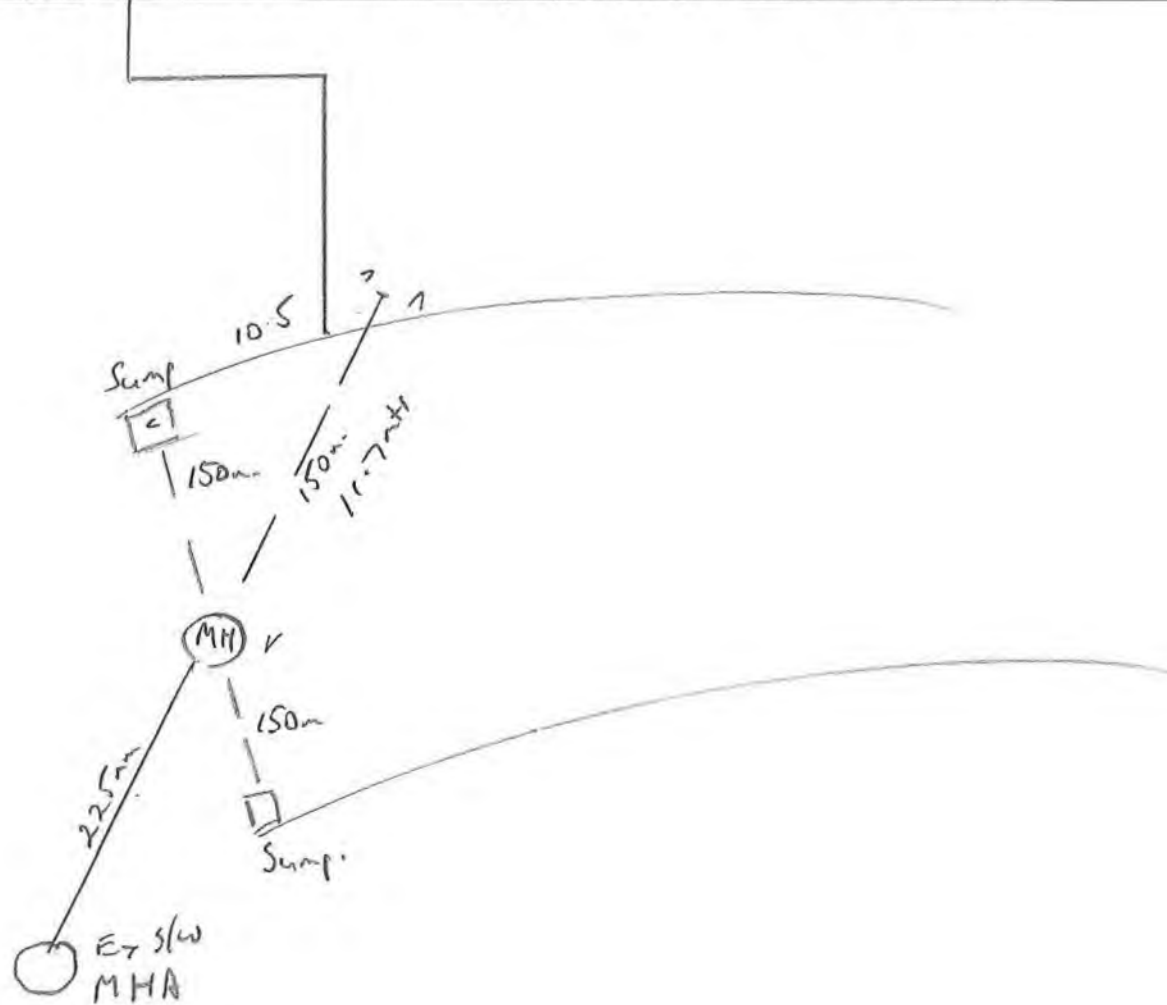
Owner:

Project No: 100-76086

Drainlayer: Teraco

Date: 10.7.07

Inspector: Mortlock





DRAINAGE PICKUP

CONNECTION NUMBER

Address: 15 Mary Muller Drive

Owner:

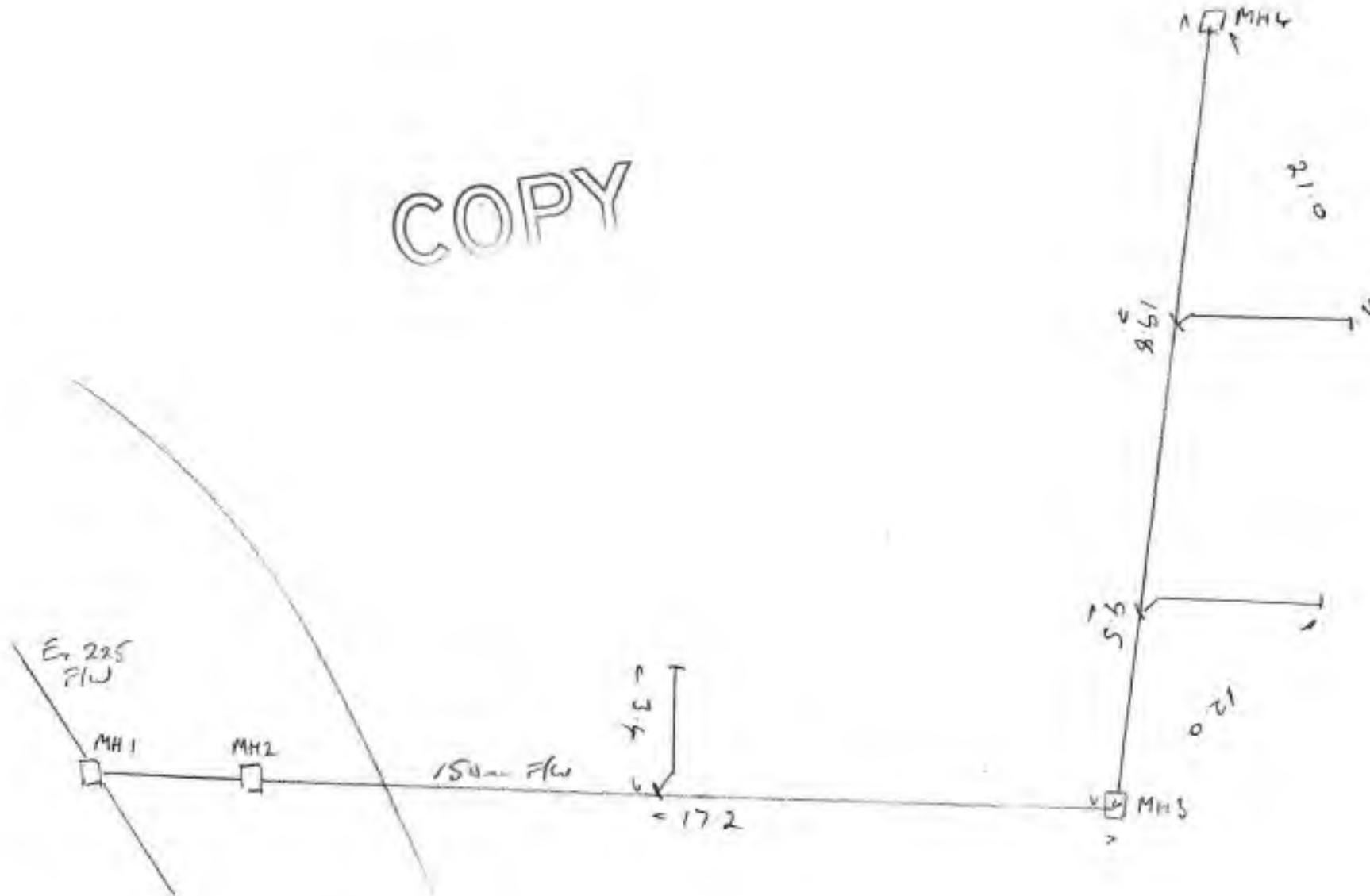
Project No: 100-76086

Drainlayer: Tetaco

Date: 3-7-07

Inspector: Mori Hoch

COPY





ABA1011 8267

1819

family owned and operated



Peter Diver

44 Maces Rd, Bromley, Christchurch 8062
Ph 384-8111
Fax 384-8748
www.peterdiver.co.nz

plumbing - drainage ltd

Builder: CASTLE ROCK PROPERTIES

NRS

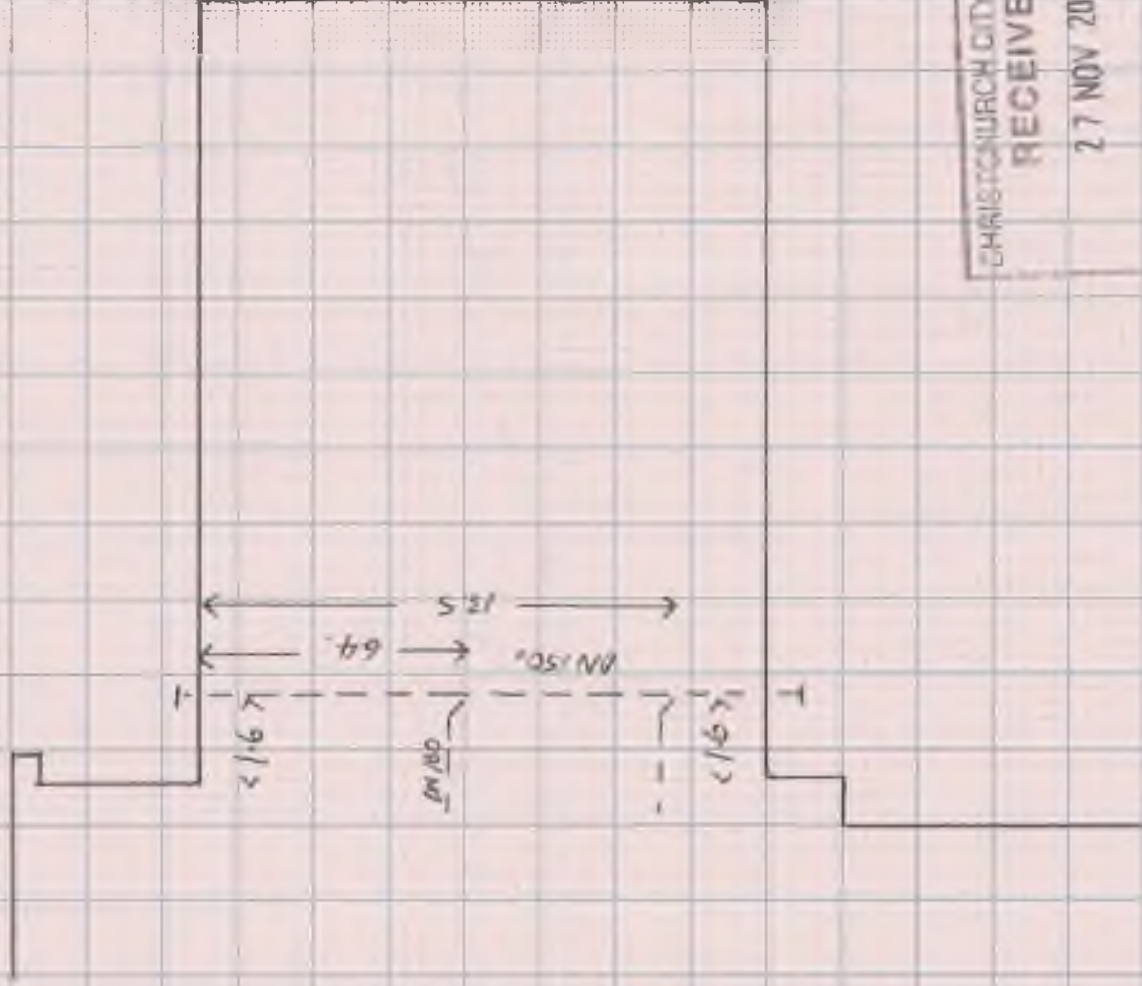
Drainlayer/Staff NAME 24/66

Job No: 35892

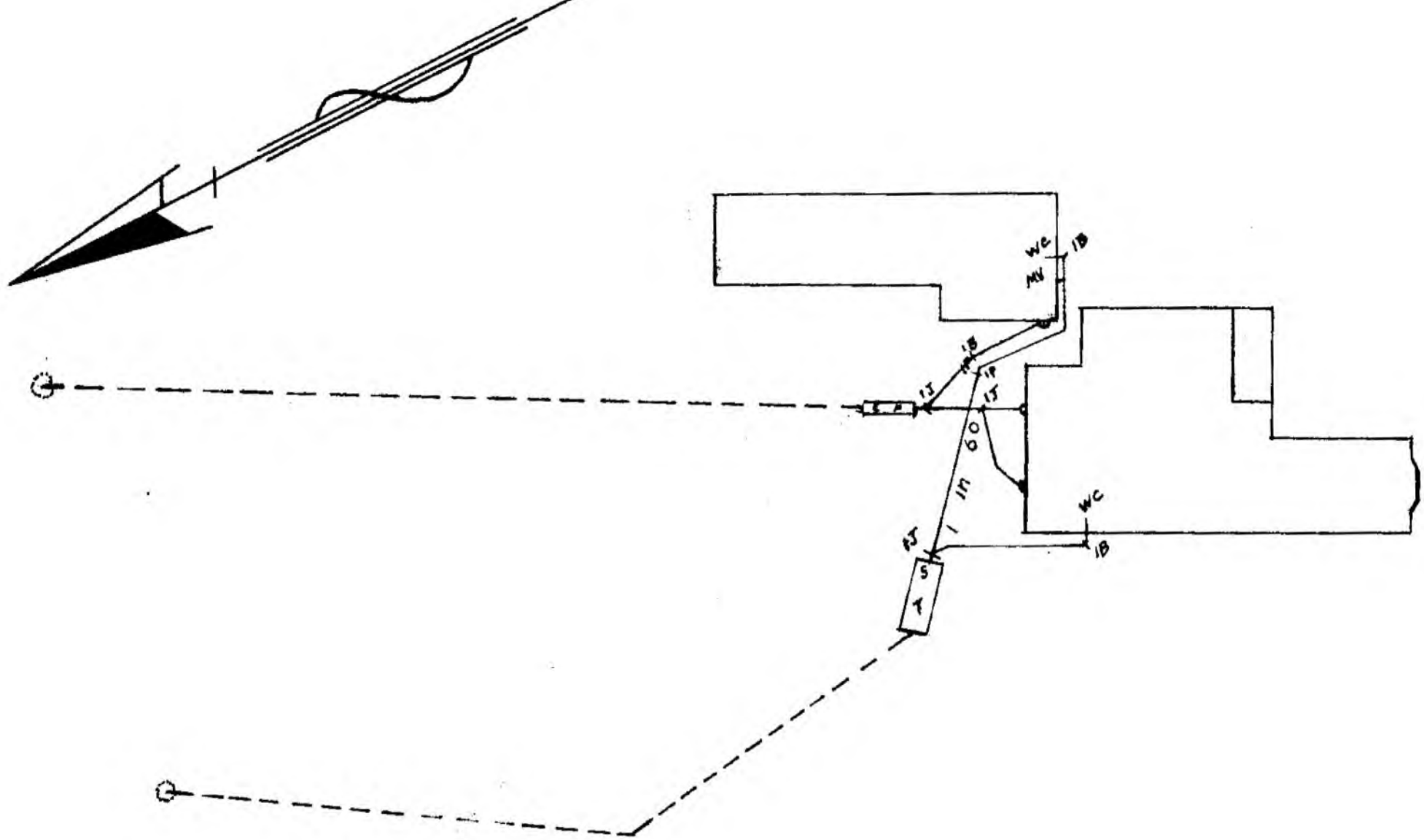
Job Address: 10 MARY MULLER DRIVE,

Date: 27/10/2012

SW GRADE 1:200



CHRISTCHURCH CITY COUNCIL
 RECEIVED
 27 NOV 2012
 CIVIC OFFICES



F. ULRICH

212 Port Hills Rd

CHRISTCHURCH CITY COUNCIL - DRAINAGE PICK UP



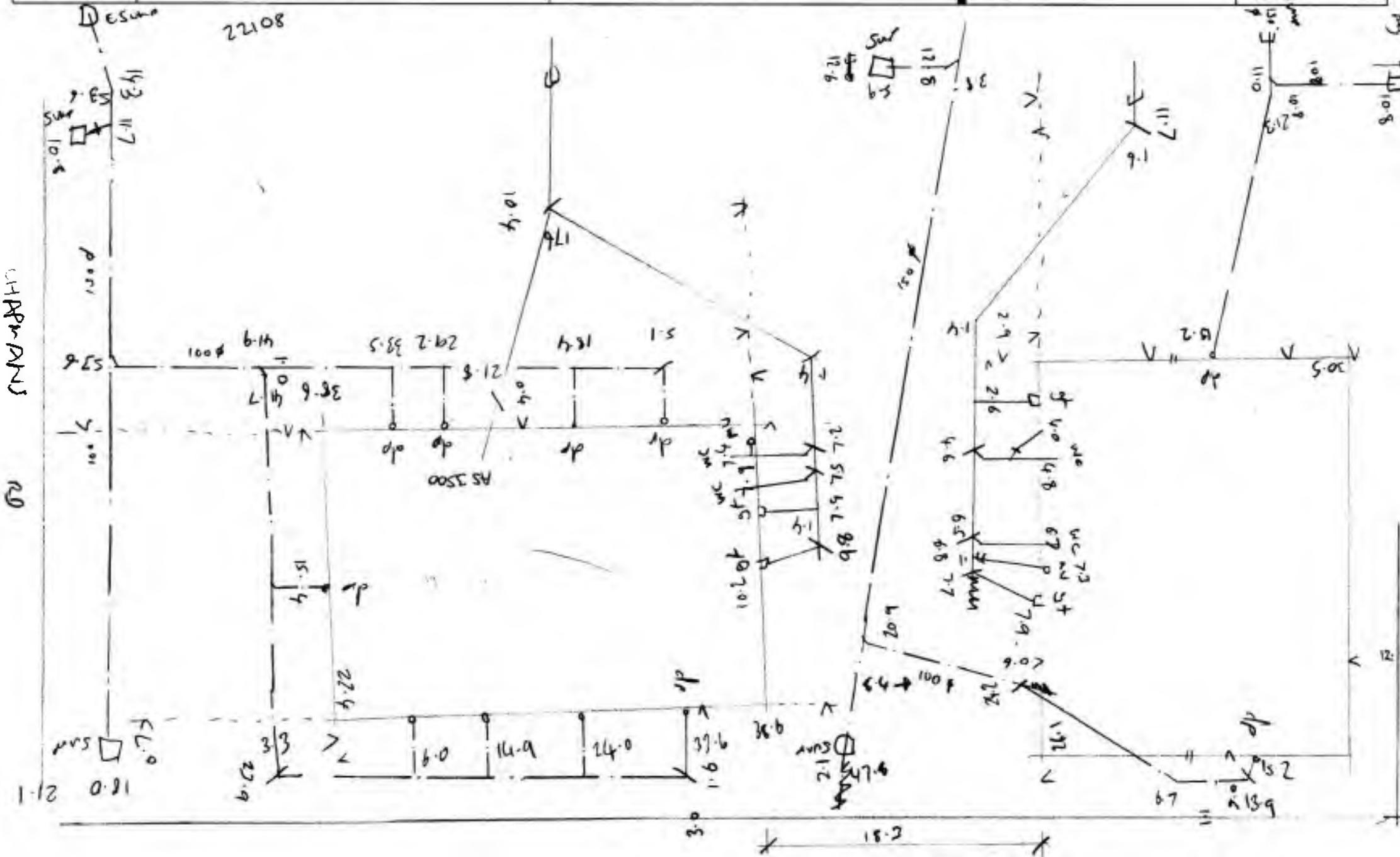
ADDRESS: 212 PORT HULLS RD
 LEGAL DESCRIPTION:
 PROJECT No: 10000375
 DATE: 25-2-00

OWNER:
 DRAINLAYER: GORDON KENNEDY
 PLUMBER:
 FIELD OFFICER: K.S. WALSH

RECEIVED: 05 APR 2000
 BLOCK PLAN: ST 74
 PLOTTED:
 EYE BOOK:

CONNECTION NUMBER

74



26167

CHRISTCHURCH CITY COUNCIL - DRAINAGE PICK UP

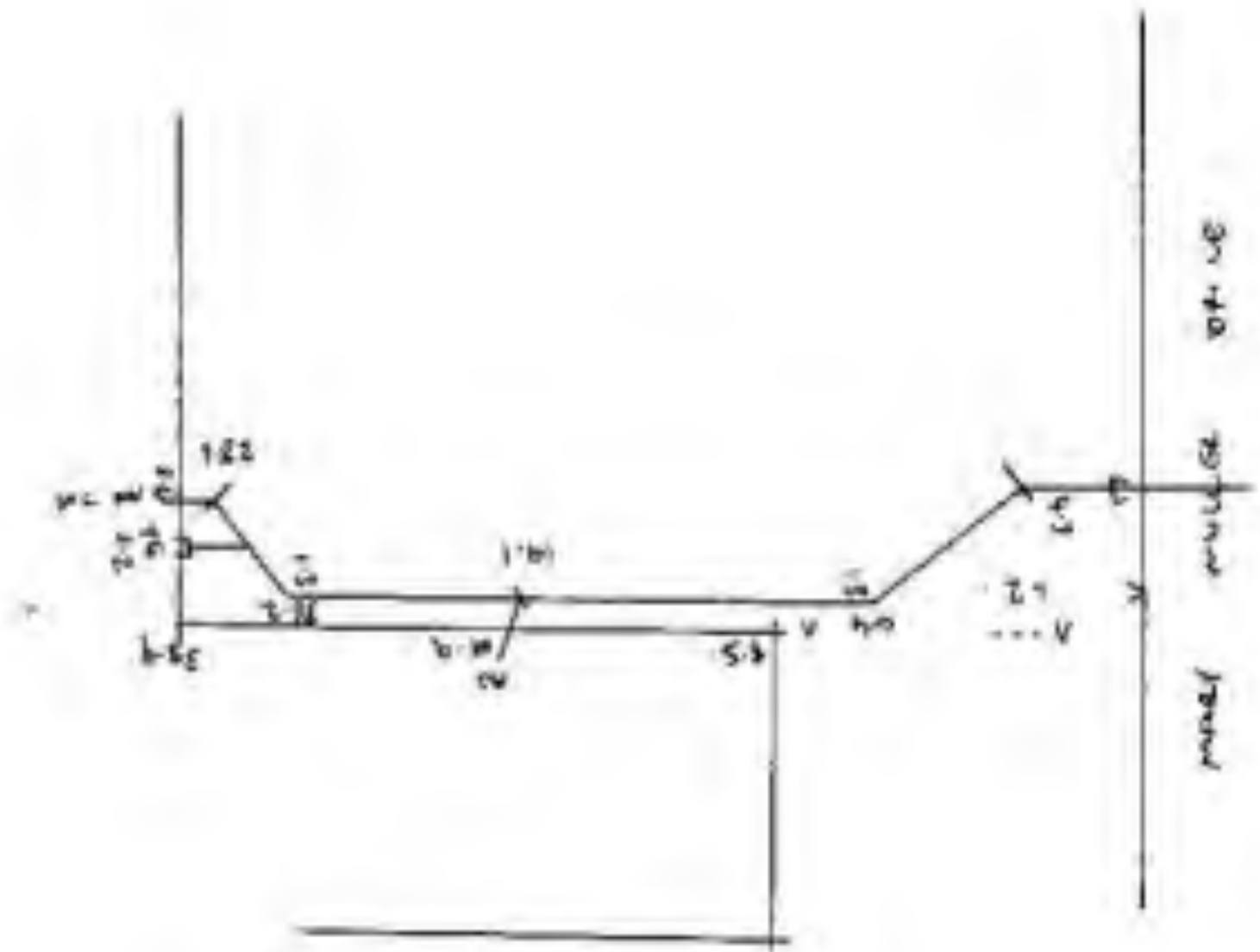


ADDRESS: 212 PORT HILLS RD
 LEGAL DESCRIPTION
 PROJECT No: 10003925
 DATE: 25-7-0

OWNER:
 DRAINLAYER: GOLDEN KENNEDY
 PLUMBER:
 FIELD OFFICER: K. J. WALSH

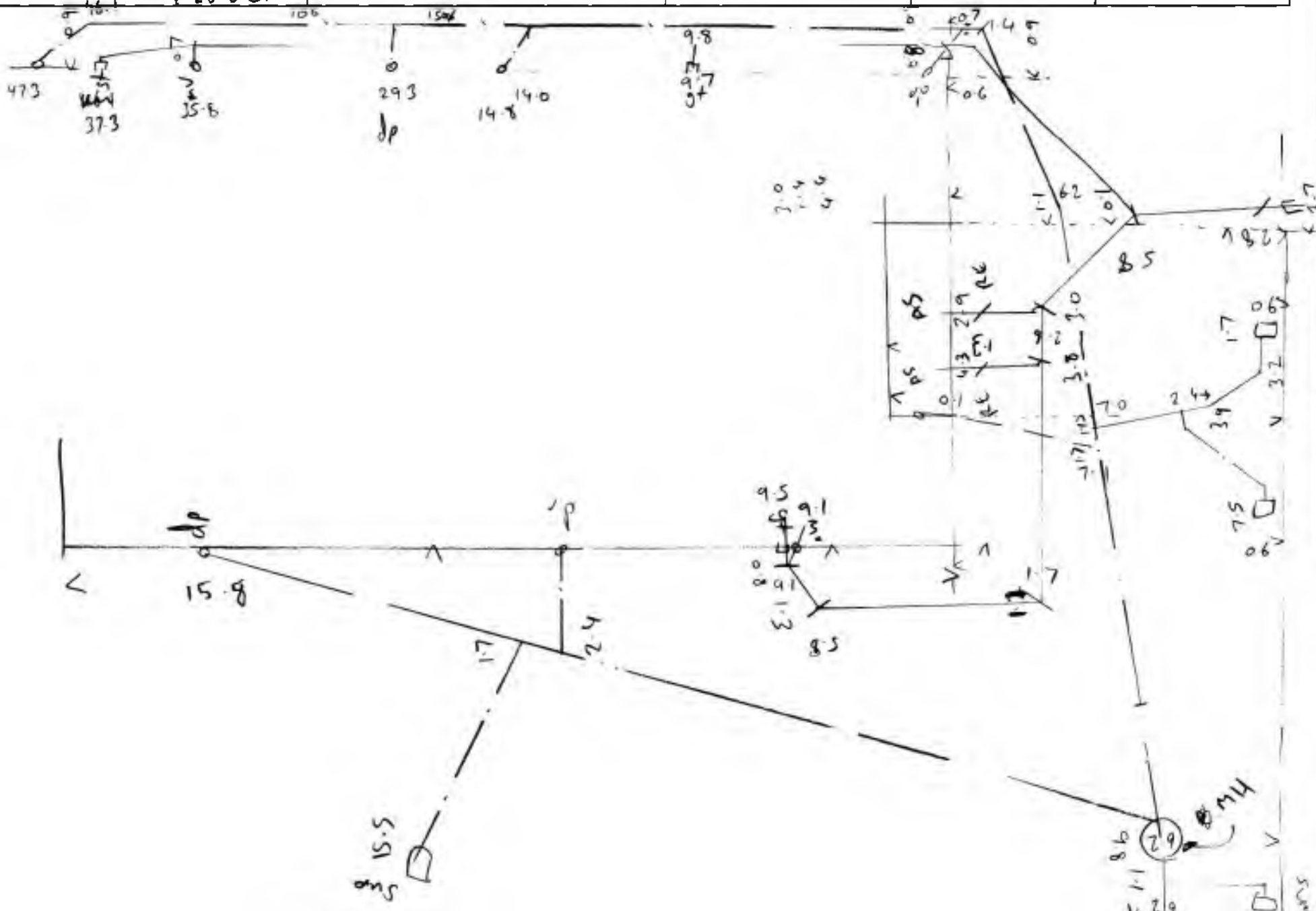
RECEIVED:
 BLOCK PLAN:
 PLOTTED: / /
 EYEBOOK:

CONNECTION NUMBER





28348	DRAINAGE PICKUP	RECEIVED	PICKUP CHECKED <input type="checkbox"/>	CONNECTION NUMBER
ADDRESS 212 PORT HILLS RD	OWNER	BLOCK PLAN	PLOTTED BY	
PROJECT NO. 10013483	DRAIN LAYER THEO NEWFIELD	1/2000 NO.	DATE PLOTTED	
DATE 2.8.01 & 17.8.01 & 21.08.01 & 28.8.01	FIELD OFFICER K.J. WALSH	DATABASE ID	D/B UPDATED <input type="checkbox"/>	





CHRISTCHURCH
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28351

DRAINAGE PICKUP

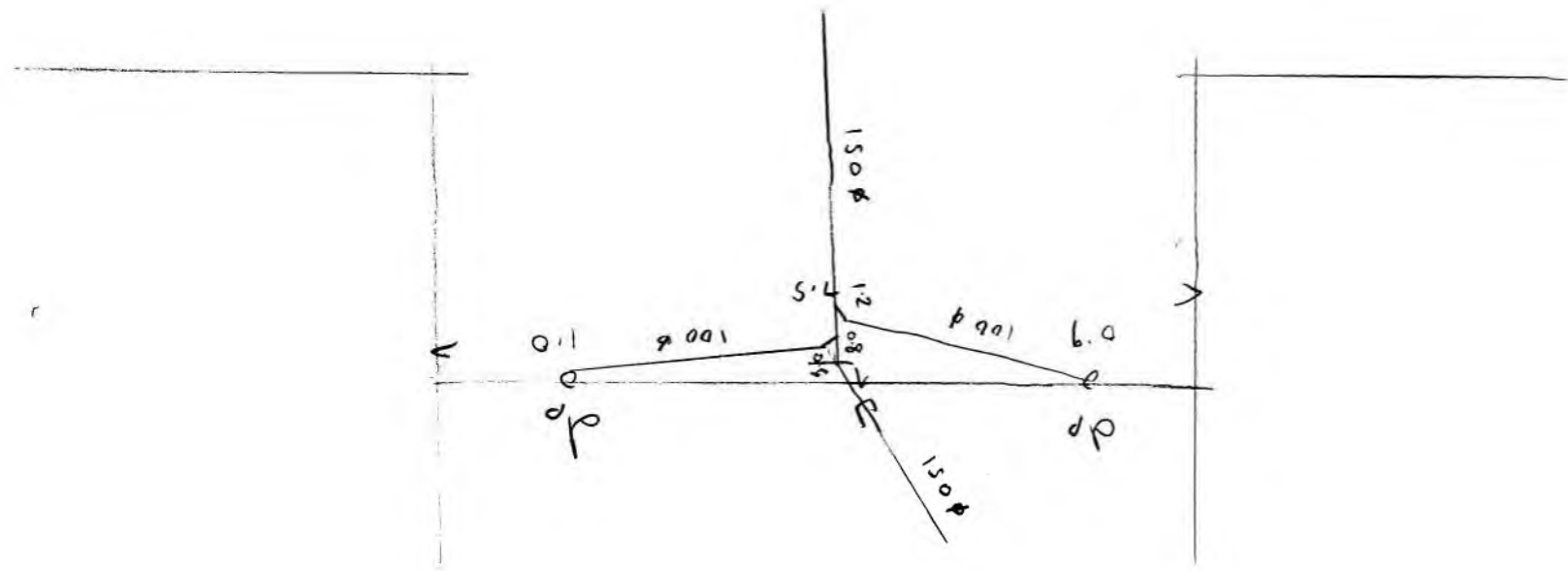
ADDRESS 218 PORT HILLS
PROJECT NO. (2 MART MURDER DR)
DATE 10013568

OWNER
DRAIN LAYER GORDON KENNEDY
FIELD OFFICER K.J. WALSH

RECEIVED
BLOCK PLAN
1/2000 NO.
DATABASE ID

PICKUP CHECKED
PLOTTED BY
DATE PLOTTED
D/B UPDATED

CONNECTION NUMBER



29230

2 MARY MULLER DRIVE

CHRISTCHURCH CITY COUNCIL - DRAINAGE PICK UP

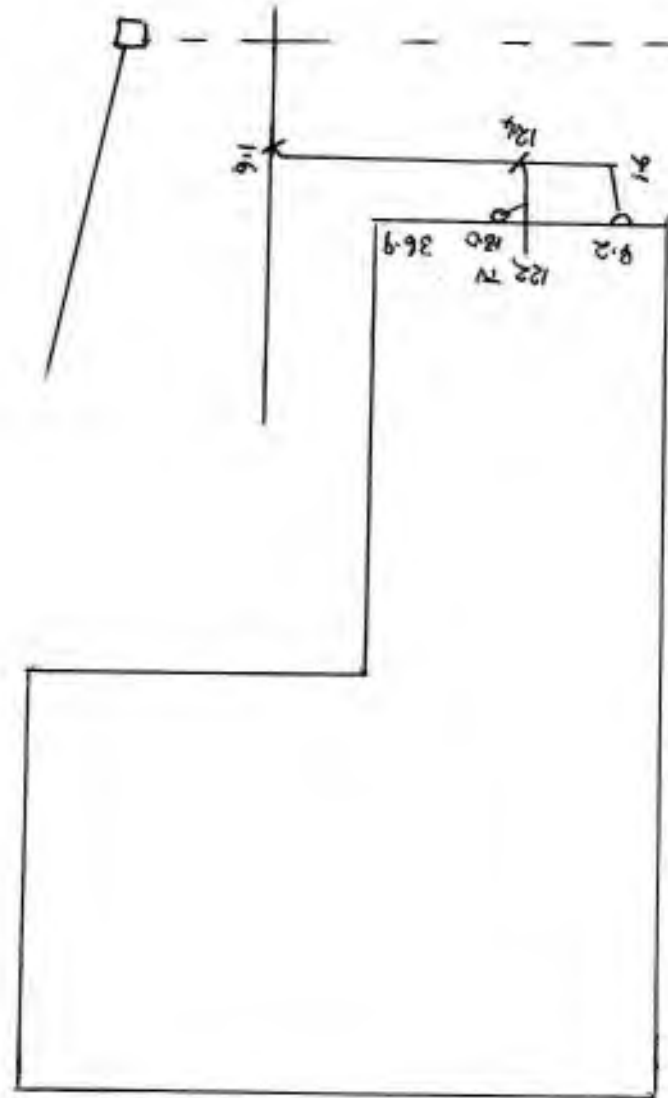


ADDRESS: 212 PORT HILLS RD
 LEGAL DESCRIPTION:
 PROJECT No.: 100.104.04
 DATE: 6.6.2001

OWNER:
 DRAINLAYER: KT
 PLUMBER:
 FIELD OFFICER: RA

RECEIVED:
 BLOCK PLAN:
 PLOTTED: / /
 EYE BOOK:

CONNECTION NUMBER



CHRISTCHURCH CITY COUNCIL - DRAINAGE PICK UP

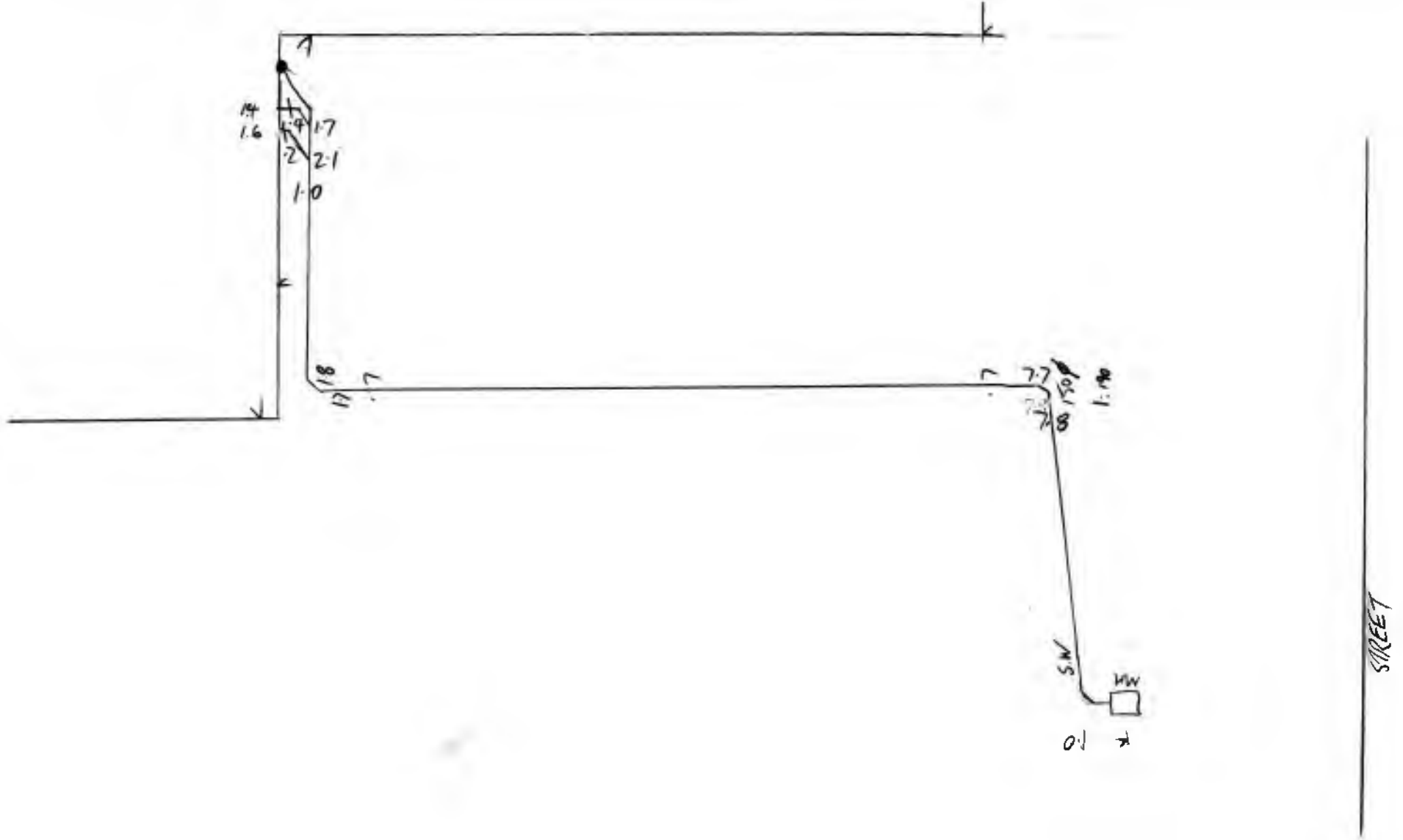


ADDRESS: *2 Mary Muller*
 LEGAL DESCRIPTION:
 PROJECT No.: *100*
 DATE: *25/11/03*

OWNER:
 DRAINLAYER: *Peter Barrett*
 PLUMBER:
 FIELD OFFICER: *S. WALTERS*

RECEIVED:
 BLOCK PLAN:
 PLOTTED: / /
 EYE BOOK:

CONNECTION NUMBER





ABA 1011 8267, 0123 family owned and operated
 44 Maces Rd, Bromley, Christchurch 8062
 Ph 384-8111
 Fax 384-8748
 www.peterdiver.co.nz

family owned and operated



Peter Diver

plumbing + drainage ltd

Builder:

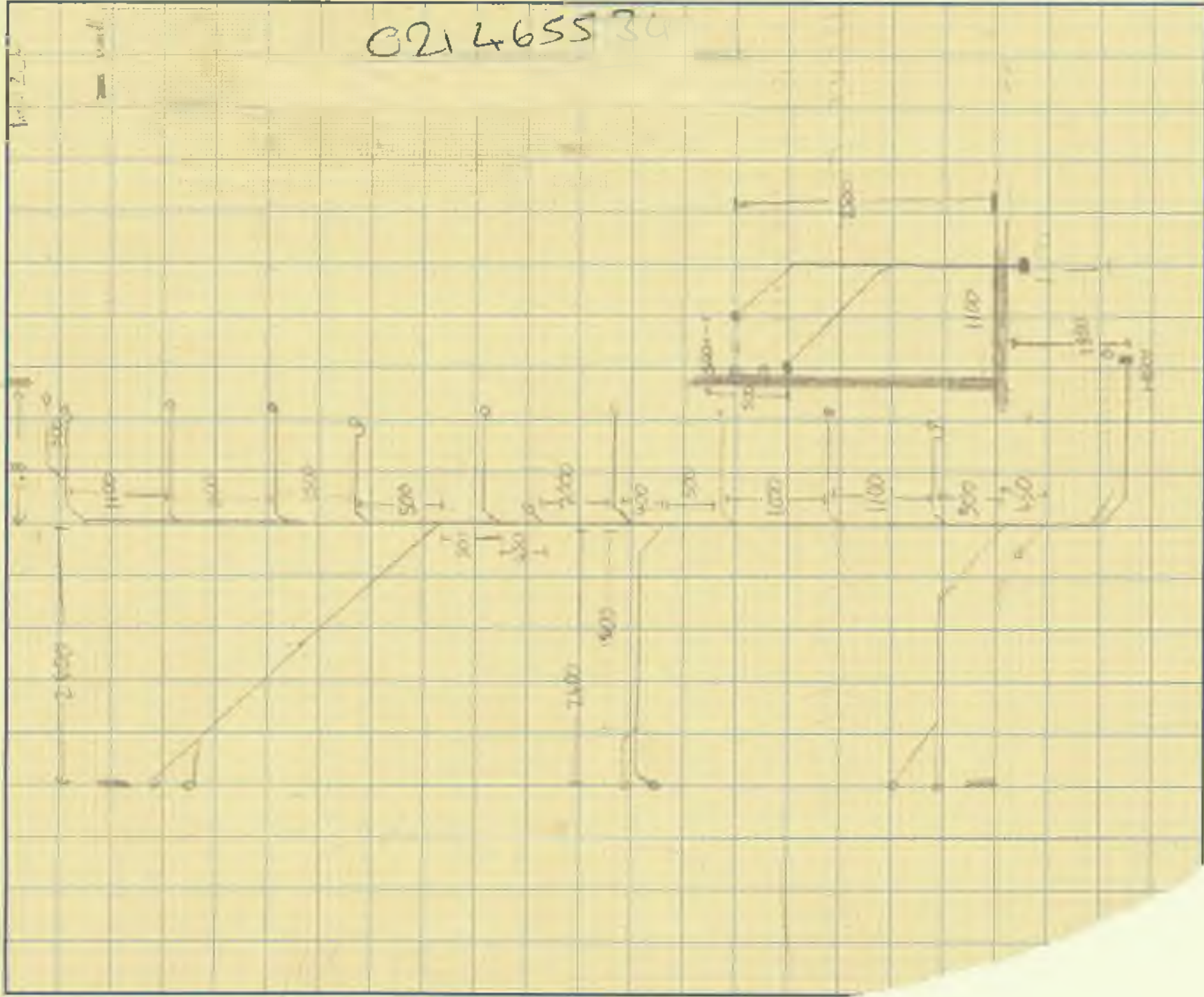
Drainlayer/Staff Wayne Collier

Job No: 12393

Job Address:

10 Mary Miller

Date: 26/12/12





DRAINAGE PICKUP

CONNECTION NUMBER

Address: 15 MARYMULLEN DR

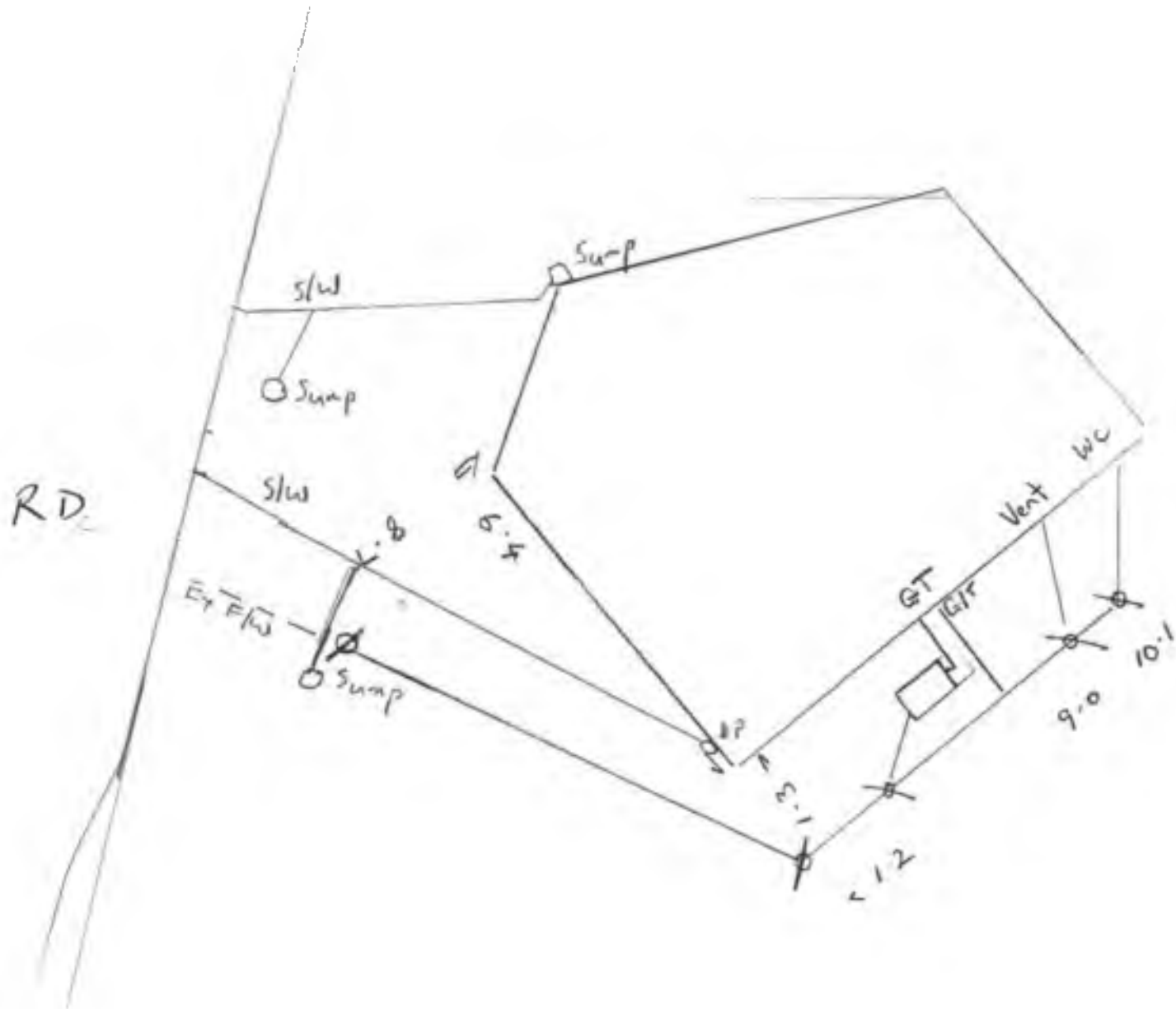
Owner: CASTLE ROCK RENTALS

Project No: 10062360

Drainlayer: PATE JAMES

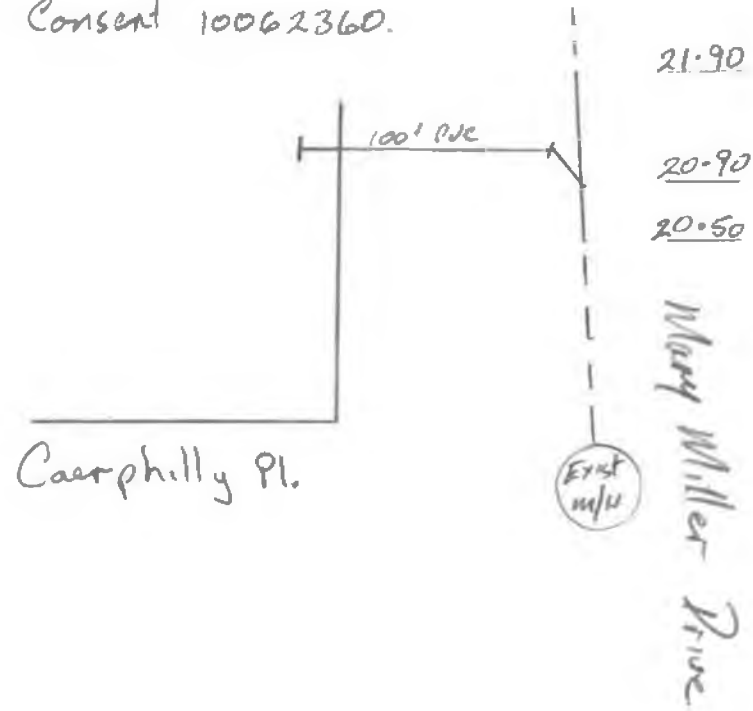
Date: 22/5/06

Inspector: LUCY BOWEN



Catv John
R... ..

Mary Miller Drive 225' Sewer
Main Repair and New 100' PVC
Sewer Lat to No 5/06 Calcom
Consent 10062360.





DRAINAGE PICKUP

CONNECTION NUMBER

Address: 15 MARIAMULLEN DRIVE

Owner: CASTLE ROCK RANGERS

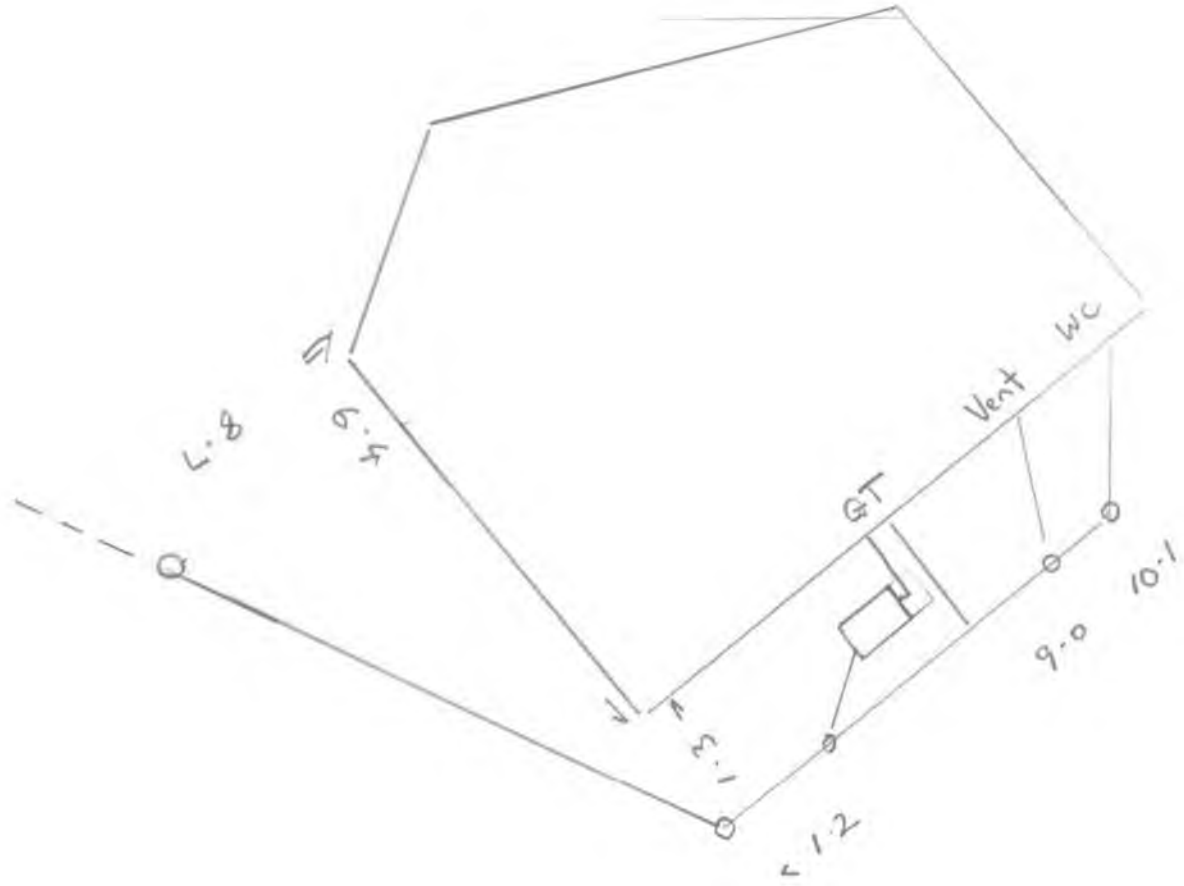
Project No: 10062360

Drainlayer: PATE DEWNEY

Date: 22/5/06

Inspector: LUCY BROWN

RD₀



CHRISTCHURCH CITY COUNCIL - DRAINAGE PICK UP

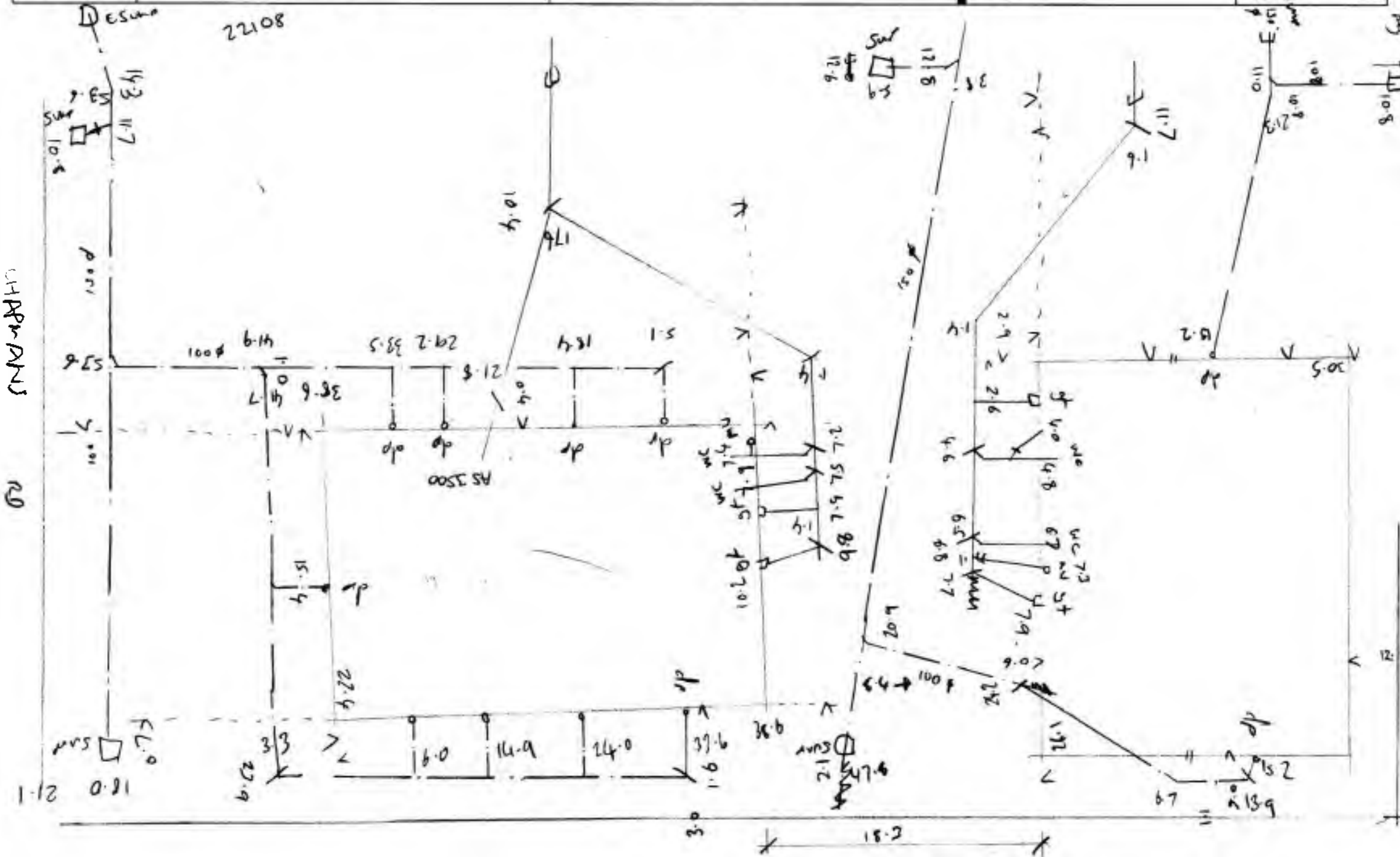


ADDRESS: 212 PORT HULLS RD
 LEGAL DESCRIPTION:
 PROJECT No: 10000375
 DATE: 25-2-00

OWNER:
 DRAINLAYER: GORDON KENNEDY
 PLUMBER:
 FIELD OFFICER: K.J. WALSH

RECEIVED: 05 APR 2000
 BLOCK PLAN: ST 74
 PLOTTED:
 EYE BOOK:

CONNECTION NUMBER



26167

CHRISTCHURCH CITY COUNCIL - DRAINAGE PICK UP

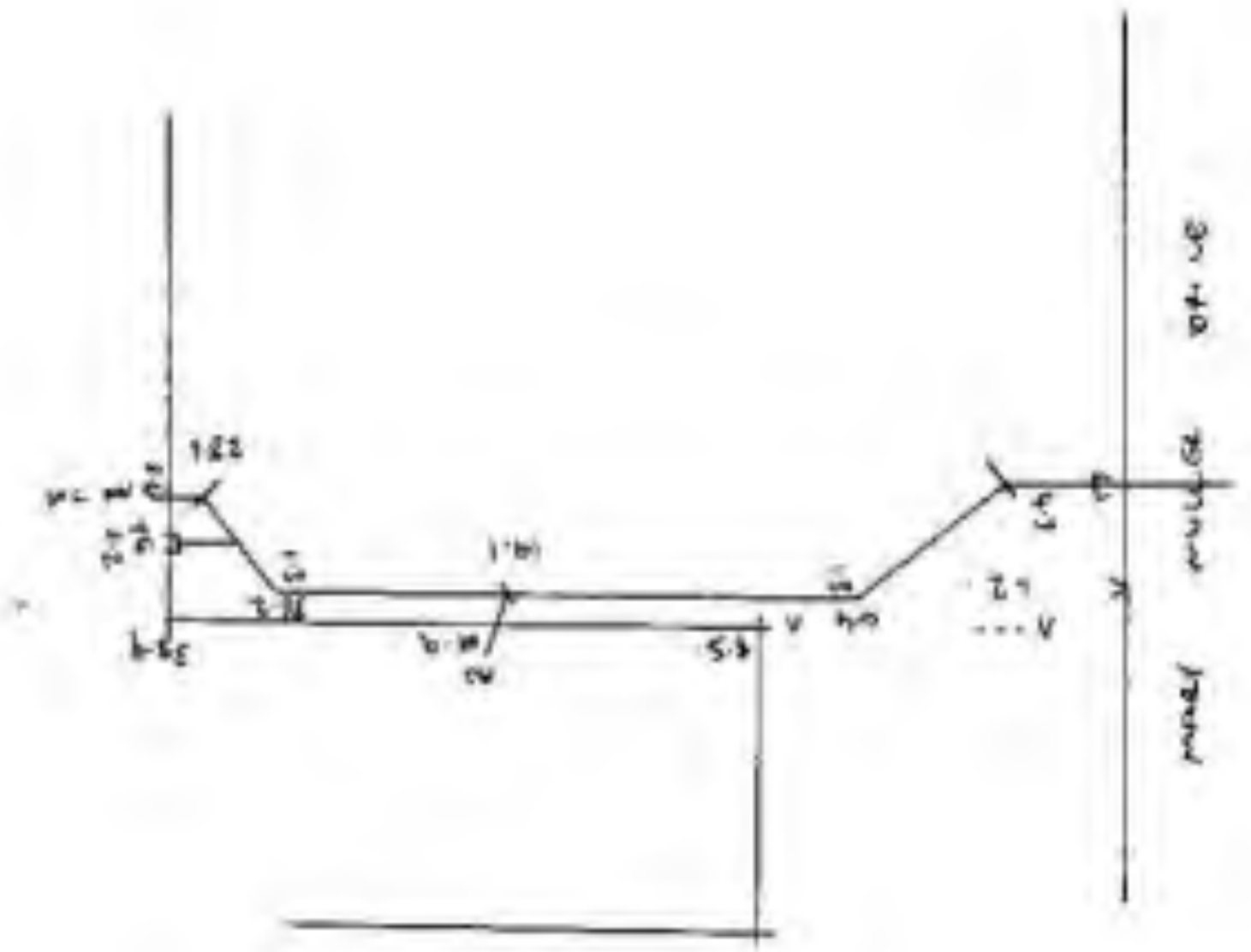


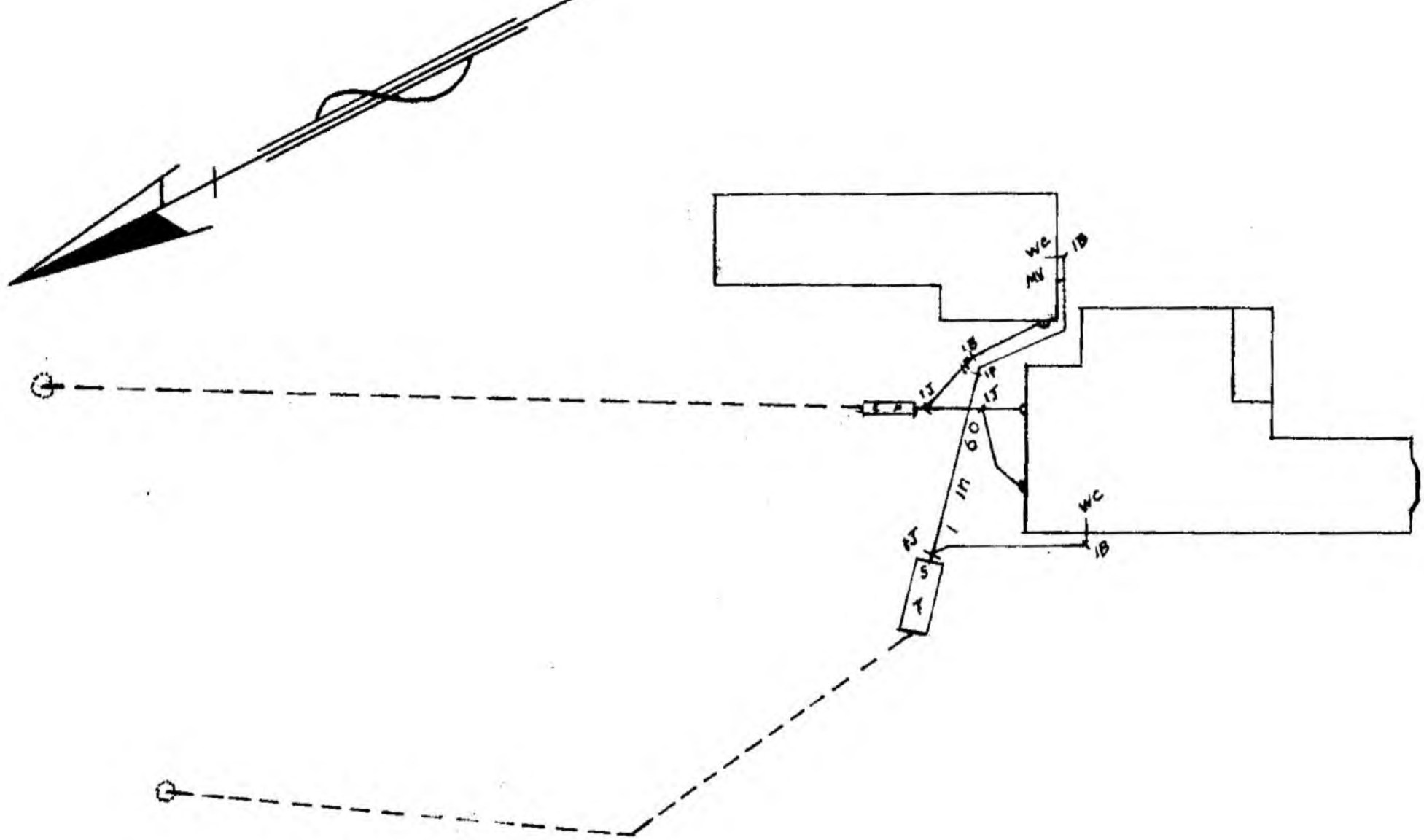
ADDRESS: 212 PORT HILLS RD
 LEGAL DESCRIPTION
 PROJECT No: 10003925
 DATE: 25-7-0

OWNER:
 DRAINLAYER: GOLDEN KENNEDY'S
 PLUMBER:
 FIELD OFFICER: K. J. WALSH

RECEIVED:
 BLOCK PLAN:
 PLOTTED: / /
 EYEBOOK:

CONNECTION NUMBER





F. ULRICH

212 Port Hills Rd

29230

2 MARY MULLER DRIVE

CHRISTCHURCH CITY COUNCIL - DRAINAGE PICK UP

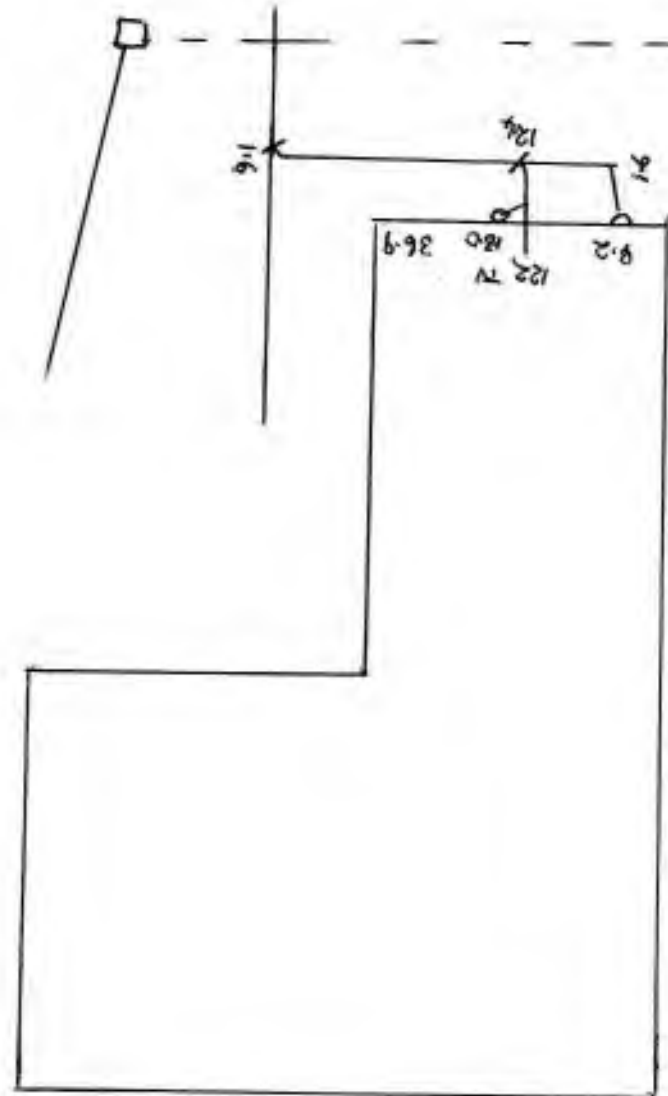



ADDRESS: 212 PORT HILLS RD
 LEGAL DESCRIPTION:
 PROJECT No.: 100.10404
 DATE: 6.6.2001

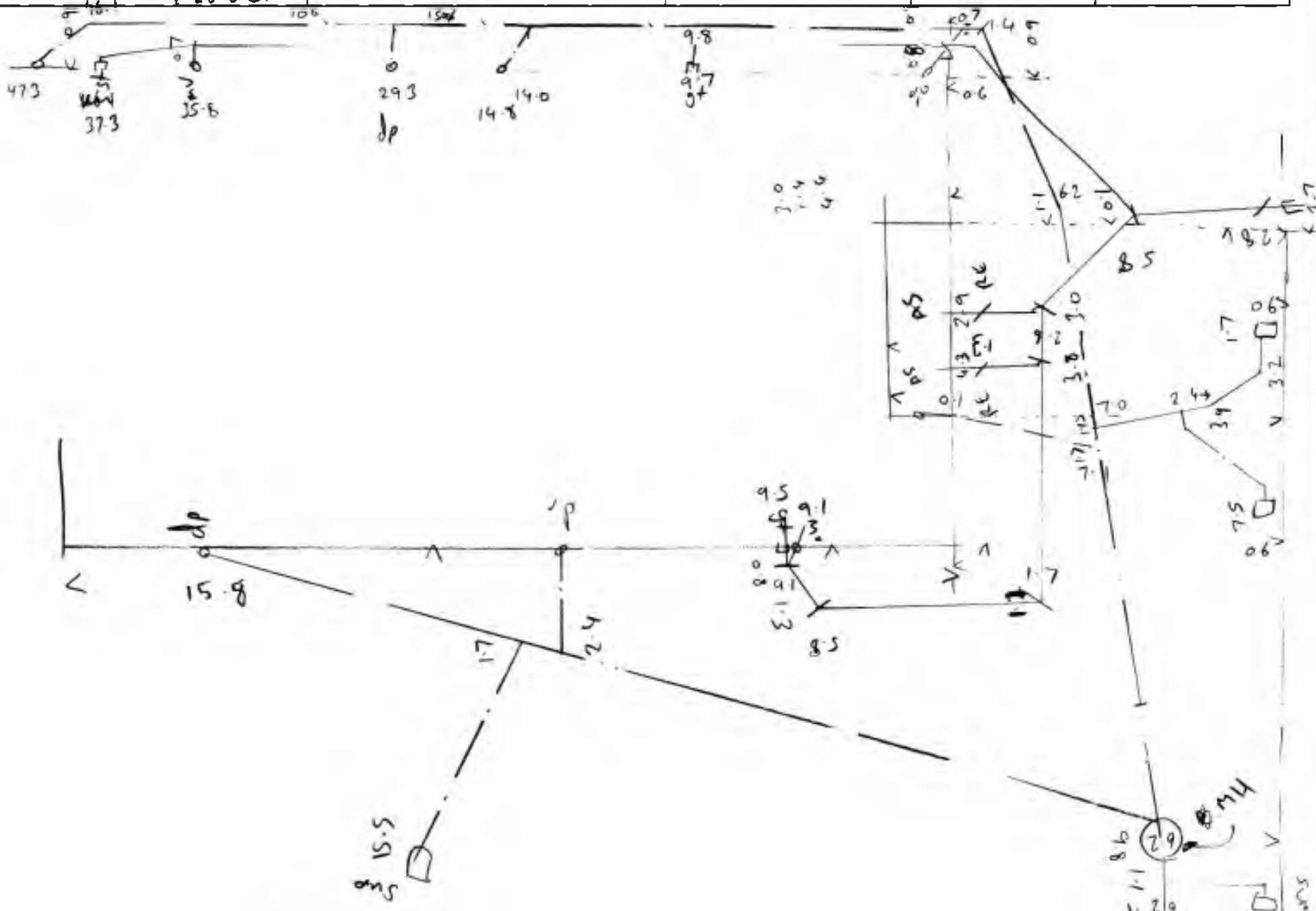
OWNER:
 DRAINLAYER: KT
 PLUMBER:
 FIELD OFFICER: RA

RECEIVED:
 BLOCK PLAN:
 PLOTTED: / /
 EYE BOOK:

CONNECTION NUMBER



 <p>CHRISTCHURCH CITY COUNCIL - YOUR PEOPLE YOUR CITY</p>	28348 DRAINAGE PICKUP		RECEIVED	PICKUP CHECKED <input type="checkbox"/>	CONNECTION NUMBER	
	ADDRESS 212 PORT HILLS RD		BLOCK PLAN	PLOTTED BY		
	PROJECT NO. 10013483		OWNER	DATE PLOTTED		
DATE 2.8.01 & 17.8.01 & 21.08.01 & 28.8.01		DRAIN LAYER THEO NEWFIELD	1/2000 NO.	D/B UPDATED <input type="checkbox"/>		
		FIELD OFFICER K.J. WALSH	DATABASE ID			





CHRISTCHURCH
CITY COUNCIL - YOUR PEOPLE - YOUR CITY

28351

DRAINAGE PICKUP

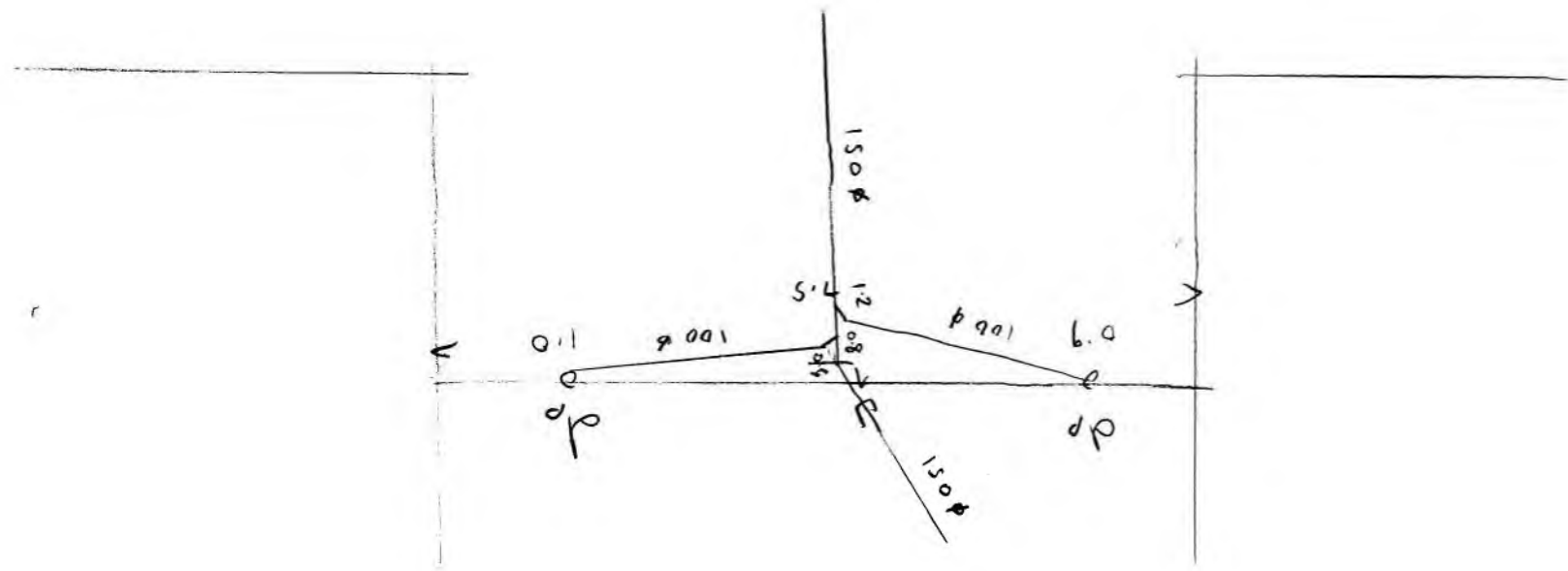
ADDRESS 218 PORT HILLS
PROJECT NO. (2 MART MURDER DR)
DATE 10013568

OWNER
DRAIN LAYER GORDON KENNING
FIELD OFFICER K.J. WALSH

RECEIVED
BLOCK PLAN
1/2000 NO.
DATABASE ID

PICKUP CHECKED
PLOTTED BY
DATE PLOTTED
D/B UPDATED

CONNECTION NUMBER



CHRISTCHURCH CITY COUNCIL - DRAINAGE PICK UP

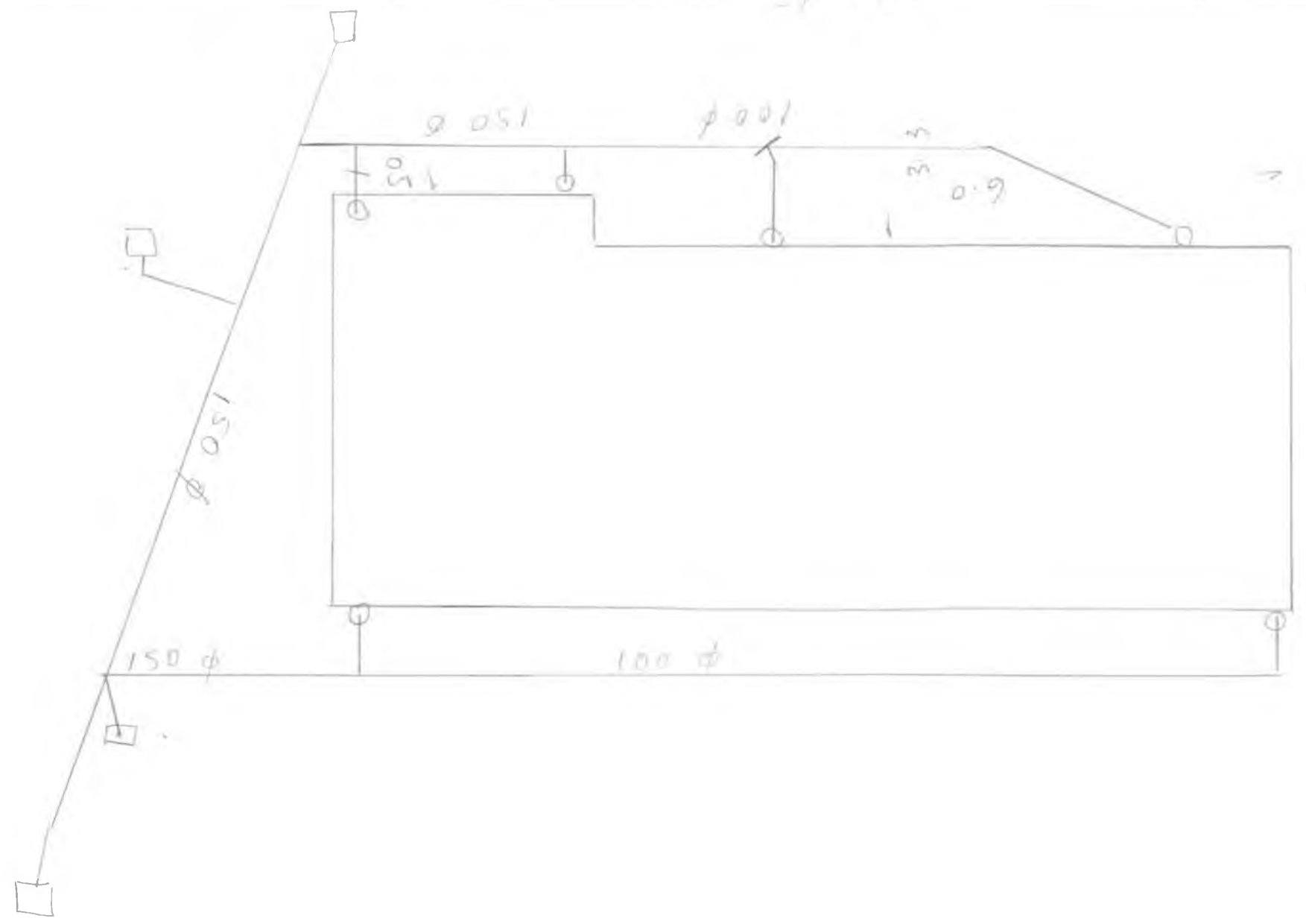


ADDRESS: 15 MARY MULLER
 LEGAL DESCRIPTION:
 PROJECT No.: 10054080.
 DATE: 2.9.5

OWNER:
 DRAINLAYER: D WILKIE
 PLUMBER:
 FIELD OFFICER: STUBBS

RECEIVED:
 BLOCK PLAN:
 PLOTTED: / /
 EYE BOOK:

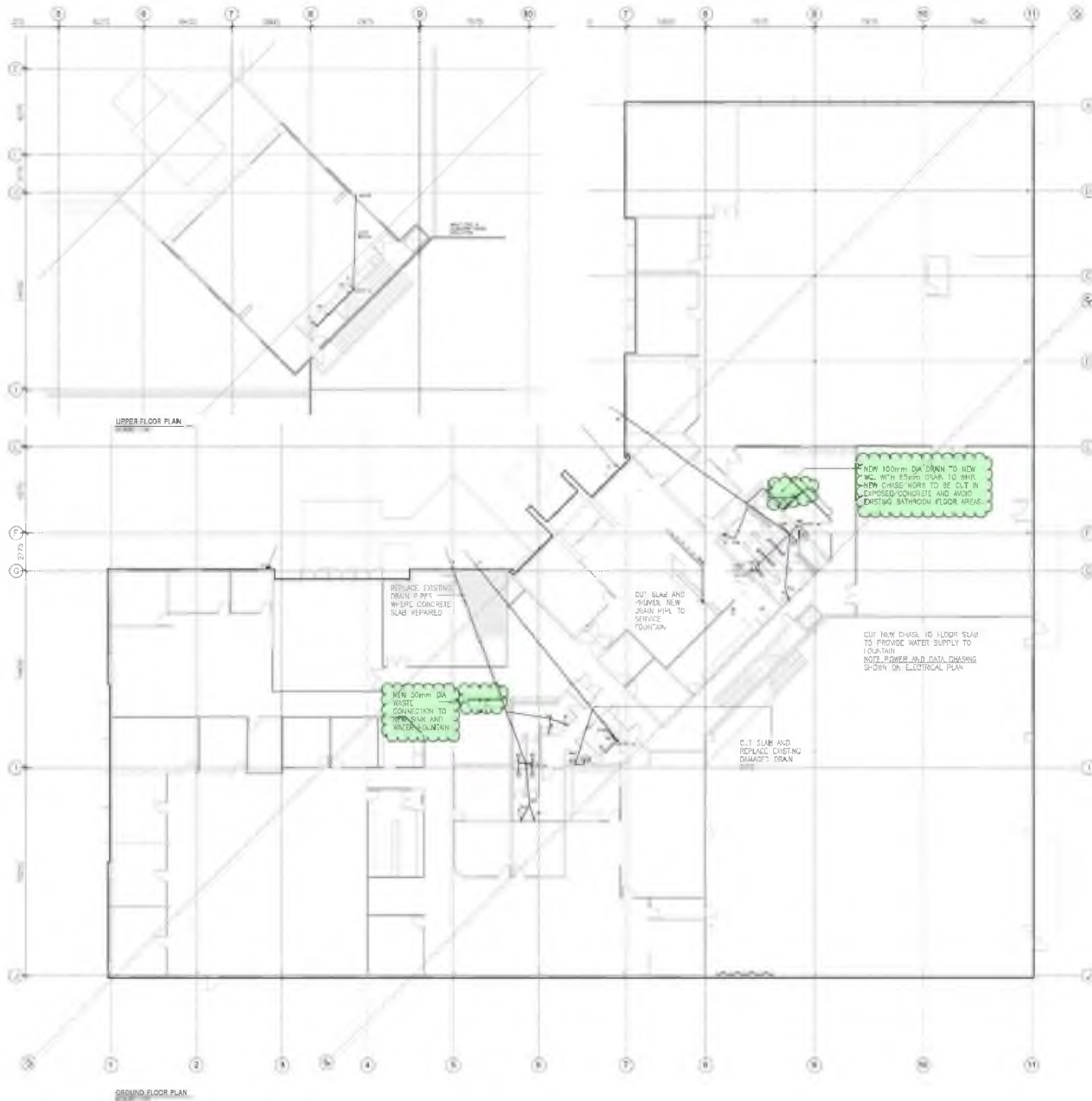
CONNECTION NUMBER



Ex MH

Project: 11 Mary Muller Drive Hillsborough

BCN/2016/7711
 18/01/2017 1:52:03 p.m.



ITEMS:

1. 100mm dia drain to new WC, with 50mm drain to whirl. New chase hole to be cut in exposed concrete and avoid existing bathroom floor areas.

ITEM	QTY	UNIT	PRICE	TOTAL
100mm dia drain	1	m	150.00	150.00
50mm dia drain	1	m	100.00	100.00
chase hole	1	sqm	200.00	200.00
concrete repair	1	sqm	100.00	100.00
total				550.00

2. 50mm dia waste connection to trench and woods laundry.

ITEM	QTY	UNIT	PRICE	TOTAL
50mm dia waste connection	1	m	100.00	100.00
total				100.00

3. Cut slab and repair new drain pipe to service fountain.

ITEM	QTY	UNIT	PRICE	TOTAL
cut slab and repair new drain pipe	1	m	100.00	100.00
total				100.00

4. Cut new chase to floor slab to provide water supply to fountain. Note power and data chassing shown on electrical plan.

ITEM	QTY	UNIT	PRICE	TOTAL
cut new chase to floor slab	1	m	200.00	200.00
total				200.00

5. Cut slab and replace existing damaged drain.

ITEM	QTY	UNIT	PRICE	TOTAL
cut slab and replace existing damaged drain	1	m	100.00	100.00
total				100.00

6. Track work

ITEM	QTY	UNIT	PRICE	TOTAL
Track work	1	m	100.00	100.00
total				100.00

7. Waterproofing

ITEM	QTY	UNIT	PRICE	TOTAL
Waterproofing	1	m	100.00	100.00
total				100.00

BCN/2016/7711

BC Plumbers As Built
 18/01/2017 1:52:03 p.m.

Brent Cowan REG 10589
 BC Plumbers Ltd
 P.O. Box 7214
 Christchurch 8140
 0274 525-510

Three Sixty Architecture

LONGBEACH
 LONGBEACH APPAREL
 SWANNDRILL
 MARY MULLER DRIVE
 CHRISTCHURCH
 DRAINAGE LAYOUT PLAN

Scale: 1:100
 Date: 18/01/2017
 209

Three Sixty Architecture
 ASQ House, Level 1
 61 Cambridge Terrace
 Christchurch 8013
 03 365 3349
 office@thesixtyarch.co.nz
 thesixtyarch.co.nz

CHRISTCHURCH CITY COUNCIL - DRAINAGE PICK UP

pm m2 ph 3842043 Day - Ross



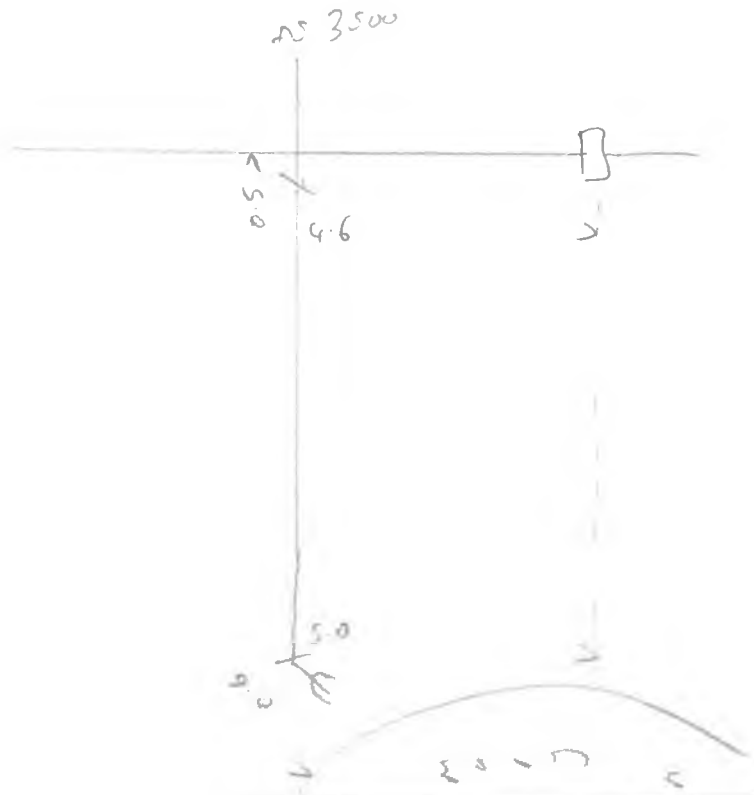
ADDRESS: 15 Mars Miller Dr
 LEGAL DESCRIPTION:
 PROJECT No.: 10054080
 DATE:

OWNER:
 DRAINLAYER: Dean Wilkie
 PLUMBER:
 FIELD OFFICER: K. Wilkie

RECEIVED:
 BLOCK PLAN:
 PLOTTED: / /
 EYE BOOK:

CONNECTION NUMBER

port 115

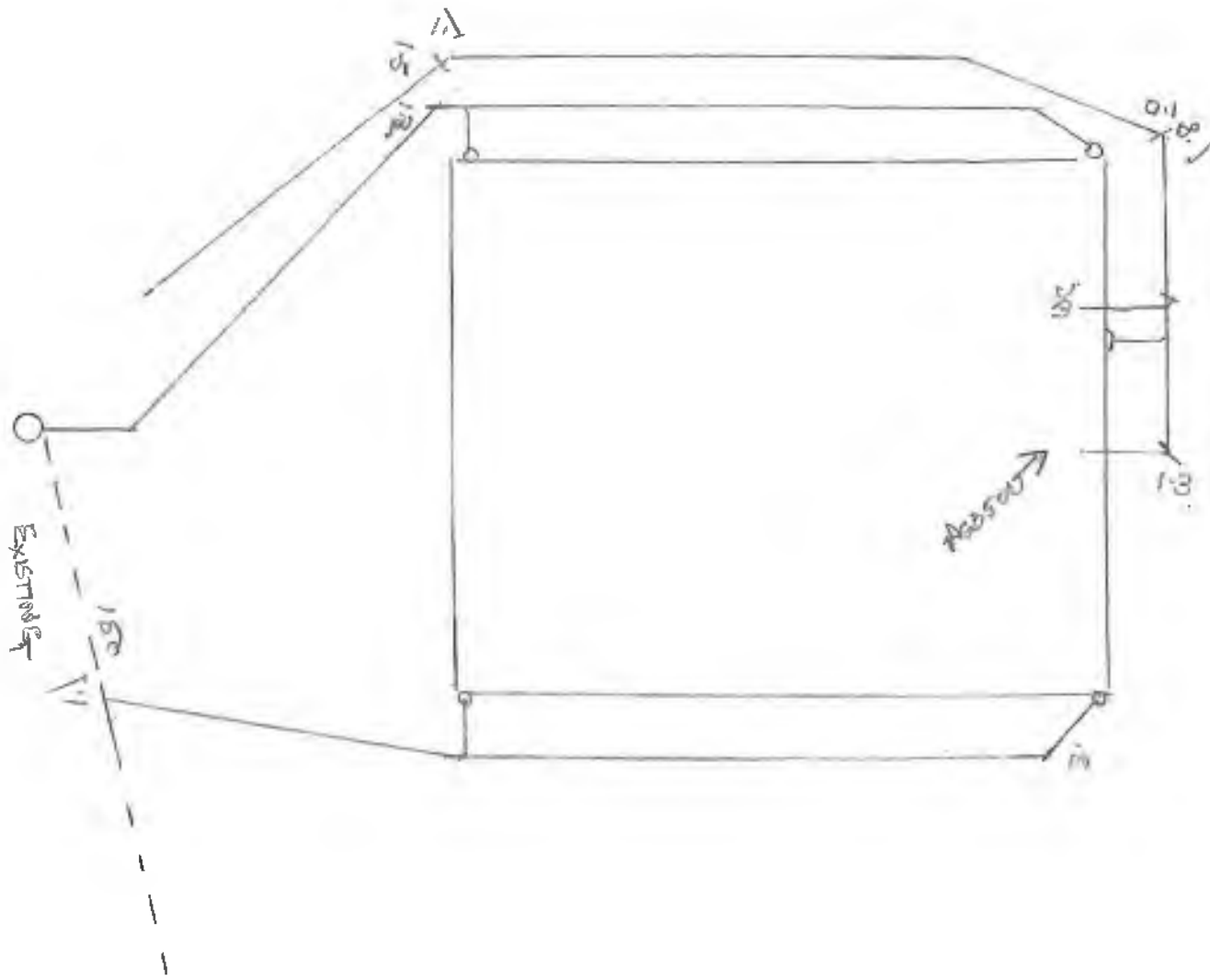





DRAINAGE PICKUP

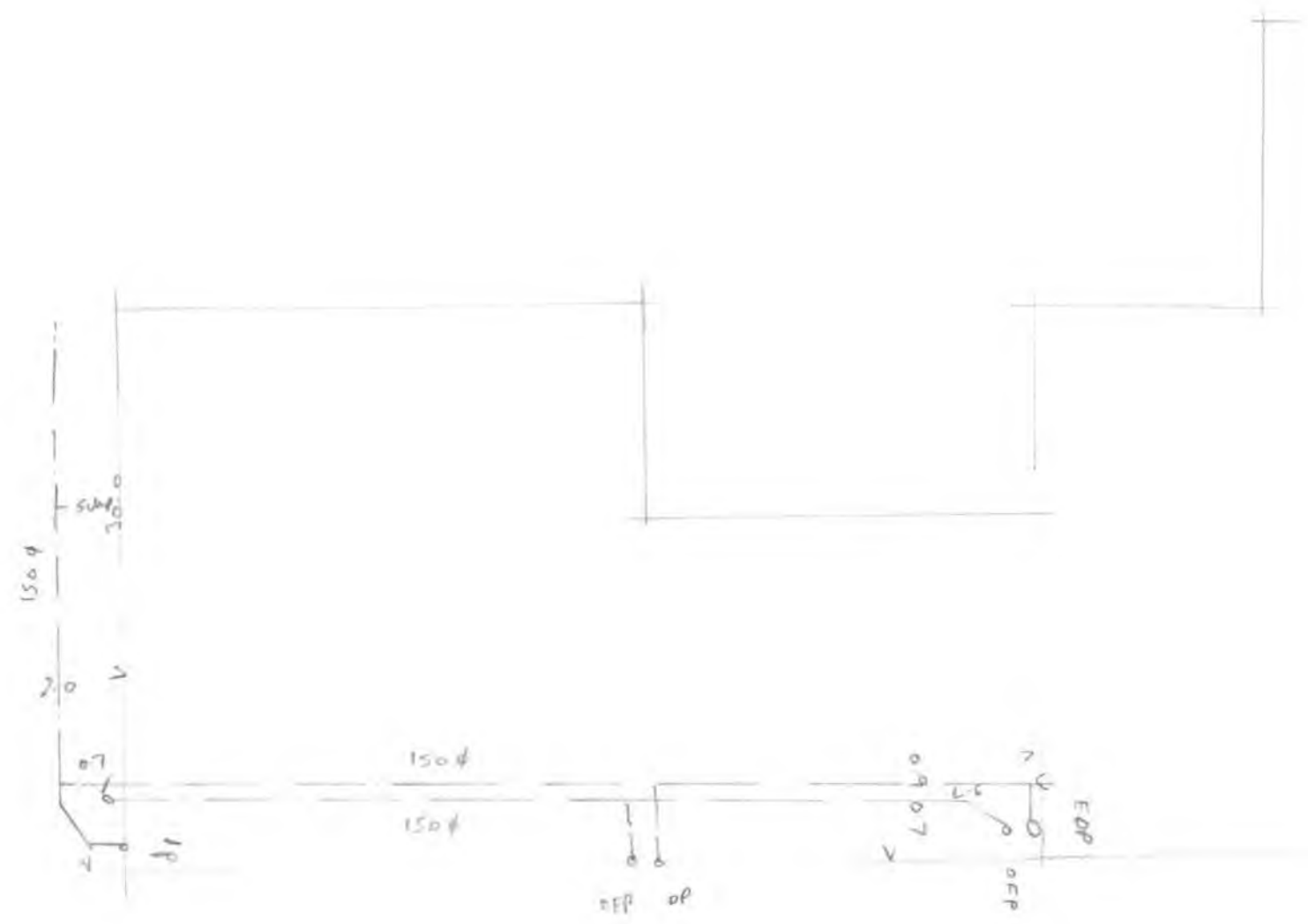
CONNECTION NUMBER

Address: 2 MARY MULLER DRIVE Owner: _____
Project No: 10080970 Drainlayer: D WINKIE
Date: 30-1-8 Inspector: RA




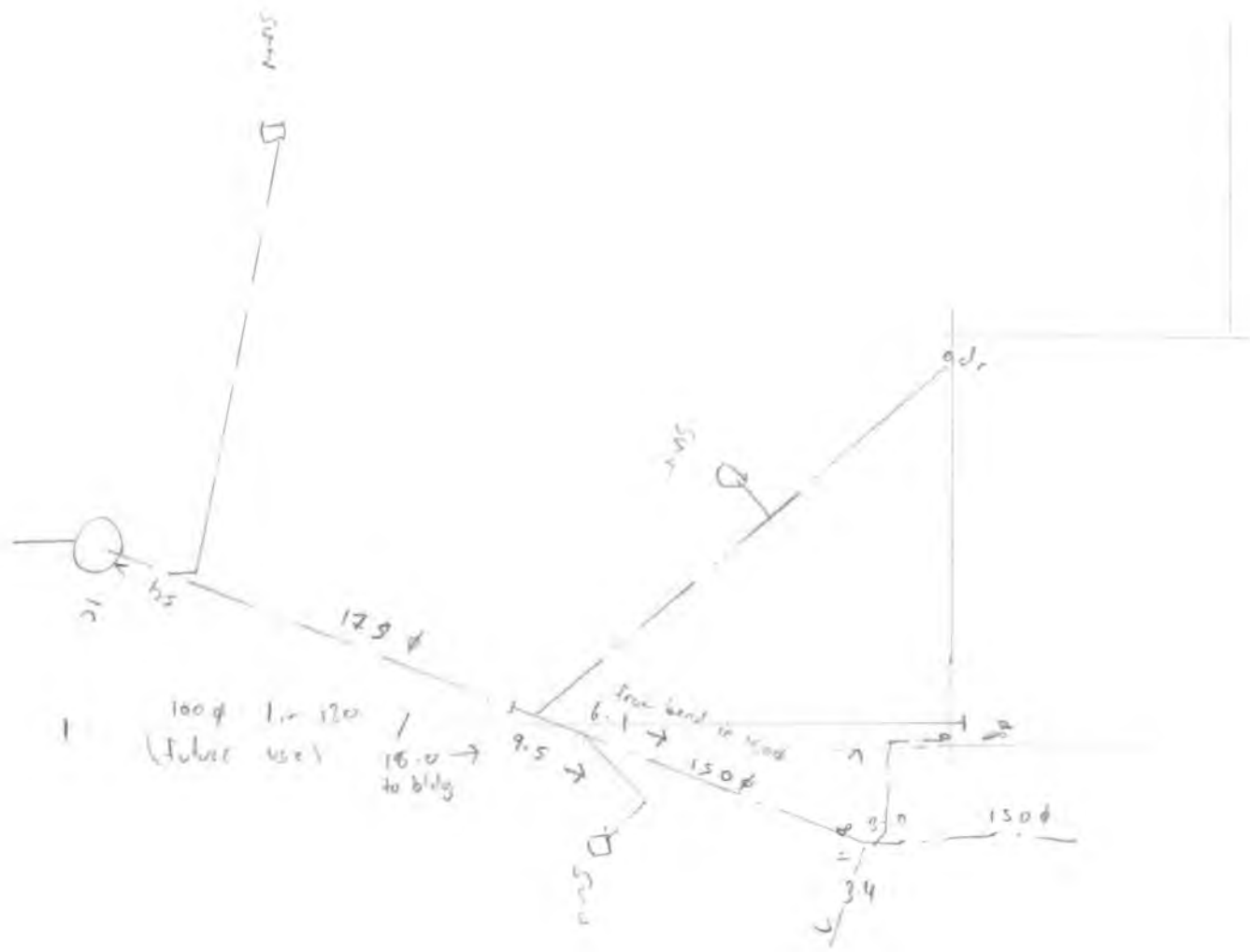
CHRISTCHURCH CITY COUNCIL - DRAINAGE PICK UP

 <p>CHRISTCHURCH CITY COUNCIL The City That Works</p>	ADDRESS: 2 Mary Miller Dr.	OWNER:	RECEIVED:	CONNECTION NUMBER
	LEGAL DESCRIPTION:	DRAINLAYER: Dean Wilkie	BLOCK PLAN:	
	PROJECT No.: 10056499	PLUMBER:	PLOTTED: / /	
	DATE: 6-10-5	FIELD OFFICER: K J Walsh	EYE BOOK:	



CHRISTCHURCH CITY COUNCIL - DRAINAGE PICK UP

 <p>CHRISTCHURCH THE GARDEN CITY <i>the only smart choice</i></p>	ADDRESS: 2 Mary Muller Dr.	OWNER:	RECEIVED:	CONNECTION NUMBER
	LEGAL DESCRIPTION:	DRAINLAYER: Dean W. Kie	BLOCK PLAN:	
	PROJECT No: 10056499	PLUMBER:	PLOTTED: / /	
DATE: 18.11.5	FIELD OFFICER: K J Walsh	EYE BOOK:		



**Christchurch City Council
Approved Consent Plan
ABA10117386
Michael Nilsson
418 Pages
28/08/2012**

GEOTECHNICAL REPORT

12 Mary Muller Drive

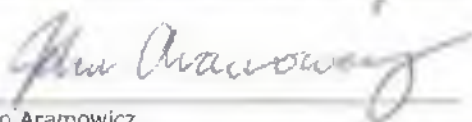

For Apollo Projects Ltd.

