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79 CAMBRIDGE TERRACE
CHRISTCHURCH

SITE INVESTIGATION REPORT

Reference Number: 3984

Date: 29 November 2011

CONTENTS

SITE INVESTIGATION REPORT
79 CAMBRIDGE TERRACE

1.0 INTRODUCTION

The existing 7 storey office building (Bradley Nuttall House) at 79 Cambridge Terrace in Christchurch has undergone some structural damage during the earthquake event of 22 February 2011, and subsequent aftershocks. The building has settled differentially to a small degree (up to approximately 50mm), and some slumping around the area suggests that the site has undergone some liquefaction. Lidar information suggests general liquefaction settlements in this area of 0 to 200mm; and lateral movements up to 250mm.

A subsurface investigation has been carried out to determine the severity of the liquefaction problem at this site.

This report summarises the results of the site investigation and the design implications that can be interpreted from those results.

2.0 THE SITE

The site is level, and is occupied by a single 7 story office building, totalling about 600 square metres in plan area.

The site is bounded to the southeast by Cambridge Terrace – to the southwest is a smaller commercial building; to the northwest is a small carparking area and then a large high-rise building to the north (currently undergoing demolition); to the northeast is a further carparking area and basement access ramp to the commercial mid-rise building to the northeast. The Avon River is some 35m to the southeast of the site.

3.0 THE SITE INVESTIGATION

3.1 Objectives

This site investigation was carried out to provide information about the composition, spatial relationships and geotechnical properties of the materials that underlie the site. In particular the following information was sought:

- Definition of the quality and variability of the soils underlying the site.
- Water table depth.
- Liquefaction potential.
- Permissible foundation types and associated bearing capacities.
- Settlements.
- Site subsoil category.

3.2 Methodology

Two dual tube boreholes have been drilled, at the western and eastern corners of the building. These extended to between 21.4m and 22.8m below ground level. SPT testing was carried out at regular intervals down each borehole.

Appendix 1 contains a site plan showing the locations of the boreholes.

3.3 Subsurface Conditions

The boreholes indicate (at the locations drilled) a surface layer of asphalt and sandy gravel fill to 1.1 m depth. This is underlain with very loose to loose sand to 2.6m depth, where a layer of sandy gravel to gravely sand is located. This layer is medium dense to dense, and extends to 7.2m depth. Below this is a very loose to loose sand layer (with some interbedded gravels and silts) to 12.5 – 13m depth, where the sands become dense to very dense. At 19.1 – 19.3m depth below ground a layer of very soft silt is encountered, extending to about 20.5m (in one hole there was a layer of peaty silt in the lower 300mm of this unit). Below this is a sandy gravel layer (first artesian aquifer), and the borholes terminated in this at 21.4 – 22.8m depth.

A well log has been retrieved (from the Ecan database) of what appears to be a borehole drilled here in 1985 for the construction of the original building – this indicates sand to 1.2m depth and then 'sand and gravels' to 3.5m depth. A 'sandy gravel' then extends to 10.2m depth, overlying silts and interbedded silts and sands to 12.5m depth. A sand then extends to 17.8m depth, silt to 19.2m and then sand and gravels. The Ecan well logs are not as reliable as actual borehole logs (and Ecan rate the reliability of this particular well log as 4 (worst) on a scale of 1-4), however the well log does appear to be slightly at odds with what we have found in the dual tube boreholes.

Borelogs are appended to this report, in Appendix 2.

4.0 INTERPRETATION

4.1 Liquefaction Potential

The saturated sandy materials below the water table do have potential for liquefaction in a large earthquake. For smaller earthquake events this is mainly from 7m – 11m depth, however for larger events there is potentially a small pocket directly under the water table at about 2m depth, and then the materials from 5 – 13m depth are potentially liquefiable (Although this does vary between the two boreholes).

The CPT profiles have been analysed using a relatively recent method (Idriss & Boulanger (2006)), which is an update of the more commonly used NCEER procedure from Youd et al.,(2001). This semi-empirical cyclic stress based method ('CSR' method) is the currently accepted 'industry standard' in New Zealand.

For the design input ground motion accelerations in the past we would normally adopt the NZGS recommended approach of using NZS 1170.5 to calculate ultimate limit state and serviceability limit state accelerations, with the "Z" and "R" factors upgraded as recently recommended (May 2011) by the Department of Building and Housing. However we are now aware that the design ground motions for the purposes of liquefaction analysis will be upgraded in the near future, and some account has been taken of the likely new design values in our analyses.

We calculate current ULS ('Ultimate Limit State') post liquefaction ground settlements at the site in the order of 85 - 270mm, based on a method by Ishihara and Yoshimine (1992), using the CSR analysis method.

We have also calculated liquefaction potential and ground settlements from the smaller Serviceability Limit Sate ('S.L.S.') earthquake – the CSR method indicates ground settlements of about 5–90mm. For the anticipated future design SLS level, we calculate post liquefaction ground settlements in the order of 15–170mm – however the thickness of the overlying 'crust' in this case, and the performance of the site to date suggests that surface manifestation of damage in this case is unlikely.

Design Event	Design Ground Acceleration	Ground Settlement (CSR Method)
25 years (S.L.S.) (current level)	0.11g / M7.5	5 - 90mm
25 years (S.L.S.) (anticipated new level)	0.14g / M7.5	15 - 170mm
500 years (U.L.S.) (current level)	0.34g / M7.5	85 - 270mm
22 February 2011	0.45g / M6.3	75 - 265mm

Table 1 – Calculated Liquefaction Induced Settlements

4.1.1 Lateral Spread

Lateral spread is the post-liquefaction movement of either level liquefied ground towards a free edge (for example a river channel or lake edge) or of sloping liquefied ground downhill.

There are approximate theoretical methods available for the prediction of lateral spread – however these are based on databases of past measured movements in a very small number of overseas events. Lateral spread movements in the order of 50 – 100mm are indicated in a SLS event. In the order of 1 – 3.5m is indicated at ULS. It appears that this has not manifested in the recent earthquake events (in the order of 250mm is an estimated measured value in this area) and therefore these figures are likely to be over-estimates, however this is not a guarantee that some movement won't occur in future events.

A better estimate of likely movements would come from a slope stability analysis (Newmark Sliding Block analysis) following a survey of the land and river cross section, and further drilling nearer to the river.

4.2 Capacity of Existing Building Foundation Elements

4.2.1 Bulb Piles

The building drawings show that the existing shear walls are supported on reinforced concrete bulb piles, which presumably act as tension piles. According to the design drawings these are founded at about 9.5m below ground level, unfortunately in what we now know to be the main liquefiable sand layer. The design diameter of the expanded bulbs is 1.25m, with a 0.5m diameter shaft. We have carried out calculations of the uplift capacity of the piles in a liquefaction event, based on estimated post liquefied strengths in the sand layer – we estimate a geotechnical ultimate capacity in the order of **120kN**. (This should have a capacity reduction factor of 0.5 applied to it and compared with fully factored loads.)

Information to hand indicates that general lateral movements of the ground in the order of 250mm has taken place during the earthquake series – the ductility of the existing piles will need to be checked to determine whether this is sufficient to cope with this movement without damage.

4.2.2 Shallow Foundations

A bearing capacity analysis has been carried out based on the limited number of SPT N values in the upper soil layers. This has yielded very low capacities, which are probably unrealistic and reflect the coarse nature of the SPT test.

The design drawings indicate that the foundation beams are located some 1.8m below ground level and footing dimensions are in the order of 2-3m wide – at this depth and for these footing dimensions we estimate Ultimate Bearing capacities of **390 kPa** for strip footings and **590 kPa** for pad footings. (These values should have a capacity reduction factor of 0.5 applied to it and compared with fully factored loads.) (For shallower depths and/or smaller dimensioned footings, bearing capacities are very much reduced.)

To limit settlements under static loads to 25mm we estimate allowable bearing pressures under working loads to be 350 kPa for strip footings and 550 kPa for pad footings, at 1.8m depth and for footing dimensions in the order of 2-3m wide. (Again, for shallower depths and/or smaller dimensioned footings, bearing capacities are very much reduced.)

The calculation of bearing pressures is influenced considerably by the low SPT value obtained in BH2 at 1.1m depth – more accurate calculations could be carried out if continuous CPT testing was done at the site.