GEOTECHNICAL REPORT

12 Mary Muller Drive

QUALITY CONTROL CERTIFICATE

All relevant information is identified, has been reviewed, and is approved for release.

Prepared by:	 John Aramowicz Associate, Civil/Geotechnical Engineer	BE (Hons), MIPENZ, CPEng, IntPE(NZ)
Reviewed and approved for release by:	 Marton D Sinclair Director	BE, BSc, FNZIS, MIPENZ, CPEng, IntPE(NZ), RPS
Date: Reference: Status:	24 April 2012 348392_Geotech Report_smh.doc FINAL	
Distribution:	2 x copies 1 x file copy	for Apollo Projects Ltd. Eliot Sinclair

Limitations

This report has been prepared for Apollo Projects Ltd. according to their instructions, for the particular objectives described in the report. The information contained in the report should not be used by anyone else or for any other purposes.

Executive Summary

Apollo Projects Ltd. have engaged Eliot Sinclair to undertake a geotechnical investigation, liquefaction assessment and report for 12 Mary Muller Drive, Hillsborough, at the southwest part of Lot 2 DP 392999. We understand that a single-storey office, warehouse and factory building is proposed to be constructed at the south part of the site.

Ground damage and liquefaction evidence was visible to the south of the site after the 22 February 2011 earthquake (M6.3, peak ground acceleration approximately 0.65g in general area).

The geotechnical site investigation comprised 6 CPTu tests, with refusal at 18.7 to 27.7m depth below ground level. The soil types inferred from the CPT tests were mostly uniform across the site, comprising of uncontrolled fill materials to 1.5m top 2m depth, over layers of clayey silts and sandy silts, with a layer of 'sand & silty sand' at 5-7m and at 15-16m depth below ground level.

Computer analysis indicates the presence of thin liquefiable layers throughout the soils profile. Calculated total and differential vertical settlements are up to 120mm in a proposed SLS (0.13g) event, and up to 220mm in a ULS (0.35g) event.

Due to the presence of deep liquefiable soils underlying this site, piled foundations are unlikely to be economical for the proposed single storey building.

Based on the CPT test data, it would be possible to construct shallow foundations for small, lightweight structures (ie ≤ 2 stories) over a compacted gravel/subsoil raft of around 2m depth, and with stiffened raft-type foundations.

This foundation system would not prevent differential settlement entirely, but would assist in reducing the effects of deeper liquefaction, and limit the degree of differential settlement. Should differential settlement occur due to future earthquake shaking then it would be reasonably straightforward to relevel a stiffened building foundation and floor slab system by injection grouting between the ground and the stiffened shallow foundations.

Consideration will need to be given to the likelihood of encountering contaminated materials, and appropriate approval will need to be obtained from the Council, and precautions taken, before any test pits can be excavated, or ground improvement earthworks or foundation excavations commenced.

In order to confirm the nature and depth of the fill materials across the site, we recommend at least six test pits be undertaken across the site. Whilst the results of the test pits is not likely to change the recommendations of this report, the information will be useful in determining the quantity of uncontrolled fill material across the site, its composition, and any specific requirements for remediation.

Based on the results of the CPTu testing and analysis, we recommend the insitu uncontrolled fill materials are excavated, screened to remove unsuitable debris and organic material, and replaced with controlled compaction. Some additional make up clean gravel material may be

needed due to compaction losses, etc. This will form a raft upon which to build the foundation to the building.

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1. INTRODUCTION

Apollo Projects Ltd. have engaged Eliot Sinclair to undertake a geotechnical investigation, liquefaction assessment, and report for the Xtendlife site at 12 Mary Muller Drive, legally described as Lot 2 DP 392999.

The geotechnical report is required to characterise the general geotechnical conditions across the site, and to advise on likely foundations requirements for the proposed building.

2. BUILDING PROPOSAL

Concept drawings provided to us indicate a single-storey building with lightweight cladding to the west, north and east parts of the building, concrete tilt panel firewall along the southern lease boundary, and with a sloping steel roof.

A large part of the building will be used for commercial, retail and office space, with a small warehouse with 5.7m high stud at the northwest part of the building. There is provision for a future extension to the west of the building.

A sealed carpark area is proposed along the northern part of the site. Refer to Appendix K.

3. SCOPE OF WORK

The scope of work for the geotechnical assessment of the site was;

- Review published geology,
- Review Environment Canterbury's well card database for nearby bore log data,
- Review floor level data for adjacent building,
- Review of GNS Science's strong motion data for 4 September 2010 and 22 February 2011 earthquakes,
- Undertake 6 x Cone Penetration Tests with measurement of pore water pressures (CPTu),
- Liquefaction analysis,
- Reporting and Recommendations.

4. DISCLAIMER

Comments made in this geotechnical report are based on test records obtained from CPTu testing undertaken in March 2012, inspection of the general area, and liquefaction modelling using Geologimiski's CLiq software.

Whilst every care was taken during our investigation and interpretation of subsurface conditions, there may well be subsoil strata and features that were not detected. Additionally, on-going seismicity in the general area may lead to deterioration or additional ground settlement that could not have been anticipated at time of writing of this report.

The exposure of such conditions, or occurrence of additional strong seismicity, may require a review of our recommendations.

This report has been prepared for the benefit of Apollo Projects Ltd. at time of writing of this report, in accordance with the scope of work. No liability is accepted by this company or any employee of this company with respect to the use of this report by any other party for any other purpose.

5. ENGINEERING GEOLOGY

*Christchurch is situated on the middle part of the east coast of the South Island of New Zealand. The city is located on Holocene deposits at the Pegasus Bay coast of the Canterbury Plains, and on the northern slopes of the adjacent Port Hills of Banks Peninsula. Brown and Webber (1992) describe the Christchurch CBD area being underlain by 'a sequence of gravel strata interbedded with silt, clay, peat, shelly sand and clay down to 400m.'*¹

Brown and Webber (1992) note the general area of the site being underlain by: "Dominantly alluvial sand and silt overbank deposits." The general subsoil geology of Hillsborough will be similar to that of the Christchurch CBD with respect to soil types and presence of younger sediments in the upper layers overlying sandy gravels at some depth.

The key characteristics of the Christchurch CBD soils are summarised by Cubrinovski & McCahon² as;

- The top 20-25 m of the CBD soils are relatively recent alluvial soils overlying 300m to 500m thick gravelly deposits.
- The recent alluvial soils in the top 20 m of the deposits are the most important for foundations of multi-storey buildings and liquefaction evaluation. These soils comprise gravels, sands, silts, peat and their mixtures, and are highly variable both horizontally and vertically.
- The soils within the CBD are fully saturated below 1.0 m to 1.5 m depth
- Considering their composition (sandy soils and non-plastic silts), age (recent deposits, few hundreds to a few thousand years old) and depositional environment (river, swamp and marine sediments), these soils are generally considered susceptible to

¹ Brown, L.J.; Weeber, J.H. 1992 Geology of the Christchurch urban area. Scale 1:25 000. Institute of Geological & Nuclear Sciences geological map 1. 1 sheet + 104p. Institute of Geological and Nuclear Sciences Limited, Lower Hutt, New Zealand.

² Cubrinovski, M., McCahon, I. 2011 Foundations on Deep Alluvial Soils – Technical Report Prepared for the Canterbury Earthquake Royal Commission. University of Canterbury, Christchurch, New Zealand.

liquefaction, and in some cases (when deposited in a loose state) they have very low resistance to liquefaction.

- By and large, the foundation conditions within CBD are very complex and challenging for geotechnical engineers, particularly in regard to their performance during strong earthquakes.
- The presence of aquifers at depths of about 20 m to 25 m (and in some cases even at shallower depths) is a relatively unique feature that potentially may exacerbate the seismic response of the soils above the aquifers during strong earthquakes (by providing an additional mechanism for increase in the groundwater pressure through upward flow of water fed by the aquifers).
- The presence of aquifers at depths of about 20 m to 25 m (and in some cases even at shallower depths) is a relatively unique feature that potentially may exacerbate the seismic response of the soils above the aquifers during strong earthquakes (by providing an additional mechanism for increase in the groundwater pressure through upward flow of water fed by the aquifers).

1.1. EXISTING BORE HOLE DATA

Bore log records sourced from Environment Canterbury were reviewed to determine typical subsoil geology of the general area.

Well M36/1170 located approximately 185m southeast of the site indicates '*brown sandy silty clay*' from 0.4m to 2m depth below ground level, '*blue grey silty sand*' to 3.8m, '*silty clay*' to 6.4m and '*sandy silty clay*' to 18.2m depth below ground level where the log terminates.

Well M36/20624 located approximately 220m southwest of the site indicates '*fill*' to 0.9m below ground level, '*grey silt*' to 7.2m, and '*brown fine sandy silt*' to 12m depth below ground level where the log terminates.

Well M36/3883 located approximately 560m northwest of the site generally indicates organic silts to 4.5m, sandy silts to 7m, sandy clay to 9.5m and clay to 16.0m depth below ground level where the log terminates. Refer to Appendix A.

6. CANTERBURY EARTHQUAKE

The M7.1 Darfield earthquake on 04 September 2010 occurred on a previously unknown fault, producing peak horizontal ground accelerations (pga) of 0.24g at 'Christchurch Cathedral College' (CCCC). It is likely that this would have been similar to the horizontal ground accelerations experienced at 12 Mary Muller Drive, Hillsborough.

The earthquake resulted in liquefaction within the soft alluvial soils of eastern Christchurch.

A M6.3 aftershock on 22 February 2011 occurred on a previously unknown fault located under the Port Hills. Whilst the magnitude of the earthquake was lower than the M7.1 September 2010 event, the epicentre of the February 2011 event was located much closer to the urban areas of Christchurch, between Lyttelton and Heathcote Valley.

The M6.3 February 2011 event produced higher peak horizontal ground accelerations (pga) in Christchurch compared to the September event, with 0.49g recorded at Christchurch Cathedral College, and estimated at around 0.65g in the area of the site

A subsequent M6.4 aftershock on 13 June 2011 located around Taylors Mistake, caused further liquefaction across the CBD and eastern parts of Christchurch, although this was generally not as extensive in comparison to the February 2011 event.

1.2. DAMAGE MAPPING

Inspection of the ground surface across the Christchurch area was undertaken by the EQC in the days after the 4 September 2010, 22 February 2011 and 13 June 2011 earthquakes. Mapping clearly shows the general area around the site to have been effected by liquefaction, particularly in the February 2011 event.

7. SITE DESCRIPTION

1.3. GENERAL

The Xtendlife site at 12 Mary Muller Drive forms part of Lot 2 DP 306637, with other existing buildings across the lot. The lease site is to the west of Mary Muller Drive and has flat, grassed ground surface.

The Wyatt Print building is located immediately south of the site, and the Bidvest coolstore is located to the west. Refer to Diagram 1.



Diagram 1: Site location plan (approximate boundary in yellow), (source: Canterbury Recovery, Project Orbit, photo from 24 February 2011).



Diagram 2: Lot 2 DP306637, (shaded in black) Source: Quick Map, March 2012.

1.4. LAND CLASSIFICATION

On the 28th October 2011, the Canterbury Earthquake Recovery Authority (CERA) released site-specific classification for Canterbury properties. As part of this, the Department of Building and Housing (DBH) developed three new technical categories for residential foundation design as part of its guidance for repairing and rebuilding earthquake damaged homes in Canterbury. These new categories apply to liquefaction prone residential flat land in the green zone in the greater Christchurch urban area, but not to commercial land.

The site is noted as Green Zone, N/A Non Residential, however, residential properties nearby the site have been categorised as Technical Category 2 (TC2, yellow) which indicates, "*minor to moderate land damage from liquefaction is possible in future significant earthquakes*".

Refer to Appendix B.

8. SITE INVESTIGATION

1.5. CPTU TEST RESULTS

Six CPTu tests were undertaken across the site, being:

- CPTu01 is located near the north eastern corner of the site,
- CPTu02 is located near the south eastern corner of the site,
- CPTu03 is located near the south western corner of the site
- CPTu04 is located near the north western corner of the site,
- CPTu05 is located between CPTu01 and CPTu04
- CPTu06 is located between CPTu02 and CPTu03.

Refer to Appendix C for the CPTu test location plan.

The data from this testing indicates multiple thin layers of liquefiable soils throughout the entire soil profile.

The inferred soils from the CPTu analysis generally comprise multiple thin layers of silty clay and sandy silt with a layer of '*sandy & silty sand*' at 5-7m and at 15-16m depth below ground level.

The CPT's met practical refusal between 18.7m and 27.7m depth below ground level, presumably on dense sand or sandy gravels.

Refer to Appendix D.

1.6. GROUNDWATER

The depth to groundwater is noted as 1.0m below ground level on the Canterbury Recovery, Project Orbit database.

Well log M36/1170, approximately 185m southeast of the site, notes the minimum ground water level as 2.1m below ground level.

1.7. OBSERVED LAND DAMAGE

The areas surrounding Mary Muller Drive experienced land damage due to liquefaction in the February 2011 earthquake event, and this is visible in the aerial photograph taken on 24 February 2011 where there is evidence of ejected sediment and groundwater at the carpark to the south and southeast, and around the Bidvest building to the west.

Survey measurements taken across the concrete floor of the Wyatt Print building, south of the site, indicate minor differential settlement has occurred from the earthquake shaking.

We understand the northern foundations of the Bidvest building settled differentially by around 80mm in the February 2011 earthquake.

Refer to Appendix E.

9. LIQUEFACTION ASSESSMENT

1.8. SEISMIC HAZARD FOR CHRISTCHURCH

Subsequent to the M6.3 Christchurch earthquake, the hazard factor for Christchurch and Canterbury was revised on 19 May 2011 to a minimum of $Z=0.30$.

Table 3.2 of NZS1170.0:2002 notes single-family dwellings as Importance Level 2.

The serviceability limit state return period factor, R_s , for Christchurch was revised on 1 August 2011. NZS1170.0:2002 Table 3.3 sets out that the annual probability of exceedance for of an Importance Level 2 building is 1/25 for the serviceability limit state and 1/500 for the ultimate limit state.

$$R_s \text{ (SLS, 1/25)} = 0.33 \text{ (Ref: Amendment 11, 2.2.14c as amended by DBH, 1 August 2011)}$$

$$R_u \text{ (ULS, 1/ 500)} = 1.00$$

The site is underlain by deep alluvial soils, and are considered Type D in terms of NZS1170.5:2004. NZGS guidelines specifies;

$$C=1.12 \text{ for Type D soils.}$$

1.9. CURRENT PEAK HORIZONTAL GROUND ACCELERATION

The New Zealand Geotechnical Society's "*Geotechnical earthquake engineering practice – module 1 – Guideline for the identification, assessment and mitigation of liquefaction hazards*" sets out the method of determining the peak ground acceleration;

$$\text{Peak ground acceleration, } a_h = Z R C$$

Where Z = base pga called '*hazard factor*', given by NZS1170.5:2004 Table 3.3 and Figures 3.3 and 3.4

R = Return period factor, given by NZS1170.5:2004, Table 3.5

C = Site response factor called '*spectral shape factor*' in NZS1170.5:2004.

In summary

$$a_h \text{ (SLS)} = 0.30 \times 0.33 \times 1.12 = 0.11g$$

$$a_h \text{ (ULS)} = 0.30 \times 1.00 \times 1.12 = 0.34g$$

1.10. PROPOSED REVISION OF PEAK HORIZONTAL GROUND ACCELERATION

We understand that the design SLS and ULS pga's for liquefaction assessment are to be revised upwards slightly by GNS/DBH to 0.13g and 0.35g, respectively, and that these will apply equally to the Christchurch City, Waimakariri and Selwyn Districts, except where pga's calculated in accordance with NZS1170 exceed these values, then the values set out by NZS1170 should be adopted.

1.11. COMPARISON TO ACTUAL INTENSITIES

Strong motion records from GNS indicate peak horizontal accelerations were much higher in the 22 February 2011 earthquake than the 4 September 2010 earthquake. Refer to Table 1.

Results from the nearest monitoring station, *Christchurch Cathedral College (CCCC)*, indicate that peak horizontal ground accelerations were much higher than the proposed ULS pga in the 22 February 2011 event, and above proposed SLS pga in the 04 September 2010 earthquake.

Table 1: Comparison of peak horizontal ground accelerations close to site.

<i>PGA (horizontal)</i>	SLS (1/25, M7.5)	ULS (1/500, M7.5)	04 Sept 2010 ³ (M7.1)	22 Feb 2011 ⁴ (M6.3)
Current design pga (1 Aug 2011)	0.11g	0.34g		
Proposed design pga (12 Dec 2011)	0.13g	0.35g		
Christchurch Cathedral College (CCCC)			0.24g	0.49g
Estimate at Site			0.30g	0.65g

1.12. METHODOLOGY

The site investigation and analysis was generally undertaken in accordance with the New Zealand Geotechnical Society's '*Geotechnical Earthquake Engineering Practice; Module 1-Guideline for the identification, assessment and mitigation of liquefaction hazards*', published in 2010.

The CPTu data was analysed using the following procedures;

- Liquefaction assessment: NCEER (1998)⁵, and Robertson (2009)⁶

³ Darfield (Canterbury) earthquake strong motion data, GNS Science, 04 September 2010.

⁴ Christchurch earthquake strong motion data, GNS Science, 22 February 2011.

⁵ Analysis and Fines Correction Methods; Youd & Idris et al, 1998. *Liquefaction resistance of soils*:

For the purpose of this assessment, the depth to groundwater for liquefaction assessment is conservatively assumed 0.5m below existing ground level.

The nature of materials underlying the areas of refusal have not been specifically determined as part of this site investigation.

1.13. LIQUEFACTION POTENTIAL INDEX

The liquefaction potential index (LPI) in a proposed SLS (0.13g) event is estimated to be 3 to 6 at the full depth of testing, generally indicating a 'low' to 'high' risk of liquefaction damage to shallow building foundations.

Under the proposed ULS (0.35g) scenario, the LPI is estimated to range from 16 to 22 at the full depth of testing. These values indicate a 'very high' risk of liquefaction damage to building foundations at the various CPT test sites.

The liquefaction analysis shows a 'very high risk' of liquefaction damage to building foundations around 11m depth below ground level.

Refer to Appendix D.

1.14. VERTICAL SETTLEMENT DUE TO LIQUEFACTION

Estimates of vertical settlement by computer modelling are summarised in Table 2.

We note vertical settlements calculated by CLiq software use the method by Zhang (2002)⁷, for a range of parameters that are estimated from the four basic CPTu parameters of depth, cone tip resistance, skin friction, and pore water pressure, and therefore the settlements shown are not guaranteed or an exact figure.

The depth to refusal of the CPTu probe ranged across the site from 18.7m to 27.7m depth, presumably on dense sands or sandy gravels.

Summary report for the 1996 NCEER and 1998 NCEER/SF Workshop of Evaluation of Liquefaction Resistance of Soils. NCEER, ASCE Journal of Geotechnical & Geoenvironmental Engineering, Vol 127, pp 817-833

⁶ Robertson, P.K. 2009. Interpretation of cone penetration tests – a unified approach. Canadian Geotechnical Journal 2009, 46:1337-1355.

⁷ Zhang, G., Robertson, P.K, Brachman, R. 2002, *Estimating Liquefaction Induced Ground Settlements from the CPT*, Canadian Geotechnical Journal, 39: pp 1168-1180.

Table 2: Summary of predicted liquefaction induced settlements, at selected test locations.

Test Reference (depth to refusal)	CPTu01 (22.9m)	CPTu02 (27.7m)	CPTu03 (18.7m)	CPTu04 (18.9m)	CPTu05 (20.9m)	CPTu06 (27.6m)
Proposed SLS (M7.5, 0.13g)	120mm	120mm	150mm	75mm	80mm	75mm
Proposed ULS (M7.5, 0.35g)	210mm	210mm	220mm	130mm	140mm	150mm
22 Feb 2011 M6.3, 0.49g)	220mm	240mm	250mm	160mm	160mm	170mm

1.15. COMMENT ON EARTHQUAKE SETTLEMENT

1.15.1. SERVICEABILITY LIMIT STATE

Settlements predicted in a SLS (0.13g) event range from 75mm to 120mm, refer to Table 2.

NZGS's Geotechnical Earthquake Engineering Practice guidelines (Table 6.1) indicates that, based on the modelling results for an SLS event, a general performance level of L3 (high) for this site, ie: " $F_L < 1.0$; "Liquefaction occurs in significant portion of the deposit resulting in differential movements, large settlements (few hundreds of millimetres) and lateral displacements".

1.15.2. ULTIMATE LIMIT STATE

Settlements predicted in a ULS (0.35g) event range from 130mm to 220mm, refer to Table 2.

NZGS's Geotechnical Earthquake Engineering Practice guidelines (Table 6.1) indicates that, based on the modelling results for a ULS event, a general performance level of L3 (high) for this site, ie: " $F_L < 1.0$; "Liquefaction occurs in significant portion of the deposit resulting in differential movements, large settlements (few hundreds of millimetres) and lateral displacements".

1.15.3. ACTUAL DAMAGE

There was no visual evidence of liquefaction of ground settlement across the Xtendlife site.

Differential settlement of up to 50mm was recorded by a survey of floor levels of the adjacent commercial printing building south of 12 Mary Muller Drive. Also around 30mm of horizontal displacement was measured between survey marks, south of the site on the roadside of Mary Muller Drive was noted.

1.16. LATERAL SPREADING

The topography of the site is flat with no major watercourses within 100m of the site, and therefore lateral spreading analysis was not undertaken for this site.

10. FOUNDATION OPTIONS

1.17. GENERAL

Test pits have not been excavated across the site as the fill materials are understood to be contaminated.

Based on our observations during the excavation and formation of Mary Muller Drive, it is likely that there will be around 1.5m to 2m depth of uncontrolled fill material across the site. The CPTu test results indicate highly variable penetration resistances in the upper layers, and appear to confirm the presence of uncontrolled fill materials to these depths.

Based on previous excavations during road construction and photographic records on our file, the fill materials may contain various amounts of leather scraps, topsoil, timber, rubber tyres, bricks, boulders, gravels, gasworks waste, silt, clay pipes, concrete slab debris, and scrap steel. The large debris and scrap pipes tend to form underground voids.

Due to the nature of the uncontrolled fill materials there is a high risk of large differential settlement under static conditions due to consolidation of the uncompacted materials, decay of organic matter, and loss of fines into underground voids.

Road formation works for Mary Muller Drive were previously undertaken by excavation of the uncontrolled fill materials, disposal offsite of obvious organic or unsuitable materials, screening to remove large debris, etc and relaying the remaining screened soils in thin layers using controlled compaction and moisture control. These works appear to have been very successful as the surface of Mary Muller drive in the area of the site appears to be in good condition with no obvious evidence of earthquake related damage due to cracking, settlement, heaving, etc.

1.18. DEEP PILED FOUNDATIONS

The depth down to firm bearing, or non-liquefiable soils, is around 18-27m, and due to the presence of deep uncontrolled fill down to around 1.5m-2.0m depth across the site, and the presence of liquefiable insitu soils below this, floor slabs would also need to be supported on piles to avoid excessive differential settlement between the foundation and floor slab.

It is therefore unlikely that piled foundations will be an economic solution for the proposed single-storey building, and we have not considered this option any further.

1.19. SHALLOW STIFFENED RAFT FOUNDATION (≤ 2 STORIES)

Buildings of up to 2 stories typically have lower foundation loads and can be designed to tolerate a degree of differential settlement, and could therefore be constructed with a stiffened raft-type foundation over a compacted gravel/subsoil raft.

We note that use of raft foundations will not necessarily limit total or differential settlements to less than 50mm in a SLS event, or 100mm in a ULS event however, construction of stiffened raft-foundation over a compacted gravel/subsoil raft will assist in limiting the amount of differential settlement and assist in releveling of any future earthquake induced differential settlement.

Should differential settlement occur due to future earthquake shaking then it would be reasonably straightforward to relevel a building with stiffened raft/waffle slab foundations by injection grouting between the compacted gravel/subsoil raft and the concrete raft foundation.

1.19.1. FOUNDATION RAFT CRITERIA

Calculation of design bearing strength for shallow foundations assumes the following parameters;

- Depth to underside of raft foundation = 0m below existing GL.
- Uncontrolled fill materials excavated out and disposed off site.
- Compacted gravel/subsoil raft; min 2m thickness, compacted in 200mm layers to extend at least 1.5m laterally beyond any foundation.
- Triax TX160 geogrid at regular intervals.
- Stiffened concrete raft foundations (specific engineering design) constructed over compacted subbase,
- Suitable for lightweight buildings of two storeys (ie ground + 1st storey).
- Refer to Appendix H, I & J.

1.19.2. BEARING CAPACITY OF SHALLOW FOUNDATIONS

Assessment of design bearing strength was undertaken using Hansen's equation for static loading conditions, and assume that foundations are located over a min 2m depth compacted subbase. Refer to Appendix H.

1.19.3. SETTLEMENT CONTROL OF SHALLOW FOUNDATIONS

Primary and secondary settlements were estimated using the Schmertmann Method outlined in Bowles international edition (1997)⁸, and assume that foundations are located over a 2m deep compacted sgravel/subsoil raft. Design bearing strengths were calculated as the maximum strength resulting in 25mm total settlement over a 50-year design life. Refer to Appendix I.

⁸ Bowles, J. 1996 "Foundation analysis and design, International Edition 1997" McGraw Hill

The bearing capacity for foundations reduces with increasing width due to the increasing depth of influence of the foundation pressure, and the presence of weaker subsoil layers identified by the subsoil testing. Foundation widths outside of the range shown on the summary charts should be reviewed by the geotechnical engineer in order to confirm the design bearing capacity.

It is noted that foundation settlement shown on the design chart are for static conditions only, and any ground settlements that may arise from seismic shaking would be additional to the design static settlement of 25mm over 50 years.

1.19.4. COMMENT

The design bearing strength (typically referred to as 'allowable') shown for both bearing capacity and foundation settlement takes into account the strength reduction factor of 0.45, being within the range of accepted values set out in Table 1 of NZBC Verification Method BM1/VM4.

The structural engineer will need to refer to both charts in order to identify the lesser of the two Design Bearing Strengths for a particular foundation width (B), and we note that settlement control of foundations is typically the limiting consideration for large foundations with high static loads.

Refer to charts in Appendix H and I.

We note that from a geotechnical engineering perspective, it would be preferable to use lightweight construction for the whole of the building, in order to avoid large differences in foundation pressures.

1.19.5. LIMITATIONS

The strengths shown on the charts, and specified elsewhere in this report, are based on the assumed excavation of uncontrolled fill materials and replacement with controlled compaction of uniform clean fill materials, and with a stiffened raft foundation over (refer to Appendix J) with static, vertical loadings.

Should eccentric foundation loads be proposed then the geotechnical engineer should review the proposed foundation design and loadings to confirm the foundation design is suitable for the conditions encountered.

1.19.6. EXTENT OF COMPACTED GRAVEL/SUBSOIL RAFT

We note the south wall of the building is shown with a concrete tilt panel fire wall along the southern part of the lease site. It would be preferable that the compacted gravel/subsoil raft extends at least 1.5m beyond any foundations to ensure adequate load spread and avoid failure of the compacted gravel raft.

Consideration should be made to either relocating the south wall of the building to the north, or alternatively, extending the compacted gravel/subsoil raft further south to provide adequate setback.

Similar consideration should be made for the future extension, however, it is unlikely that the gravel raft could be extended beyond the western boundary. Also, it would be preferable to construct the gravel raft for both the current and future building footprint in a single operation to ensure consistency and to minimise the risk of differential settlement.

1.19.7. REMEDIATION OF UNCONTROLLED FILL

Serious consideration will need to be given to the presence of uncontrolled and possibly contaminated fill materials down to around 1.5-2.0m depth across the site, as the cost to excavate and dispose these materials offsite, and to import and place clean compacted gravels, will be considerable.

An alternative approach would be to excavate the uncontrolled fill materials, dispose of unsuitable materials, screen to remove debris and oversize materials, and relay in layers using controlled compaction to create a relatively uniform, dense fill material. This approach appears to have been very successful along Mary Muller Drive and could provide an alternative to disposing all existing fill materials offsite.

1.19.8. CARPARK AREAS

We note the large carpark area will need to be constructed so that differential settlement is kept to a minimum, and to provide adequate bearing strength for delivery trucks, etc.

Due to the likely presence of highly variable fill with areas of unsuitable organic matter and voids, we consider the most practical option would be to remediate the insitu fill in the same manner as proposed for the compacted gravel/subsoil raft, before construction of the carpark formation.

1.20. OTHER GROUND IMPROVEMENT OPTIONS

There are a number of ground remediation technologies available that are capable of mitigating the risk of consolidation of the uncontrolled fill and liquefaction by either chemical stabilisation, or densification of the loose soils.

It may be possible that one or more of the various ground improvement methods could be used at this site to increase bearing capacity and decrease risk of liquefaction, although alternative ground improvement methods are not yet common in Christchurch and are therefore likely to be relatively costly. Some methods considered are;

1.20.1. DYNAMIC COMPACTION

Insitu dynamic compaction of the existing fill materials is likely to induce ground vibrations in the general area, and given the presence of the printing operation to the south, may be impractical at this stage.

Further, dynamic compaction of the uncontrolled fill materials will not address the presence of large debris, decomposable organic material, or large concentrations of topsoil within the fill that may consolidate over time and result in differential settlement.

In summary, the use of dynamic compaction is not a suitable remediation technique for this site.

1.20.2. CEMENT STABILISATION

Recent trials by the Department of Building & Housing on a test area established at QEII Park was undertaken to investigate ground improvement options for residential foundations, and was not specifically intended for commercial use or over uncontrolled fill. It is likely the conclusions of this investigation may also be applicable to small lightweight commercial types of buildings, but not large multi storey buildings with high foundation loads.

Whilst the results of the DBH investigation have not been officially released, a progress briefing by the Department of Building & Housing to Christchurch geotechnical engineers on 8 February 2012 indicated that paddle-mixed cement stabilisation down to around 2m depth provides cost-effective ground improvement that, in conjunction with a stiffened raft foundation over, may achieve acceptable performance in sands and silty sands.

We note that remediation by cement stabilisation would be problematic at this site due to the likely presence of highly variable fill materials, large debris, and the presence of timber, topsoil, organic matter, etc, and is therefore not a suitable technique for the site.

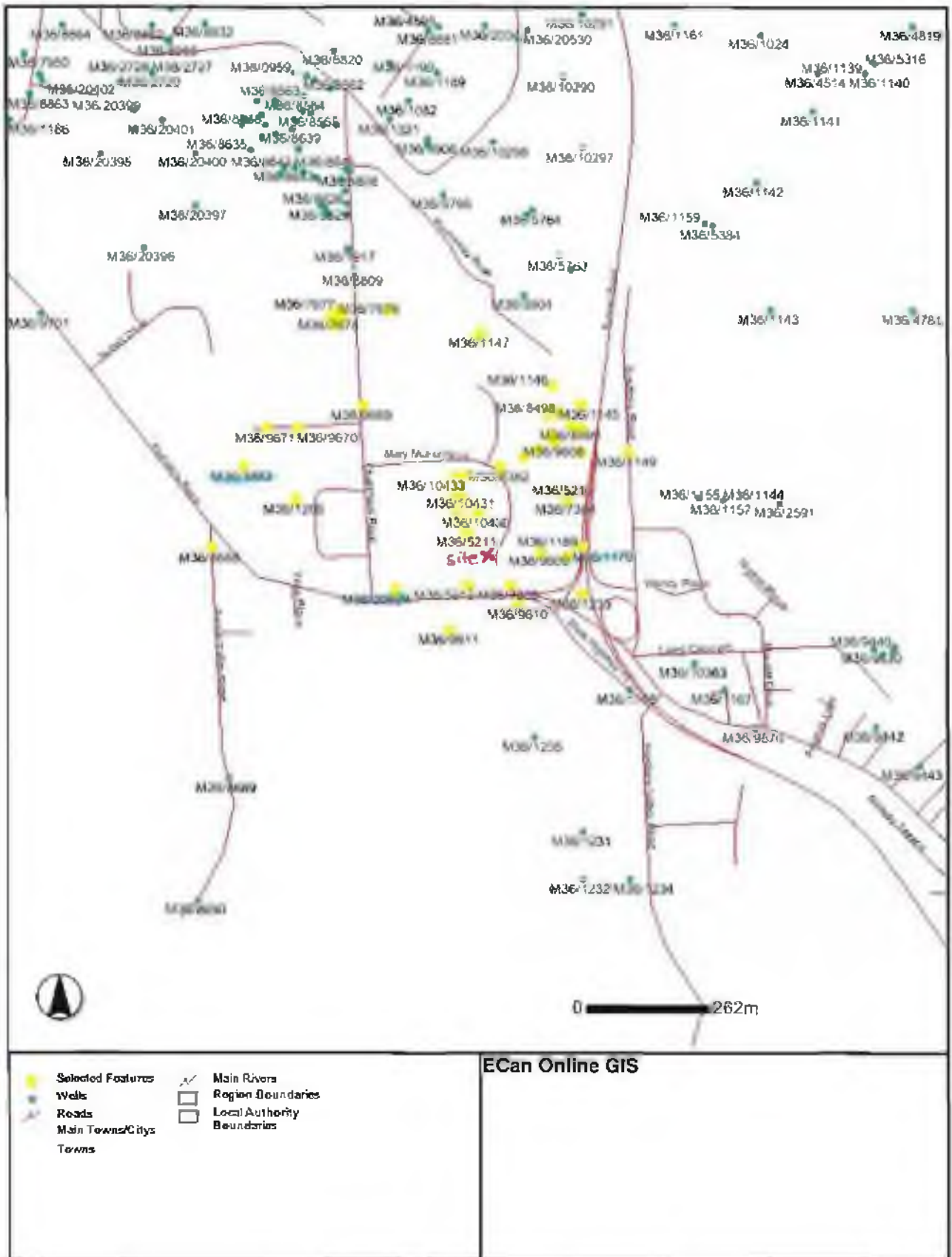
11. RECOMMENDATIONS

Consideration will need to be given to the likelihood of encountering contaminated materials, and appropriate approval will need to be obtained from the Council, and precautions taken, before any test pits can be excavated, or ground improvement earthworks or foundation excavations commenced.

In order to confirm the nature and depth of the fill materials across the site, we recommend at least six test pits be undertaken across the site. Whilst the result of the test pits is not likely to change the recommendations of this report, the information will be useful in determining the quantity of uncontrolled fill material across the site, its composition, and any specific requirements for remediation.

Based on the results of the CPTu testing and analysis, we recommend the insitu uncontrolled fill materials are excavated, screened to remove unsuitable debris and organic material, and replaced with controlled compaction. Some additional make up clean fill material may be needed due to compaction losses, etc. The building should be supported over a stiffened raft-type foundation and floor slab subject to specific engineering design.

12. APPENDIX A – ENVIRONMENT CANTERBURY WELL LOGS



Bore or Well No: M36/20624

Well Name:

Owner: BIDVEST NZ LIMITED



Street of Well: PORT HILLS ROAD

File No: CO6C/28258

Locality: CHRISTCHURCH

Allocation Zone: Christchurch/West Melton

NZGM Grid Reference: M36:8481-3741 QAR 3

NZGM X-Y: 2484810 - 5737410

Location Description:

Uses: Foundation/Investigation Bore

ECan Monitoring:

Well Status: Active (exist, present)

Drill Date: 18 Apr 2011

Water Level Count: 0

Well Depth: 12.00m -GL

Strata Layers: 4

Initial Water Depth: 0.33m -MP

Aquifer Tests: 0

Diameter: 100mm

Isotope Data: 0

Yield/Drawdown Tests: 0

Measuring Point Ait:

Highest GW Level:

GL Around Well: 0.00m -MP

Lowest GW Level:

MP Description: TOC

First Reading:

Last Reading:

Driller: McMillan Water Wells Ltd

Calc. Min. GWL:

Drilling Method: Rotary Rig

Last Updated: 08 Dec 2011

Casing Material: PVC

Last Field Check:

Pump Type:

Screens:

Yield:

Screen Type: DRILLED PVC

Drawdown:

Top GL: 2.90m

Specific Capacity:

Bottom GL: 5.90m

Aquifer Type:

Aquifer Name:

Date

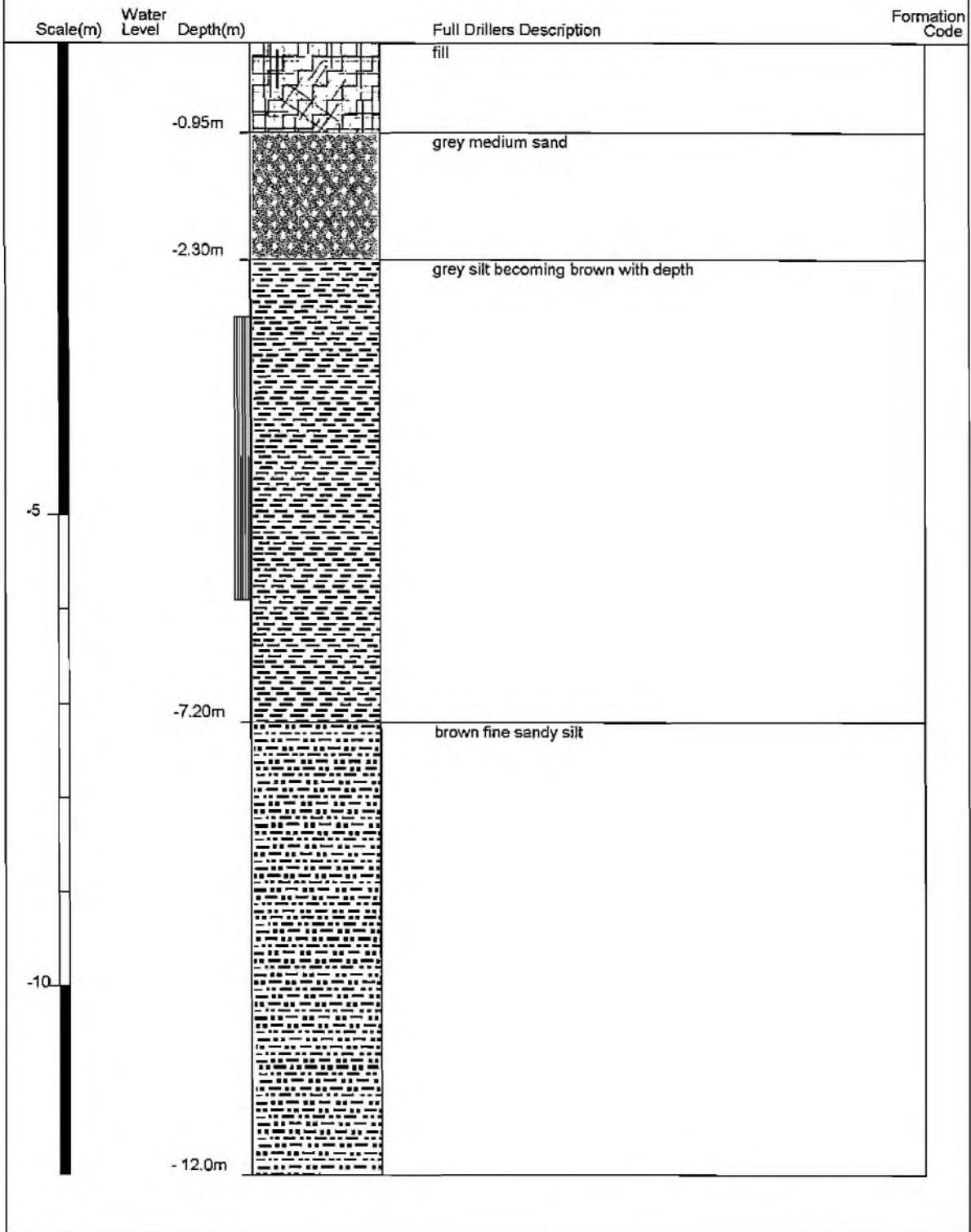
Comments

20 May 2011

Gridref changed from: M36:8486-3749 to M36:8481-3741 location error Plan confirms

Borelog for well M36/20624

Gridref: M36:8481-3741 Accuracy : 3 (1=high, 5=low)
 Driller : McMillan Water Wells Ltd
 Drill Method : Rotary Rig
 Drill Depth : -12m Drill Date : 18/04/2011



Bore or Well No: M36/1170

Well Name:

Owner: M.O.W.



Street of Well: TUNNEL ROAD

Locality: HEATHCOTE VALLEY

NZGM Grid Reference: M36:852-375 QAR 4

NZGM X-Y: 2485200 - 5737500

Location Description:

ECan Monitoring:

Well Status: Casing Retrieved /
Abandoned

File No:

Allocation Zone: Christchurch/West Melton

Uses: Foundation/Investigation Bore

Drill Date: 01 May 1961

Well Depth: 18.20m -GL

Initial Water Depth: -0.76m -MP

Diameter:

Water Level Count: 0

Strata Layers: 5

Aquifer Tests: 0

Isotope Data: 0

Yield/Drawdown Tests: 0

Measuring Point Ait: 4.80m MSD QAR 3

GL Around Well: 0.00m -MP

MP Description:

Driller: Job Osborne (& Co/Ltd)

Drilling Method: Unknown

Casing Material:

Pump Type: Unknown

Yield:

Drawdown:

Specific Capacity:

Aquifer Type: Unknown

Aquifer Name:

Highest GW Level:

Lowest GW Level:

First Reading:

Last Reading:

Calc. Min. GWL: -2.10m -MP

Last Updated: 18 Oct 2006

Last Field Check:

Screens:

Screen Type:

Top GL:

Bottom GL:

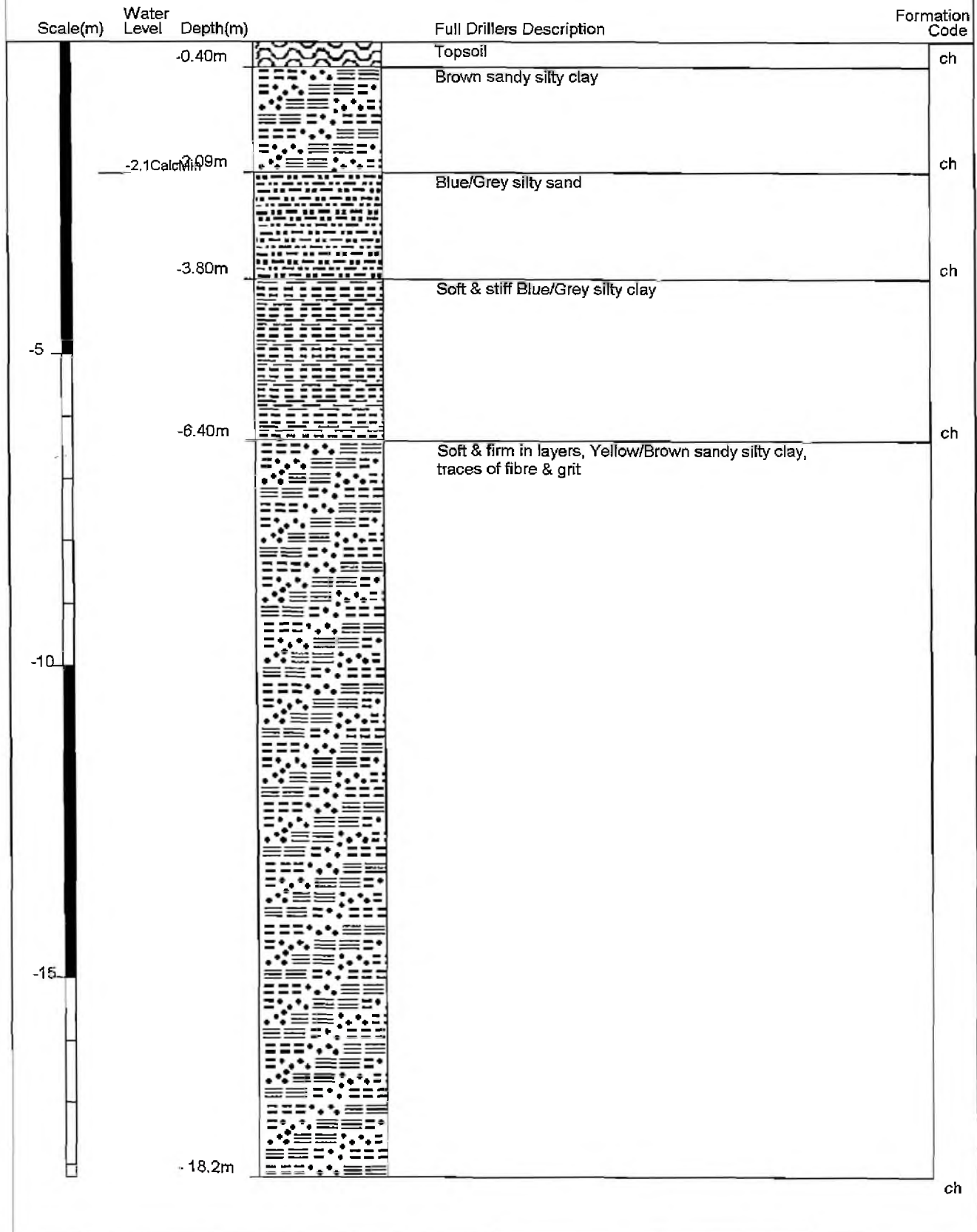
Date

Comments

BOREHOLE NO 2.

Borelog for well M36/1170

Gridref: M36:852-375 Accuracy : 4 (1=best, 4=worst)
 Ground Level Altitude : 4.8 +MSD
 Driller : Job Osborne (& Co/Ltd)
 Drill Method : Unknown
 Drill Depth : -18.2m Drill Date : 1/05/1961



Bore or Well No: M36/3883

Well Name:

Owner: OASIS INDUSTRIES LTD



Street of Well: BRIGHTLINGS RD

Locality: HILLSBOROUGH

NZGM Grid Reference: M36:8449-3767 QAR 4

NZGM X-Y: 2484490 - 5737670

Location Description:

ECan Monitoring:

Well Status: Casing Retrieved /
Abandoned

File No:

Allocation Zone: Christchurch/West Melton

Uses: Foundation/Investigation Bore

Drill Date: 12 Mar 1987

Well Depth: 16.00m -GL

Initial Water Depth:

Diameter: 150mm

Water Level Count: 0

Strata Layers: 13

Aquifer Tests: 0

Isotope Data: 0

Yield/Drawdown Tests: 0

Measuring Point Ait: 2.60m MSD QAR 3

GL Around Well: 0.00m -MP

MP Description:

Driller: Canterbury Drilling Company

Drilling Method: Cable Tool

Casing Material: STEEL

Pump Type: Unknown

Yield: 0 l/s

Drawdown: 0 m

Specific Capacity:

Aquifer Type: Unknown

Aquifer Name:

Highest GW Level:

Lowest GW Level:

First Reading:

Last Reading:

Calc. Min. GWL: -2.70m -MP

Last Updated: 21 Sep 2006

Last Field Check:

Screens:

Screen Type:

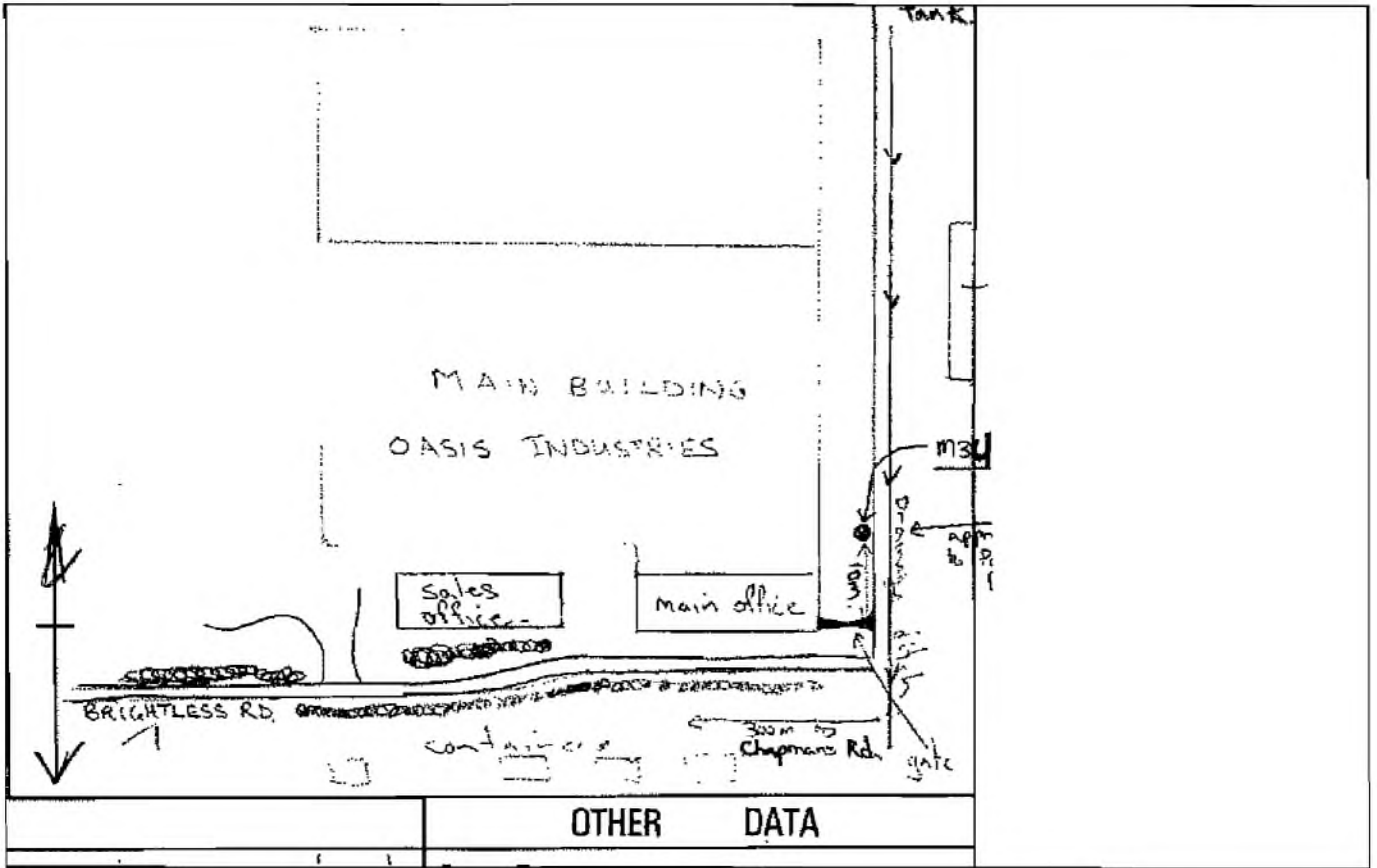
Top GL:

Bottom GL:

Date

Comments

CASING RETREIVED



Borelog for well M36/3883

Gridref: M36:8449-3767 Accuracy : 4 (1=best, 4=worst)
 Ground Level Altitude : 2.6 +MSD
 Driller : Canterbury Drilling Company
 Drill Method : Cable Tool
 Drill Depth : -16m Drill Date : 12/03/1987



Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
		-1.20m	Unknown	ch
		-3.00m	Soft organic silts	ch
	-2.7CalcMin	-3.00m	Soft organic silts with some small peat layers	ch
		-4.00m	Soft silts with some timber	ch
		-4.50m	Shells & silts 50/50	ch
-5		-5.50m	Sandy silts some shells	ch
		-6.00m	Sand silts some shells	ch
		-6.50m	Sandy silts (Some sand lenses present in the silts up to 200mm thick, lenses are clean. Coarse Grey sand)	ch
		-7.00m	Sandy clay (Br)	ch
		-9.50m	Clay some sand becoming dry	ch
-10		-11.5m	Clay rare small stones dry	ch
		-13.0m	Clay	ch
-15		-15.5m	Clay dry crumbles when broken. Possibly rotten volcanics rock turning Re/Br	co?
		-16.0m		co

13. APPENDIX B – CERA CLASSIFICATION

CERA
Canterbury Earthquake Recovery Authority

Home My Property Recovery Strategy Search

12 Mary Muller Drive, Hillsborough 8022

G Green Zone, N/A - Urban Nonresidential

Land Status: **Green** | **N/A** | **Urban Nonresidential**

Land classified as green means that homes are suitable for repair and rebuild.

Key points to note

- Land generally suitable for houses to be repaired or rebuilt;
- Property owners should talk directly with their insurer or EQC about repairs;
- Property owners no longer have to wait for the results of any area-wide land assessment reports by EQC or their engineering consultants Tankin & Taylor;
- There will be some isolated exceptions where geotechnical assessments will be required due to major land damage;
- Repair and rebuilding work should take into consideration the risk of ongoing aftershocks, so some finishing tasks such as brick and driveway concrete laying should be delayed until that risk decreases.

What does Technical Category "not applicable" mean?

Some properties in the green zone have experienced liquefaction-related land damage and considerable settlement during the sequence of Canterbury earthquakes. While land in the green zone is still generally considered suitable for residential construction, houses in some areas will need more robust foundations or site foundation design where foundation repairs or rebuilding are required.

Technical Category not applicable means that non-residential properties in urban areas, properties in rural areas or beyond the extent of land damage mapping, and properties in the Port Hills and Banks Peninsula have not been given a Technical Category.

Normal consenting procedures will apply in these areas.

What happens next?

- You should make contact with your insurer or EQC to progress repairs.
- [Download the Green Zone factsheet](#) (PDF 720KB) for more information on properties classified as Green.

The above zoning information is correct to the best of our knowledge at the time of publishing.

CERA
Canterbury Earthquake Recovery Authority
The Canterbury Earthquake Recovery Authority (CERA)

My Property
Residential red zone
Support and Assistance
Land announcements
Land damage information
[Green zone/technical category](#)

Library
Maps
Legislation
Cabinet Papers
Science and Data
Plan Building

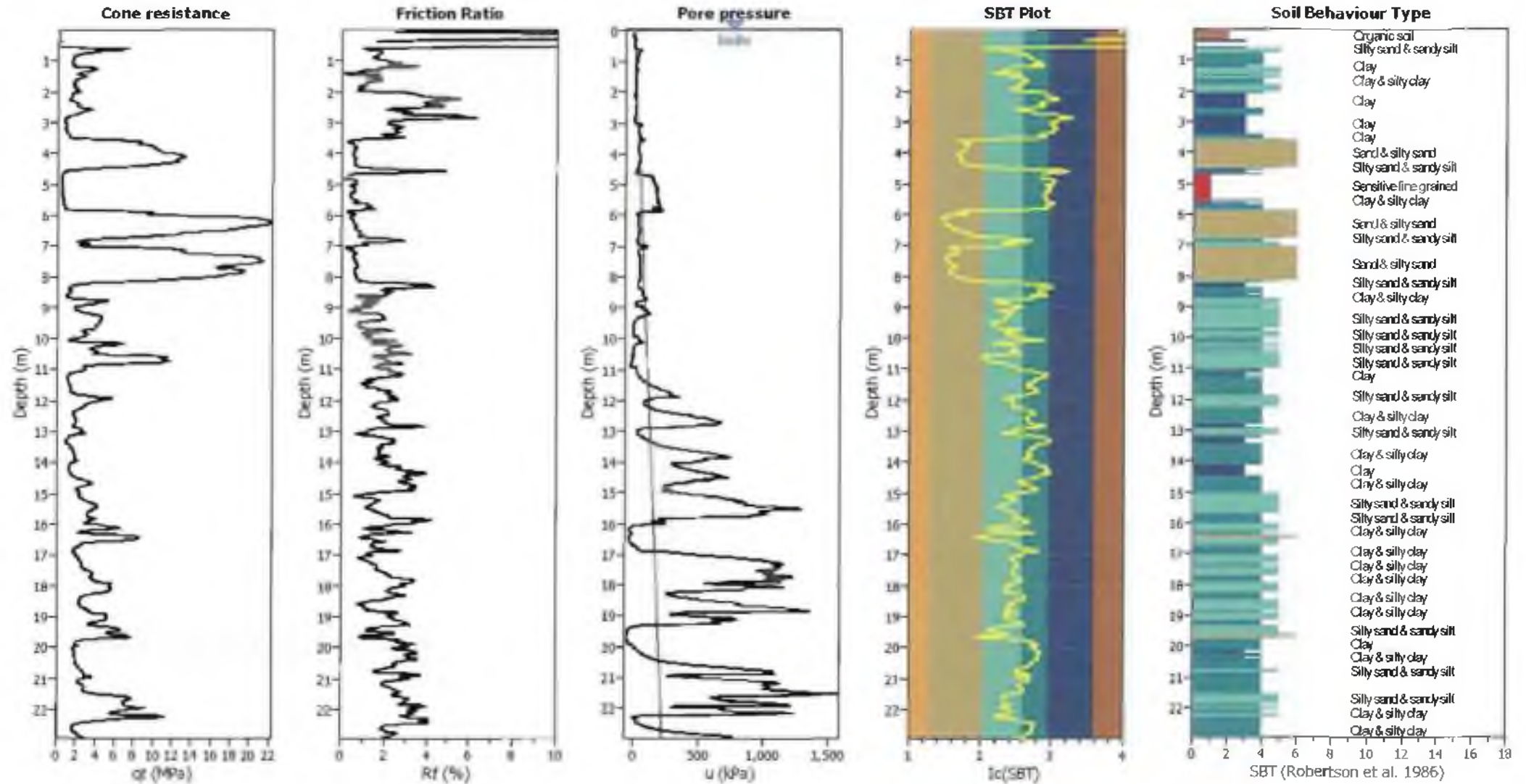
Questions and Answers
[Red zone purchase offer](#)
[Your Property](#)
[Zonings](#)
[Structural Assessment](#)
[Health and Safety](#)

14. APPENDIX C – CPTU TEST LOCATION PLAN



15. APPENDIX D – CPTU TEST RESULTS & ANALYSIS

CPT basic interpretation plots



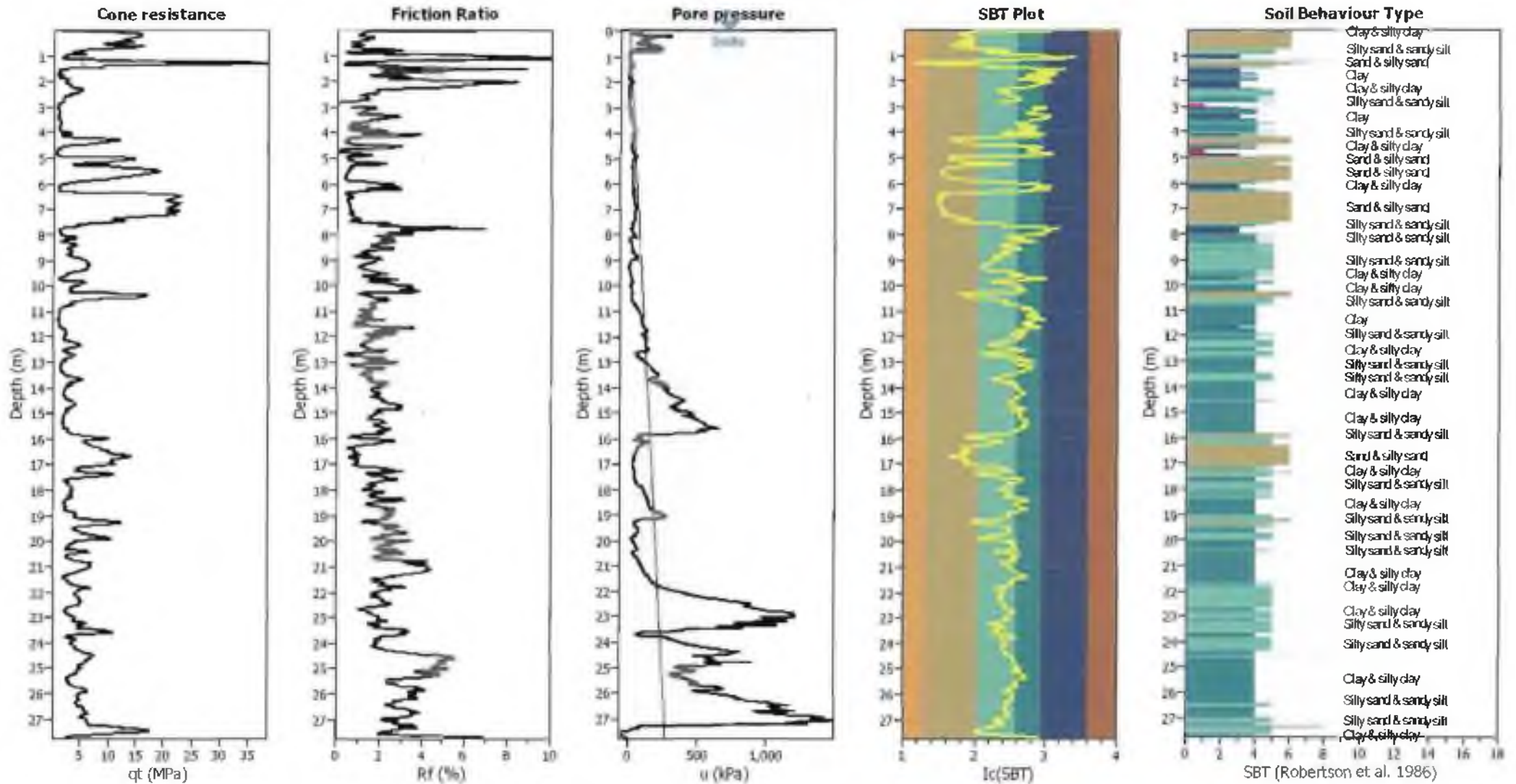
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.15	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	0.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots



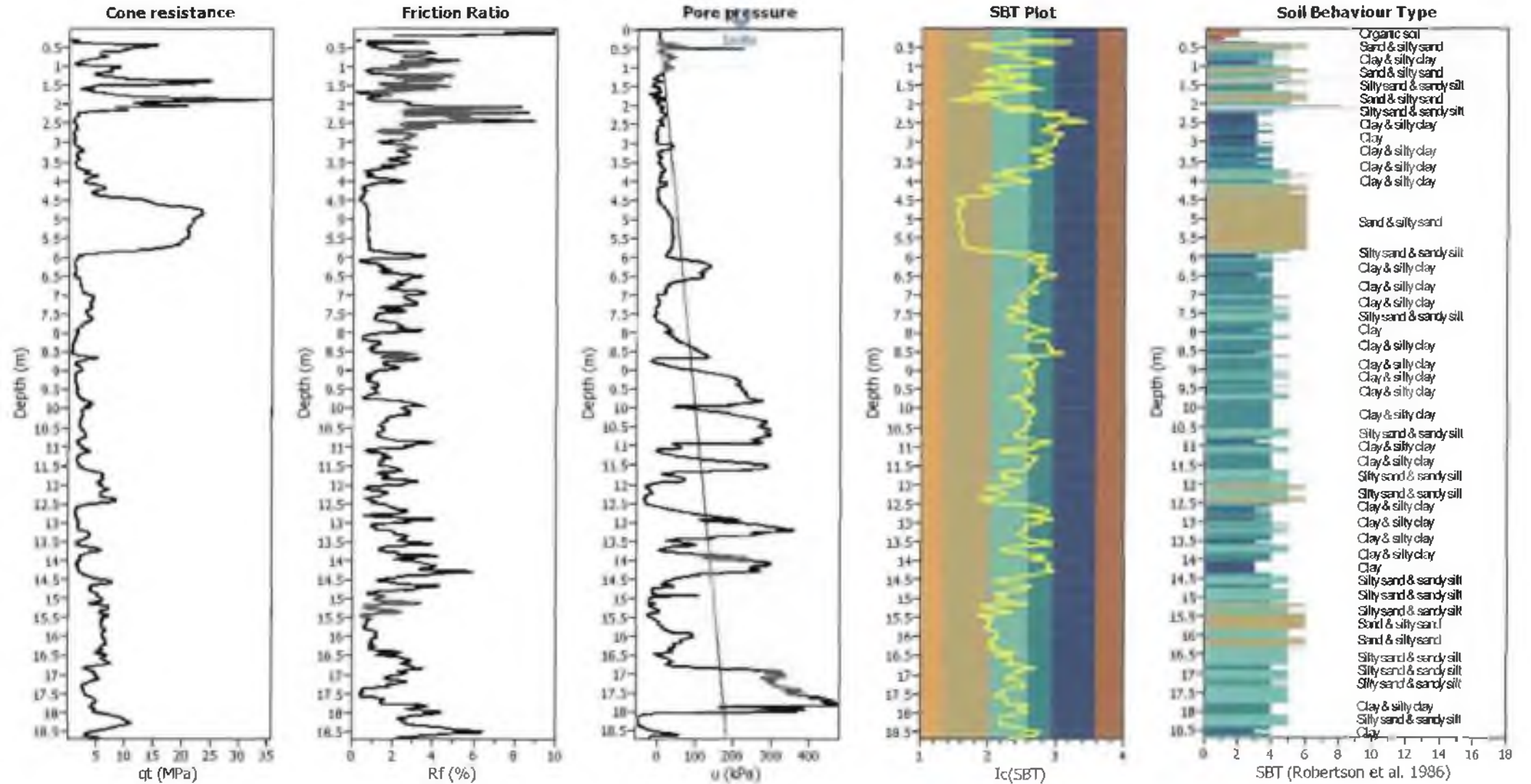
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.15	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	0.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots



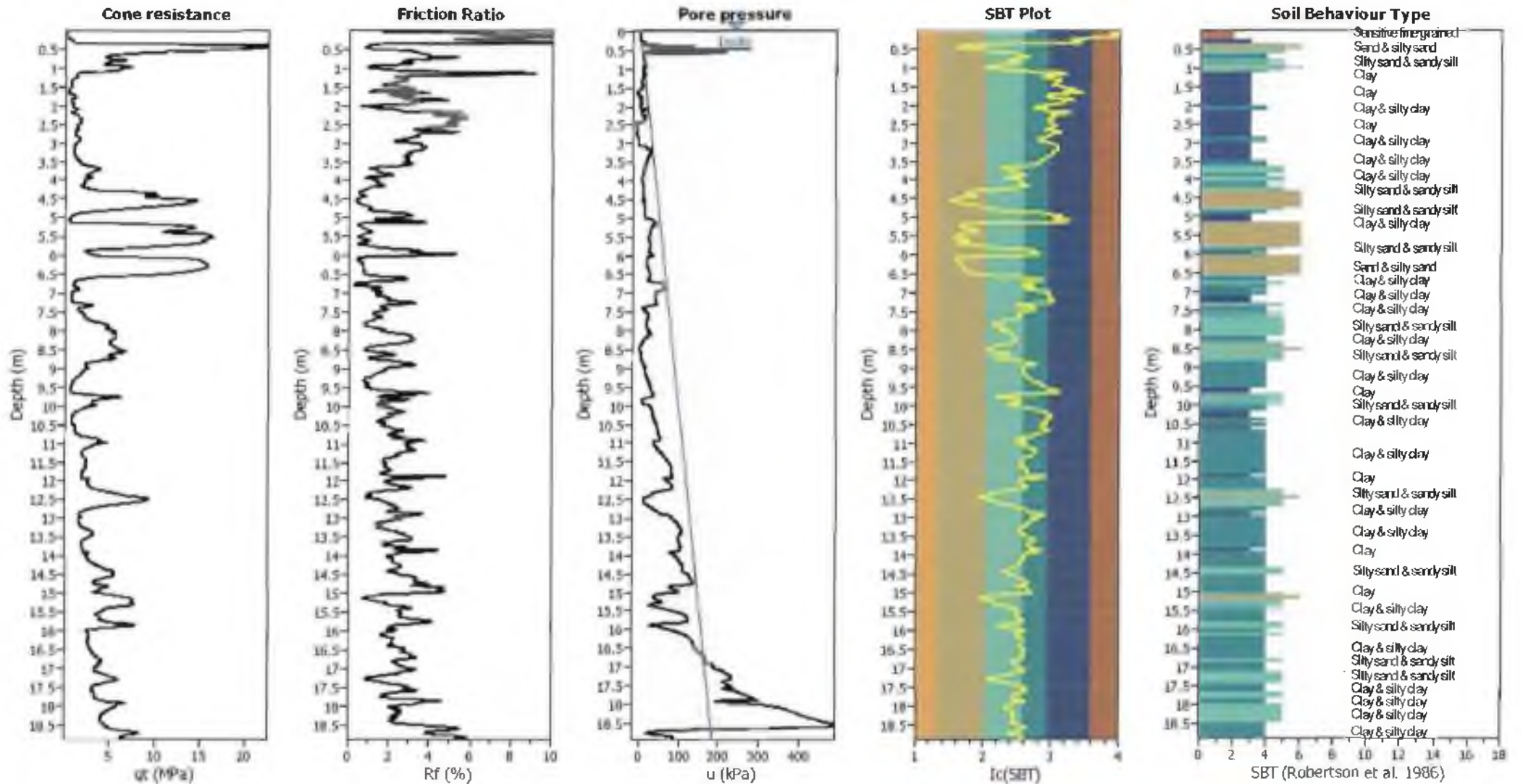
Input parameters and analysis data

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Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.15	Use fill:	No	Limit depth applied:	No
Depth to water table (Insitu):	0.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots



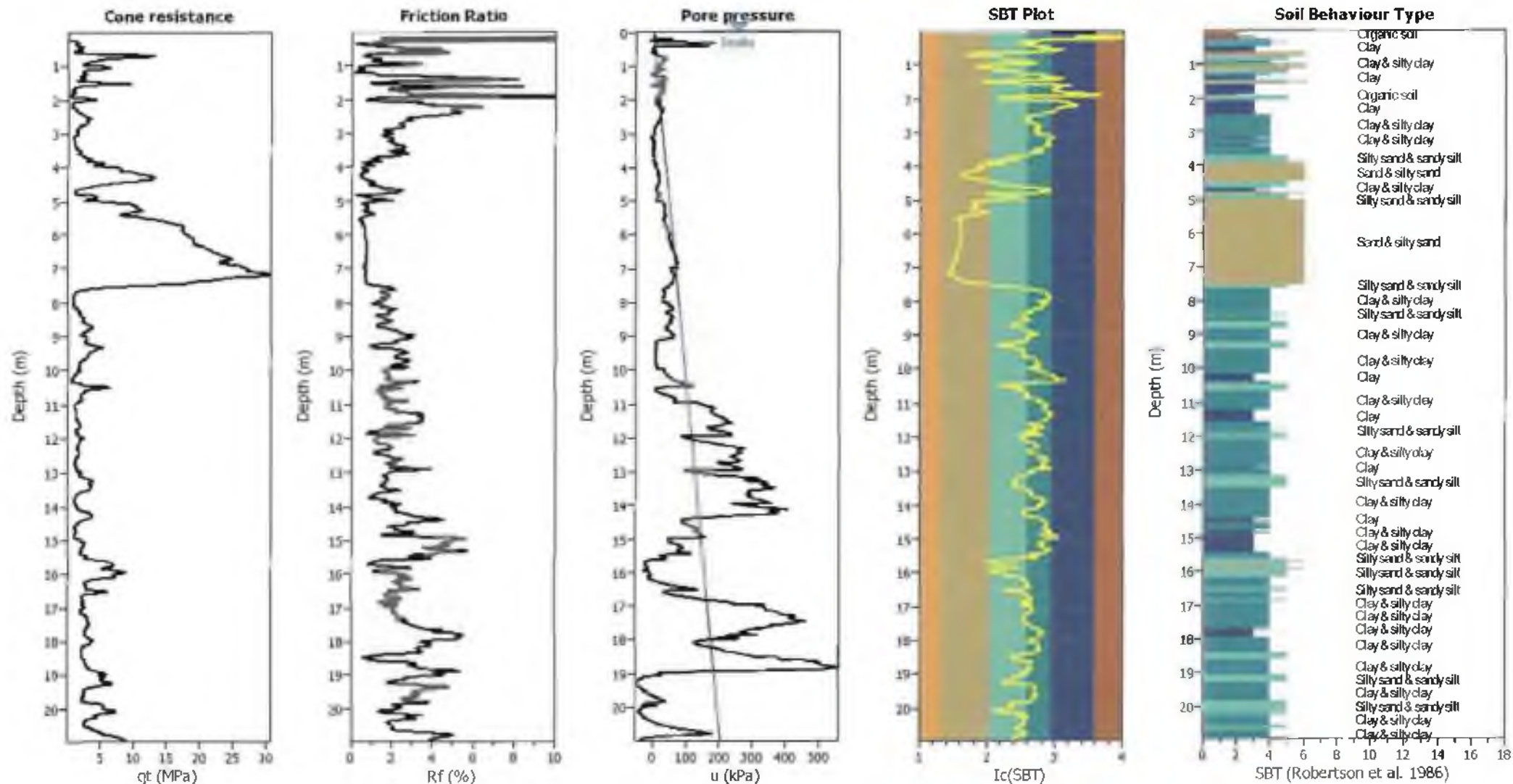
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on I_c value	I_c cut-off value:	2.60	K_0 applied:	Yes
Earthquake magnitude M_w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.15	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	0.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots



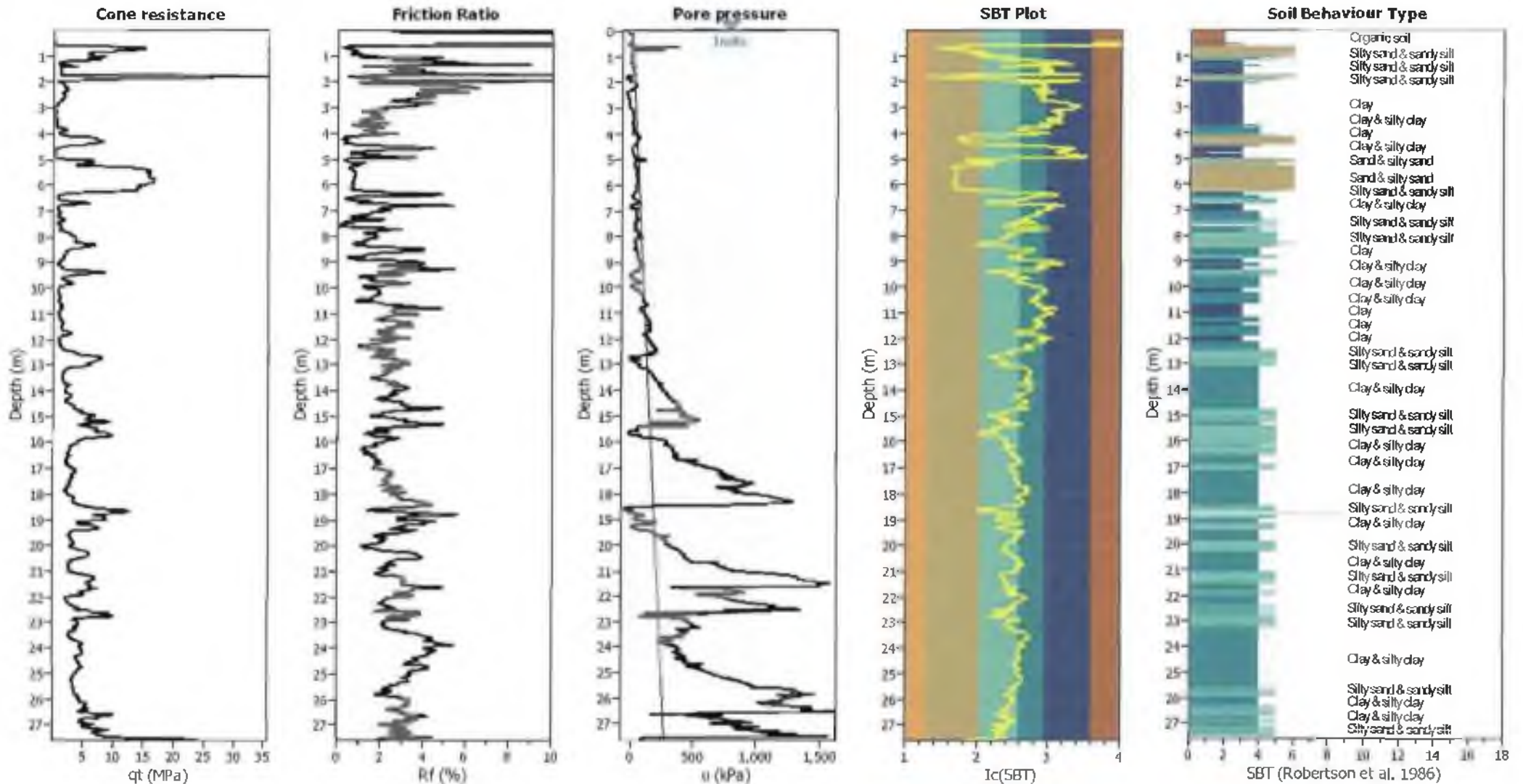
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _a applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.15	Use fill:	No	Limit depth applied:	No
Depth to water table (Insitu):	0.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots



Input parameters and analysis data

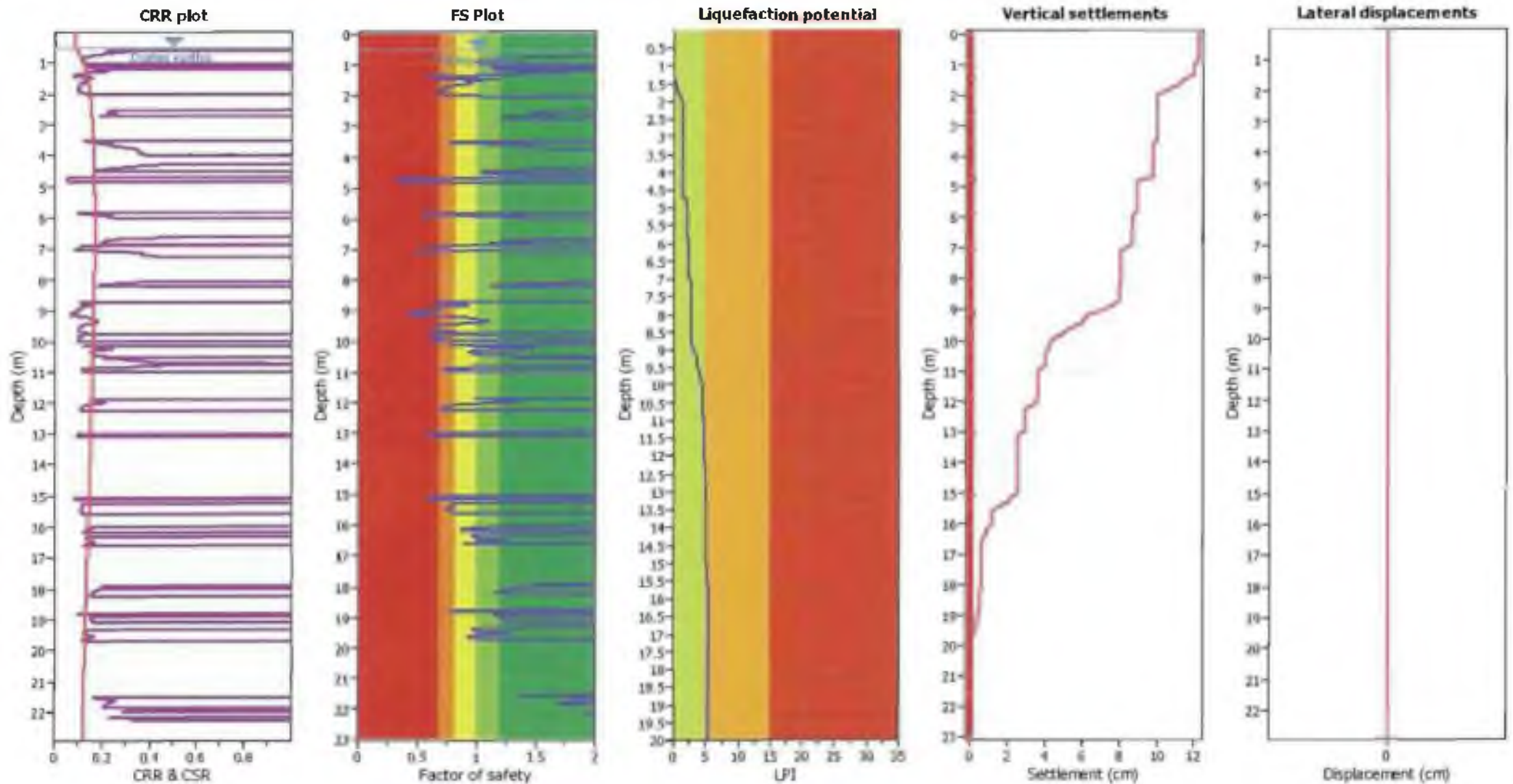
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Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.15	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	0.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Serviceability Limit State (SLS): M7.5, 0.13g peak horizontal ground acceleration

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _w applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

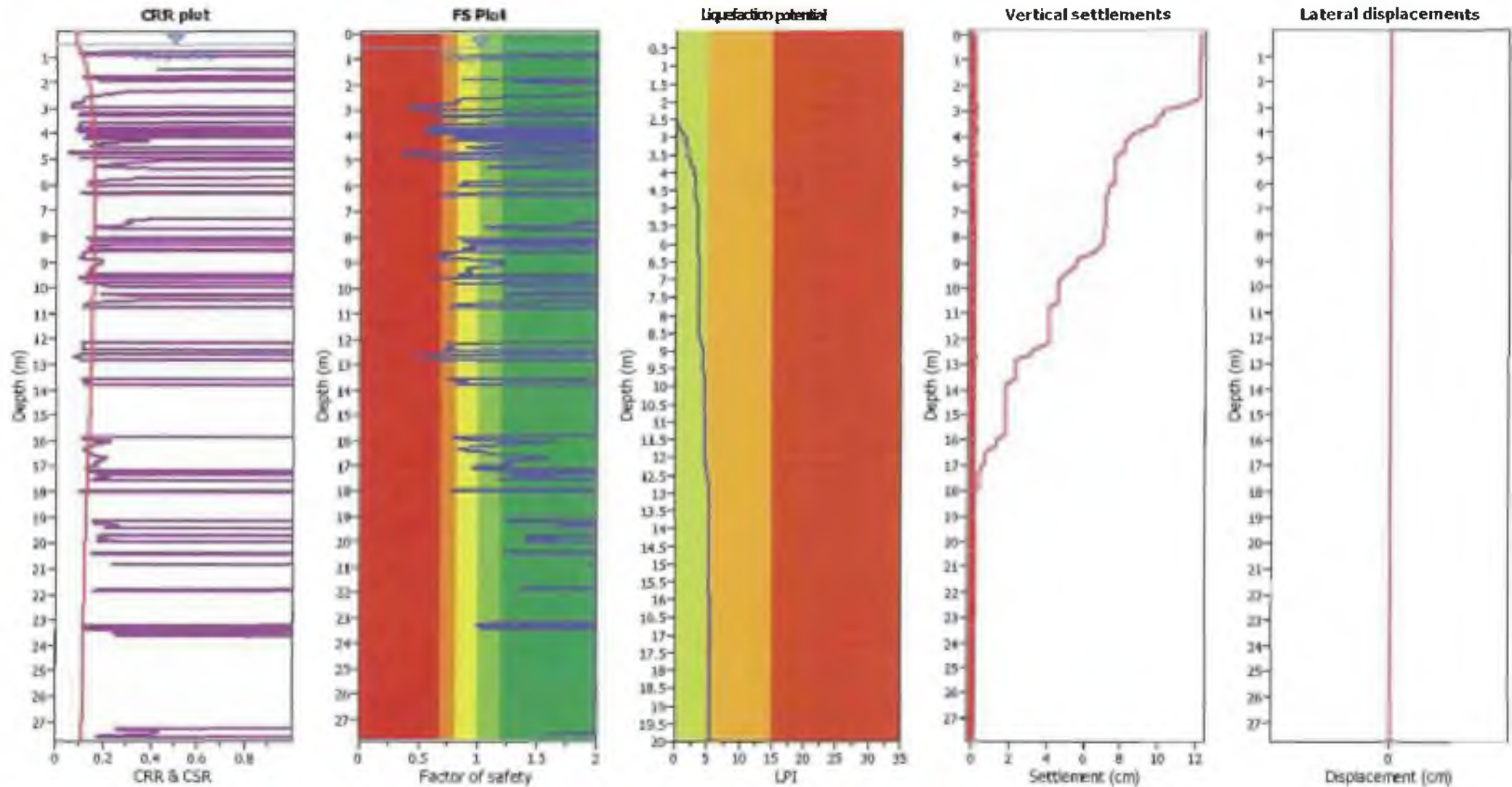
F.S. color scheme

- Almost certain it will liquify
- Very likely to liquify
- Liquefaction and no liquefaction are equally likely
- Unlike to liquify
- Almost certain it will not liquify

LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	0.50 m	F ₀ weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on I _c value	I _c cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (Insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

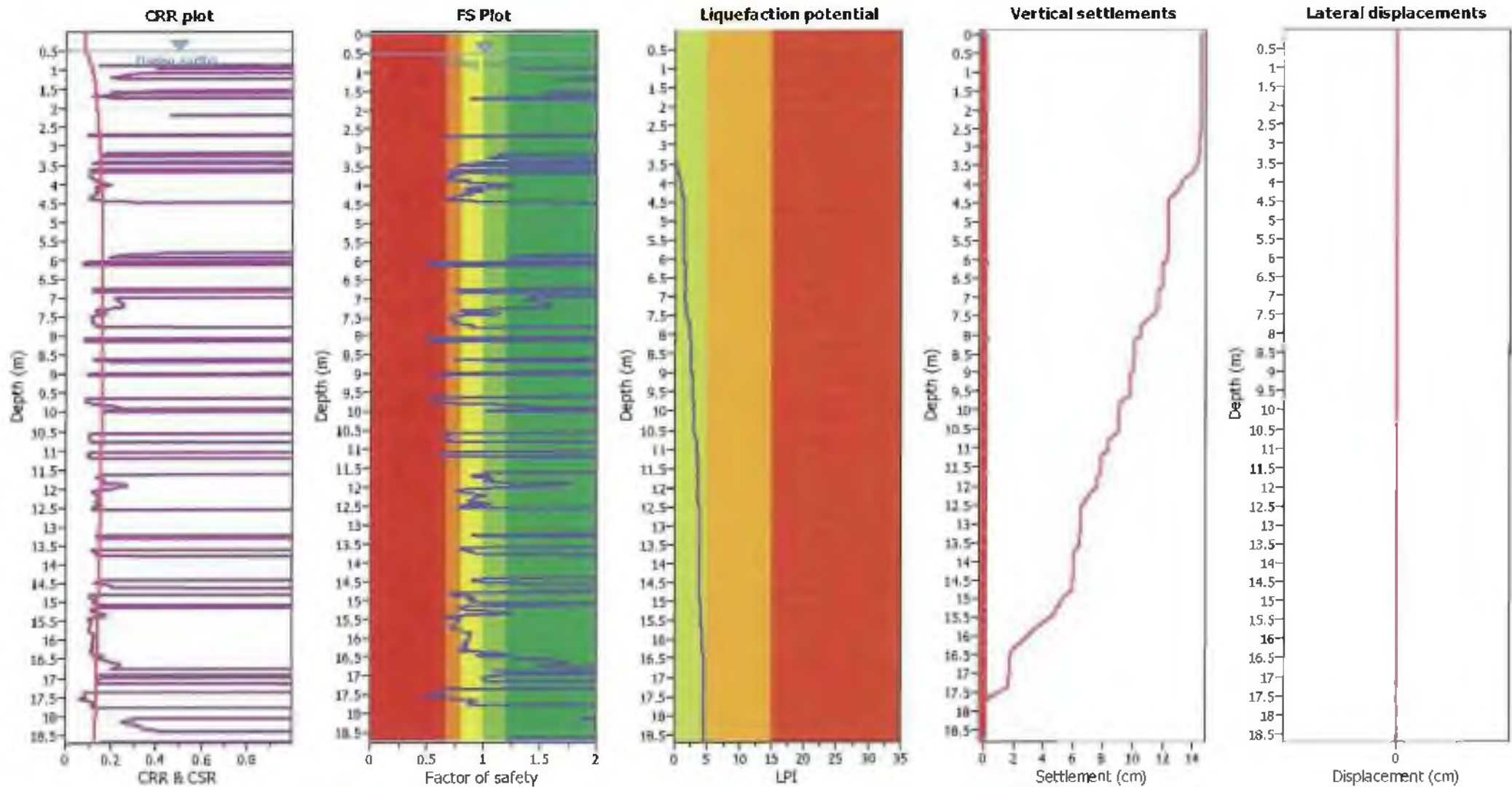
F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _σ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (Insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

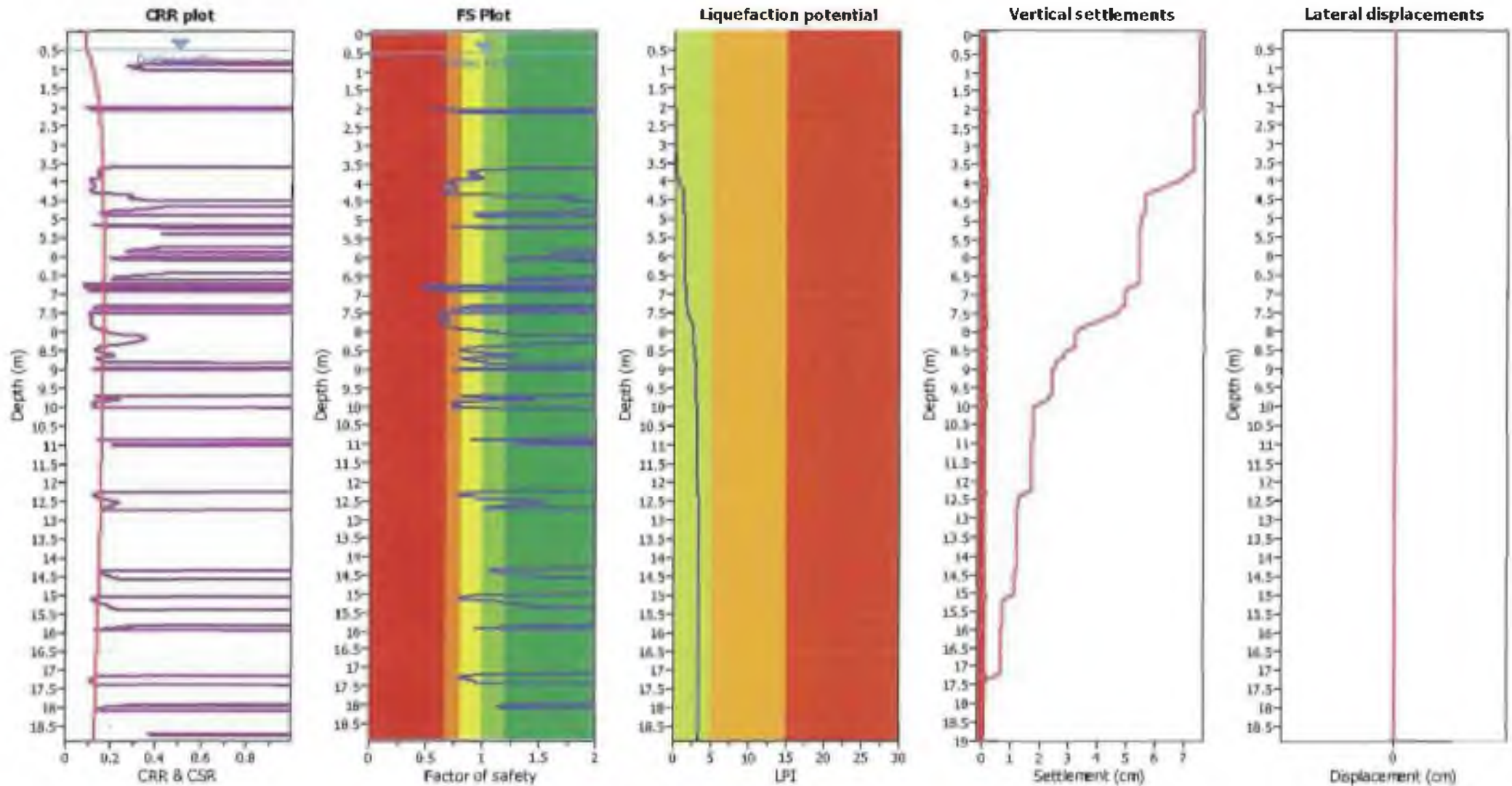
F.S. color scheme

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LPI color scheme

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- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.50	K_v applied:	Yes
Earthquake magnitude M_w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

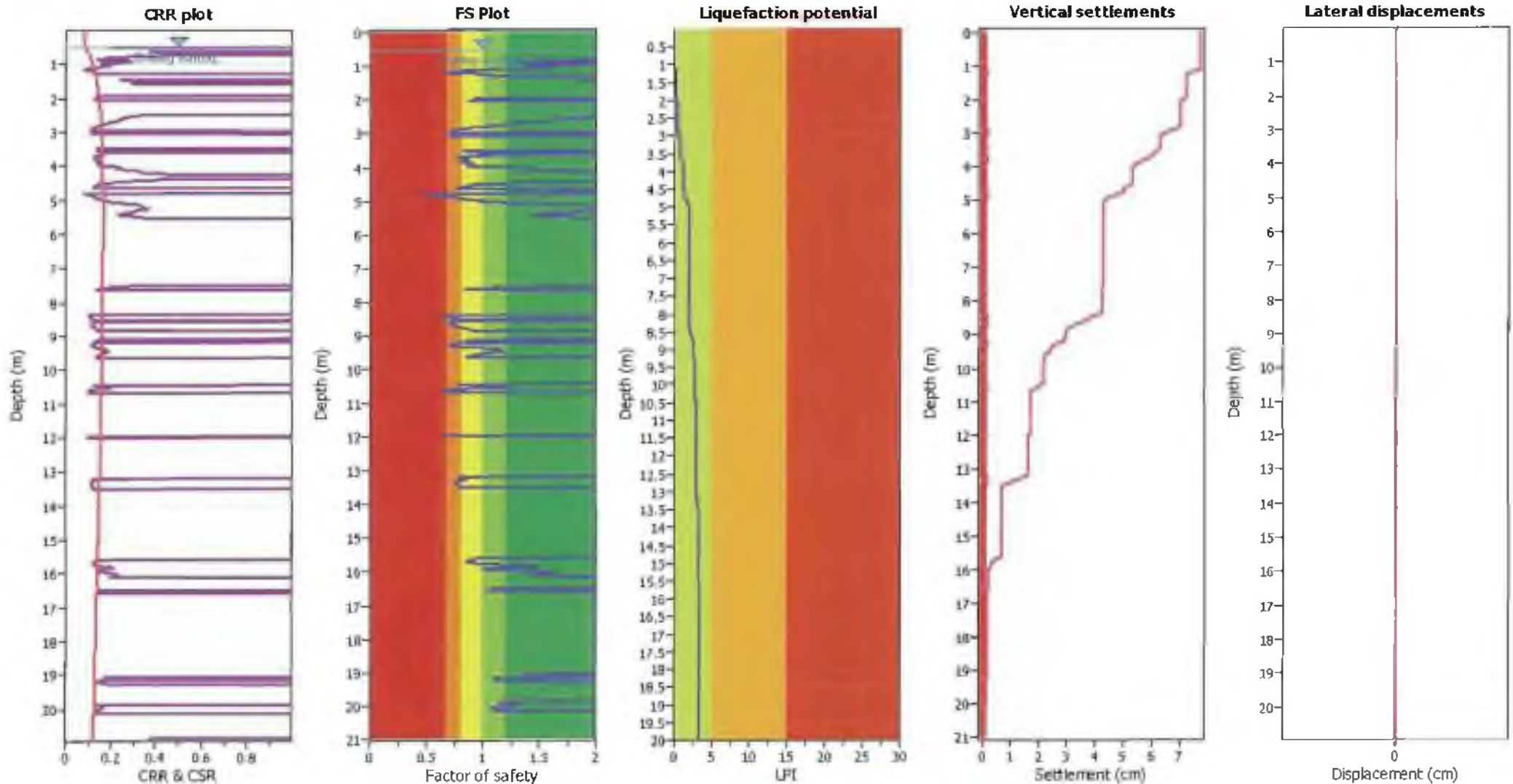
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LPI color scheme

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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on I _c value	I _c cut-off value:	2.60	K _σ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Unit depth applied:	No
Depth to water table (Inst):	2.50 m	Fill height:	N/A	Unit depth:	N/A

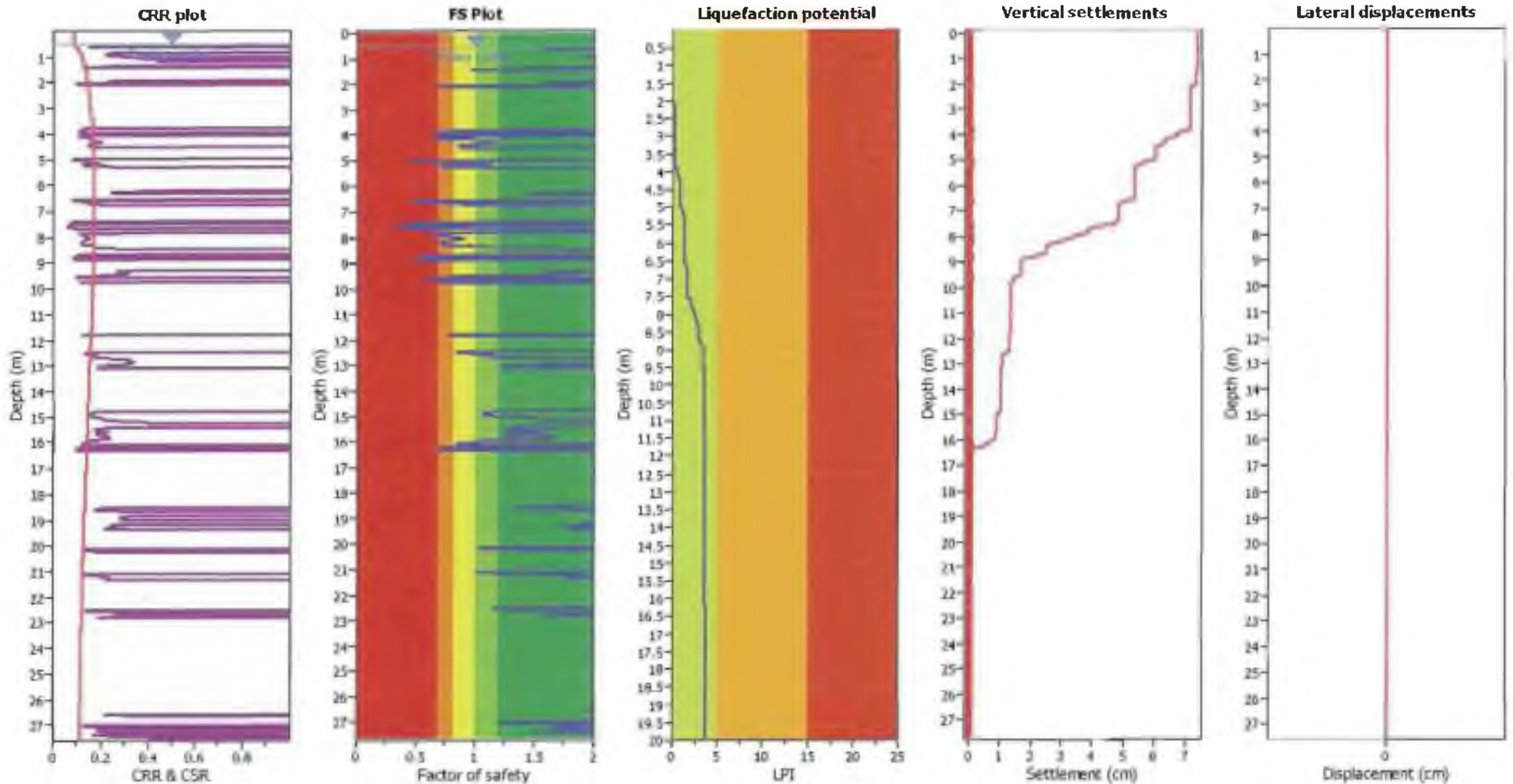
F.S. color scheme

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- Unlikely to liquefy
- Almost certain It will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect, applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_v applied:	Yes
Earthquake magnitude M_w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

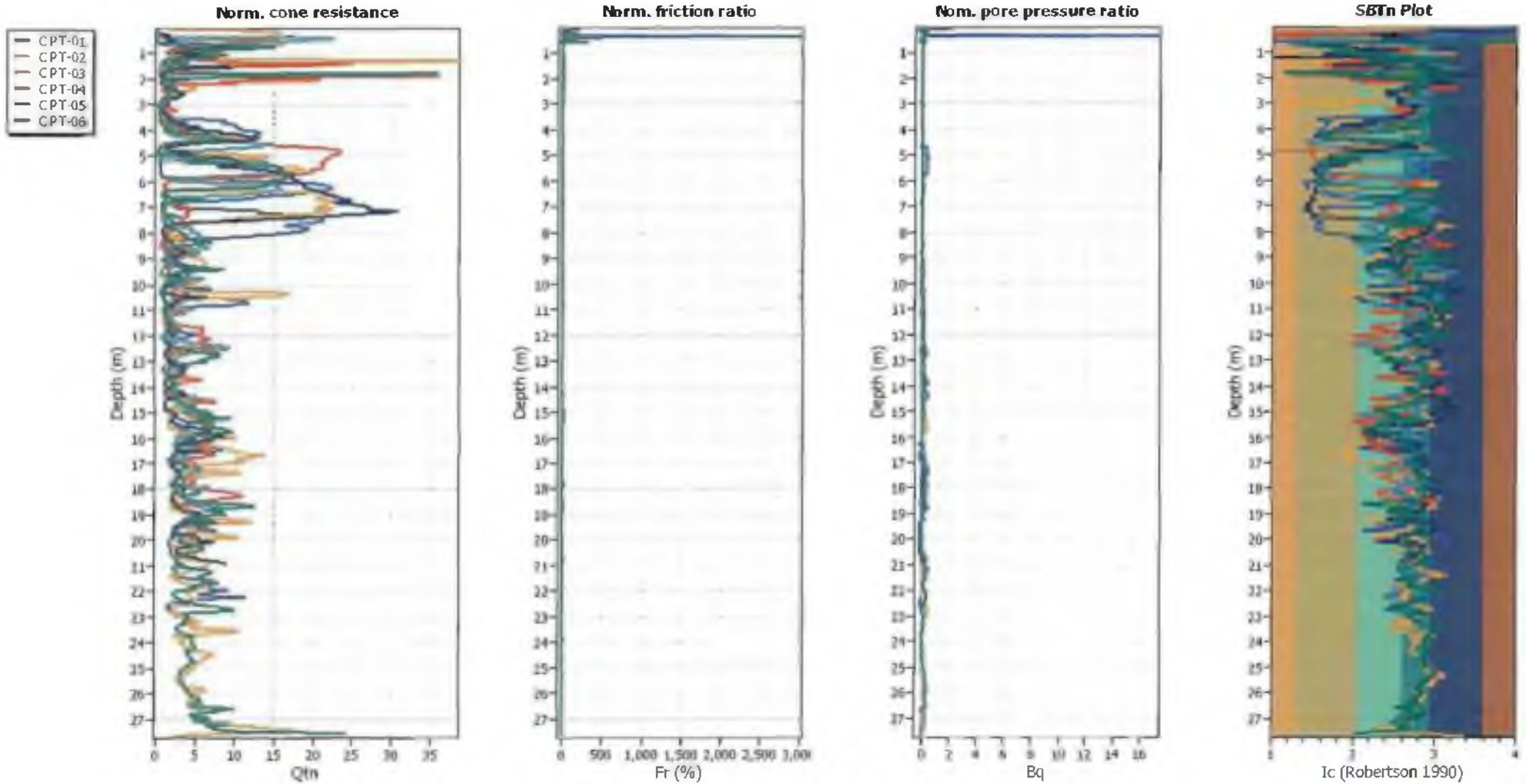
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LPI color scheme

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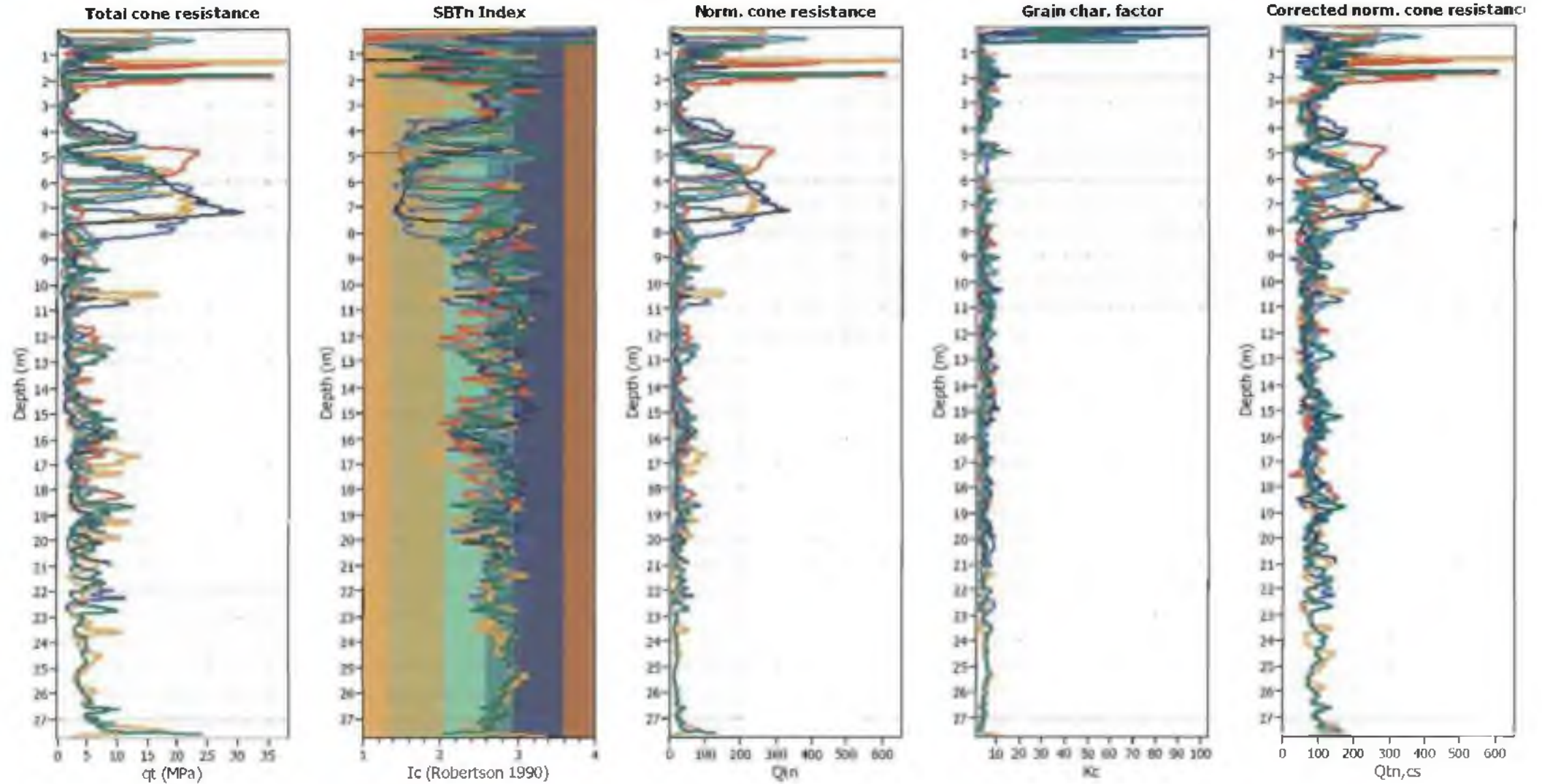
Project:

Overlay Normalized Plots



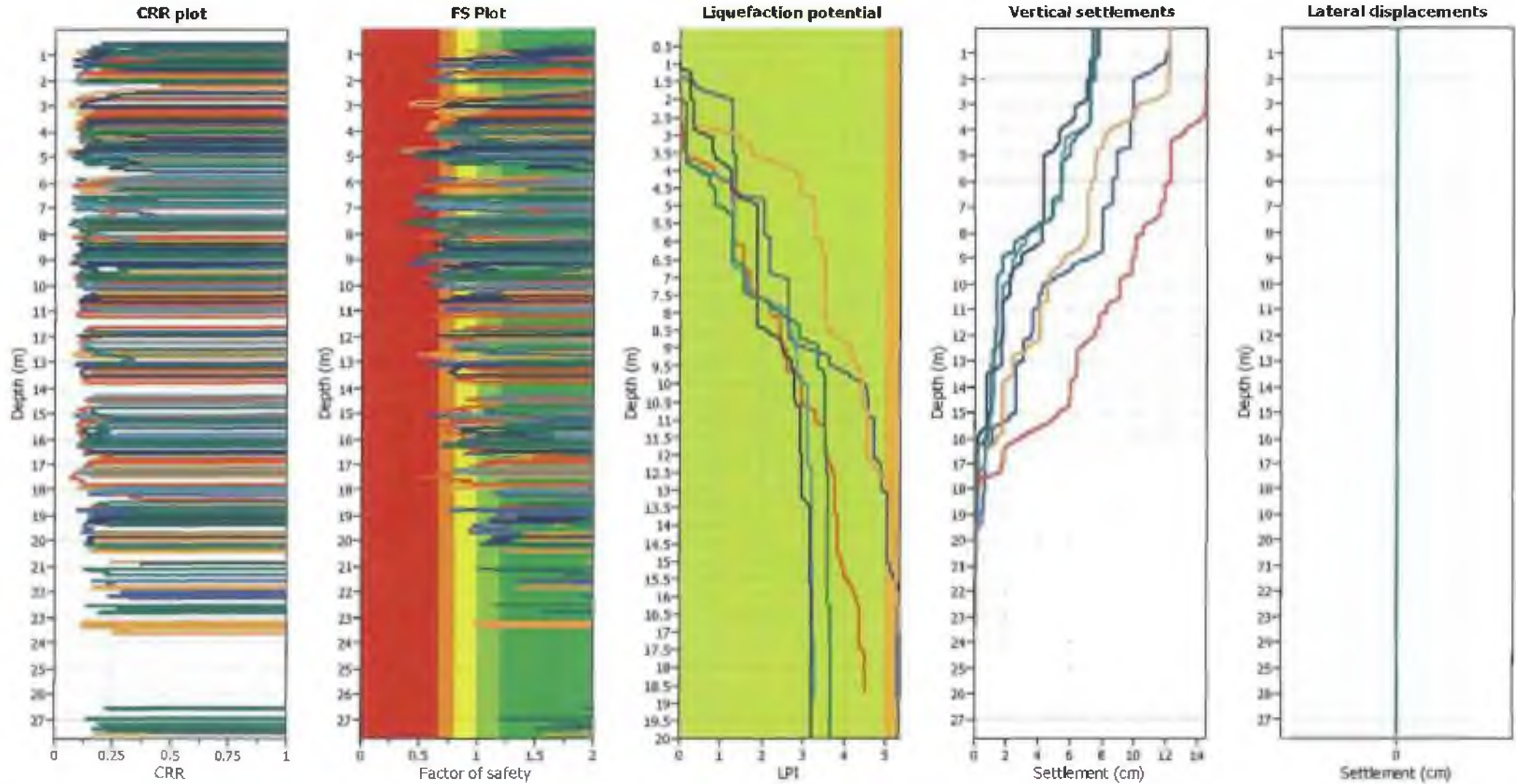
Project:

Overlay Intermediate Results



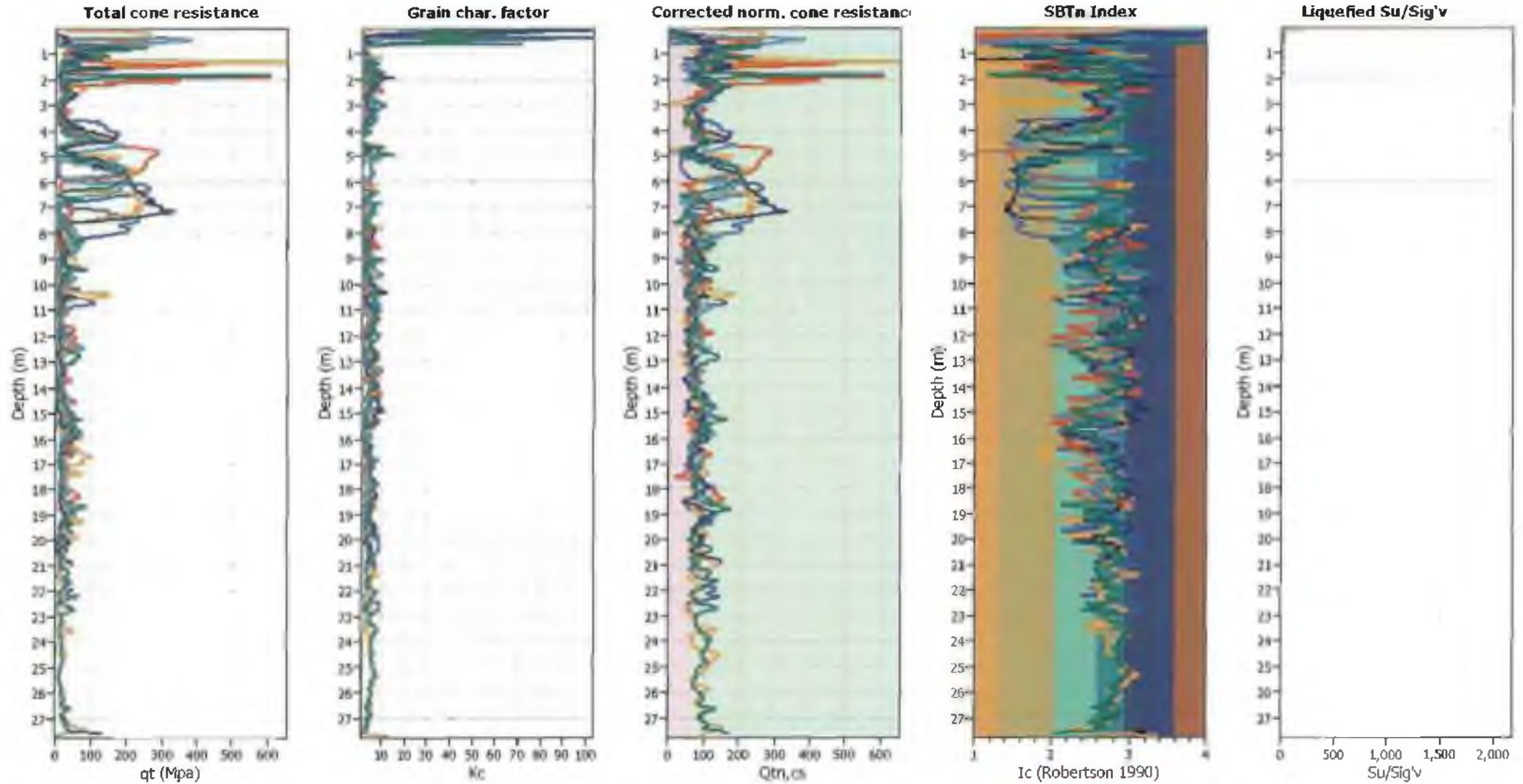
Project:

Overlay Cyclic Liquefaction Plots



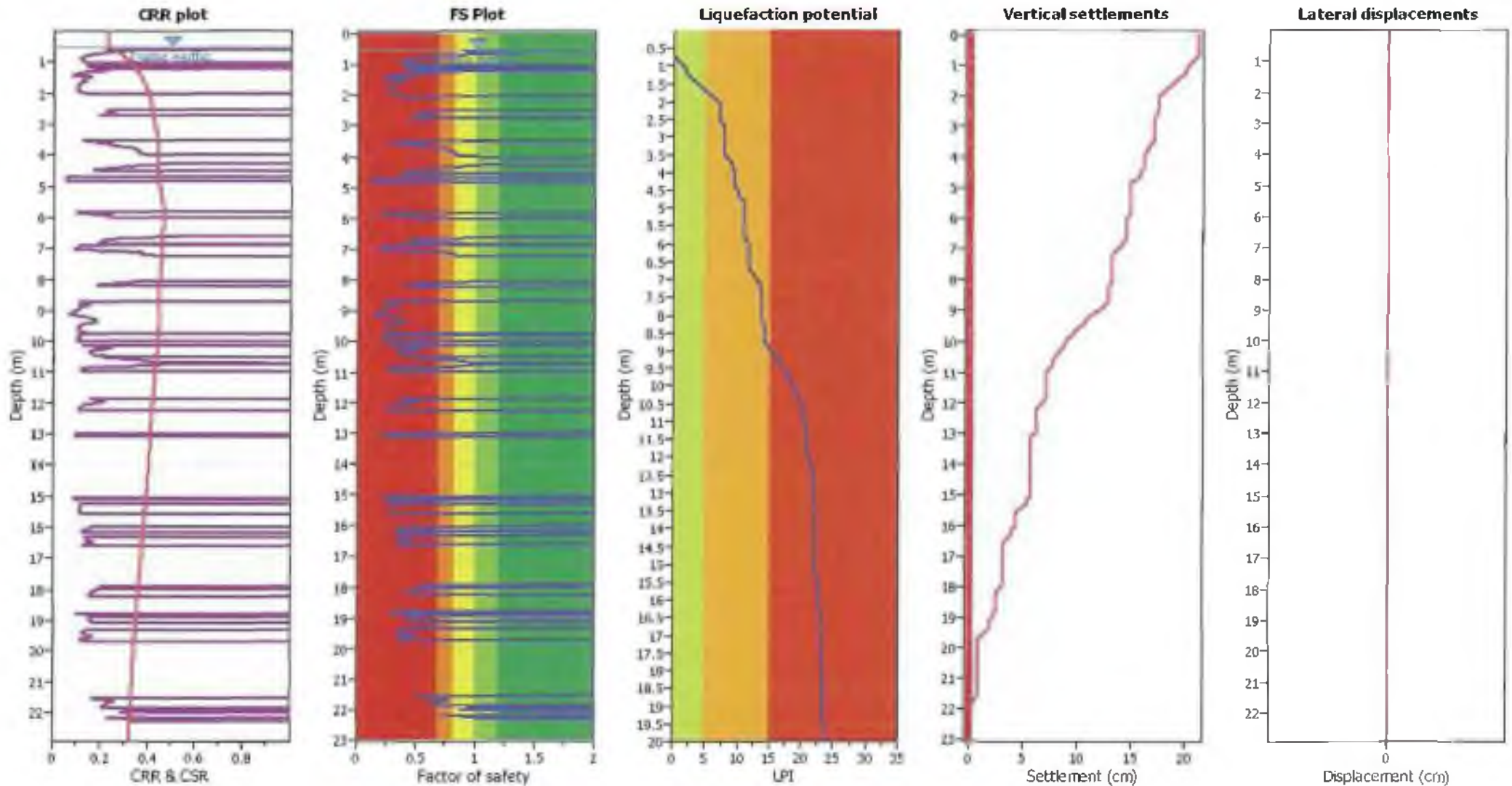
Project:

Overlay Strength Loss Plots



Ultimate Limit State (ULS): M7.5, 0.35g peak horizontal ground acceleration

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earth.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_0 applied:	Yes
Earthquake magnitude M_w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

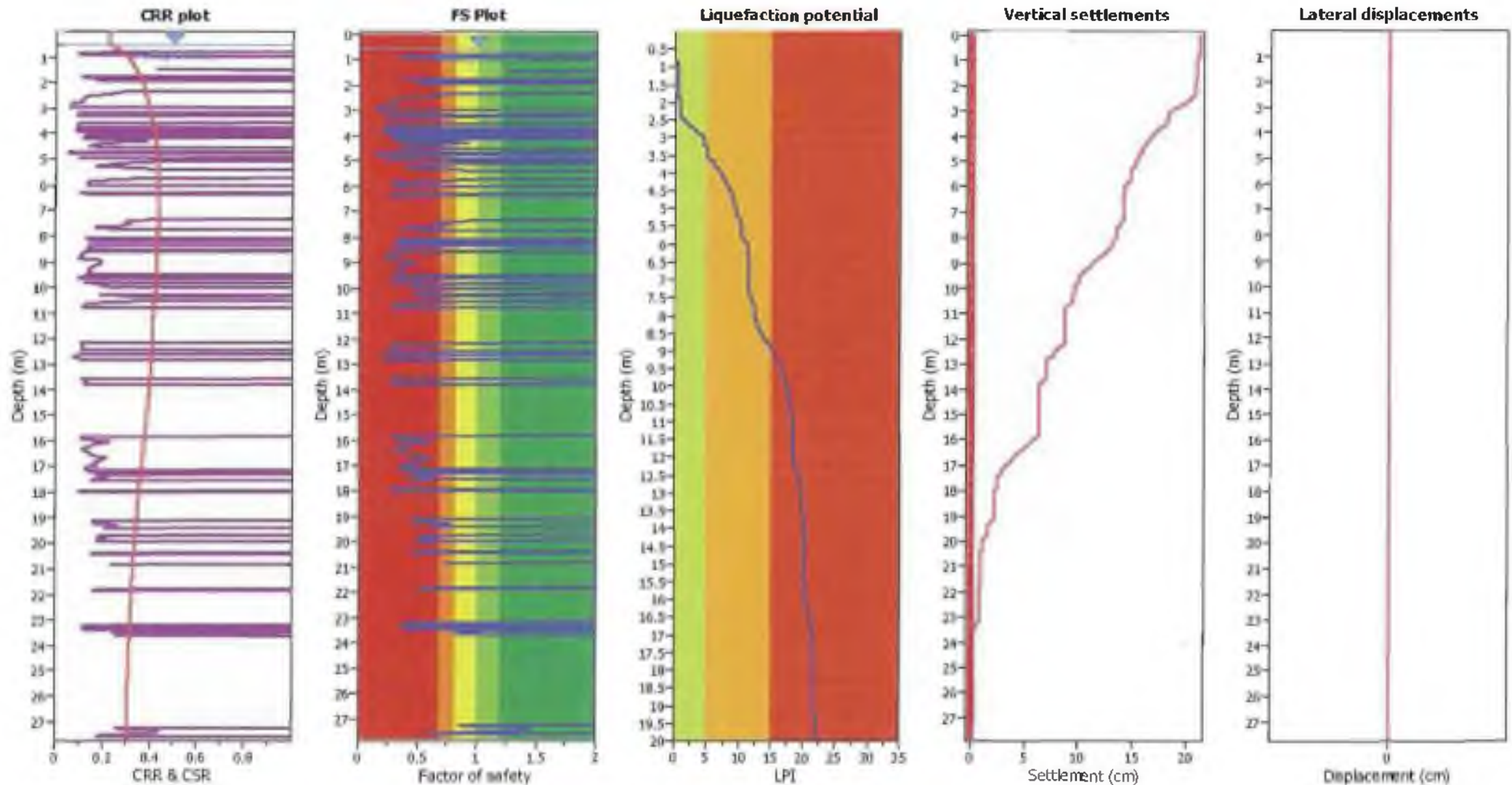
F.S. color scheme

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- Liquefaction and no liquefaction are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	No
Depth to water table (In situ):	2.50 m	Fill height:	N/A	Limit depth:	N/A

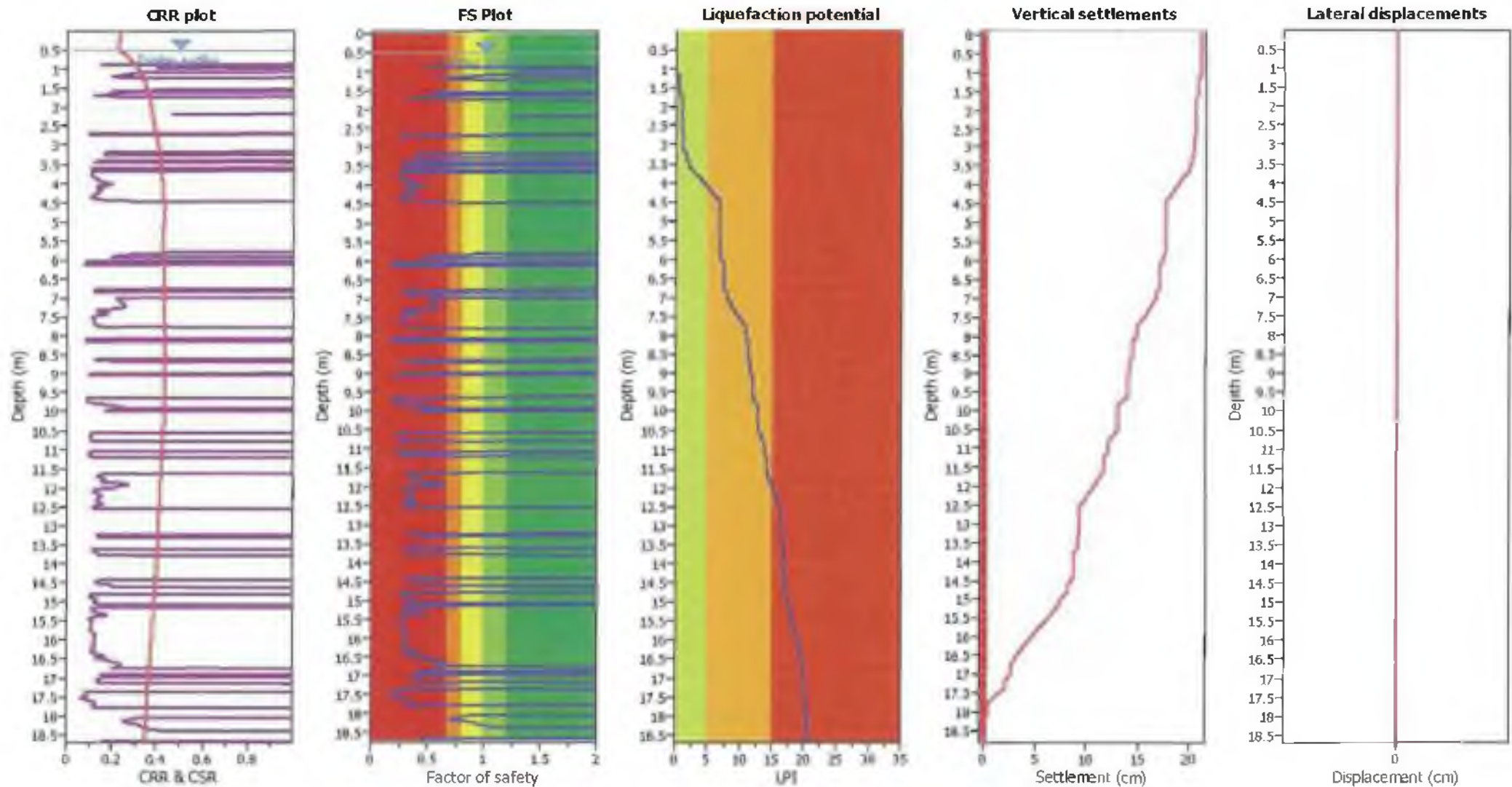
F.S. color scheme

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LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_0 applied:	Yes
Earthquake magnitude M_w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

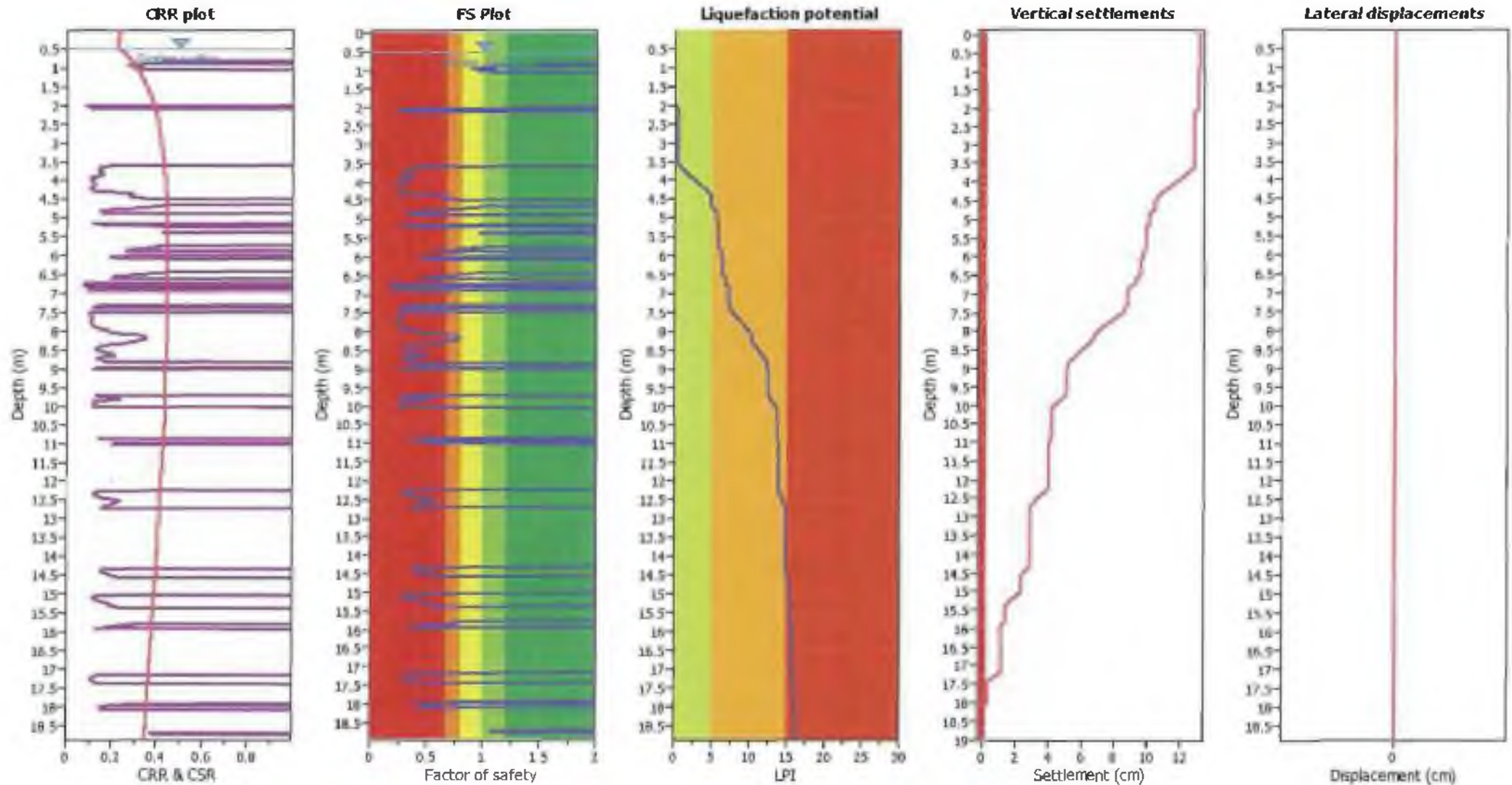
F.S. color scheme

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LPI color scheme

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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

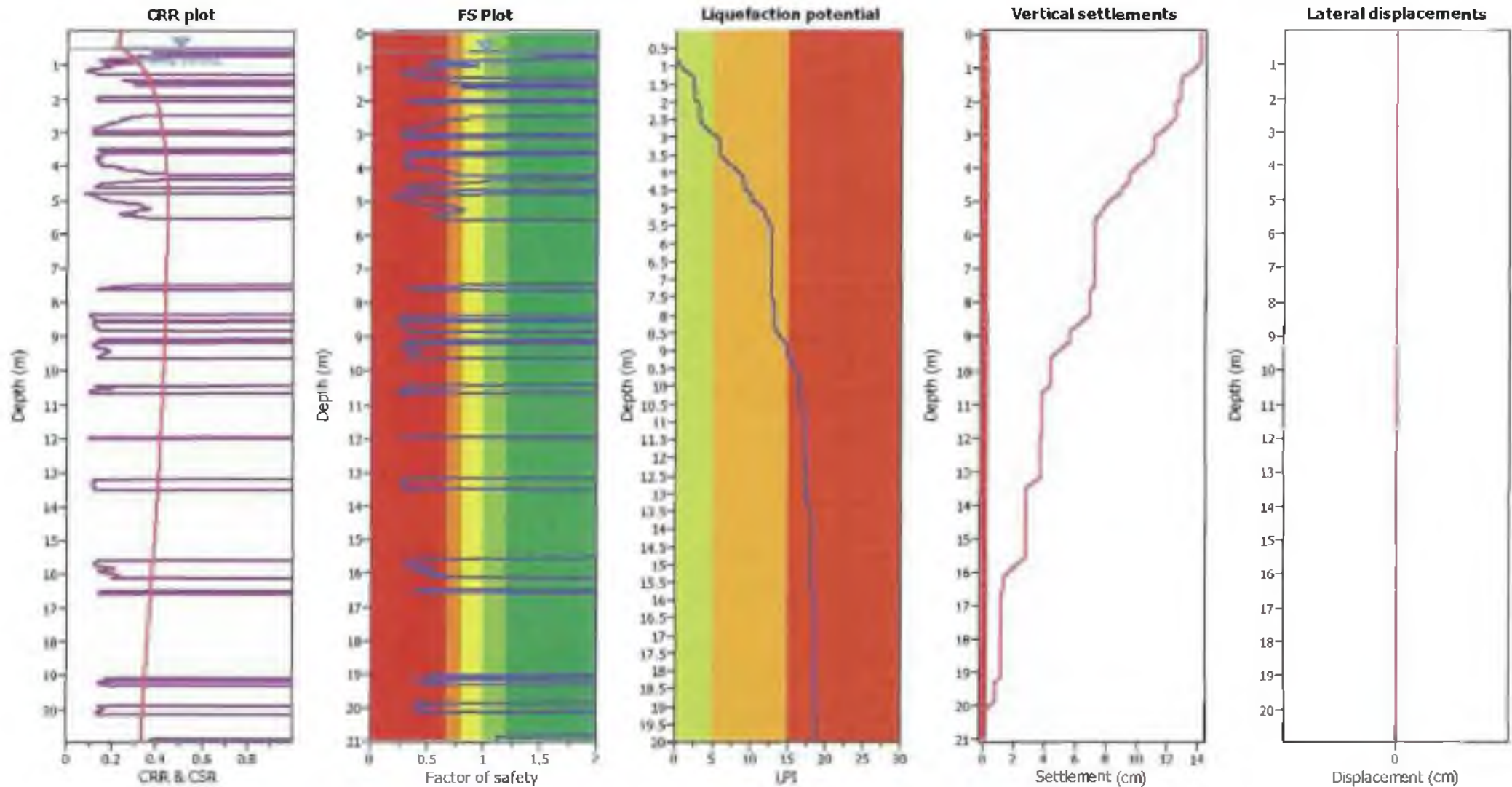
F.S. color scheme

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LPI color scheme

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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _σ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	No
Depth to water table (Insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

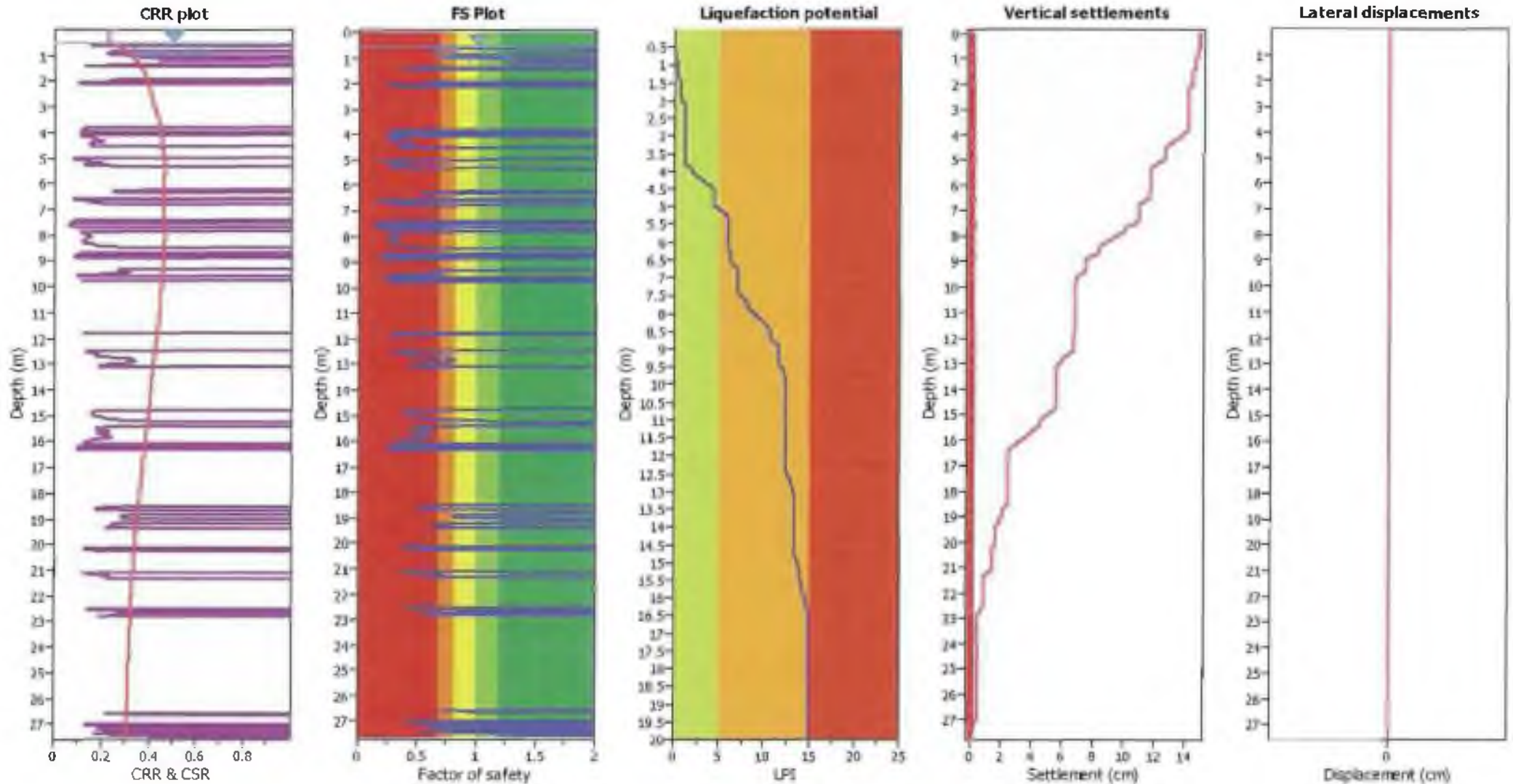
F.S. color scheme

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LPI color scheme

- Very high risk
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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	No
Depth to water table (Inst):	2.50 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

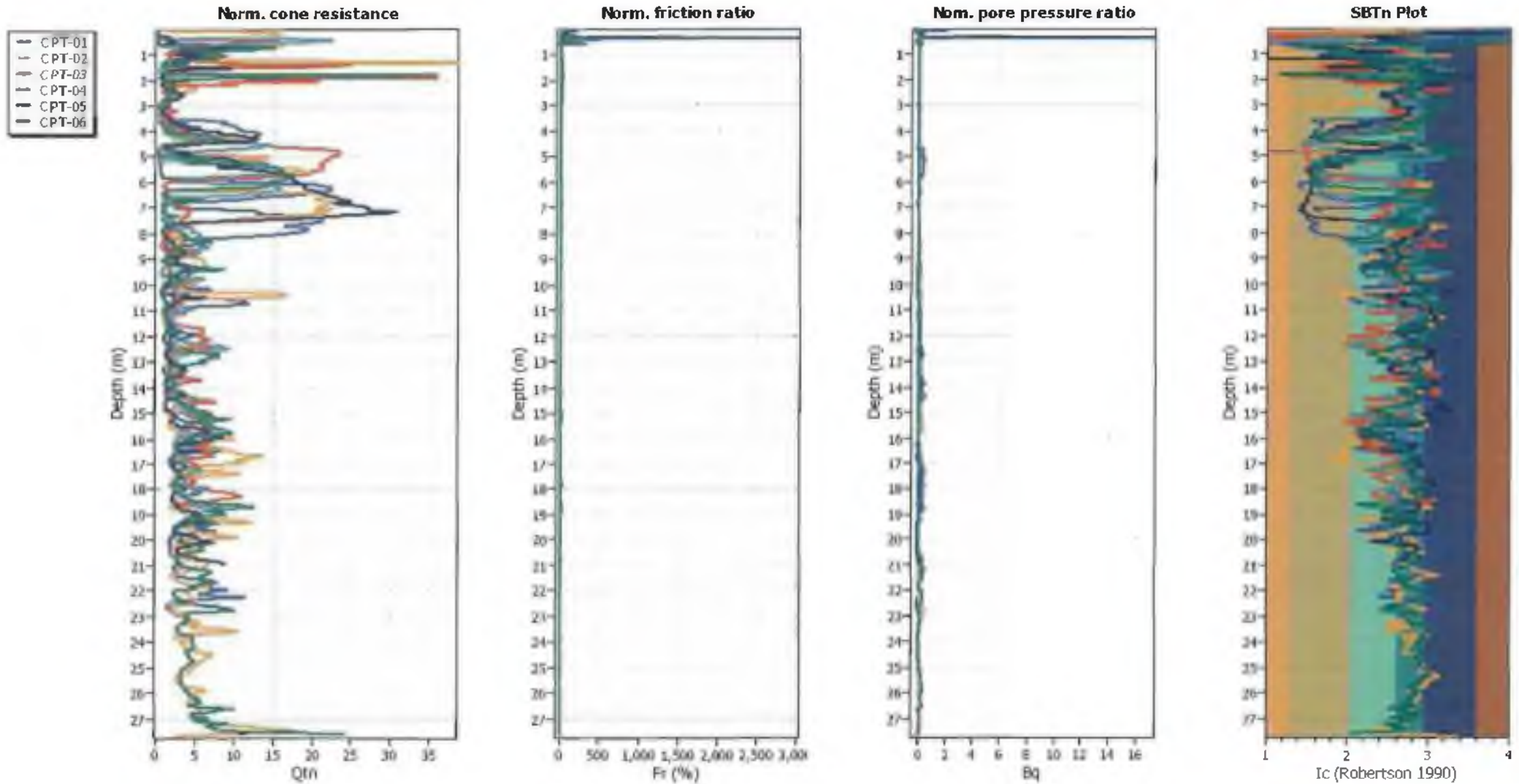
- Almost certain it will liquefy
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LPI color scheme

- Very high risk
- High risk
- Low risk

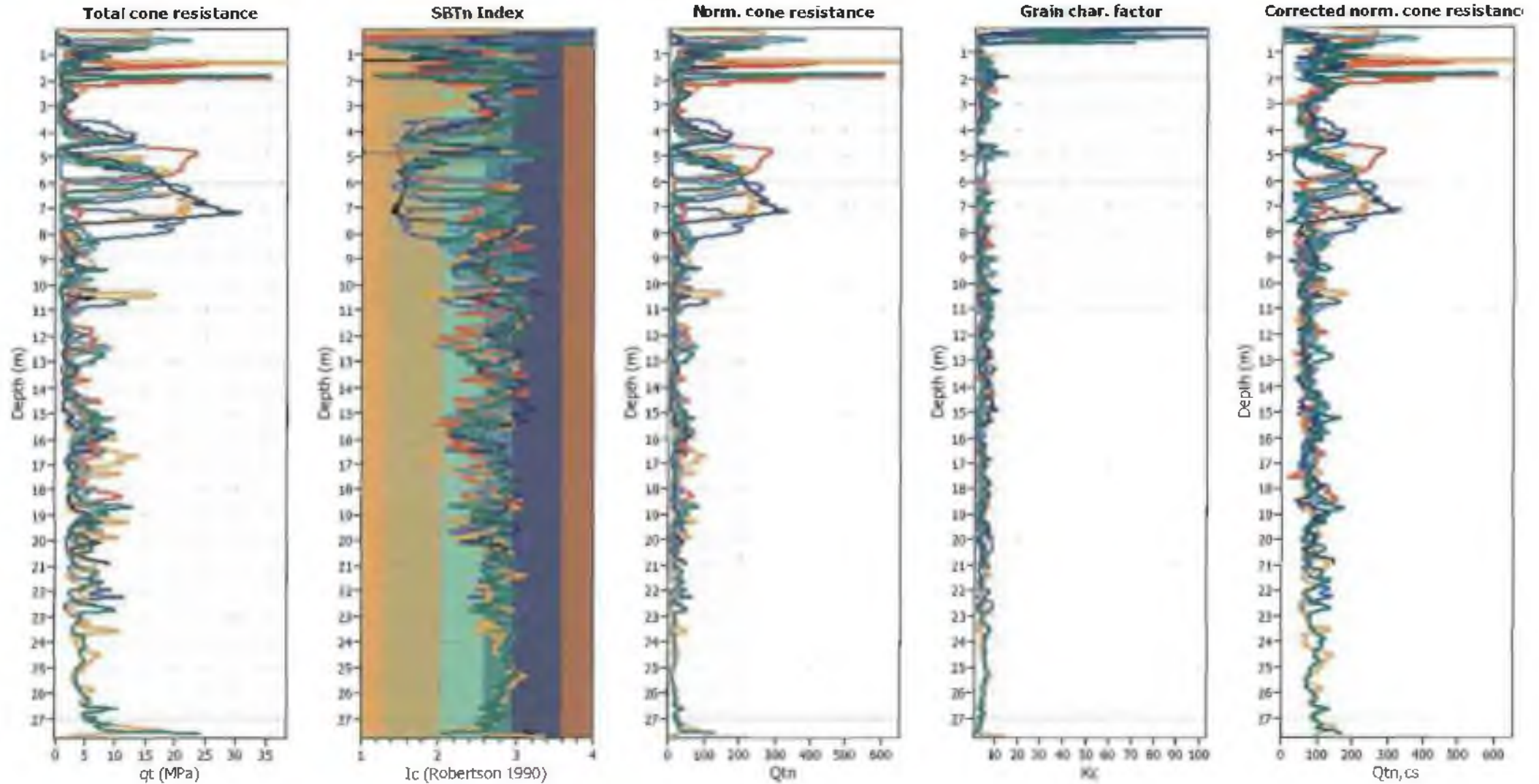
Project:

Overlay Normalized Plots



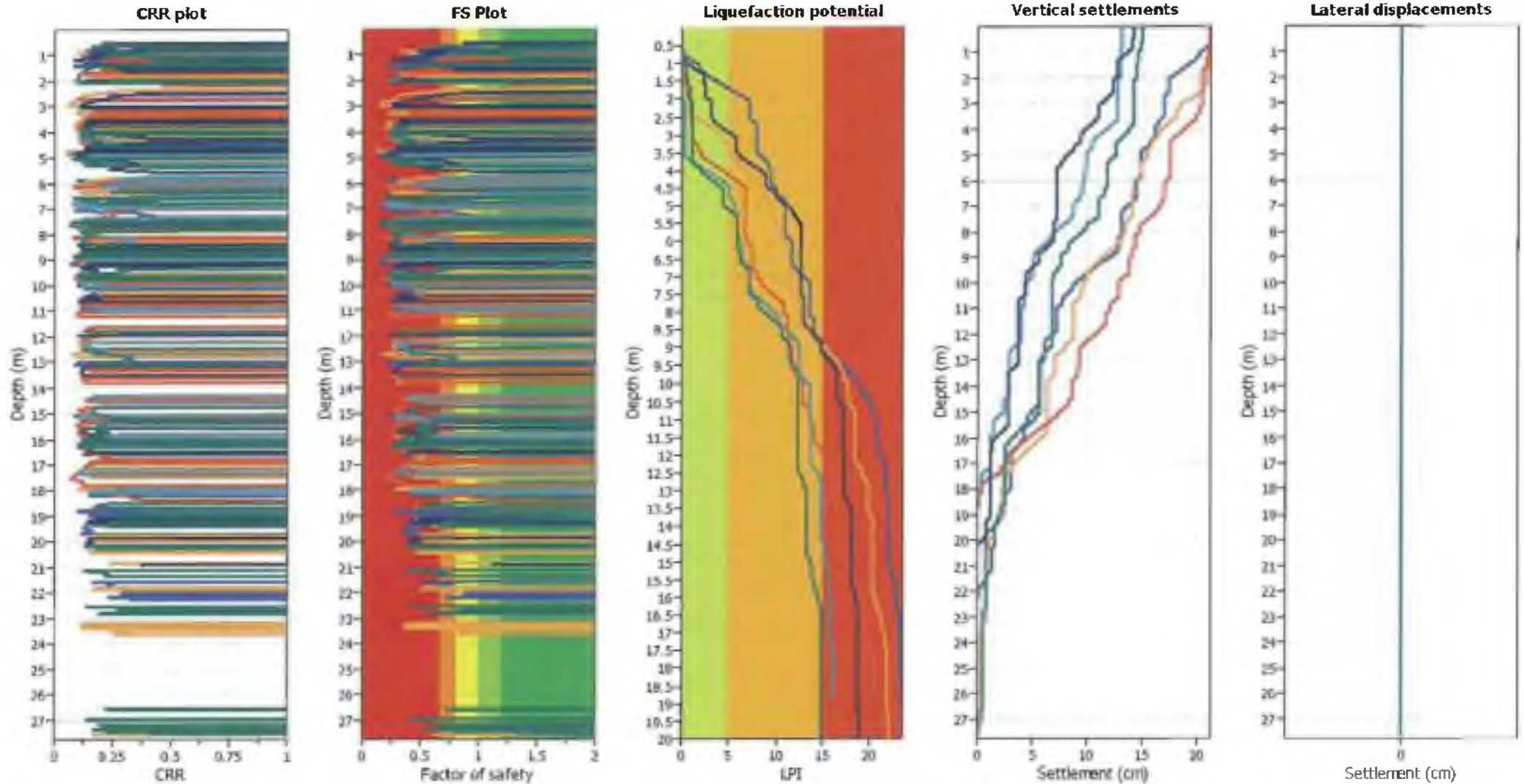
Project:

Overlay Intermediate Results



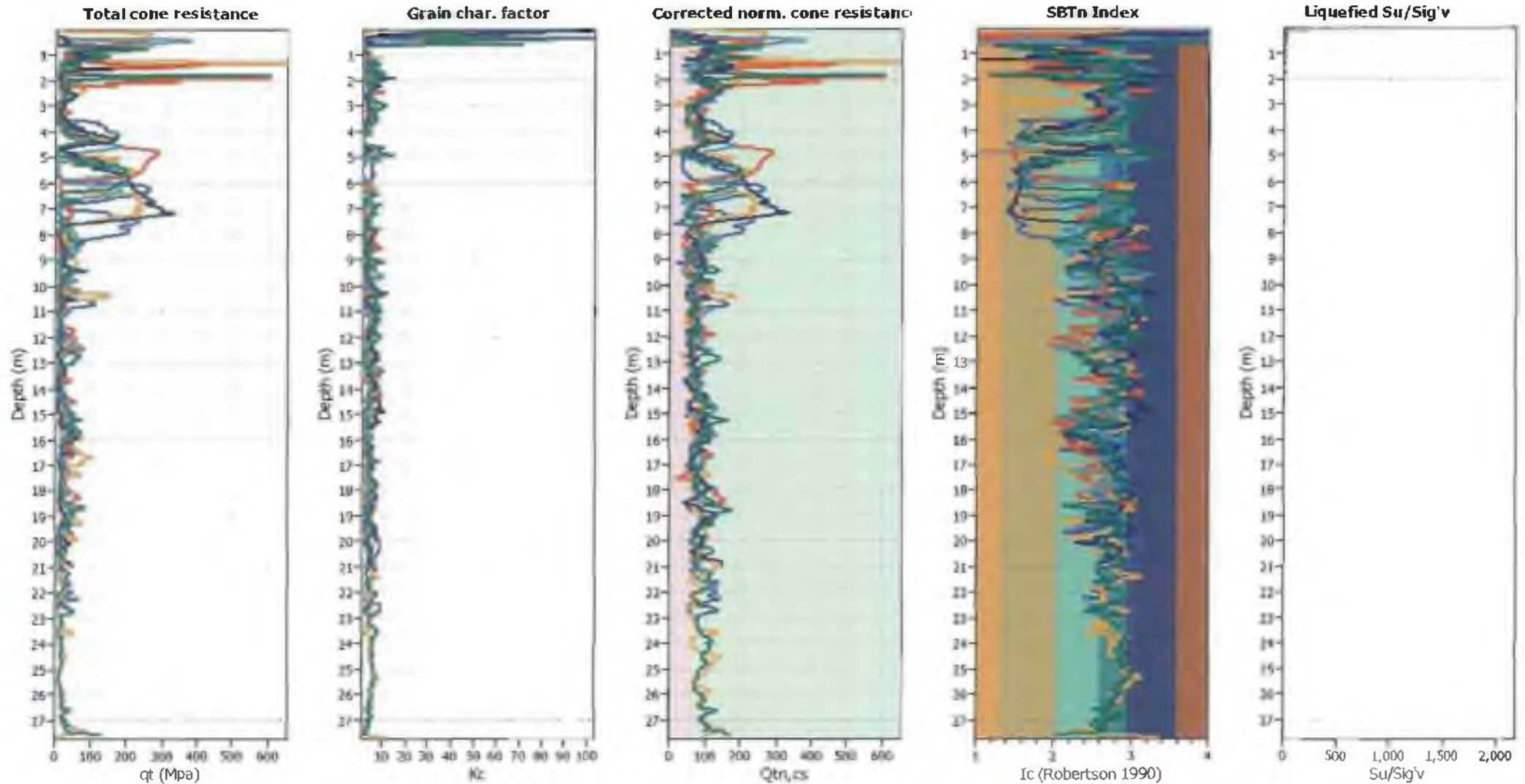
Project:

Overlay Cyclic Liquefaction Plots



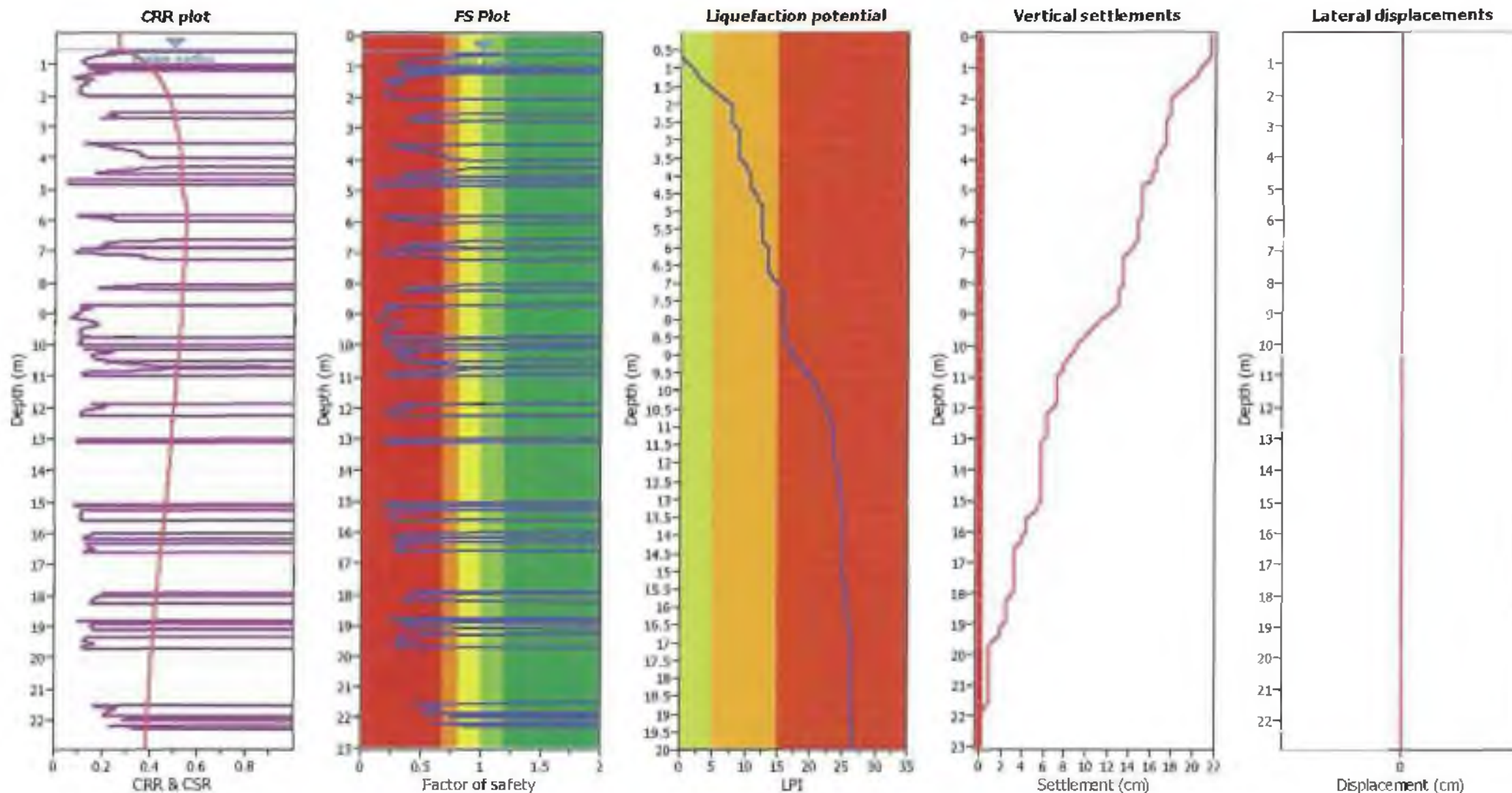
Project:

Overlay Strength Loss Plots



22 February 2011 M6.3, 0.65g peak horizontal ground acceleration

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on t_c value	t_c cut-off value:	2.60	K_0 applied:	Yes
Earthquake magnitude M_w :	6.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (Insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

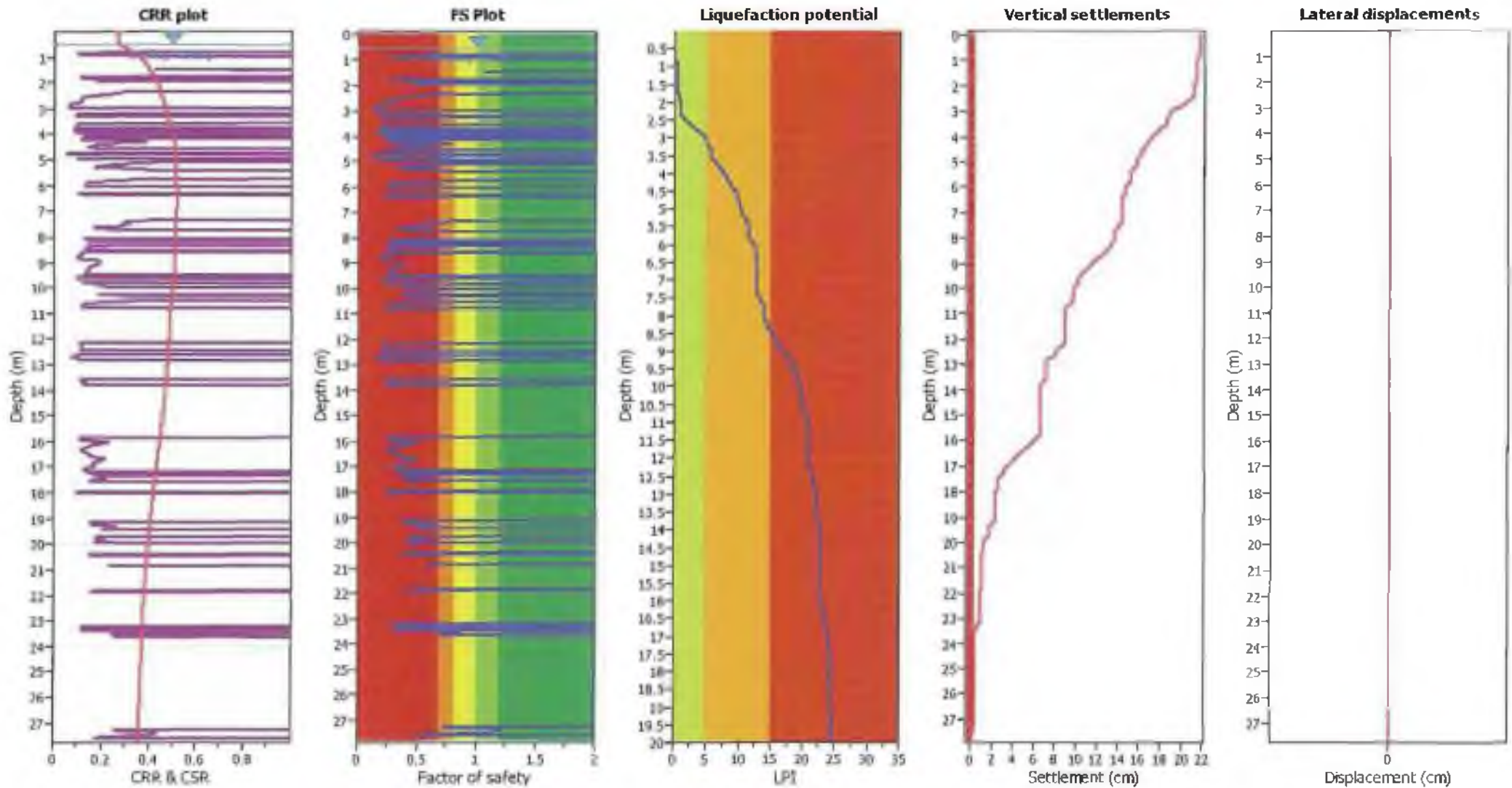
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
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LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1990)	Depth to water table (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_0 applied:	Yes
Earthquake magnitude M_w :	6.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (Instk):	2.50 m	Fill height:	N/A	Limit depth:	N/A

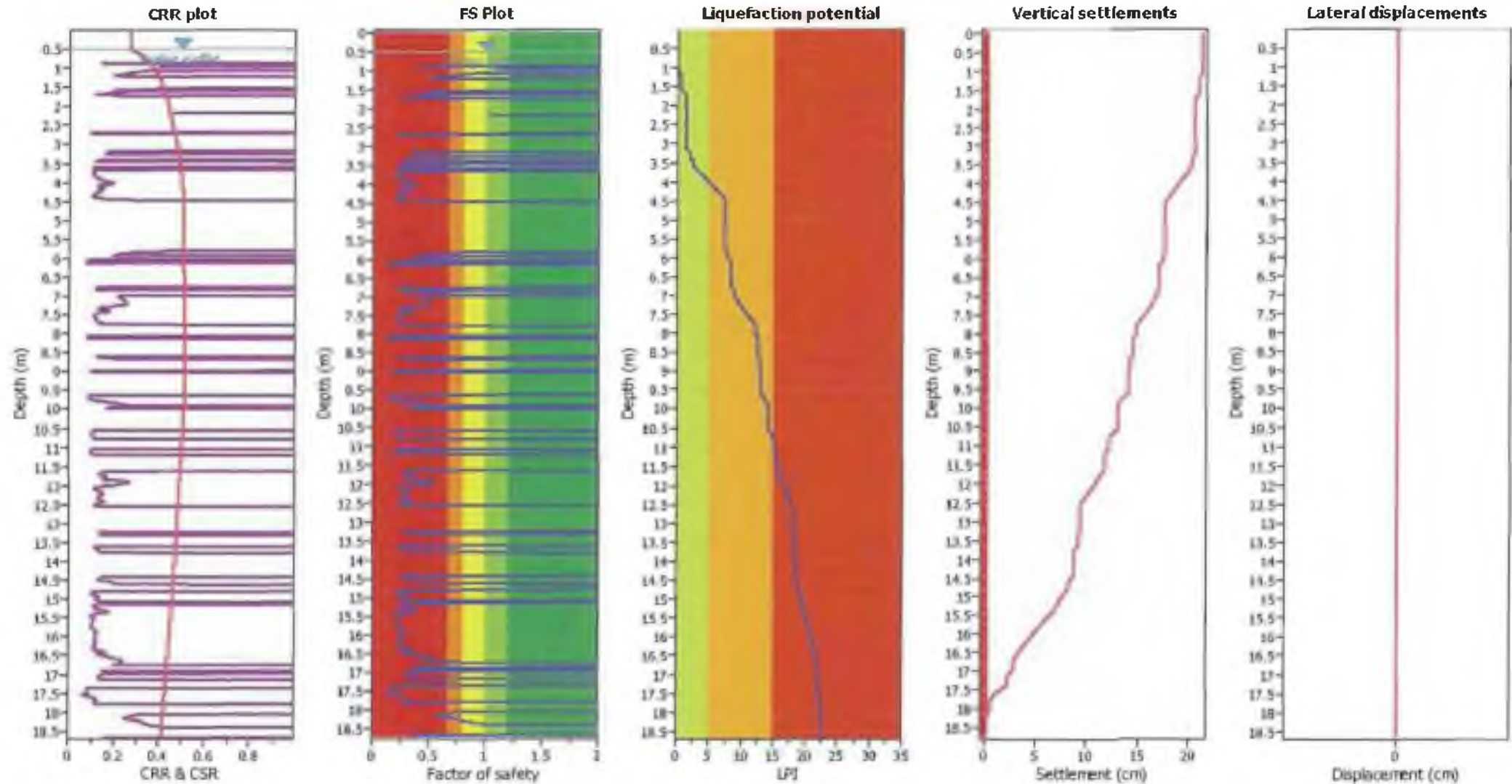
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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on I_c value	I_c cut-off value:	2.60	K_{α} applied:	Yes
Earthquake magnitude M_w :	6.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.6S	Use fill:	No	Limit depth applied:	No
Depth to water table (Instu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

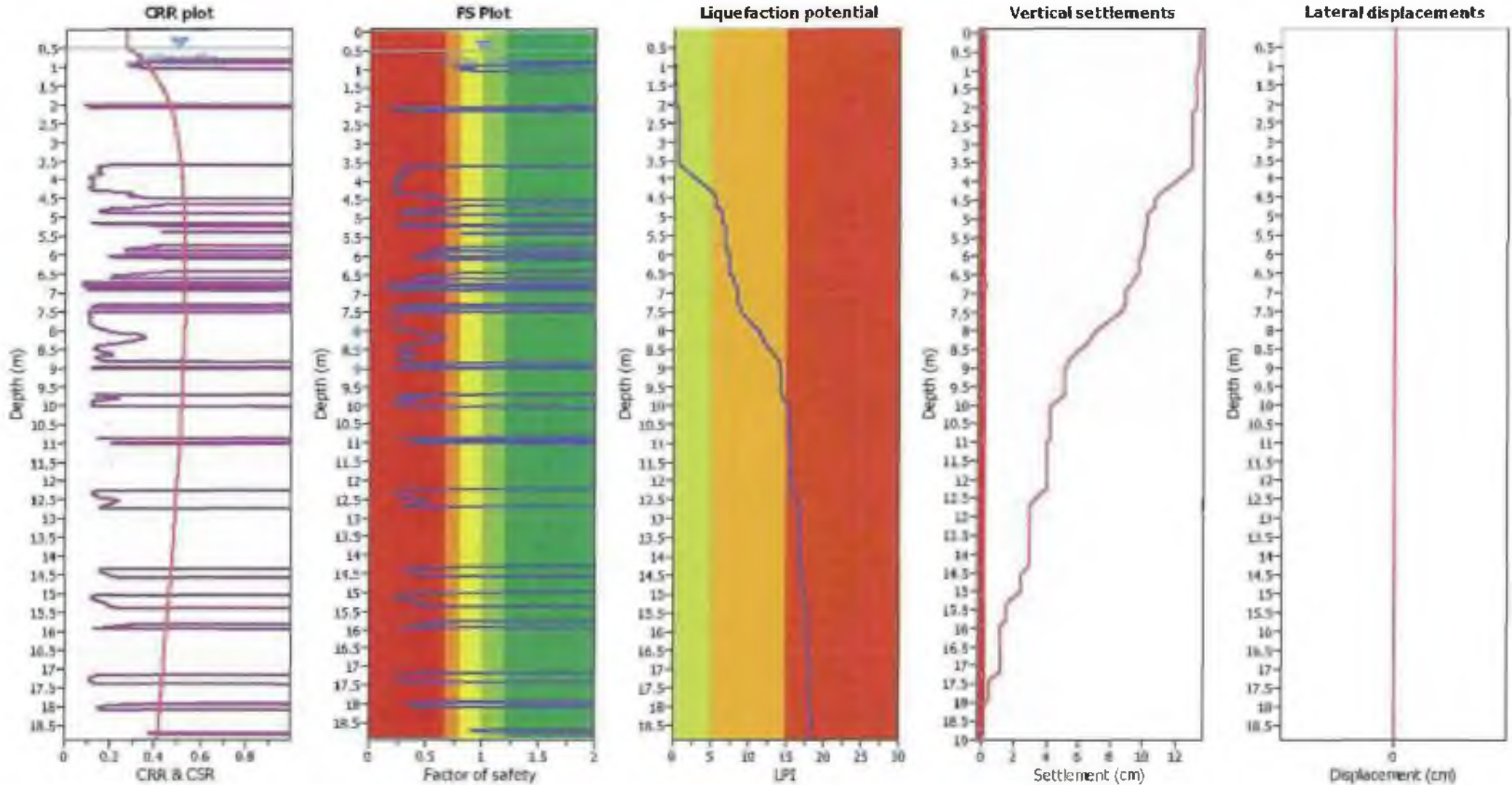
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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results Interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_v applied:	Yes
Earthquake magnitude M_w :	6.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (instiu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

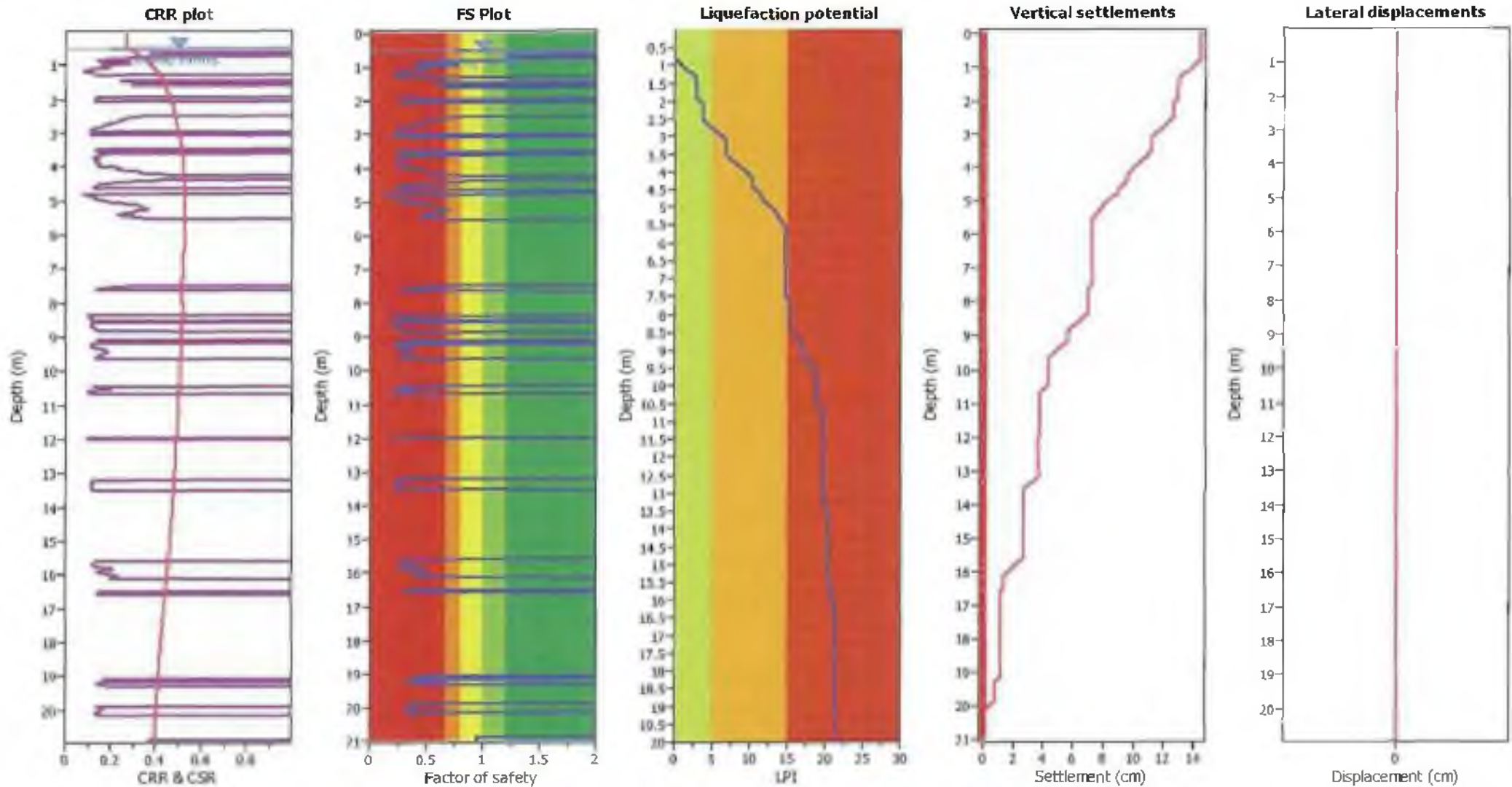
F.S. color scheme

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LPI color scheme

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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results Interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	6.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (INSRU):	2.50 m	Fill height:	N/A	Limit depth:	N/A

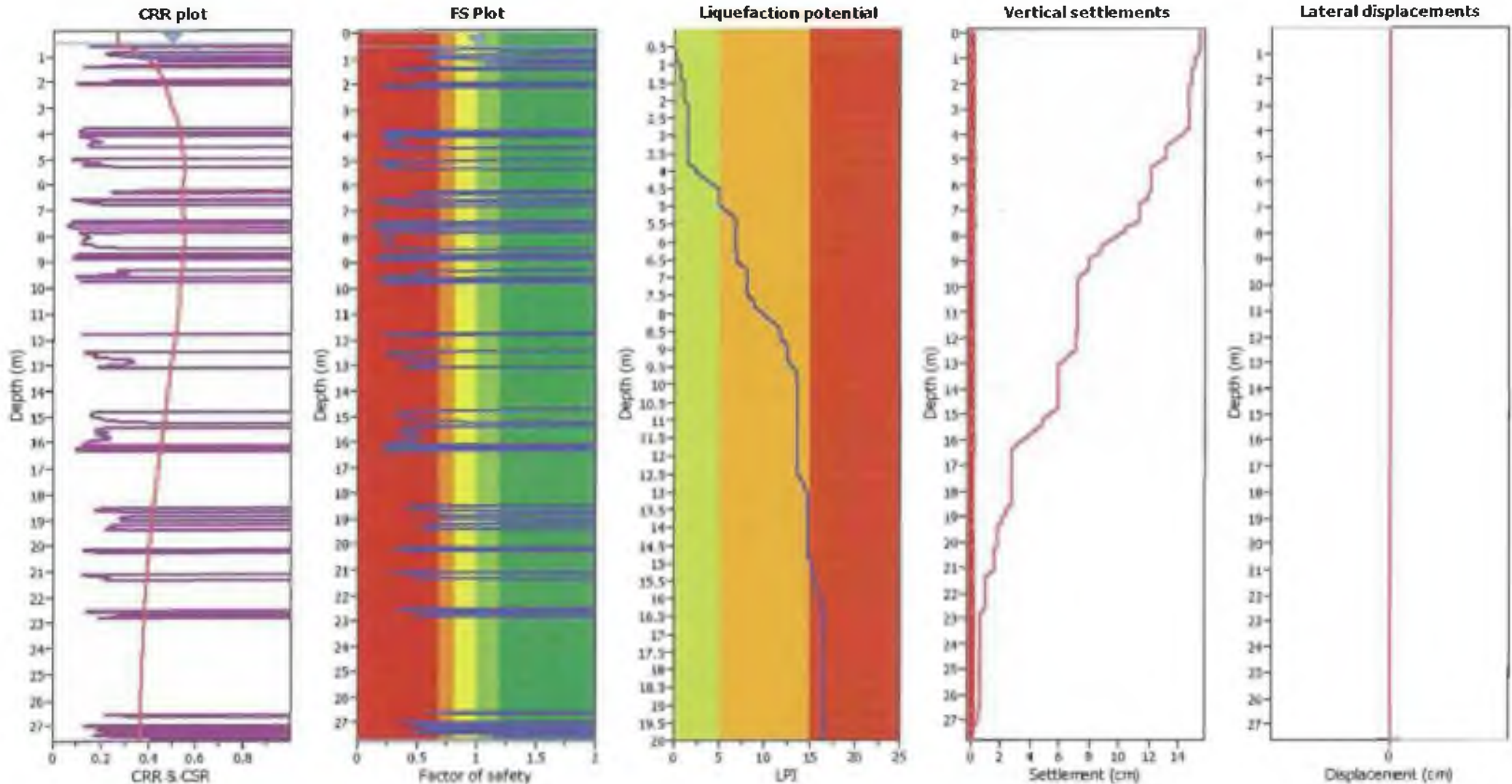
F.S. color scheme

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LPI color scheme

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- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on I _c value	I _c cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	6.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (instn):	2.50 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

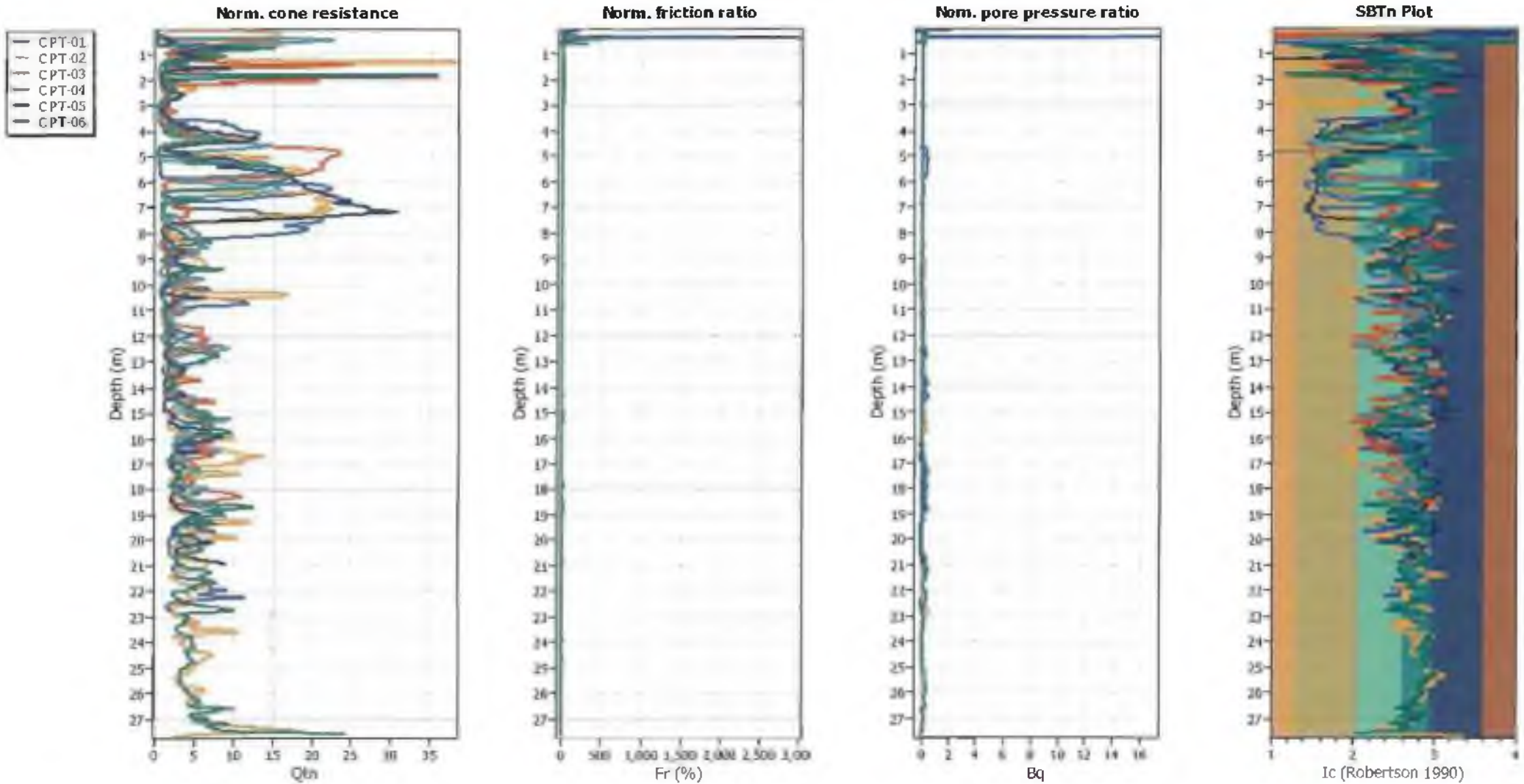
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LPI color scheme

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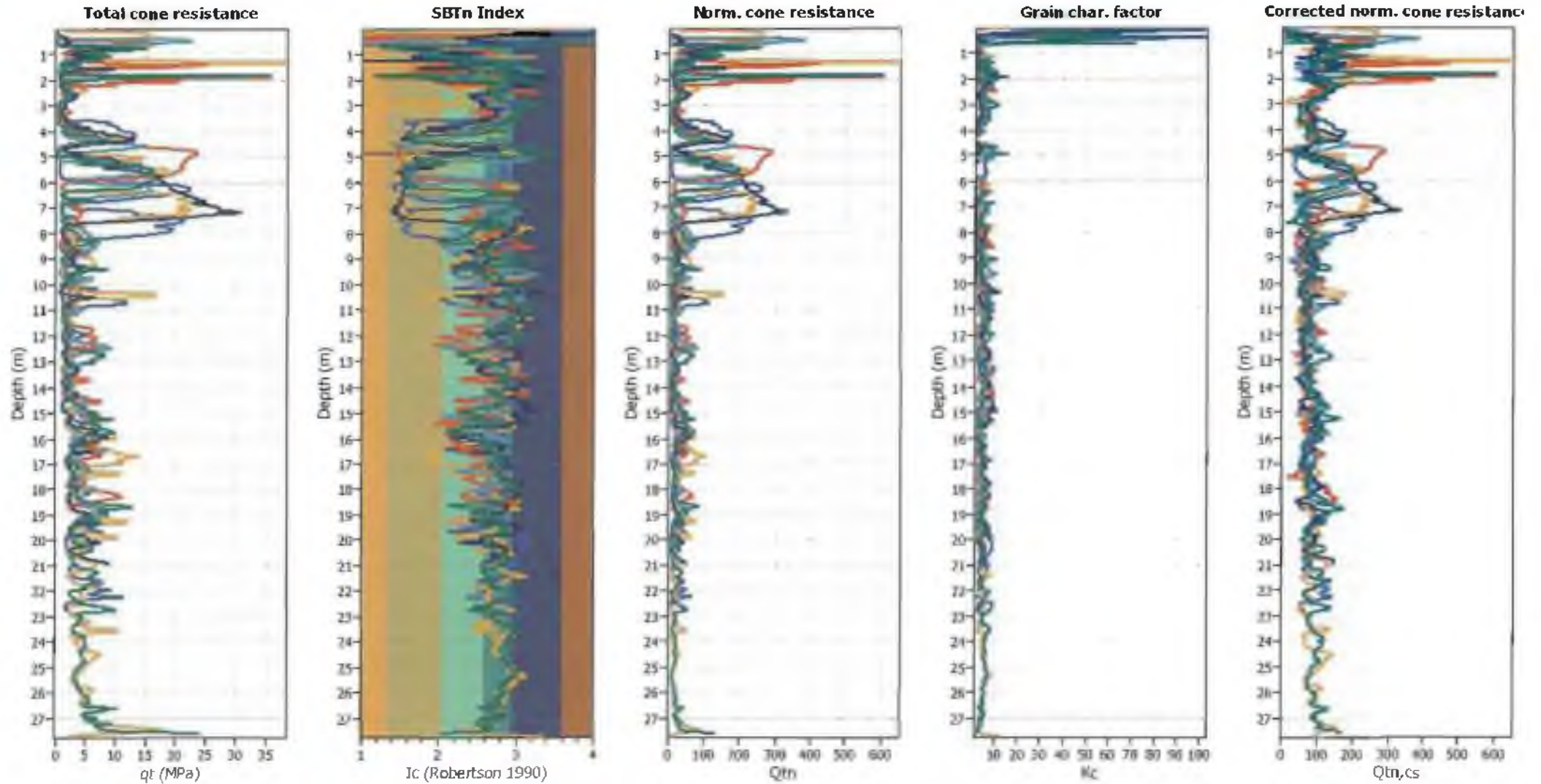
Project:

Overlay Normalized Plots



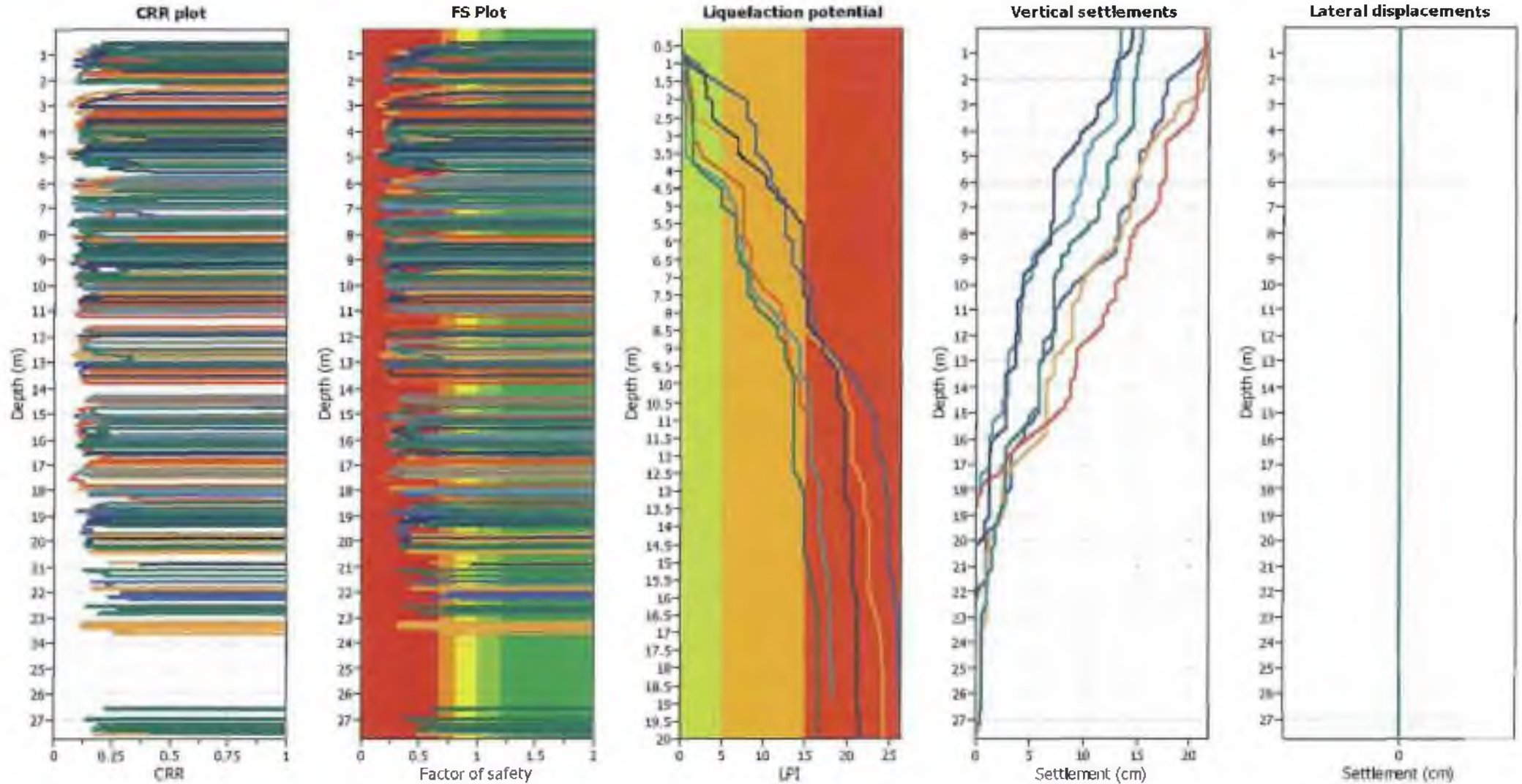
Project:

Overlay Intermediate Results



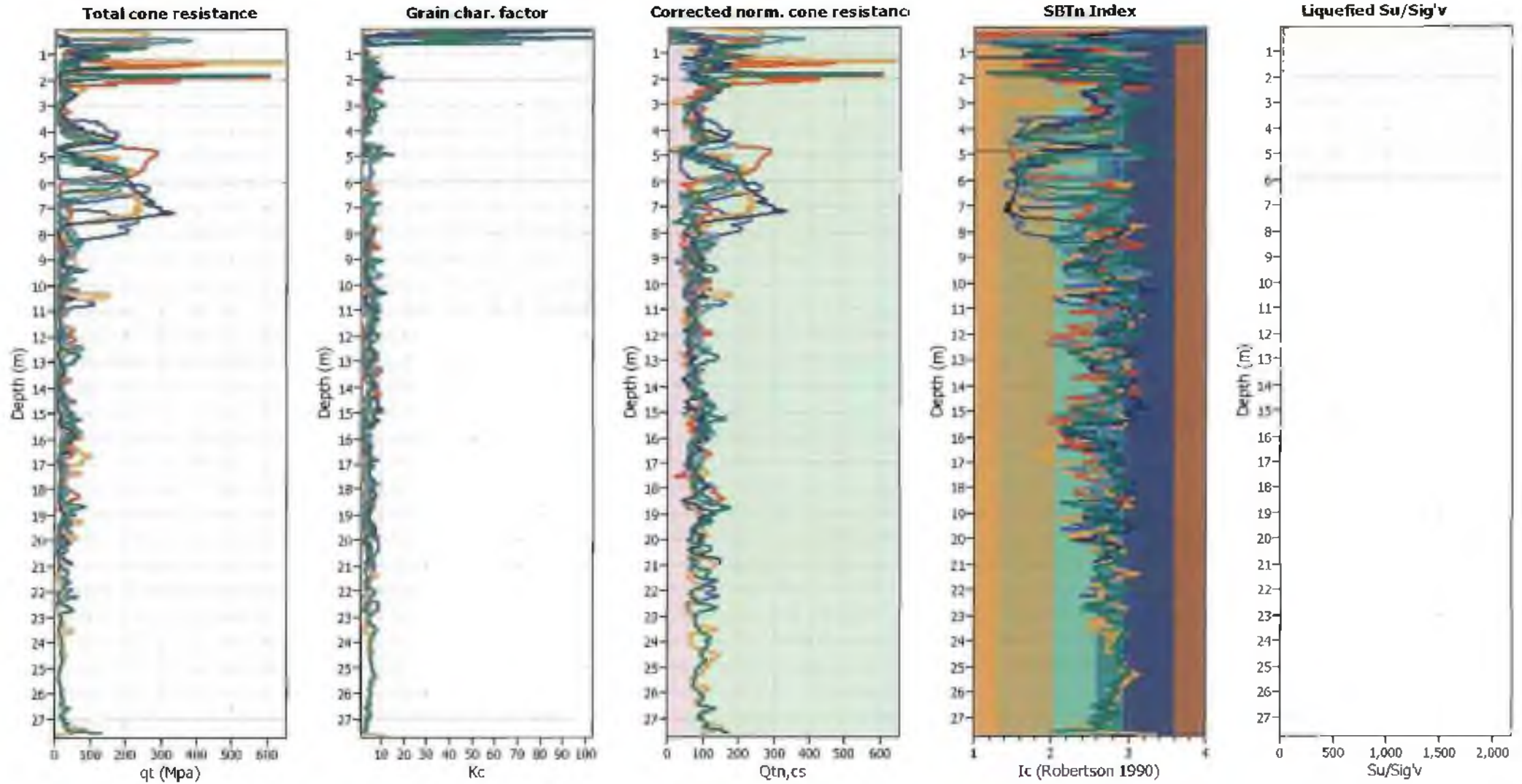
Project:

Overlay Cyclic Liquefaction Plots



Project:

Overlay Strength Loss Plots



16. APPENDIX E – OBSERVED LAND DAMAGE



Land Damage on Mary Muller Drive. The approximate site boundary is in yellow. Source: Canterbury Recovery, Project Orbit. Photographs taken 24 February 2011.

17. APPENDIX F – DBH GUIDELINES, TABLE B2.1

Table B2.1: Liquefaction deformation limits and house foundation implications

Liquefaction Hazard	Liquefaction Deformation Limits (mm)				Implications for House Foundations
	Vertical settlement		Lateral spread		
	SLS	ULS	SLS	ULS	
TC1	15 mm	25 mm	nil	nil	Standard 3604-like foundations with tied slabs ²
TC2	50 mm	100 mm	50 mm	100 mm	The Department's enhanced foundation solutions (section 5.2)
TC3	>50 mm	>100 mm	>50 mm	>100 mm	Site-specific measures - piles, ground improvement, etc

² Note that certain foundation details included in NZS 3604 are precluded from use in Canterbury (refer to: www.dbh.govt.nz/information-sheet-seismicity-changes).

Source: Department of Building and Housing, November 2011. 'Revised guidance on repairing and rebuilding houses affected by the Canterbury earthquake sequence'. New Zealand

18. APPENDIX G – NZGS GUIDELINES, TABLE 6.1

Performance levels

The magnitude of liquefaction-induced ground displacements is generally related to the liquefaction triggering factor F_L and to the overall thickness of the liquefied layer (Ishihara, 1985; Ishihara and Yoshimine, 1992). Based on general interpretation of these relations, Table 6.1 summarizes performance levels for liquefied soil deposits.

Table 6.1 General performance levels for liquefied deposits

Performance Level	Effects from excess pore water pressure and liquefaction	Characteristics of liquefaction and its consequences
L0	Insignificant	$F_L > 1.5$; No significant excess pore water pressures.
L1	Mild	$F_L > 1.2$; Limited excess pore water pressures without complete liquefaction; relatively small deformation of the ground with relatively small settlements (few tens of millimetres).
L2	Moderate	$F_L < 1.0$; Liquefaction occurs in layers of limited thickness (small proportion of the deposit); ground deformation results in differential settlements.
L3	High	$F_L < 1.0$; Liquefaction occurs in significant portion of the deposit resulting in differential movements, large settlements (few hundreds of millimetres) and lateral displacements.
L4	Severe	$F_L < 1.0$; Complete liquefaction develops in most of the deposit resulting in very large settlements (total and differential) and lateral displacements of the ground.
L5	Very severe	$F_L < 1.0$; Liquefaction resulting in lateral spreading (flow).

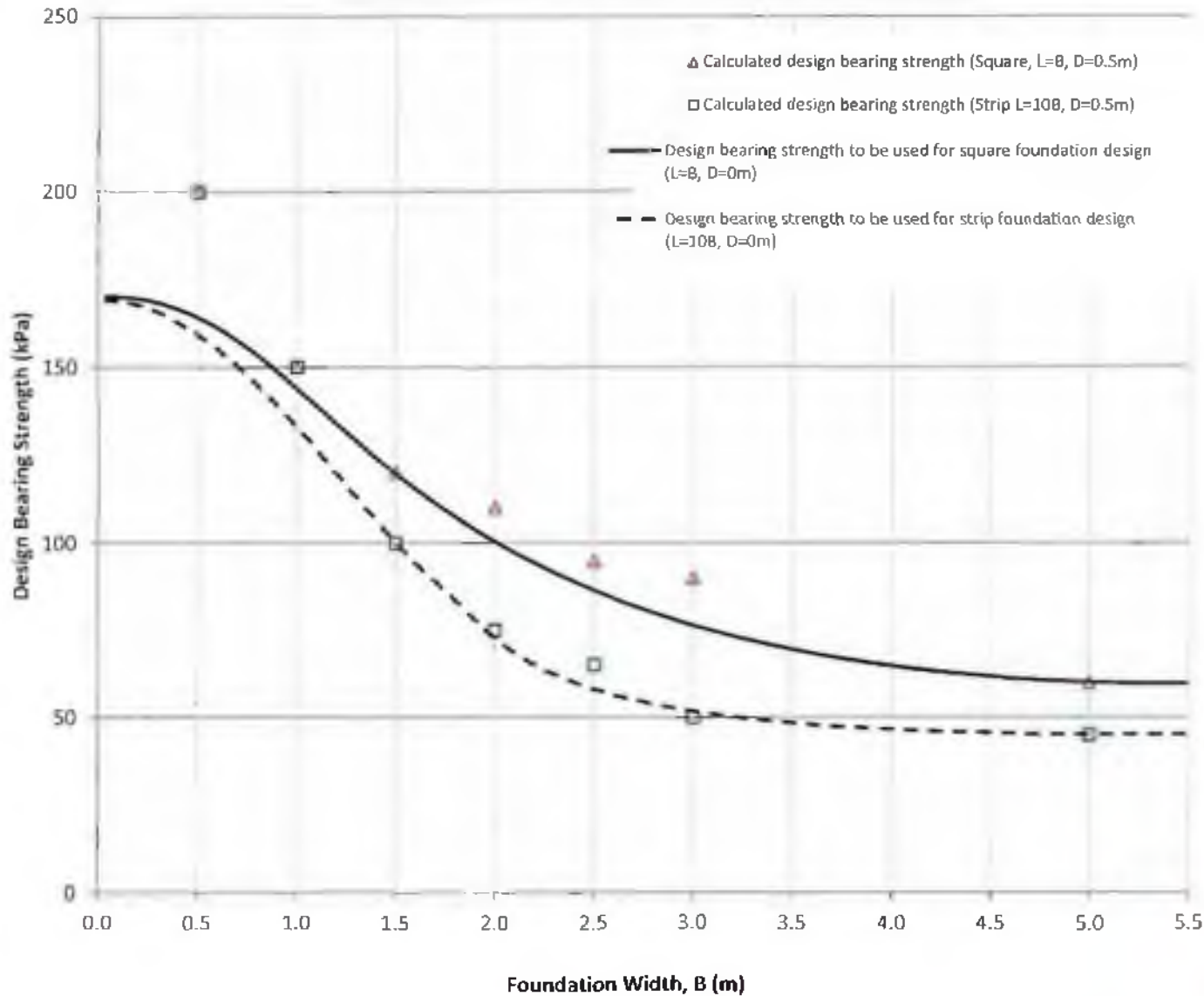
Table 6.1 should be used for general guidance only

Other factors

There are considerable uncertainties regarding the stiffness and strength of liquefying soils and consequent ground deformation. The magnitude and spatial distribution of lateral spreading displacements are particularly difficult to predict. These uncertainties should be considered in the design.