4.3 Remedial Measures

4.3.1 Piles

If the building cannot cope with the calculated settlements outlined in section 4.1 of the report, or new tension piles are required to replace or augment the existing bulb piles, then one possible solution is to support the building on piles taken down to the dense sand layers below 12m depth. We recommend a good embedment into these materials – when the overlying loose sands liquefy, the tension capacity of the piles will be reduced - the greater the depth of embedment, the less will be this effect. Additionally, the dense founding sands will undergo some softening, even in the absence of liquefaction – again this effect should reduce with depth.

For ease of retro-installation, and also to provide some ductility in the event of lateral movements, we recommend the use of steel screw piles.

Piletech provide their own in-house design service, however we estimate the bearing capacity of a 600mm diameter screw pile founded at a nominal 15m depth to be in the order of **1700 kN.** The estimated tension capacity is **850 kN.** These are ultimate (geotechnical) pile capacities - and should have a reduction factor of 0.5 applied to them and then compared with fully factored Ultimate Limit State (ULS) loads.

Installation of screw piles around the building perimeter should be achievable, and for the interior Piletech have advised that they can operate piling rigs inside buildings with headroom in the order of 3m; they are also currently developing a rig that requires less headroom than this.

4.3.2 Ground Improvement

Another possible option is ground improvement. The liquefaction potential could be reduced by improving the ground down to the denser materials at 12m depth. This could be achieved by installing a grid of compaction grout columns under and around the building, using Low Mobility Grout. The presence of the large footings might impede this operation however.

Another option could be to improve the tension capacity of the existing bulb piles by the use of permeation grouting, targeting the areas that are in the immediate proximity of the piles in order to reduce the liquefaction there.

Any ground improvement will require extensive consultation with a specialist contractor however, to determine the viability of the option.

4.3.3 Foundation Integration

We note that many of the foundation elements are sitting in isolation from one another, with the exception of those associated with the shear walls, where substantial ground beams exist. In order to reduce the risk of lateral drift of isolated foundation elements, which could lead to the undermining and collapse of supported columns, we strongly recommend that all foundation elements are tied together with ground beams in two orthogonal directions.

4.4 Seismic Category

The consistency and depth of the alluvial formations underlying this site makes it a ‘Class D’ site in terms of the seismic design requirements of NZS1170.5:2004.

4.5 Further Work

The two boreholes drilled on site show different ground conditions than are indicated by the original 1985 well log, and results from other sites nearby indicate that ground conditions can be highly variable in this locale. We would recommend that if the project is to be taken much further, at least two additional deep boreholes are carried out, at the other two diagonal corners of the building (access should become available once the adjacent demolition is complete). We would also advise laboratory testing of some of the retrieved soil samples to confirm liquefaction susceptibility.

Some shallow CPT probes (which will refuse on the gravels at 2-3m depth) would also be advisable, if the bearing capacities of shallow foundations are critical.

To improve the estimates of lateral spread potential, a further borehole near the river to the south would be needed, along with more sophisticated slope stability analyses and a survey of the ground surface profile.

5.0 SUMMARY & CONCLUSIONS

The ground conditions at the site consist of a surface layer of asphalt and sandy gravel fill to 1.1 m depth. This is underlain with very loose to loose sand to 2.6m depth, where a layer of sandy gravel to gravely sand is located. This layer is medium dense to dense, and extends to 7.2m depth. Below this is a very loose to loose sand layer (with some interbedded gravels and silts) to 12.5 – 13m depth, where the sands become dense to very dense. At 19.1 – 19.3m depth below ground a layer of very soft silt is encountered, extending to about 20.5m. Below this is a sandy gravel layer (first artesian aquifer).

Liquefaction analyses indicate that in a S.L.S. seismic event ground settlements of up to 170mm are possible (but surface effects in this case will likely be suppressed), and 270mm in a ULS.

The existing bulb piles under the building shear walls are located in a liquefiable layer and therefore will have very low capacities in a seismic event. The existing surface foundations (which are unusually deep) have reasonable static bearing capacity.

Steel screw piles could be retrofitted to reduce post-liquefaction settlements and provide tensile capacity for shear walls; alternatively it may be possible to undertake ground improvement, in close consultation with a specialist contractor.

Further investigation work is recommended and is outlined in the report.

Yours faithfully,

Geotech Consulting Ltd per:

Nick Traylen

6.0 LIMITATIONS

This report has been prepared solely for the benefit of, and under specific instruction from Buchanan & Fletcher Ltd as our client with respect to the brief, for use for this specific project. The reliance by other parties on the information or opinions contained in the report shall be at such parties' sole risk.

Recommendations and opinions (not to be construed as guarantees) in this report are based on data from boreholes and probings. The borelogs are an engineering interpretation of the subsurface conditions. The nature and continuity of subsoil conditions away from the test locations are inferred and it must be appreciated that actual conditions could vary from the assumed model. Any construction costs estimates contained in this report are indications only – a qualified quantity surveyor should be engaged if more accurate costs are required.

During excavation and construction, the site should be examined by an Engineer or Engineering Geologist competent to judge whether the exposed subsoils are compatible with the inferred conditions on which the report has been based. It is possible that the nature of the exposed subsoils may require further investigation, and the modification of any design work that may have been based on this report.

Geotech Consulting Ltd would be pleased to provide this service and believe that the project would benefit from such continuity. In any event it is important that Geotech Consulting Ltd is contacted if there is any variation in subsoil conditions from those described in this report as it may affect opinions expressed and any design parameters recommended in this report.

Regulatory and insurance issues may arise from some of the recommendations in this report; the client should seek independent advice on these aspects. This opinion is not intended to be advice that is covered by the Financial Advisers Act 2010.

7.0 REFERENCES

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Appendix 1

Site Plan

GEOTECH consulting ltd	PROJECT: 79 Cambridge Terrace	DRAWING: Site Investigation Plan	ISSUE / DATE / AMMENDMENT DETAILS / CHKD

SCALES:	DESIGNED	NJT	09/11
1:400 @ A4 approx	DRAWN	NJT	09/11
	CHECKED		

PROJECT No.	SHEET No.	ISSUE
3984	SK1	A

Appendix 2

Borelogs

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DRILLHOLE BORELOG

Hole ID:	BH1
Sheet:	1 of 3
Date:	27/10/2011

Project No:	3984	Equipment:	DT 45	G.L. R.L.:		Logged by:	PEW
Project:	79 Cambridge Tce.	Drilling Co:	McMillan	Max Depth:	21.4m	Checked by:	NJT
Client:	Buchanan & Fletcher	Operator:	D. Keown	Location:	Refer site plan		

Geological Formation	STRATA DESCRIPTION SOIL DESCRIPTION Major colour, second colour, Subordinate fraction, minor fractions, - plasticity, bedding, moisture, structures ROCK DESCRIPTION Colour, fabric, rock name	Graphic Log	Depth	Piezo-meter and Water Levels	COMMENTS	Drill method	Samples	Tests	SPT blows/mm (20 40 60 80)
	Asphalt with **Sandy GRAVEL** (**FILL**).		0.0		**-0.0 - 1.2m,** 33% sample recovery.	Dual Tube Ø70mm			
	SAND with minor **Silt**; yellow brown. Sand, fine; loose.		1.0		**-1.2 - 2.7m**, 20% sample recovery.			1.2 SPT	7, 3, 5 8/300mm
	Silt Sandy GRAVEL; grey brown. Gravel, fine to coarse, subrounded, dense.		2.0, 3.0		**-2.7 - 4.2m**, 26% sample recovery.			2.7 SPT	14, 16, 19 35/300mm
	Sandy GRAVEL; dark grey. Sand, coarse; Gravel fine to coarse, sub-rounded; minor **Silt**; medium dense to dense. -5.71m, less Sand.		4.0, 5.0, 6.0, 7.0		**-4.2 - 5.7m**, 33% sample recovery. **-5.7 - 7.2m**, 39% sample recovery.	Dual Tube		4.2 SPT 5.7 SPT	7, 9, 9 18/300mm 18, 17, 14 31/300mm
	SAND; dark grey. Sand, fine to medium, medium dense. -7.63m, coarser with depth.				**-7.2 - 8.8m**, 79% sample recovery.			7.2 SPT	5, 6, 11 17/300mm
	Sandy GRAVEL; dark grey. Sand, coarse; gravel, fine to coarse, sub-rounded.		8.0						
	SAND; dark grey, coarse.					Dual Tube			
	No Sample.							8.8 SPT	5, 3, 8 11/300mm
	SAND with minor **Silt**; dark grey. Sand, fine; medium dense. . -9.15m, wood fragment. -9.45m, coarser Sand below.		9.0, 10.0		**-8.8 - 10.3m**, 76% sample recovery.				