19. APPENDIX H – BEARING CAPACITY CONTROL OF SHALLOW FOUNDATIONS

BEARING CAPACITY CONTROL OF SHALLOW FOUNDATIONS



Notes:

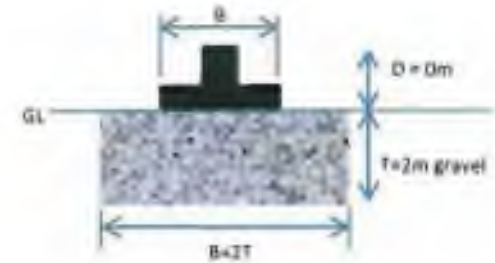
Foundation depth, $D = 0m$ below existing GL

Assumes 2m depth of compacted AP65 hardfill to underside of foundation, with min width $B+2T$

Structural Engineer should also design bearing strength of chart 'Settlement Control of Shallow Foundations'.

Design bearing strength = $0.5 \times$ Ultimate bearing strength [in accordance with NZBC Verification Method BM1/VMA4, Table 1, For bearing and passive earth pressure (all other load combinations)]

Design bearing strength at smaller foundation width has been conservatively limited as shown to take into account the presence of lower soil layers, or inferred geotechnical conditions indicated from other test results at an adjacent site.



Project: 12 Mary Muller Drive , Christchurch

Client: Castle Rock Properties Ltd.

Job #: 348392

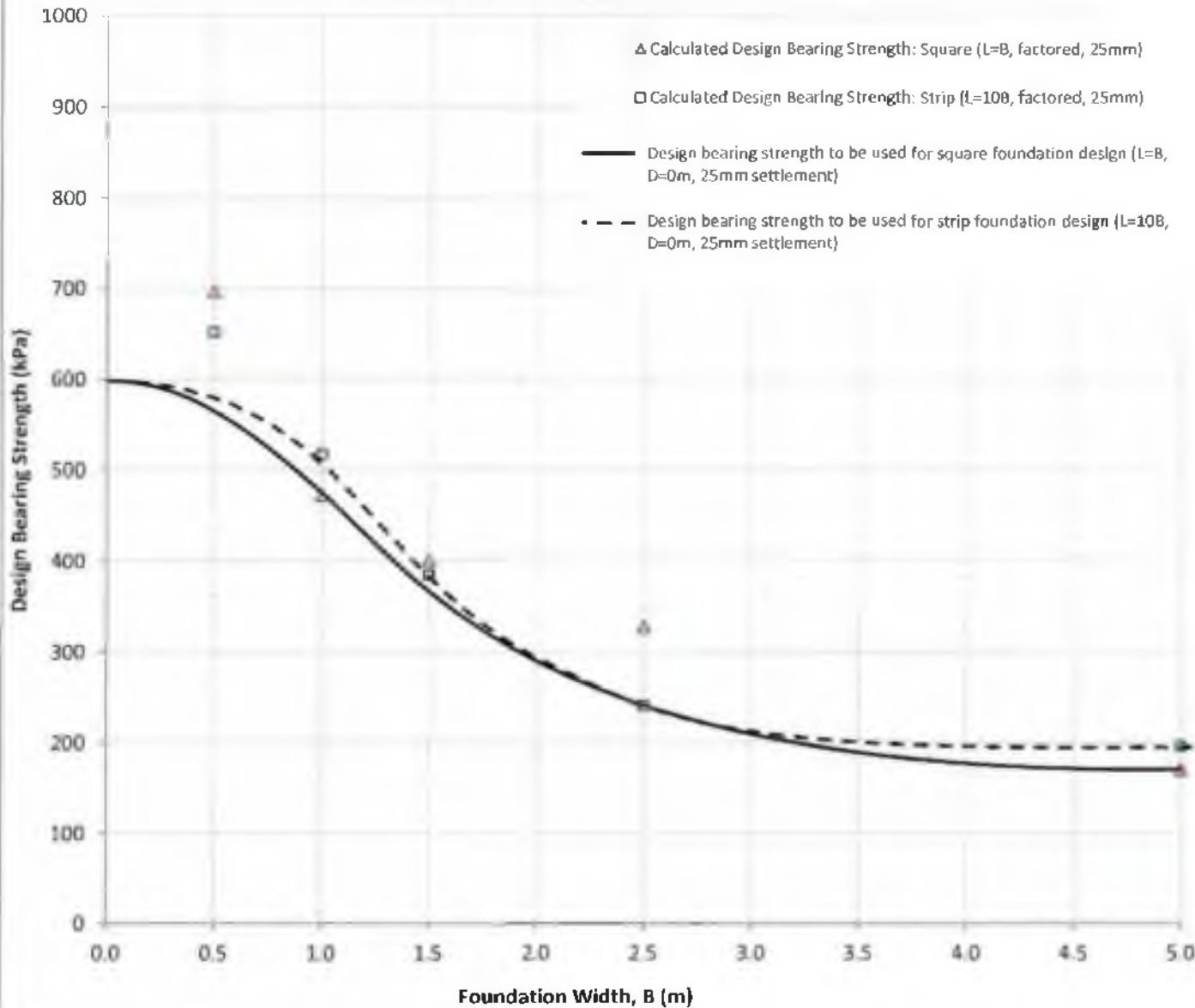
Date: 20 April 2012

Title: Bearing Capacity of Shallow Foundations

Eliot Sinclair
surveyors | engineers | planners

20. APPENDIX I – SETTLEMENT CONTROL OF SHALLOW FOUNDATIONS

SETTLEMENT CONTROL OF SHALLOW FOUNDATIONS



Notes:

Foundation depth, $D = 0\text{m}$ below existing GI.

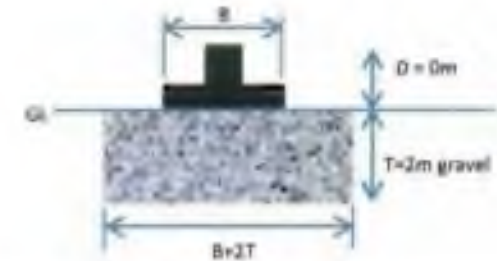
Assumes 2m depth of compacted AP65 hardfill to underside of foundation, with minimum width $B \geq 2T$ (m)

Duration = 50 years, max 25mm total settlement under non-seismic conditions, calculated using Schmertmann method.

Structural Engineer should also check bearing capacity of foundations, refer to 'Bearing Capacity of Shallow Foundations' chart.

Design bearing strength = $0.5 \times$ Ultimate bearing strength [in accordance with NZBC Verification Method BM1/VM4, Table 1, For bearing and passive earth pressure (all other load combinations)]

Design bearing strength at smaller foundation width has been conservatively limited as shown to take into account the presence of lower soil layers, or inferred geotechnical conditions indicated from other test results at an adjacent site.



Project: 12 Mary Muller Drive, Christchurch

Client: Castle Rock Properties Ltd.

Job #: 348392

Date: 20 April 2012

Title: Settlement control of shallow foundations

Eliot Sinclair
surveyors | engineers | planners

6782/W
19-12-00.

10010404

CHRISTCHURCH CITY COUNCIL
Rec'd 22 DEC 2000
Civic Offices
Application No.....

Mary Muller Place

**Proposed Printing Press
Development**

Geotechnical Report

REFERENCE NUMBER: 2164

DATE: December 2000

PREPARED FOR: ALAN REAY CONSULTANTS LTD

PREPARED BY: GEOTECH CONSULTING LIMITED

ENQUIRIES TO: Ian McCahon

1 Introduction

A new building to house a printing press and associated facilities is to be constructed on a site to be accessed off the end of Mary Muller Place in Woolston, Christchurch. This report outlines the site investigation and geotechnical issues to be addressed for this development.

The new building will be a warehouse 39m by 56m and an adjoining single level office 23m by 37m. The warehouse will contain two 17 tonne and one 35 tonne printing presses. An area at the north end will be used for storing paper and may carry floor loads of up to 25 kPa.

2 Site

The site will have access off the yet to be formed extension of Mary Muller Place. The site also has a frontage onto Port Hills Road. The ground is essentially level and is currently covered in long grass with large areas also covered in dumped used tyres. A fill stockpile extends into the northeast corner, and a depression partly filled with rubbish lies close to the northwest corner. The site is known to have been filled.

3 Site Investigation

Considerable subsurface investigations have been carried out on the site and the adjacent area. The known information consists of:

- Alan Reay Consultants Ltd, December 1997:
21 test pits to a maximum depth of 2.5m. 15 of these pits were on the site and 8 under the building footprint.
- Eliot Sinclair and partners Ltd, September 2000:
11 test pits to a maximum depth of 4m on the centre line of the extension to Mary Muller Place. Two pits (nos 10 and 11) were in the turning circle area at the end of the proposed road and are relevant to the site
- Geotech Consulting Ltd, December 2000:
6 test pits to 2.5m maximum depth, with limited SPT testing
5 Static Cone Penetration (CPT) Tests, to between 5.5 and 19m depth, under the building footprint.

The locations of the investigation bores are shown on the site plan, Figure 1, attached. The CPT test profiles and the Geotech Consulting borelogs are appended, along with copies of the ARC borelogs.

4 Subsurface Conditions

There is a liquefaction risk on the site. Most of the soil profile is silty and the soil grading is too fine to be very susceptible to liquefaction. However, there are thin layers of sand throughout the soil profile investigated, and most of them are loose enough to liquefy in a strong earthquake. The sand layer at about 4m deep is probably too deep to liquefy.

Liquefaction in an earthquake is unlikely to cause great effects at the ground surface, but would probably lead to some ground distortion and settlement, of perhaps 30 to 50mm.

5.4 Soil Properties

The soils are essentially cohesionless sands and silty sands. Soil properties suitable for design are:

	Density (kN/m ³)	Angle of internal friction	Cohesion (kPa)
Fill	17	27	0
Silts and Sands 2 – 3.5m	18	28	0
Sand below 3.5m	18	30	0

5.5 Groundwater

The depth to the groundwater measured is assessed as 1.2 to 1.5m.

6 Foundations

6.1 Foundation Options

The site is geotechnically constrained by the presence of 1.5 to 2m of variable and loose fill, overlying a deep sequence of soft silts and loose silty sands. The scale of the development does not necessarily warrant extensive ground improvement, and construction over the fill is possible provided that the associated risks are fully appreciated.

The options include:

- Ground improvement by excavating the fill, removing all the unsuitable material and backfilling with the better quality material supplemented with imported hardfill.
- Use of piles to support the main load bearing foundations
- Use of shallow foundations throughout.

In many respects the best option is to excavate and replace the fill. This removes much of the risk and allows shallow footings to be used throughout. However it would be an expensive and time consuming operation.

Shallow foundations on the existing fill would be subject to potentially damaging differential settlement because of the variability in the fill, and would have to be

	Capacity (kN)	Friction (kN)	Capacity (kN)
0.2	8	5	3
0.3	18	7	11
0.4	30	9	21
0.5	45	11	34
0.6	60	13	47
0.8	100	18	82
1.0	150	22	128

Driven piles could be used. There is little advantage in driving them deeper than the sand layer at between 2.7 and 4m depth, as this is the densest layer found in the CPT tests to between 12 and 19m depth. The sand layer is thin – generally about 1m thick, and only small section piles could be used in order not to overstress the bearing layer. Care would be needed not to overdrive the piles.

Table 6.2 Ultimate Bearing Capacities for Short Driven Piles

Pile Size (m)	Ultimate bearing Capacity (kN)	Negative Skin Friction (kN)	Net Ultimate Capacity (kN)
0.15	60	5	55
0.275	200	10	190

The ultimate capacities for either the bored piles or the driven piles should be reduced by a bearing capacity reduction factor to give values of “allowable ultimate” bearing stresses to be used with fully factored loads in accordance with NZS 4203:1992. A capacity reduction factor of 0.5 should be used for all load combinations except those including earthquake overstrength when a value of 0.8 is applicable.

Installation of piles may be straightforward in some areas and difficult in others due to debris in the fill. It is recommended that driven piles are predrilled through the fill to clear any obstructions.

6.4 Shallow Footings

As noted above, shallow pad and strip foundations are not recommended for the proposed development on this site because of their vulnerability to settlement damage.

Footings for light structures for which settlement is acceptable should be sized on an allowable bearing pressure of not more than 50 kPa to minimise the possible settlement and vulnerability to local variation in the fill.

6.5 Settlement

The proposed development incorporates floor areas which are likely to carry significant loads.

The paper store may carry up to 25 kPa loading. An estimate of settlement of an area 39m by 16m at 25 kPa is 50 to 70mm. The bulk of this settlement is in the underlying

8 Limitations

The subsurface conditions and the interpretations reported are those identified at the locations of the investigations at the time of the investigation and are subject to the limitations of the investigation methods.

The borelogs are an engineering/geological interpretation of the subsurface conditions dependent on the method and frequency of sampling and testing. The boreholes represent only a very small sample of the total subsurface soils. The interpretation of the information and its application must take into account the spacing of the boreholes, the frequency of sampling and testing and the possibility of undetected variations in soils between the boreholes.

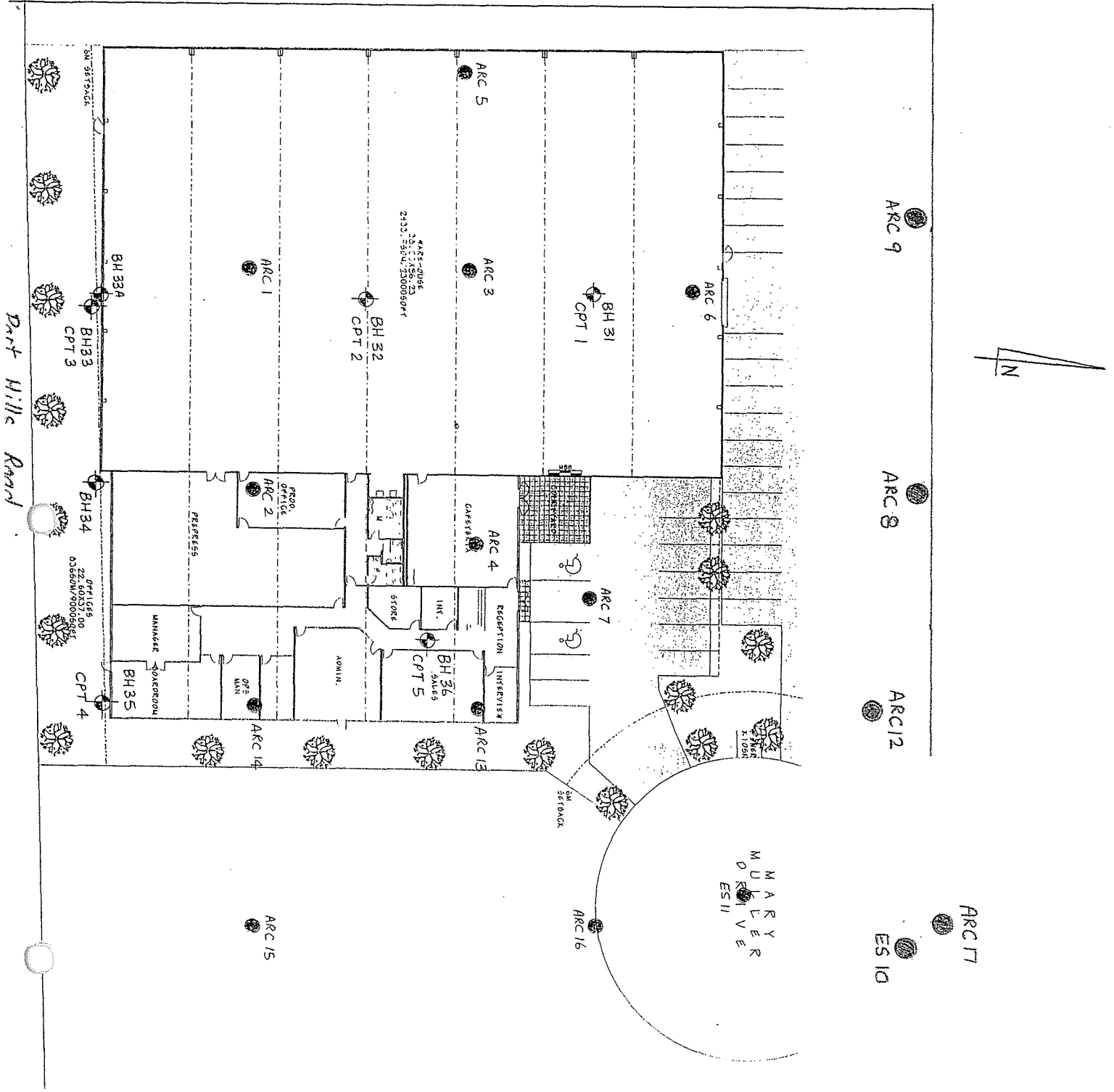
While care has been taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical aspects and recommendations or suggestions for design and construction, Geotech Consulting Ltd cannot anticipate or assume responsibility for unexpected variations in ground conditions or the actions of contractors. If conditions encountered on site during construction appear to vary from those which can be expected from the information contained in this report, Geotech Consulting Ltd requests that it be notified immediately.

This report has been prepared for the proposal as outlined in the introduction and the information and interpretation may not be relevant if the proposed development is changed. If the form and details of the proposed development are changed, Geotech Consulting Ltd will be pleased to review the report and the sufficiency of the investigation and appropriateness of the recommendations.

This report has been prepared solely for the benefit of Alan Reay Consultants Ltd and the Christchurch City Council. No liability is accepted by this Company or any employee or sub-consultant of this company with respect to its use by any other person.

This disclaimer shall apply notwithstanding that the report may be made available to other persons for an application for permission or approval or to fulfil a legal requirement.

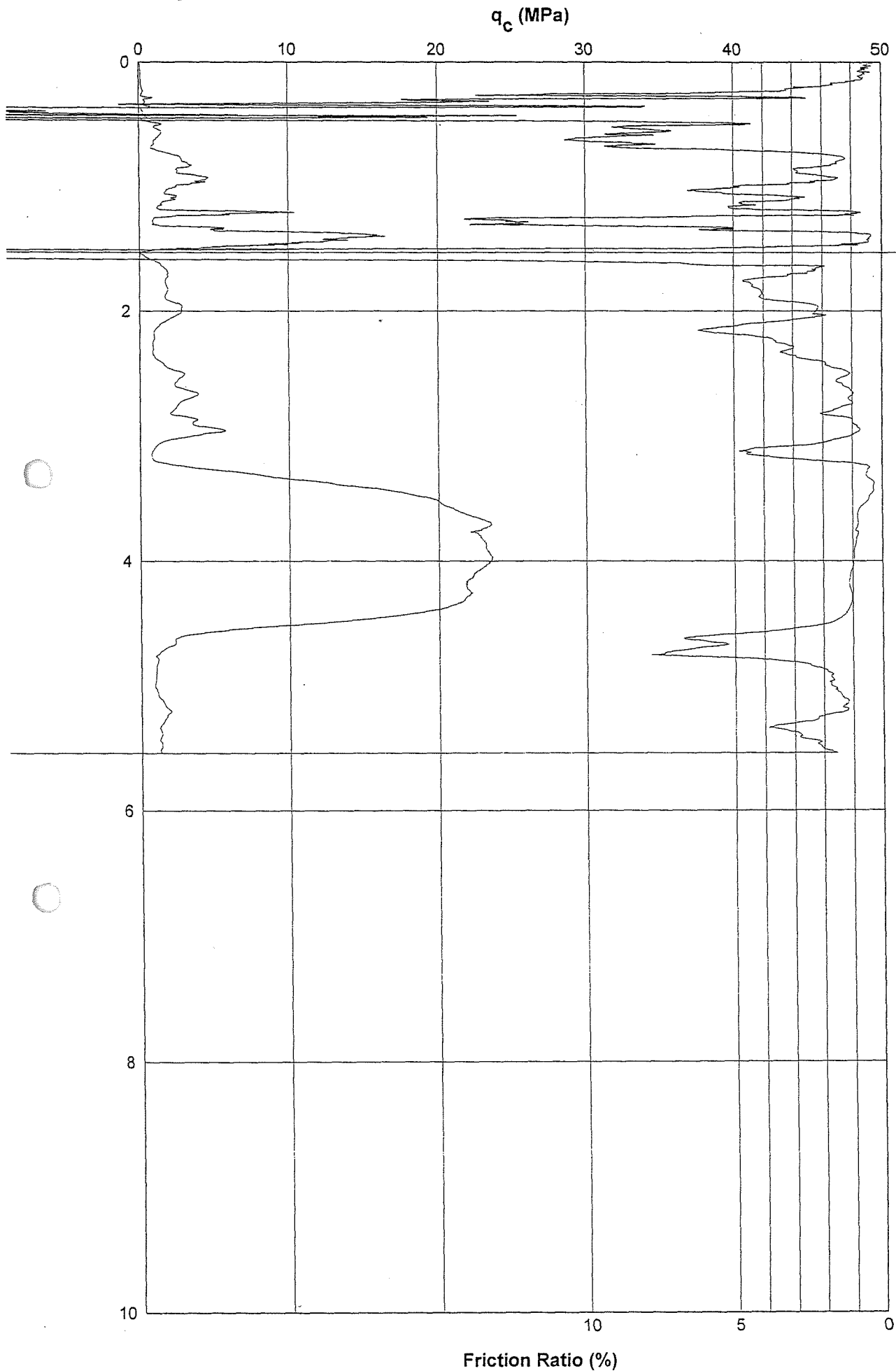
Geotech Consulting Ltd
Ian McCahon

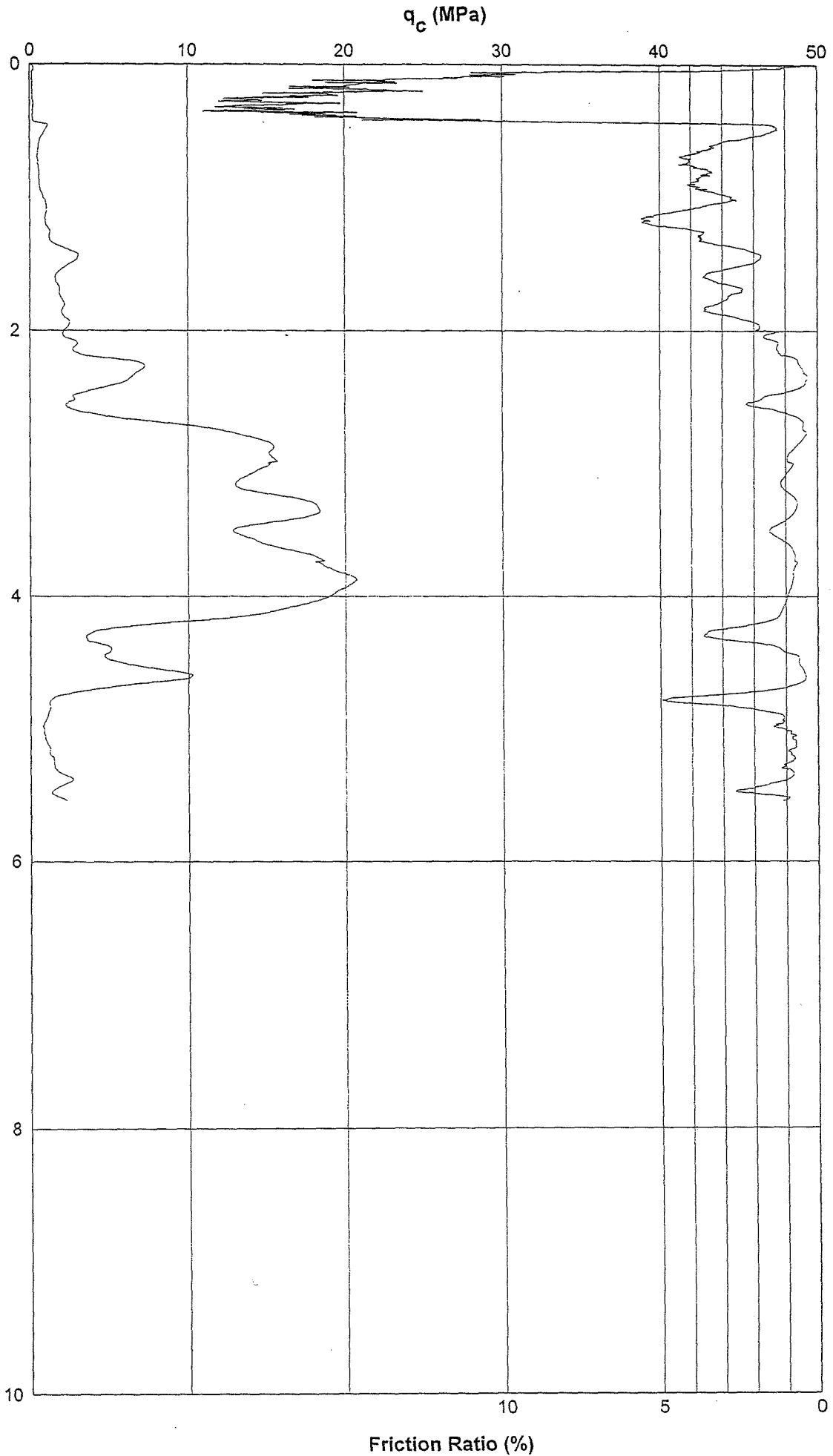


SITE PLAN

Test Pit Locations approximate only
 Transferred from location plans by
 ARC & ES.

FIGURE 1







GEOTECH

SHALLOW BOREHOLE LOG GEOTECH CONSULTING LTD

Hole No: **BH 31**

Job No: **2164**

Logged by: **imcc**

Date drilled: **12/6/00**

Checked by:

Date checked: **12/16/00**

Project: **Mary Muller Place**

Client: **Alan Reay Consultants Ltd**

Hole location: **North end of warehouse**

Driller: **Texco**

Contractor: **Texco**

Equipment: **Machine Auger**

R.L.:

Max depth: **2.00**

Notes:

Depth (m)	STRATA DESCRIPTION	USCS	Graphic Log	Water Table	S.P.T		SCALA PENETROMETER								
					50	100	34	50	100	150					
0.0	Brown Gravel SILT - FILL gravel round to angular, fine to medium 12mm dia steel rod @ 0.3m		FILL												
0.5	dark grey-black Sandy Silt - ash @ 0.6m lenses of wet, green/blue sand. Strong smell of tar		FILL												
1.0	SPT, N=2.5		FILL												
1.5	dark brown Peat / organic silt		PT												
2.0	grey SILT, some rootlets SPT, N = 10		ML												
2.5															
3.0															
3.5															
4.0															
4.5															
5.0															

FILL

PT

ML

USCS

Graphic Log

Water Table

S.P.T

SCALA PENETROMETER

50 100

34 50 100 150

0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5 5.0

2.4

10.0

X

X

X

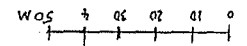
X

X

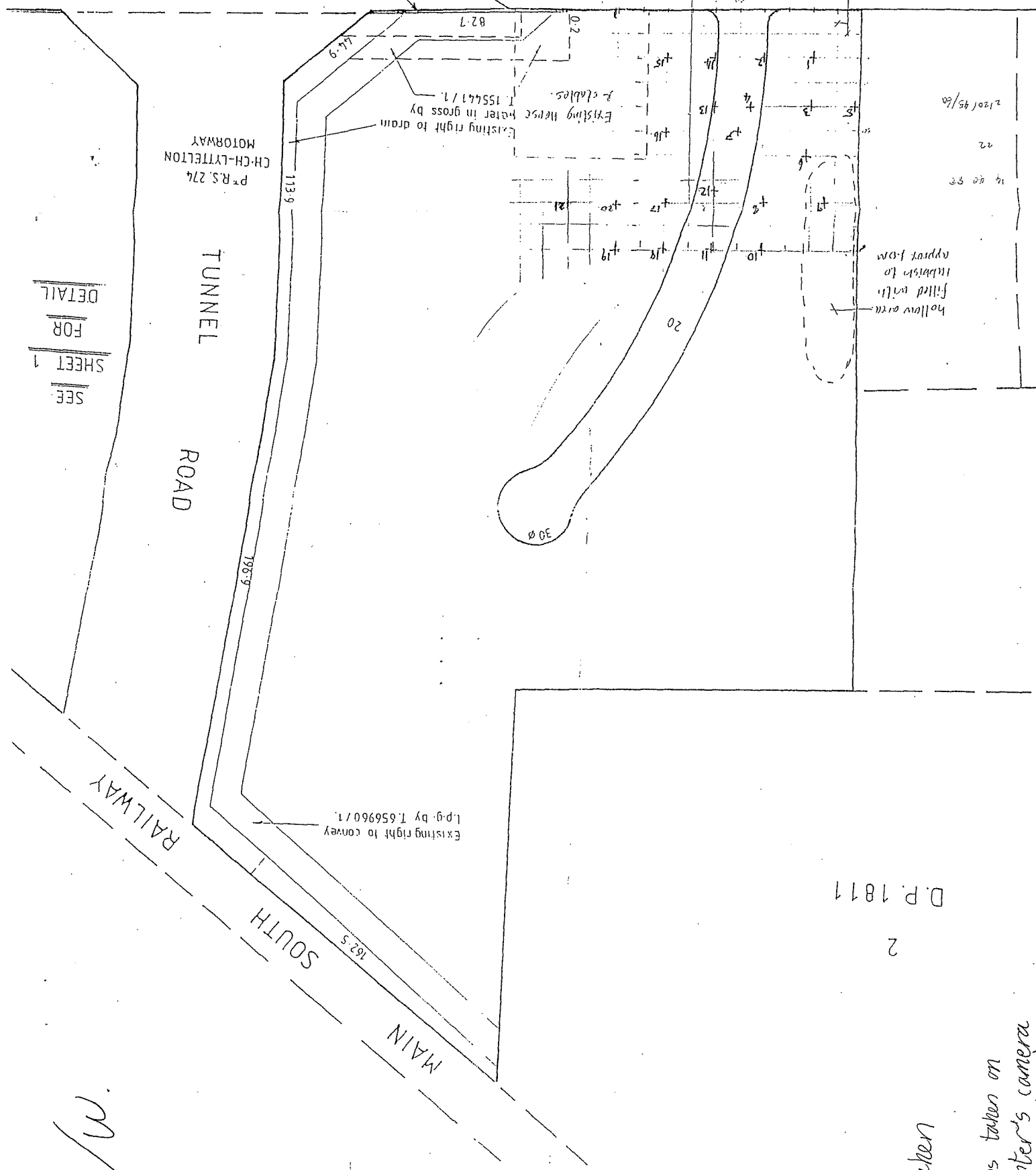
X

X

SCALE 1:1500



ORT HILLS ROAD (SH. 74)



SEE SHEET 1 FOR DETAIL

2120145/50
22
14 00 33
filled with approx. form
hollow area

D.P. 1811

2

6152/W.

Photos taken 3/12/97. - undated photos taken on Peter Parmenter's camera (no. hmc. no. 41.000)

SITE INVESTIGATION

PROJECT: PROPOSED SUBDIVISION OF PT. R.S. 274

INST DIGGER-900 wide.

FILE 6152.

DATE 4/12/97.

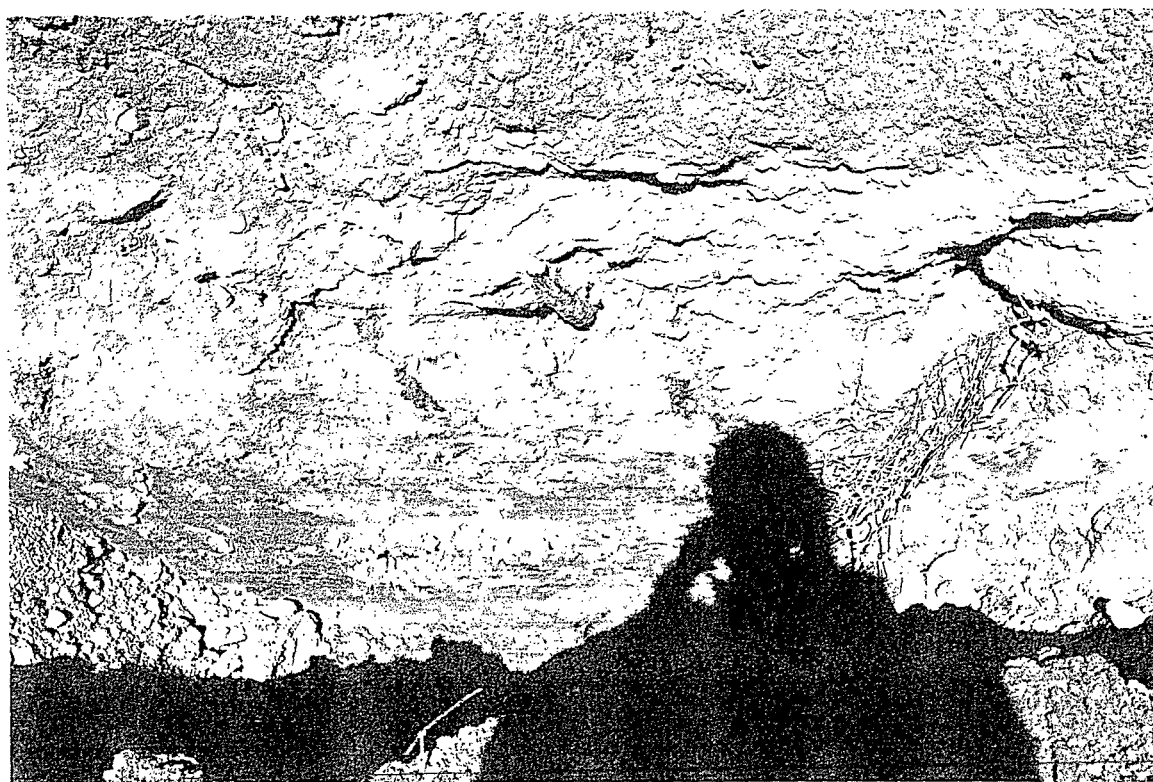
DEPTH GL= 0.0	HOLE No. 21.	
	Gravelly silt.	
0.5	Silt with some building rubble and rusted steel.	0.5
1.0	becoming grey/brown silt.	1.0
1.5		1.5
2.0		2.0
2.5		2.5

NOTES.

Density test of fill material at 0.9 m.

Dry Density = 1465 kg/m³.

% Moisture = 20-7%



SITE INVESTIGATION

PROJECT: PROPOSED SUBDIVISION OF PT. R.5.274

INSTR	DIGGER - 900 wide
FILE	6152.
DATE	4/12/97.

DEPTH GL= 0.0	HOLE No. 20.	
0.5	Gravelly silt.	0.5
1.0	1 silt - layers of black ash and some building rubble.	1.0
1.5	becoming uniform silt. (grey brown).	1.5
2.0		2.0
2.5		2.5

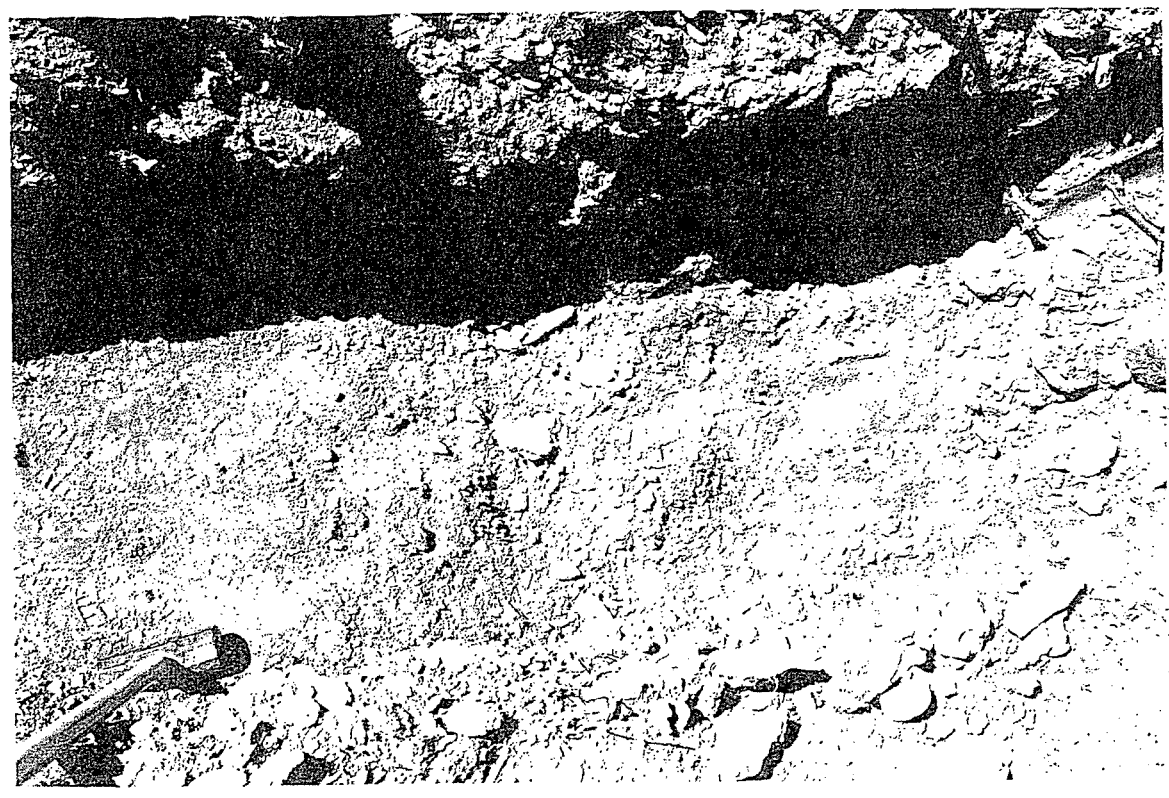
NOTES.
 Density test of fill material
 at 1.1M.
 Dry Density = 1939 kg/m³.
 % Moisture = 11.8%



SITE INVESTIGATION
 PROJECT: PROPOSED SUBDIVISION OF PT. R.S.274

DEPTH GL= 0.0	HOLE No. 19.	
	Firm Gravelly Silt.	
0.5	silt, some stones & building rubble.	0.5
1.0	limit of trench. solid layer of brick prevented further excavation	1.0
1.5		1.5
2.0		2.0
2.5		2.5

NOTES.
 No density test.



SITE INVESTIGATION

PROJECT: PROPOSED SUBDIVISION OF PT. R.S. 274

INST	DIGGER - 900 wide
FILE	6152
DATE	4/12/97

DEPTH GL= 0.0	HOLE No. 18	
		Gravelly silt.
0.5		layers of fill, - Gravelly silt with mix of building rubble and metal scraps - greenish sandy silt.
1.0		
1.5		layers of fill - black ash & silt mixed with building rubble.
2.0	$\frac{\nabla}{3}$	becoming silt. water table?
2.5		

NOTES.

Density test of fill material
at 1.3 M. (green sandy/silt clay)

Dry Density = 1098 kg/m³.
% Moisture = 24.6%



SITE INVESTIGATION

PROJECT: PROPOSED SUBDIVISION OF PT. R.S. 274

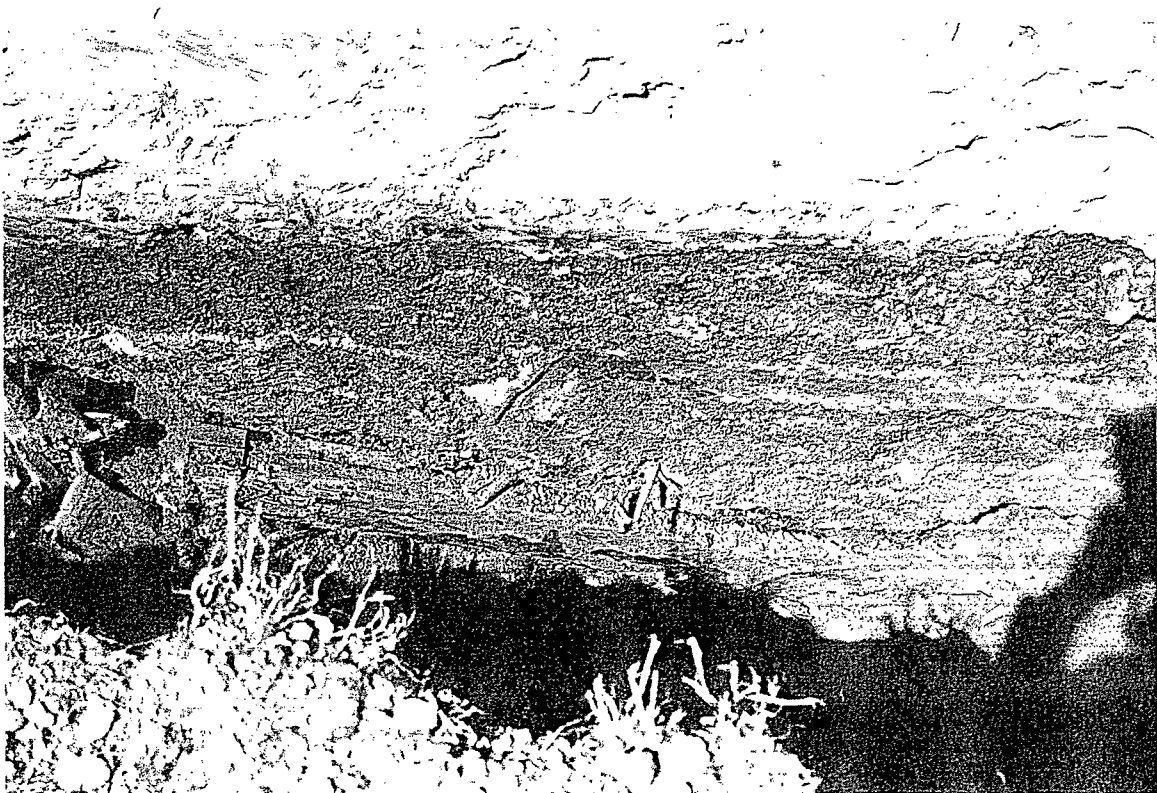
INST	DIGGER - 900 wide
FILE	6152.
DATE	4/12/97.

DEPTH GL= 0.0	HOLE No. 17
	Gravelly-silt - firm.
0.5	layered black ash & silty fill.
1.0	
1.5	fill with high % of rotten timbers & building rubble.
	fill.
2.0	wet fill water table?
	wet silt (below water table).
2.5	

NOTES.

Density test of fill material at base of rotten timber layer at 1.5M

Dry Density = 1166 kg/m³
% moisture = 30.0%



SITE INVESTIGATION

PROJECT: PROPOSED SUBDIVISION OF PT. R.S. 274

INST DIGGER-900 wide

FILE 6152.

DATE 4/12/97.

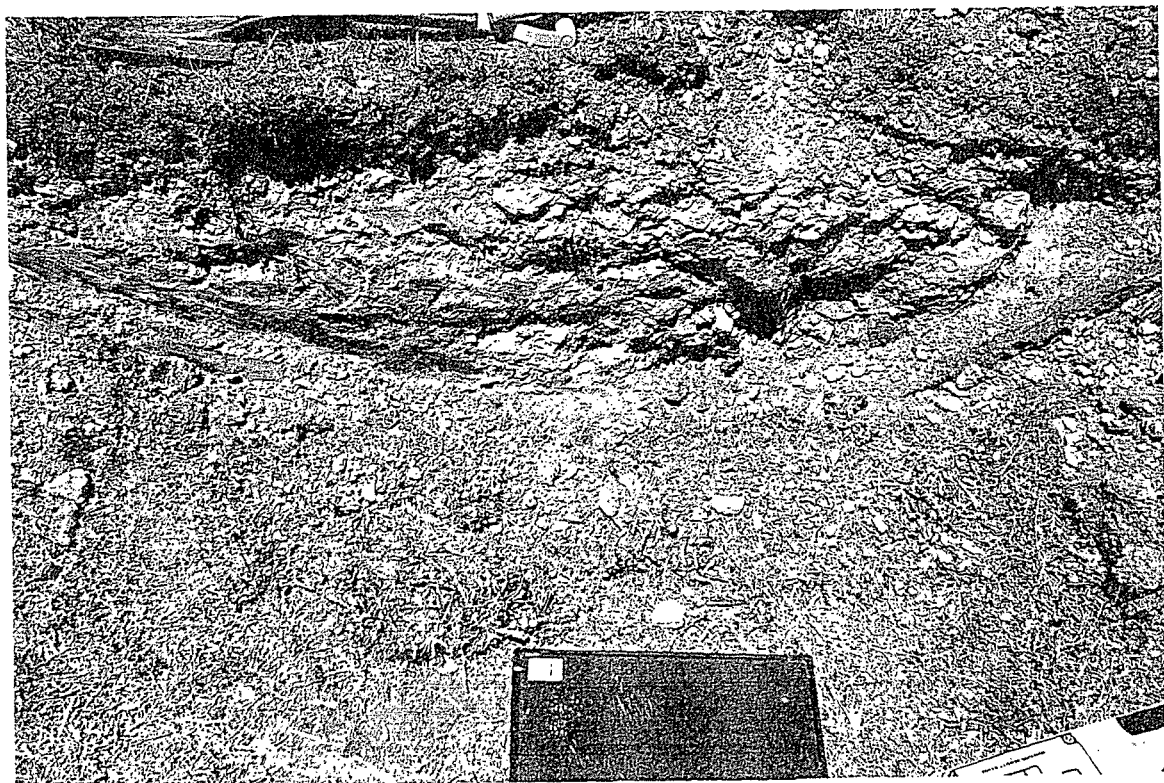
DEPTH GL= 0.0	HOLE No. 16.
0.5	Gravelly silt fill occasional building rubble.
1.0	black ash layer
1.5	Gravelly silt fill + building rubble including corrugate iron.
2.0	
2.5	

NOTES.

Density test of fill material
at 1.0M.

Dry Density = 1604 kg/m³.

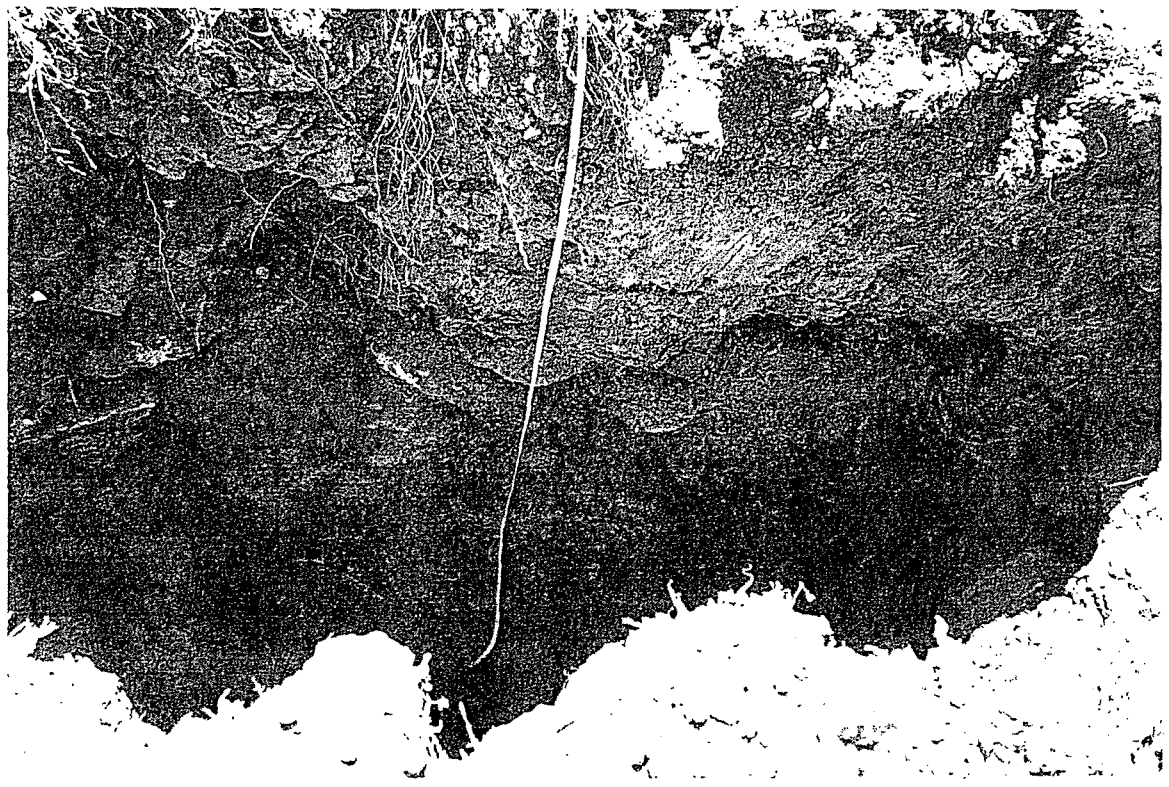
% Moisture = 17.0%



SITE INVESTIGATION
 PROJECT: PROPOSED SUBDIVISION OF PT. R.S. 274

DEPTH GL= 0.0	HOLE No. 15	
	Gravelly silt fill.	
0.5	layer of concentrated building rubble.	0.5
1.0	Gravelly silt fill with thin layers of black ash	1.0
1.5		1.5
2.0	becoming generally uniform silt. water table at approximately 1-8M.	2.0
2.5		2.5

NOTES.
 No Density test.





PAGE	
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FILE	6152
DATE	4/12/97

PROJECT: PROPOSED SUBDIVISION OF PT. R.S. 274

SITE INVESTIGATION

DEPTH HOLE NO. 14

NOTES.
 Density test of natural material (below fill level) at 1.7m
 Dry Density = 1383 kg/m³
 % Moisture = 30.5%

2.5			
2.0	damp generally uniform silt.		
1.5	silt occasional building rubble material.		
1.0	layer of ash/silt mix.		
0.5	silt plus some fill material small ash deposits.		
0.0	hard gravelly silt.		

SITE INVESTIGATION

PROJECT: PROPOSED SUBDIVISION OF PT. R.S. 274

INST DIGGER-900 wide

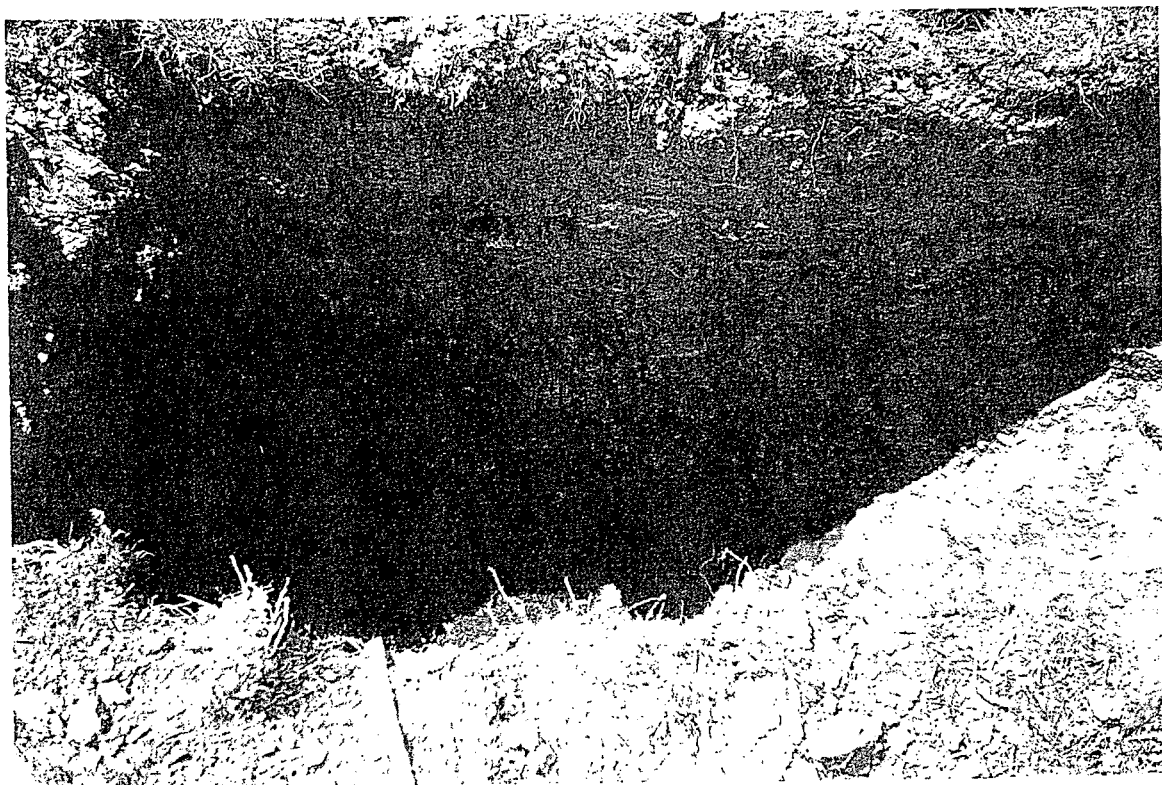
FILE 6152.

DATE 4/12/97.

DEPTH GL= 0.0	HOLE No. 13.	
0.5	Gravelly silt mixed with occasional small layers of black ash and building rubble	0.5
1.0		1.0
1.5		1.5
2.0	becoming uniform silt.	2.0
2.5		2.5

NOTES.

- No density test.



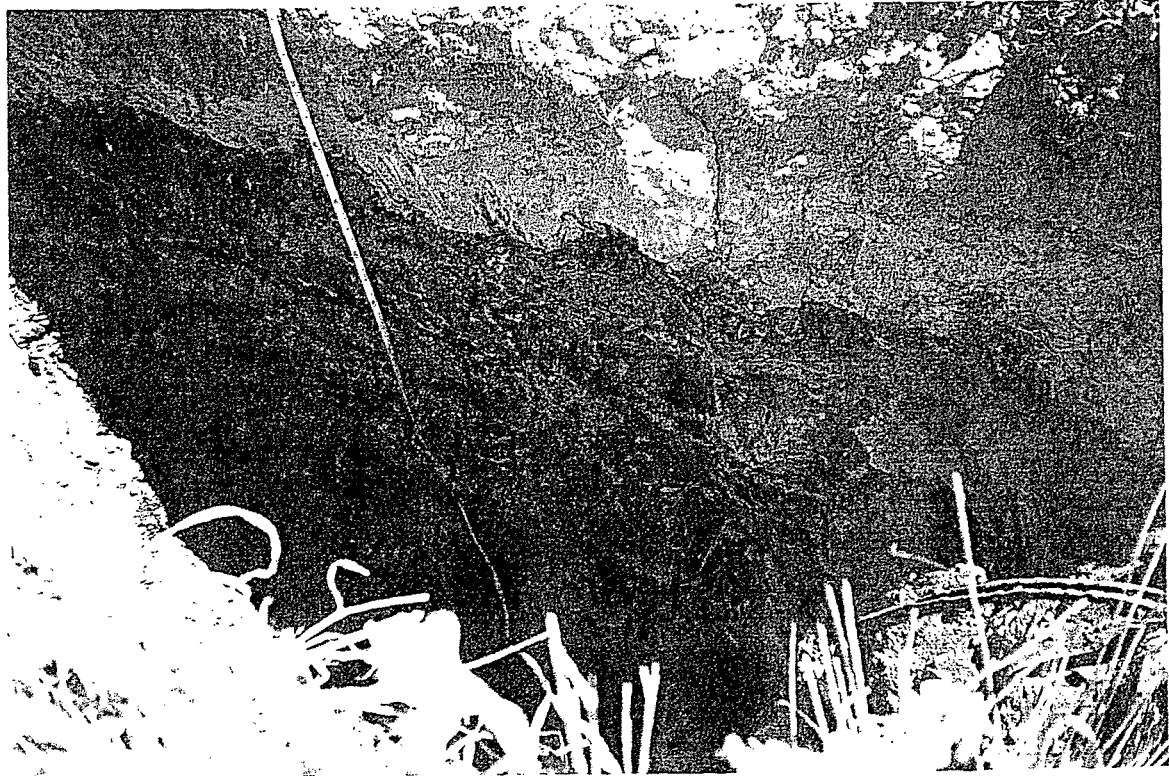
SITE INVESTIGATION

PROJECT: PROPOSED SUBDIVISION OF PT. R.S.274

DEPTH GL= 0.0	HOLE No. 12.	
	Gravelly silt - Dry.	
0.5	Layers of Varying thickness of black ash type material & gravelly silt fill mixed with occasional brick and woodpieces (<0.1%)	0.5
1.0		1.0
1.5		1.5
2.0	Becoming uniform silt. water table at approximately 1.8m.	2.0
2.5		2.5

NOTES.

Density test of fill layers at 1-1m
 Dry Density = 1251 kg/m³
 %Moisture = 31.5%



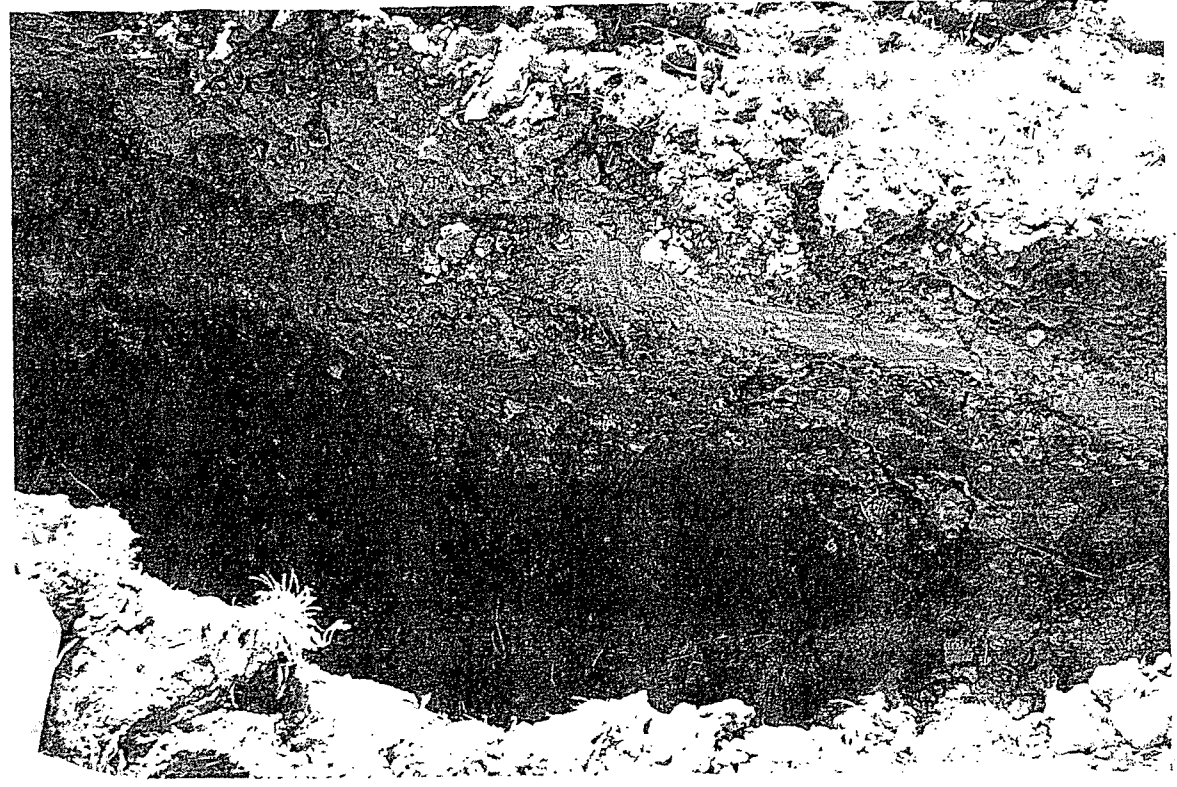
SITE INVESTIGATION

PROJECT: PROPOSED SUBDIVISION OF PT. R.S.274

DEPTH GL= 0.0	HOLE No. 11
0.5	Gravelly silt.
0.5	Gravelly silt fill including some building rubble.
1.0	50-200 thick black ash layer.
1.0	silt/building rubble. some ash.
1.5	
2.0	becoming uniform silt.
2.5	

NOTES.

Density test of underlying silt. at 1.7M
 Dry Density = 1165 kg/m³.
 % Moisture = 23.9%



SITE INVESTIGATION

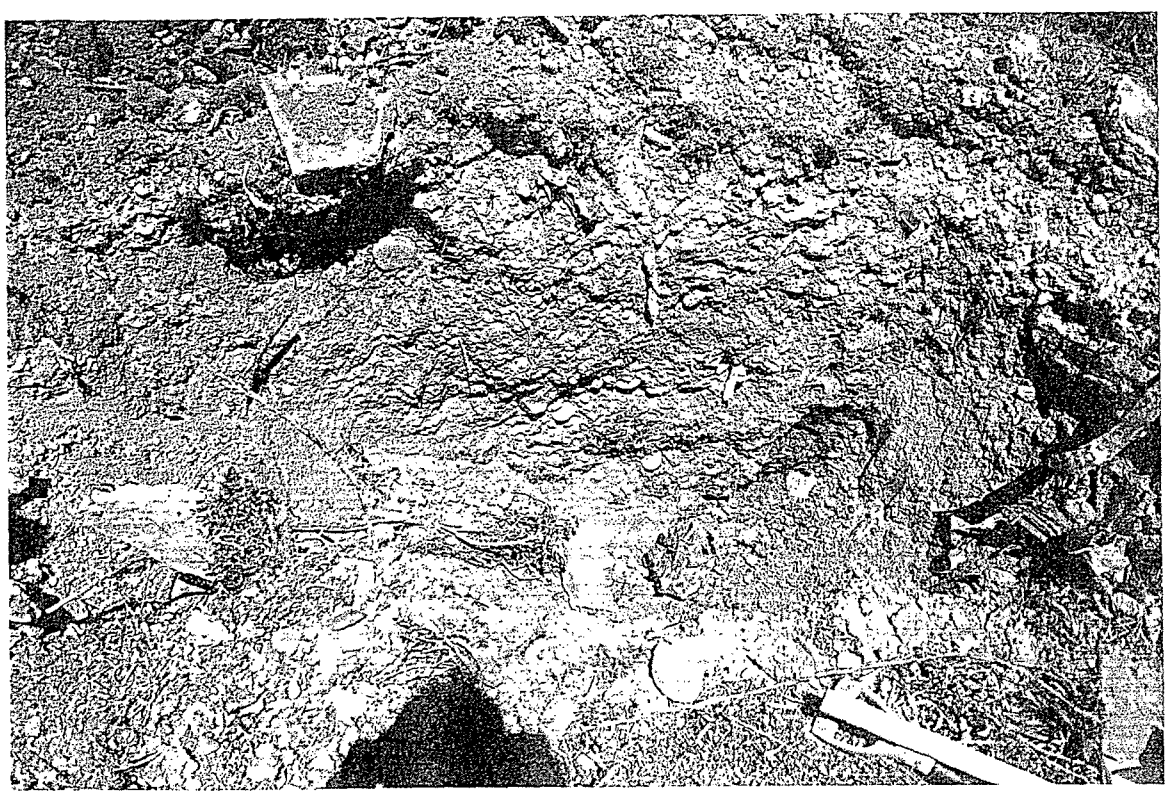
PROJECT: PROPOSED SUBDIVISION OF PT. R.S. 274

INST	DIGGER - 900 wide
FILE	6152.
DATE	4/12/97.

DEPTH GL= 0.0	HOLE No. 10.
0.5	Gravelly silt hard top layer Gravelly silt mixed with building rubble - concrete/bricks
1.0	layers of stoney fill & black ash material - some wood (<0-1%). - some inorganic refuse.
1.5	
2.0	becoming uniform silt.
2.5	

NOTES.

Density test of fill material
at 1-2M.
Dry Density = 1551 kg/m³
% Moisture = 13.1%



SITE INVESTIGATION

PROJECT: PROPOSED SUBDIVISION OF PT. R.S. 274

INST	DIGGER-900 wide
FILE	6152.
DATE	4/12/97.

DEPTH GL= 0.0	HOLE No. 9.	
0.5	Refuse including iron, wire, wood, organics to 800-1000 mm depth.	0.5
1.0	<i>W O X W O X W X W X</i> Gravelly silt laced with tree roots.	1.0
1.5	<i>O X O O X O X X O O X O X O X O X O X X O X X O X X O X X O X X O X X O</i> Generally even slightly gravelly silt fill small amount of brick material. ↓ becoming uniform silt	1.5
2.0	<i>X X</i>	2.0
2.5		2.5

NOTES.

Density test of fill at 1.2 m

Dry Density = 1791 kg/m³

% Moisture = 10.0%

- Natural hollow region near boundary, approximate area shown on trench location plan.



SITE INVESTIGATION
 PROJECT: PROPOSED SUBDIVISION OF PT. R.S. 274

DEPTH GL= 0.0	HOLE No. 8.
	Gravelly silt.
0.5	silt fill, some building refuse & rubble two noticable but thin layers of black ash material
1.0	
1.5	
2.0	becoming silt. uniform.
2.5	

NOTES.

Density test of fill material at 1.0m.
 Dry Density = 1428 kg/m³
 % moisture = 14.5%.

No Photograph.

SITE INVESTIGATION

PROJECT: PROPOSED SUBDIVISION OF PT. R.S. 274

INST DIGGER-900 wide

FILE 6152

DATE 4/12/97

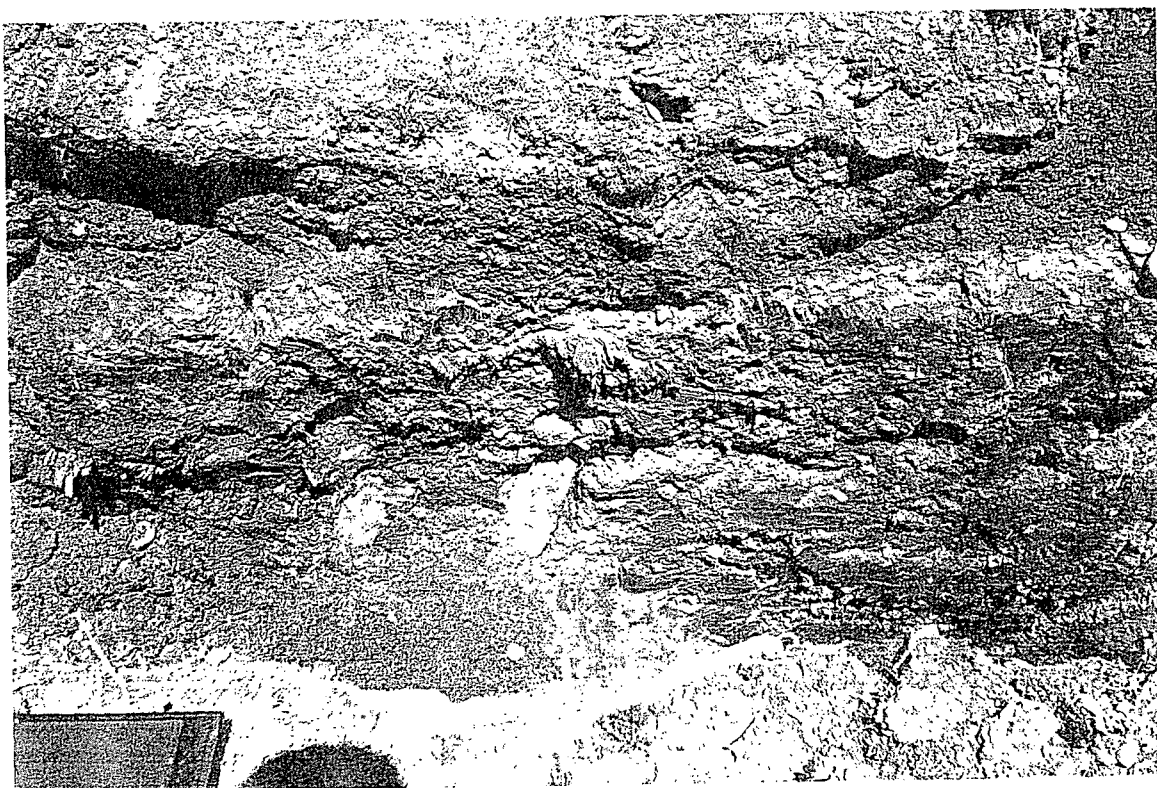
DEPTH GL= 0.0	HOLE No. 7.	
		Gravelly silt - Dry.
		Black Ash layer
0.5		silt/clay fill mixed with black ash, concrete & brick with small amount of refuse.
1.0		
1.5		
2.0		bluish clay/silt
2.5		

NOTES.

Density test of fill at 1.1m.

Dry Density = 1521 kg/m³.

% Moisture = 21-3%



SITE INVESTIGATION

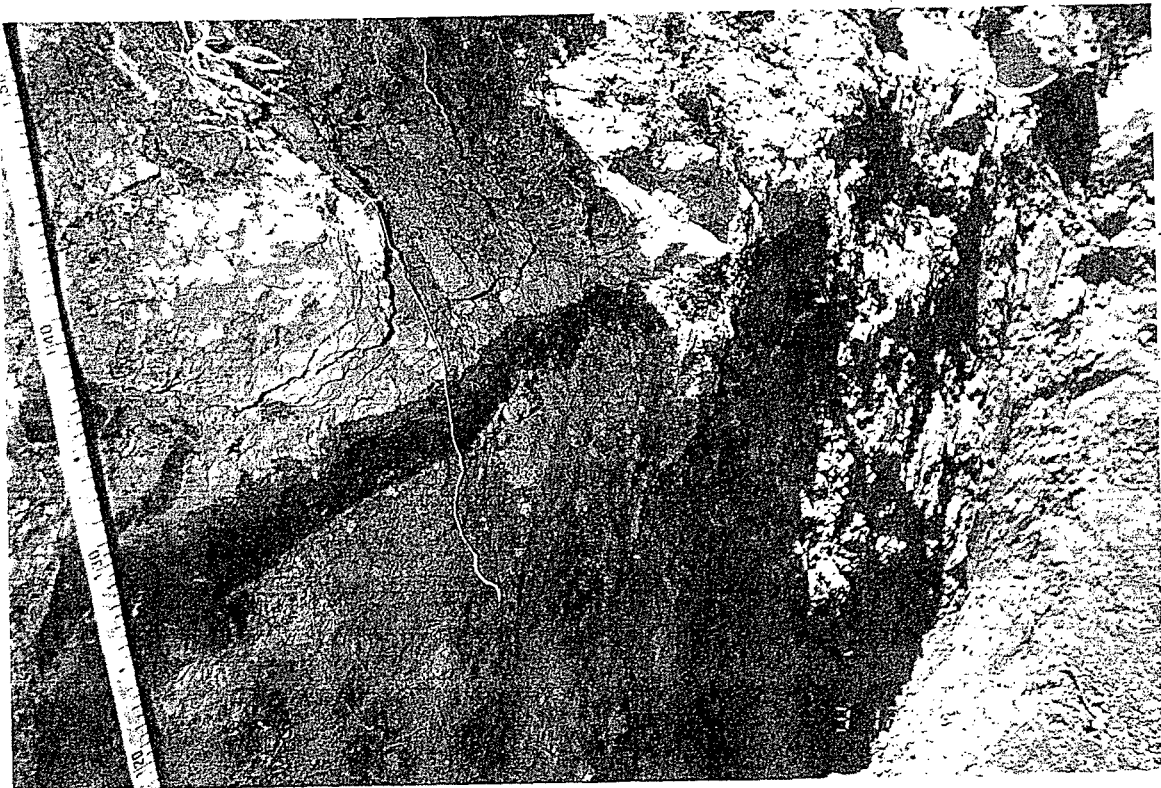
PROJECT: PROPOSED SUBDIVISION OF PT. R.S. 274

INST	DIGGER-900 wide b
FILE	6152.
DATE	14/12/97.

DEPTH GL= 0.0	HOLE No. 6.	
	scrub, organics plus some refuse.	
	Gravel silt.	
0.5	black Ash layer.	0.5
1.0	Gravel silt fill with mixture of hard fill tailings some organics (tree roots) < 0.1%.	1.0
1.5		1.5
2.0	becoming bluish silt/clay	2.0
2.5		2.5

NOTES.

Density test of fill material at 1.6m
 Dry Density = 12.97
 % moisture = 32%



SITE INVESTIGATION

PROJECT: PROPOSED SUBDIVISION OF PT. R.S. 274

PAGE	
INST	DIGGER-900 wide-b
FILE	6152.
DATE	4/12/97.

DEPTH GL= 0.0	HOLE No. 5 (boundary)
	Gravelly silt.
0.5	silt fill small amount of building rubble
1.0	slightly gravelly silt ↓becoming uniform silt.
1.5	
2.0	
2.5	

NOTES.

Density test at 1.0m
 Dry Density 1732 kg/m³
 % Moisture 12.7%

No photograph.

SITE INVESTIGATION

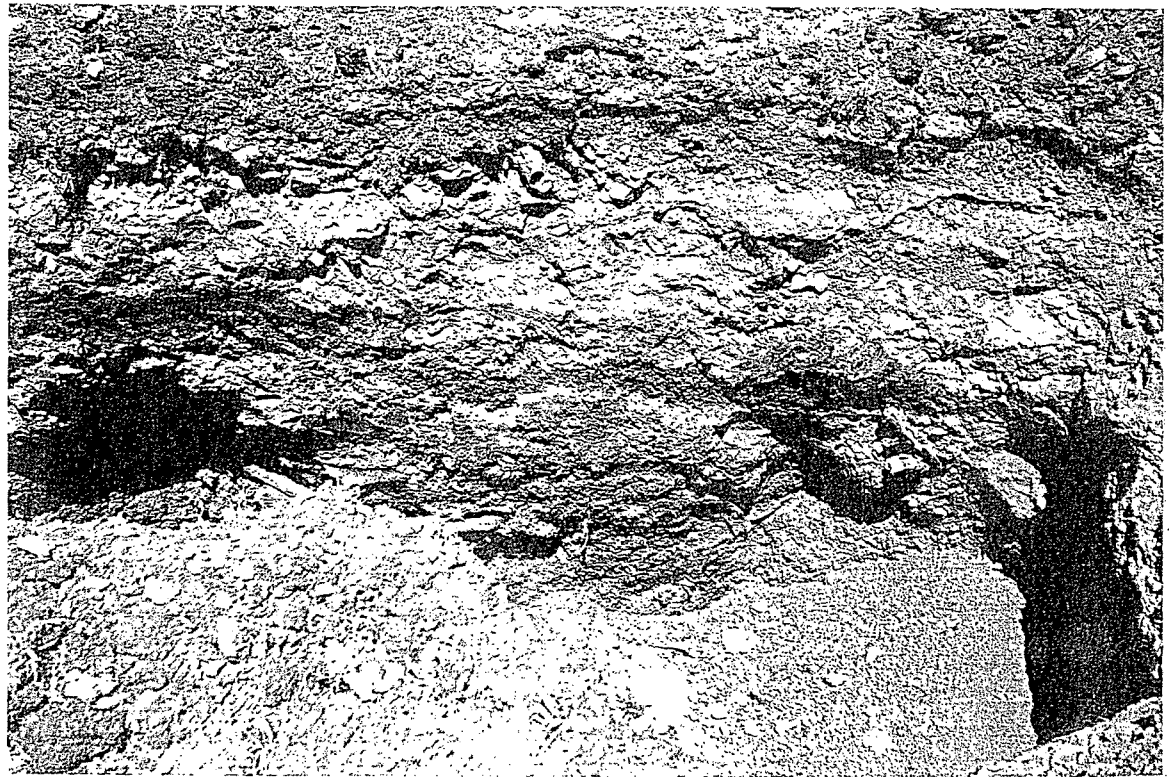
PROJECT: PROPOSED SUBDIVISION OF PT. R.S. 274

INST	DIGGER - 900 wide
FILE	6152.
DATE	4/12/97.

DEPTH GL= 0.0	HOLE No. 4.	
0.5	stoney silt. / Gravelly silt.	0.5
1.0	layers of varying thickness of black ash type material and gravelly silt fill mixed with building rubble including brick and wood. (< 0.1%)	1.0
1.5		1.5
2.0	becoming generally uniform silt.	2.0
2.5		2.5

NOTES.

Density test of fill material
at 1.0M.
Dry Density = 1022
% Moisture = 33.7%



SITE INVESTIGATION

PROJECT: PROPOSED SUBDIVISION OF PT. R.5.274

INST DIGGER-900 wide

FILE 6152.

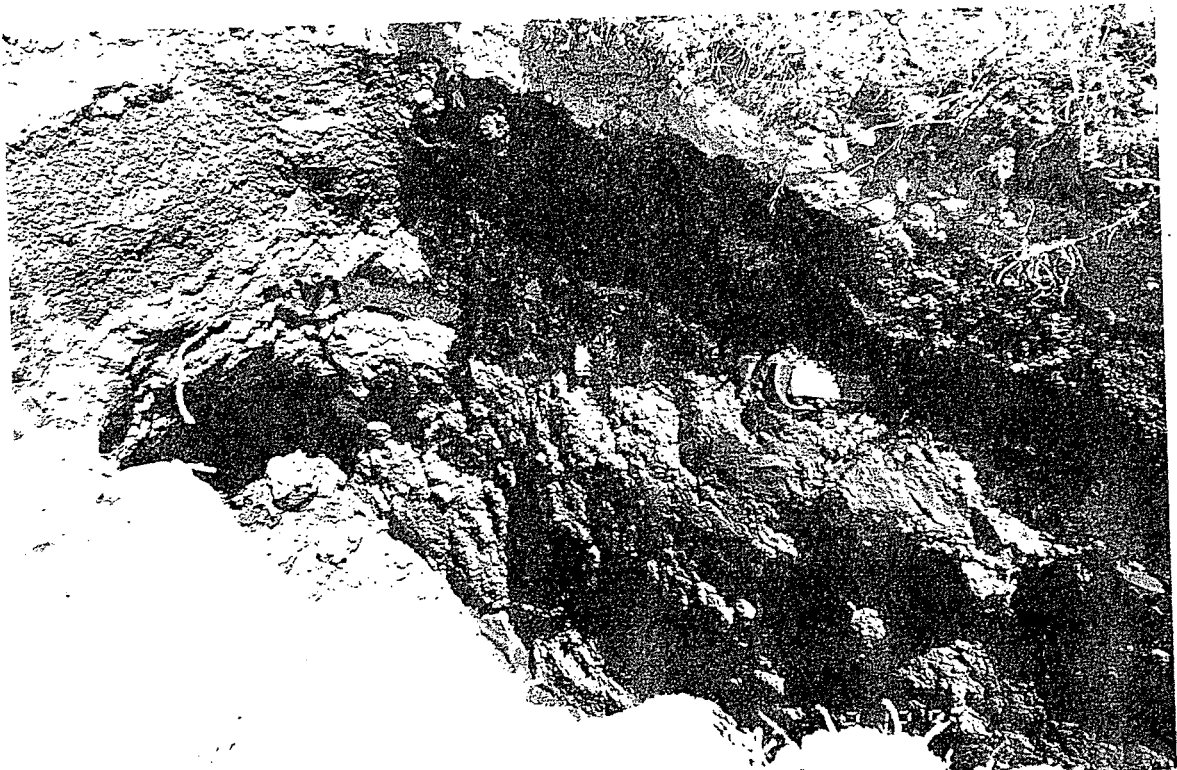
DATE 4/12/97.

DEPTH GL= 0.0	HOLE No. 3.
	Gravelly silt - Dry.
	black ash layer
0.5	Gravelly silt mixed with black ash, bricks, concrete fill and thin layers of black ash material.
1.0	
1.5	
2.0	becoming generally uniform silt. 1750 approx. water level.
2.5	

NOTES.

Density test of fill material at 1.4M.

Dry Density = 1422 kg/m³.
% moisture = 30.2%



SITE INVESTIGATION

PROJECT: PROPOSED SUBDIVISION OF PT. R.S. 274

INST DIGGER-900 wide

FILE 6152.

DATE 4/12/97.

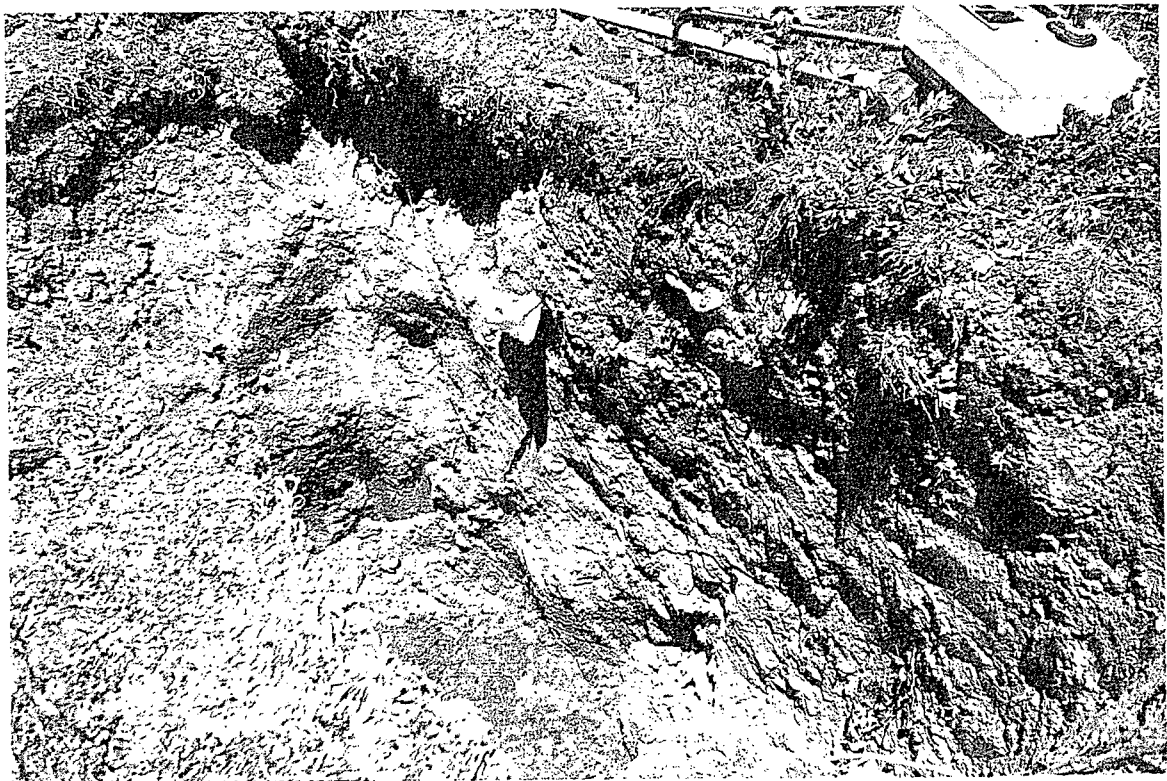
DEPTH GL= 0.0	HOLE No. 2.
	Gravelly silt.
0.5	Gravelly silt with small layers of black ash and mix of building rubble.
	approx 150 thick layer of black ash. → 200
1.0	Silt mixed with some building rubble & black ash.
1.5	
2.0	becoming uniform silt.
2.5	

NOTES.

Density test of underlying silt at 1-7M

Dry Density = 1343 kg/m³.

% moisture = 41.1%



SITE INVESTIGATION

PROJECT: PROPOSED SUBDIVISION OF P^T. R.5.274

INST	DIGGER - 900 wide
FILE	6152.
DATE	4/12/97.

DEPTH GL= 0.0	HOLE No. 1.	
	x 0 x o o x y o x 5 x x o	
0.5	Firm Gravelly silt.	
1.0	Gravelly silt mixed with concrete tailings, and building rubble. Thin ash layers.	0.5
1.5	black ash layer approximately 400 thick.	1.5
2.0	silt mixed with black ash & building rubble. becoming uniform silt.	2.0
2.5		2.5

NOTES.

Density test of Ash layer at 1.5 m.

Dry Density = 1345 kg/m³

% Moisture = 24.6%



ALAN REAY CONSULTANTS SEIVED

SITE INVESTIGATION

PROJECT: DEVELOPMENT FOR PULSE DATA
MARY MULLER DRIVE

28 AUG 2008

INST 300 dia AUGER

FILE 7735

DATE 1-11-04

DEPTH CL=	BORE No. I 13m from West Bdy 88m from South Bdy	DEPTH CL=	BORE No. II 33m from West Bdy 88m from South Bdy
0.0	Light brown sandy Silt Some stones & bricks	0.0	Light brown sandy Silt, some stones & bricks.
0.5	Soft dark ash, silt, sand mix, some pieces rotting wood.	0.5	
1.0	Brown Silt, some bricks.	1.0	Dark Silt mixed with white "Quick Lime" bricks & pieces of wood.
1.5	Blue Silt, some pieces ew. pipe. Slight odour.	1.5	
2.0	Dark Silt mixture of bricks, stones, pieces wood & humus material. Slight odour.	2.0	W.T. @ 1.850 after drilling.
2.5	Soft dark silty Clay (Pug) Some stones, bricks. Slight odour	2.5	Dark blue silt mixed with pieces corrugated iron, stones & bricks
3.0	W.T. @ 2.1m after drilling Stiff blue/grey clayey Silt, dark ash & broken concrete.	3.0	Stiff grey/brown clayey Silt. Some corr. iron & bricks
3.5	Grey/brown sandy Silt	3.5	Grey/brown clayey Silt
4.0		4.0	Grey/brown Sand
4.5	Soft grey silty Clay (Pug) Some pieces roots, tree. Drilled to 5m & struck coarse grey Sand.	4.5	Drilled to 4m still sand.

SITE INVESTIGATION

PROJECT:

DEPTH CL= 0.0	BORE No. III 53m from West Bdy 88m from South Bdy	DEPTH CL= 0.0	BORE No. IV 73m from West Bdy 88m from South Bdy
0.5	Brown sandy Silt, some stones, coal.	0.5	Dark brown sandy Silt some bricks & stones
1.0	Pieces of wood at 1m	1.0	
1.5		1.5	Silty Sand mixed with stones, bricks, coal etc
2.0		2.0	
2.5	Stiff grey/brown clayey Silt.	2.5	Stiff blue/grey clayey Silt
3.0	Grey/brown silty Sand	3.0	Grey/brown sandy Silt
3.5	Grey/brown clayey Silt.	3.5	
4.0	Drilled to 3.5m No W.T. struck.	4.0	Drilled to 3.5m No W.T. struck
4.5		4.5	

SITE INVESTIGATION

PROJECT:

DEPTH GL= 0.0	BORE No. V 108m from West Bdy 88m from South Bdy	DEPTH GL= 0.0	BORE No. VI 123m from West Bdy 88m from South Bdy
0.5	Brown sandy Silt mixed with bricks, stones & trace of wood.	0.5	Brown sandy Silt, some stones. Struck concrete @ 600
1.0		1.0	"old" dark silty Top Soil Slight peat odour.
1.5		1.5	
2.0	Brown/grey Silt mixed with trace top soil, stones, bricks, pieces of rubber. Slight peat odour.	2.0	Stiff grey/brown clayey Silt
2.5		2.5	
3.0	Stiff grey/brown clayey Silt	3.0	Grey/brown sandy Silt
3.5	Grey/brown sandy Silt Drilled to 3.1m No W.T. struck	3.5	Drilled to 3.1m No W.T. struck
4.0		4.0	
4.5		4.5	

SITE INVESTIGATION

PROJECT:

DEPTH CL= 0.0	BORE No. VII 13m from West Bdy 68m from South Bdy	DEPTH CL= 0.0	BORE No. VIII 33m from West Bdy 68m from South Bdy
0.5	Brown sandy Silt, stones, bricks, concrete	0.5	Brown sandy Silt, Some bricks & stones
1.0		1.0	
1.5		1.5	Numerous brick pieces @ 1.2
2.0	Dark sandy mix of stones, bricks & trace fibrous plant material	2.0	W.T. @ 1.8m after drilling.
2.5		2.5	Stiff grey clayey Silt
3.0	Stiff blue silt, some stones & bricks	3.0	Grey/brown sandy Silt
3.5	Blue/grey Silt	3.5	Drilled to 3.1m Struck water at 2.4m whilst drilling.
4.0	Drilled to 3.1m No W.T. struck	4.0	
4.5		4.5	

SITE INVESTIGATION

INST 300 dia AUGER

PROJECT:

FILE 7735

DATE 1-11-04

DEPTH GL= 0.0	BORE No. IX 53m from West Bdy 68m from South Bdy	DEPTH GL= 0.0	BORE No. X 73m from West Bdy 68m from South Bdy
0.5	Brown sandy Silt, some stones & bricks	0.5	Brown sandy Silt, some bricks, stones & trace tree roots.
1.0	trace wood @ 800	1.0	
1.5		1.5	Struck concrete 1.5m moved Bore 1.5m South.
2.0	Struck concrete @ 1.8m moved Bore 2m East Dark Silt, trace organic soil, pieces metal.	2.0	Dark sandy Silt, bricks, metal, & wood (old oak barrel). Dark sand mixed with bricks, wood, & metal Slight odour.
2.5	Stiff blue/grey clayey Silt	2.5	Stiff grey clayey Silt. Grey/brown Silt.
3.0	Grey/brown Silt.	3.0	Grey/brown sandy Silt.
3.5	Grey/brown clayey Silt.	3.5	Drilled to 2.6m No w.t. struck.
4.0		4.0	
4.5	Drilled to 3.1m No w.t. struck though bore material wet.	4.5	

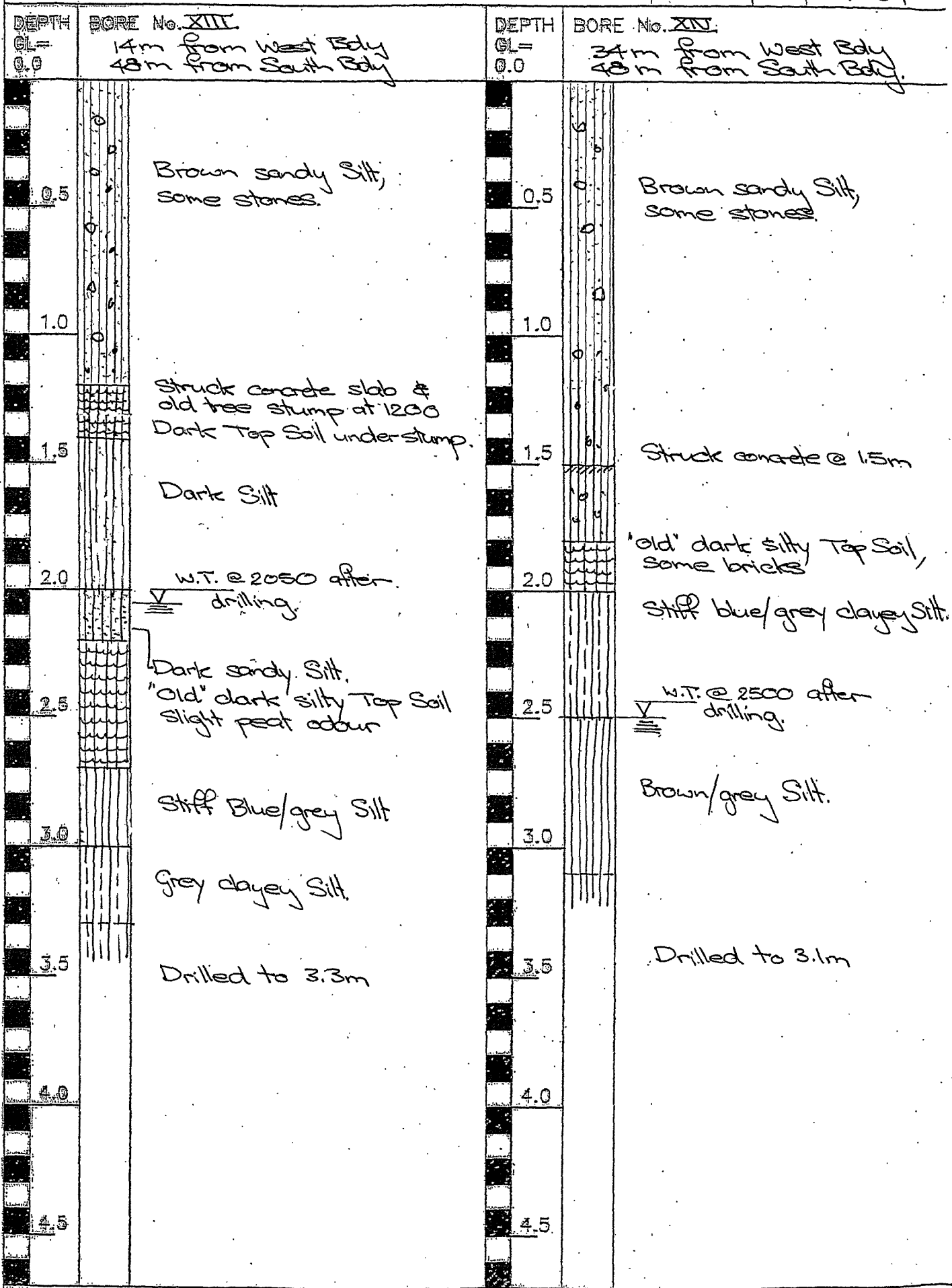
SITE INVESTIGATION

PROJECT:

DEPTH CL=	BORE No. XI 93m from West Bdy 68m from South Bdy	DEPTH CL=	BORE No. XII 106m from West Bdy 68m from South Bdy
0.0		0.0	
0.5	Brown sandy Silt, some stones, bricks, & concrete	0.5	Brown sandy Silt, some stones, bricks, & concrete
1.0		1.0	
1.5		1.5	Dark silt layer, some bricks Dark silty Top Soil.
2.0	Struck layer of cloth rags i.e. clothing man. trimmings. Stiff grey clayey Silt	2.0	Stiff grey/brown clayey Silt.
2.5		2.5	Grey/brown sandy Silt
3.0	W.T. @ 2.7m after drilling Grey/brown Silt. Grey/brown sandy Silt.	3.0	Grey/brown clayey Silt
3.5	Drilled to 3.1m	3.5	Drilled to 3.1m No W.T. struck
4.0		4.0	
4.5		4.5	

SITE INVESTIGATION

PROJECT:



SITE INVESTIGATION

PROJECT:

DEPTH GL= 0.0	BORE No. <u>XV</u> 54m from West Bdy 48m from South Bdy	DEPTH GL= 0.0	BORE No. <u>XVII</u> 74m from West Bdy 48m from South Bdy
0.5	Brown sandy Silt, some stones.	0.5	Brown sandy Silt, stones, bricks, & concrete
1.0		1.0	Struck concrete slab @ 300
1.5	Silt/sand mix with bricks, stones & concrete	1.5	
2.0	Dark silt layer	2.0	
2.5	Stiff Grey clayey Silt	2.5	Stiff Grey clayey Silt
2.5	W.T. @ 2.5m after drilling	2.5	W.T. @ 2.5m after drilling.
3.0	Grey/brown sandy Silt	3.0	Grey/brown sandy Silt.
3.0	Drilled to 3.1m	3.0	Drilled to 3.1m
3.5		3.5	
4.0		4.0	
4.5		4.5	

SITE INVESTIGATION

PROJECT:

DEPTH GL= 0.0	BORE No. XVII 94m from West Bdy 48m from South Bdy	DEPTH GL= 0.0	BORE No. XVIII 104m from West Bdy 48m from South Bdy
0.5	Brown sandy Silt, some stones, bricks & concrete	0.5	Brown sandy Silt, some stones, bricks & concrete
1.0	Struck concrete slab	1.0	Sandy coal/coke ash.
1.5	Stiff grey silt, some stones	1.5	Trace "old" top soil @ 1500
2.0	"old" silty top soil layer	2.0	Grey/brown sandy Silt.
2.5	Stiff Grey/brown clayey Silt	2.5	Drilled to 2.1m No w.t. struck
3.0	Grey/brown sandy Silt.	3.0	
3.5	Drilled to 2.3m No w.t. struck	3.5	
4.0		4.0	
4.5		4.5	

SITE INVESTIGATION

PROJECT:

DEPTH CL= 0.0	BORE No. XIX 18m from West Bdy 28m from South Bdy	DEPTH CL= 0.0	BORE No. XX 30m from West Bdy 28m from South Bdy
0.5	Brown sandy Silt, some stones.	0.5	Light Brown sandy Silt, some stones.
1.0		1.0	Brown/yellow Silt, some stones & concrete.
1.5	Dark brown sandy Silt Some stones bricks, concrete & metal.	1.5	Stiff Grey/brown clayey Silt
2.0	W.T. @ 2m after drilling	2.0	
2.5	Stiff dark blue clayey Silt	2.5	
	Grey clayey Silt. Drilled to 2.6m		Grey/brown sandy Silt.
3.0		3.0	Drilled to 2.6m No W.T. struck
3.5		3.5	
4.0		4.0	
4.5		4.5	

SITE INVESTIGATION

PROJECT:

DEPTH CL= 0.0	BORE No. XXI 58m from West Bdy 28m from South Bdy	DEPTH CL= 0.0	BORE No. XXII 78m from West Bdy 28m from South Bdy
0.5	Brown sandy Silt, some stones.	0.5	Brown sandy Silt, some stones.
1.0	Pieces concrete @ 800	1.0	Brown Sand, some stones
1.5	Dark stiff silt "Old Top Soil layer"	1.5	Silty Sand mixed with coal/coke ash, some stones
2.0	W.T. @ 1.8m after drilling. Stiff blue/grey clayey Silt	2.0	W.T. @ 1.8m after drilling. Stiff dark blue clayey Silt
2.5	Grey/brown Silt.	2.5	Grey/brown sandy Silt.
3.0	Grey/brown sandy Silt. Drilled to 2.6m	3.0	Drilled to 2.5m
3.5		3.5	
4.0		4.0	
4.5		4.5	

SITE INVESTIGATION

PROJECT:

DEPTH GL= 0.0	BORE No. <u>XXIII</u> 93m from West Bay 23m from South Bay	DEPTH GL= 0.0	BORE No. <u>XXIV</u> 25m from West Bay 8m from South Bay
0.5	Brown sandy Silt, some stones	0.5	Brown sandy Silt some stones & concrete
1.0		1.0	Struck concrete 800
1.5		1.5	
2.0	W.T. @ 1650 after drilling stiff blue clayey Silt	2.0	stiff grey Silt
2.5	Grey/brown sandy Silt.	2.5	Dark Silt "old" Top Soil layer
3.0		3.0	Grey/brown Silt.
3.5		3.5	
4.0		4.0	
4.5		4.5	
	Drilled to 2.6m		Drilled to 2.6m

SITE INVESTIGATION

PROJECT:

DEPTH CL= 0.0	BORE No. XXV 45m from West Bdy 8m from South Bdy	DEPTH CL= 0.0	BORE No. XXVI 65m from West Bdy 8m from South Bdy
0.5	Light Brown sandy Silt Some stones	0.5	Brown sandy Silt some stones
1.0		1.0	
1.5	Brown sand Some stones	1.5	Brown/grey Silt, Some stones
2.0	Dark clayey Silt & top soil mix. (Pug)	2.0	
2.5	Stiff dark Silt. "old" top soil layer	2.5	Stiff blue clayey Silt
	Stiff Blue/grey clayey Silt		Grey/brown sandy Silt.
	grey/brown sandy Silt.		
3.0	Drilled to 2.7m No W.T. struck	3.0	Drilled to 2.6m No W.T. struck
3.5		3.5	
4.0		4.0	
4.5		4.5	

SITE INVESTIGATION

PROJECT:

DEPTH CL= 0.0	BORE No. <u>XXVII</u> 85m from West Rd 8m from South Bdy	DEPTH CL= 0.0	BORE No. <u>XXVIII</u> 105m from West Rd 8m from South Bdy
0.5	Brown sandy Silt, trace stones.	0.5	Brown sandy Silt, some stones, & concrete
1.0		1.0	Silty Sand, some stones
1.5	Grey/brown Silt, some stones, brick, & concrete	1.5	Dark silty Top Soil, some stones
2.0	Sandy Silty Top Soil	2.0	Dark sandy Silt, some rocks.
	Stiff Blue clayey Silt.		Stiff blue clayey Silt.
	Grey/brown Silt.		Grey/brown Silt.
2.5	Grey/brown sandy Silt.	2.5	Stiff blue clayey Silt W.T. @ 2.4m after drilling.
	Drilled to 2.6m No W.T. struck.		Grey/brown Silt
			Grey/brown sand Silt.
			Grey/brown clayey Silt.
			Drilled to 3m
3.5		3.5	
4.0		4.0	
4.5		4.5	


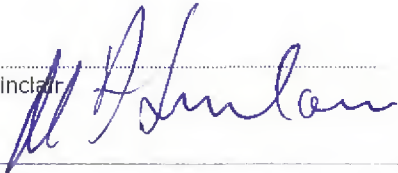
GEOTECHNICAL REPORT – REVISION A

10 Mary Muller Drive

For Castle Rock Properties Ltd

GEOTECHNICAL REPORT – REVISION A

10 Mary Muller Drive

QUALITY CONTROL CERTIFICATE		
All relevant information is identified, has been reviewed, and is approved for release.		
Prepared by:	 John Aramowicz Associate, Civil/Geotechnical Engineer	BE(Hons), MIPENZ, CPEng, IntPE(NZ)
Reviewed and approved for release by:	 Marton D Sinclair Director	BE, BSc, FNZIS, MIPENZ, CPEng, IntPE(NZ), RPS
Date:	24 July 2012	
Reference:	352691_Geotech Report_FINAL-RevA.doc	
Status:	FINAL	
Distribution:	1 x original 1 x file copy	for Castle Rock Properties Ltd Eliot Sinclair

Limitations

This report has been prepared for Castle Rock Properties Ltd. according to their instructions, for the particular objectives described in the report. The information contained in the report should not be used by anyone else or for any other purposes.

Executive Summary

Castle Rock Properties Ltd. have engaged Eliot Sinclair to undertake a geotechnical investigation, liquefaction assessment and report for the Wormald's site at 10 Mary Muller Drive, Hillsborough, at the southwest part of Lot 2 DP 392999.

We understand that a single-storey office and factory is proposed to be constructed across the central and western parts of the site.

The M6.3 earthquake of 22 February 2011 resulted in a peak ground acceleration of approximately 0.65g in the general area, and cause some liquefaction to the south of the site, and foundations settlement to a large tilt-panel coolstore to the west of the site.

Geotechnical investigation of the site comprised six deep CPTu tests, that extended to 19.5 to 25.5m depth below ground level. The soil types inferred from the CPT tests were mostly uniform across the site, comprising of uncontrolled fill materials to 1.5m top 2m depth, over layers of clayey silts and sandy silts, with a layer of 'sand & silty sand' at 4-9m and at 15-16m depth below ground level.

Analysis of the CPTu test results indicates the presence of thin liquefiable layers throughout the soils profile. Calculated total vertical settlements range from 80mm to 120mm in a proposed SLS (0.13g) event, and from 150mm to 210mm in a ULS (0.35g) event.

Differential movements within adjacent buildings are in the order of 50% of the calculated total settlements.

Consideration will need to be given to the likelihood of encountering contaminated materials, and appropriate approval may need to be obtained from the Council before any test pits can be excavated, or ground improvement earthworks or foundation excavations commenced.

Based on the CPT test data, a practical foundation system could be undertaken by compaction of the insitu fill materials using a square-impact roller to as much of the site as is practical, before constructing shallow strip and pad foundations for the small, lightweight office building, and the factory part of the building.

Foundations for the factory building will require careful consideration due to the presence of the historic fill, location of the heavy foundation loads along the boundary, and the presence of an existing foundation immediately to the west.

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1. INTRODUCTION

Castle Rock Properties Ltd. have engaged Eliot Sinclair to undertake a geotechnical investigation, liquefaction assessment, and report for the Wormald site at 10 Mary Muller Drive, legally described as Lot 2 DP 392999.

The geotechnical report is required to characterise the general geotechnical conditions across the site, and to advise on likely foundations requirements for the proposed building.

2. BUILDING PROPOSAL

Concept drawings provided to us indicate a single-storey office building at the centre of the site, and a factory with 6~7m stud height between the office and the western boundary.

The concept drawings appear to show lightweight cladding to the west, north and east parts of the office building, with the factory constructed using a portal frame, steel roof, and concrete tilt-panel walls. There is provision shown on the drawings for a southern extension to the factory.

A sealed driveway and carpark area is to be located to the north, east and south of the office and factory buildings. Refer to Appendix D.

3. SCOPE OF WORK

The scope of work for the geotechnical assessment of the site was;

- Review published geology,
- Review Environment Canterbury's well card database for nearby bore log data,
- Review floor level data for adjacent buildings,
- Review of GNS Science's strong motion data for 4 September 2010 and 22 February 2011 earthquakes,
- Undertake 6 x Cone Penetration Tests with measurement of pore water pressures (CPTu),
- Liquefaction analysis,
- Reporting and Recommendations.

4. DISCLAIMER

Comments made in this geotechnical report are based on test records obtained from CPTu testing undertaken in March 2012, inspection of the general area, and liquefaction modelling using Geologimiski's CLiq software.

Whilst every care was taken during our investigation and interpretation of subsurface conditions, there may well be subsoil strata and features that were not detected. Additionally, on-going seismicity in the general area may lead to deterioration or additional ground settlement that could not have been anticipated at time of writing of this report.

The exposure of such conditions, or occurrence of additional strong seismicity, may require a review of our recommendations.

This report has been prepared for the benefit of Castle Rock Properties Ltd. at time of writing of this report, in accordance with the scope of work. No liability is accepted by this company or any employee of this company with respect to the use of this report by any other party for any other purpose.

5. ENGINEERING GEOLOGY

*Christchurch is situated on the middle part of the east coast of the South Island of New Zealand. The city is located on Holocene deposits at the Pegasus Bay coast of the Canterbury Plains, and on the northern slopes of the adjacent Port Hills of Banks Peninsula. Brown and Webber (1992) describe the Christchurch CBD area being underlain by 'a sequence of gravel strata interbedded with silt, clay, peat, shelly sand and clay down to 400m.'*¹

Brown and Webber (1992) note the general area of the site being underlain by: "*Dominantly alluvial sand and silt overbank deposits.*" The general subsoil geology of Hillsborough will be similar to that of the Christchurch CBD with respect to soil types and presence of younger sediments in the upper layers overlying sandy gravels at some depth.

The key characteristics of the Christchurch CBD soils are summarised by Cubrinovski & McCahon² as;

- The top 20-25 m of the CBD soils are relatively recent alluvial soils overlying 300m to 500m thick gravelly deposits.
- The recent alluvial soils in the top 20 m of the deposits are the most important for foundations of multi-storey buildings and liquefaction evaluation. These soils comprise

¹ Brown, L.J.; Weeber, J.H. 1992 Geology of the Christchurch urban area. Scale 1:25 000. Institute of Geological & Nuclear Sciences geological map 1. 1 sheet + 104p. Institute of Geological and Nuclear Sciences Limited, Lower Hutt, New Zealand.

² Cubrinovski, M., McCahon, I. 2011 Foundations on Deep Alluvial Soils – Technical Report Prepared for the Canterbury Earthquake Royal Commission. University of Canterbury, Christchurch, New Zealand.

gravels, sands, silts, peat and their mixtures, and are highly variable both horizontally and vertically.

- The soils within the CBD are fully saturated below 1.0 m to 1.5 m depth
- Considering their composition (sandy soils and non-plastic silts), age (recent deposits, few hundreds to a few thousand years old) and depositional environment (river, swamp and marine sediments), these soils are generally considered susceptible to liquefaction, and in some cases (when deposited in a loose state) they have very low resistance to liquefaction.
- By and large, the foundation conditions within CBD are very complex and challenging for geotechnical engineers, particularly in regard to their performance during strong earthquakes.
- The presence of aquifers at depths of about 20 m to 25 m (and in some cases even at shallower depths) is a relatively unique feature that potentially may exacerbate the seismic response of the soils above the aquifers during strong earthquakes (by providing an additional mechanism for increase in the groundwater pressure through upward flow of water fed by the aquifers).
- The presence of aquifers at depths of about 20 m to 25 m (and in some cases even at shallower depths) is a relatively unique feature that potentially may exacerbate the seismic response of the soils above the aquifers during strong earthquakes (by providing an additional mechanism for increase in the groundwater pressure through upward flow of water fed by the aquifers).

5.1 EXISTING BORE HOLE DATA

Bore log records sourced from Environment Canterbury were reviewed to determine typical subsoil geology of the general area.

Well M36/1170 located approximately 200m southeast of the site indicates '*brown sandy silty clay*' from 0.4m to 2m depth below ground level, '*blue grey silty sand*' to 3.8m, '*silty clay*' to 6.4m and '*sandy silty clay*' to 18.2m depth below ground level where the log terminates.

Well M36/20624 located approximately 250m southwest of the site indicates '*fill*' to 0.9m below ground level, '*grey silt*' to 7.2m, and '*brown fine sandy silt*' to 12m depth below ground level where the log terminates.

Well M36/3883 located approximately 550m northwest of the site generally indicates organic silts to 4.5m, sandy silts to 7m, sandy clay to 9.5m and clay to 16.0m depth below ground level where the log terminates.

6. SITE DESCRIPTION

6.1 GENERAL

The site at 10 Mary Muller Drive forms part of Lot 2 DP 306637, with other existing buildings across the lot. The lease site is to the west of Mary Muller Drive and has flat, grassed ground surface.

The Wyatt Print building is located south of the site, Geon to the west, and the Bidvest coolstore is located to the southwest. Refer to Diagram 1.



Diagram 1: Site location plan with approximate boundary in yellow (source: Canterbury Recovery, Project Orbit, photo from 24 February 2011).

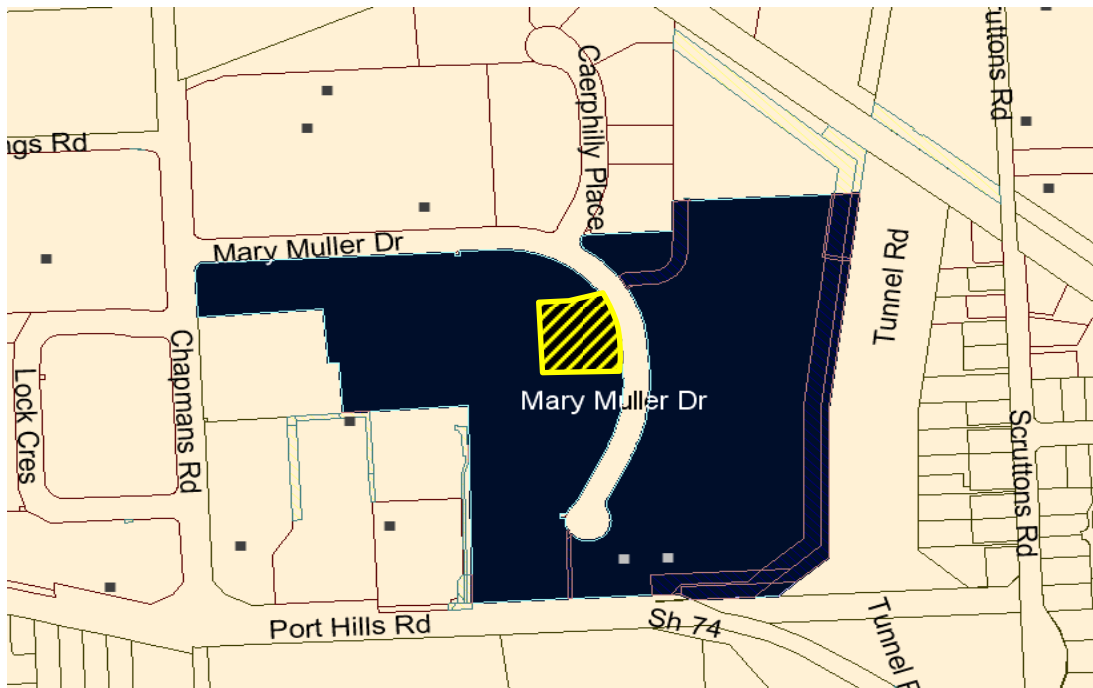


Diagram 2: Lot 2 DP306637 shaded in black, with 10 Mary Muller Drive in yellow hatch (Source: Quick Map, March 2012)

6.2 LAND CLASSIFICATION

On the 28th October 2011, the Canterbury Earthquake Recovery Authority (CERA) released site-specific classification for Canterbury properties. As part of this, the Department of Building and Housing (DBH) developed three new technical categories for residential foundation design as part of its guidance for repairing and rebuilding earthquake damaged homes in Canterbury. These new categories apply to liquefaction prone residential flat land in the green zone in the greater Christchurch urban area, but not to commercial land.

The site is noted as Green Zone, N/A Non Residential, however, residential properties nearby the site have been categorised as Technical Category 2 (TC2, yellow) which indicates, *“minor to moderate land damage from liquefaction is possible in future significant earthquakes”*. Refer to Appendix B.

6.1 GEON BUILDING

The building located along the western boundary, referred to as the Geon building in this report, is constructed with a portal frame structure, shallow foundation pads for the portal columns, precast concrete tilt panel exterior walls, concrete slab on ground and lightweight steel roof.

7. SITE INVESTIGATION

7.1 CPTU TEST RESULTS

Six CPTu tests were undertaken across the site, being:

- CPTu01 is located near the north western corner of the site,
- CPTu02 is located at the mid-north, between CPTu01 and CPTu03,
- CPTu03 is located near the north eastern corner of the site,
- CPTu04 is located near the south eastern corner of the site,
- CPTu05 is located at the mid-south, between CPTu04 and CPTu06,
- CPTu06 is located near the south western corner of the site

Refer to Appendix A for the CPTu test location plan.

The inferred soils from the CPTu analysis generally comprise multiple thin layers of silty clay and sandy silt with a predominantly *‘sandy & silty sand’* layer between 4 to 9m and from 15 to 16m depth below ground level.

The CPT's were terminated at 19.5m and 25.5m depth below ground level, however, dense sand or gravel was not encountered at the full depth of testing. Refer to Appendix B.

The nature of materials underlying the full depth of CPTu testing have not been confirmed as part of this site investigation by deep boreholes.

7.2 GROUNDWATER

The depth to groundwater is noted as 1.0m below original ground level on the Canterbury Recovery, Project Orbit database.

Well log M36/1170, approximately 200m southeast of the site, notes the minimum ground water level as 2.1m below ground level.

7.3 FILL MATERIALS

Test pits have not been excavated across the site as the upper layers are understood to comprise fill materials that may be contaminated, however, based on our observations across the site to the south, and during the formation of Mary Muller Drive, it is likely that there will be around 1.5m to 2m depth of uncontrolled fill material across the site.

7.4 OBSERVED LAND DAMAGE

The areas surrounding Mary Muller Drive experienced land damage due to liquefaction in the February 2011 earthquake event, and this is visible in the aerial photograph taken on 24 February 2011 where there is evidence of ejected sediment and groundwater at the carpark to the south and southeast, and around the Bidvest building to the west.

Survey measurements taken across the concrete floor of the Wyatt Print building, south of the site, indicate minor differential settlement has occurred from the earthquake shaking.

A report by Alan Reay Consultants, dated 21 December 2011, shows detailed floor levels across the Geon building, immediately west of the site. Differential floor levels of around 50-60mm were recorded, with the larger settlements recorded in the area of the 'Press Pad' at the centre of the building. The report outlines the damage caused by the February 2011 earthquake and aftershocks, including opening of control joints and cracking to the concrete floor slab, portal frame base spreading, foundation settlement, and distortion of precast panel joints due to foundation movement.

We also understand that the northern foundations of the Bidvest building, located to the southwest of the site, settled differentially by around 80mm in the February 2011 earthquake. Refer to Appendix C.

8. LIQUEFACTION ASSESSMENT

8.1 CANTERBURY EARTHQUAKE

The M7.1 Darfield earthquake on 04 September 2010 occurred on a previously unknown fault, producing peak horizontal ground accelerations (pga) of 0.24g at 'Christchurch Cathedral College' (CCCC). It is likely that this would have been similar to the horizontal ground accelerations experienced at 10 Mary Muller Drive, Hillsborough.

The earthquake resulted in liquefaction within the soft alluvial soils of eastern Christchurch.

A M6.3 aftershock on 22 February 2011 occurred on a previously unknown fault located under the Port Hills. Whilst the magnitude of the earthquake was lower than the M7.1 September 2010 event, the epicentre of the February 2011 event was located much closer to the urban areas of Christchurch, between Lyttelton and Heathcote Valley.

The M6.3 February 2011 event produced higher peak horizontal ground accelerations (pga) in Christchurch compared to the September event, with 0.49g recorded at Christchurch Cathedral College, and estimated to be around 0.65g in the area of the site.

Whilst much higher peak ground accelerations were recorded at Heathcote Valley Primary School, they were unlikely to be representative of the pga's at Mary Muller Drive due to the large differences in the underlying geology between the sites.

A subsequent M6.4 aftershock on 13 June 2011 located around Taylors Mistake, caused further liquefaction across the CBD and eastern parts of Christchurch, although this was generally not as extensive in comparison to the February 2011 event.

8.2 DESIGN PEAK GROUND ACCELERATIONS

The 'Interim guidance for repairing and rebuilding foundations in Technical Category 3' released by the Department of Building and Housing on 27 April 2012 specifies the peak ground accelerations to be adopted for liquefaction assessment in Canterbury, being 0.13g for an SLS event, and 0.35g for a ULS event.

Whilst this guidelines applies specifically to residential property, these values are now commonly adopted for assessment of other non-residential land.

Please note these pga's are not for use in structural design.

8.3 COMPARISON TO ACTUAL INTENSITIES

Strong motion records from GNS indicate peak horizontal accelerations were much higher in the 22 February 2011 earthquake than the 4 September 2010 earthquake.

Table 1: Comparison of peak horizontal ground accelerations close to site.

<i>PGA (horizontal)</i>	SLS (1/25, M7.5)	ULS (1/500, M7.5)	04 Sept 2010 ³ (M7.1)	22 Feb 2011 ⁴ (M6.3)
Design	0.13g	0.35g		
Estimate at site (based on seismic contouring)			0.30g	0.65g
Actual (CCCC)			0.24g	0.49g
Actual (HVPS)			0.66g	1.50g

³ Darfield (Canterbury) earthquake strong motion data, GNS Science, 04 September 2010.

⁴ Christchurch earthquake strong motion data, GNS Science, 22 February 2011.

Actual peak horizontal ground accelerations exceeded a ULS event in the 22 February 2011 event, and were close to a ULS event in the 04 September 2010 earthquake. Refer to Table 1.

8.4 METHODOLOGY

The site investigation and analysis was generally undertaken in accordance with the New Zealand Geotechnical Society's '*Geotechnical Earthquake Engineering Practice; Module 1-Guideline for the identification, assessment and mitigation of liquefaction hazards*', published in 2010.

For the purpose of this assessment, the depth to groundwater for liquefaction assessment is conservatively assumed 0.5m below existing ground level.

CPTu data was analysed for liquefaction using Geologimiski's CLiq software which uses the NCEER (1998)⁵ and Robertson (2009)⁶ method.

8.5 LIQUEFACTION POTENTIAL INDEX

The liquefaction potential index (LPI) in a proposed SLS (0.13g) event is estimated to be 4 to 6 at the full depth of testing, generally indicating a '*low*' to '*high*' risk of liquefaction damage to shallow building foundations.

Under the proposed ULS (0.35g) scenario, the LPI is estimated to range from 16 to 23 at the full depth of testing. These values indicate a '*very high*' risk of liquefaction damage to building foundations at the various CPT test sites.

Refer to Appendix D.

8.6 VERTICAL SETTLEMENT DUE TO LIQUEFACTION

Estimates of vertical settlement by computer modelling are summarised in Table 2.

Table 2: Summary of predicted liquefaction induced settlements down to 15m depth, at selected test locations.

Test Reference (depth to refusal)	CPTu01 (19.5m)	CPTu02 (21.9m)	CPTu03 (25.5m)	CPTu04 (21.8m)	CPTu05 (21.8m)	CPTu06 (25.5m)
SLS (M7.5, 0.13g)	85mm	80mm	90mm	80mm	70mm	110mm
ULS (M7.5, 0.35g)	160mm	150mm	150mm	130mm	110mm	150mm
22 Feb 2011 (M6.3, 0.49g)	160mm	160mm	160mm	140mm	110mm	150mm

⁵ Analysis and Fines Correction Methods; Youd & Idriss et al, 1998. *Liquefaction resistance of soils: Summary report for the 1996 NCEER and 1998 NCEER/SF Workshop of Evaluation of Liquefaction Resistance of Soils*. NCEER, ASCE Journal of Geotechnical & Geoenvironmental Engineering, Vol 127, pp 817-833

⁶ Robertson, P.K. 2009. Interpretation of cone penetration tests – a unified approach. *Canadian Geotechnical Journal* 2009, 46:1337-1355.

We note vertical settlements calculated by CLiq software use the method by Zhang (2002)⁷, for a range of parameters that are estimated from the four basic CPTu parameters of depth, cone tip resistance, skin friction, and pore water pressure, and therefore the settlements shown are not guaranteed or an exact figure.

8.7 COMMENT ON EARTHQUAKE SETTLEMENT

8.7.1 SERVICEABILITY LIMIT STATE

Settlements predicted in a SLS (0.13g) event range from 80mm to 120mm, refer to Table 2. NZGS's Geotechnical Earthquake Engineering Practice guidelines (Table 6.1) indicates that, based on the modelling results for an SLS event, a general performance level of L3 (high) for this site, ie: " $F_L < 1.0$; "Liquefaction occurs in significant portion of the deposit resulting in differential movements, large settlements (few hundreds of millimetres) and lateral displacements".

8.7.2 ULTIMATE LIMIT STATE

Settlements predicted in a ULS (0.35g) event range from 150mm to 210mm, refer to Table 2. NZGS's Geotechnical Earthquake Engineering Practice guidelines (Table 6.1) indicates that, based on the modelling results for a ULS event, a general performance level of L3 (high) for this site, ie: " $F_L < 1.0$; "Liquefaction occurs in significant portion of the deposit resulting in differential movements, large settlements (few hundreds of millimetres) and lateral displacements".

8.7.3 ACTUAL DAMAGE

Enquiries have indicated that there was no recorded settlement of building structures in the vicinity during Sept 2010 earthquake despite the peak ground accelerations substantially exceeding the SLS peak ground accelerations.

Whilst there was no visual evidence of liquefaction of ground settlement across the site during the Feb 2011 earthquake, differential settlement of foundations and floors is reported to have occurred to adjacent buildings.

Differential settlement of up to 50mm was recorded by a survey of floor levels of the adjacent commercial printing building south of 12 Mary Muller Drive, and 60mm to the Geon building to the west.

Also, around 30mm of horizontal displacement was recorded at the south part of Mary Muller Drive.

It appears from these records that the earthquake has caused differential settlement due to liquefaction, along with lateral displacement of foundations of the Geon building due to the horizontal component of the earthquake shaking (not lateral spreading).

⁷ Zhang, G., Robertson, P.K, Brachman, R. 2002, *Estimating Liquefaction Induced Ground Settlements from the CPT*, Canadian Geotechnical Journal, 39: pp 1168-1180.

Alan Reay Consultants Ltd.'s report for the Geon building describes damage due to opening of the slab control joints, likely damage to reinforcing across the control joints, and settlement of the foundations in relation to the floor slab at a number of locations.

The evidence therefore indicates that the settlements predicted by the analysis are conservative for both the SLS and ULS seismic events and much lower settlements can be expected in future SLS earthquakes.

One cause of this difference between observed and predicted settlements will be because many of the liquefiable layers are very thin and probably laterally discontinuous. These thin layers will tend not to fully liquefy in an SLS event with consequent reduced settlements. A further cause is that the levels recorded in the buildings are differential settlements not total settlements and do not fully reflect the movements of the ground within the vicinity.

8.8 LATERAL SPREADING

The topography of the site is flat with no major watercourses within 100m of the site, and therefore lateral spreading analysis was not undertaken for this site. The site is not likely to be subject to risk of liquefaction-induced lateral spreading.

9. FOUNDATION OPTIONS

9.1 GENERAL

The CPTu test results indicate highly variable penetration resistances in the upper fill layers, and appear to confirm the presence of a dense capping layer of silty gravel fill over uncontrolled fill materials to around 1.5 to 2m.

Based on previous excavations during road construction and photographic records on our file, the fill materials may contain a variety of materials, including leather scraps, topsoil, timber, rubber tyres, bricks, boulders, gravels, gasworks waste, silt, clay pipes, concrete slab debris, and scrap steel including silt and gravels. The large debris and scrap pipes can tend to form underground voids.

Due to the presence of the uncontrolled fill materials underlying the denser compacted silty gravel fill, there is a risk of differential settlement under static conditions due to consolidation of the deeper uncompacted materials, decay of organic matter, and loss of fines into underground voids. The foundation system for the proposed building will need to take the presence of these materials into account.

Further, the western wall of the proposed factory building is located adjacent to the eastern wall of the Geon building. Foundation construction for the new Wormald building will need to take into account the presence of existing foundations to avoid foundation failure and loss of lateral support.

9.2 DEEP PILED FOUNDATIONS

The factory building is to be constructed along the western boundary, adjacent to the Geon building, and a possible option is for the factory to be supported on a piled foundation to avoid construction issues with the adjacent foundations.

CPTu testing did not encounter any shallow dense sand or sandy gravel layers that would provide a suitable foundation for shallow driven piles.

The depth down to the inferred non-liquefiable silts and clayey silts is around 16-20m.

Due to the depth to non-liquefiable materials, driven piles are not practical for this site. Deep screw pile foundations may be able to be provided, however, this would be subject to specific engineering design by the supplier.

9.3 FLOOR SLABS

Due to the presence of dense gravel fill overlying uncontrolled fill down to around 1.5m-2.0m depth across the site, the design bearing pressure for dead and long-term live loads should not exceed $q_d = 20\text{kPa}$ in order to limit calculated settlements to around 25mm over 50 years.

The design bearing pressure could be increased by impact rolling of the surface, however, this could not be done in close proximity to the Geon building due to the risk of causing excessive ground vibrations.

Based on our experience, we estimate that all of the office floor and around 50-75% of the factory building area could be treated using impact roller compaction, resulting in a risk of long-term differential settlement of the untreated part of the factory floor area.

Use of impact roller compaction too close to the western boundary is likely to result in high ground vibrations that could cause damage to the Geon building. Square impact rolling cannot be undertaken where excessive ground vibrations are recorded, and these areas of the factory building will need to have any uncontrolled fill excavated, screened to remove large debris and unsuitable materials, and then backfilled with controlled compaction using a conventional steel-drum roller compactor. The smaller compaction equipment will produce lower ground vibrations and enable the fill materials to be compacted close to the western boundary.

We note that floor slabs should be well-reinforced and adequately connected to building foundations in order to provide a resilient construction that resists the effects of horizontal earthquake shaking.

9.4 SHALLOW FOUNDATIONS

The office building is well clear of the Geon building and could be constructed with shallow pad or strip-type footings that bear onto the insitu silts at around 2m depth. It is anticipated that any excavations would be backfilled with compacted sandy river-run or pit-run gravels, after construction of the foundations.

The factory building could have similar foundations at its eastern side, but there are difficulties in providing a similar foundation at the western side against the Geon building.

In our opinion, based on the observations of the existing structures in the locality, the use of shallow foundations should limit total settlements to less than 50mm in a SLS event, or 100mm in a ULS event however, construction of deepened footings that are well-connected to the floor slab will assist in limiting the amount of differential settlement.

Differential settlements across the structure are often found to be in the order of 50% of the total predicted settlements and on this basis the differential settlements likely to be experienced by the various foundations across the building are likely to be less than 25mm in an SLS event.

9.4.1 BEARING STRENGTH

Assessment of design bearing strength for both the factory and office foundations was undertaken using an average of the cone tip resistance under the proposed foundations, and assume pad or strip footings extend through any fill materials and bear onto insitu silts at 2m depth.

Where foundations bear onto insitu silts at 2m below existing ground level, the ultimate bearing strength to be used shall be determined from the attached chart "Bearing capacity control of shallow foundations" in Appendix E.

The geotechnical engineer should specify the design bearing strength to be used where foundation widths outside of this range are required.

Alternatively, shallow foundations could bear onto compacted gravel backfill. The gravel backfill will increase the allowable bearing pressures at the base of the foundations, but the excavation width will have to be increased to account for the load-spread effect of shallow concrete foundations bearing onto the gravel backfill. Alternative bearing strengths could be determined to account for a gravel layer under the foundation, if this option is selected however we understand that this is not a preferred option.

9.4.2 SETTLEMENT CONTROL

Primary and secondary settlements were estimated using proprietary software⁸, and assume that shallow foundations extend to 2m below existing ground level onto insitu silts. The ultimate bearing strength shown on the attached chart "Settlement control of shallow foundations", in Appendix F, is estimated to result in up to 25mm total settlement over a 50-year design life.

We note that foundation settlement resulting from the ultimate bearing strength is for static conditions only, and any ground settlements that may arise from seismic shaking would be additional to a static settlement of 25mm over 50 years.

⁸ CPTet, Geologimiski, using Terzaghi's 1-D theory of consolidation from inferred constrained modulus.

9.4.3 COMMENT

Foundation widths outside of the 0.5 to 4m width range shown on the charts in Appendix E and F should be reviewed by the geotechnical engineer in order to confirm the ultimate bearing strength to be used for foundation design.

We recommend a strength reduction factor of 0.55, being within the range of accepted values set out in Table 1 of NZBC Verification Method BM1/VM4.

We note that from a geotechnical engineering perspective, it would be preferable to use lightweight construction for the whole of the building, in order to avoid large differences in foundation pressures.

Should eccentric foundation loads be proposed then the geotechnical engineer should review the proposed foundation design and loadings to confirm the foundation design is suitable for the conditions encountered.

9.5 OTHER GROUND IMPROVEMENT OPTIONS

There are a number of ground remediation technologies available that could be used to mitigate the risk of consolidation of the uncontrolled fill and liquefaction by either chemical stabilisation, or densification of the loose soils.

Whilst it may be possible to use one or more of the various ground improvement methods to increase bearing capacity and decrease risk of liquefaction at this site, alternative ground improvement methods are not yet common in Christchurch and are therefore likely to be relatively costly. Some of the methods are;

9.5.1 INSITU COMPACTION

In situ compaction of the existing fill materials was trialled at the adjacent site using a square impact roller and appears to have provided a modest improvement in compaction of the underlying fill, although monitoring of ground vibrations at adjacent buildings indicates that this method may not be practical within 10m of existing structures, unless special precautions are taken to avoid damage.

Given the proposed factory is to be located adjacent to the Geon building, use of dynamic compaction for the factory area, and the driveway area south of this, is not practical, unless the Geon building were to be demolished.

Dynamic compaction would be a suitable method of ground improvement across the proposed office building and carpark areas north and south of this.

9.5.2 CEMENT STABILISATION

We note that remediation by cement stabilisation would be problematic at this site due to the likely presence of highly variable fill materials, large debris, and the presence of timber, topsoil, organic matter, etc, and is therefore not a suitable technique for the site.

9.5.3 REMEDIATION OF UNCONTROLLED FILL

Road formation works for Mary Muller Drive comprised excavation of the uncontrolled fill materials, disposal offsite of obvious organic or unsuitable materials, screening to remove large debris, etc and relaying the remaining screened soils in thin layers using controlled compaction and moisture control. These works appear to have been very successful as the surface of Mary Muller drive in the area of the site appears to be in good condition with no obvious evidence of earthquake related damage due to cracking, settlement, heaving, etc.

It would be possible to excavate most of the uncontrolled fill materials, screen and dispose unsuitable materials, and relay in layers using controlled compaction to create a relatively uniform, dense fill material although this method is likely to be quite costly.

10. FURTHER INVESTIGATIONS

Test pits have not been undertaken on this site to this stage as the areas is well known from previous investigations for the road and adjacent buildings.

The fill is up to 2m depth and consists of a gravelly silt to approximately 1m depth overlying a layer of less compact material which may contain organic materials, concrete, and some voids.

Some test pits could be excavated in the vicinity of the proposed footings to confirm the quality and depth of the fill layers, but this would be better done when the location and preferred option for the shallow foundations is known. It is anticipated that up to six test pits could be excavated at relatively short notice to complete this confirmation work.

11. DIFFERENTIAL SETTLEMENT

It is anticipated that foundations for the factory building will be more heavily loaded than those of the office building. There will be construction difficulties in excavating close to the Geon building, however, providing the exposure of the Geon foundations is limited to small areas at any one time, and temporary shoring provided as necessary to any excavations below the base of existing foundation pads, then we are satisfied that new foundation pads for the factory building can be constructed along the western boundary.

Differential settlement between the western wall of the factory and the eastern wall of the factory/office have been estimated on the assumption that the eastern portal bases to the eastern wall are supported on a pad or strip type foundation.

It would be desirable to detail a movement joint between the factory and office structures.

An estimate of the likely settlements are given below.

11.1 SLS

Static load settlement	25mm
Earthquake settlement with the top 15m of soils, and assuming that differential settlement is 50% of total settlement	25mm
Total	50mm

11.2 ULS

Static load settlement	25mm
Earthquake settlement with the top 15m of soils, and assuming that differential settlement is 50% of total settlement	90mm
Total	115mm

These ULS settlements are consistent with those observed in buildings in the locality after the February 2011 earthquake. In order to account for likely movements between the factory and the office, assuming independent foundations, the movement joint should be detailed to accommodate up to 50mm of long-term and earthquake movement.

If the eastern wall of the factory and the office are both founded on common strip or pad footings, vertical movement will not need to be accommodated especially if differential movement between the foundations and floor slabs of the two portions of the building are constrained by effective connections.

12. RECOMMENDATIONS

Consideration will need to be given to the likelihood of encountering contaminated materials, and appropriate approval will need to be obtained from the Council, and precautions taken, before any test pits can be excavated, or ground improvement earthworks and foundation excavations commenced.

In order to confirm the nature and depth of the fill materials across the site, we recommend at least six test pits be undertaken across the. Whilst the results of the test pits is unlikely to alter the recommendations of this report, the information will be useful in determining the depth and composition of the uncontrolled fill, and any other specific requirements for remediation/foundation construction.

Based on the results of the CPTu testing and analysis, we recommend the following ground improvement and foundation design parameters for the proposed building;

- Square impact roller compaction of the existing fill materials across the office and factory building footprint, and the carpark/driveway areas to the north and south, with care to be taken to limit ground vibrations at adjacent buildings to acceptable levels, and
- Where excessive ground vibrations make impact rolling impractical, close to the Geon foundations, the uncontrolled fill materials will need to be excavated and screened to remove unsuitable materials and debris, before using the screened

soil for backfill using controlled compaction with a steel-drum roller compactor, and

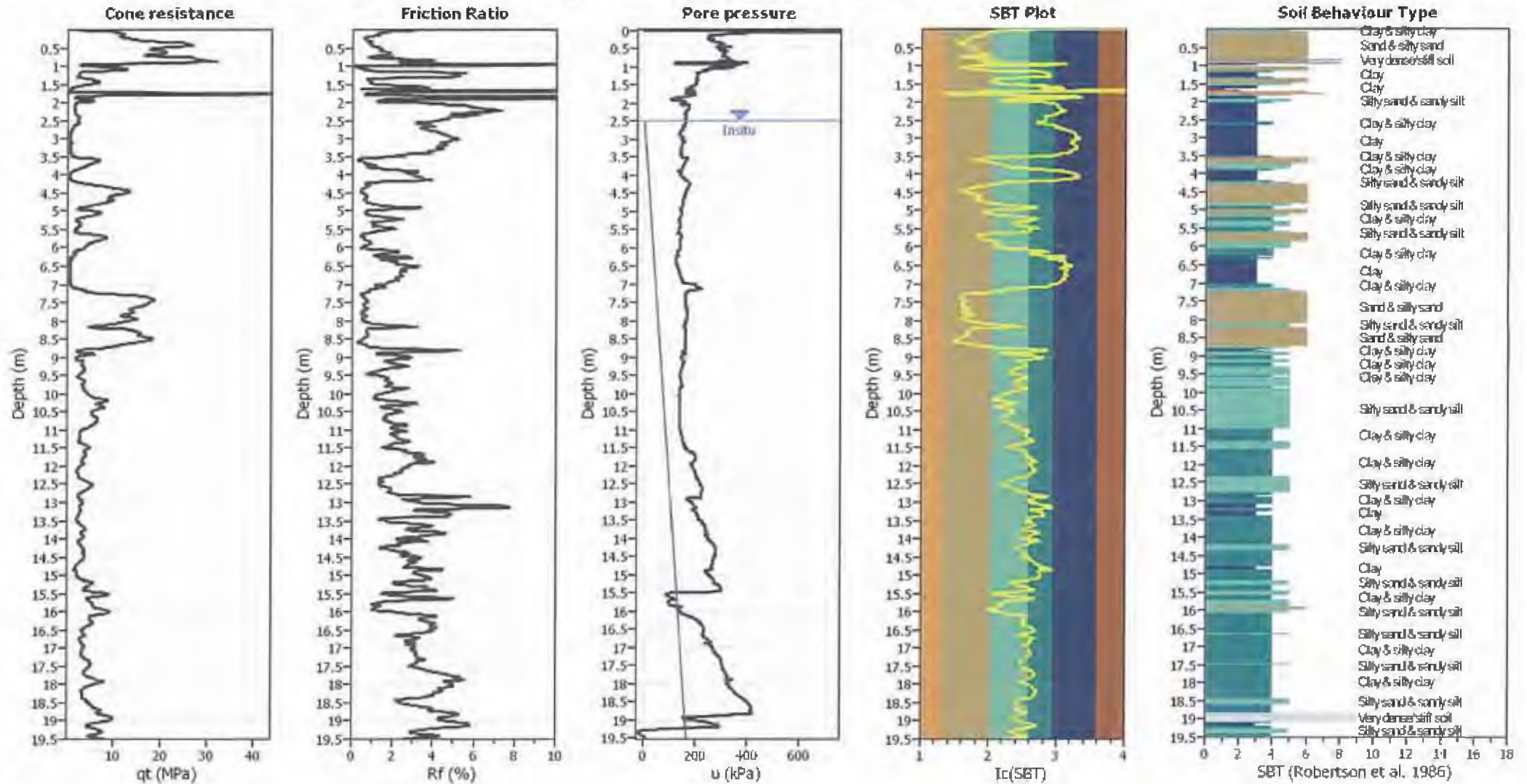
- Support the building on strip or pad foundations for the office and factory buildings that extend through all fill materials and bear onto insitu silts at around 2m depth, with a maximum design bearing strength determined using the ultimate bearing capacities shown on the charts in Appendix E and F, and
- Ensure that all building foundations and floor slabs are well-reinforced and connected to limit the effects of differential settlement and provide a resilient foundation system, and
- Limit the floor load for all floor slabs constructed over densified historic fill to **20kPa** in order to limit calculated floor settlements under static conditions to around 25mm over 50 years.

13. APPENDIX A – CPTU TEST LOCATION PLAN



14. APPENDIX B – CPTU TEST RESULTS & ANALYSIS

CPT basic interpretation plots



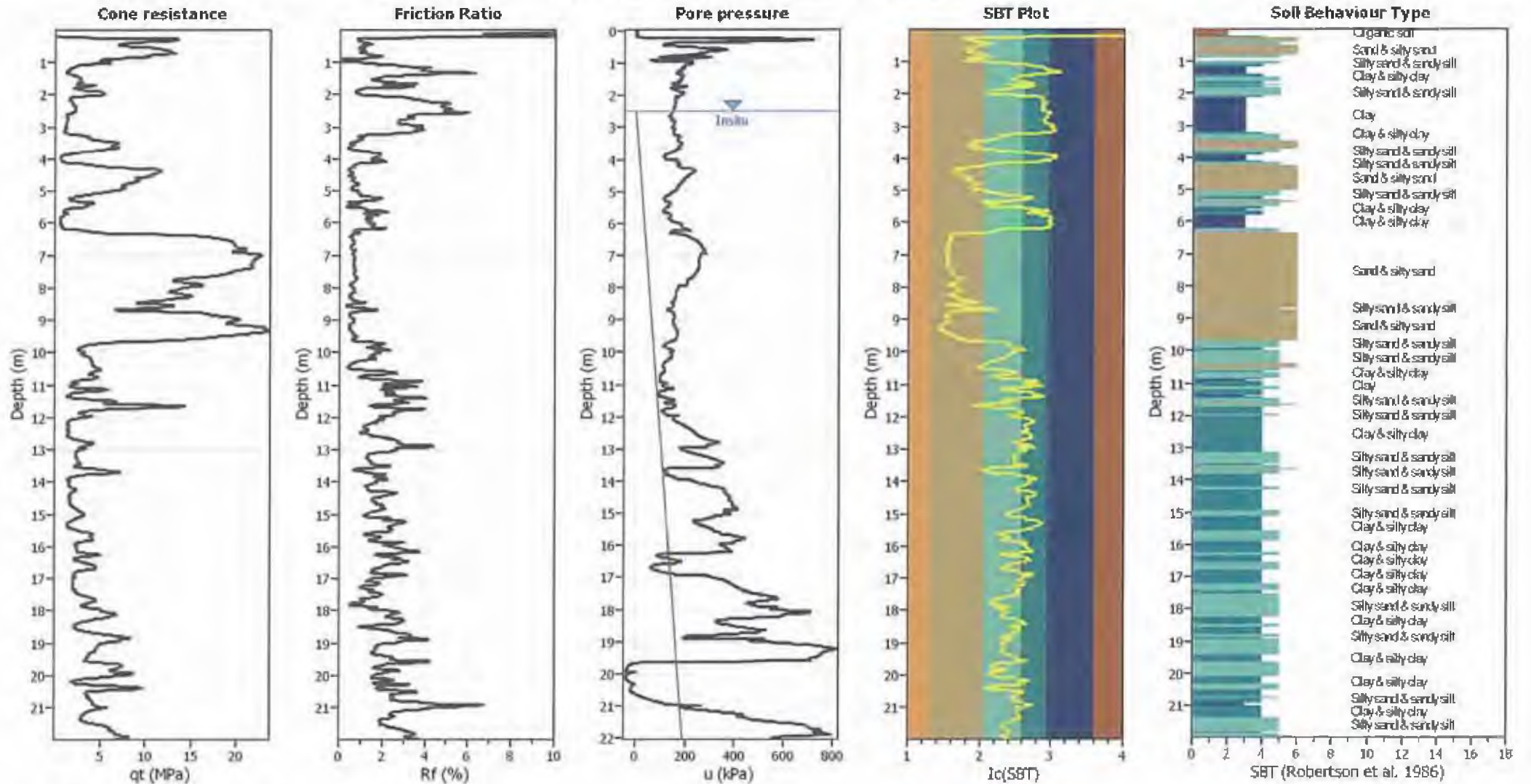
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _s applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots



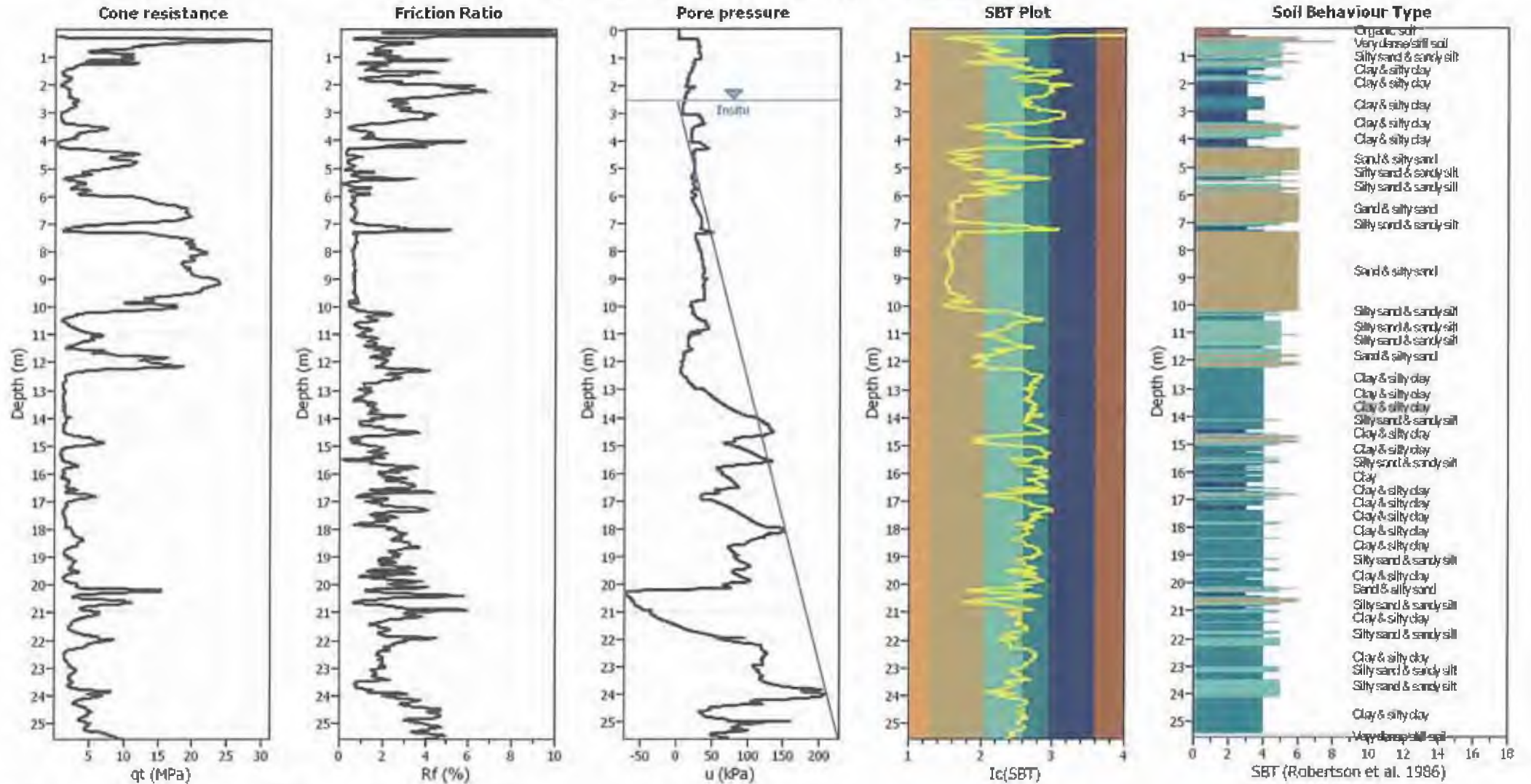
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _s applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic Interpretation plots



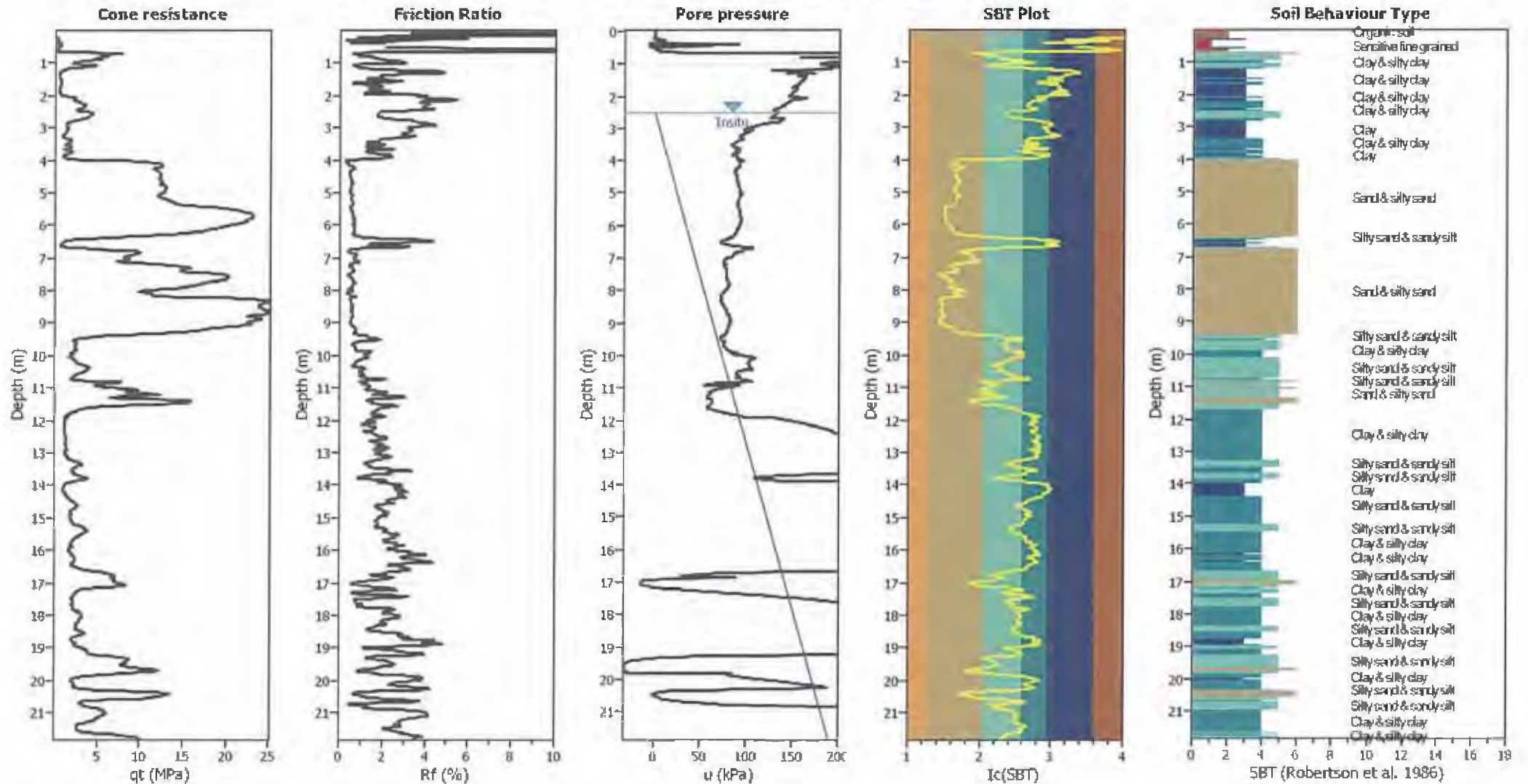
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Flare correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _c applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (in situ):	2.50 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots



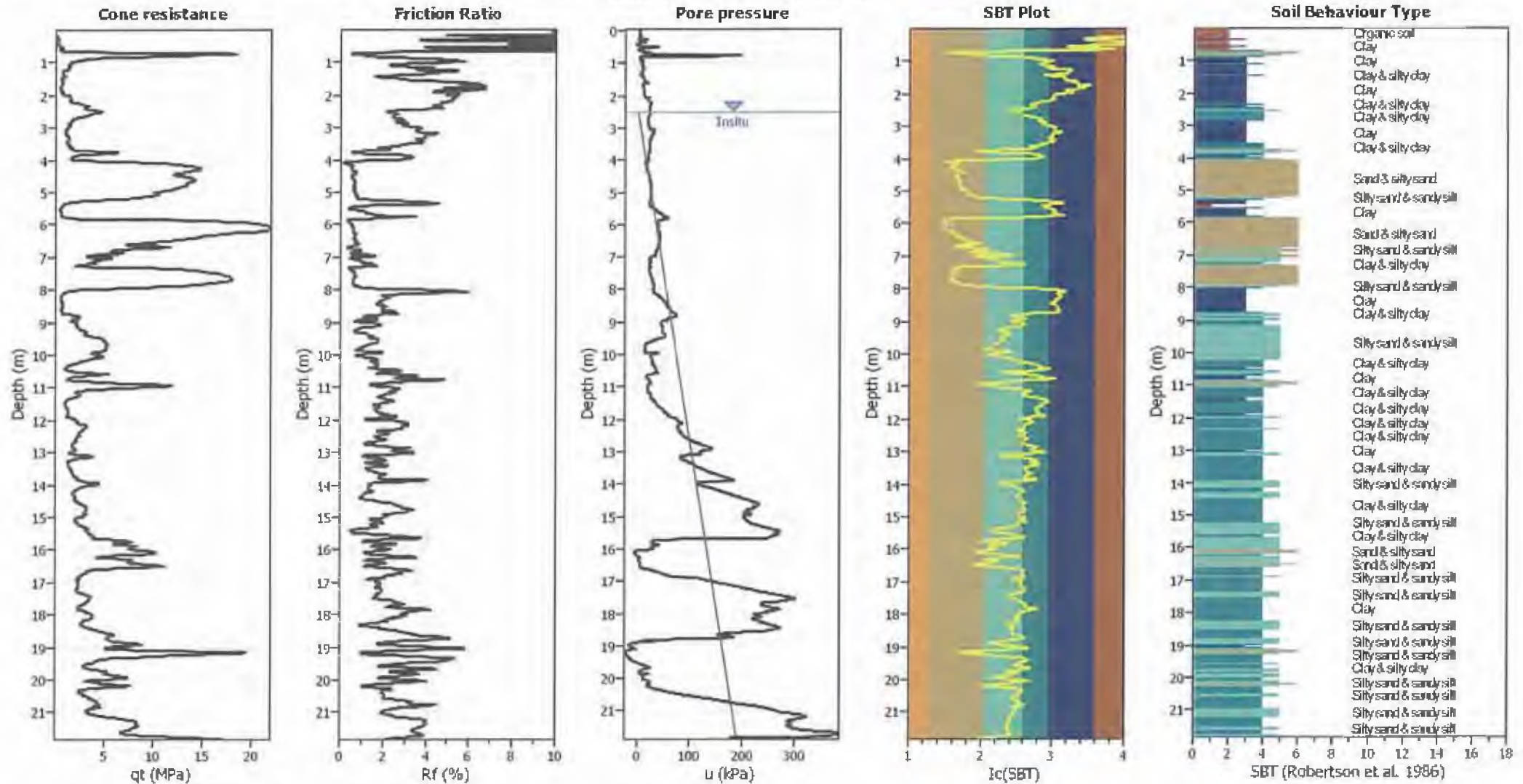
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (ortho.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_0 applied:	Yes
Earthquake magnitude M_w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (ortho.):	2.50 m	Fill height:	N/A	Unit depth:	N/A

SBT legend

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

CPT basic interpretation plots



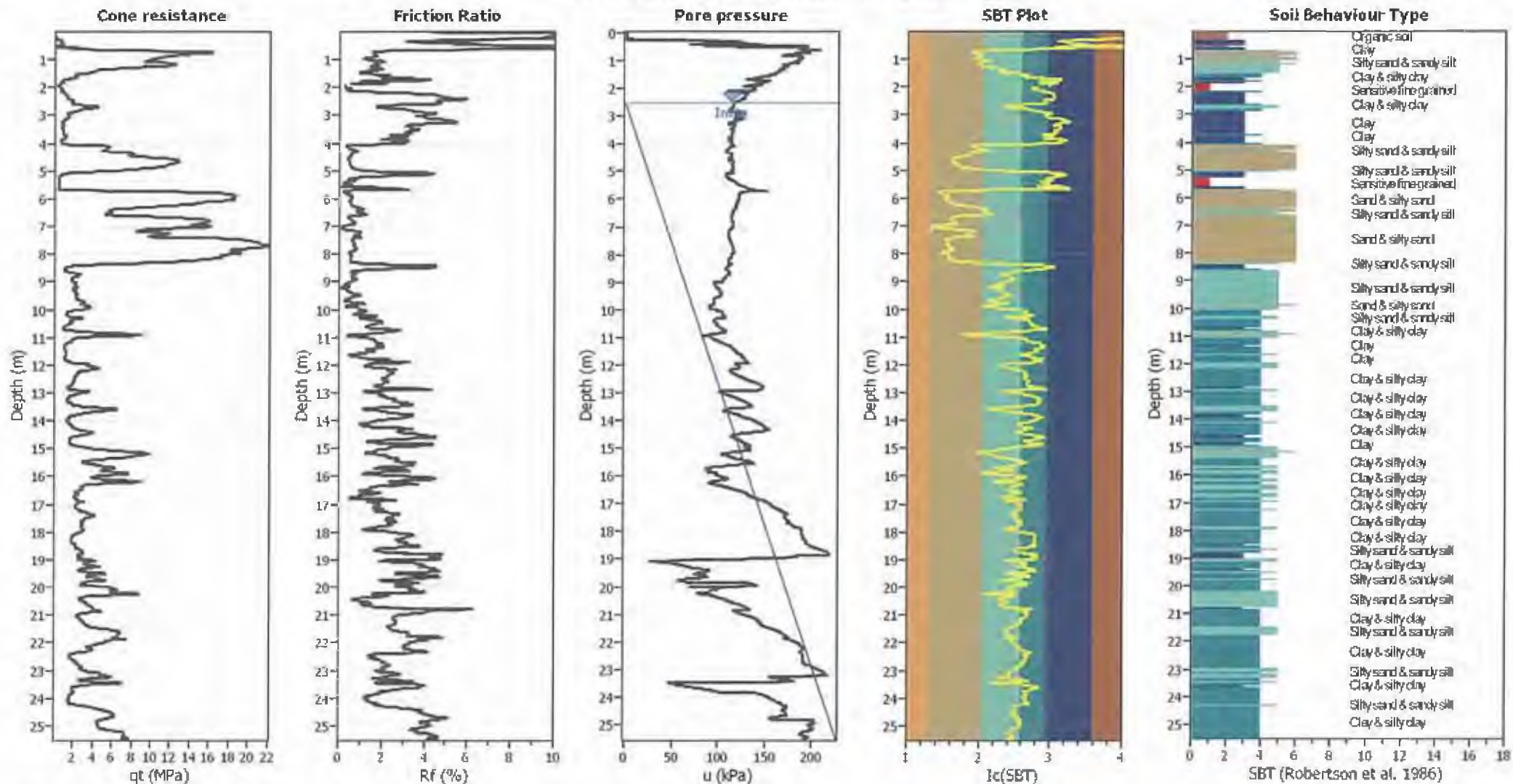
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (ortho.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.50	K _o applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots



Input parameters and analysis data

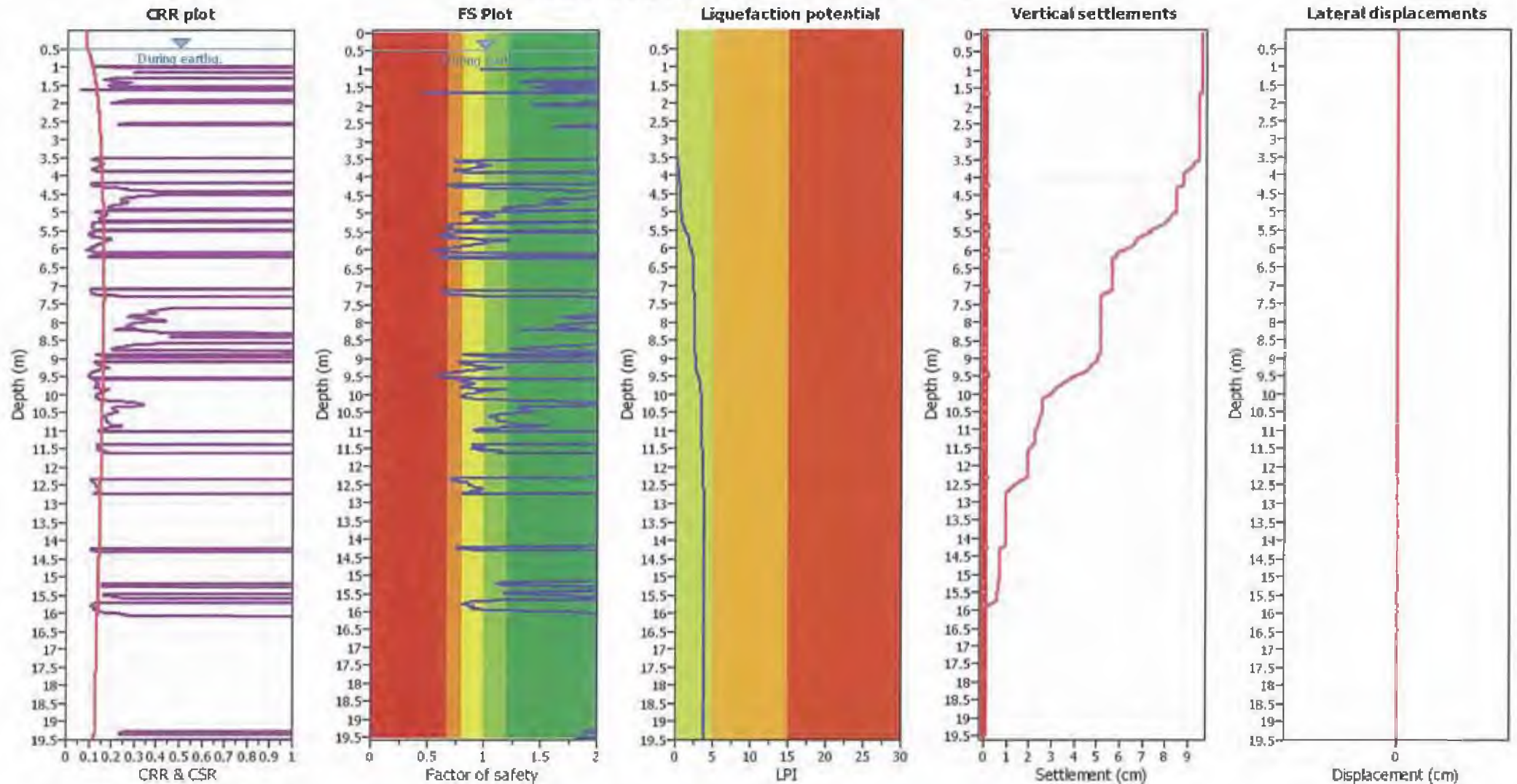
Analysis method:	NCEER (1998)	Depth to water table (enhsq):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.50	K _s applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Serviceability Limit State (SLS): M7.5, 0.13g peak horizontal ground acceleration

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on I_c value	I_c cut-off value:	2.60	K_0 applied:	Yes
Earthquake magnitude M_w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

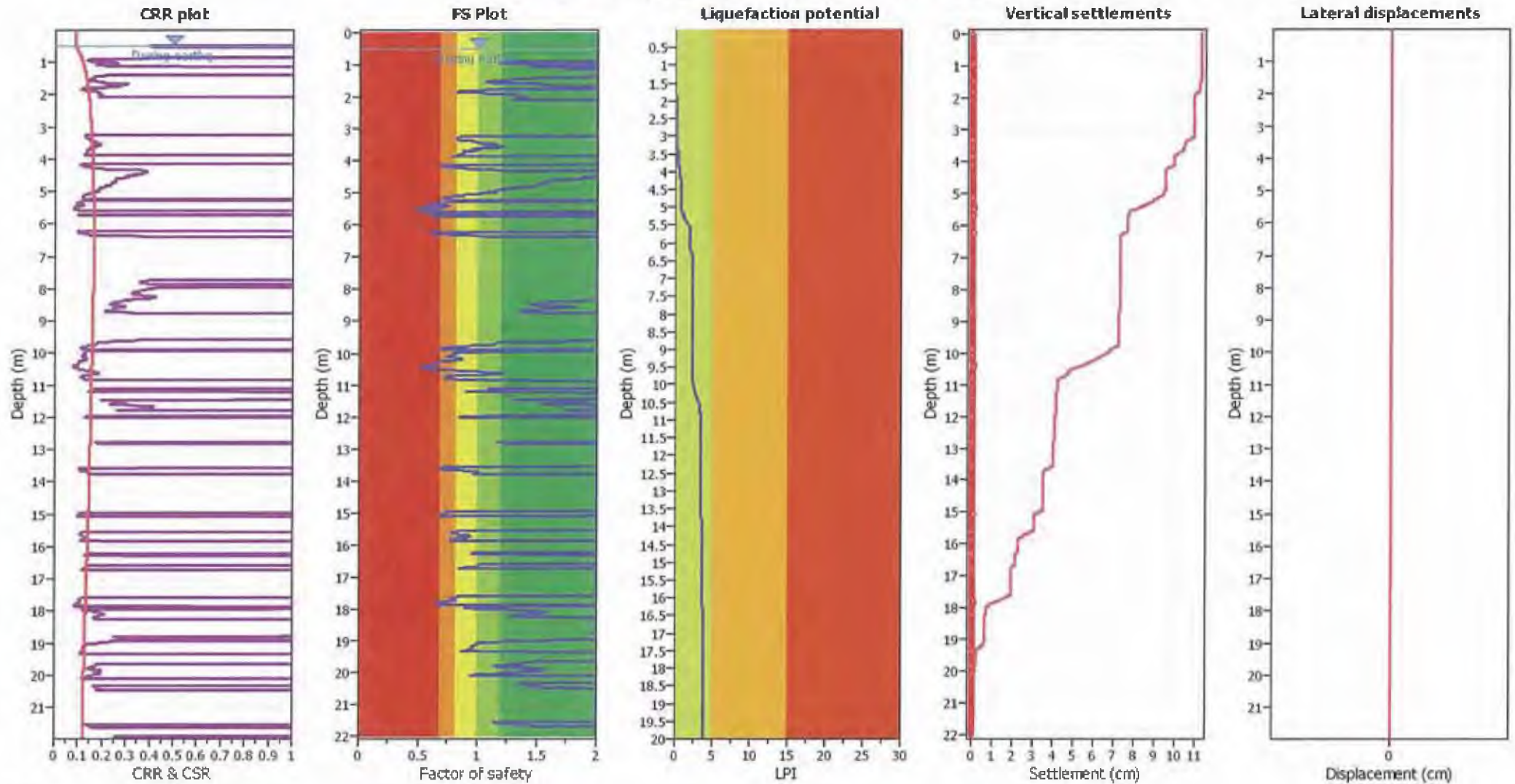
F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (ortho):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _o applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Unit depth:	N/A

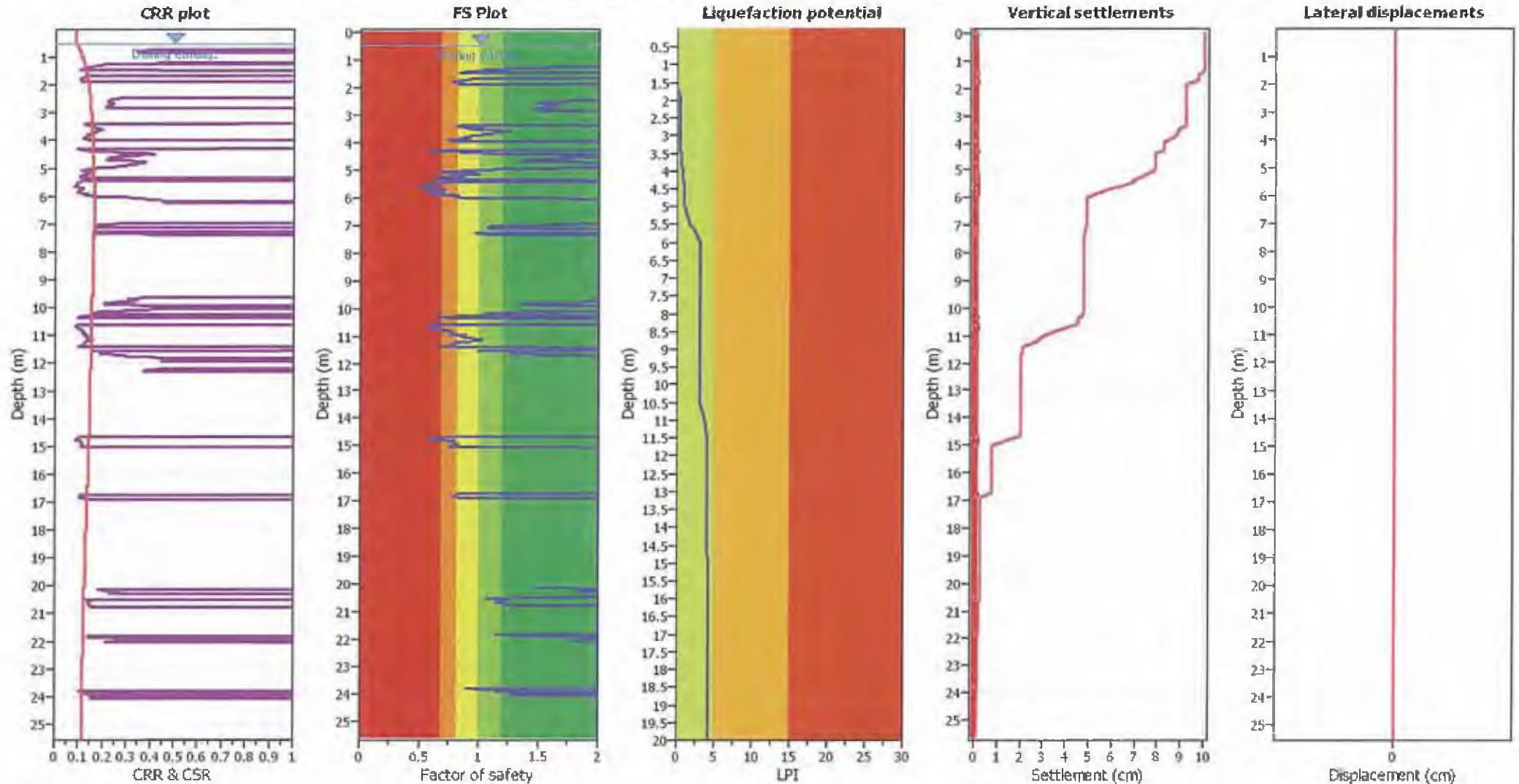
F.S. color scheme

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LPI color scheme

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- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_v applied:	Yes
Earthquake magnitude M_w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (instq):	2.50 m	Fill height:	N/A	Limit depth:	N/A

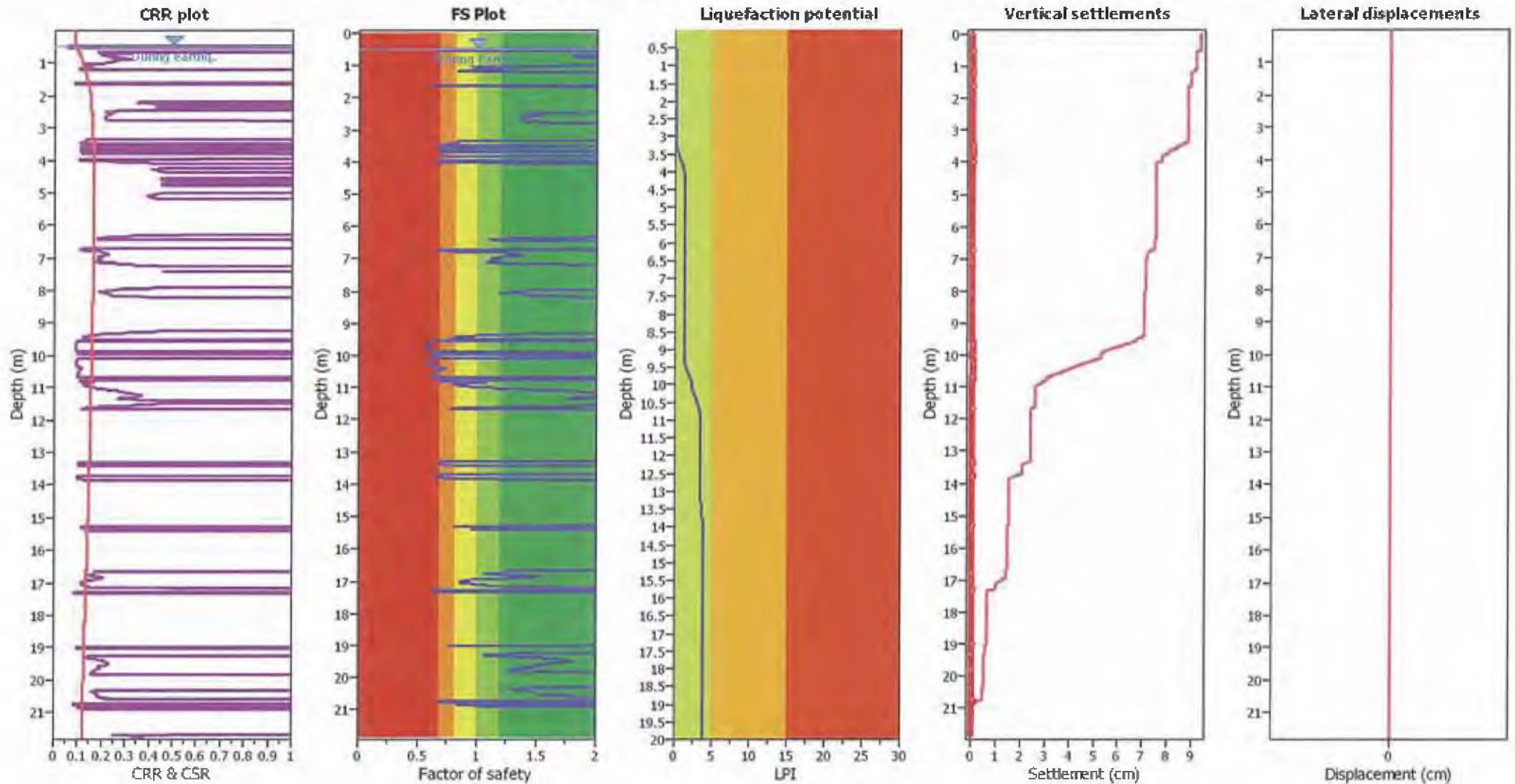
F.S. color scheme

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LPI color scheme

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- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (ortho.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _v applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Unit depth:	N/A

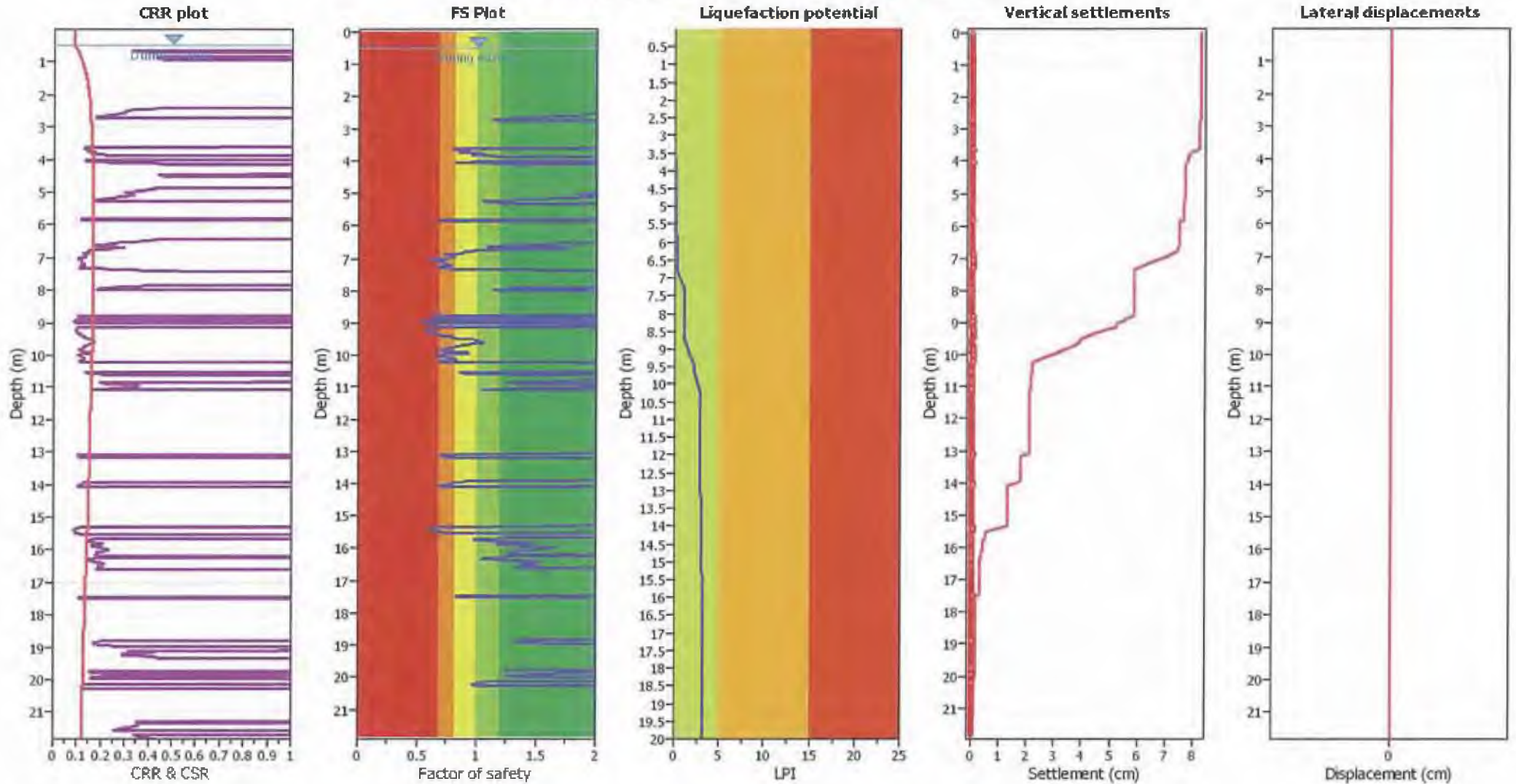
F.S. color scheme

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LPI color scheme

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- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_0 applied:	Yes
Earthquake magnitude M_w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

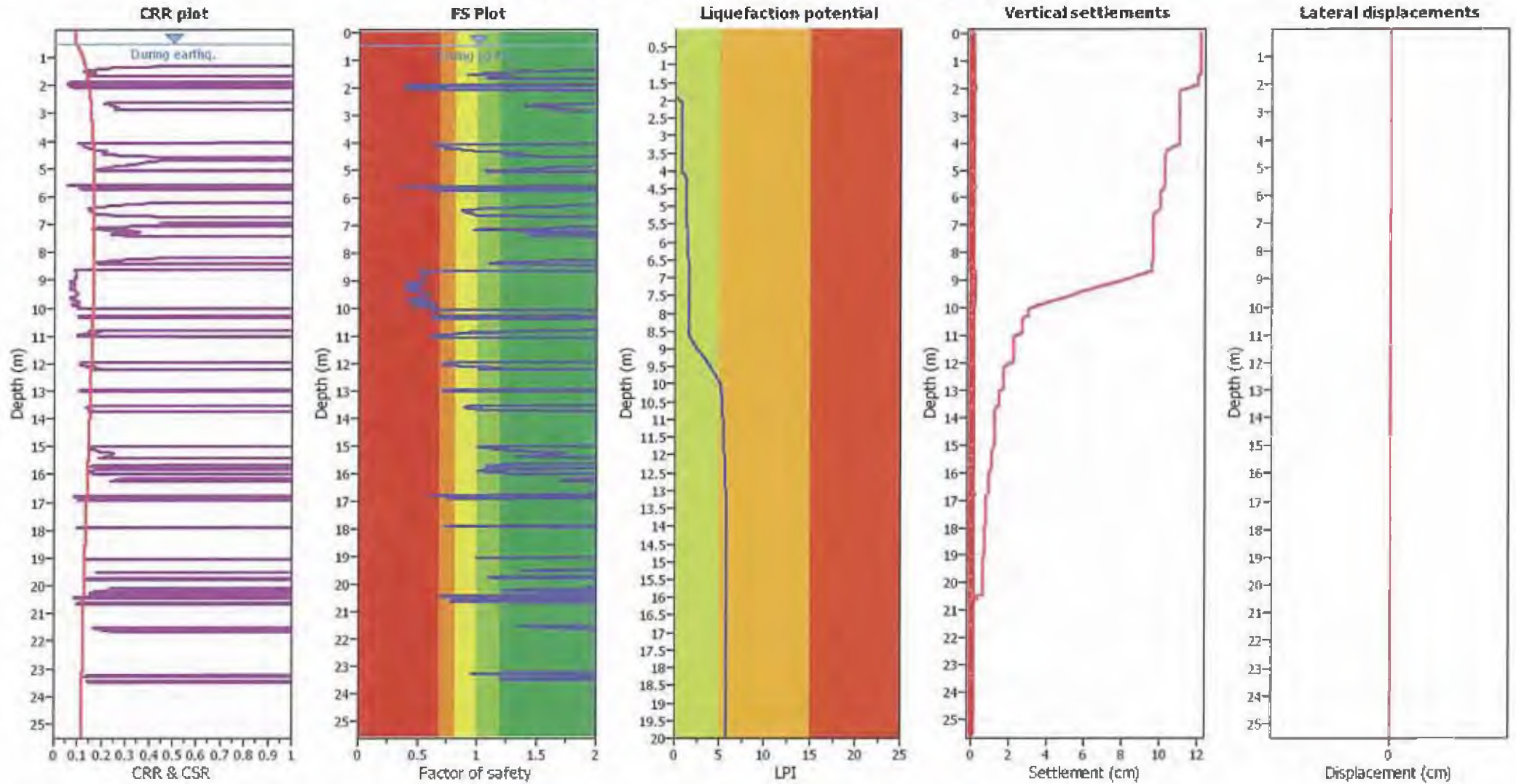
F.S. color scheme

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LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

F.5. color scheme

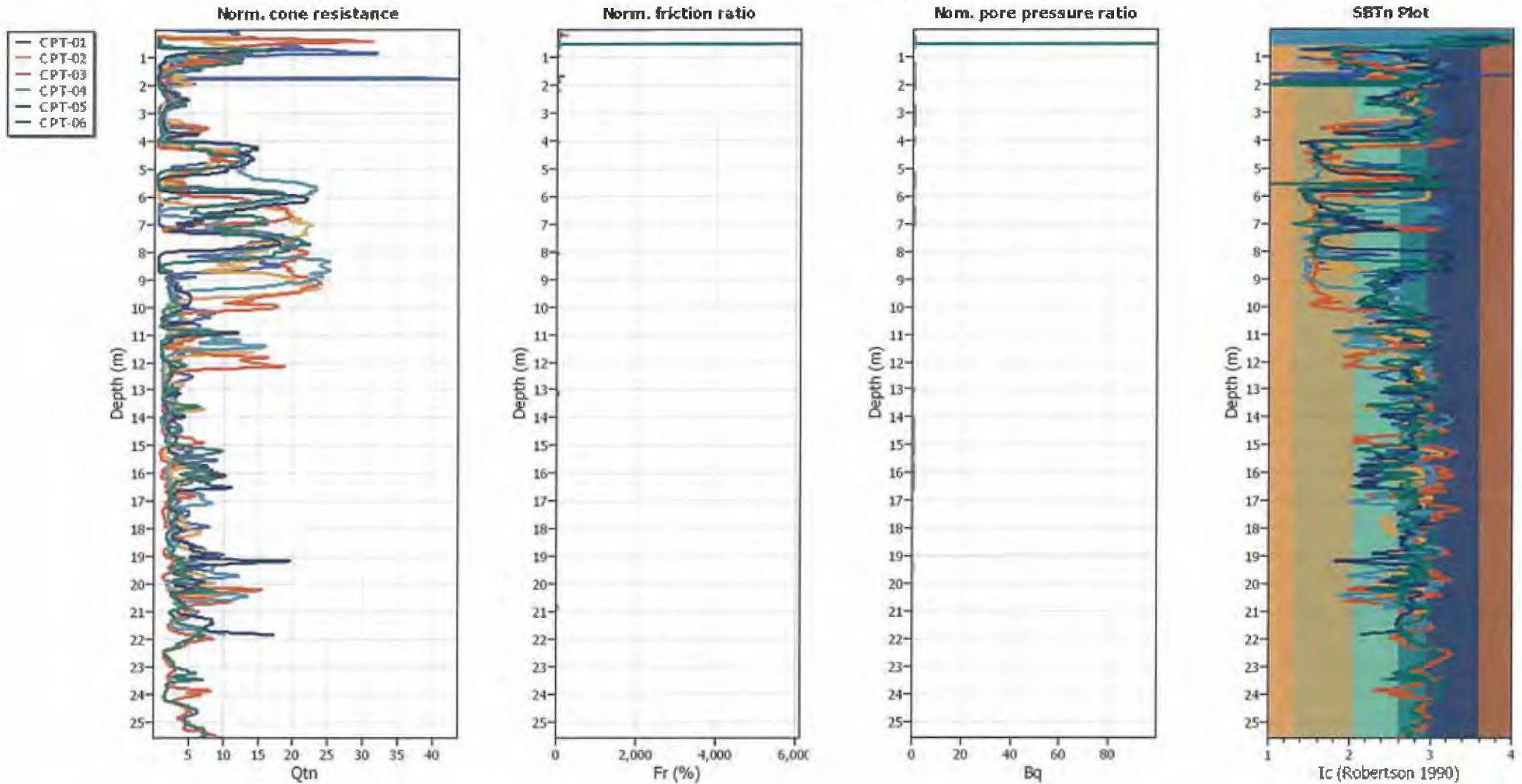
- Almost certain it will liquefy
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- Liquefaction and no liquefaction are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

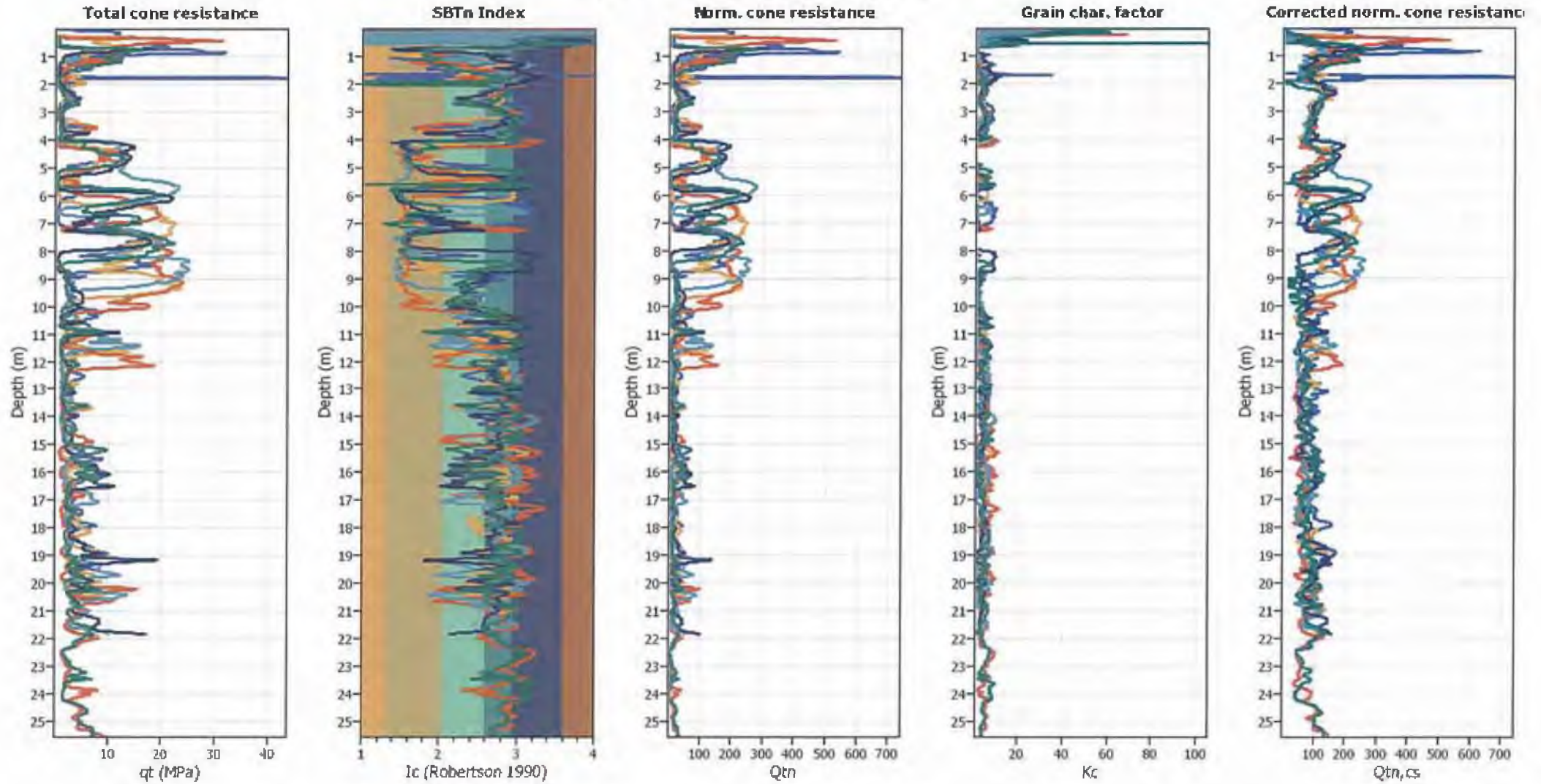
Project:

Overlay Normalized Plots



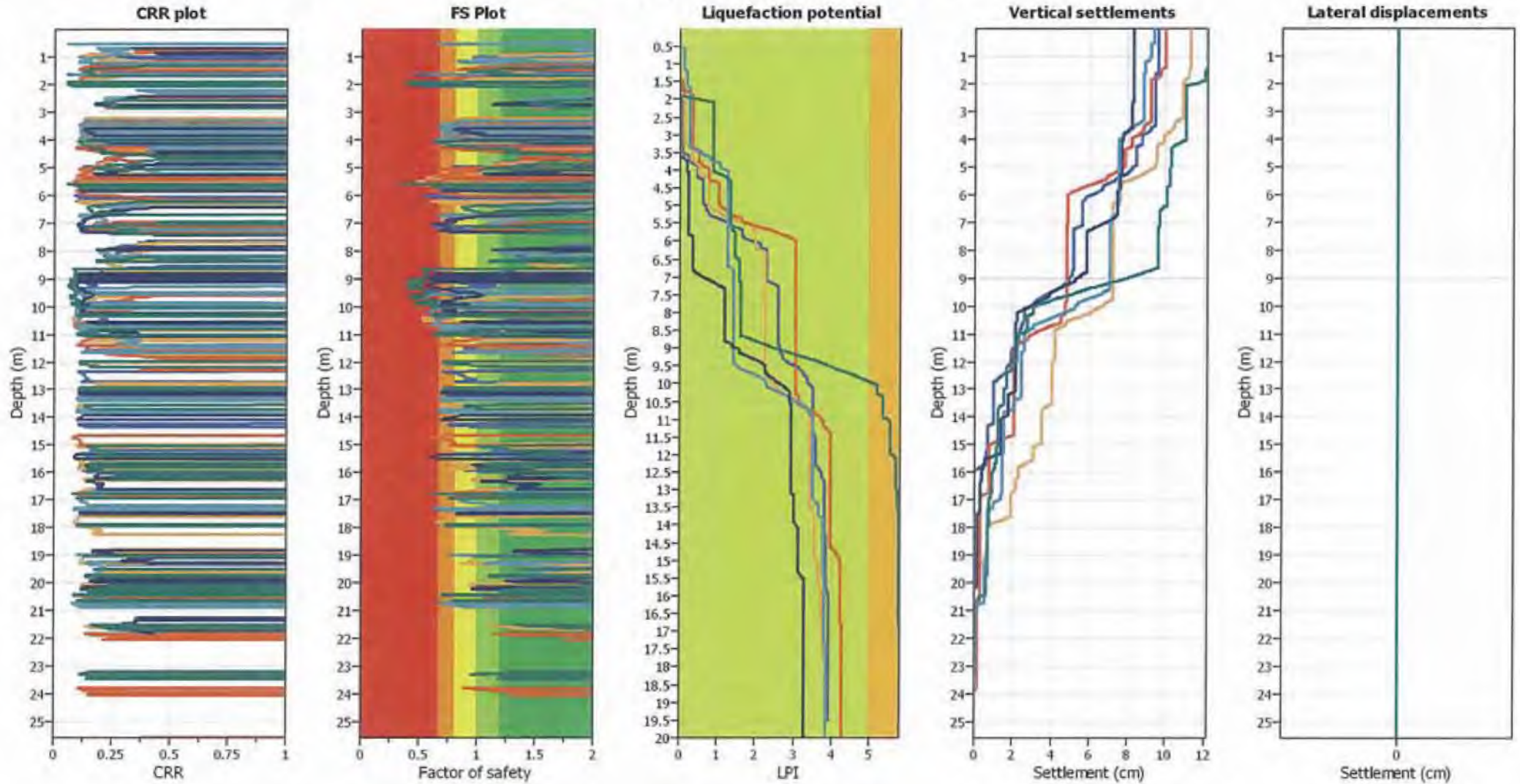
Project:

Overlay Intermediate Results



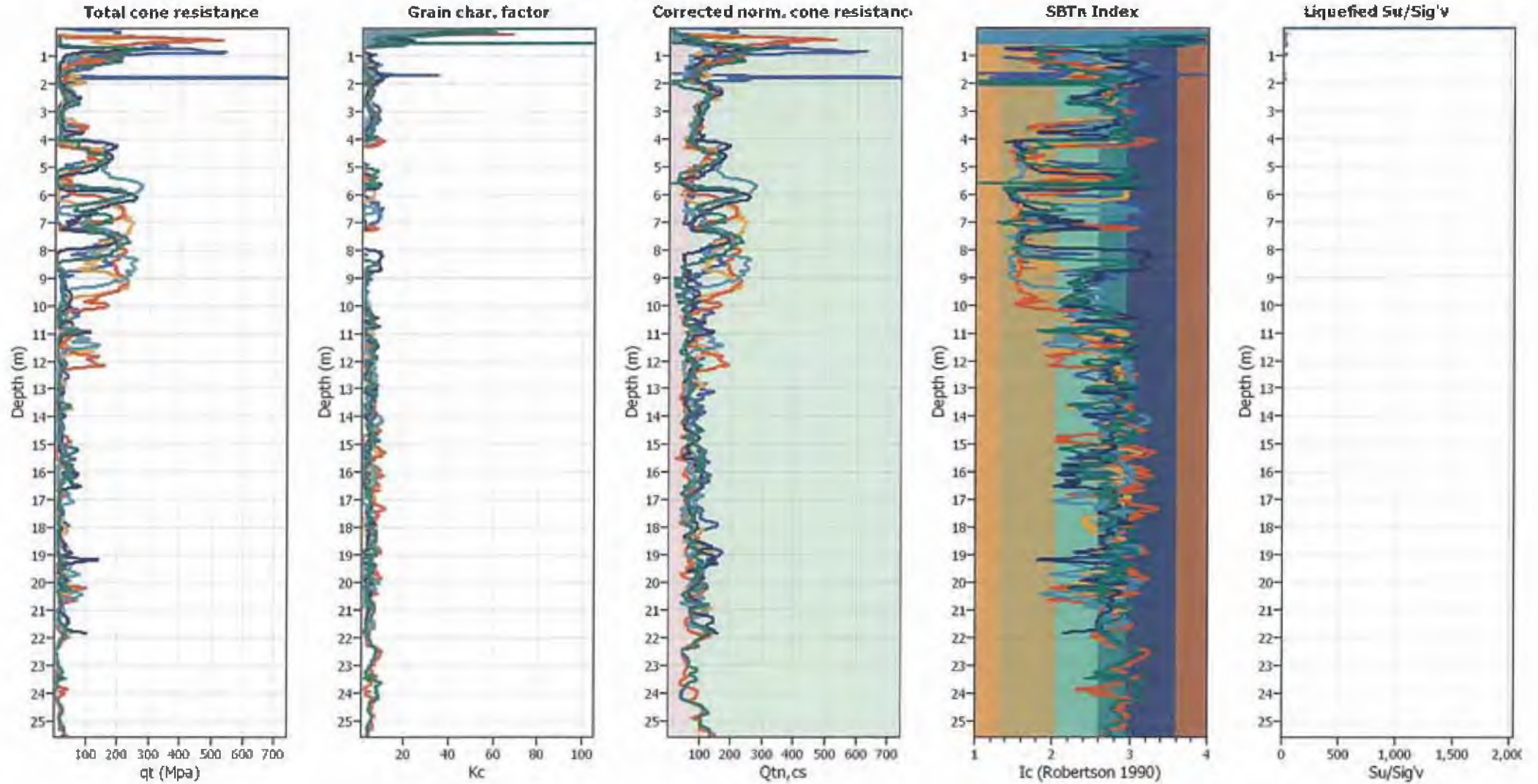
Project:

Overlay Cyclic Liquefaction Plots



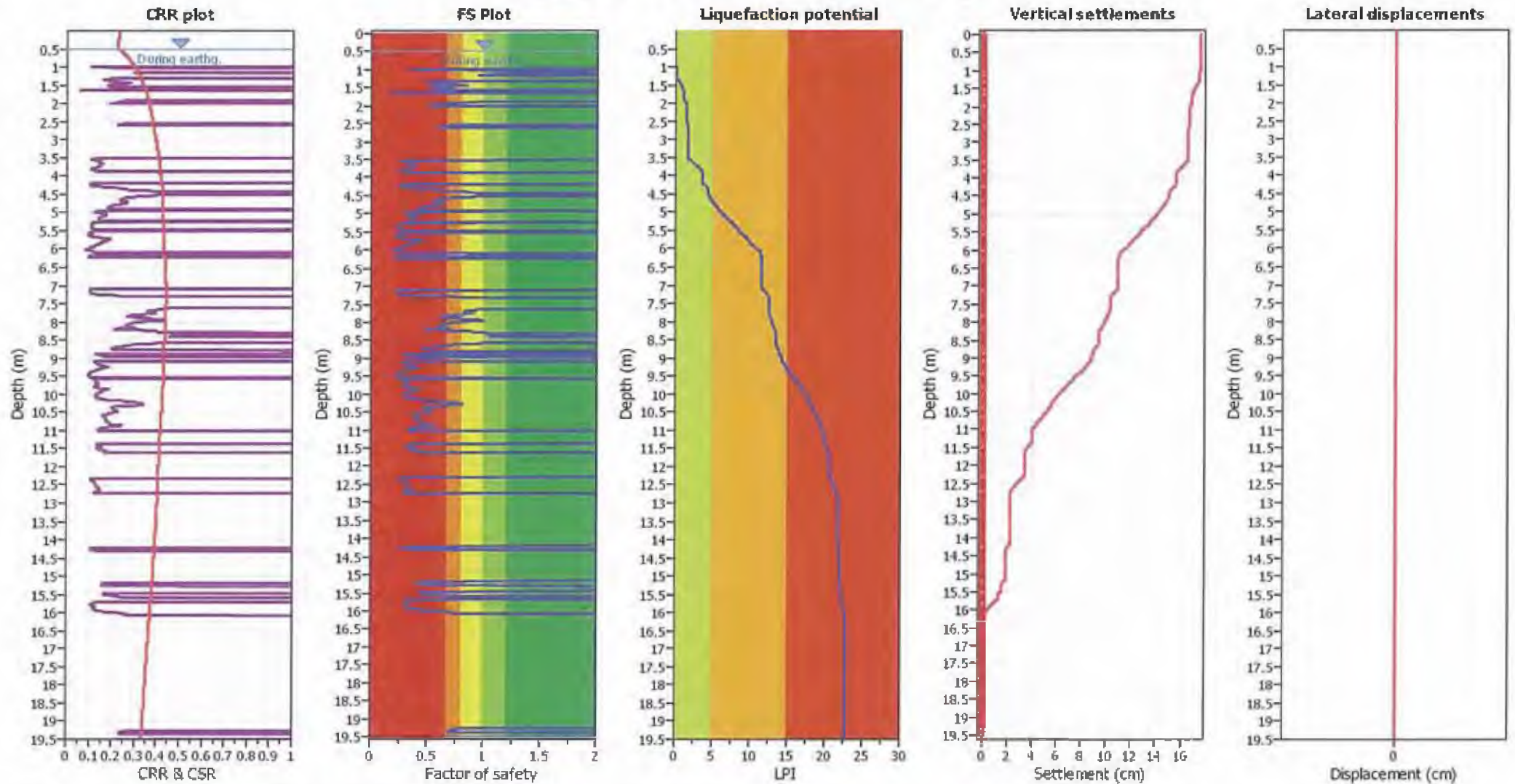
Project:

Overlay Strength Loss Plots



Ultimate Limit State (ULS): M7.5, 0.35g peak horizontal ground acceleration

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

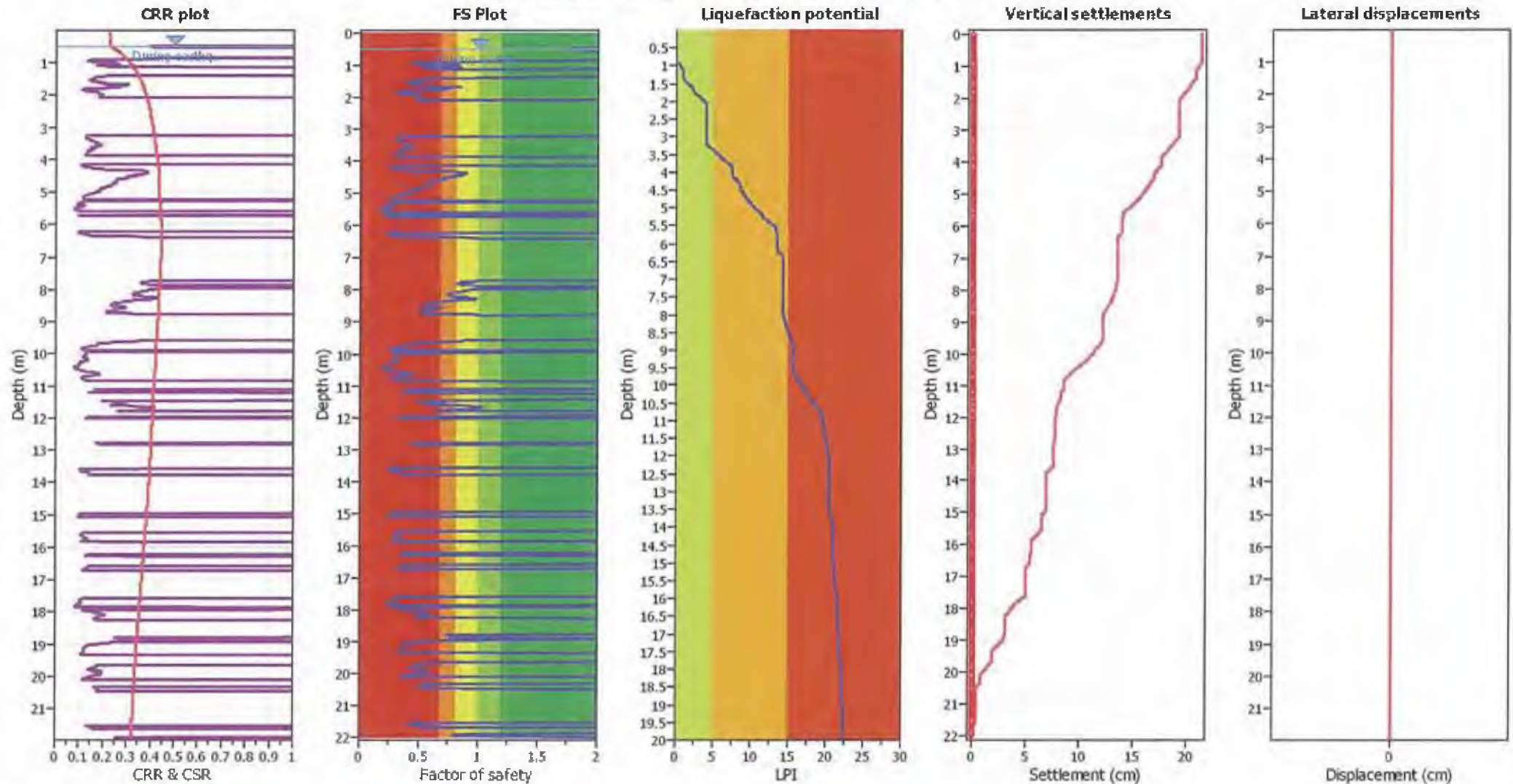
F.S. color scheme

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LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on I_c value	I_c cut-off value:	2.60	K_o applied:	Yes
Earthquake magnitude M_w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	No
Depth to water table (in situ):	2.50 m	Fill height:	N/A	Unit depth:	N/A

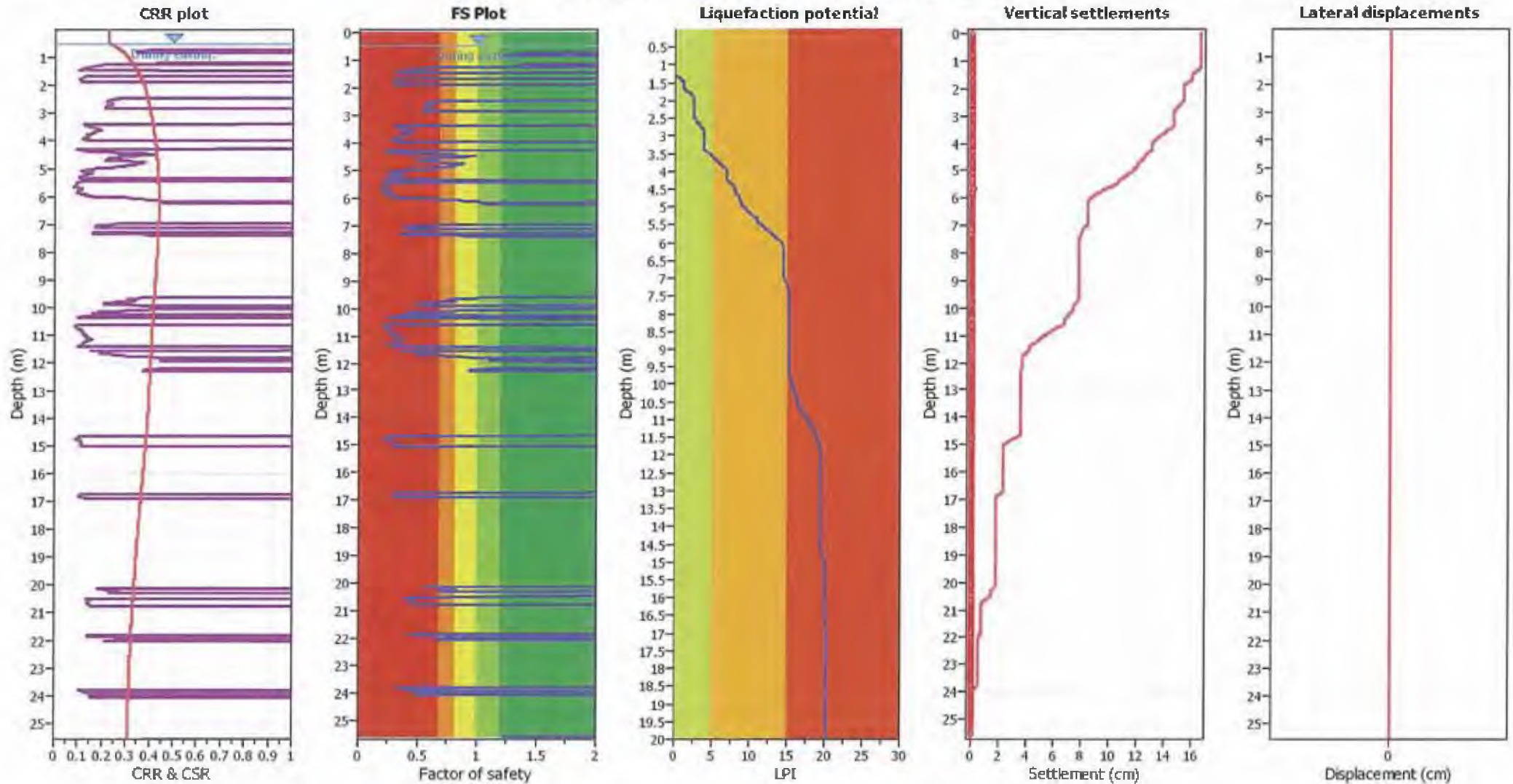
F.S. color scheme

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LPI color scheme

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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

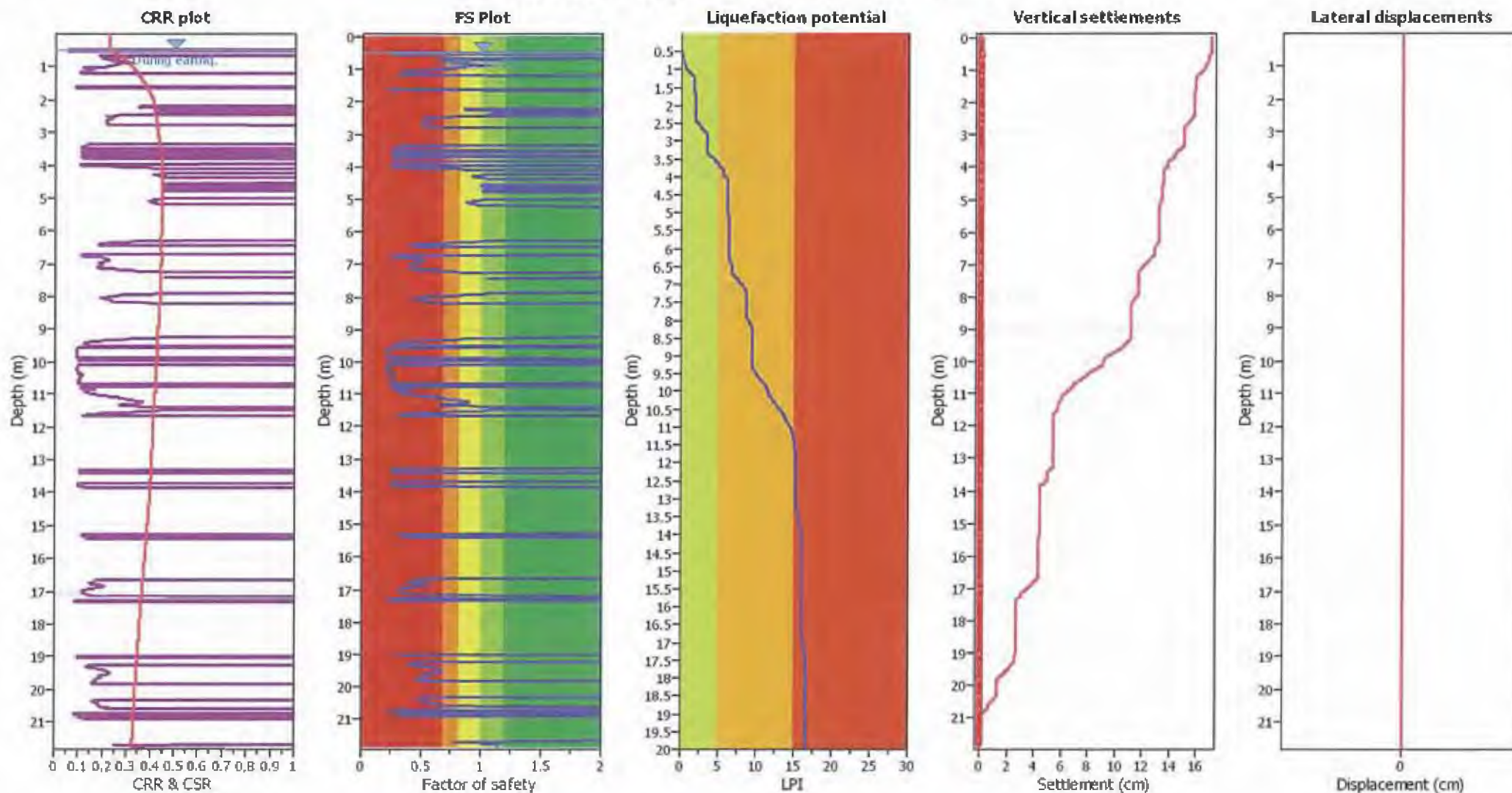
F.S. color scheme

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LPI color scheme

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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on I_c value	LC cut-off value:	2.60	K_v applied:	Yes
Earthquake magnitude M_w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

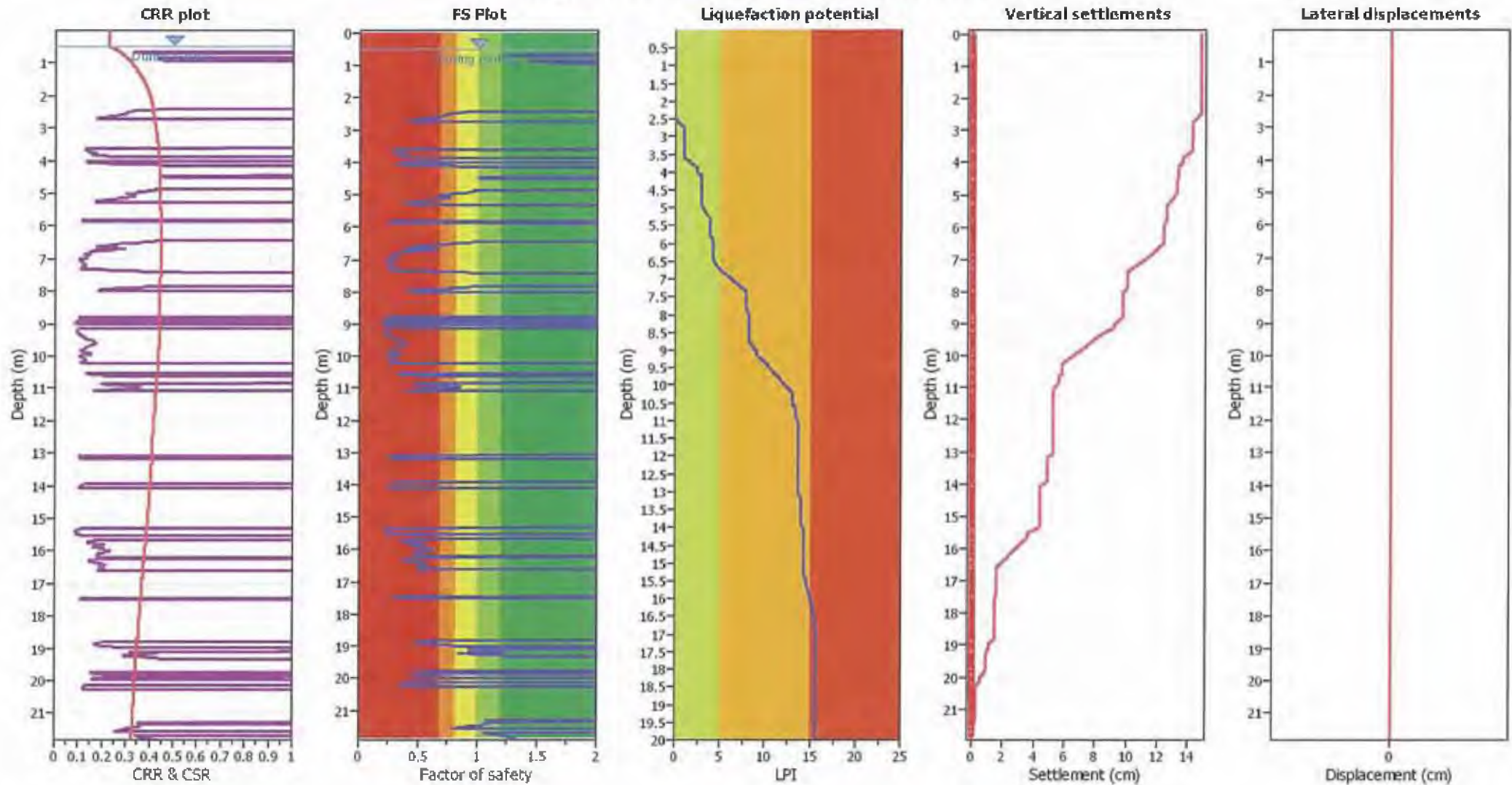
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LPI color scheme

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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Flines correction method:	NCEER (1998)	Average results interval:	3	Transition detect, applied:	No
Points to test:	Based on t_c value	t_c cut-off value:	2.60	K_0 applied:	Yes
Earthquake magnitude M_w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	No
Depth to water table (instku):	2.50 m	Fill height:	N/A	Limit depth:	N/A

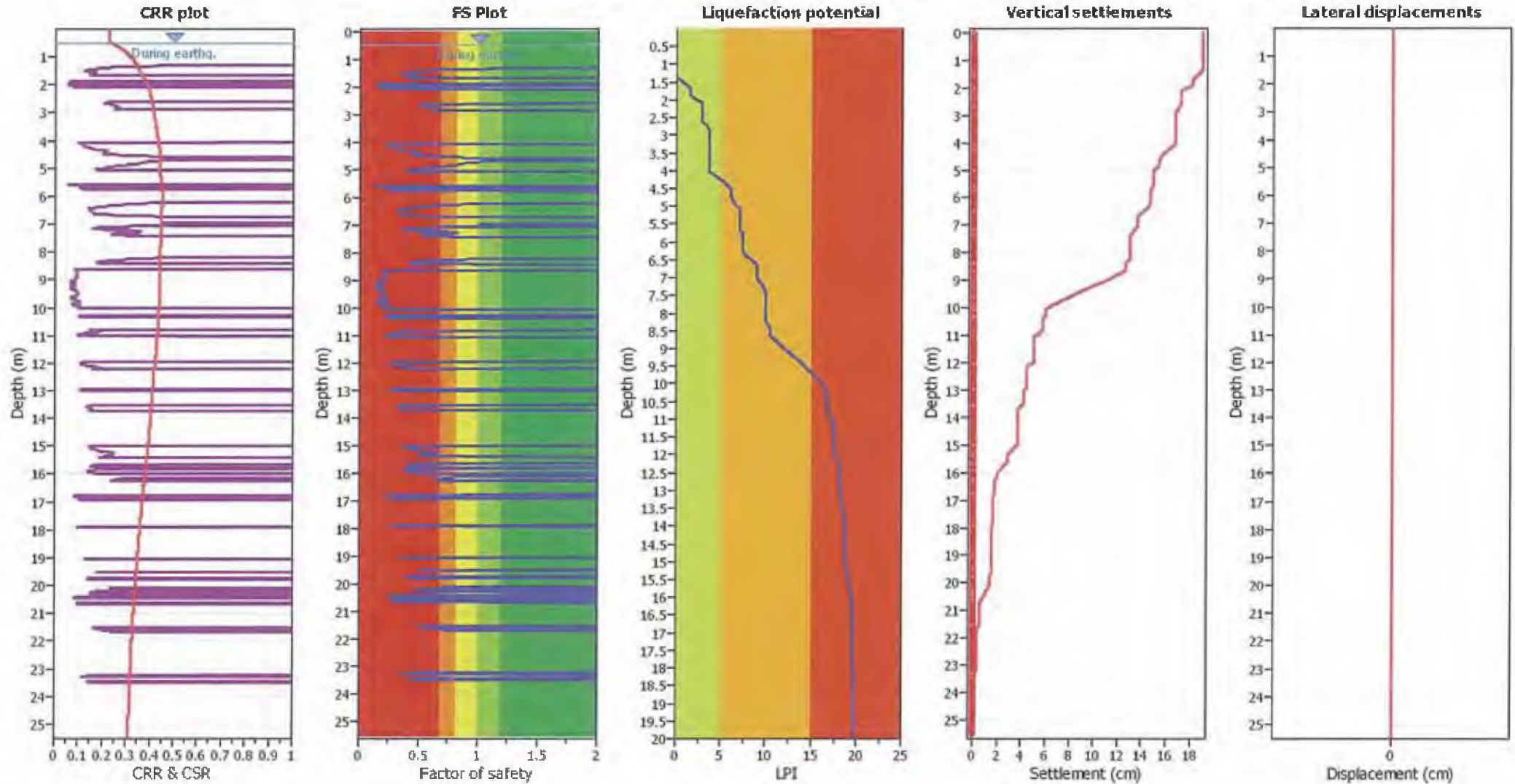
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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Flines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on I _c value	I _c cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

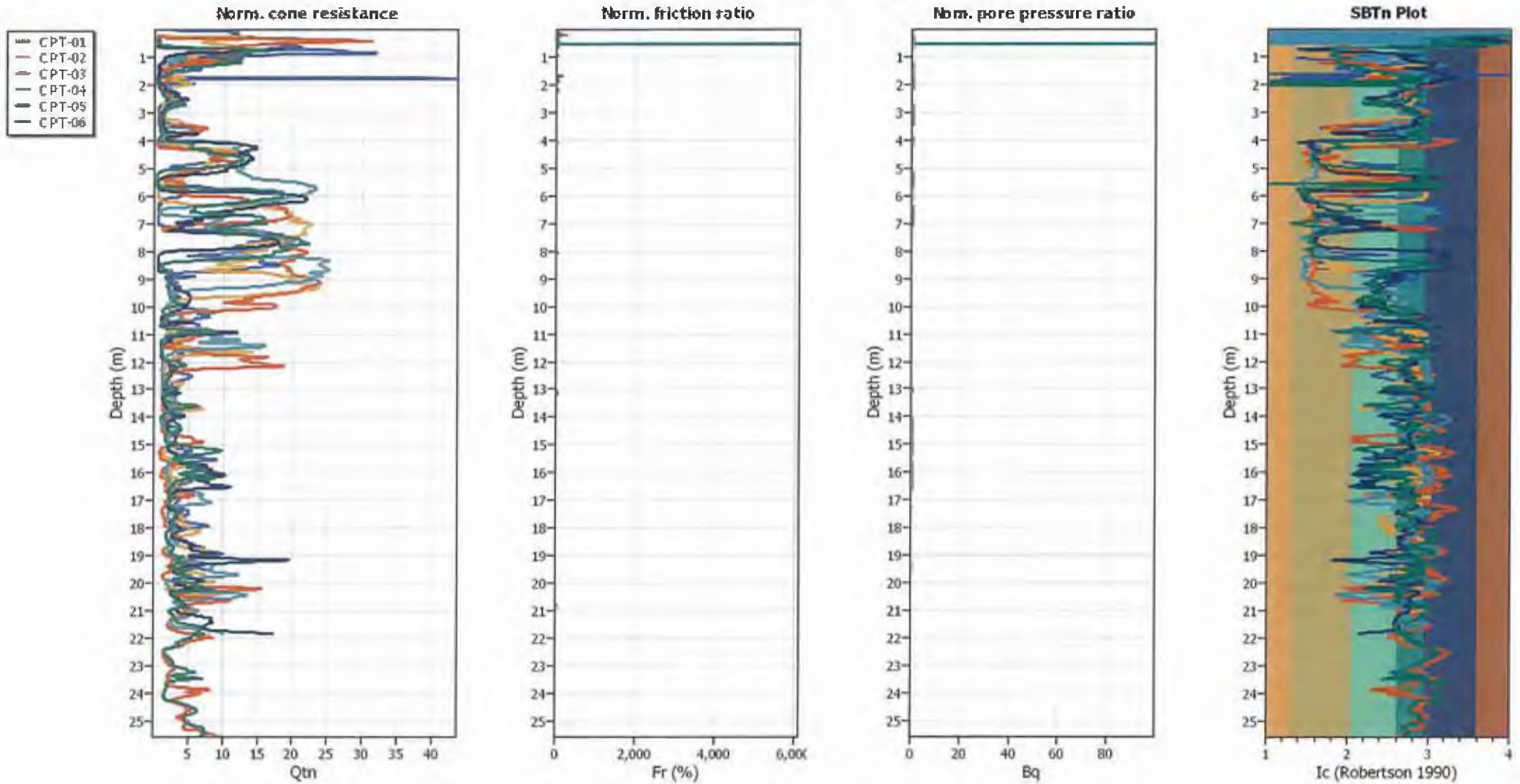
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LPI color scheme

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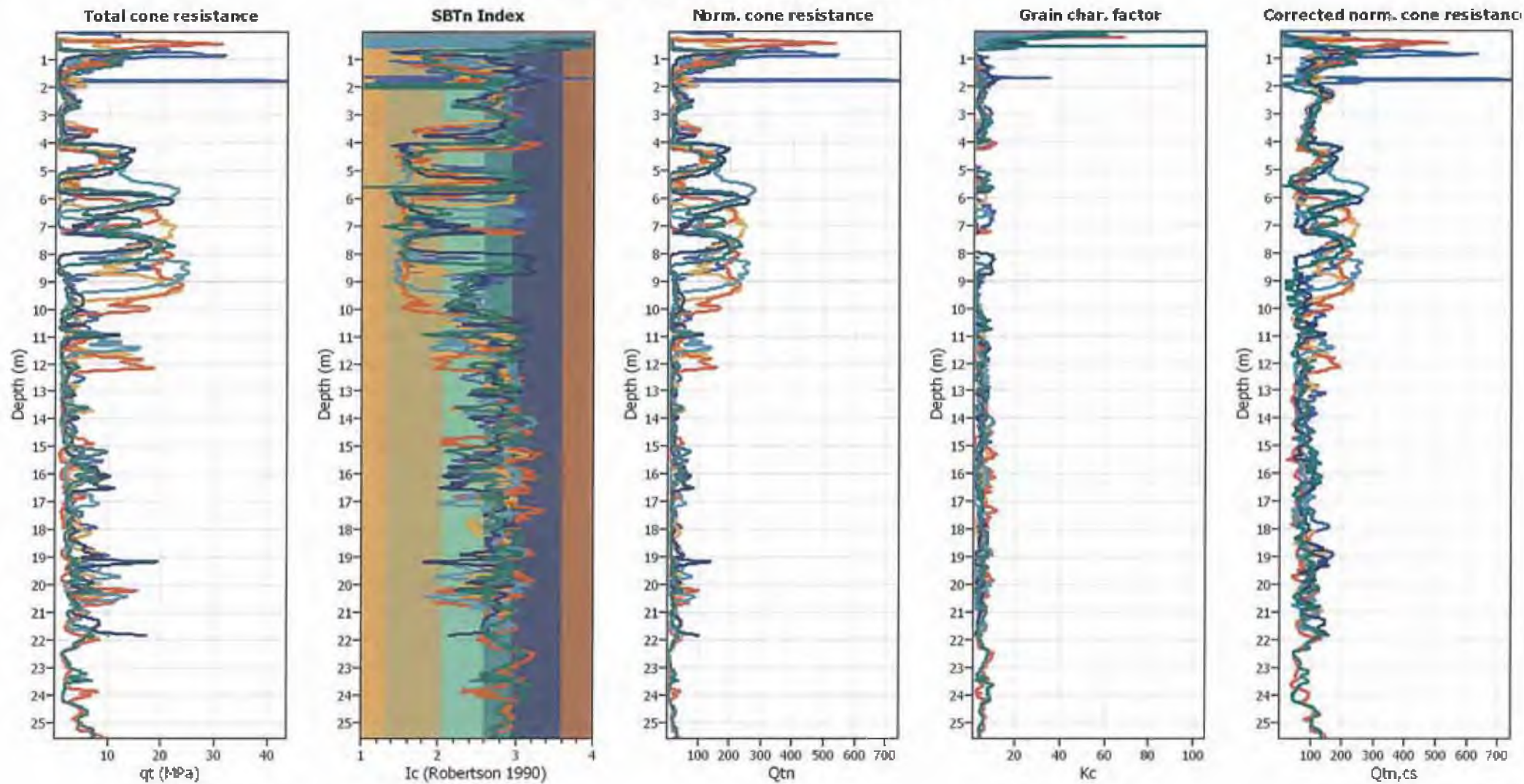
Project:

Overlay Normalized Plots



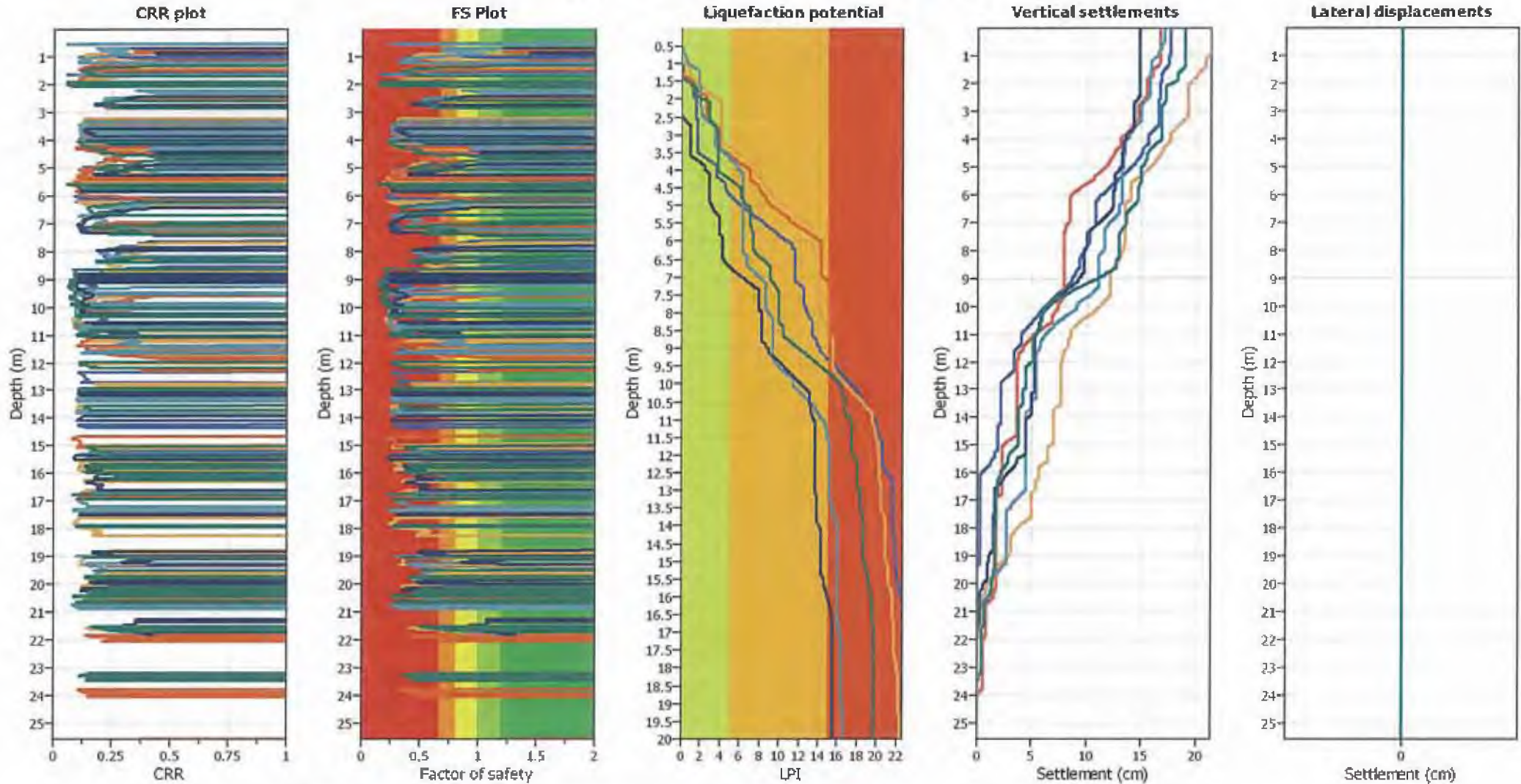
Project:

Overlay Intermediate Results



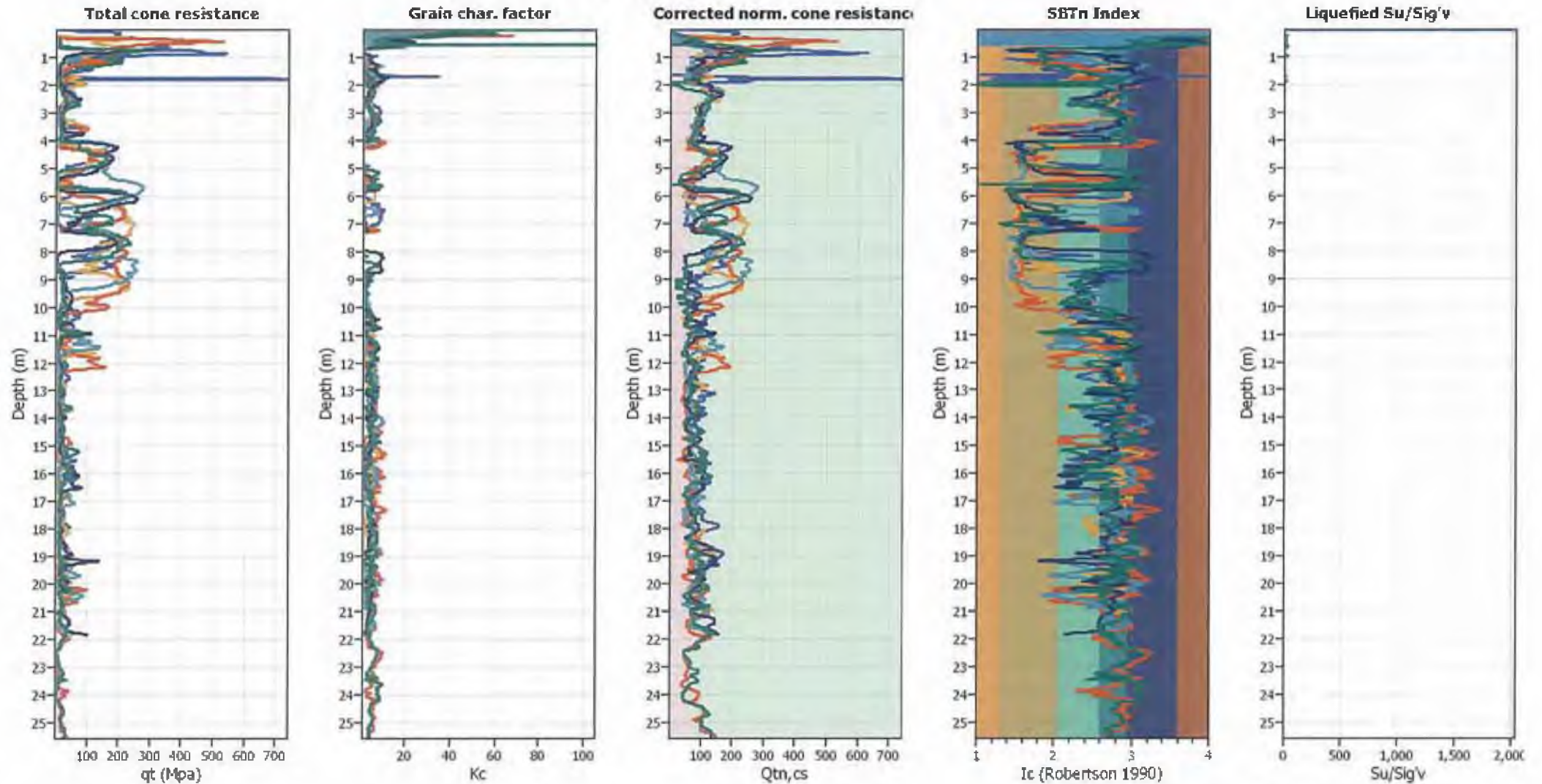
Project:

Overlay Cyclic Liquefaction Plots



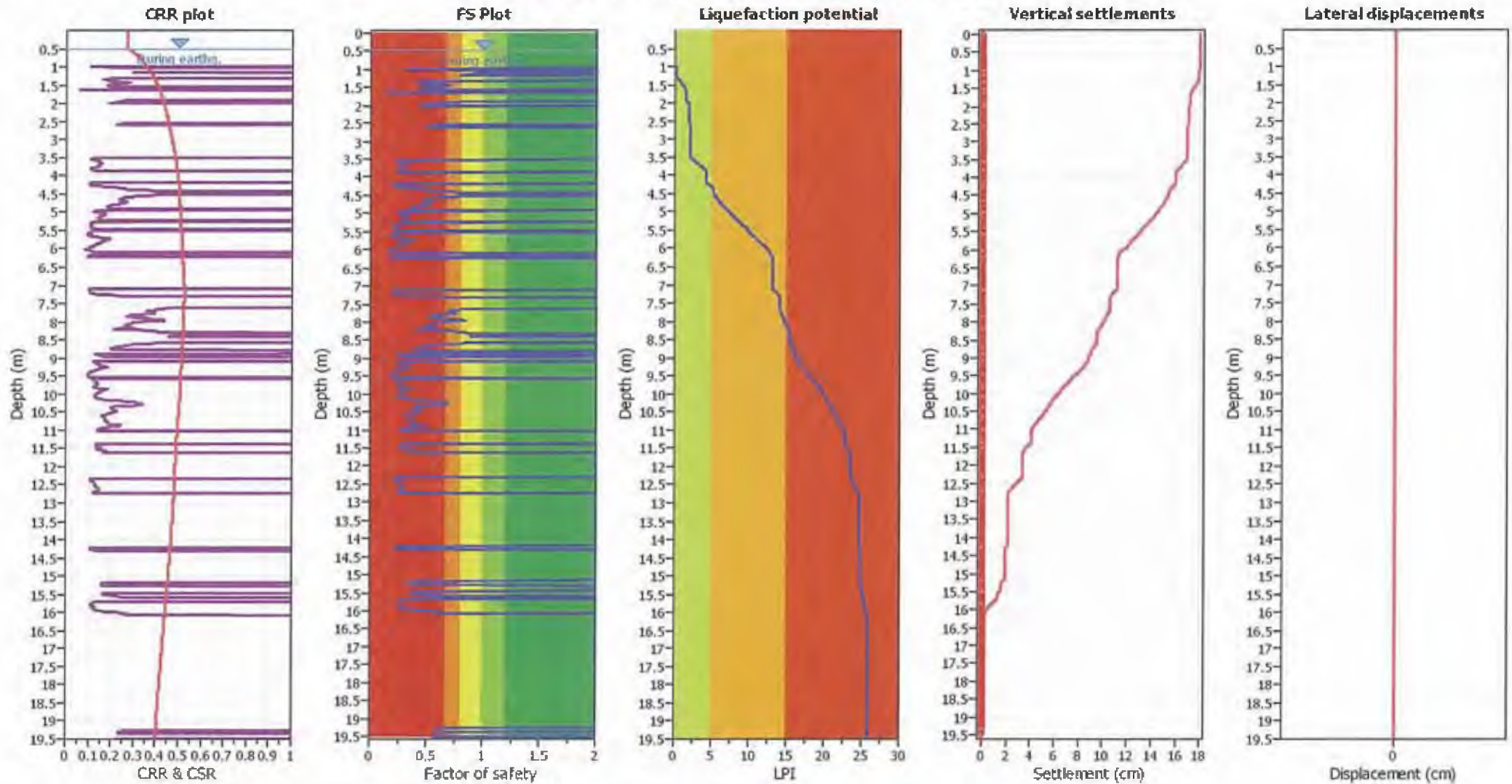
Project:

Overlay Strength Loss Plots



22 February 2011 M6.3, 0.65 g peak horizontal ground acceleration

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	6.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (static):	2.50 m	Fill height:	N/A	Limit depth:	N/A

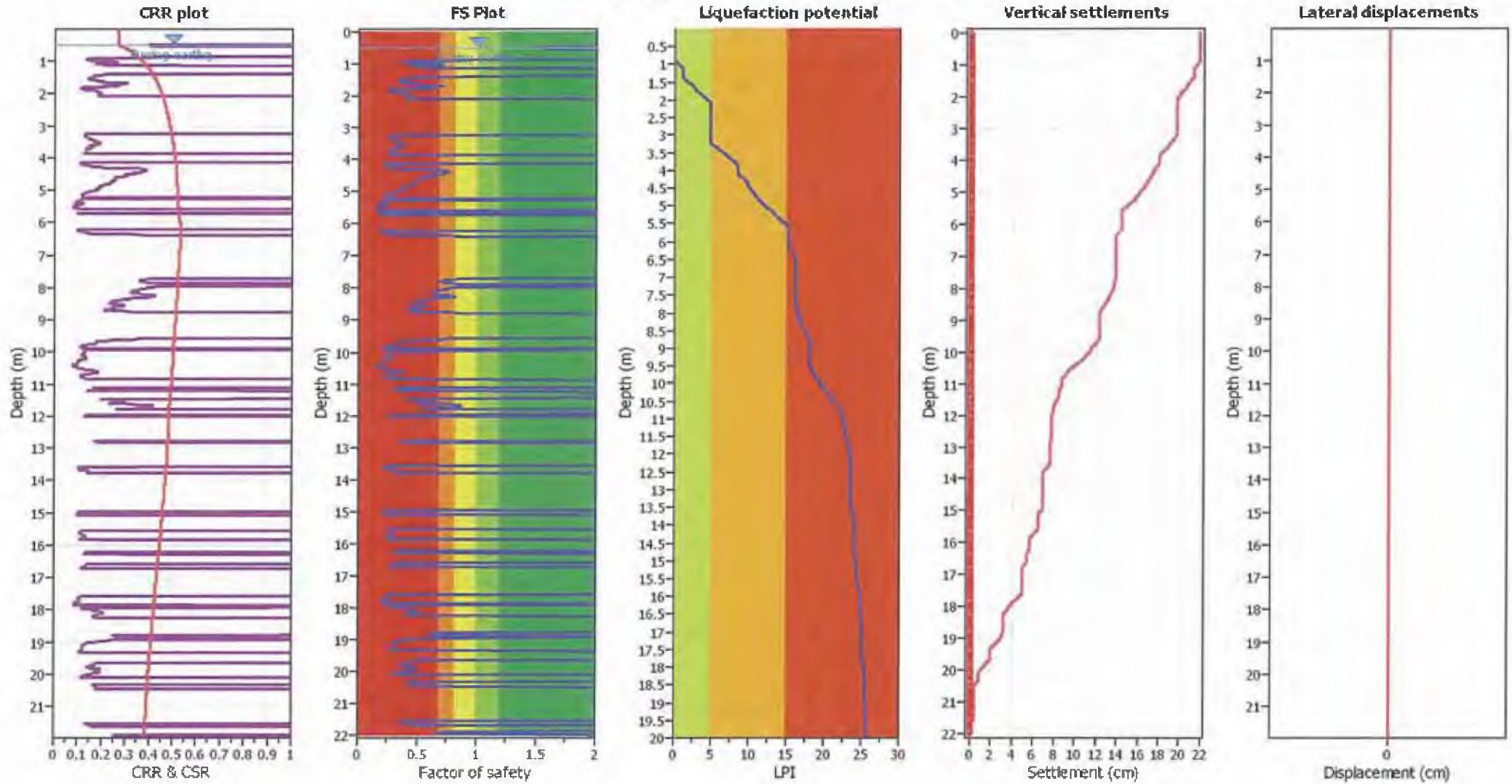
F.S. color scheme

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LPI color scheme

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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect, applied:	No
Points to test:	Based on I _c value	I _c cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	6.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

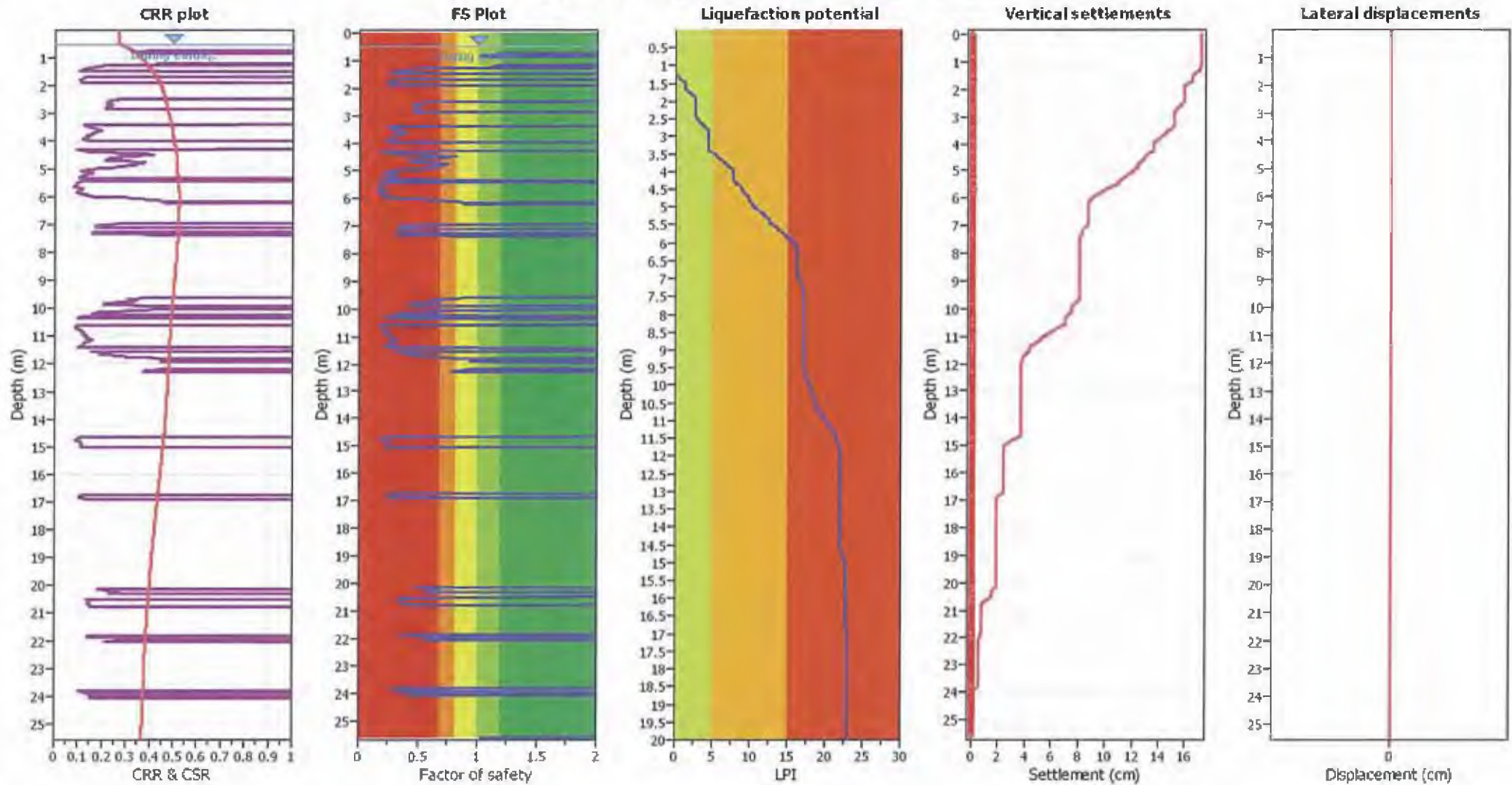
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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on I_c value	I_c cut-off value:	2.60	K_v applied:	Yes
Earthquake magnitude M_w :	6.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (Insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

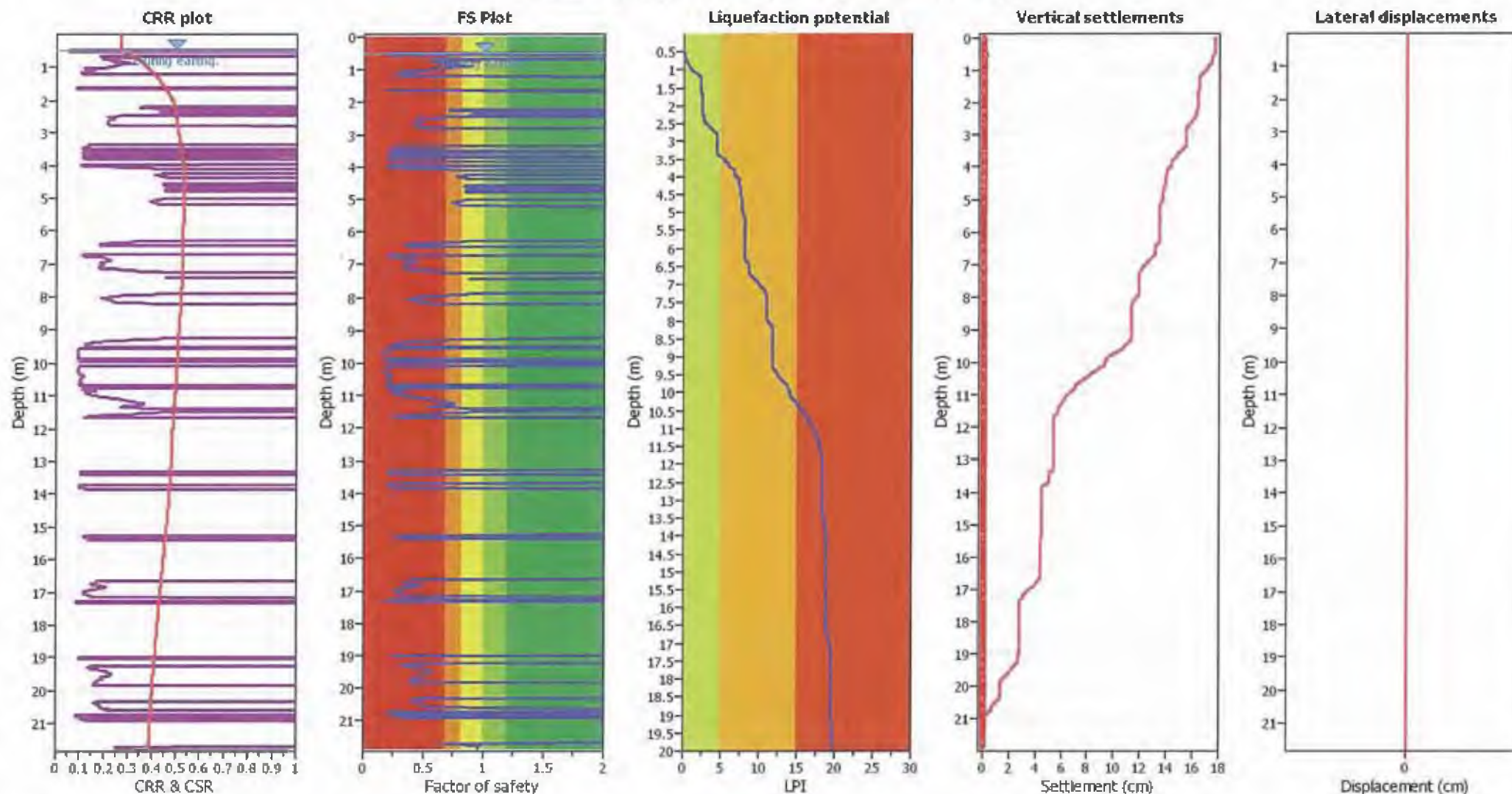
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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect, applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _s applied:	Yes
Earthquake magnitude M _w :	6.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (Insitu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

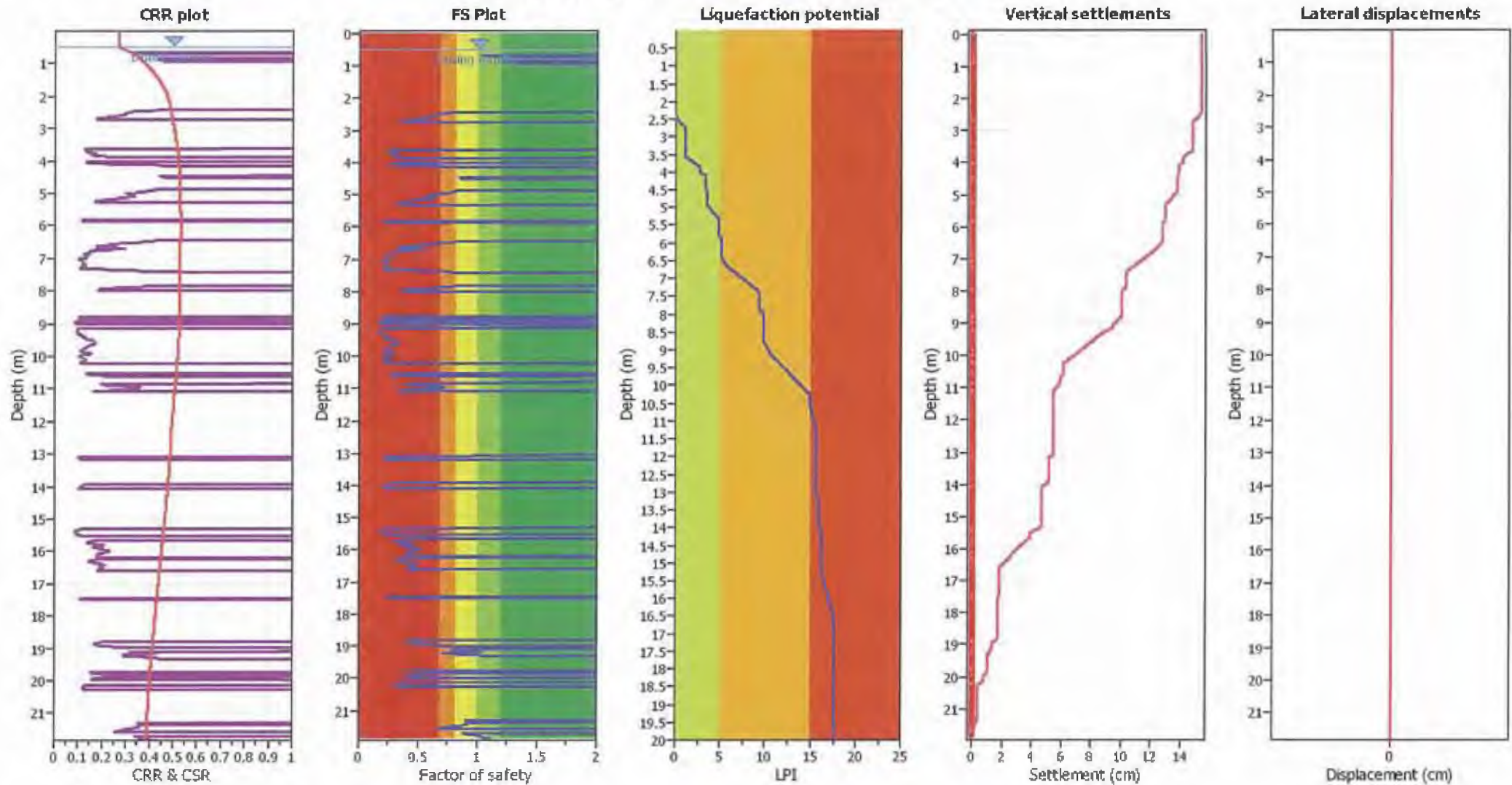
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LPI color scheme

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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _v applied:	Yes
Earthquake magnitude M _w :	6.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.65	Use fill:	No	Limit depth applied:	No
Depth to water table (lnsktu):	2.50 m	Fill height:	N/A	Limit depth:	N/A

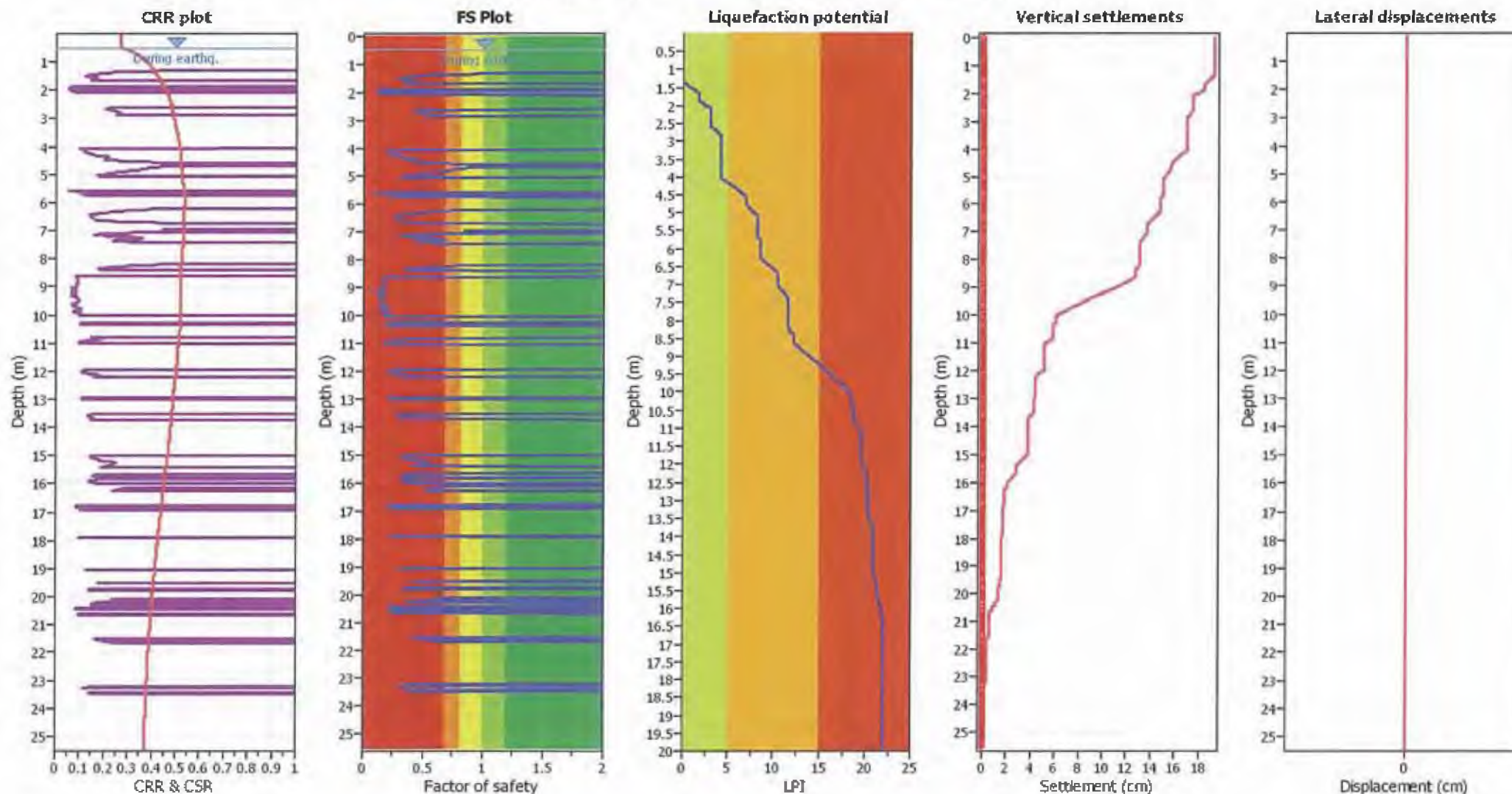
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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	0.50 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_v applied:	Yes
Earthquake magnitude M_w :	6.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.65	Use fill:	No	Link depth applied:	No
Depth to water table (instk):	2.50 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

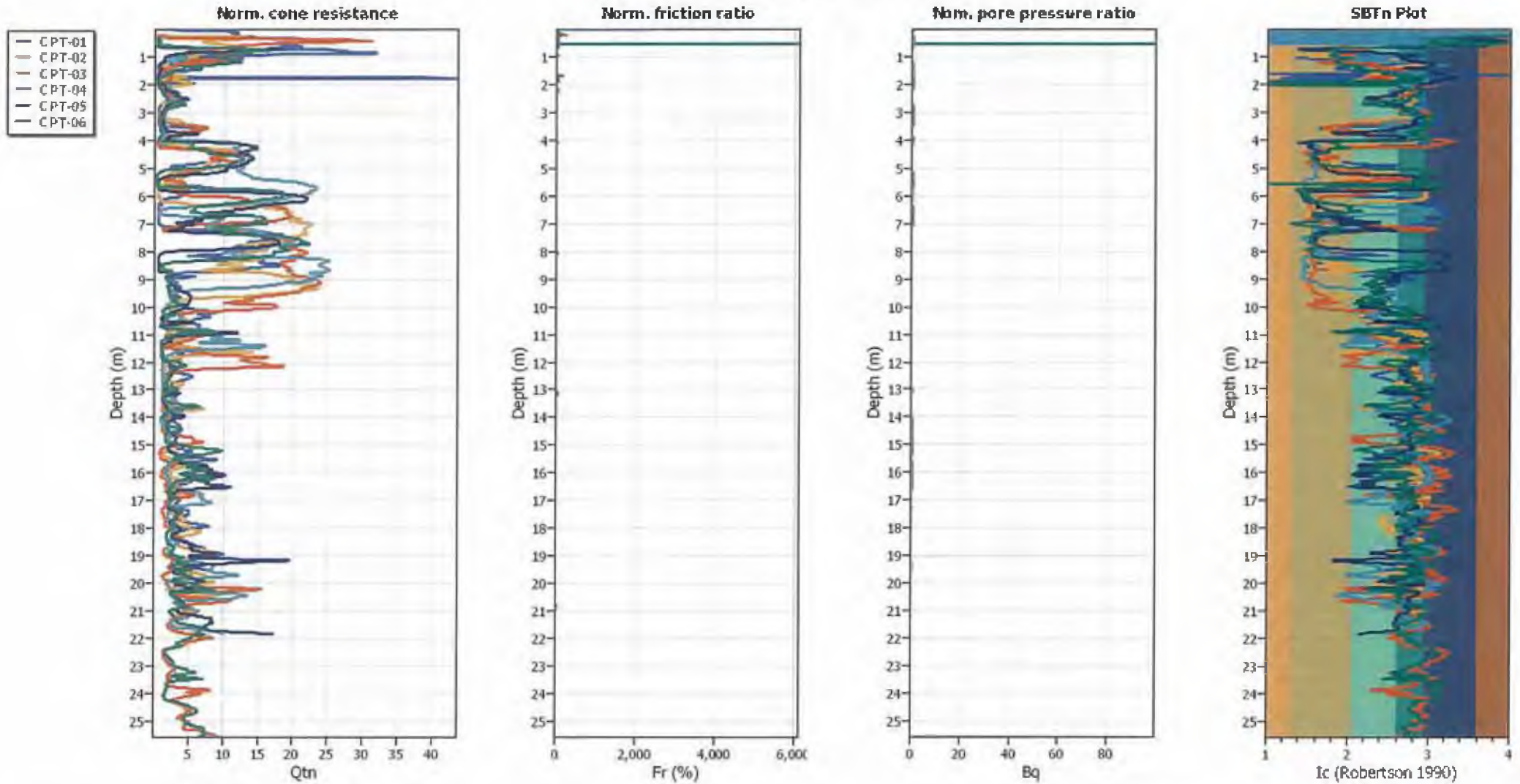
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liquefaction are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

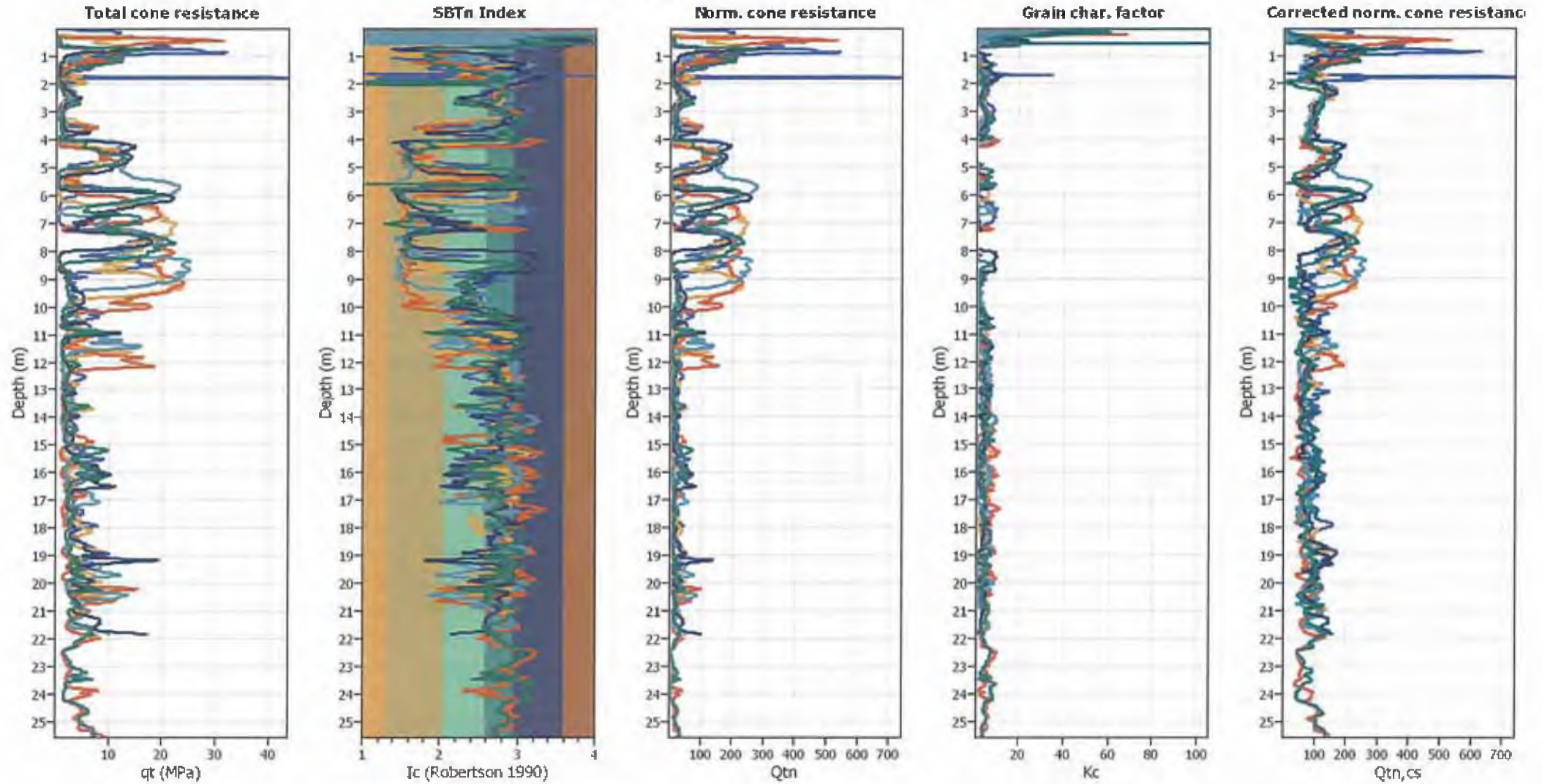
Project:

Overlay Normalized Plots



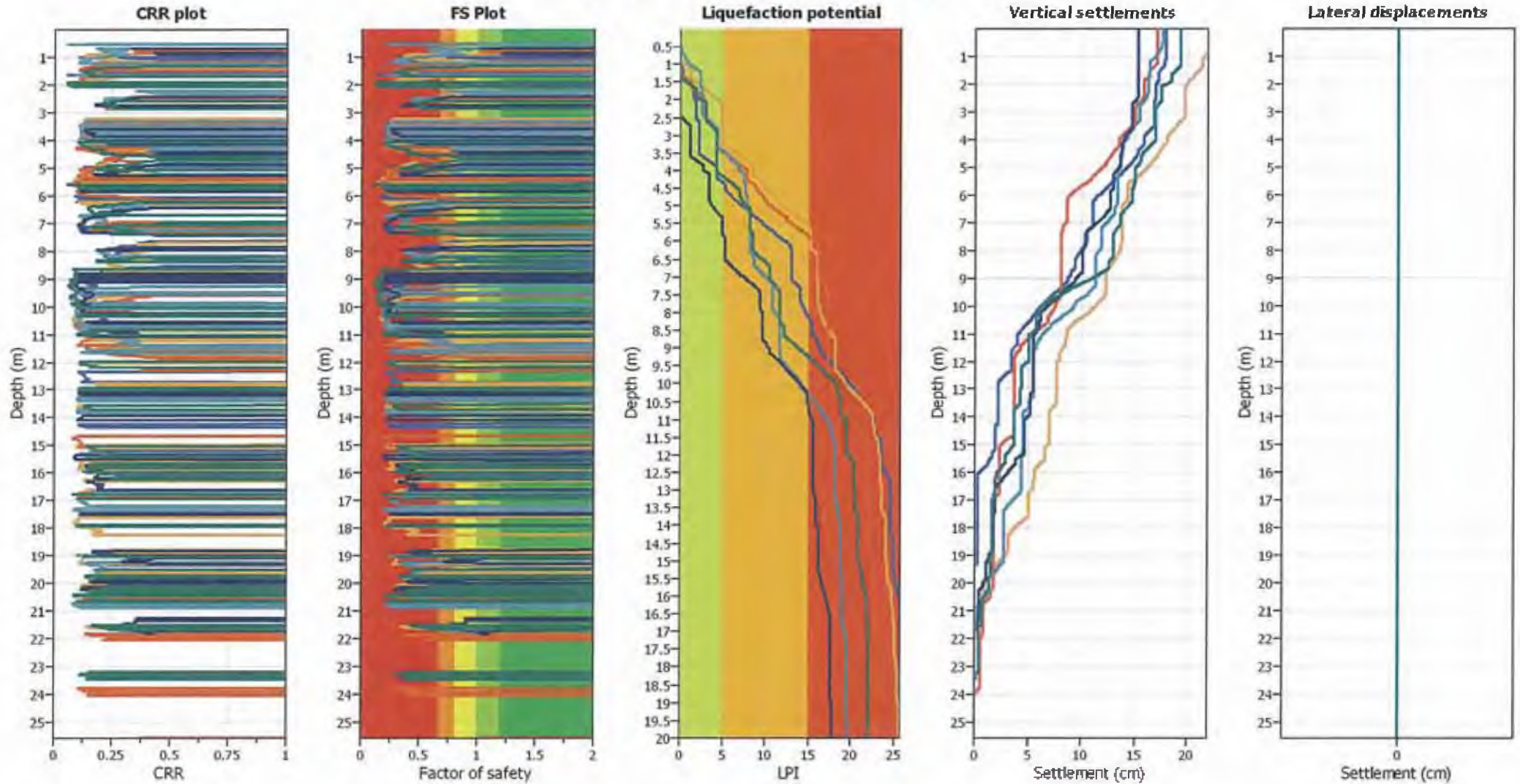
Project:

Overlay Intermediate Results



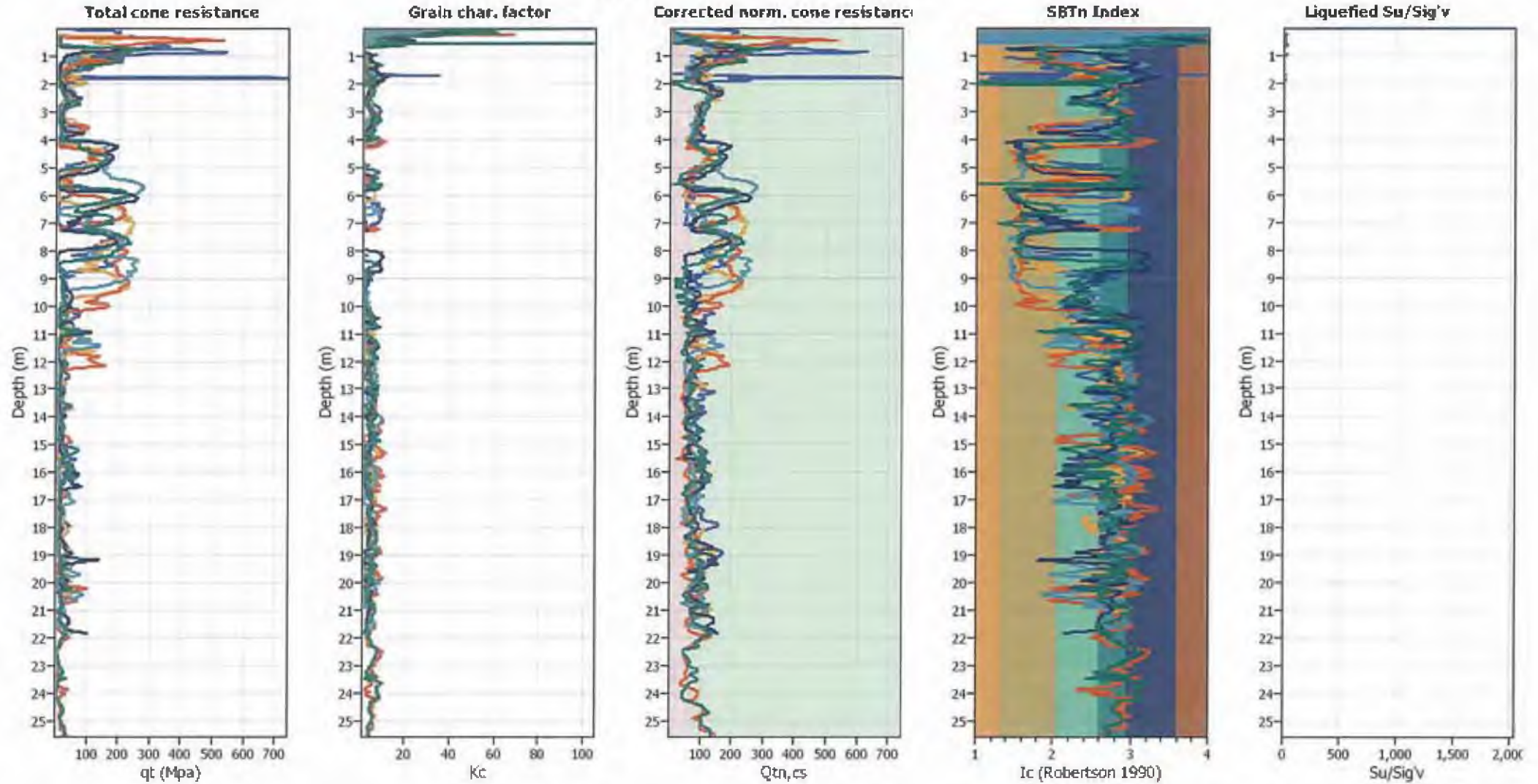
Project:

Overlay Cyclic Liquefaction Plots



Project:

Overlay Strength Loss Plots



15. APPENDIX C – OBSERVED LAND DAMAGE



Land Damage on Mary Muller Drive. The approximate site boundary is in yellow. Source: Canterbury Recovery, Project Orbit. Photographs taken 24 February 2011.

16. APPENDIX D – BUILDING PROPOSAL



EAST ELEVATION.



NORTH ELEVATION.



FLOOR PLAN.

SITE AREA: 8245 sqm.
 OFFICES FLOOR AREA: 750 sqm.
 FACTORY FLOOR AREA: 375 sqm.
 TOTAL: 1125 sqm.

CARPARKING REQUIREMENTS
 STAFF: 24
 VISITORS: 2
 DISABLED: 2
 95 PERCENTILE: 1
 HQV: 1
 CYCLES: 4
 CAR PARKS PROVIDED: 27

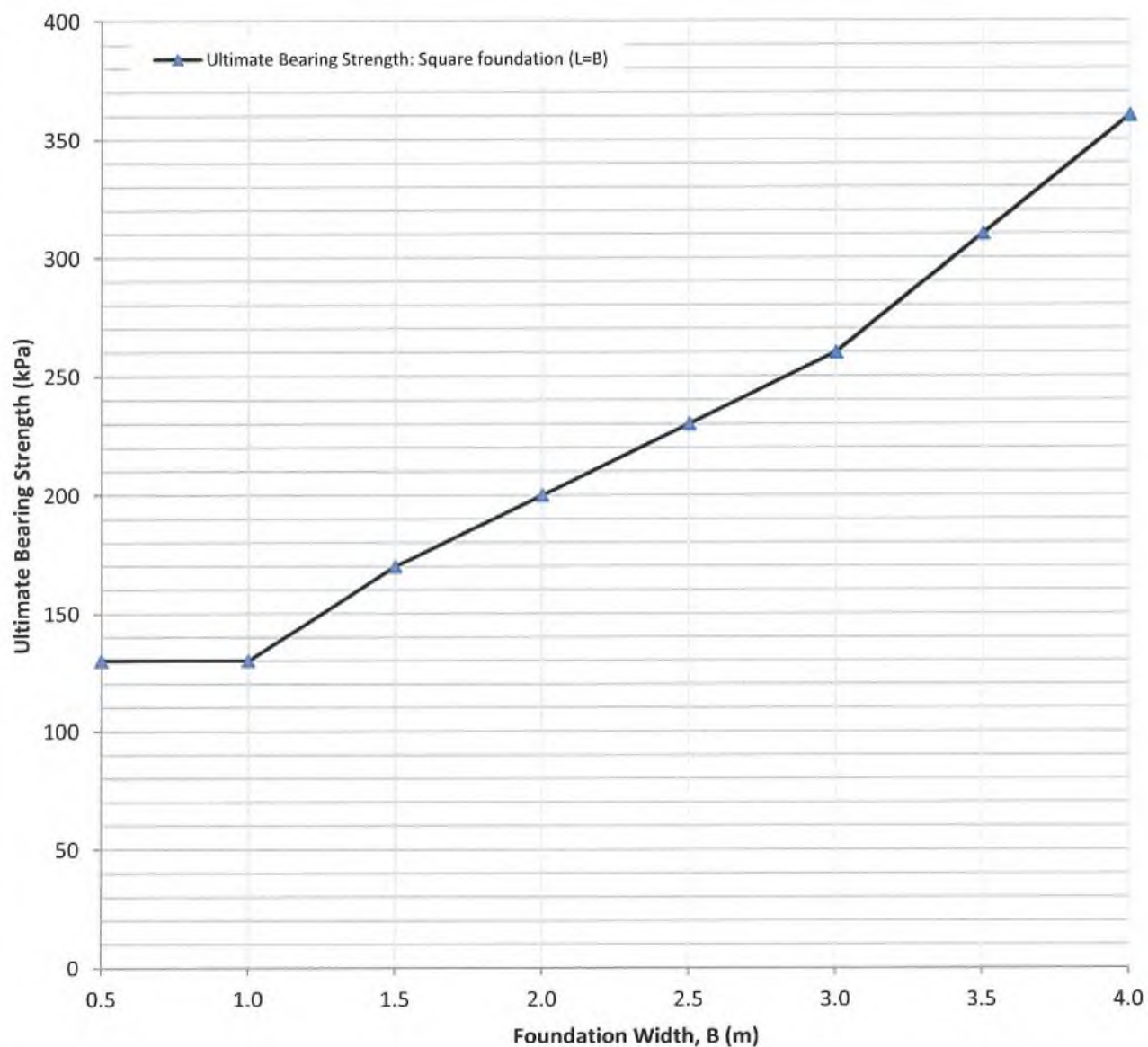


LOCATION PLAN.

WORMALDS MARYMULLER DRIVE

17. APPENDIX E – BEARING CAPACITY OF SHALLOW FOUNDATIONS

BEARING CAPACITY OF SHALLOW FOUNDATIONS



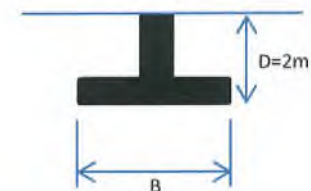
Notes:

Foundation depth = 2m below existing GL.

Assumes foundation extends through any fill materials, and bears onto insitu soils.

For rectangular footings with length(L) : width(B) ratio upto 3 (ie L=3B), use smallest dimension (B) to determine ultimate bearing strength.

Structural Engineer should also design bearing strength of chart 'Settlement Control of Shallow Foundations'.



Project: Wormald: 10 Mary Muller Drive

Client: Castle Rock Properties Ltd.

Job #: 351691

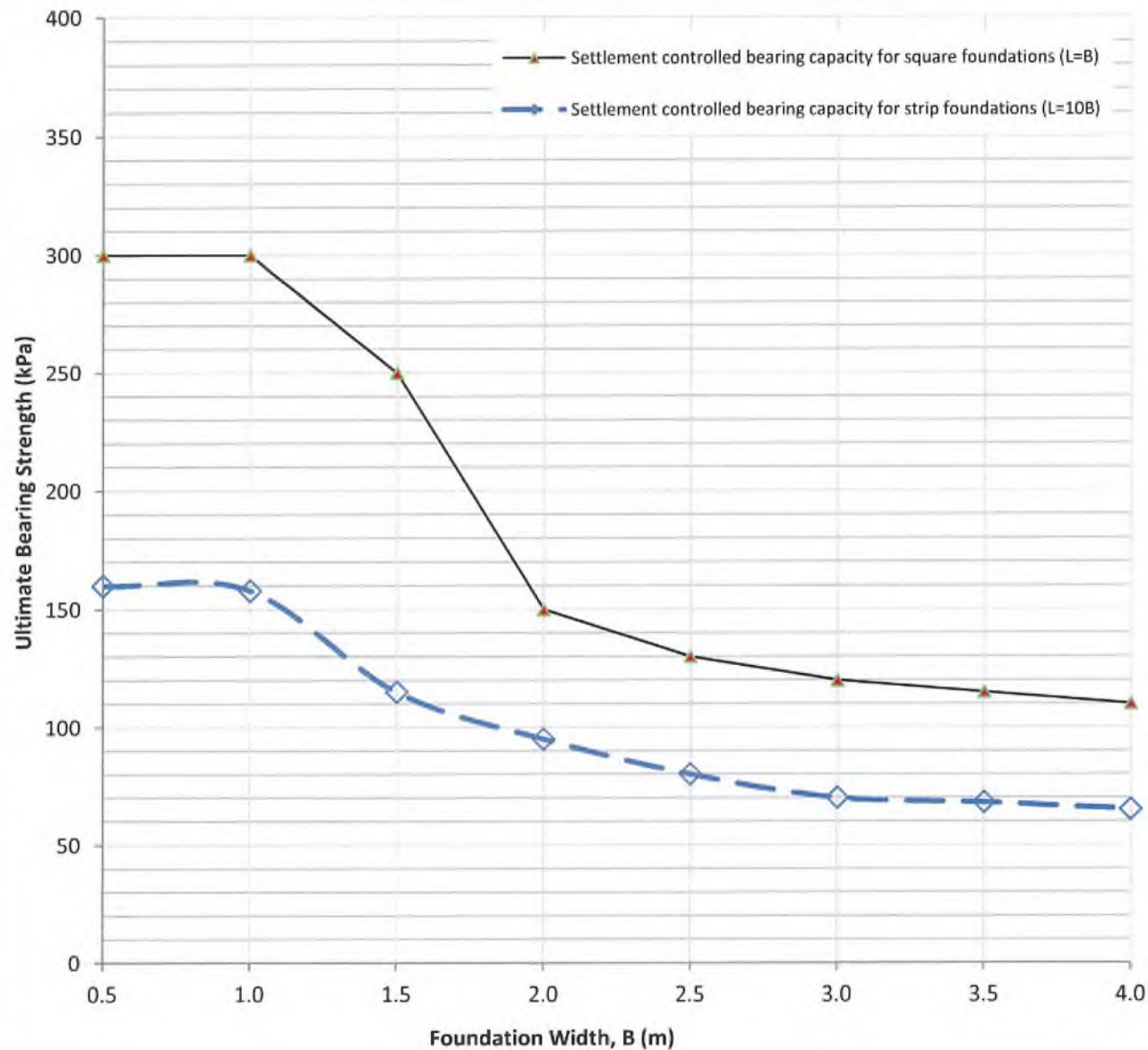
Date: 24 July 2012

Title: Bearing Capacity of Shallow Foundations

Eliot Sinclair
surveyors | engineers | planners

18. APPENDIX F – SETTLEMENT CONTROL OF SHALLOW FOUNDATIONS

SETTLEMENT CONTROL OF SHALLOW FOUNDATIONS



Notes:

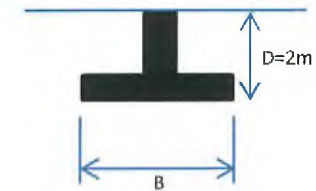
Foundation depth = 2m below existing GL.

Assumes foundation extends through any fill materials, and bears onto insitu soils.

Duration = 50 years, max 25mm total settlement under non-seismic conditions, calculated using Modulus of Elasticity.

Structural Engineer should also check bearing capacity of foundations, refer to 'Bearing Capacity of Shallow Foundations' chart.

Ultimate bearing capacity at smaller foundation width has been conservatively limited as shown to take into account the presence of lower soil layers.



Project: Wormald: 10 Mary Mullar Drive
Client: Castle Rock Properties Ltd
Job #: 352691
Date: 24 July 2012
Title: Settlement control of shallow foundations
Eliot Sinclair surveyors engineers planners

GEOTECHNICAL INTERPRETIVE REPORT
14 Mary Muller Drive, Hillsborough, Christchurch
For **Castlerock Properties Ltd**

18 December 2013

Eliot Sinclair
surveyors | engineers | planners

GEOTECHNICAL INTERPRETIVE REPORT

14 Mary Muller Drive, Hillsborough, Christchurch

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Date:	18 December 2013	
Reference:	382221_13304165215_letter_mdsmic.docx	
Status:	FINAL	
Distribution:	1 Original	Castlerock Properties Ltd
	File copy	Eliot Sinclair

Limitations: This report has been prepared according to the instructions from Castlerock Properties Ltd, for the particular objectives described in the report. The information contained in the report should not be used by anyone else or for any other purposes.

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1. INTRODUCTION

Eliot Sinclair were engaged by Castlerock Properties Ltd to prepare a geotechnical interpretive report for 14 Mary Muller Drive, Hillsborough, Christchurch and to comment on likely foundation requirement for a proposed warehouse with two-storey office building.

2. SCOPE OF WORK

The scope of work for this interpretive geotechnical report was;

- Review Eliot Sinclair's Geotechnical Factual Report dated 18 December 2013,
- Review available data from Canterbury Geotechnical database (CGD), Environment Canterbury's GIS database, and the Institute of Geological and Nuclear Sciences (GNS) database,
- Calculate the risk of liquefaction in accordance with the *Ministry for Business, Innovation and Employment's* guidelines,
- Prepare geotechnical interpretive report to comment on the general geotechnical conditions encountered across the site and comment on foundation system/s likely to be suitable for a future commercial building typical of the surrounding area.

3. DISCLAIMER

Comments made in this geotechnical interpretive report are based on information shown on the Canterbury Geotechnical Database, Environment Canterbury's GIS, GNS database, the results of the recent deep testing, our inspection of the general area, and the Ministry of Business, Innovation and Employment (MBIE)'s December 2012 guidelines.

Whilst every care was taken during our interpretation of the subsurface conditions, there may be subsoil strata and features that were not detected. Additionally, on-going seismicity in the general area may lead to deterioration or additional ground settlement that could not have been anticipated at time of writing of this report. The exposure of such conditions, occurrence of additional strong seismicity, or any future updates of MBIE's guidelines may require a review or additional investigations.

This report has been prepared for the benefit of Castlerock Properties Ltd in accordance with the Scope of Work.

No liability is accepted by Eliot Sinclair or any employee of Eliot Sinclair with respect to the use of this report by any other party, for any other purpose other than outlined in the Scope of Work.

4. SITE INVESTIGATION

4.1. Borehole with Standard Penetration Testing (SPT)

One deep borehole with SPT was carried out in October 2013 to 15m depth in order to confirm the nature of deeper geological conditions, with SPT testing was undertaken at 1.5m centres.

BH001 was located in the central part of the site, between the existing office building and the warehouse. This borehole generally encountered deep gravel/silt fill of medium density to 2.2m depth, over low plasticity silt to 4.4m, medium sand to 4.9m, over low plasticity silt to 15m where the test was terminated.

The depth to groundwater was not recorded in this test-hole.

Refer to Eliot Sinclair's Geotechnical Factual Report dated 18 December 2013.

4.2. Cone Penetration Testing (CPTu)

Four deep CPTu tests were undertaken in October 2013.

CPTu001 was undertaken north of the site within the car park and required 0.5m of predrilling to penetrate through the asphalt. CPTu002 was located in the garden area near the northeast corner of the existing office building and started from the ground level without predrilling. Both CPTu001 & CPTu001 were undertaken using a track mounted rig that has the ability to push the cone through the gravel/silt fill layers.

CPTu003 & CPTu004 were located south of the site along Port Hills Road. Due to limited access to these locations, a small mounted rig was used that requires a predrilling through the fill layers to 2.4m depth where the test started.

CPTu1 to CPTu3 generally encountered low cone tip resistance of 5MPa or less to the 15m target depth inferring a silty soil, with an exception of sandy soil presented between 4 and 6m depth having cone tip resistances of 20MPa.

CPTu004 was terminated at 11.28m below ground level encountered silt to the full depth with low cone tip resistances.

Refer to Eliot Sinclair's Geotechnical Factual Report dated 18 December 2013.

5. DESKTOP INVESTIGATION

5.1. Canterbury Geotechnical Database

The Canterbury Geotechnical Database (CGD) contains a large range of photographic, topographic, geological, geotechnical, land classification, survey records and field observations that relate to the Canterbury earthquake sequence. The database is coordinated by the Canterbury Earthquake Recovery Authority (CERA). A number of the following comments are based on information sourced from the CGD.

5.2. Geological Maps

The geological map of Christchurch indicates the site is underlain by 'Sand, silt and peat of lagoons and estuaries (Avon/Heathcote)' and is considered a 'Holocene estuary deposits'¹. Refer to Figure 1 and **Error! Reference source not found.**



Figure 1: Geological map of Christchurch with aerial photograph.

5.3. CERA Land Classification

The Ministry of Business, Innovation and Employment (MBIE) defines three technical categories for residential foundation design described in its guidance for repairing and rebuilding earthquake damaged homes in Canterbury. These categories apply to liquefaction prone flat land in the green zone in the greater Christchurch urban area and surrounding communities, predominantly for residential land. We note that there is no specific site classification for commercial properties.

This site has been classified by CERA as 'Green Zone, Urban Non-residential'.

5.4. Active Faults

The 4 September 2010 M7.1 earthquake occurred on the Greendale Fault, with its eastern limit of surface rupture located around 18km to the west.

The 22 February 2011 M6.2 earthquake occurred on a blind fault located under the Port Hills, with its projected intersection with the ground surface located between the Christchurch CBD and the lower northern slopes of the Port Hills.

¹ GNS Geological Map of Christchurch, showing the surficial soil geology and surrounding area (source: GNS, November 2013).

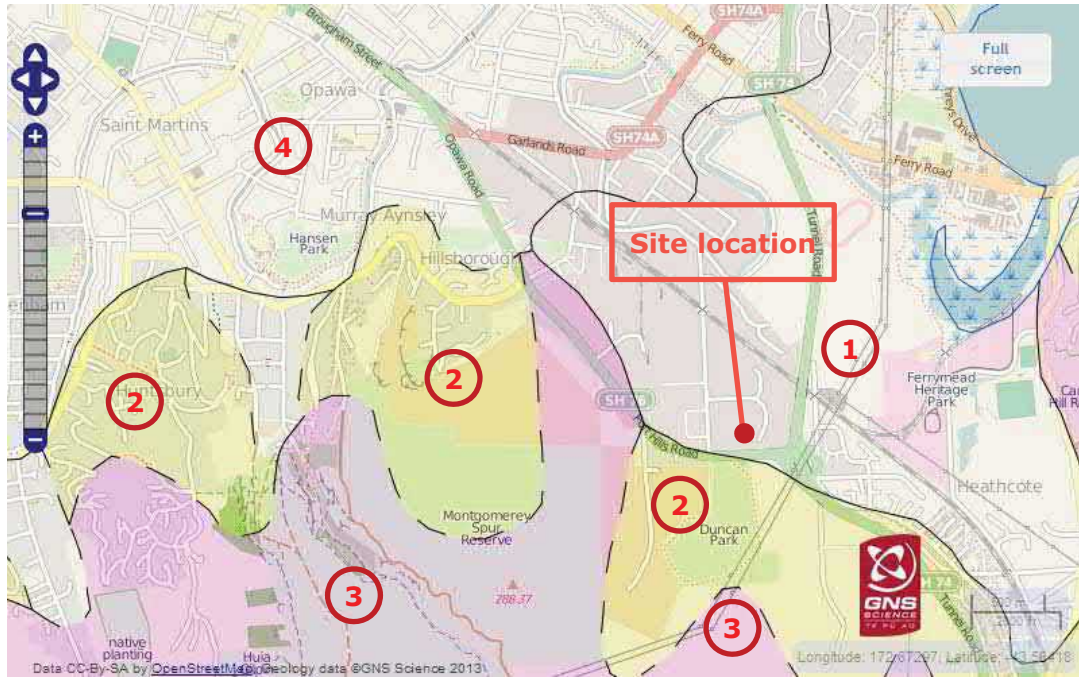


Figure 2: Geological map of Christchurch.

1	Sand, silt and peat of lagoons and estuaries (Avon/Heathcote)
2	Yellow-brown windblown silt deposits, locally with fine sand or clay; >3m thick & commonly in multiple layers; thicker downslope
3	Basaltic (hawaiite) to trachytic lava flows interbedded with tuff and breccia (including lahars), many dikes & minor lava domes
4	Modern river floodplain/low-level degradation terrace. Unweathered, variably sorted gravel/sand/silt/clay. Surfaces <2 degree slope

The 13 June 2011 M6.0 earthquake was located on a NW-SE fault that is estimated to be approximately parallel to, and just east of the New Brighton coast. Refer to Figure 3.

5.5. EQC Vertical Ground Movements

LiDAR survey data recorded in September 2010 and February 2011 does not extend across the area. However, LiDAR data taken after the 13 June 2011 event indicates vertical ground movement of '-0.2 to -0.1m' occurred in the mid to south parts of the site, while it was insignificant in the north part and recorded as '-0.1 to 0.1m'.

5.6. EQC Horizontal Ground Movements

Given the absence of LiDAR data, horizontal ground movements are not shown for each of the September 2010 and the February 2011 events, while 'local' horizontal ground movement data for the June 2011 event indicates horizontal movements of around 60 to 105mm occurred to the southwest.

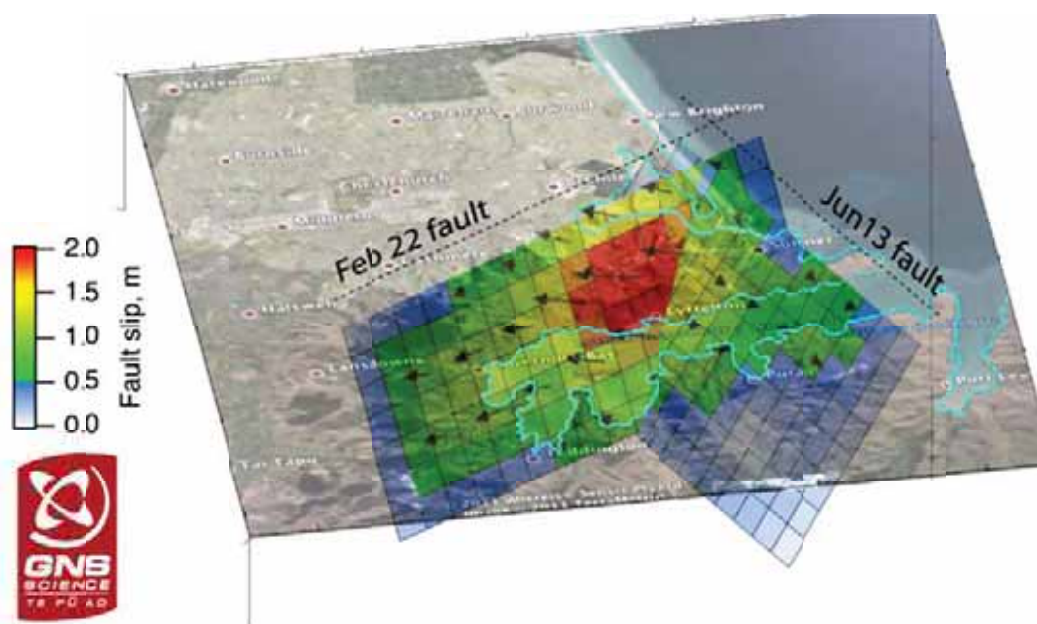


Figure 3: Semi-transparent graphic giving a simplified view of the two faults that ruptured causing the earthquakes on 22 February and 13 June in Christchurch (GNS, 06/09/2011, <http://www.gns.cri.nz/Home/News-and-Events/Media-Releases/Two-main-faults>).

5.7. Conditional PGA for Liquefaction Assessment

The Ministry for Business, Innovation and Employment's (MBIE) 'Guidance for repairing and rebuilding houses affected by the Canterbury earthquakes' (December 2012) specifies, for residential land, the peak ground acceleration ($PGA_{M7.5}$) to be adopted for liquefaction assessment in a serviceability limit state (SLS) event as $PGA_{M7.5} = 0.13g$, and $PGA_{M7.5} = 0.35g$ in an ultimate limit state (ULS) event.

The equivalent peak horizontal ground accelerations in each of the September 2010 and June 2011 events were in excess of the Serviceability Limit State (SLS, $PGA_{M7.5} = 0.13g$), but were less than the Ultimate Limit State (ULS, $PGA_{M7.5} = 0.35g$). The February 2011 earthquake exceeded the ULS. Refer to Table 1.