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DRILLHOLE BORELOG

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Hole ID:	BH1
Sheet:	2 of 3
Date:	27/10/2011

Project No:		Equipment:		G.L. R.L.:		Logged by:	
Project:		Drilling Co:		Max Depth:	21.4m	Checked by:	NJT
Client:		Operator:		Location:			

Geological Formation	STRATA DESCRIPTION SOIL DESCRIPTION Major colour, second colour, Subordinate fraction, minor fractions, - plasticity, bedding, moisture, structures ROCK DESCRIPTION Colour, fabric, rock name	Graphic Log	Depth	Piezometer and Water Levels	COMMENTS	Drill method	Samples	Tests	SPT blows/mm (20 40 60 80)
	No Sample.		10.0						
	SAND with trace of **Silt**; dark grey. Sand, coarse; medium dense.				**-10.3 - 11.8m**, 86% sample recovery.	Dual Tube Ø70mm		10.3 SPT	8, 9, 10; 19/300mm
			11.0						
	-11.79m, Finer Sand below.							11.8 SPT	4, 4, 2; 6/300mm
			12.0						
	SILT interbedded with **Sand** layers; dark grey. Medium to high plasticity; soft.				**-11.8 - 13.3m**, 89% sample recovery.				
	Silty SAND; dark grey. Sand, fine grained; dense.								
			13.0					13.3 SPT	22, 19, 25; 44/300mm
					-13.3 - 14.8m, 99% sample recovery.				
			14.0						
	-14.53m, medium Sand, Silt decreasing with depth.					Dual Tube			
	-14.83m, fine Sand.							14.8 SPT	16, 21, 29; 50/300mm
			15.0		**-14.8 - 16.4m**, 92% sample recovery.				
	-15.85m, coarse Sand.								
			16.0					16.4 SPT	17, 21, 22; 43/300mm
	-16.55m, shell fragment. -16.65m, fine Sand below.								
	-16.95m, coarse Sand below.				**-16.4 - 17.9m**, 100% sample recovery.				
			17.0						
								17.9 SPT	11, 12, 19; 31/300mm
			18.0		**-17.9 - 19.4m**, 100% sample recovery.	Dual Tube			
	-18.92m, Silty SAND.								
			19.0						
	SILT; dark grey. Very low plasticity; soft.				**-19.4 - 20.9m**, 100% sample recovery.			19.4 SPT	3, 2, 2, 3; 5/300mm
			20.0						

DRILLHOLE BORELOG

GEOTECH

Hole ID:	BH1
Sheet:	3 of 3
Date:	27/10/2011

Project No:		Equipment:		G.L. R.L.:		Logged by:	PEW
Project:		Drilling Co:		Max Depth:	21.4m	Checked by:	NJT
Client:		Operator:		Location:	Refer site plan		

Geological Formation	STRATA DESCRIPTION SOIL DESCRIPTION Major colour, second colour, Subordinate fraction, minor fractions, - plasticity, bedding, moisture, structures ROCK DESCRIPTION Colour, fabric, rock name	Graphic Log	Depth	Piezometer and Water Levels	COMMENTS	Drill method	Samples	Tests	SPT blows/mm (20 40 60 80)
	-20.11m, high plasticity. -20.31m - 20.41m, wood fragment.		20.0			Dual Tube Ø70mm			
	Silty Sandy GRAVEL; dark grey. Gravel, fine to coarse, sub-rounded; very dense.							20.9 SPT	
			21.0		**-20.9 - 21.4m**, 100% sample recovery.				25 35 17 52/300mm
	-21.4m, E.O.H								
			22.0						
			23.0						
			24.0			Dual Tube			
			25.0						
			26.0						
			27.0						
			28.0			Dual Tube			
			29.0						
			30.0						

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DRILLHOLE BORELOG

Hole ID:	BH2		
Sheet:	1 of 3		
Date:	25/10/2011		

Project No:	3984	Equipment:	DT45	G.L. R.L.:		Logged by:	PEW
Project:	79 Cambridge Tce.	Drilling Co:	McMillan	Max Depth:	22.8m	Checked by:	NJT
Client:	Buchanan & Fletcher	Operator:	Keown	Location:	Refer site plan		

Geological Formation	STRATA DESCRIPTION SOIL DESCRIPTION Major colour, second colour, Subordinate fraction, minor fractions, - plasticity, bedding, moisture, structures ROCK DESCRIPTION Colour, fabric, rock name	Depth	Piezo-meter and Water Levels	COMMENTS	Drill method	Samples	Tests	SPT blows/mm
	Asphalt with **Silty Sandy Gravel** (**FILL**); yellow brown. Gravel, fine to coarse; trace of cobbles, less than 70mm diameter, sub-angular to sub-rounded.	0.0		**-0.0 - 1.1m**, 20% sample recovery.	Dual Tube Ø70mm			
	SAND with some **Silt**; yellow brown. Fine Sand, very loose.	1.0		**-1.1 - 2.6m**, 26% sample recovery.			1.1 SPT	2, 1, 1 — 2/300mm
		2.0	▽					
	Gravelly SAND; grey brown. Sand, coarse; gravel, fine to coarse, sub-rounded; trace of **Silt**; medium dense	3.0		**-2.6 - 4.1m**, 15% sample recovery.			2.7 SPT	10, 9, 15 — 24/300mm
	-4.14m, Sandy GRAVEL.	4.0		**-4.1 - 5.7m**, 20% sample recovery.	Dual Tube		4.1 SPT	14, 17, 8 — 25/300mm
	-5.66m, less Sand.	5.0					5.7 SPT	10, 7, 7 — 14/300mm
		6.0		**-5.7 - 7.2m**, 7% sample recovery.				
	SAND with minor **Silt**; dark grey. Sand, medium to coarse, loose.	7.0		**-7.2 - 8.7m**, 86% sample recovery.			7.2 SPT	4, 2, 3 — 5/300mm
	Gravelly SAND; dark grey. Sand, coarse; gravel fine to coarse, sub-rounded.							
	SAND with minor **Silt**; dark grey. Sand, medium to coarse, loose.	8.0			Dual Tube			
	No Sample.						8.7 SPT	4, 3, 5 — 8/300mm
	SAND with minor **Silt**; Sand, coarse. -timber fragment at 9.1m. -9.6m, timber fragment at 9.6m.	9.0		**-8.7 - 10.2m**, 69% sample recovery.				
		10.0						

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DRILLHOLE BORELOG

Hole ID:	BH2
Sheet:	2 of 3
Date:	25/10/2011

Project No:	3984	Equipment:	9700-D (DT)	G.L. R.L.:		Logged by:	PEW
Project:	79 Cambridge Tce.	Drilling Co:	McMillan	Max Depth:	22.8m	Checked by:	NJT
Client:	Buchanan & Fletcher	Operator:	Keown	Location:	Refer site plan		

STRATA DESCRIPTION
SOIL DESCRIPTION Major colour, second colour, Subordinate fraction, minor fractions, - plasticity, bedding, moisture, structures
ROCK DESCRIPTION Colour, fabric, rock name