Table 1: Comparison of peak horizontal ground accelerations close to site

PGA (horizontal)	SLS (1/25, M7.5)	ULS (1/500, M7.5)	04 Sept 2010 (M7.1)	22 Feb 2011 (M6.2)	13 June 2011 (M6.0)
Design (as of April 2012)	0.13g	0.35g			
Conditional Median PGA			0.29g	0.67g	0.45g
Magnitude Scaling Factor (MSF)			1.11	1.41	1.48
Equivalent to $PGA_{M7.5}$			0.26g	0.48g	0.30g

5.8. EQC liquefaction Interpreted from Aerial Photography

The liquefaction interpreted from aerial photography as shown in the CGD website does not extend to the area in September 2010. However, data provided in 22 February 2011 event indicates 'minor observed liquefaction' was recorded and 'no observed liquefaction' was recorded for 13 June 2011 event.

Inspection of the aerial photography provided by the CGD for both events did not indicate any evidence of sand boils within the site or the surrounding area. The raft of fill material over this site and the presence of voids in the fill may have suppressed any indications of liquefaction at the surface.

5.9. EQC Observed Ground Crack Locations

No ground cracks were found within the site or the nearby properties.

5.10. Liquefaction Hazard Mapping

Environment Canterbury's reviewed of the liquefaction hazard information² and notes a 'low liquefaction potential may be expected' for the area.

5.11. Flood Hazard

The Christchurch City Council records note the site is not located in Flood Management Area or Flood Awareness Area. Refer to Appendix A.

5.12. Previous Geotechnical Investigation Data

Review of the CGD did not reveal any geotechnical investigation data that is nearby or close to the site.

However, Eliot Sinclair undertook several cone penetration tests within 10 and 12 Mary Muller Drive, north of the site.

CPTu002, CPTu003 and CPTu006 (located near to the boundary line between 12 and 14 Mary Muller Drive) encountered low cone penetration resistances of less than 5MPa down to 15m depth, with the exception between 5 to 7m depth where the cone penetration resistances were increased to more than 15MPa presumably inferred dense sand. Refer to Appendix B.

6. LIQUEFACTION ASSESSMENT

6.1. Method

The calculation of liquefaction triggering was undertaken using the method outlined in both Idriss & Boulanger (2008)³. The estimation of post-liquefaction

² Brackley, H.I. (2012): Review of liquefaction hazard information in eastern Canterbury, including Christchurch City and parts of Selwyn, Waimakariri and Hurunui Districts - Environment Canterbury, Report No. R12/83

³ Idriss, I.M. & Boulanger, R.W. (2008): *Soil liquefaction during earthquakes* - Earthquake Engineering

induced settlements for CPTu used the method outlined by Zhang et al (2002)⁴, and SPT analysis used the method outlined by Yoshimine, et al (2006)⁵.

The MBIE's guidelines prescribed the peak ground acceleration values to be adopted for liquefaction assessment of residential land and residential dwelling are considered to have an importance level of IL2 in terms of NZS1170. The MBIE's guidelines do not specifically relate to non-residential land. However, there is no other New Zealand standard and therefore, the values set out in the MBIE's guidelines have been adopted as a default. These are 0.13g for a Serviceability Limit State (SLS) event and 0.35g for the Ultimate Limit State (ULS).

The results of the SPT were analysed for both the Serviceability Limit State (SLS)⁶ and the Ultimate Limit State (ULS)⁶. Refer to Appendix C.

6.2. Analysis considerations

The borehole confirms the presence of silt and gravel fills to around 2.4m below ground level. CPTu003 and CPTu004 required predrilling to that depth before tests started and therefore all the liquefaction analyses of the CPTu using CLiQ software were analysed starting from 2.4m depth in order to provide consistency in the results.

In addition, for the purpose of this assessment, the depth to groundwater for liquefaction assessment was conservatively assumed to be 1.0m below ground level.

6.3. Liquefaction Potential Index from CPTu Testing

The liquefaction potential index (LPI) in a SLS (0.13g) scenario is estimated to range between less than 5 to less than 15 at the full depth of testing. These values indicate a 'low to high risk' of liquefaction damage to shallow building foundations.

Under the ULS (0.35g) scenario, the LPI increased to 'high to very high risk' of liquefaction damage to shallow building foundations. Refer to Appendix C.

6.4. Liquefaction Severity Number (LSN)

"The liquefaction severity number (LSN) is a new calculated parameter developed by Tonkin and Taylor to reflect the more damaging effects of shallow liquefaction on residential land and foundations"⁷. This parameter is limited to the upper 10m of the CPTu profile.

Research Institute Monograph MNO12

⁴ Zhang, G., Robertson, P.K. & Brachman, R. (2002): Estimating Liquefaction Induced Ground Settlements from the CPT, *Canadian Geotechnical Journal*, 39: pp 1168-1180.

⁵ Yoshimine, M., Nishizaki, H., Amano, K. & Hosono, Y. (2006): Flow deformation of liquefied sand under constant shear load and its application to analysis of flow slide in infinite slope, *Soil Dynamics and Earthquake Eng.* 26, 253-264.

⁶ Guidance for Repairing and rebuilding houses affected by the Canterbury earthquakes, version 3, Ministry of Business, Innovation and Employment (MBIE), December 2012

⁷ Tonkin & Taylor's (T&T) report 'Liquefaction Vulnerability Study', February 2013, T&T

The maximum LSN calculated is around 24 under SLS event and indicates 'Moderate expression of liquefaction, with sand boils and some structural damage' is likely to occur. While the maximum LSN in a ULS event is calculated to be 34 and indicates 'Moderate to severe expression of liquefaction, settlement can cause structural damage' is likely.

6.5. Vertical settlement due to liquefaction (index value at SLS)

Estimates of vertical settlement are summarised in Table 2.

Table 2: Summary of liquefaction-induced settlements, at selected test locations (limited to the first 10m depth).

Test No. (depth to refusal)	SLS (M7.5, 0.13g)	ULS (M7.5, 0.35g)
BH01 (15m)	99mm	137mm
CPTu001 (15m)	70mm	86mm
CPTu002 (15m)	110mm	125mm
CPTu003 (15m)	75mm	100mm
CPTu004 (11.28m)	110mm	145mm
CPTu002(27.68m) 12 Mary Muller Drive	95mm	123mm
CPTu003(18.62m) 12 Mary Muller Drive	94mm	104mm
CPTu006(27.52m) 12 Mary Muller Drive	76mm	90mm

We note vertical settlements calculated by CLiq software are based on the method by Zhang et al (2002)⁴, for a range of parameters that are estimated from the four basic CPTu parameters of depth, cone tip resistance, skin friction, and pore water pressure, and therefore the settlements shown are not guaranteed or an exact figure.

7. BUILDING FOUNDATIONS

7.1. Proposal Construction

Drawings provided to us indicate a large warehouse occupying the majority of the site area and comprising a combination of lightweight wall panel cladding and heavyweight precast concrete panels and attached two-storey office building located on the southeast corner of the site having lightweight cladding.

7.2. Floor level

The Christchurch City Council does not specify a minimum floor level and therefore we assume future floor level needs only to comply with the relevant requirements of the NZBC. Refer to Appendix A.

7.3. Foundation system

7.3.1. General

Based on previous excavations during road construction and photographic records on our file, the fill materials may contain various amounts of leather scraps, topsoil, timber, rubber tyres, bricks, boulders, gravels, gasworks waste, silt, clay pipes, concrete slab debris, and scrap steel. The large debris and scrap pipes tend to form underground voids.

Due to the nature of the uncontrolled fill materials there is a high risk of large differential settlement under static conditions due to consolidation of the uncompacted materials, decay of organic matter, and loss of fines into underground voids.

Road formation works for Mary Muller Drive were previously undertaken by excavation of the uncontrolled fill materials, disposal offsite of obvious organic or unsuitable materials, screening to remove large debris, etc and relaying the remaining screened soils in thin layers using controlled compaction and moisture control. These works appear to have been very successful as the surface of Mary Muller Drive in the area of the site appears to be in good condition with no obvious evidence of earthquake related damage due to cracking, settlement, heaving, etc.

7.3.2. Pile foundations

Due to the nature of the underlain soil down to the full depth of investigation of 15m, a deep pile foundation will not be a suitable foundation system. Our previous knowledge in the site and its surrounding properties indicates the depth to firm bearing strata will be at least 18-20m below ground level, and this would tend to result in a pile foundation being very costly in relation to shallow foundation systems.

7.3.3. Shallow foundations

The adjacent buildings were supported on foundation pads and strip footings excavated to the top of the insitu silt material. The fill material was compacted by rolling with a heavy impact roller. This technique resulted in severe vibration felt in adjoining properties and is not recommended for the present site.

7.3.4. Replacing uncontrolled filling with clean materials

Based on the soils encountered by the deep tests and the presence of topsoil and uncontrolled fill materials encountered at the upper surficial soil, the inferred bearing capacity within the uncontrolled fill materials down to at least 2.4m is low and variable and should not be relied upon to support building foundations or floor slabs without remedial action.

The uncontrolled fill materials are also potentially contaminated.

For this site the recommended method of remediation is to excavate and screen the fill materials down to the top of the insitu silt layers. The screened fill would then be replaced and compacted in layers.

The removal of the larger debris will result in a deficiency of fill material and imported silt or granular fill will need to be imported to make up the difference to the underside of the foundation. Some silt material is already available on site for this purpose. The fill material contains a perched watertable and strong short-term inflows into the excavation can be expected. This water will need to be appropriately disposed of during the earthworks and compaction; the water levels will need to be maintained below the level of filling.

The fill material will need to be compacted at a suitable moisture content to a standard that meets the requirements of NZS4431:1989. This document, whilst originally intended for filling for residential construction, will also provide suitable standard of earthworks compaction for light industrial construction.

Nuclear Densometer testing of the fill to confirm that the standard of compaction achieves at least 95% of maximum dry density will be required.

Depending on the soil consistency at the bottom of excavation, geogrid may be required to be placed between the in-situ soils and the control filling materials. In order to confirm the necessity of geogrid, Eliot Sinclair & Partners should inspect all excavation before any placement of controlled fill.

There is sufficient clearance from the site boundaries to enable excavation within the whole of the building footprint except along the south boundary where the proposed building will be located close to the boundary and excavation will be extended as close to the boundary. Excavation will not be sufficient to fully support the perimeter foundation. The foundation design engineer will need to provide cantilever support for any structure along the boundary. Alternatively, a series of deep concrete pads extended to the insitu silts will need to be provided.

7.3.5. Bearing strength

The process of backfilling the site with clean controlled materials will increase the ultimate bearing capacity to at least 300kPa. A geotechnical strength reduction factor $\phi_{bc}=0.45$ should be adopted. A design ultimate bearing strength of around $q_d=135\text{kPa}$ can be used.

Along the south boundary, foundation pads may need to extend to the insitu silts at approximately 2.4m depth. These pads should be designed for the range of ultimate bearing capacities shown in Appendix D, again with a geotechnical strength reduction factor of 0.45.

7.3.6. Settlement control of shallow foundations

Primary and secondary settlements were estimated using the Schmertmann Method outlined in Bowles international edition (1997)⁸, and assume that

⁸ Bowles, J. 1996 "Foundation analysis and design, International Edition 1997" McGraw Hill

foundations are located over insitu silts at approximately 2.4m depth. Design bearing strengths were calculated as the maximum strength resulting in 25mm total settlement over a 50-year design life. Refer to Appendix E.

The bearing capacity for foundations reduces with increasing width due to the increasing depth of influence of the foundation pressure, and the presence of weaker subsoil layers identified by the subsoil testing. The geotechnical engineer should review foundation widths outside of the range shown on the summary charts in order to confirm the design bearing capacity.

It is noted that foundation settlement shown on the design chart are for static conditions only, and any ground settlements that may arise from seismic shaking would be additional to the design static settlement of 25mm over 50 years.

7.3.7. Settlement

The LIDAR data available in the Canterbury Geotechnical Database indicates settlements in the order of $\pm 100\text{mm}$ may have occurred at the site. However, the underlying insitu soils are relatively uniform and differential settlements over the building footprint would be small. Inspection of the roads in the area does not indicate significant differential settlements. Further, recent testing of deep raft construction has confirmed that these structures are effective in controlling differential settlements⁹.

7.3.8. Other soil parameters

Other soil geotechnical parameters that may be required by the foundation design engineer, such as Young's Modulus and Constrained Modulus, are shown graphically in Appendix F. Conservative values should be adopted from the range of interpreted values shown.

7.3.9. Carpark areas

We note the large carpark area will need to be constructed so that differential settlement is kept to a minimum, and to provide adequate bearing strength for delivery trucks, etc.

Due to the likely presence of highly variable fill with areas of unsuitable organic matter and voids, we consider the most practical option would be to remediate the in-situ fill in the same manner as proposed for the compacted gravel/subsoil raft, before construction of the carpark formation.

7.4. Building Services

It would be good practice to, where practical, allow for building services to exit through the side of the foundations and not under foundations.

⁹ Bowen, H.J. & Jacka, M.E. (2013): Liquefaction induced ground damage in the Canterbury earthquakes: predictions vs. reality, *Proc. 19th NZGS Geotechnical Symposium*, Queenstown, NZ, Ed. CY Chin, pp 45-52.

In addition, where they exit the foundations, a flexible or sliding connection to services should be used to allow for differential movement in both a vertical and lateral direction in the event of strong earthquake shaking. These measures will assist in providing a resilient construction that will allow continuing use of the building services in the event of any future large earthquakes.

Appendix A : COUNCIL CORRESPONDENCE

Firas Salman

From: Wells, Heath <Heath.Wells@ccc.govt.nz>
Sent: Monday, 4 November 2013 7:04 a.m.
To: Firas Salman
Subject: RE: [#382221] 14 Mary Muller Drive, Hillsborough

12d Synergy: -1
12d Synergy Job: [#382221]
12d Synergy Project: [#382221]

Hi,

This site is not in the FMA and not in a modelled floodplain.

Mean ground level at this site is recorded (based on post-quake LiDAR) as RL 12.05m.

There are no minimum ground or floor levels associated with this site in relation to flood limitation.

"Standard" / "normal" heights are all that is required.

Regards,

Heath

From: Firas Salman [<mailto:Firas.Salman@eliotsinclair.co.nz>]
Sent: Friday, 1 November 2013 11:26 AM
To: Wells, Heath
Cc: Keenan, Sheryl
Subject: [#382221] 14 Mary Muller Drive, Hillsborough

Dear Heath/ Sheryl

I'm working on a geotechnical report for a new commercial building.

The site is part of a big lot that is legally described as Lot 2 DP 392999 and occupies its southwest corner. Kindly see the below photos.

This site appears to be within the grey zone on the CCC-floor levels database.

So, kindly help and provide the available information in this regard (including minimum floor level, average ground level).

Kind regards,

Firas A. Salman
Civil/Geotechnical Engineer

firas.salman@eliotsinclair.co.nz

Eliot Sinclair

surveyors | engineers | planners

Eliot Sinclair & Partners Ltd. 20 Troup Drive, PO Box 9339, Tower Junction, Christchurch 8149, NZ
phone 03 379 4014, fax 03 365 2449

www.eliotsinclair.co.nz

[facebook](#)



We have moved to 20 Troup Drive, Tower Junction, Christchurch.
Our new address is: **PO Box 9339, Tower Junction, Christchurch 8149.**

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The views expressed in this message are those of the individual sender and may not necessarily reflect the views of the Christchurch City Council.

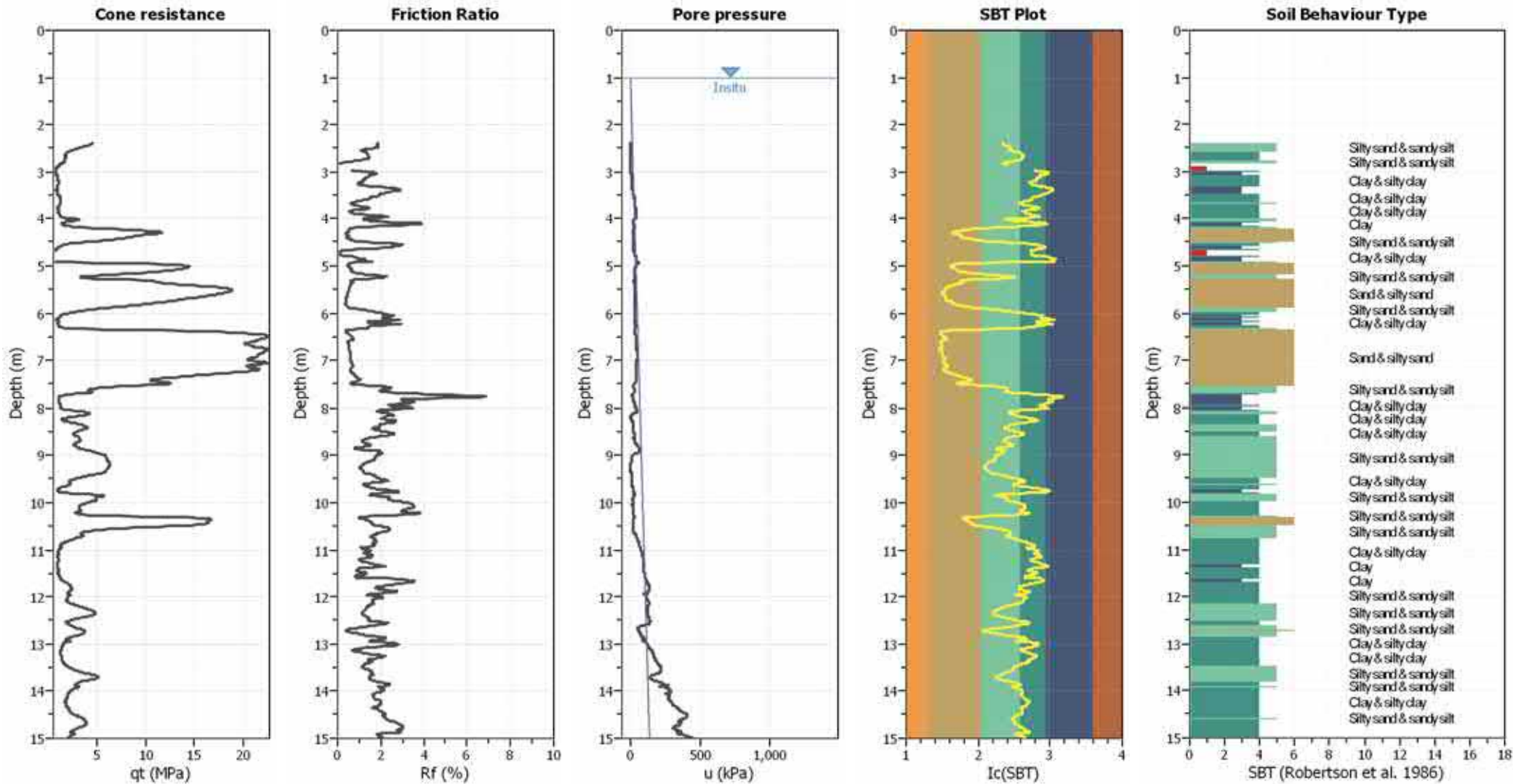
If you are not the correct recipient of this email please advise the sender and delete.

Christchurch City Council

<http://www.ccc.govt.nz>

Appendix B : PREVIOUS GEOTECHNICAL TEST DATA

CPT basic interpretation plots



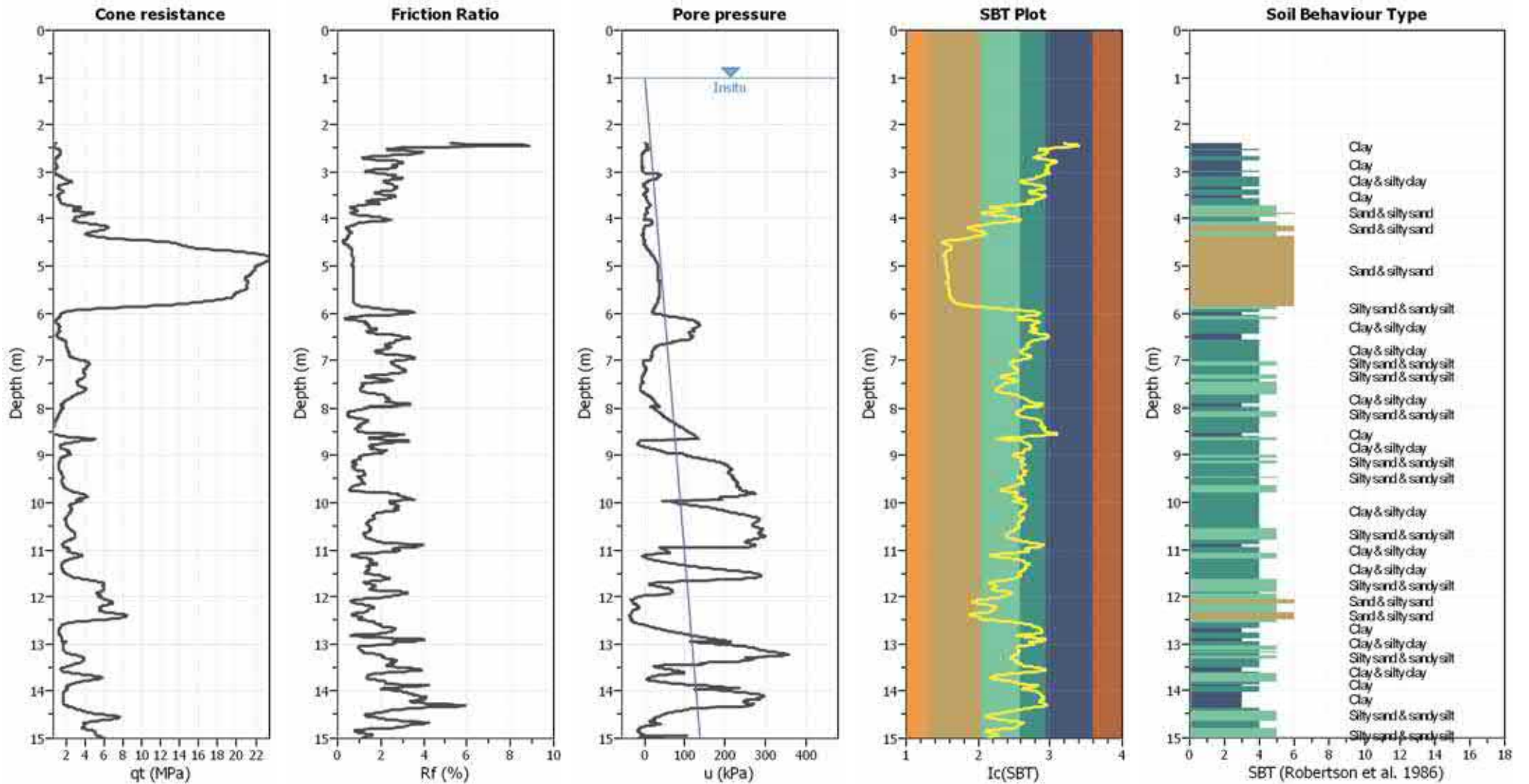
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Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _p applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	10.00 m

SBT legend

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

CPT basic interpretation plots



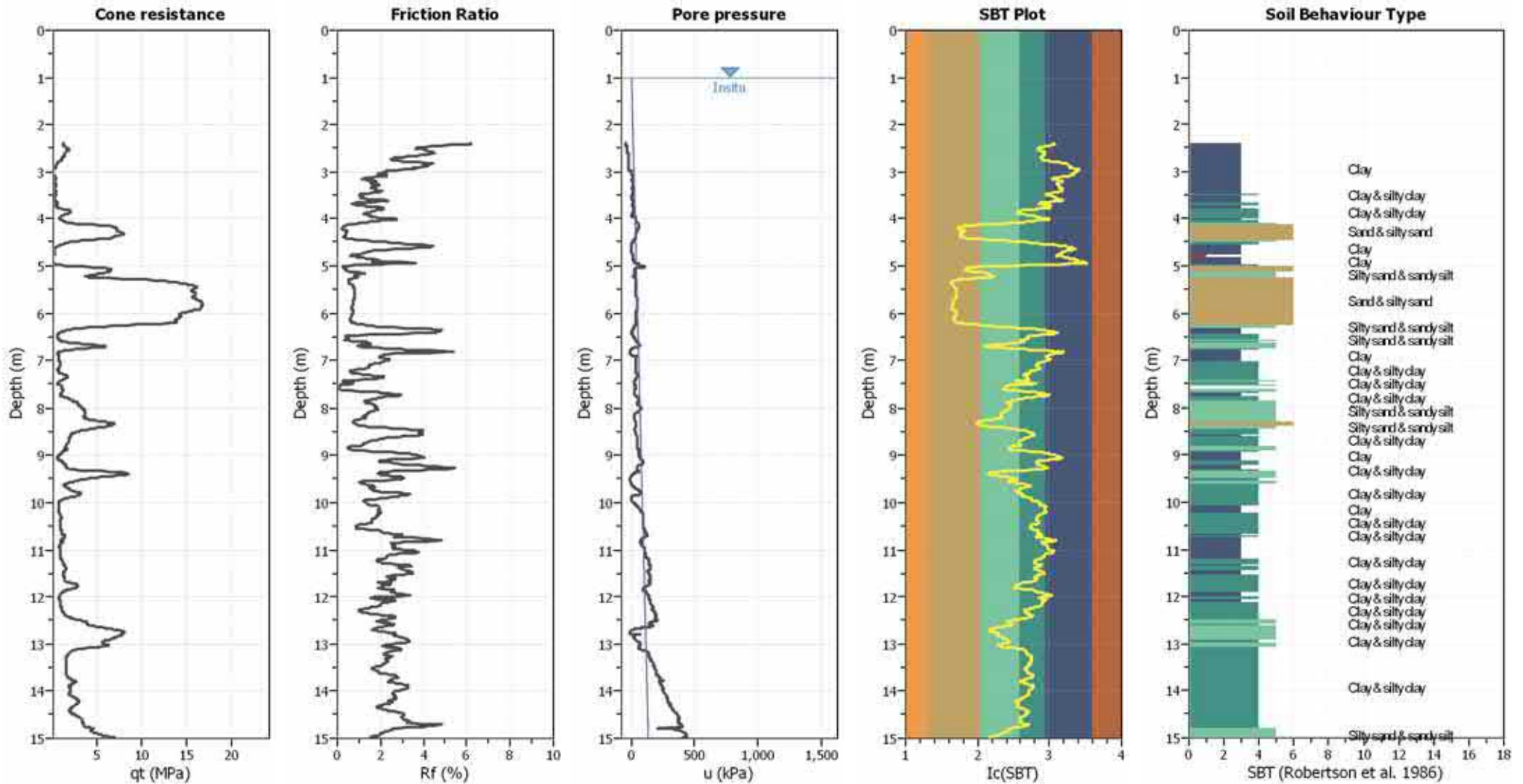
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _o applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	10.00 m

SBT legend

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2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

CPT basic interpretation plots



Input parameters and analysis data

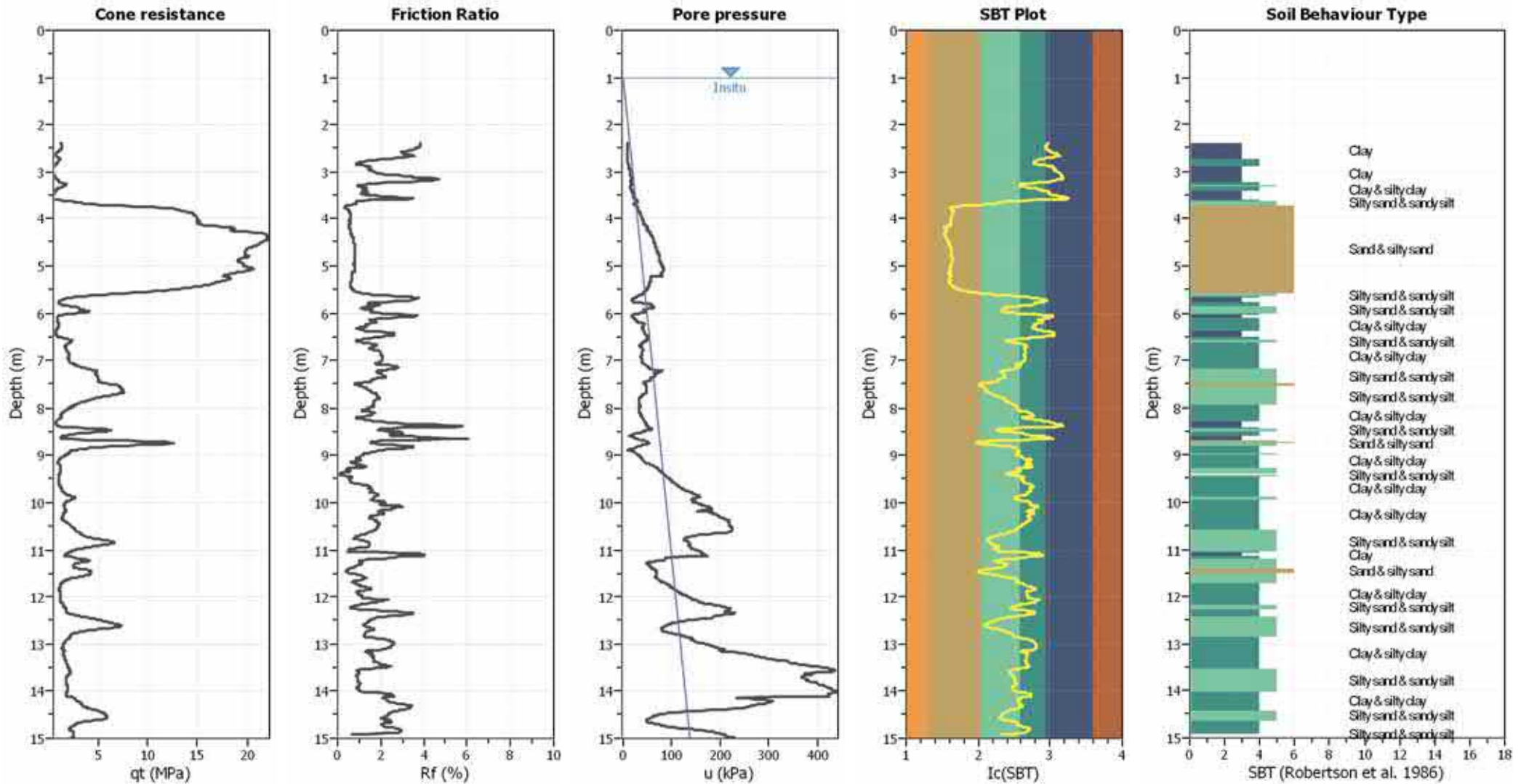
Analysis method:	I&B (2008)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	10.00 m

SBT legend

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

Appendix C : LIQUEFACTION ANALYSIS

CPT basic interpretation plots



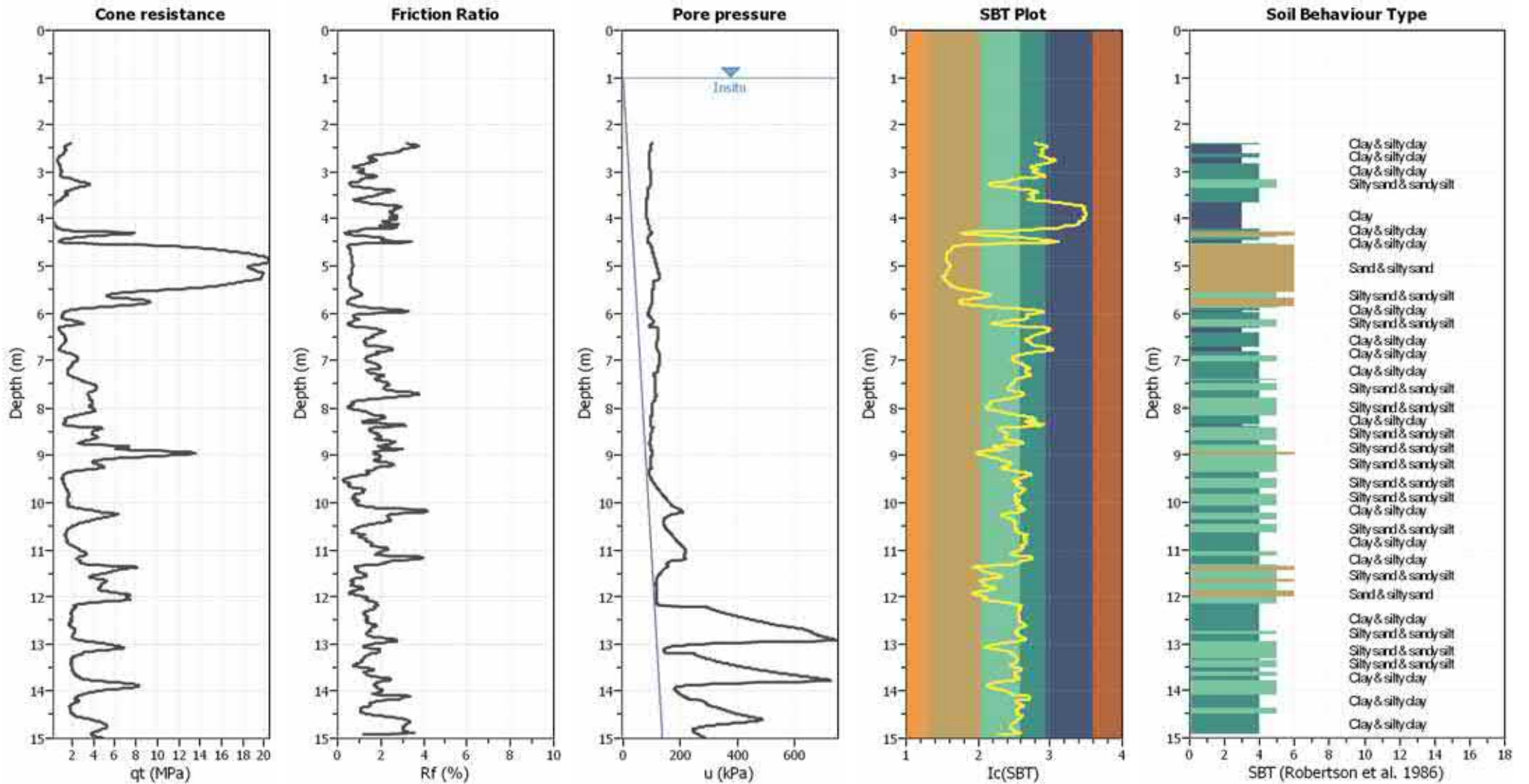
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	10.00 m

SBT legend

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

CPT basic interpretation plots



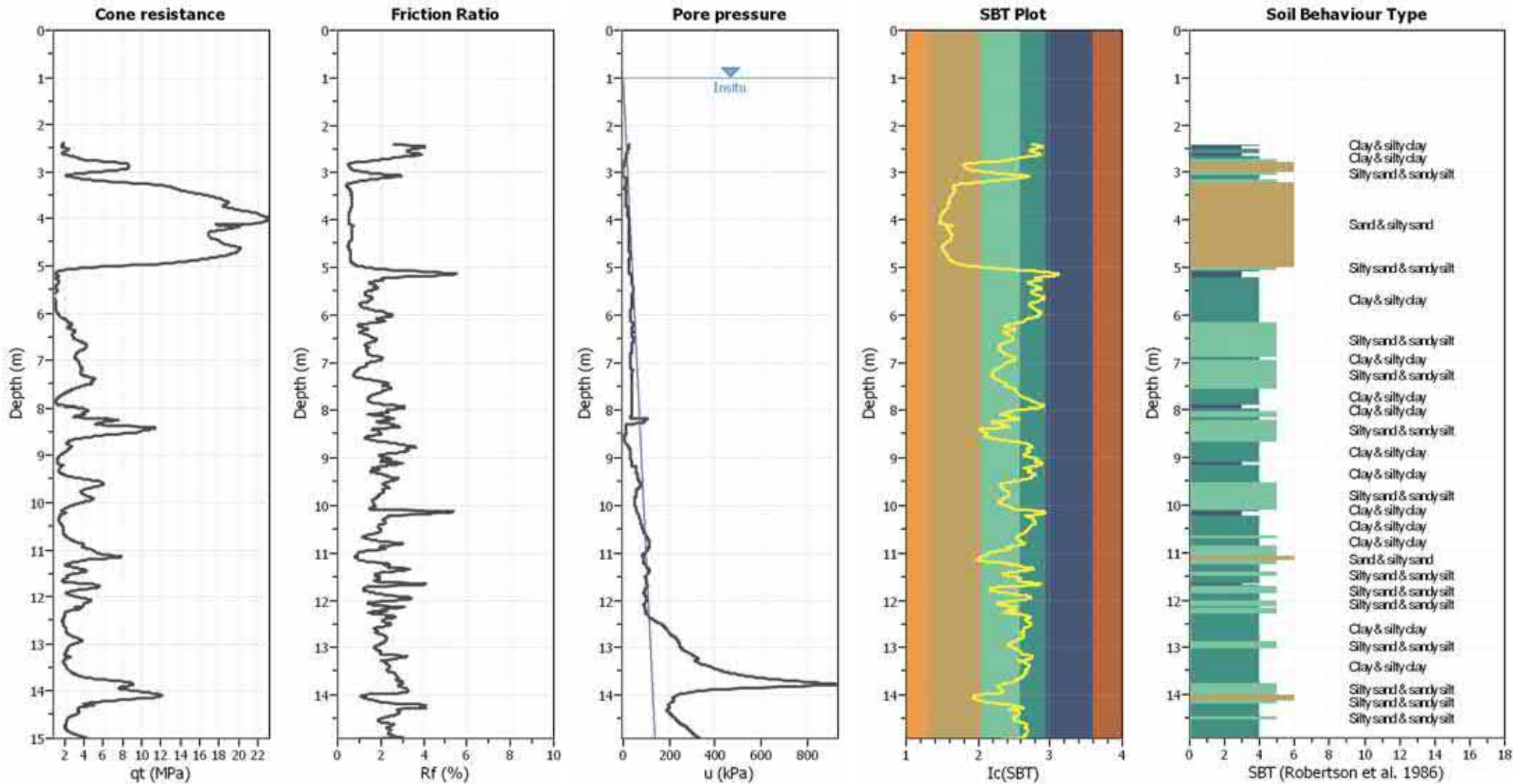
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_p applied:	Yes
Earthquake magnitude M_w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	10.00 m

SBT legend

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

CPT basic interpretation plots



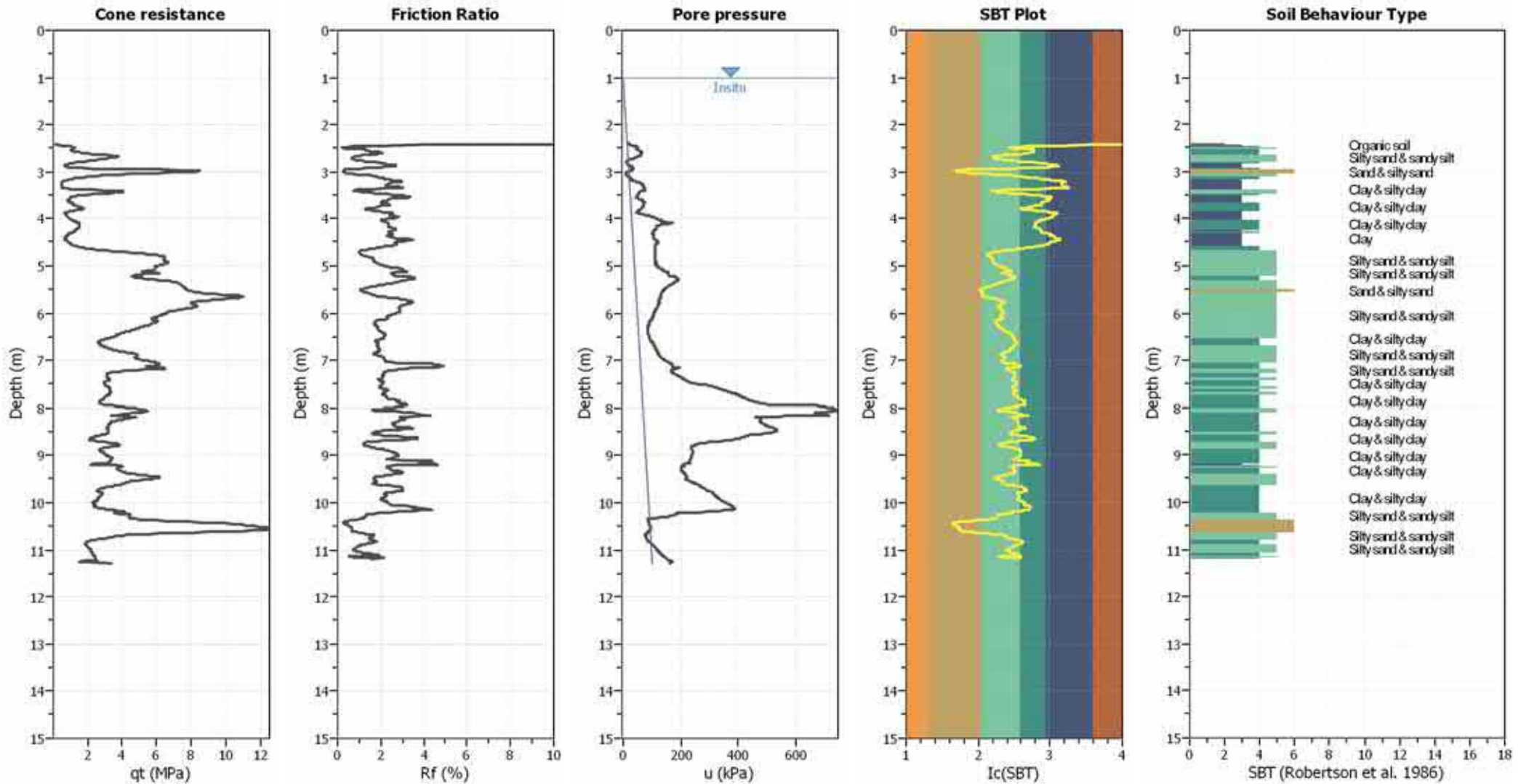
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_p applied:	Yes
Earthquake magnitude M_w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	10.00 m

SBT legend

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

CPT basic interpretation plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	10.00 m

SBT legend

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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

Serviceability Limit State (SLS, limited to 10m depth): M7.5, 0.13g peak horizontal ground acceleration



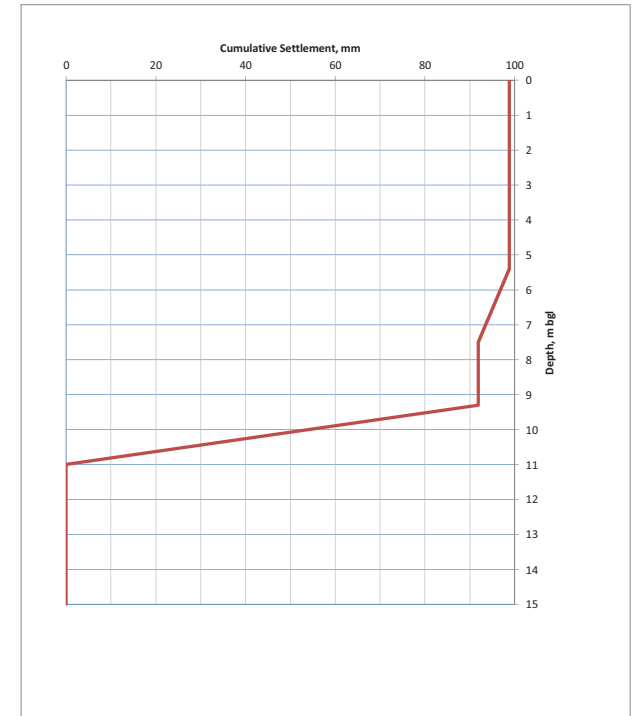
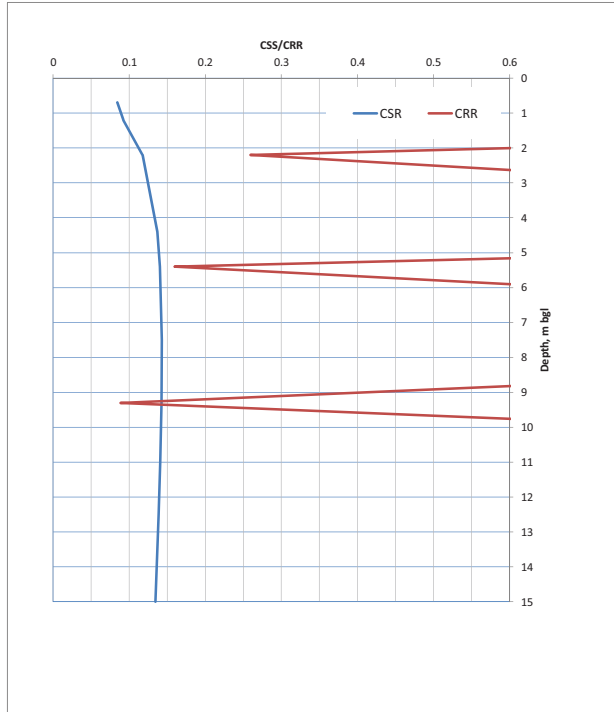
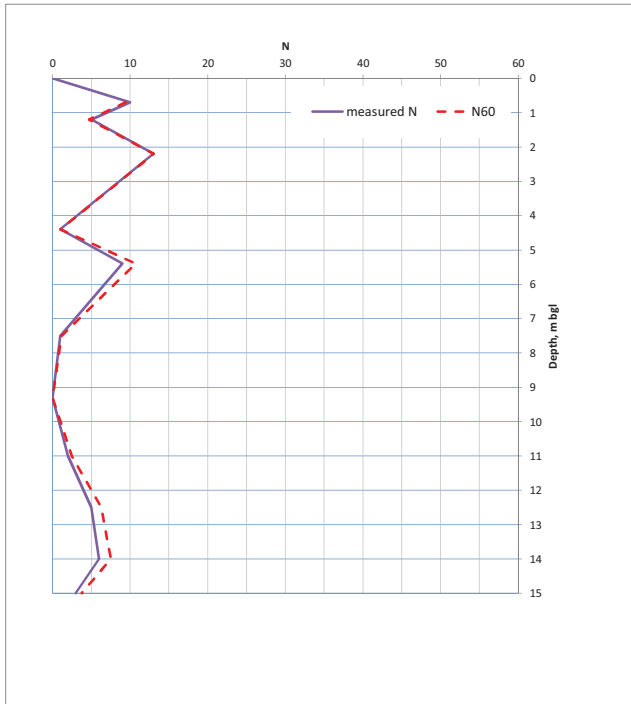
Input Parameters		Magnitude scaling factor, MSF = 1.00
Atmospheric pressure, Pa kPa	101	
Peak ground acceleration, pga =	0.13	
Earthquake magnitude, M=	7.5	
Water table depth, m =	1.0	
Average γ_T above water table, kN/m3 =	18.0	
Average γ_T below water table, kN/m3 =	19.0	
Borehole diameter (mm)	70	
Requires correction for sample liners (Yes/No)	No	
Rod Lengths (m)	1.5	

SLS event - (Settlement is limited to the top 10m)

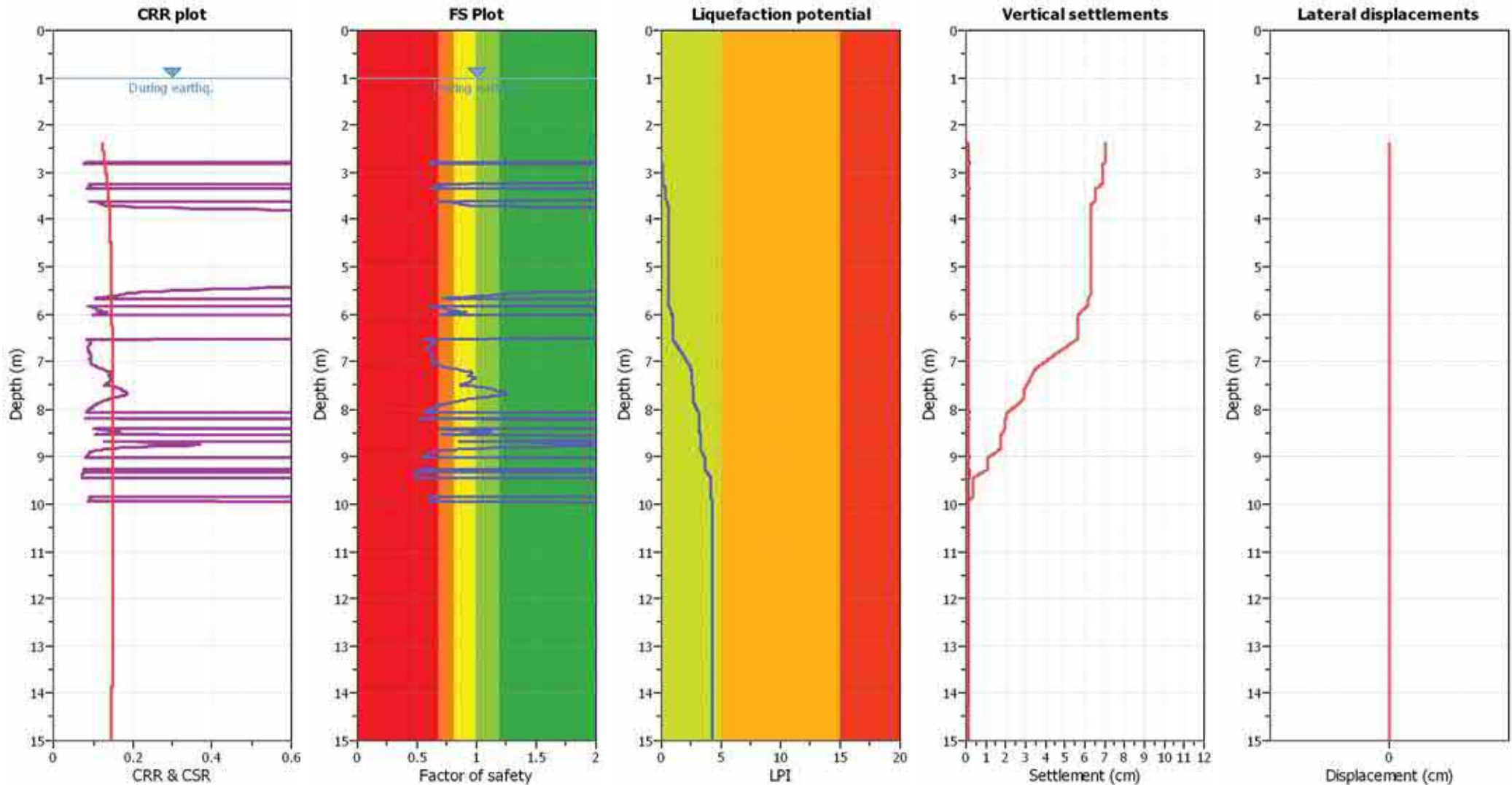
BH001

14 Mary Muller Drive, Hillsborough

SPT sample number	Depth to bottom of layer (m)	measured N	Description	Soil Classification	Flag "Clay" "Unsat" "Unreliable"	Fines Content (%)	Energy Ratio (%)	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	σ_{vc}^1 (kPa)	C_N	$(N_1)_{60}$	ΔN for fines content	$(N_1)_{60cs}$	Stress reduction coeff, r_d	CSR	MSF for sand	K_σ for sand	CRR for M=7.5 & $\sigma_{vc}^1=1atm$	CRR	FOS	Limiting shear strain, γ_{lim}	F_α	Max shear strain, γ_{max}	ΔH_{liq} (m)	ΔLDI (m)	Vertical reconsolidation strain, E_v	Settlement, ΔS_1 (mm)	Cumulative settlement, ΣS (mm)	
0	0	0																																
1	0.70	10	gravel fill		Unsat	5	75.3	1.26	1.00	0.75	1.0	9.4	13	13	1.70	16.0	0.0	16.00	1.00	0.085	1.00	1.10	0.165	2.00	2.00	0.25	0.712	0.000	0.00	0.000	0.000	0	99	
2	1.20	5	silt	silt		50	75.3	1.26	1.00	0.75	1.0	4.7	22	20	1.70	N.A.	n.a.	n.a.	1.00	0.093	1.00	1.10	n.a.	2.00	2.00	0.00	0.000	0.000	0.20	0.000	0.000	0	99	
3	2.20	13	gravel fill			0	75.3	1.26	1.00	0.80	1.0	13.1	41	29	1.70	22.2	0.0	22.19	0.99	0.117	1.00	1.10	0.236	0.260	2.00	0.12	0.398	0.000	1.00	0.000	0.000	0	99	
4	4.40	1	silt	silt		50	75.3	1.26	1.00	0.85	1.0	1.1	83	49	1.66	N.A.	n.a.	n.a.	0.97	0.137	1.00	1.10	n.a.	2.00	2.00	0.00	0.000	0.000	2.20	0.000	0.000	0	99	
5	5.40	9	sand			0	75.3	1.26	1.00	0.95	1.0	10.7	102	58	1.34	14.4	0.0	14.36	0.96	0.140	1.00	1.06	0.151	0.160	1.14	0.29	0.780	0.019	1.00	0.019	0.007	7	99	
6	7.50	1	silt	silt		50	75.3	1.26	1.00	0.95	1.0	1.2	142	78	1.20	N.A.	n.a.	n.a.	0.93	0.143	1.00	1.08	n.a.	2.00	2.00	0.00	0.000	0.000	2.10	0.000	0.000	0	92	
7	9.30	0	sandy silt			30	75.3	1.26	1.00	1.00	1.0	0.0	176	94	1.06	0.0	5.4	5.36	0.91	0.143	1.00	1.01	0.088	0.089	0.62	0.50	0.948	0.500	1.80	0.900	0.051	92	92	
8	11.00	2	silt	silt		50	75.3	1.26	1.00	1.00	1.0	2.5	208	110	0.95	N.A.	n.a.	n.a.	0.88	0.141	1.00	0.98	n.a.	2.00	2.00	0.00	0.000	0.000	1.70	0.000	0.000	0	0	
9	12.50	5	silt	silt		50	75.3	1.26	1.00	1.00	1.0	6.3	237	124	0.89	N.A.	n.a.	n.a.	0.86	0.139	1.00	0.94	n.a.	2.00	2.00	0.00	0.000	0.000	1.50	0.000	0.000	0	0	
10	14.00	6	silt	silt		50	75.3	1.26	1.00	1.00	1.0	7.5	265	137	0.84	N.A.	n.a.	n.a.	0.84	0.136	1.00	0.91	n.a.	2.00	2.00	0.00	0.000	0.000	1.50	0.000	0.000	0	0	
11	15.00	3	silt	silt		50	75.3	1.26	1.00	1.00	1.0	3.8	284	147	0.79	N.A.	n.a.	n.a.	0.82	0.135	1.00	0.89	n.a.	2.00	2.00	0.00	0.000	0.000	1.00	0.000	0.000	0	0	



Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _s applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	10.00 m

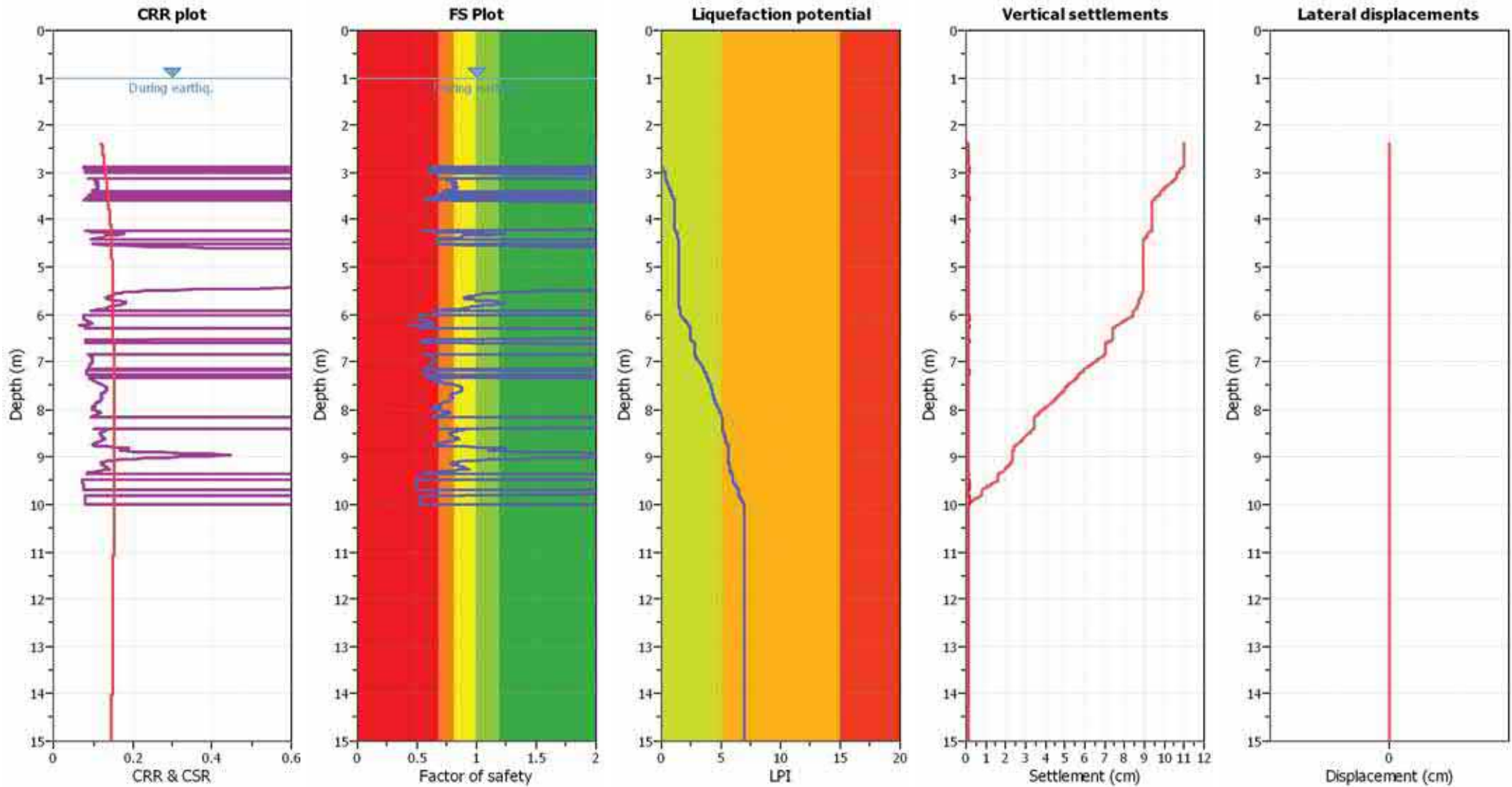
F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _σ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	10.00 m

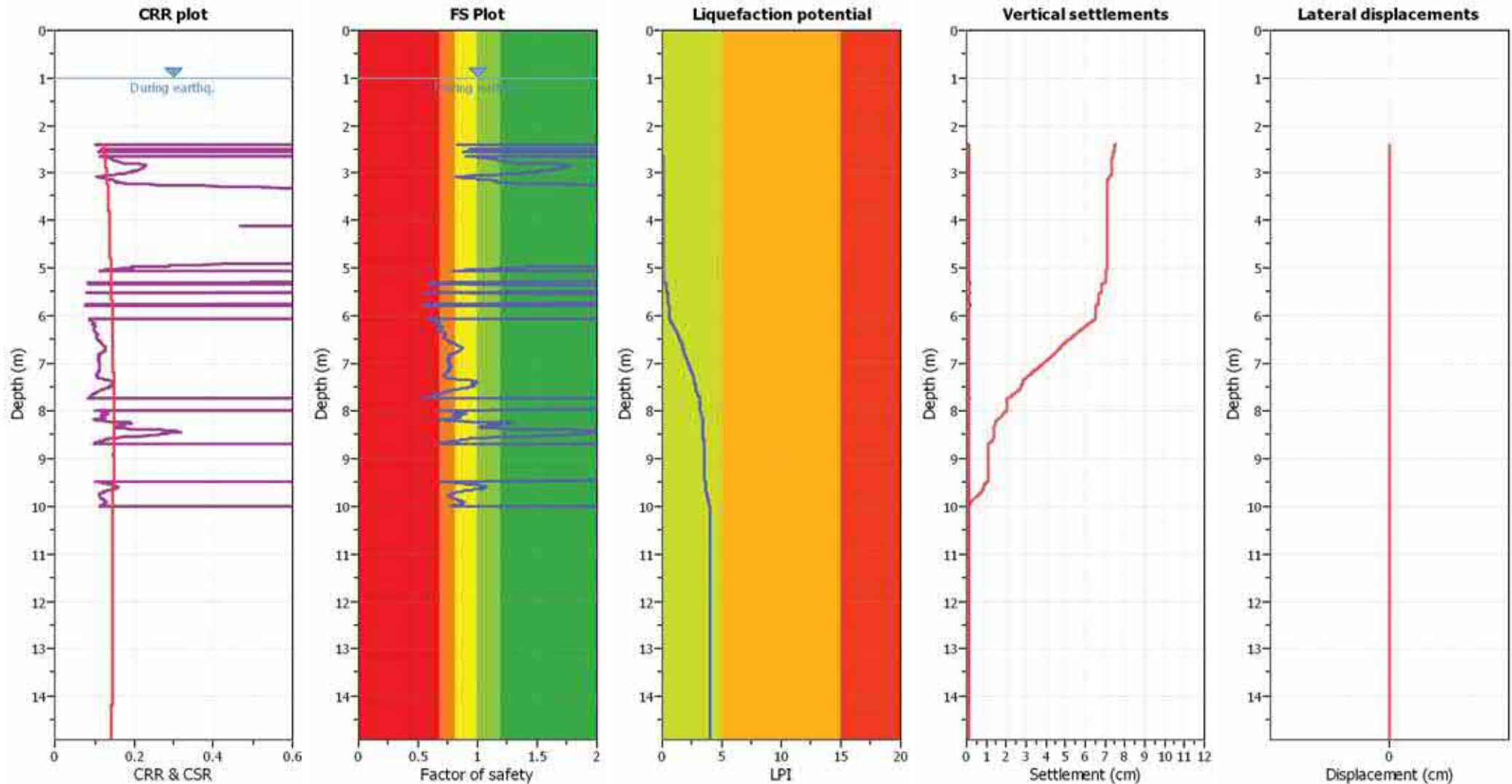
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Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	Yes
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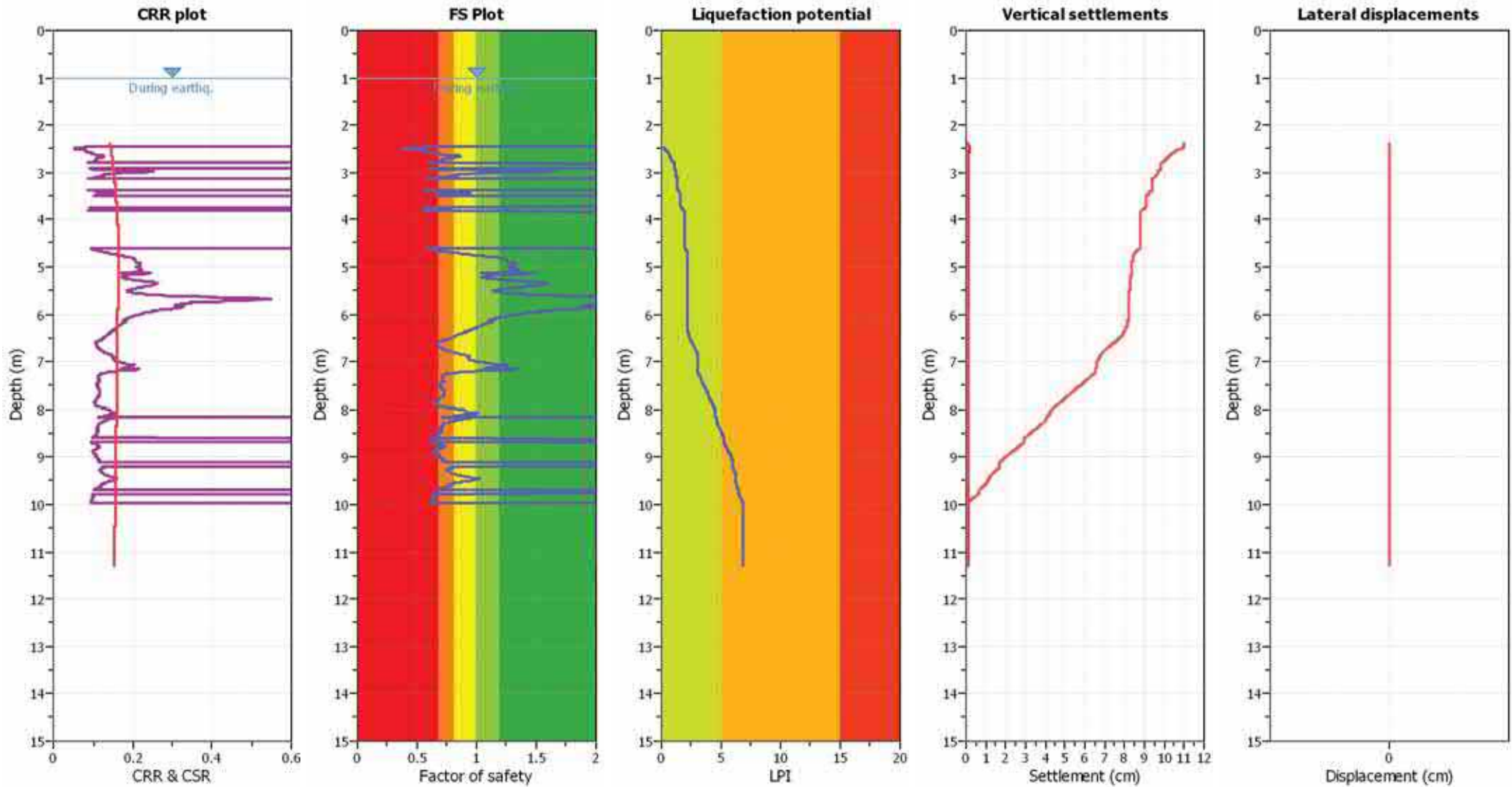
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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _σ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	10.00 m

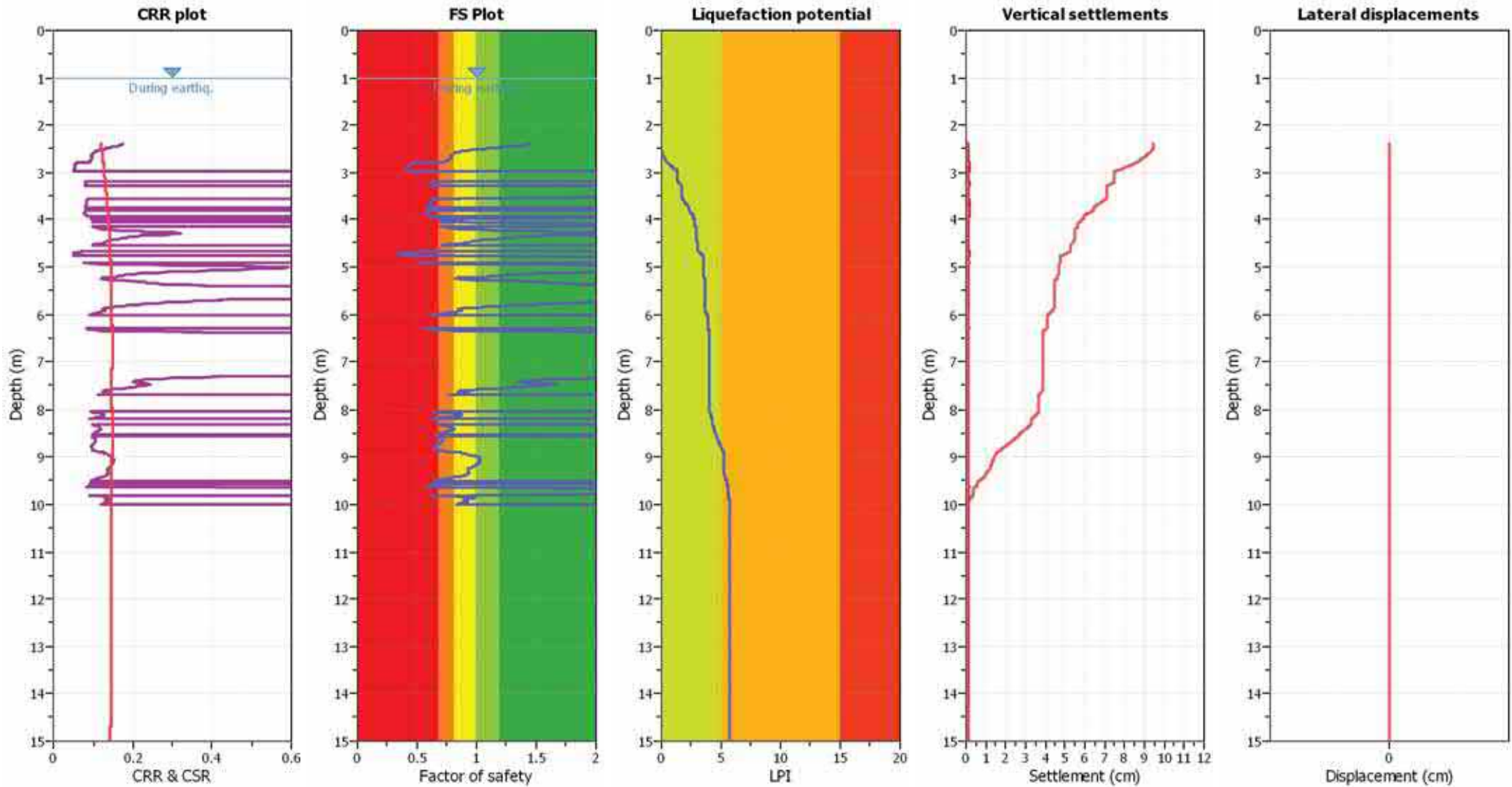
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Liquefaction analysis overall plots



Input parameters and analysis data

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Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
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Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	10.00 m

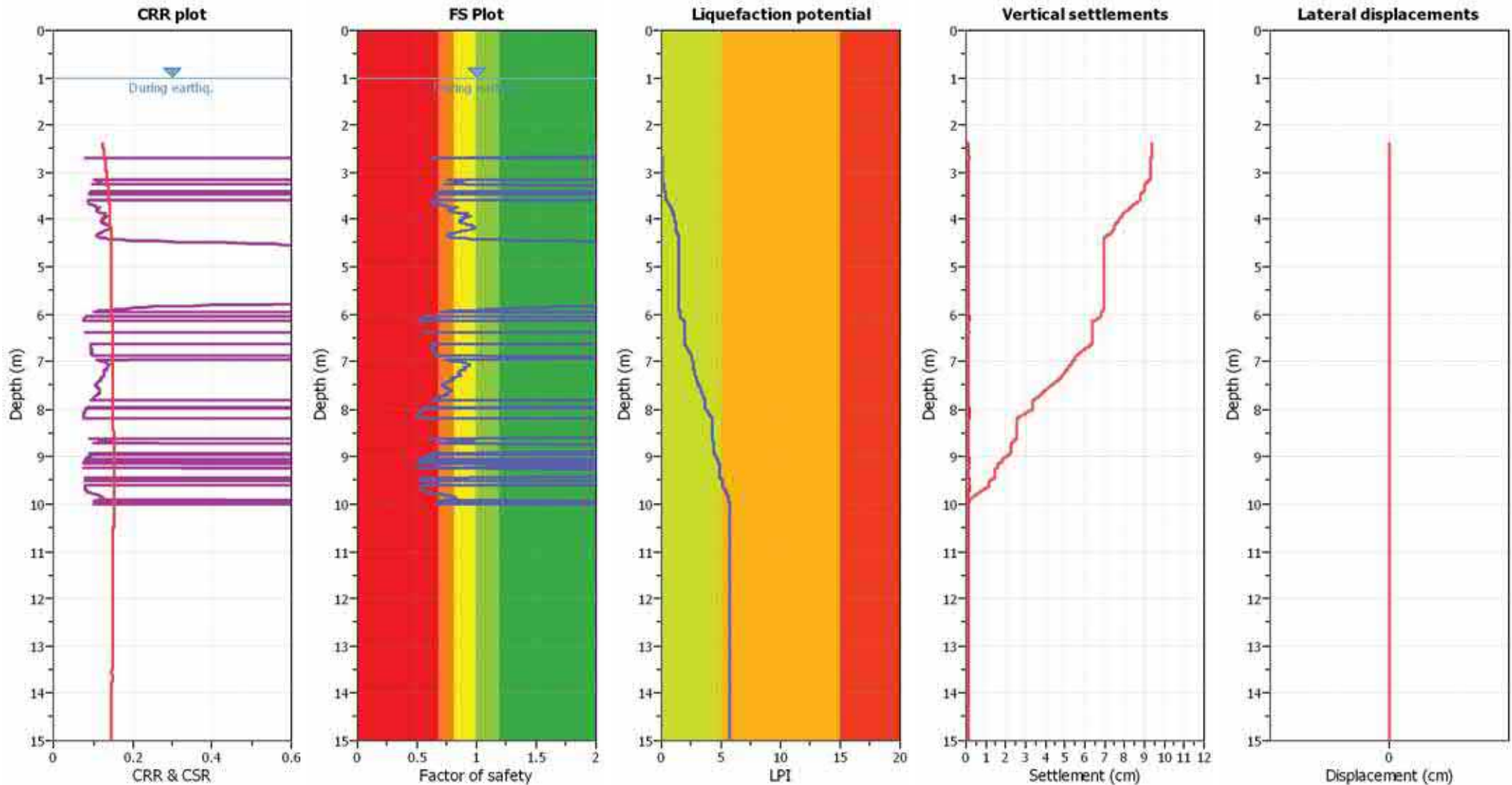
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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _σ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	10.00 m

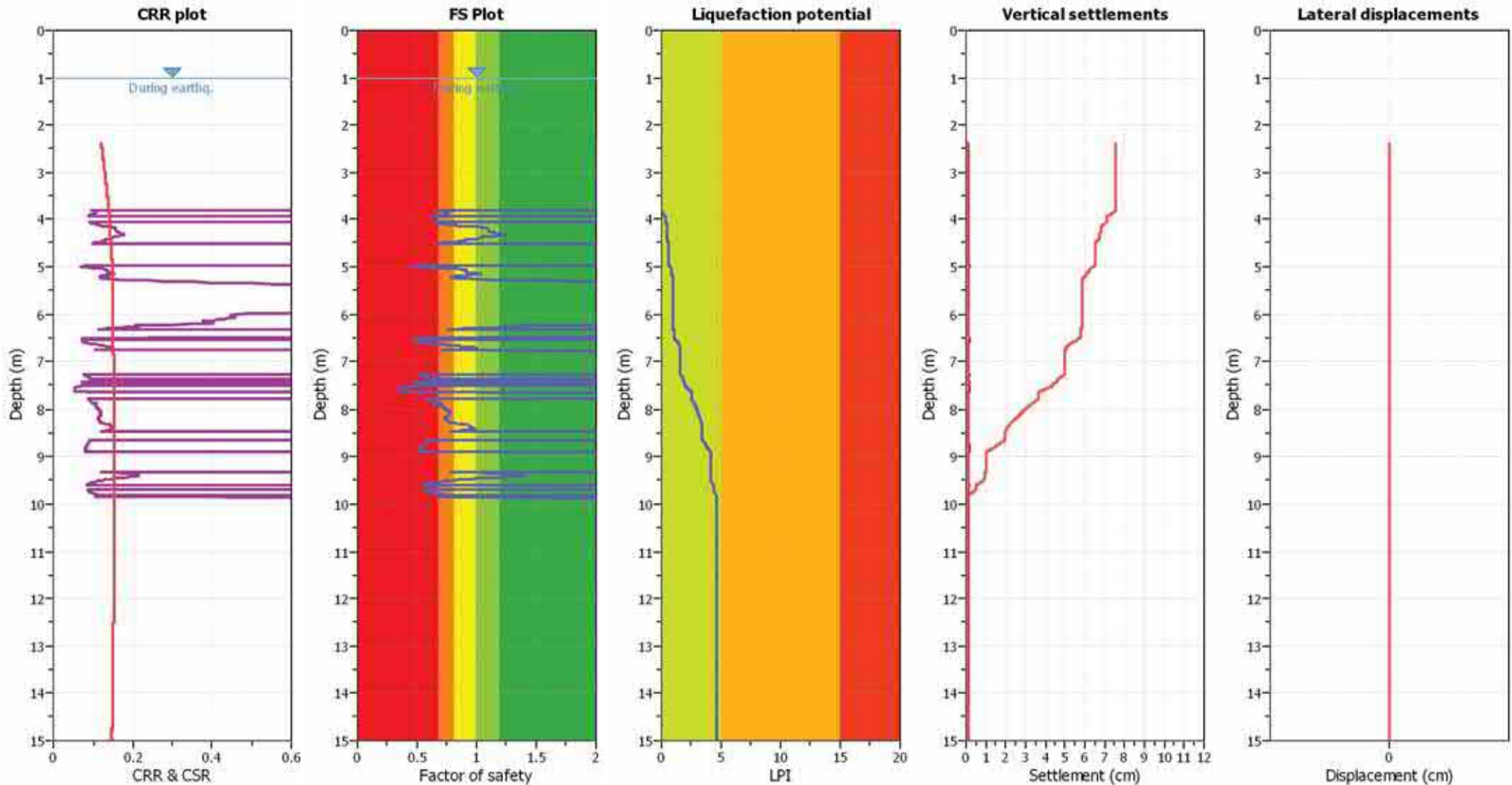
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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	10.00 m

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

Ultimate Limit State (ULS, limited to 10m depth): M7.5, 0.35g peak horizontal ground acceleration



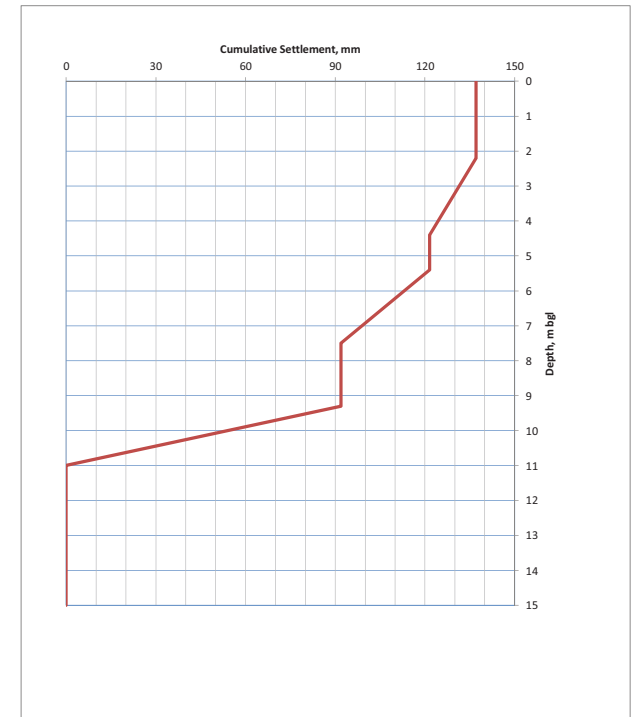
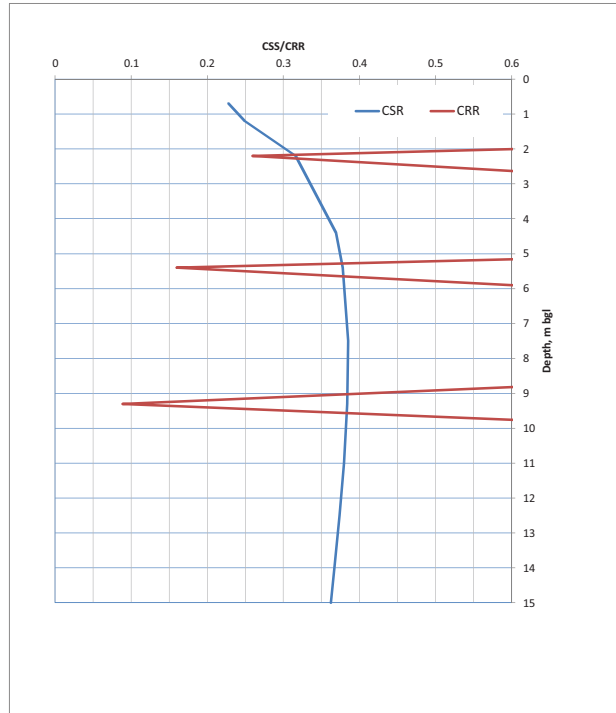
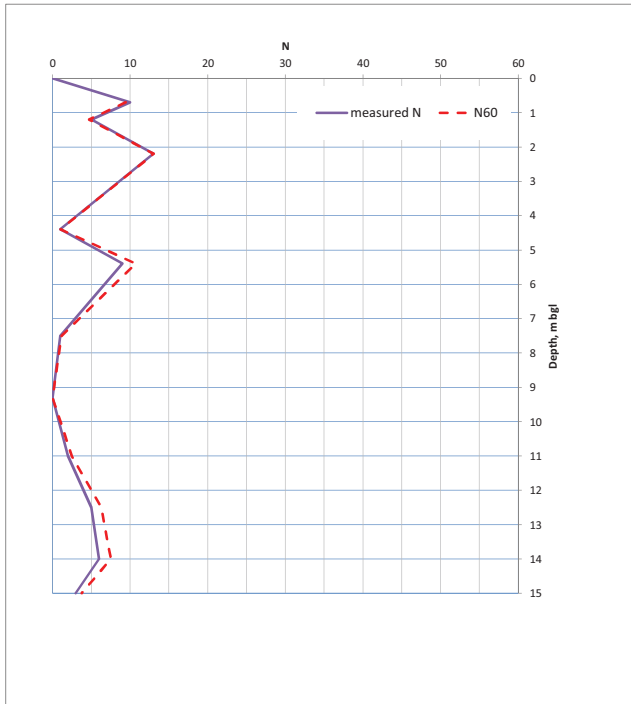
Input Parameters		Magnitude scaling factor, MSF = 1.00
Atmospheric pressure, Pa kPa	101	
Peak ground acceleration, pga =	0.35	
Earthquake magnitude, M=	7.5	
Water table depth, m =	1.0	
Average γ_T above water table, kN/m3 =	18.0	
Average γ_T below water table, kN/m3 =	19.0	
Borehole diameter (mm)	70	
Requires correction for sample liners (Yes/No)	No	
Rod Lengths (m)	1.5	

ULS event - (Settlement is limited to the top 10m)

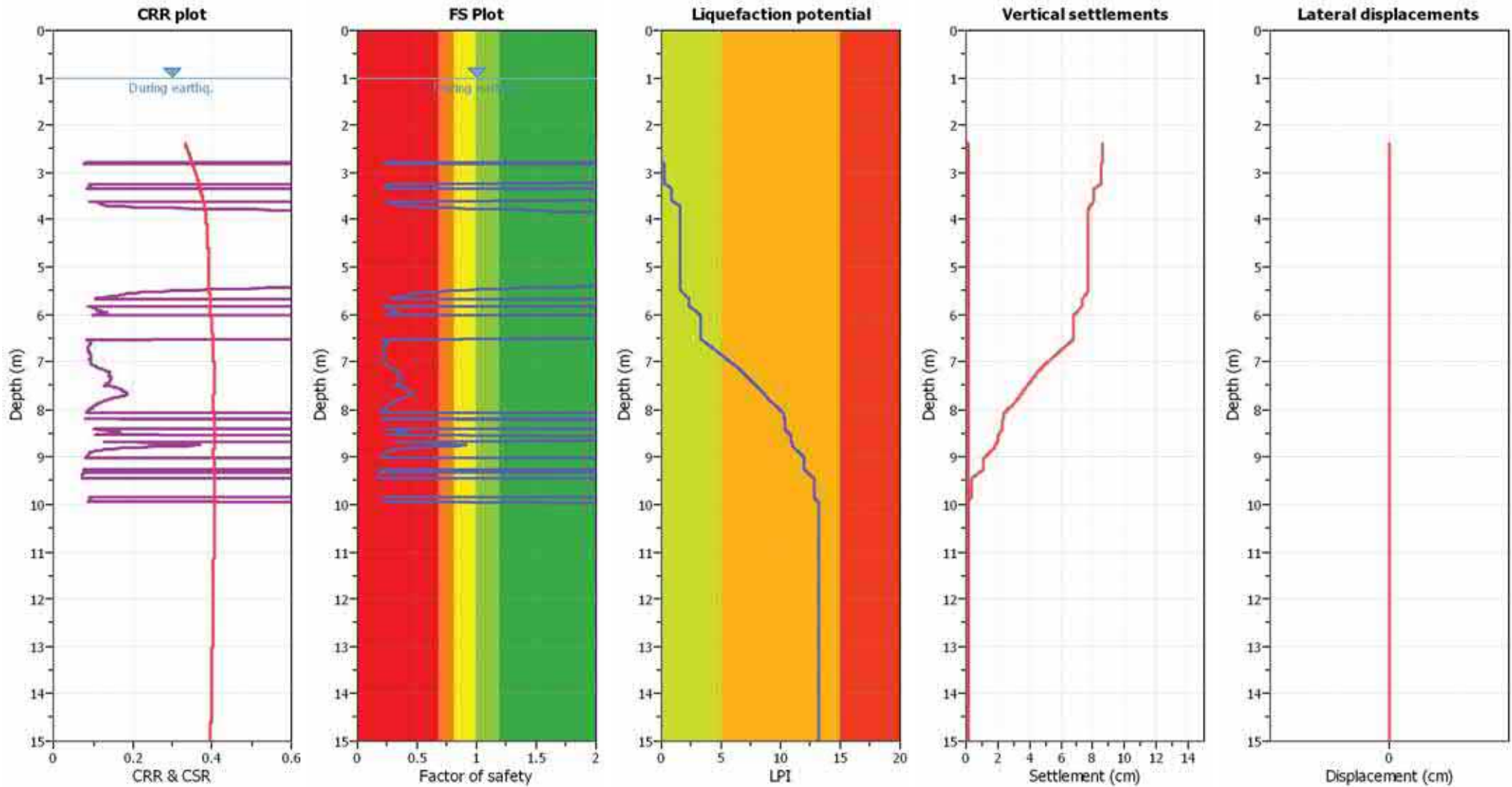
BH001

14 Mary Muller Drive, Hillsborough

SPT sample number	Depth to bottom of layer (m)	measured N	Description	Soil Classification	Flag "Clay" "Unsat" "Unreliable"	Fines Content (%)	Energy Ratio (%)	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	σ_{vc}^1 (kPa)	C_N	$(N_1)_{60}$	ΔN for fines content	$(N_1)_{60cs}$	Stress reduction coeff, r_d	CSR	MSF for sand	K_σ for sand	CRR for M=7.5 & $\sigma_{vc}^1=1atm$	CRR	FOS	Limiting shear strain, γ_{lim}	F_α	Max shear strain, γ_{max}	ΔH_{liq} (m)	ΔLDI (m)	Vertical reconsolidation strain, ϵ_v	Settlement, ΔS_1 (mm)	Cumulative settlement, ΣS (mm)		
0	0	0																																	
1	0.70	10	gravel fill	0	Unsat	5	75.3	1.26	1.00	0.75	1.0	9.4	13	13	1.70	16.0	0.0	16.00	1.00	0.228	1.00	1.10	0.165	2.000	2.00	0.25	0.712	0.000	0.00	0.000	0.000	0	137		
2	1.20	5	silt	silt	clay	50	75.3	1.26	1.00	0.75	1.0	4.7	22	20	1.70	N.A.	n.a.	16.00	1.00	0.249	1.00	1.10	0.165	2.000	2.00	0.00	0.000	0.000	0.00	0.000	0.000	0	137		
3	2.20	13	gravel fill	0	0	5	75.3	1.26	1.00	0.80	1.0	13.1	41	29	1.70	22.2	0.0	22.19	0.99	0.316	1.00	1.10	0.236	0.260	0.82	0.12	0.398	0.059	1.00	0.059	0.015	15	137		
4	4.40	1	silt	silt	clay	50	75.3	1.26	1.00	0.85	1.0	1.1	83	49	1.66	N.A.	n.a.	n.a.	0.97	0.369	1.00	1.10	n.a.	2.000	2.00	0.00	0.000	0.000	2.20	0.000	0.000	0	122		
5	5.40	9	silt	0	0	5	75.3	1.26	1.00	0.95	1.0	10.7	102	58	1.34	14.4	0.0	14.36	0.96	0.378	1.00	1.06	0.151	0.160	0.42	0.29	0.780	0.295	1.00	0.295	0.030	30	122		
6	7.50	1	silt	silt	clay	50	75.3	1.26	1.00	0.95	1.0	1.2	142	78	1.20	N.A.	n.a.	n.a.	0.93	0.385	1.00	1.08	n.a.	2.000	2.00	0.00	0.000	0.000	2.10	0.000	0.000	0	92		
7	9.30	0	sandy silt	0	0	30	75.3	1.26	1.00	1.00	1.0	0.0	176	94	1.06	0.0	5.4	5.36	0.91	0.384	1.00	1.01	0.088	0.089	0.23	0.50	0.948	0.500	1.80	0.900	0.051	92	92		
8	11.00	2	silt	silt	clay	50	75.3	1.26	1.00	1.00	1.0	2.5	208	110	0.95	N.A.	n.a.	n.a.	0.88	0.380	1.00	0.98	n.a.	2.000	2.00	0.00	0.000	0.000	1.70	0.000	0.000	0	0		
9	12.50	5	silt	silt	clay	50	75.3	1.26	1.00	1.00	1.0	6.3	237	124	0.89	N.A.	n.a.	n.a.	0.86	0.374	1.00	0.94	n.a.	2.000	2.00	0.00	0.000	0.000	1.50	0.000	0.000	0	0		
10	14.00	6	silt	silt	clay	50	75.3	1.26	1.00	1.00	1.0	7.5	265	137	0.84	N.A.	n.a.	n.a.	0.84	0.367	1.00	0.91	n.a.	2.000	2.00	0.00	0.000	0.000	1.50	0.000	0.000	0	0		
11	15.00	3	silt	silt	clay	50	75.3	1.26	1.00	1.00	1.0	3.8	284	147	0.79	N.A.	n.a.	n.a.	0.82	0.362	1.00	0.89	n.a.	2.000	2.00	0.00	0.000	0.000	1.00	0.000	0.000	0	0		



Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _σ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	10.00 m

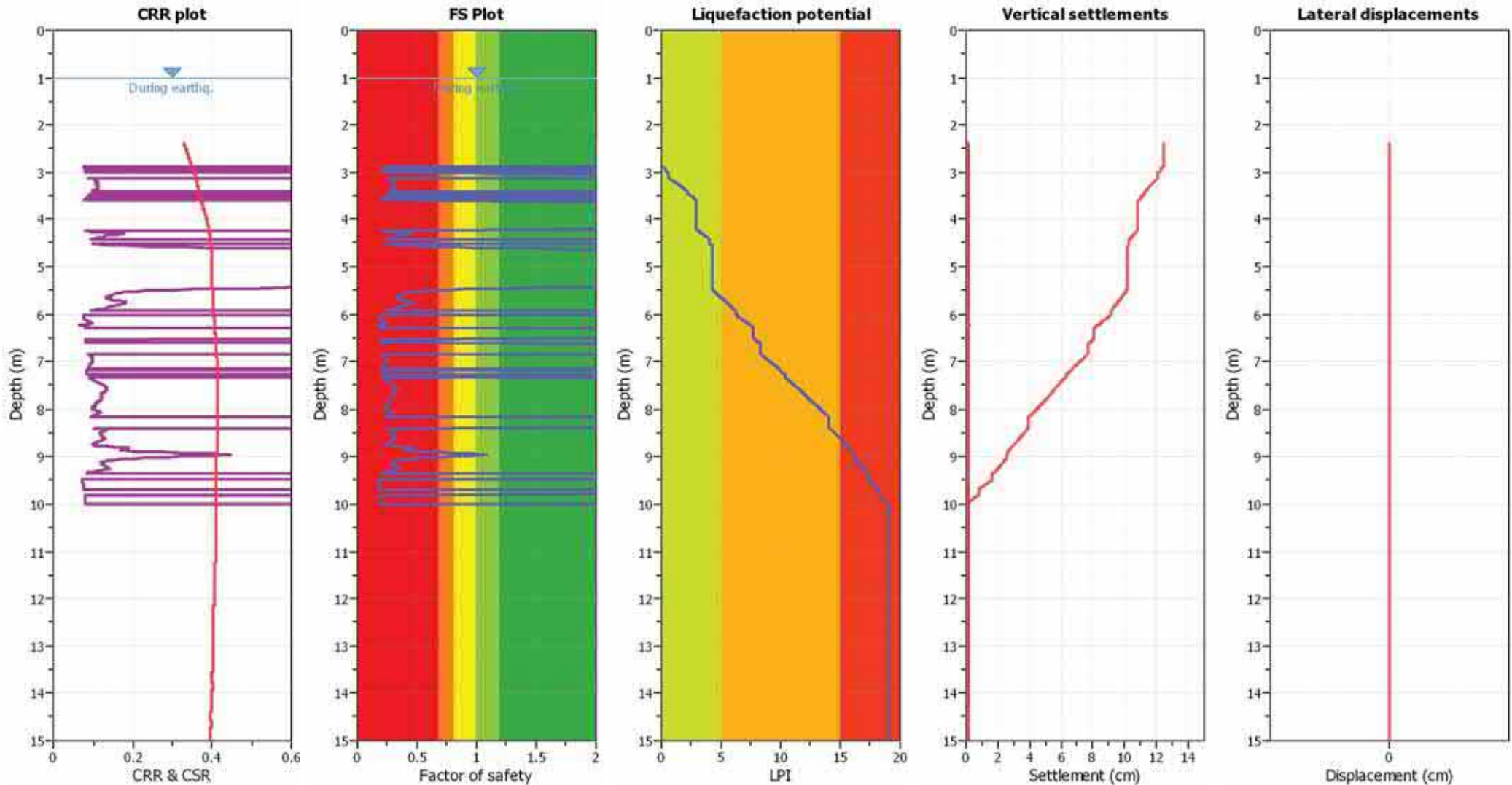
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
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LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plots



Input parameters and analysis data

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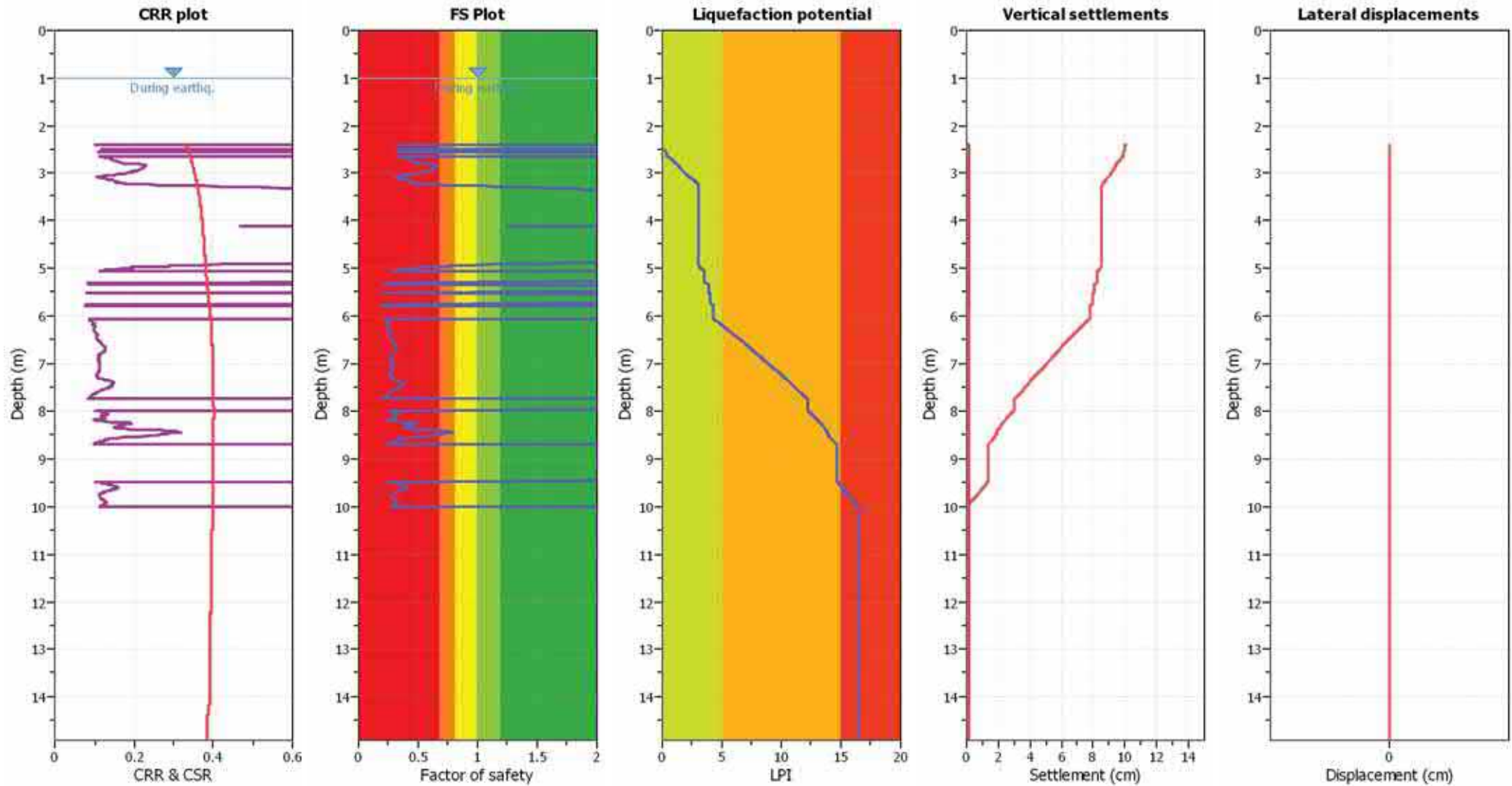
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Liquefaction analysis overall plots