Depth (m)	Strata Description	Comments	Drill method	Tests	SPT blows/mm
10.0	-10.22m, trace of Silt. -10.54m - 10.84m, some PEAT.	**-10.2 - 11.7m**, 99% sample recovery.	Dual Tube Ø70mm	10.2 SPT	5, 1, 2 — 3/300mm
11.0	-11.14m, finer Sand. -11.74m, Silty SAND.			11.7 SPT	4, 4, 3 — 7/300mm
12.0	-12.46m, Sandy SILT. -12.76m - 12.86m, SILT interbedded with Sand layers, medium to high plasticity. -12.86m, Sandy SILT.	**-11.7 - 13.3m**, 99% sample recovery.			
13.0	-13.26m, very fine Sand with minor Silt.	**-13.3 - 14.8m**, 99% sample recovery.		13.3 SPT	7, 11, 21 — 32/300mm
14.0			Dual Tube	14.8 SPT	13, 19, 26 — 45/300mm
15.0	-15.8m, coarser Sand.	**-14.8 - 16.3m**, 92% sample recovery.			
16.0		**-16.3 - 17.8m**, 86% sample recovery.		16.3 SPT	13, 19, 25 — 34/300mm
17.0				17.8 SPT	10, 17, 24 — 31/300mm
18.0		**-17.8 - 19.3m**, 15% sample recovery.	Dual Tube		
19.0	**SILT**; dark grey. Non plastic. -19.24m, medium to high plasticity. Very soft. -19.66m, low to medium plasticity. -19.96m, becoming grey brown with depth.	**-19.3 - 20.9m**, 15% sample recovery.		19.3 SPT	0, 0, 3 — 3/300mm
20.0					

Column headings: Geological Formation; Graphic Log; Depth; Piezometer and Water Levels; Comments; Drill method; Samples; Tests; SPT blows/mm (20, 40, 60, 80)

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DRILLHOLE BORELOG

Hole ID:	BH2
Sheet:	3 of 3
Date:	25/10/2011

Project No:	3984	Equipment:	9700-D (DT)	G.L. R.L.:		Logged by:	PEW
Project:	79 Cambridge Tce.	Drilling Co:	McMillan	Max Depth:	22.8m	Checked by:	NJT
Client:	Buchanan & Fletcher	Operator:	Keown	Location:	Refer site plan		

STRATA DESCRIPTION

SOIL DESCRIPTION Major colour, second colour, Subordinate fraction, minor fractions, - plasticity, bedding, moisture, structures
ROCK DESCRIPTION Colour, fabric, rock name

Geological Formation	Strata Description	Depth	Comments	Drill method	Tests	SPT blows/mm
Riccarton Gravel	**Peaty SILT**; grey brown. Medium to high plasticity.	20.0		Dual Tube Ø70mm		
	Silty Sandy GRAVEL; grey brown. Gravel, fine to coarse, subrounded, dense to loose. -20.56m, becoming yellow brown with depth, minor Silt. -20.86m, trace of Silt.	21.0	**-20.9 - 22.4m**, 80% sample recovery.		20.9 SPT	15, 22, 24 46/300mm
		22.0			22.4 SPT	2, 2, 2 4/300mm
	-22.8m, E.O.H	23.0				
		24.0		Dual Tube		
		25.0				
		26.0				
		27.0				
		28.0		Dual Tube		
		29.0				
		30.0				

Borelog for well M35/5615

Gridref: M35:8024-4144 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 5.1 +MSD
Driller : Ministry of Works
Drill Method : Cable Tool
Drill Depth : -20.2m Drill Date : 15/05/1985

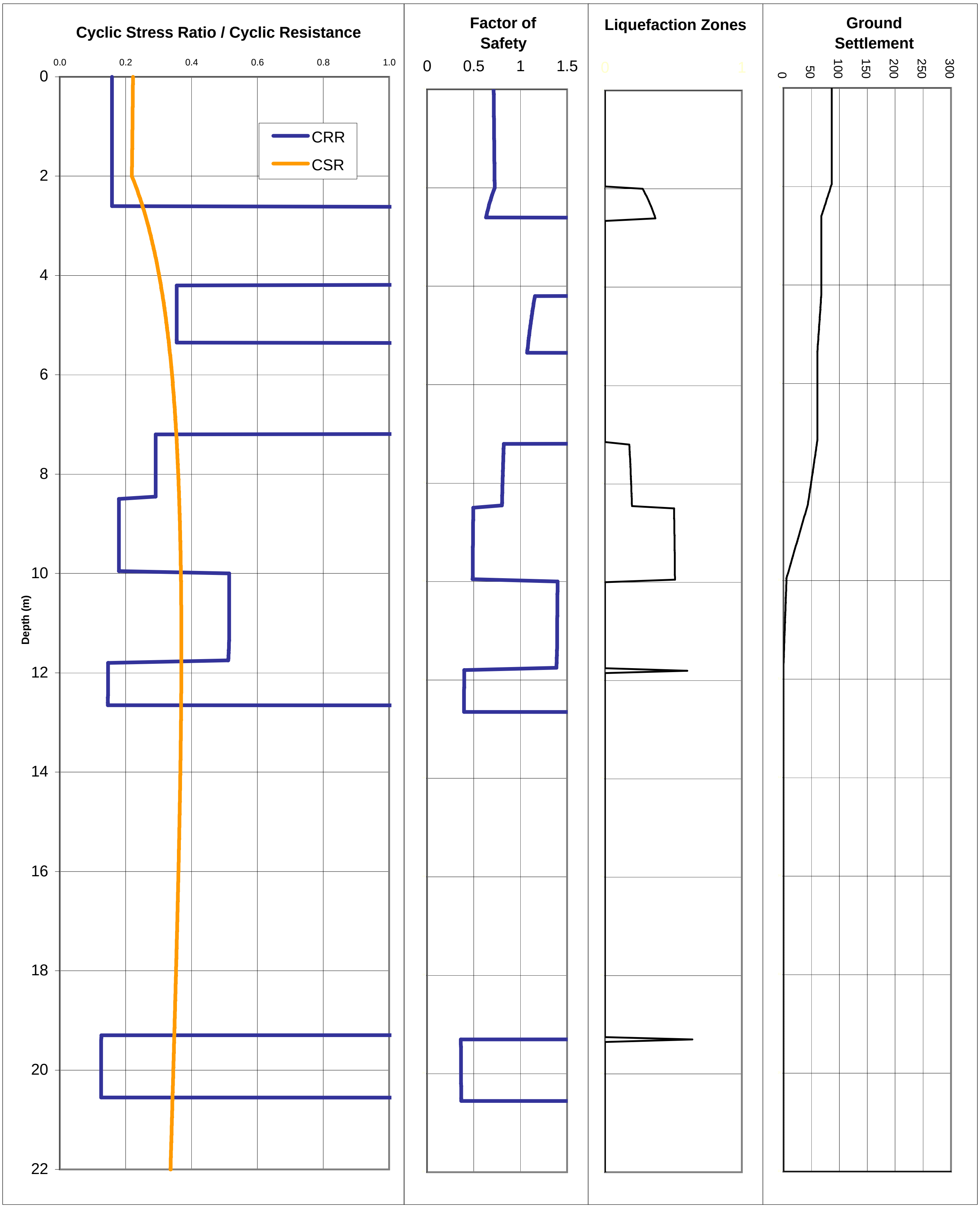
Environment Canterbury Regional Council

Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
	-0.3CalcMin	-0.50m	Grey gravel & organic silty sand	sp?
		-1.20m	Grey/Brown medium sand	sp?
		-3.50m	Grey/Brown medium sand & gravel	sp?
-5				
-10		- 10.2m	Grey/Brown sandy gravel sandier with depth rare wood at 8.5m	sp?
		- 11.0m	Grey homogeneous clayey silt	sp?
		- 12.5m	Blue/Grey interbedded clayey silt & sand	ch
-15				
		- 17.8m	Blue/Grey medium to coarse sand	ch
		- 19.2m	Grey homogeneous clayey silt & rare organics	ch
-20		- 20.2m	Blue/Grey silty sand & gravel with rare organics	ri

Appendix 3

Liquefaction Profiles

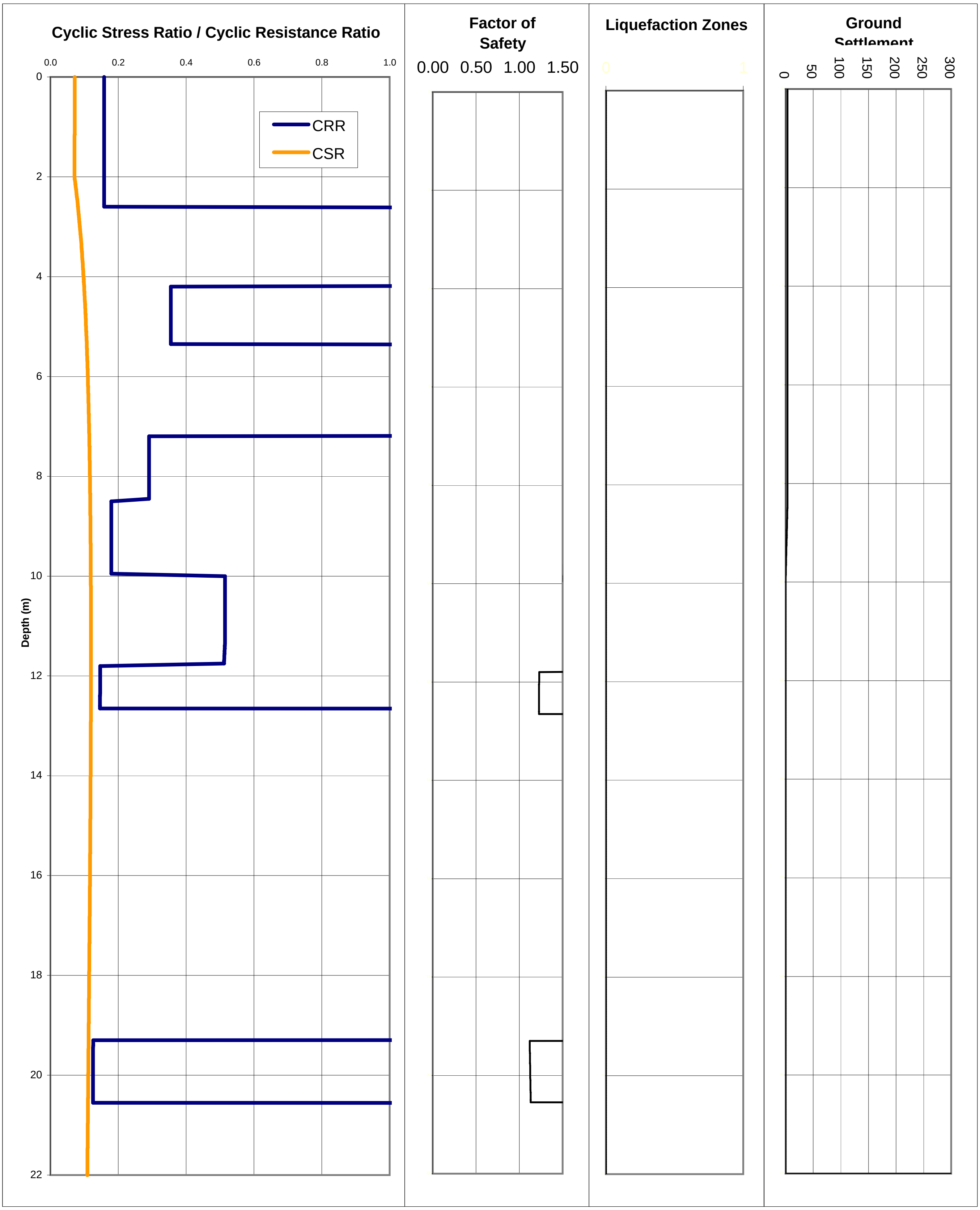
GEOTECH	Liquefaction Potential Analysis			
	GEOTECH CONSULTING LTD			
	Project:	79 Cambridge Terrace	Hole No:	**BH1**
	Client:	Buchanan & Fletcher Ltd	Job No:	3984



Note:	Based on Idriss & Boulanger (2006) and Ishihara & Yoshimine (1992)	**ULS**
	a(g) = 0.34	M = 7.5

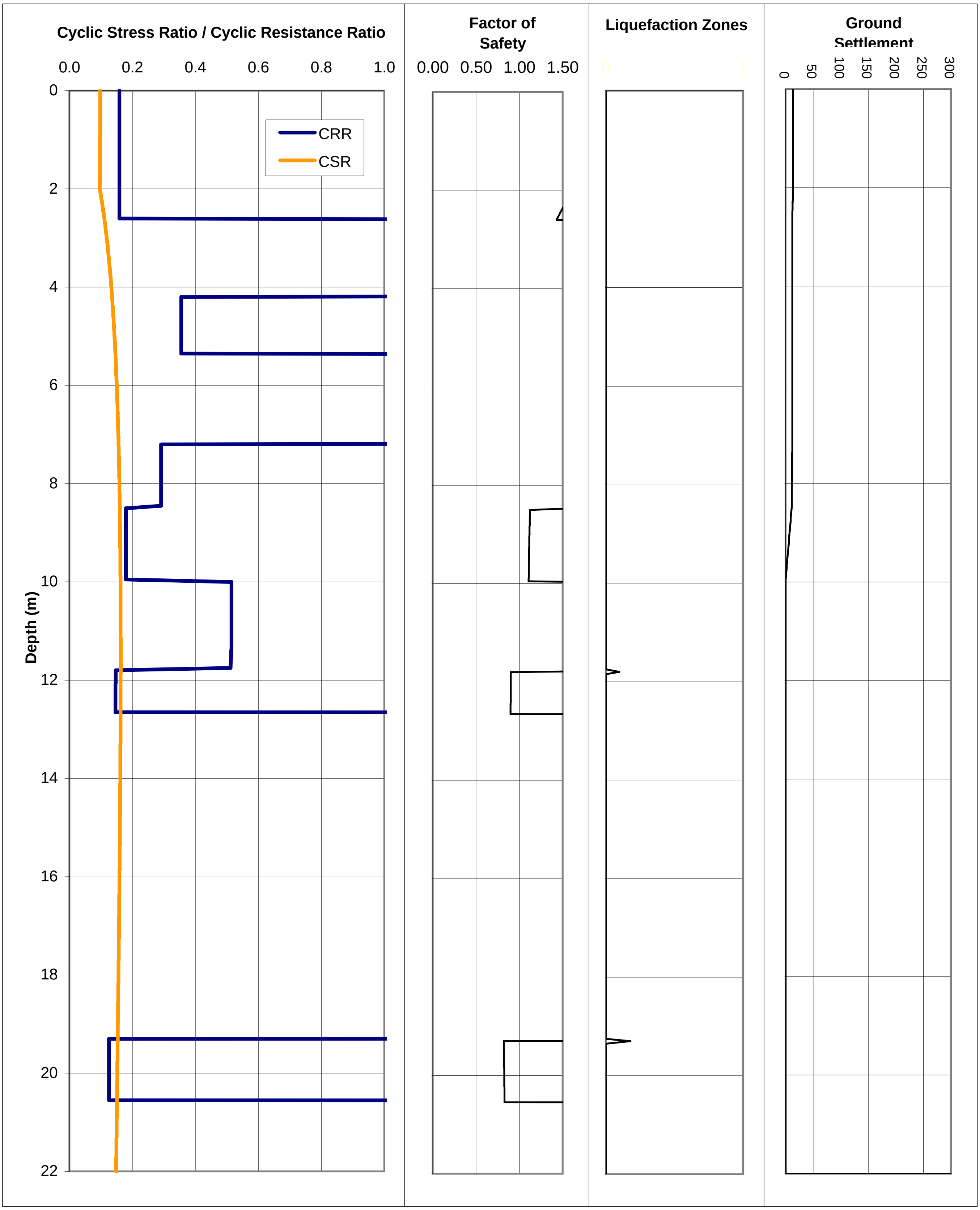
Liquefaction Potential Analysis

GEOTECH	GEOTECH CONSULTING LTD			
Project:	79 Cambridge Terrace		Hole No:	**BH1**
Client:	Buchanan & Fletcher Ltd		Job No:	3984



Note:	Based on Idriss & Boulanger (2006) and Ishihara & Yoshimine (1992)	**SLS**
	a(g) = 0.11	M = 7.5

GEOTECH	Liquefaction Potential Analysis			
	GEOTECH CONSULTING LTD			
	Project:	79 Cambridge Terrace	Hole No:	**BH1**
	Client:	Buchanan & Fletcher Ltd	Job No:	3984