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Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	Yes
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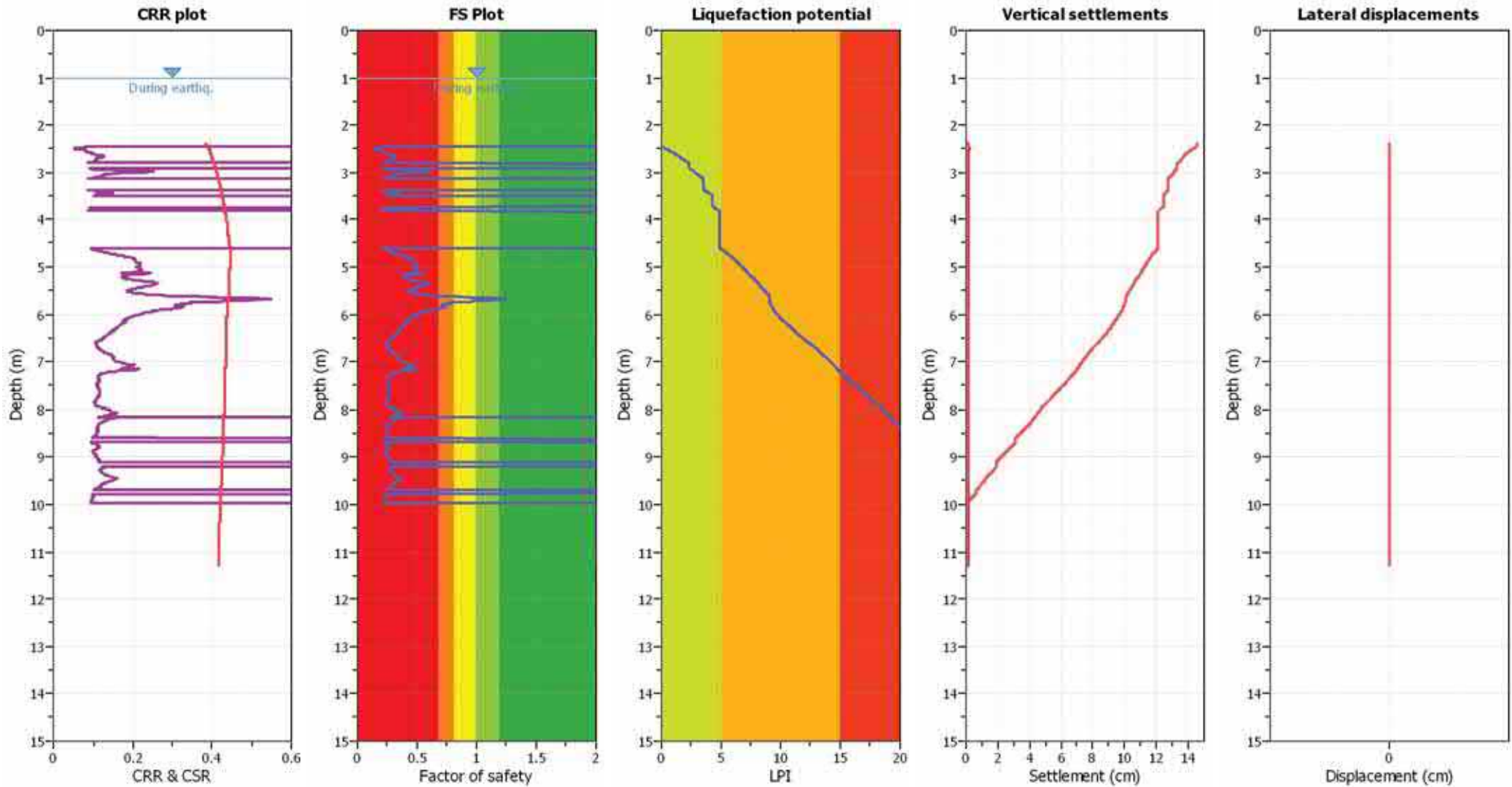
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Liquefaction analysis overall plots



Input parameters and analysis data

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Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	10.00 m

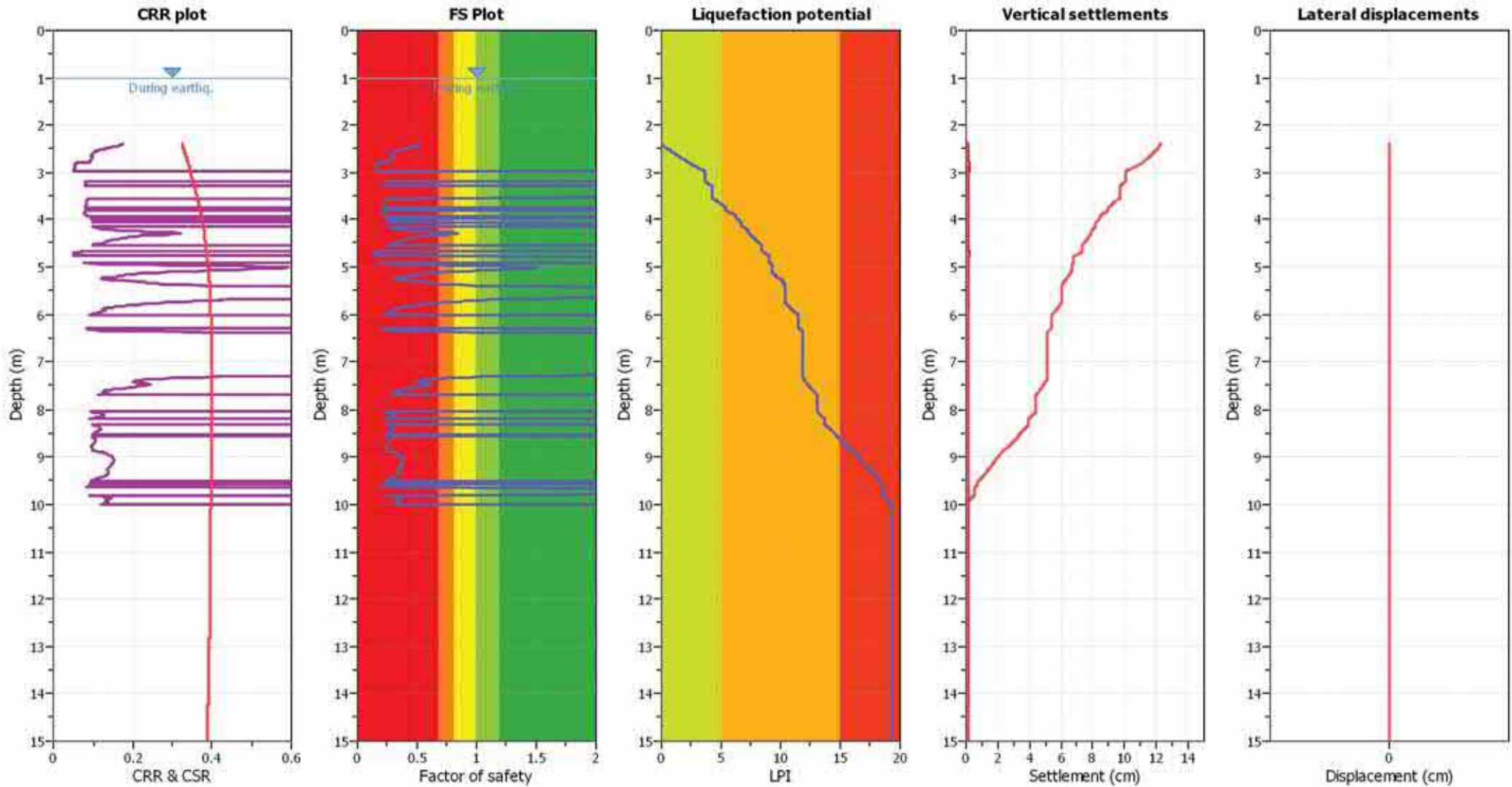
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Liquefaction analysis overall plots



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Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
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Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	10.00 m

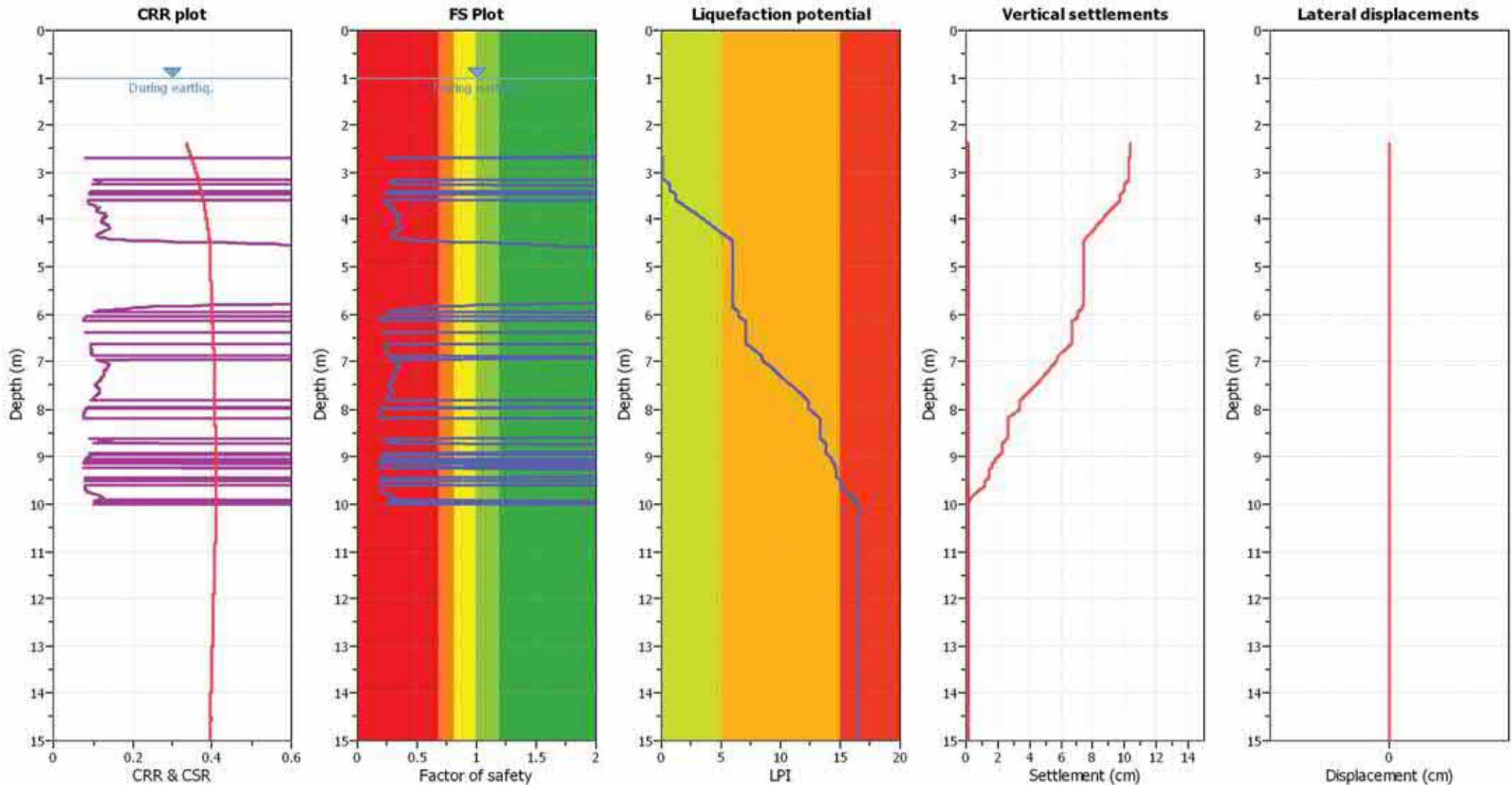
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Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _σ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	10.00 m

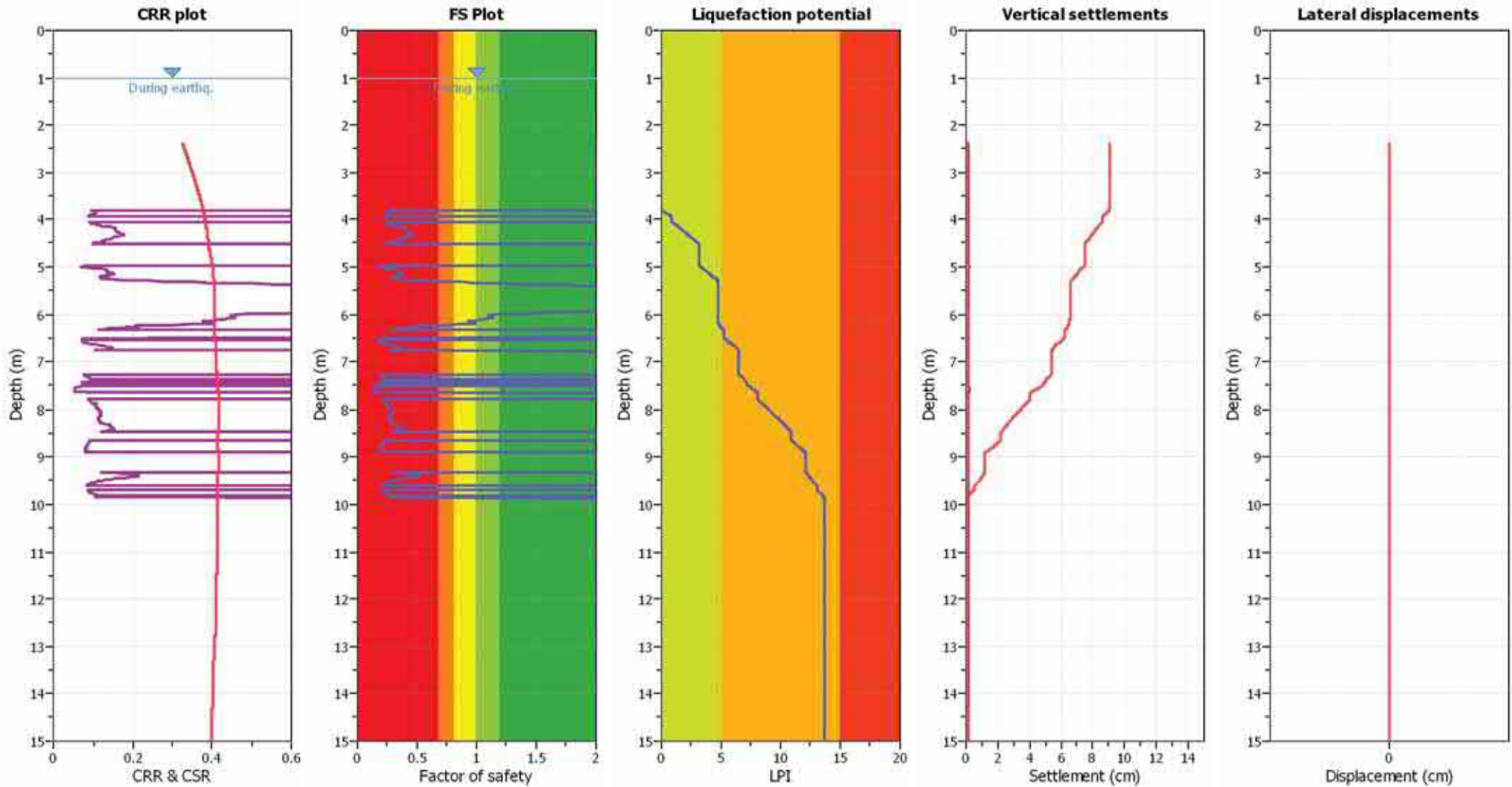
F.S. color scheme

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Liquefaction analysis overall plots



Input parameters and analysis data

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Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _σ applied:	Yes
Earthquake magnitude M _w :	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	10.00 m

F.S. color scheme

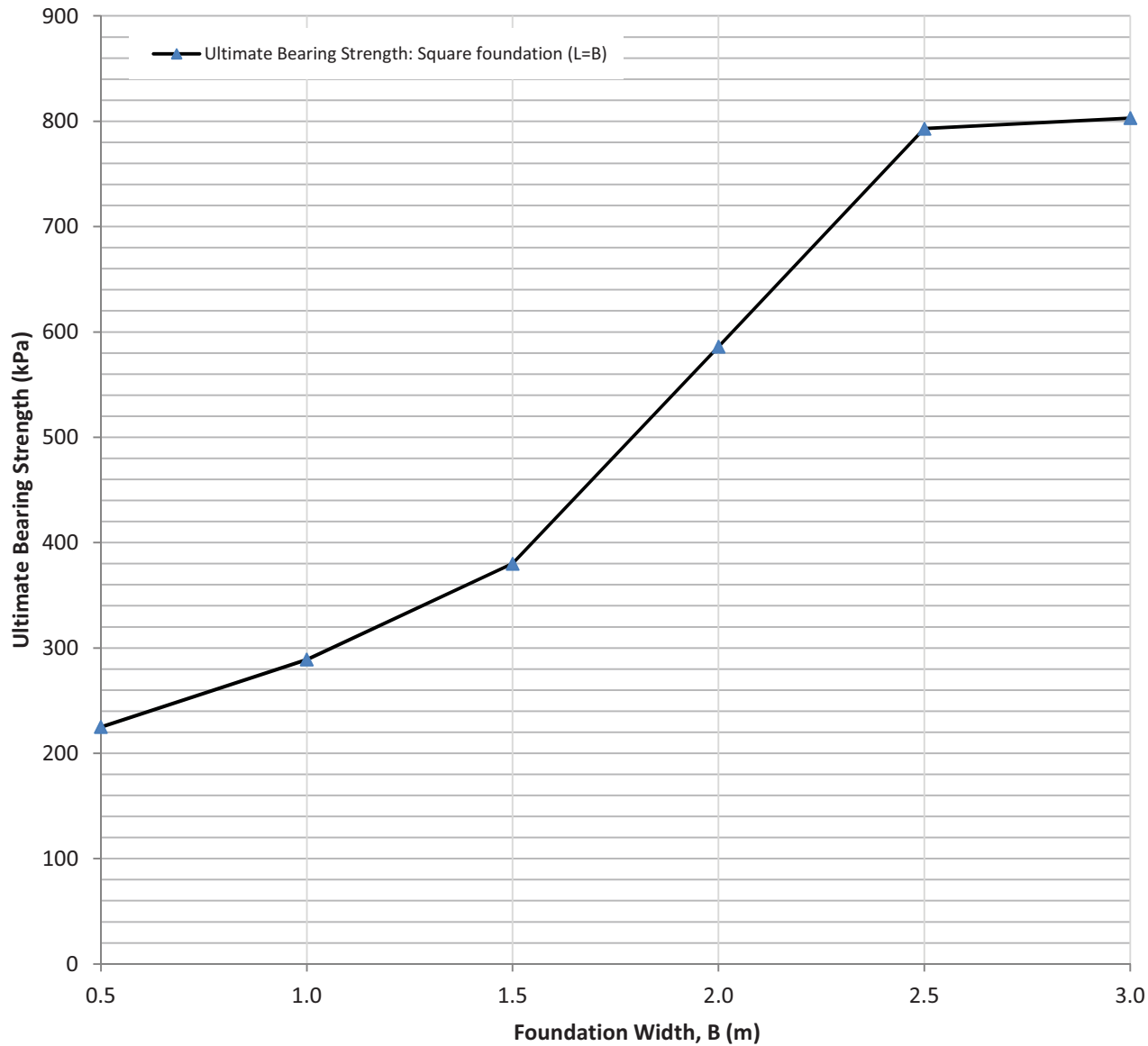
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LPI color scheme

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- Low risk

**Appendix D : BEARING CAPACITY CONTROL OF SHALLOW
FOUNDATIONS**

BEARING CAPACITY OF SHALLOW FOUNDATIONS



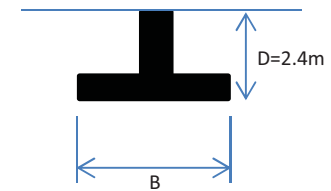
Notes:

Foundation depth = 2.4m below existing GL.

Assumes foundation extends through any fill materials, and bears onto insitu soils.

For rectangular footings with length(L) : width(B) ratio upto 3 (ie L=3B), use smallest dimension (B) to determine ultimate bearing strength.

Structural Engineer should also design bearing strength of chart 'Settlement Control of Shallow Foundations'.



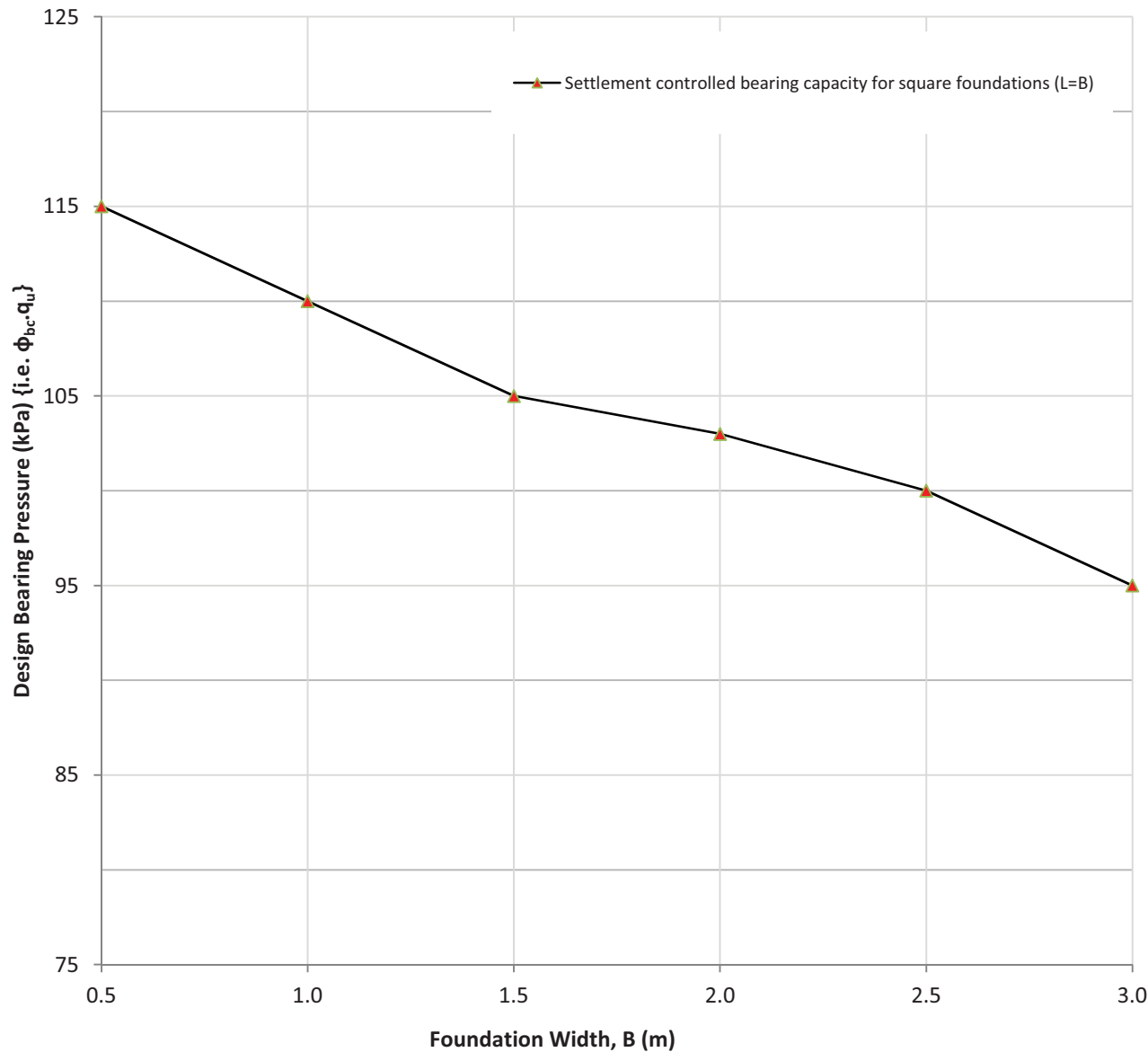
Project: 14 Mary Muller Drive
Client: Castle Rock Properties Ltd.
Job #: 382221
Date: 18 December 2013
Title: Bearing Capacity of Shallow Foundations
Eliot Sinclair surveyors engineers planners

Eliot Sinclair & Partners Ltd

Geotechnical Interpretive Report
14 Mary Muller Drive, Hillsborough, Christchurch

Appendix E : SETTLEMENT CONTROL OF SHALLOW FOUNDATIONS

SETTLEMENT CONTROL OF SHALLOW FOUNDATIONS



Notes:

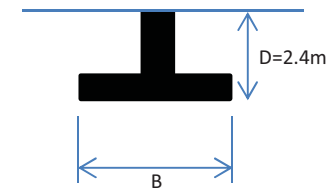
Foundation depth = 2.4m below existing GL.

Assumes foundation extends through any fill materials, and bears onto insitu soils.

Duration = 50 years, max 25mm total settlement under non-seismic conditions, calculated using Modulus of Elasticity.

Structural Engineer should also check bearing capacity of foundations, refer to 'Bearing Capacity of Shallow Foundations' chart.

Ultimate bearing capacity at smaller foundation width has been conservatively limited as shown to take into account the presence of lower soil layers.

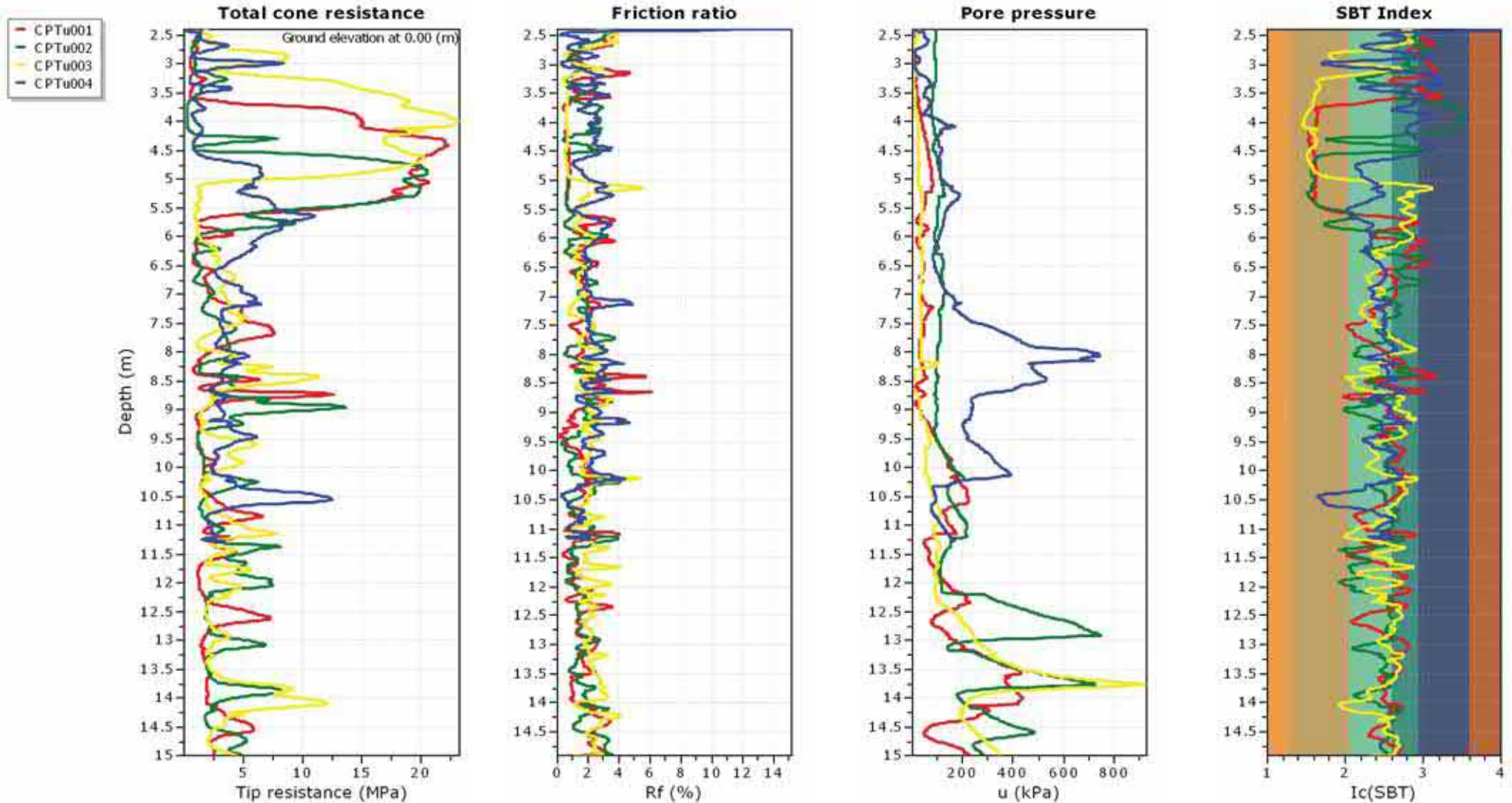


Project: 14 Mary Mullar Drive
Client: Castle Rock Properties Ltd
Job #: 382221
Date: 18 December 2013
Title: Settlement control of shallow foundations
Eliot Sinclair surveyors engineers planners

Appendix F : GEOTECHNICAL PARAMETERS

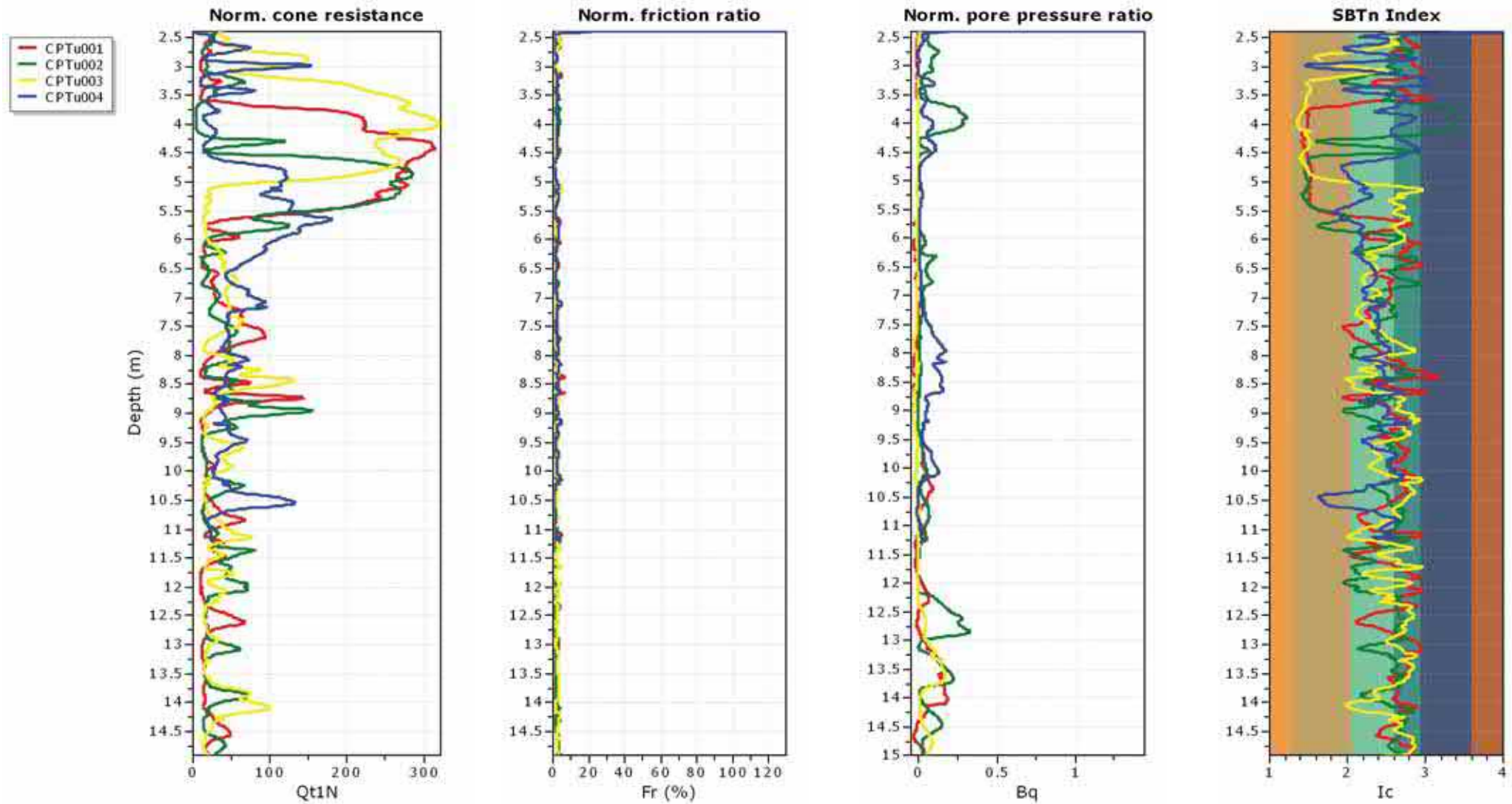
Project: 382221 - 14 Mary Muller Drive, Hillsborough
Location: 14 Mary Muller Drive, Hillsborough

Overlay basic interpretation plots



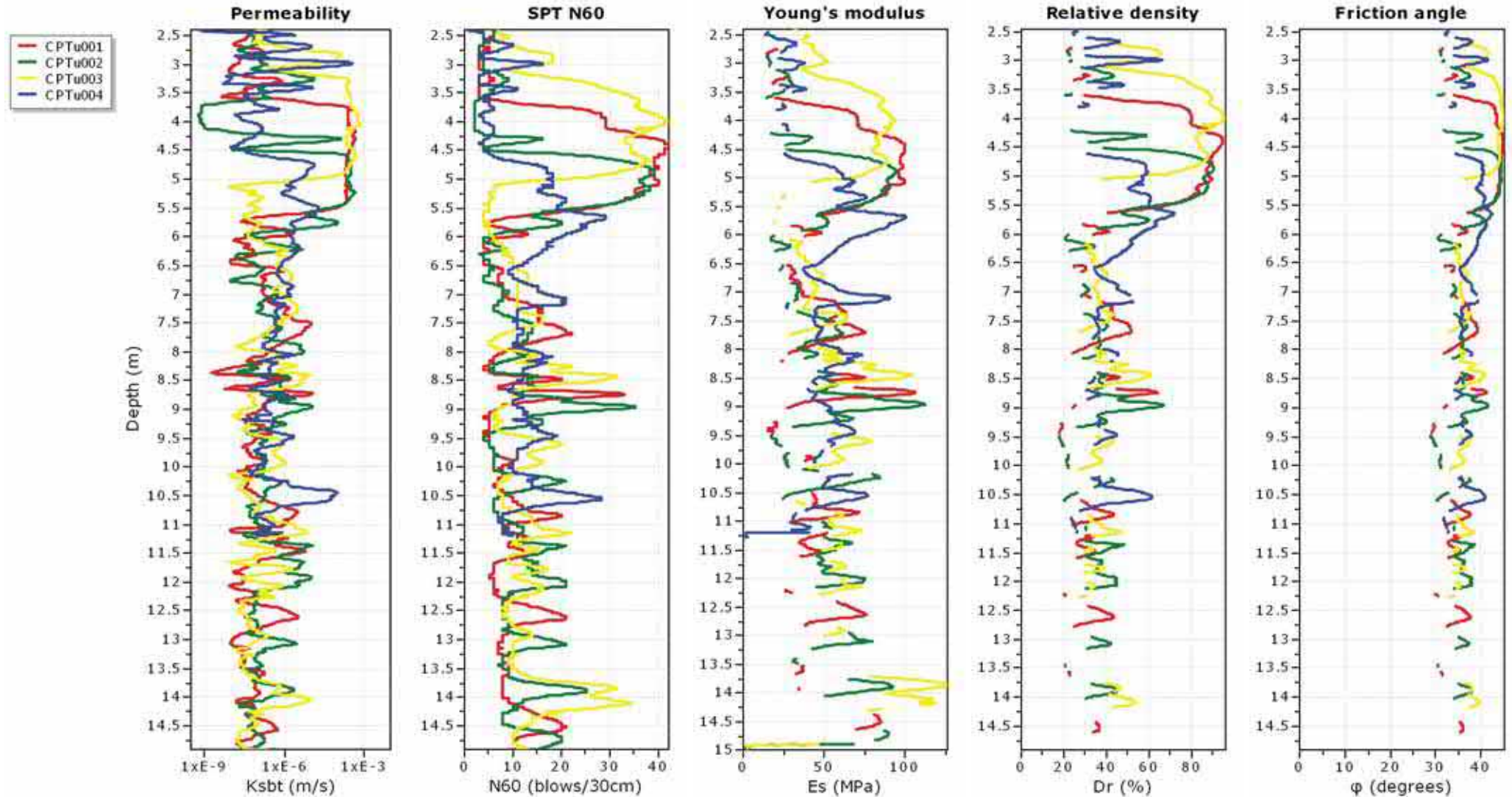
Project: 382221 - 14 Mary Muller Drive, Hillsborough
Location: 14 Mary Muller Drive, Hillsborough

Normalized basic plots



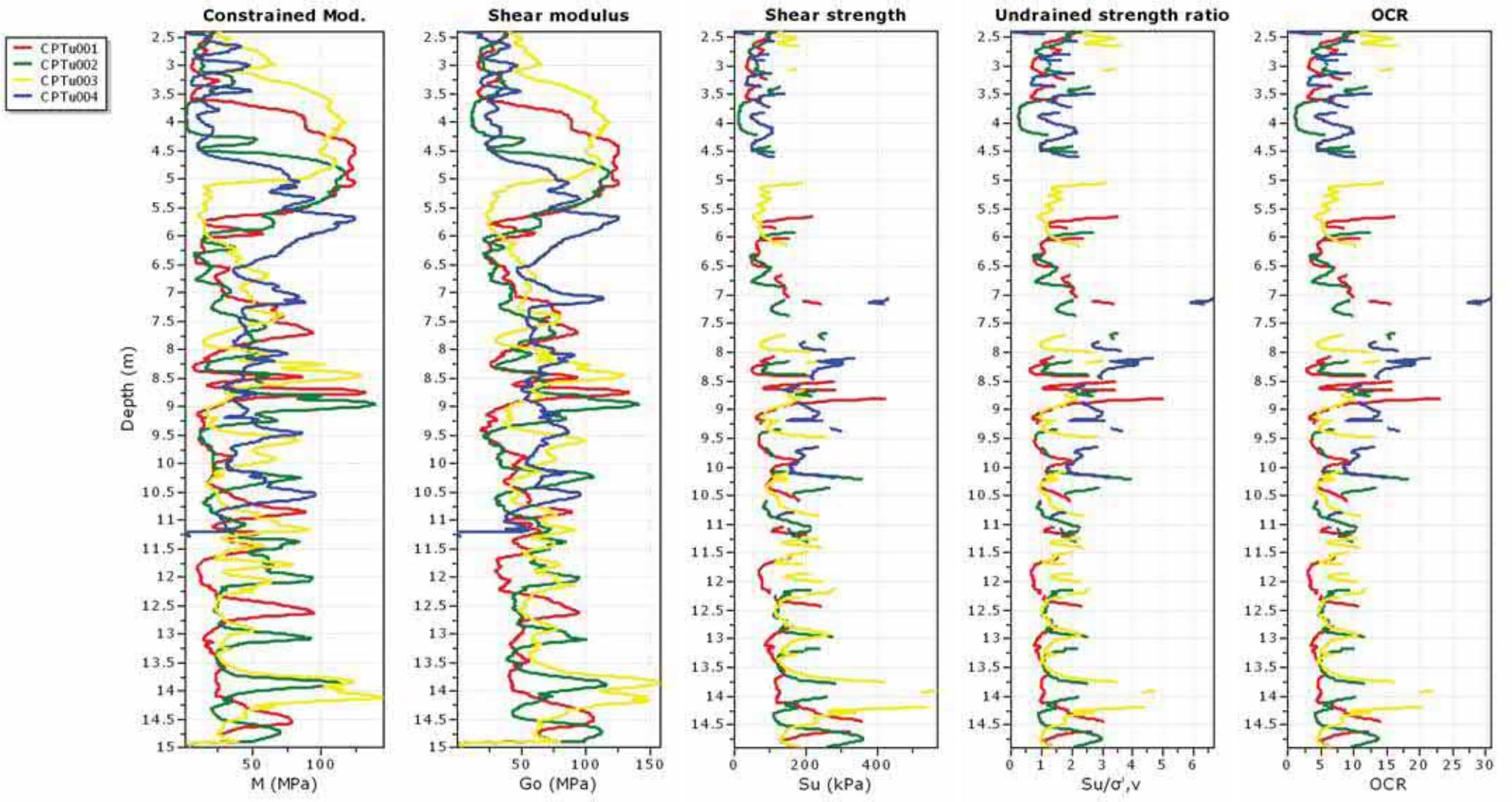
Project: 382221 - 14 Mary Muller Drive, Hillsborough
Location: 14 Mary Muller Drive, Hillsborough

Overlay estimation plots (1)



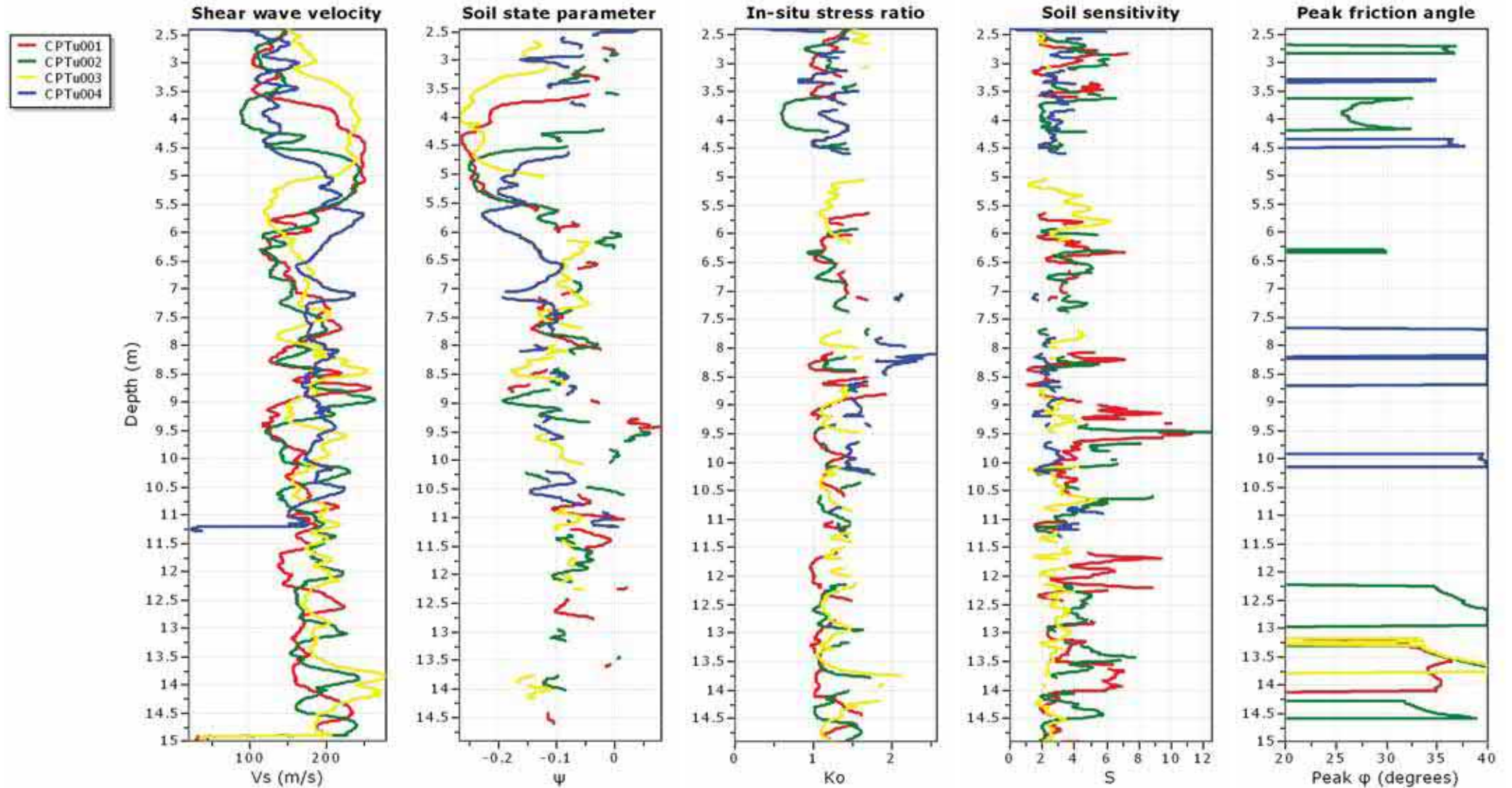
Project: 382221 - 14 Mary Muller Drive, Hillsborough
Location: 14 Mary Muller Drive, Hillsborough

Overlay estimation plots (2)



Project: 382221 - 14 Mary Muller Drive, Hillsborough
Location: 14 Mary Muller Drive, Hillsborough

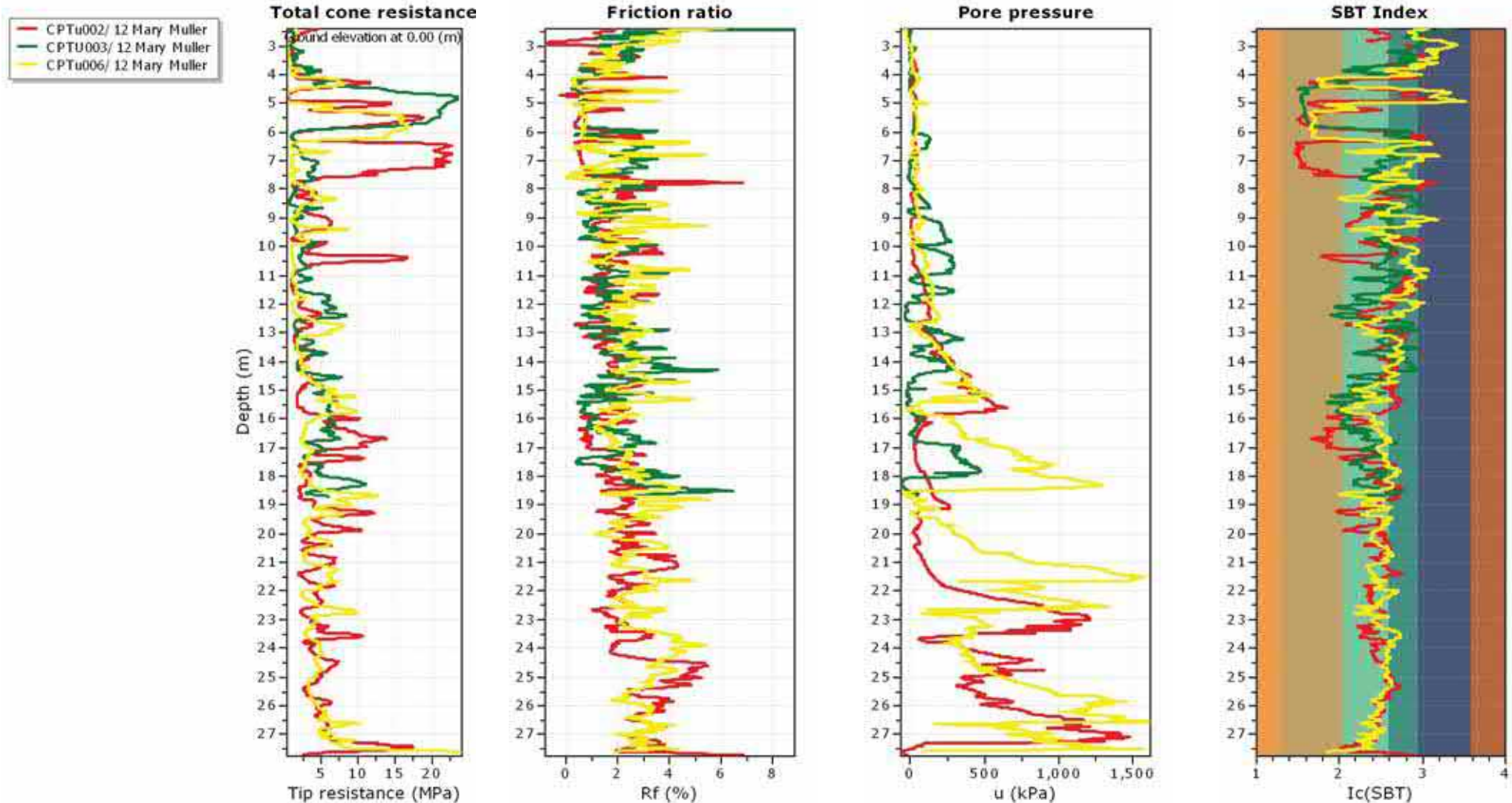
Overlay estimation plots (3)



Project: 382221 - 14 Mary Muller Drive, Hillsborough

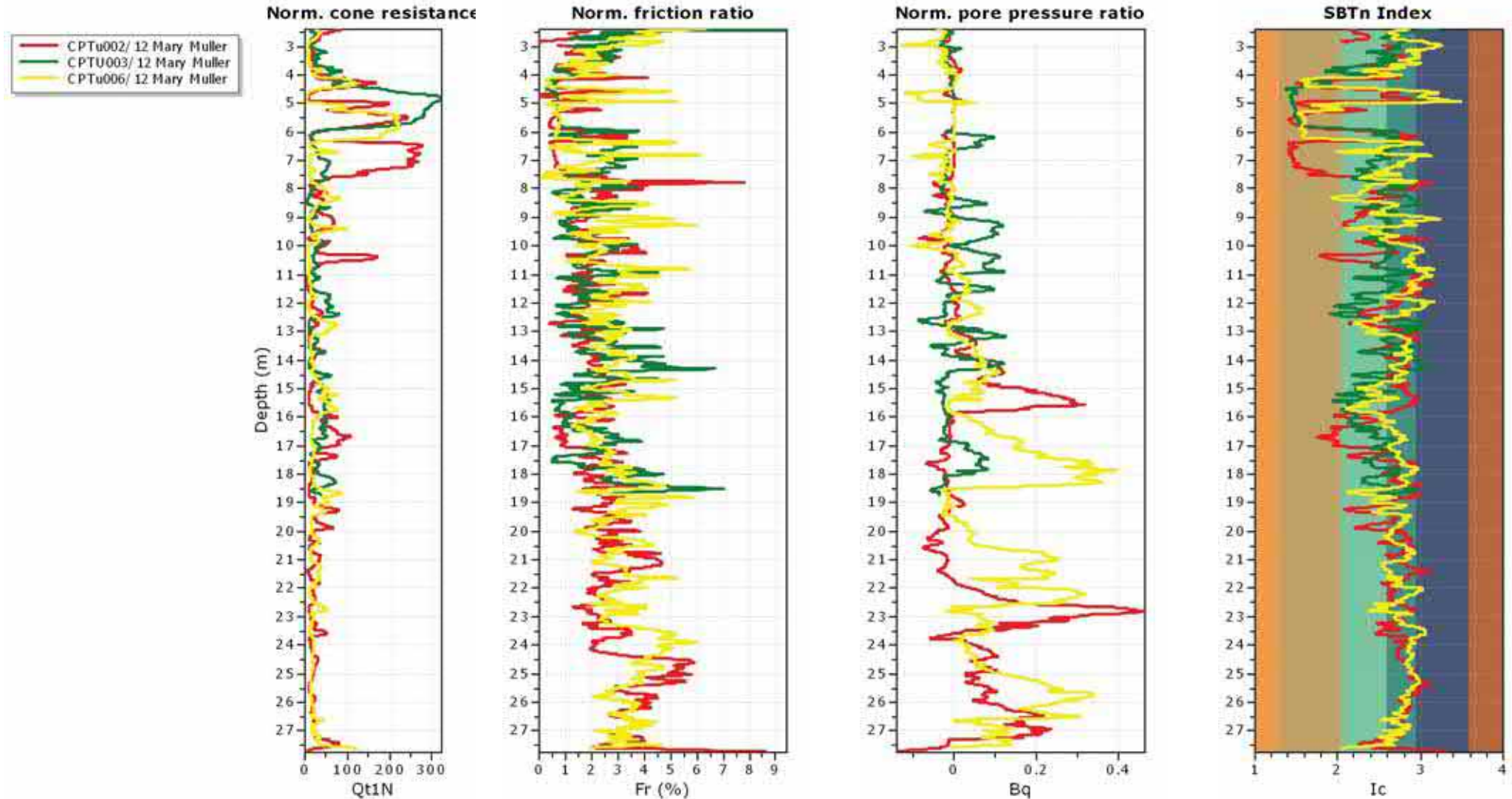
Location: 12 Mary Muller Drive, Hillsborough

Overlay basic interpretation plots



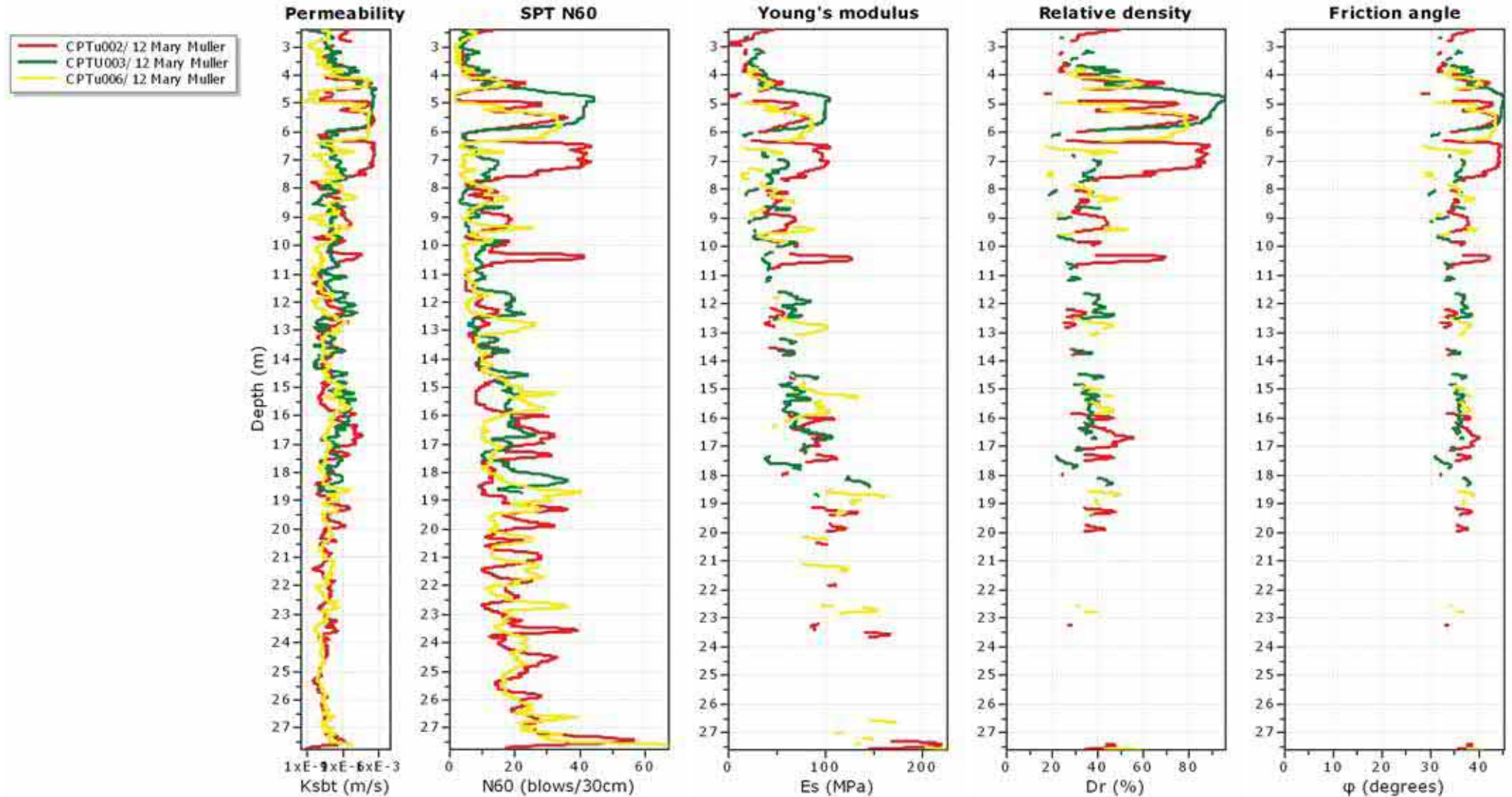
Project: 382221 - 14 Mary Muller Drive, Hillsborough
Location: 12 Mary Muller Drive, Hillsborough

Normalized basic plots



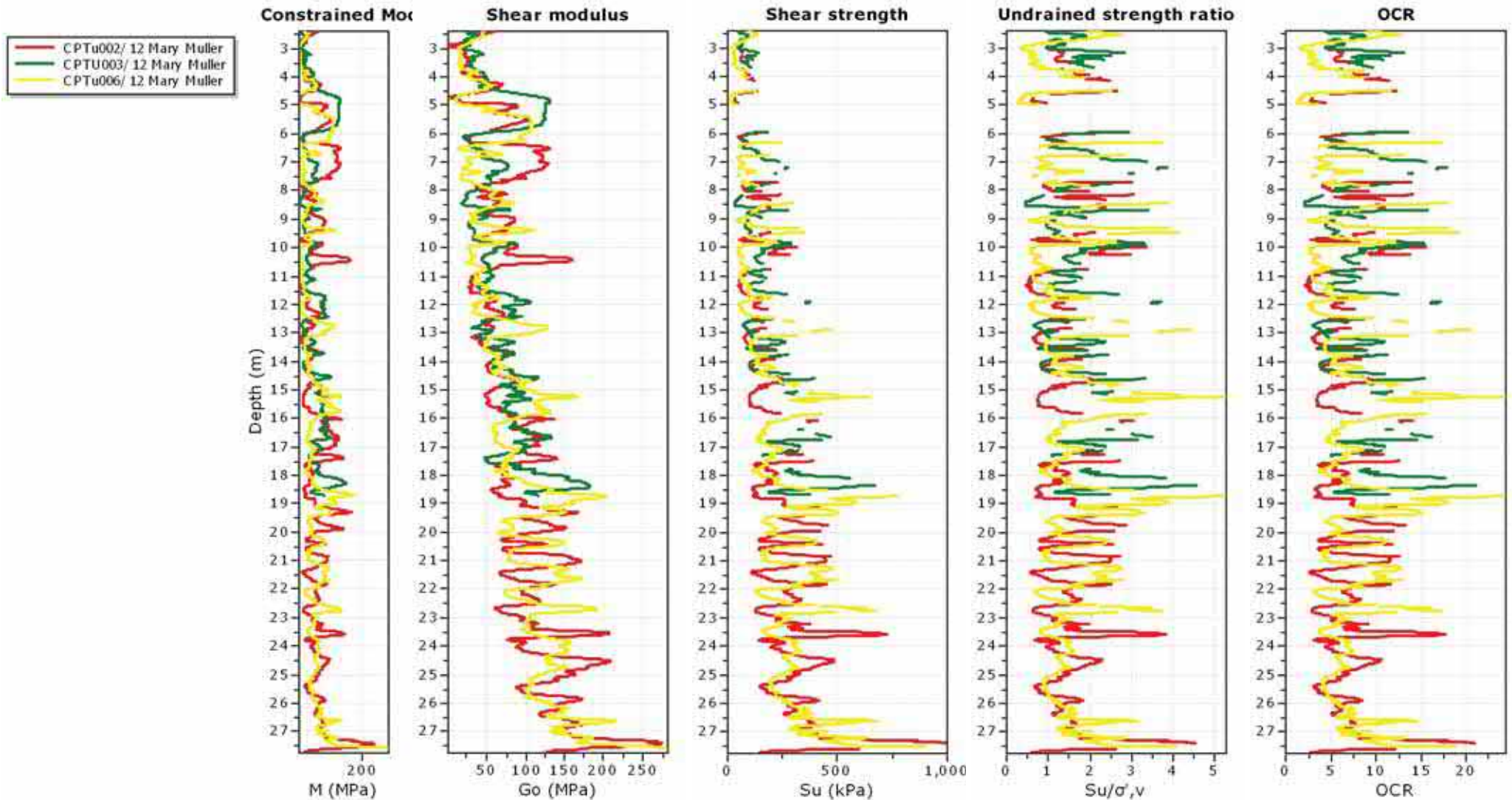
Project: 382221 - 14 Mary Muller Drive, Hillsborough
Location: 12 Mary Muller Drive, Hillsborough

Overlay estimation plots (1)



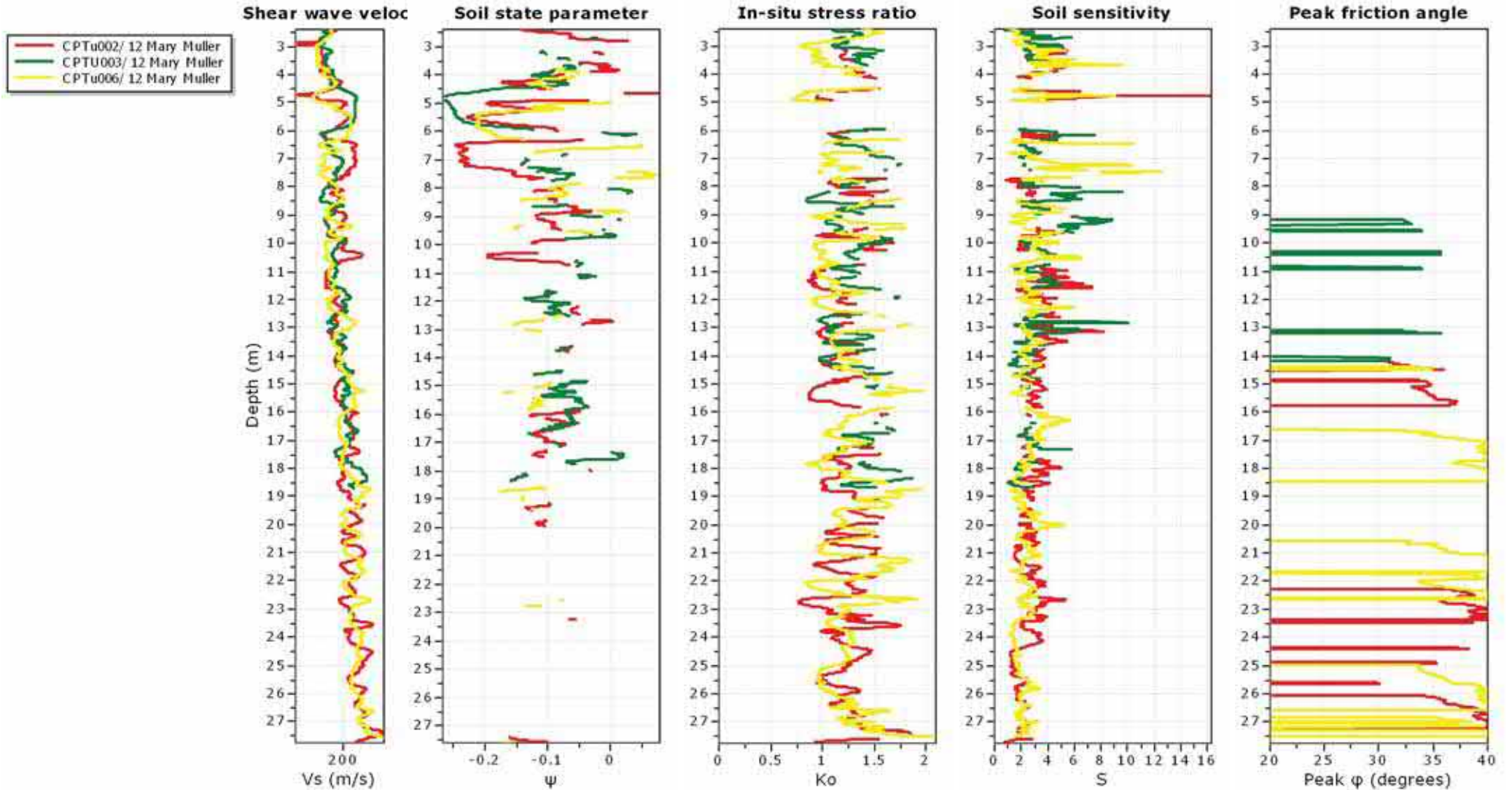
Project: 382221 - 14 Mary Muller Drive, Hillsborough
Location: 12 Mary Muller Drive, Hillsborough

Overlay estimation plots (2)



Project: 382221 - 14 Mary Muller Drive, Hillsborough
Location: 12 Mary Muller Drive, Hillsborough

Overlay estimation plots (3)



Certificate of Acceptance

Section 99, Building Act 2004

Form 9 – Building (Forms) Regulations 2004

Project number 37000664 **Date issued** 01 May 2014

The building

Street address	2 Mary Muller Drive	Location within site/block number	N/A
Legal description	Lot No: 2 DP: 3692999	Level/unit number	N/A
Building name	Macpac Distribution Centre		

The owner

Name of owner	Castle Rock Properties Ltd	Phone numbers	
Contact person	Graham Harris	Landline	03 9402300
Mailing address	PO Box 22-542 Christchurch	Mobile	N/A
		Daytime	N/A
Street address/ registered office	10A Chamans Road Christchurch	After hours	N/A
		Fax	N/A
Email	N/A	Website	N/A

First point of contact for communications with the council:

Name	Alan Reay Consultants Ltd	Phone	03 3660434
Contact person	Jeremy Mitchell	Mobile	N/A
Mailing address	PO Box 3911 Christchurch	Fax	03 3793981
		Email	jmittchell@arcl.co.nz

Acceptance of compliance

The Christchurch City Council is satisfied, to the best of its knowledge and belief and on reasonable grounds, that, insofar as it can ascertain, the building work described below complies with the building code:

The council was only able to inspect the following parts of the building work and this certificate is qualified as follows:

Description of Work:

- New roof bracing & new support to perimeter

The work is subject to the following exclusion:

- All items that are hidden from view and/or are not able to be inspected or verified
- All electrical fittings, wiring and switchgear installed as part of the fit out works

Clarification/Verification:

Clause B1 [Structure]

Complies with B1.1, B1.2, B1.3.1, B1.3.2 and B1.3.3 [a, b, f, h & j]

Verified by:

- PS1-design issued by Alan Reay Consultants Ltd [CPEng No: 34213]
- PS4-construction review issued by Alan Reay Consultants Ltd.
- PS3-construction issued by Hanham & Philp Contractors Ltd.
- Structural Design Features Report issued by Alan Reay Consultants Ltd.
- DEE Report issued by Alan Reay Consultants Ltd.
- Structural Calculations.
- Construction Drawings.
- Site Welding Inspection Report issued by Southern QA Ltd.
- Site inspection by Watkins Consultants Ltd 30 September 2013.

Clause B2 [Durability]

Complies with B2.1, B2.2 and B2.3.1 [a]

Verified by:

- PS3-construction issued by Hanham & Philp Contractors Ltd.
- Installed to Engineers specifications and of material fit for purpose.
- Site inspection by Watkins Consultants Ltd 30 September 2013.

This certificate is based on the following information:

- Certificate of Acceptance application form.
- Accompanying documents
- Inspection report, observations and photographs from Watkins Consultants

Nothing in this certificate limits the requirement that a person must not carry out building work except in accordance with a building consent, nor does it relieve any person from the requirement to obtain a building consent for building work.

Attachments:

- Stamped approved documents

Signature:



Van Heerden, Jennifer

01/05/2014 3:35 PM

Position: Building Control Officer

On behalf of the Christchurch City Council



Alan Reay Consultants

**Macpac Distribution
Centre,**

4 Mary Muller Drive

**DETAILED ENGINEERING
EVALUATION REPORT**

**PHASE 2: WAREHOUSE
AND OFFICE REPAIRS
CARRIED OUT**

Prepared for:

Castle Rock Properties Ltd

By:

**ALAN REAY CONSULTANTS
LIMITED**

Date:

10 July 2013

**Innovation
by design**

Alan Reay Consultants Ltd
395 Madras Street
P O Box 3911
Christchurch
New Zealand
Tel 03 366 0434
Fax 03 379 3981
Email eng@arcl.co.nz
Internet www.arcl.co.nz

Macpac Distribution Centre, 4 Mary Muller Drive
DETAILED ENGINEERING EVALUATION REPORT
PHASE 2: WAREHOUSE AND OFFICE REPAIRS CARRIED OUT

Prepared by:




Jeremy Mitchell (Structural Engineer)
 On behalf of
 Alan Reay Consultants Ltd

Reviewed by:



Dr Alan Reay (CPEng)
 On behalf of
 Alan Reay Consultants Ltd

Approved for
 issue by:



Dr Alan Reay
 On behalf of
 Alan Reay Consultants Ltd

DOCUMENT HISTORY

REVISION	DATE	NOTES	DISTRIBUTION
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EXECUTIVE SUMMARY

This Detailed Engineering Evaluation (DEE) Report has been prepared for Castle Rock Properties Ltd as our client in relation to the post earthquake condition of the Macpac Distribution Centre Located at 4 Mary Muller Drive, Christchurch.

The purpose of a DEE is to assist building owners to make informed decisions about the continued use of their buildings. It also provides a starting point for identifying repairs which may be required to restore the building to as near as practical to its condition prior to the earthquakes. This work is not included in this report.

The Canterbury Earthquake Recovery Authority (CERA) may request a copy of the DEE under Section 29 or 51 of the CER Act 2011.

The Macpac Distribution Centre Building consists of a warehouse and attached single level office. The office and warehouse structures consist of steel portal frames with precast concrete perimeter wall panels. The building was originally constructed in 2000 with seismic strengthening added in 2012.

The warehouse and office has been assessed in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) Initial Evaluation Procedure (IEP) as having a seismic lateral load carrying capacity of 60%-70% NBS in its pre-earthquake condition.

Alan Reay Consultants Ltd have undertaken visual observations of the building and have identified settlement of the southern end wall, cracking to concrete panels, damage to steel panel connections and damage to ground floor slab control joints as a result of the recent seismic activity in the region.

A strengthening design has been prepared in accordance with clause B1 of the New Zealand Building Code for both the warehouse and office to achieve a target lateral strength of approximately 67% of new building standard. The warehouse and office repairs have been carried out in accordance with section 41 of the New Zealand Building Act.

Based on detailed calculations, and considering the post-earthquake strengthening carried out at this time, ARCL consider that the post-earthquake lateral capacity of the warehouse and office is approximately 67% NBS.



Figure 1: Google Earth image defining the office (purple) and warehouse (red) areas.

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1. INTRODUCTION

Alan Reay Consultants Limited (ARCL) have been engaged by Peter Parmenter and Graham Harris of Castle Rock Properties Ltd to conduct a detailed engineering evaluation of the building located at 4 Mary Muller Drive following the Canterbury Earthquakes which have occurred since 4th September 2010.

1.1. Scope

The scope of work undertaken to complete this report included the following:

- Review building drawings and specification to identify the primary vertical and lateral load resisting elements.
- Using the available documentation, identify potential critical structural weaknesses or areas expected to have sustained damage.
- Observe and record building structural damage and assess its likely impact on the future seismic performance of the building.
- Complete a level survey of the building ground floor slab to identify any potential differential settlement.
- Provide a qualitative Detailed Engineering Evaluation (DEE) of the building using the '*Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury (Part 2)*' released by the DBH Engineering Advisory Group (EAG) [1].
- Provide general recommendations regarding further actions that should be considered by the building owners.

1.2. Previous Work

Since 4 September 2010 ARCL have completed the following work for the building located at 4 Mary Muller Drive.

- Post-Earthquake Site Occupancy Report dated 17 September 2012 (File 9465)
- Earthquake Structural Repair Report dated 3 December 2010 (File 9822)
- Post-Earthquake Site Occupancy Report 4 August 2012 (File 9967)
- Earthquake Damage Structural Repair Report dated 8 March 2012 (File 10402)
- Structural Engineering Evaluation Report dated 20 August 2012 (File 10402)
- Seismic Strengthening Structural Design dated 28 August 2012 (File 10402)
- Detailed Engineering Evaluation Phase 1 dated 17 October 2012 (File 10402)

1.3. Limitations

This report has been prepared solely for the benefit of Castle Rock Properties Ltd as our client, who is the owner of the building, with respect to the brief given to us. Data and opinions contained in this report may not be used in other contexts or for any other purpose without our prior review and agreement.

The observations undertaken to assess damage to this building resulting from recent earthquakes have been limited to structural aspects only. Our observations did not include an assessment of any other elements of the building or services. Items such as fire safety systems, the glazing system, racking, finishes, suspended ceilings, partitions, tenant fit-out, power, water, sewerage, mechanical services and architectural elements have not been reviewed as part of this evaluation. Where obvious damage has occurred, or the damage to these elements may be related to structural building performance, these may have been commented on.

Our investigation was limited to a high level visual examination of the building where safe and ready access existed at the time, and we have not undertaken any intrusive examinations, testing, surveying, removal or destruction of any building elements unless specifically noted otherwise in this report. Available documentation consisting of partial structural drawings (selection of drawings included in Appendix A) were reviewed as part of the investigation. No calculations or analyses were carried out, other than those required as part of the qualitative DEE process. No geotechnical investigation was carried out as part of this assessment. Seismic events and ongoing settlement subsequent to our observations may change the assessments contained in this report.

This report is necessarily limited in respect of the above, and does not address any matter that is not discoverable from such an examination, including any damage or defect in inaccessible places and latent defects. ARCL is not able to give any warranty or guarantee that all possible damage, defects, conditions or qualities have been identified. The work done by ARCL and the advice given is therefore on a reasonable endeavours basis under the circumstances.

The basis of ARCL's advice and our responsibility to our client is set out in the terms of engagement with our client, being the ACENZ Short Form Agreement for Consultant Engagement.

2. **BACKGROUND**

This section describes the background information and guidance that has been used in the preparation of this report.

2.1. ***Canterbury Earthquakes***

The following information is from the GNS Science Consultancy Report [10] prepared for the Canterbury Earthquakes Royal Commission of Inquiry.

The Canterbury Earthquake sequence was initiated at 04:35 NZST on the 4th September 2010 when the magnitude 7.1 Darfield Earthquake occurred. The earthquake occurred on a previously unknown fault now known as the Greendale Fault. The shaking caused damage to older brick and masonry buildings in the Canterbury region. A particular feature of the earthquake was the soil liquefaction that occurred in the eastern suburbs of Christchurch which caused significant lateral spreading, settlement and damage to underground services.

Since the Darfield Earthquake, there have been over 10,000 aftershocks with magnitudes up to 6.3 recorded in the Canterbury region. The most significant aftershock occurred at 12:51 NZST on the 22nd February 2011 which was recorded as a magnitude 6.3, 5km deep and centred 6km southeast of the Christchurch city centre. The aftershock caused severe damage to buildings and property throughout the Canterbury region and the casualties included 185 fatalities and several thousand injured. Widespread liquefaction occurred in eastern and central Christchurch as a result of the intense shaking. Other significant aftershocks in the sequence include a magnitude 6.3 which occurred on the 13th June 2011, and a magnitude 6.0 which occurred on the 23rd December 2011. There have been over 40 earthquakes of magnitude 5.0 and above in the Canterbury region since the 4th September 2010.

At seismological recording stations in the Christchurch CBD, shaking from the three largest earthquakes exceeded both the 1/500-year and more stringent 1/2500-year design levels in the then current New Zealand Loading Standard for particular building types. Horizontal ground accelerations of up to 2.0g and vertical ground accelerations of 2.2g have been recorded during the earthquake sequence.

There is currently an increased seismic hazard in Canterbury because of the ongoing aftershocks. In respect of this, in May 2011 the Department of Building and Housing increased the seismic hazard coefficient for the Canterbury earthquake region from 0.22 to 0.3. This effectively increases the seismic design level for new buildings by 36%. By comparison, Wellington has a seismic hazard coefficient of 0.4.

2.2. ***Detailed Engineering Evaluation (DEE)***

The purpose of a DEE is to assist building owners to make informed decisions about the continued use of their buildings. It also provides a starting point for identifying repairs which may be required to restore the building to as near as practical to its condition prior to the earthquakes.

The DEE procedure is set out in the document titled 'Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury' prepared by the Engineering Advisory Group (EAG) [1] on behalf of the DBH which was initially released in May 2011. Part 2 of the document sets out the qualitative assessment procedure which includes guidance on drawing and specification reviews, damage assessments, how to identify specific risks and

vulnerabilities (critical structural weaknesses (CSW's)), and assessment of ability of the structure to resist seismic loading in terms of the new building standard (%NBS).

Following the qualitative assessment, the guidance document [1] also provides a required course of action depending on the %NBS assessment and the level of damage observed. The options provided in the EAG guidance document [1] are summarised in the table below:

Table 1: Recommended course of action (EAG guidance document [1])

	Insignificant damage	Insignificant damage with potential collapse hazard or CSW's	Significant damage
<33%NBS	Quantitative assessment required	Quantitative assessment required	Quantitative assessment required
33-100+%NBS	No further assessment required	Mitigation of the collapse hazard or CSW is strongly recommended	Quantitative assessment required

For all of the above conditions the EAG guidance document [1] recommends strengthening for any building with %NBS<67%. For significantly damaged or 'earthquake prone' buildings (see section 2.4 below for definition) a quantitative assessment is required. The extent of this assessment is dependent of the level of damaged observed and the type of structure being assessed but is likely to necessitate further invasive investigations, materials testing and detailed geotechnical and structural analysis.

Further advice for building owners regarding the DEE process is available to view on the DBH website.

2.3. %NBS Assessment

%NBS is the percentage of New Building Standard which provides a comparison of the assessed building performance relative to the current New Zealand Building Code (NZBC) requirements for seismic design for a new building.

2.4. Earthquake Prone Building (EPB) Policy

The Building Act 2004 [5] has specific provisions relating to earthquake prone buildings. The primary purpose of this legislation is to reduce earthquake risk in the community. The definition of an '**earthquake prone**' building (under the Building Act 2004) is a building that is likely to collapse in a moderate earthquake (defined as 33%NBS of the current seismic loading standard in the Regulations) causing injury/death or damage to other property. The Building Act stipulates that Territorial Authorities are required to establish an earthquake prone building policy which discusses how to identify, assess and take action on EPB's.

The Christchurch City Council adopted the current 'Earthquake-Prone, Dangerous and Insanitary Buildings Policy 2010' [7] following the September 2010 Canterbury Earthquake. The policy references the New Zealand Society for Earthquake Engineering (NZSEE) Recommendations [3] as a basis for defining the appropriate levels of strengthening and repairs for existing buildings. A copy of the current policy can be viewed on the CCC website. Currently these NZSEE recommendations [3] suggest that any building below 67%NBS be regarded as an '**earthquake risk**'

building and therefore effort should be made to improve the structural performance of these buildings to 100%NBS where possible or at least 67%NBS. The following table taken from the NZSEE recommendations [3] indicates the relative seismic risk of a building based on the assessed %NBS. The table shows that a building with a seismic capacity of 20-33% NBS is 10-25 times more likely to have its ultimate capacity exceeded during an earthquake than a new building.

Table 2: Relative Risk Comparison

%NBS	Relative risk	Description
>100	<1 time	Low risk
80-100	1-2 times	Low risk
67-80	2-5 times	Low risk
33-67	5-10 times	Moderate risk
20-33	10-25 times	High risk
<20	>25 times	High risk

3. BUILDING DESCRIPTION AND PRE-EARTHQUAKE STRUCTURAL CONDITION

This section describes the structural features of the building based on visual observations of the building and a review of the available documentation. No assessment has been made to determine if the constructed building materials and workmanship comply with the available documentation.

3.1. *Site Description*

The building is located in an industrial area at 4 Mary Muller Drive. The building was originally constructed in 2000 with seismic strengthening added in 2012.



Figure 2: Google Earth image indicating building location with the warehouse (shaded red) and office (shaded purple) defined.

3.2. *Building Form*

The building consists of a storage warehouse measuring 78mx47m and an attached single level office measuring 43mx23m. The building is used for commercial activities and would be classified as an importance level 2 structure in accordance with AS/NZS1170.0:2002.

The warehouse and office structures consists of steel portal frames with precast concrete cladding panels, concrete end wall columns and a light weight steel roof system.

3.3. Primary Lateral Load Resisting System

The primary building lateral load resisting systems noted from the available structural drawings (selected drawings included in Appendix A) are as follows:

- Across the building (east to west) lateral loads from concrete cladding panels and the roof are resisted by steel portal frames.
- Along the building (north to south) the warehouse roof and end wall cladding panels are supported by roof cross bracing which transfers lateral loads into the side wall cantilever panels. The end wall panels are supported at mid-height by cantilever concrete columns. Office roof lateral loads are transferred to concrete end wall panels which in turn transfer loads to cantilever concrete end wall columns which support the face loading of the panels.

3.4. Foundation System and Ground Conditions

The building foundations consist of a series of deep pads to a maximum depth of 2.2 metres beneath the building floor level. The deep pads provide vertical bearing as well as overturning resistance. Geotechnical considerations are discussed in section 4.8.

3.5. Pre-Earthquake Building Seismic Performance

ARCL consider that, in its pre-earthquake condition, the primary lateral load resisting system was able to withstand loads of approximately 60%-70%NBS. This figure is based on analyses carried out according to the EAG guidance document [1] '*Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury (Part 2)*'. A printout of the spreadsheets used to calculate this figure is provided in Appendix E.

An electronic copy of this spreadsheet may be requested by CERA under the Canterbury Earthquake Recovery (CER) Act.

It should be noted that the %NBS figure stated above includes the 36% increase in the NZBC design seismic hazard factor for new building in the Christchurch region enacted by the Department of Building and Housing in May 2011.

The above figure does not account for the strengthening work carried out following the 22 February 2011 earthquake.

4. **DAMAGE ASSESSMENT AND POST-EARTHQUAKE STRUCTURAL CONDITION**

ARCL have undertaken a number of visual observations of the Macpac Building with the final visit on 12 November 2012 following the Canterbury Earthquakes and their subsequent aftershocks which have occurred since 4th September 2010. The observation was carried out at the request of Castle Rock Properties Ltd.

4.1. **Generic Building Issues**

Appendix 4A of the EAG guidance document [1] identifies expected seismic issues with single level tilt panel buildings following an earthquake. The expected issues are listed below:

- a) Brittle panel connections and/or cracked panels at the connection.
- b) Hard-drawn wire mesh reinforcing or inadequate reinforcing contents making panel prone to non-ductile face loading failure.
- c) Panel span/thickness ratio too high, leading to panel buckling concerns.
- d) Steel bracing inadequate.
- e) Inadequate seismic separation.

Particular attention was taken during the visual review of the issues listed above where applicable. Any damage related to these issues is noted in the sections below.




4.2. **Scope of Observations**

Further to the limitations set out in section 2, the following list identifies the limits of the observations undertaken. Elements of the structure not listed below have not been specifically observed. All observations were taken by standing at ground level.

- Internal and external visual observation of warehouse panels and concrete columns. Panels were partly obscured by wall linings in the north-east corner of the warehouse and by racking in the south west corner.
- Visual observation of steel to panel and panel to panel fixings in the warehouse.
- External visual observation of office panels. Office panels were obscured by linings on the inside of the building.
- Visual observation of steel to panel and panel to panel fixings in the office above ceiling level.
- Visual observation of the ground floor slab in the warehouse. The ground floor slab was obscured by carpet in the office.
- Visual observation of warehouse portal frames roof bracing and panel transoms.

4.3. Observed Damage

Table 3: ARCL Site Observations

Element	Observations	Photographs
General Site	<ul style="list-style-type: none"> One sand boil observed located at the office entrance. Sand visible below tiles. 	
Foundations	<ul style="list-style-type: none"> Settlement of southern end wall. Gradient between primary superstructure elements approximately 1:700. 	
Ground floor slab	<ul style="list-style-type: none"> Increased width of ground floor slab control joints or saw cuts. 	 <p style="text-align: center;">Photo 1</p>  <p style="text-align: center;">Photo 2</p>

Concrete wall panels

- Cracking patterns observed to the warehouse and office concrete walls, consistent with face loading damage. Crack widths up to 1mm measured.
- Cracking of panels around fixings in office.





Photo 3



Photo 4



Photo 5

<p>Concrete columns</p>	<ul style="list-style-type: none"> • Cracking to columns less than 0.2mm wide. • Damage to column to panel fixing in one location 	 <p style="text-align: center;">Photo 6</p>
<p>Southern end wall</p>	<ul style="list-style-type: none"> • Southern end wall held plumb by props. Columns rotated outward by approximately 1.2 degrees due to rotation of the column foundations. 	 <p style="text-align: center;">Photo 7</p>

Larger format representative photographs of the damage described above are included in Appendix B.

A ground floor level survey was also conducted to determine if any differential settlement had occurred over the building footprint. Further discussion of this is included below.

4.4. Building Floor Level Survey

A ground floor level survey was carried out on 26 September 2011. The survey results indicate a total variation in level of 80mm over the warehouse floor slab and 40mm over the office floor. The warehouse floor slab slope increases near the southern end wall to a gradient of approximately 1:100 indicating that the end wall has settled differentially to the slab. The level of differential settlement between elements of the superstructure appears to be minor with maximum gradients of approximately 1:700 between primary superstructure elements.

The level of global settlement of the building structure at the site has not been assessed.

4.5. Post-Earthquake Building Seismic Performance

4.5.1. Office

Cracking and spalling around the office panel fixings and flexural cracking of the panels is likely to have reduced the ductility of the panels and their fixings. Panels have been assessed based on fully elastic requirements using a structural ductility (μ_p) of 1.0 and a structural performance factor (S_p) of 1.0. On this basis the panels were assessed as having a lateral capacity of 33-50% NBS.

Due to the observed damage to the end wall columns and panels emergency stabilisation repairs were carried out. The repairs were designed in accordance with section B1 of the New Zealand Building Code to achieve a target lateral strength of approximately 67% NBS. Refer to Appendix A for details of the repair design. These repairs have now been carried out.

4.5.2. Warehouse

The end wall columns were previously the primary lateral load resisting system in the longitudinal direction of the warehouse. Damage to the warehouse slab reinforcing has reduced the strength of the structural tie at the base of the end-wall columns and settlement of the southern end wall has caused the southern end wall columns to rotate outward. This damage has reduced the lateral capacity of the end wall columns system to approximately 33%-50% NBS.

Cracking in warehouse perimeter side wall panels is likely to have reduced their ductility. Panels were assessed based on fully elastic requirements using a structural ductility (μ_p) of 1.0 and a structural performance factor (S_p) of 1.0. On this basis the panels were assessed as having a lateral capacity of 33-50% NBS.

Due to the observed damage to the end wall columns and panels emergency stabilisation repairs were carried out. The repairs were designed in accordance with section B1 of the New Zealand Building Code to achieve a target lateral strength of approximately 67% NBS. Refer to Appendix A for details of the repair design. These repairs have now been carried out.

The observed vertical settlement of the southern end wall is considered not to significantly affect the assessed strength the building.

4.6. Level of Damage

The level of damage observed in the warehouse and office is considered significant with reference to section 5 of the EAG guidance document [1]. Based on detailed calculations the warehouse and office were assessed as having a lateral capacity of 33-50% NBS prior to the installation of strengthening. Strengthening has now been carried out and the warehouse and office has been assessed to have a lateral capacity of approximately 67% NBS.

4.7. Critical Structural Weaknesses (CSW's)

From a review of the structural drawings available, no significant global Critical Structural Weaknesses (CSWs) were identified in the building.

Detail-related CSWs were also checked in accordance with the Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury (Part 2) issued by the Engineering Advisory Group. Perimeter panels are connected to horizontal steel transoms by mechanical anchors. The mechanical anchors used are undercut anchors and have a capacity greater than 67% NBS.

4.8. Geotechnical and Site Considerations

47 test bores were completed on the site in 2000. The test bores were completed using a hand auger to a maximum depth of approximately 4m. The investigations indicate that the site is overlain with approximately 2m of non-engineered fill over natural silts and sands. The water table was not recorded. Refer to Appendix C for copies of this investigation information.

A small sand boil was observed on the site in front of the office following the 22 February 2012 earthquake. The site is not adjacent to any significant waterways or slopes and no evidence of lateral spreading was observed during the observations. This site is in the Green Zone of the CERA Land Damage Hazard Map.

4.9. Neighbouring Buildings

Neighbouring buildings are located on all 4 sides of 4 Mary Muller Drive. No assessment has been undertaken on these buildings. At the time of our observation it was noted that the buildings were in use.

5. CONCLUSIONS AND RECOMMENDATIONS

An initial assessment of the building using the EAG guidance document identified that, in its pre-earthquake condition, the primary lateral load resisting system was able to withstand loads of approximately 60%-70%NBS. No critical structural weaknesses that significantly reduce the buildings capacity were identified in the available building drawings or during visual observations of the building.

Alan Reay Consultants Ltd have undertaken visual observations of the building and have identified settlement and rotation of the southern end wall, cracking to concrete panels, damage to steel panel connections and damage to ground floor slab control joints as a result of the recent seismic activity in the region.

The level of damage observed in the warehouse and office is considered significant with reference to section 5 of the EAG guidance document [1]. Based on detailed calculations the warehouse and office was assessed as having a lateral capacity of 33-50% NBS prior to the installation of strengthening. Strengthening has now been carried out and the warehouse has been assessed to have a lateral capacity of approximately 67% NBS.

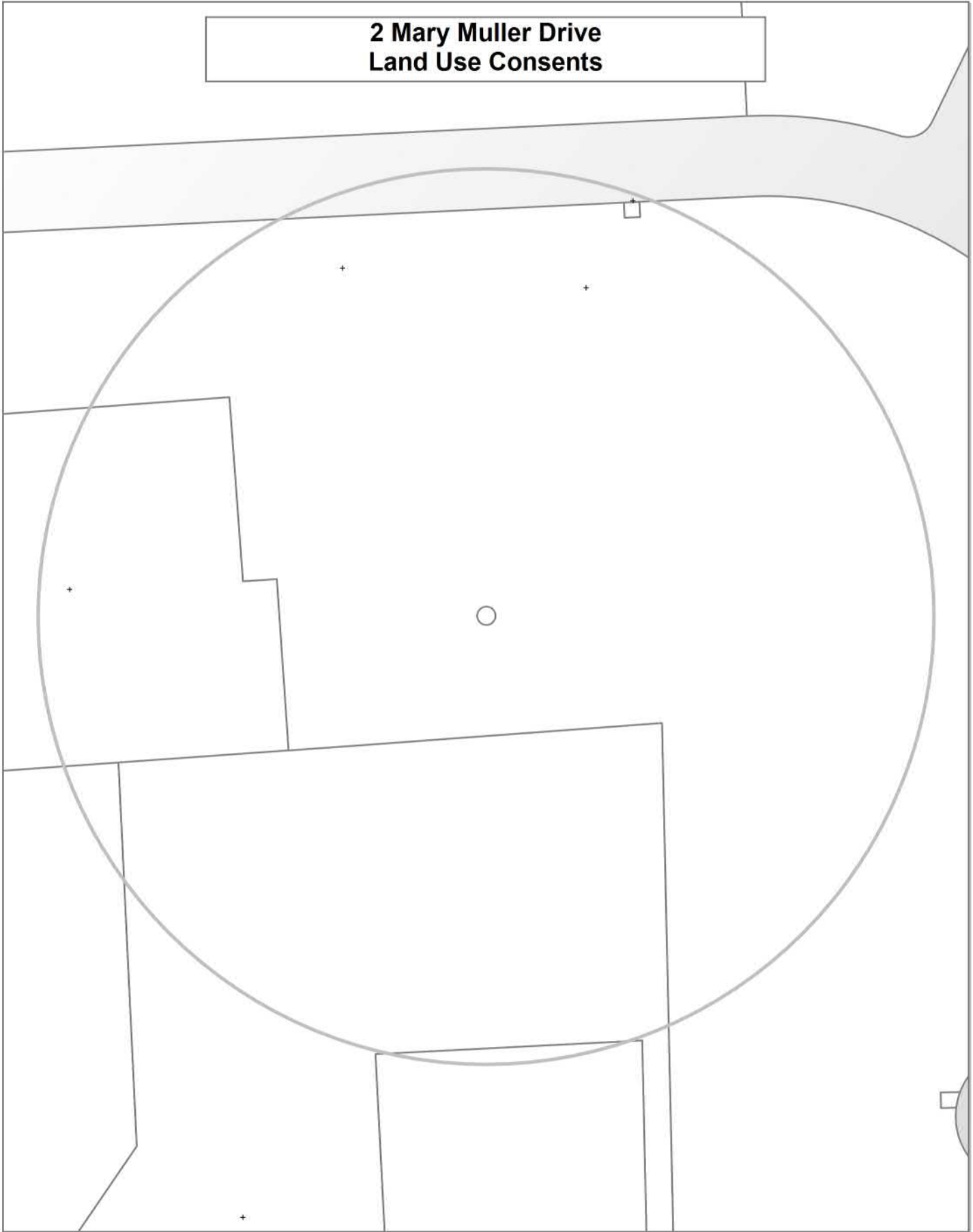
A summary DEE report following the reporting format in the EAG guidance document [1] is included in Appendix F.

This building assessment is limited to that required for a DEE. Further investigation and reporting may be required for assessment of the building for insurance purposes etc.

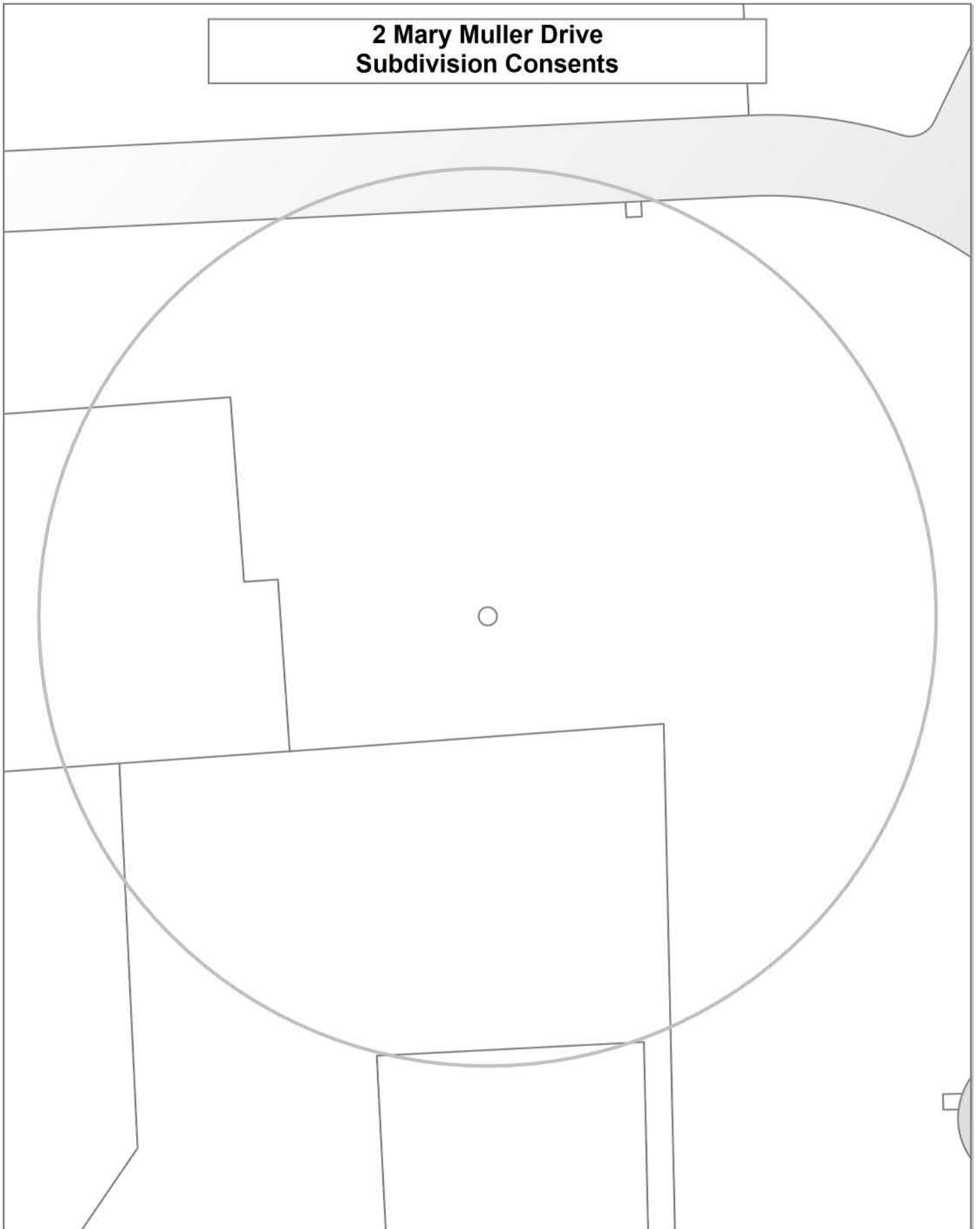
6. REFERENCES

1. Engineering Advisory Group; *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury (Revision 5)*, July 2011
2. New Zealand Society for Earthquake Engineering; *Building Safety Evaluation*, August 2009
3. New Zealand Society for Earthquake Engineering; *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*, June 2006
4. Standards New Zealand; *NZS1170 Structural Design Actions*
5. New Zealand Legislation, *Building Act 2004*, March 2012
6. New Zealand Legislation, *Building Regulations 1992*, April 2012
7. Christchurch City Council, *Earthquake-Prone, Dangerous and Insanitary Buildings Policy 2010*, September 2010
8. Department of Building and Housing; *Advice for Canterbury Building Owners: Assessing the seismic performance of non-residential and multi-unit residential buildings in greater Christchurch*, June 2012
9. Department of Building and Housing; *Guidance for engineers assessing the seismic performance of non-residential and multi-unit residential buildings in greater Christchurch*, June 2012
10. GNS Science; *The Canterbury Earthquake sequence and implications for Seismic Design Levels*, July 2011

**2 Mary Muller Drive
Land Use Consents**



**2 Mary Muller Drive
Subdivision Consents**



Land Use Resource Consents within 100 metres of 2 Mary Muller Drive

Note: This list does not include subdivision Consents and Certificates of Compliance issued under the Resource Management Act.

220 Port Hills Road

RMA/2006/598

25 lockup storage units with landscaping and parking space numbers non compliances - Historical Reference RMA20022602

Processing complete

Applied 22/03/2006

Decision issued 16/05/2006

Granted 16/05/2006

RMA/2008/794

Car parking reduction for additional freezer & drystore buildings - Historical Reference RMA92011601

Processing complete

Applied 16/04/2008

Decision issued 24/12/2008

Granted 24/12/2008

Data Quality Statement

Land Use Consents

All resource consents are shown for sites that have been labelled with an address. For sites that have been labelled with a cross (+) no resource consents have been found. Sites that have no label have not been checked for resource consents. This will be particularly noticeable on the margins of the search radius. If there are such sites and you would like them included in the check, please ask for the LIM spatial query to be rerun accordingly. This will be done free of charge although there may be a short delay. Resource consents which are on land occupied by roads, railways or rivers are not, and currently cannot be displayed, either on the map or in the list. Resource consents that relate to land that has since been subdivided, will be shown in the list, but not on the map. They will be under the address of the land as it was at the time the resource consent was applied for. Resource consents that are listed as Non-notified and are current, may in fact be notified resource consents that have not yet been through the notification process. If in doubt. Please phone (03)941 8999.

The term "resource consents" in this context means land use consents. Subdivision consents and certificates of compliance are excluded.

Subdivision Consents

All subdivision consents are shown for the sites that have been labelled with consent details. For Sites that have been labelled with a cross (+) no records have been found. Sites that have no label have not been checked for subdivision consents. This will be particularly noticeable on the margins of the search radius. If there are such sites and you would like them included in the check, please ask for the LIM spatial query to be rerun accordingly. This will be done free of charge although there may be a short delay.

The term "subdivision consents" in this context means a resource consent application to subdivide land. Non subdivision land use resource consents and certificates of compliance are excluded.

This report will only record those subdivision applications which have not been completed i.e once a subdivision has been given effect to and the new lots/properties have been established the application which created those lots will not be shown

All subdivision consent information is contained on the map and no separate list is supplied