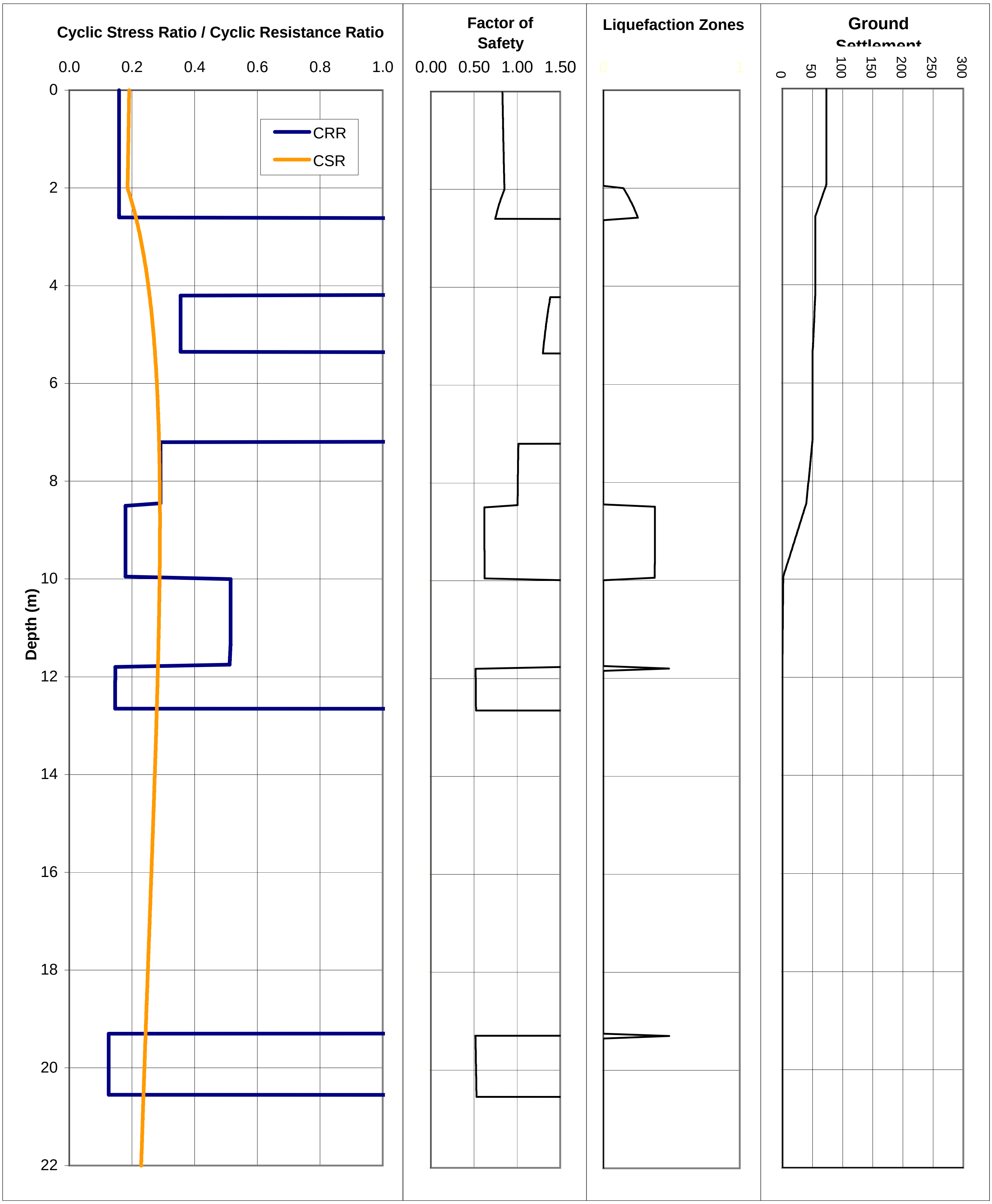
Note: Based on Idriss & Boulanger (2006) and Ishihara & Yoshimine (1992) **SLS @ 0.15g**

a(g) = 0.15 M = 7.5

Liquefaction Potential Analysis

GEOTECH CONSULTING LTD

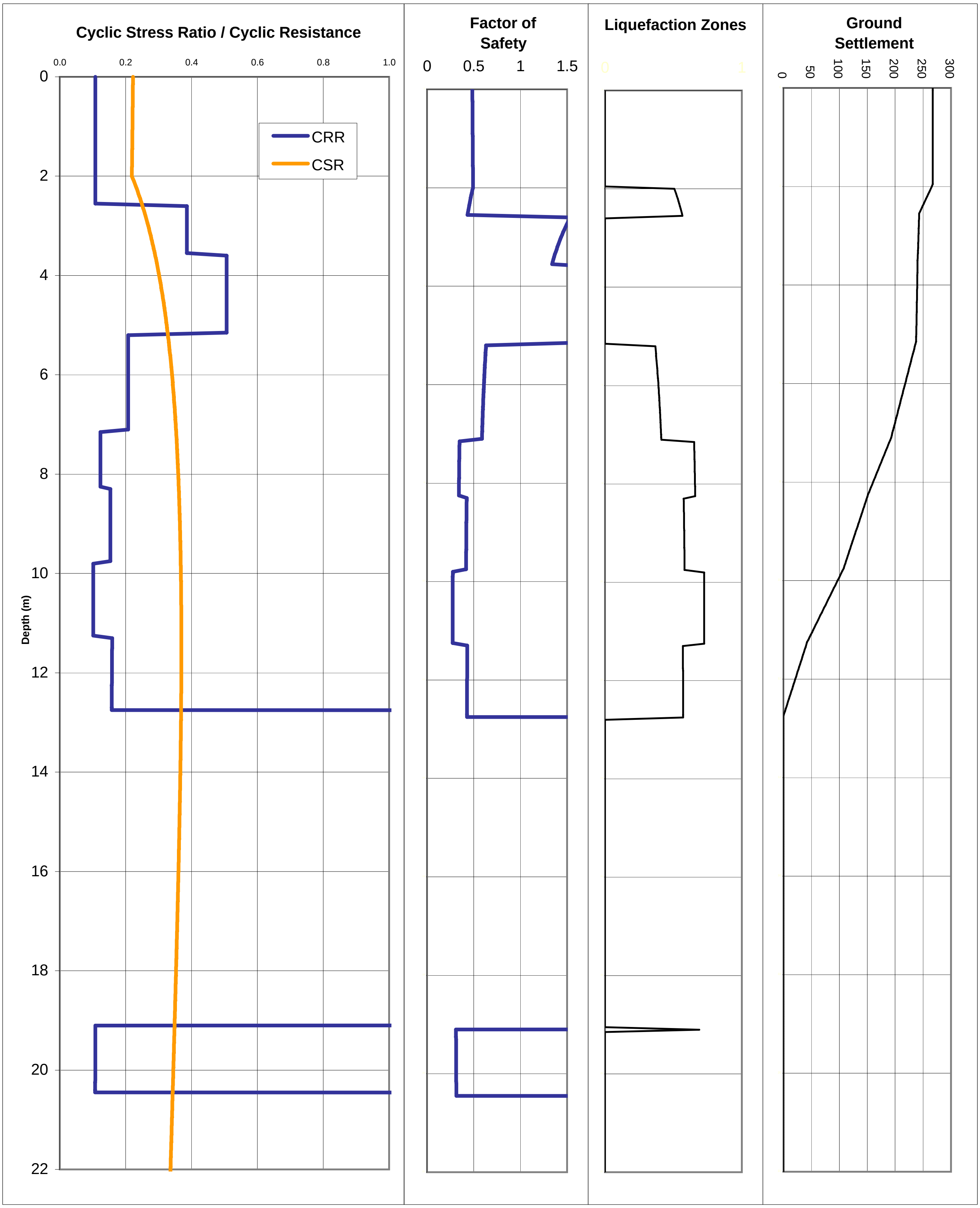
Project:	79 Cambridge Terrace	Hole No:	**BH1**
Client:	Buchanan & Fletcher Ltd	Job No:	3984



Note: Based on Idriss & Boulanger (2006) and Ishihara & Yoshimine (1992)		**22-Feb-11**
	a(g) = 0.4	M = 6.3

BUI.CAM79.0008.30

GEOTECH	Liquefaction Potential Analysis			
	GEOTECH CONSULTING LTD			
	Project:	79 Cambridge Terrace	Hole No:	BH2
	Client:	Buchanan & Fletcher Ltd	Job No:	3984

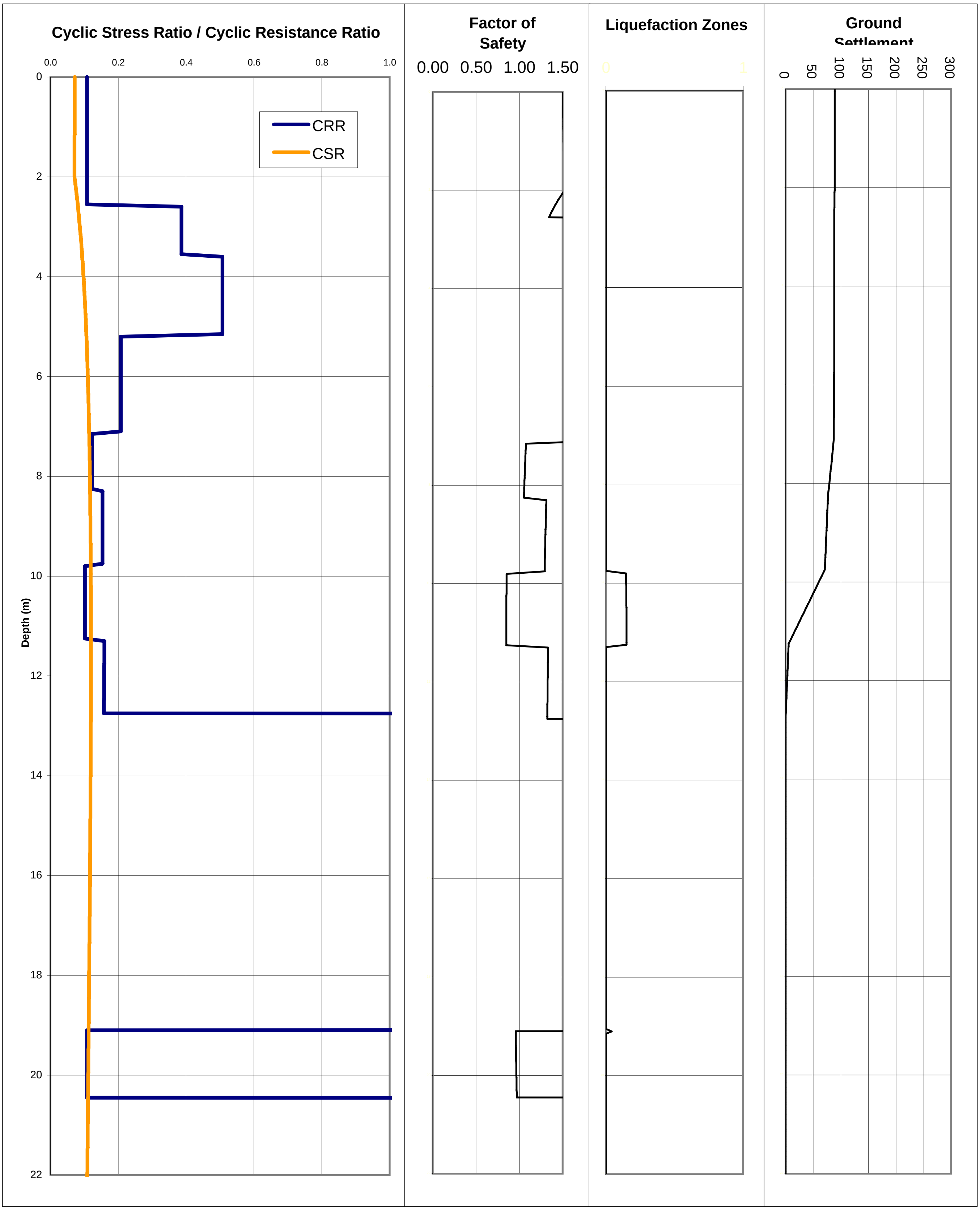


Note:	Based on Idriss & Boulanger (2006) and Ishihara & Yoshimine (1992)	ULS
	a(g) = 0.34	M = 7.5

Liquefaction Potential Analysis

GEOTECH CONSULTING LTD

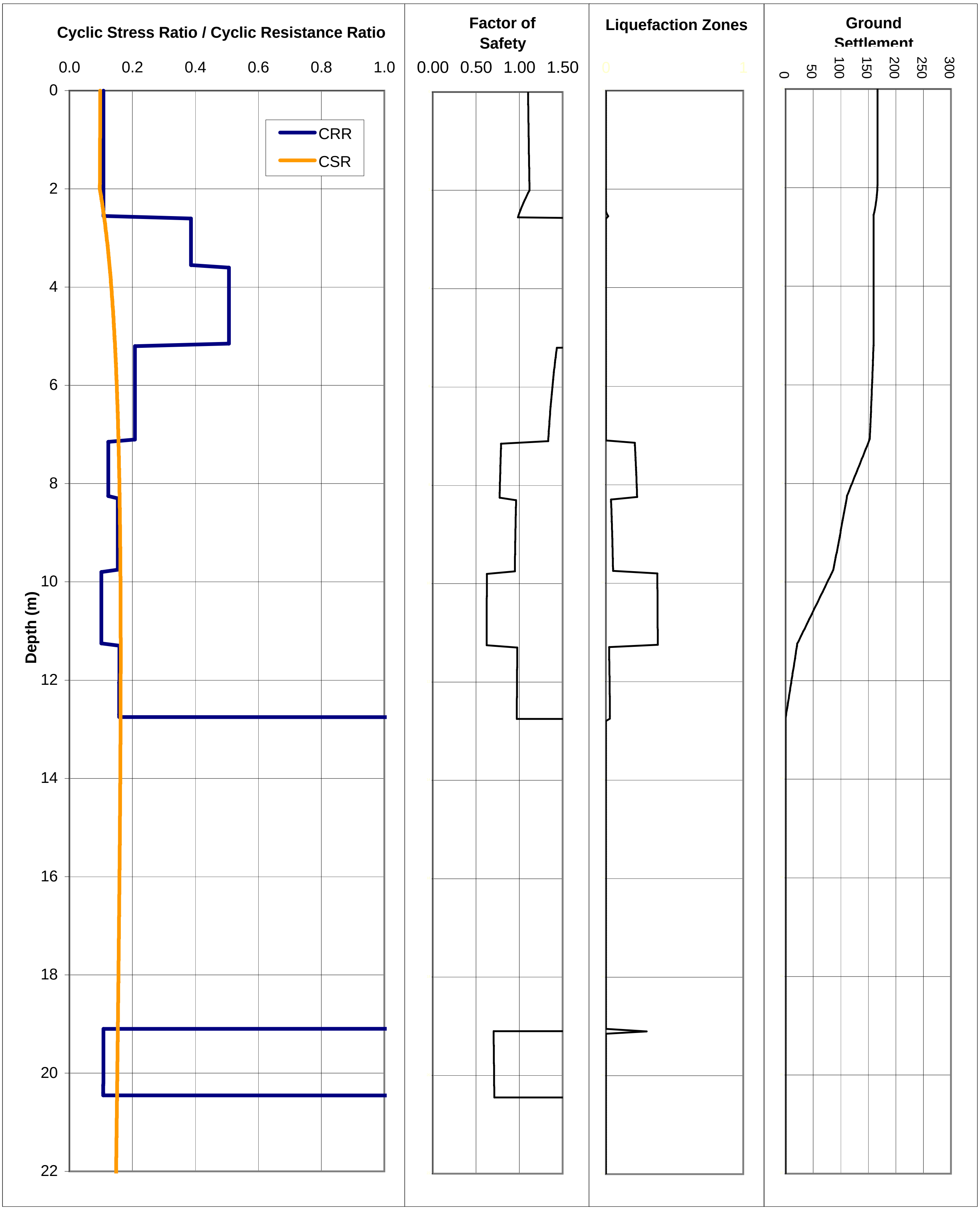
Project:	79 Cambridge Terrace	Hole No:	**BH2**
Client:	Buchanan & Fletcher Ltd	Job No:	3984



Note: Based on Idriss & Boulanger (2006) and Ishihara & Yoshimine (1992) **SLS**

a(g) = 0.11 M = 7.5

GEOTECH	Liquefaction Potential Analysis			
	GEOTECH CONSULTING LTD			
	Project:	79 Cambridge Terrace	Hole No:	**BH2**
	Client:	Buchanan & Fletcher Ltd	Job No:	3984

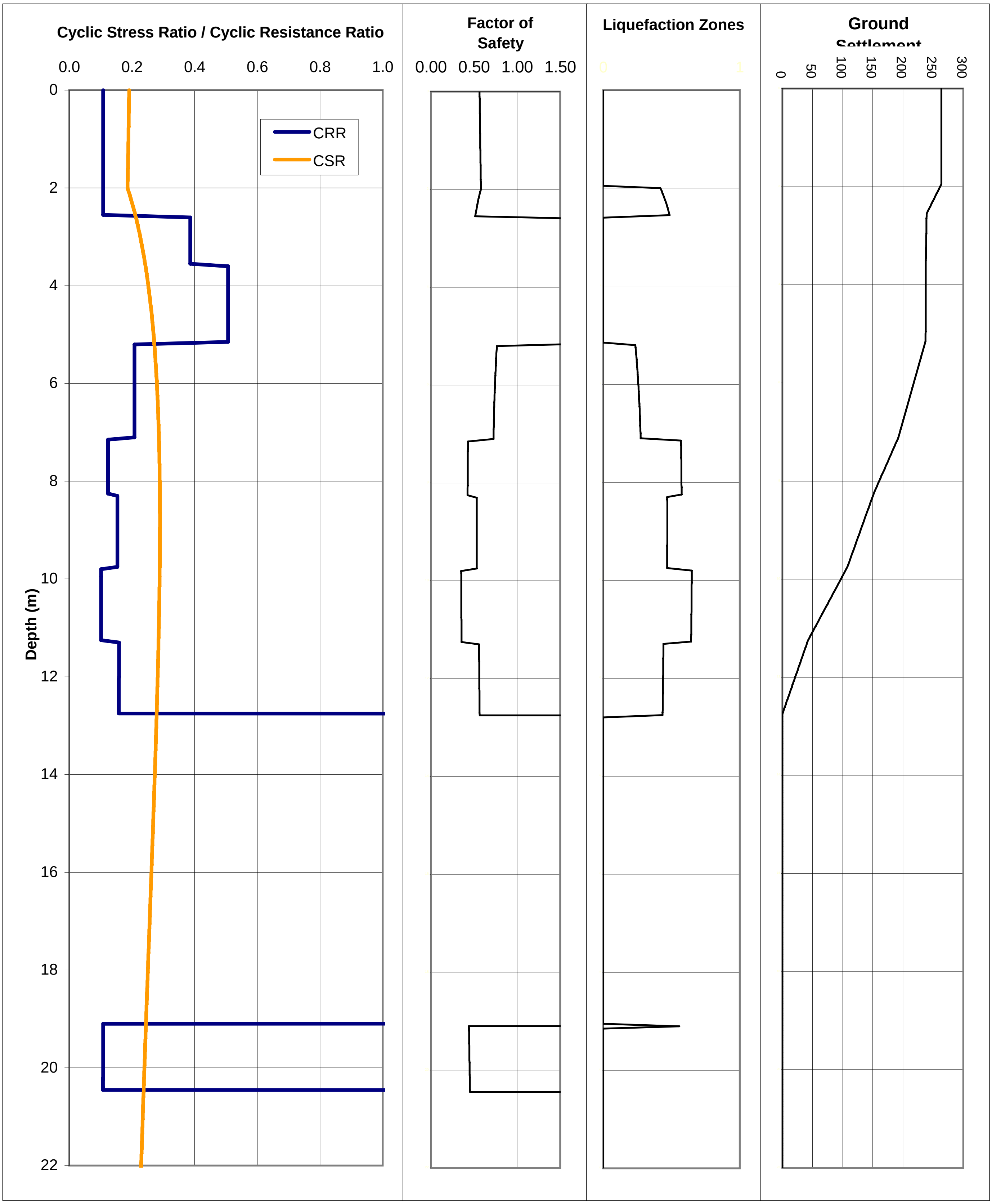


Note:	Based on Idriss & Boulanger (2006) and Ishihara & Yoshimine (1992)	**SLS @ 0.15g**
	a(g) = 0.15	M = 7.5

Liquefaction Potential Analysis

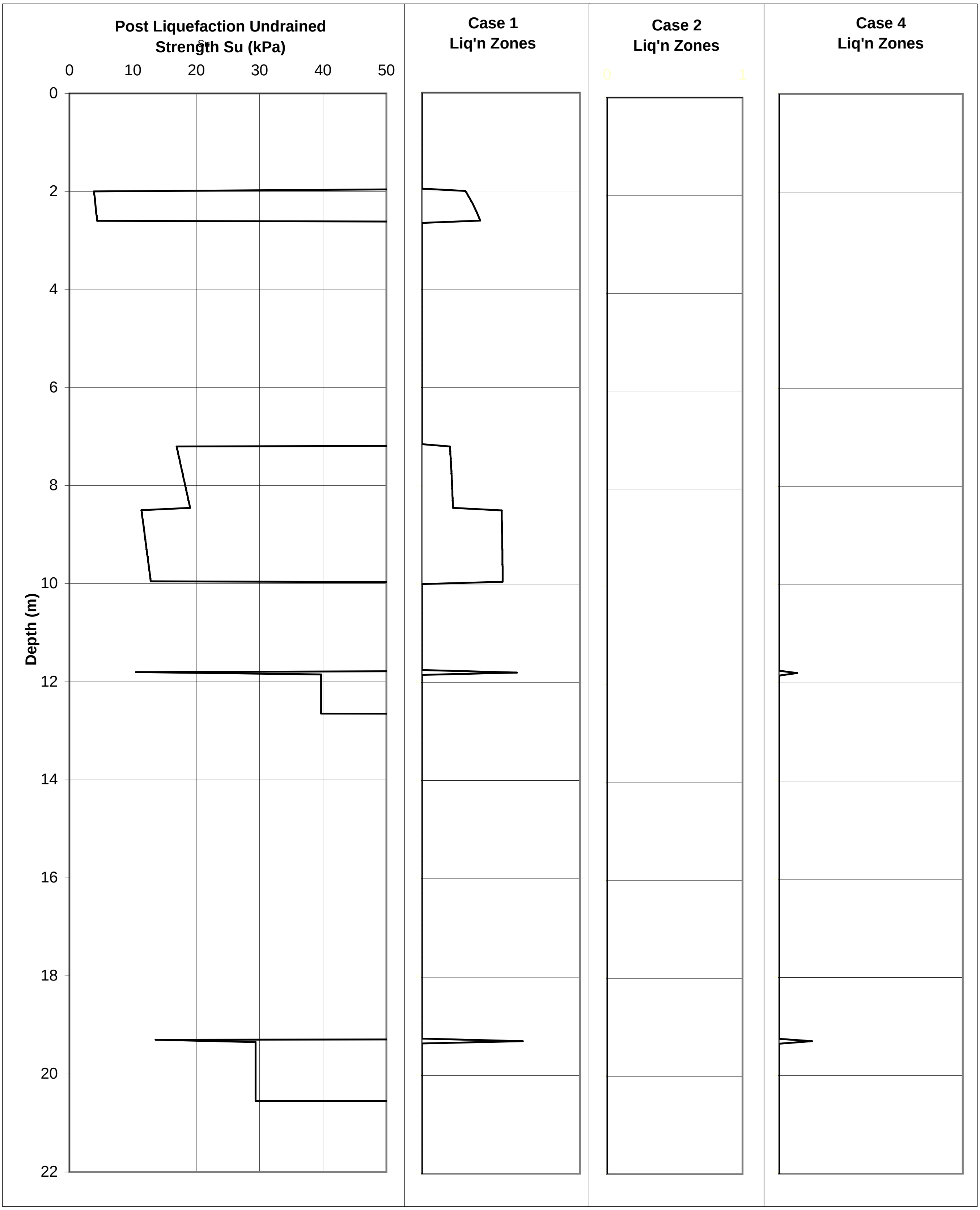
GEOTECH CONSULTING LTD

Project:	79 Cambridge Terrace	Hole No:	**BH2**
Client:	Buchanan & Fletcher Ltd	Job No:	3984



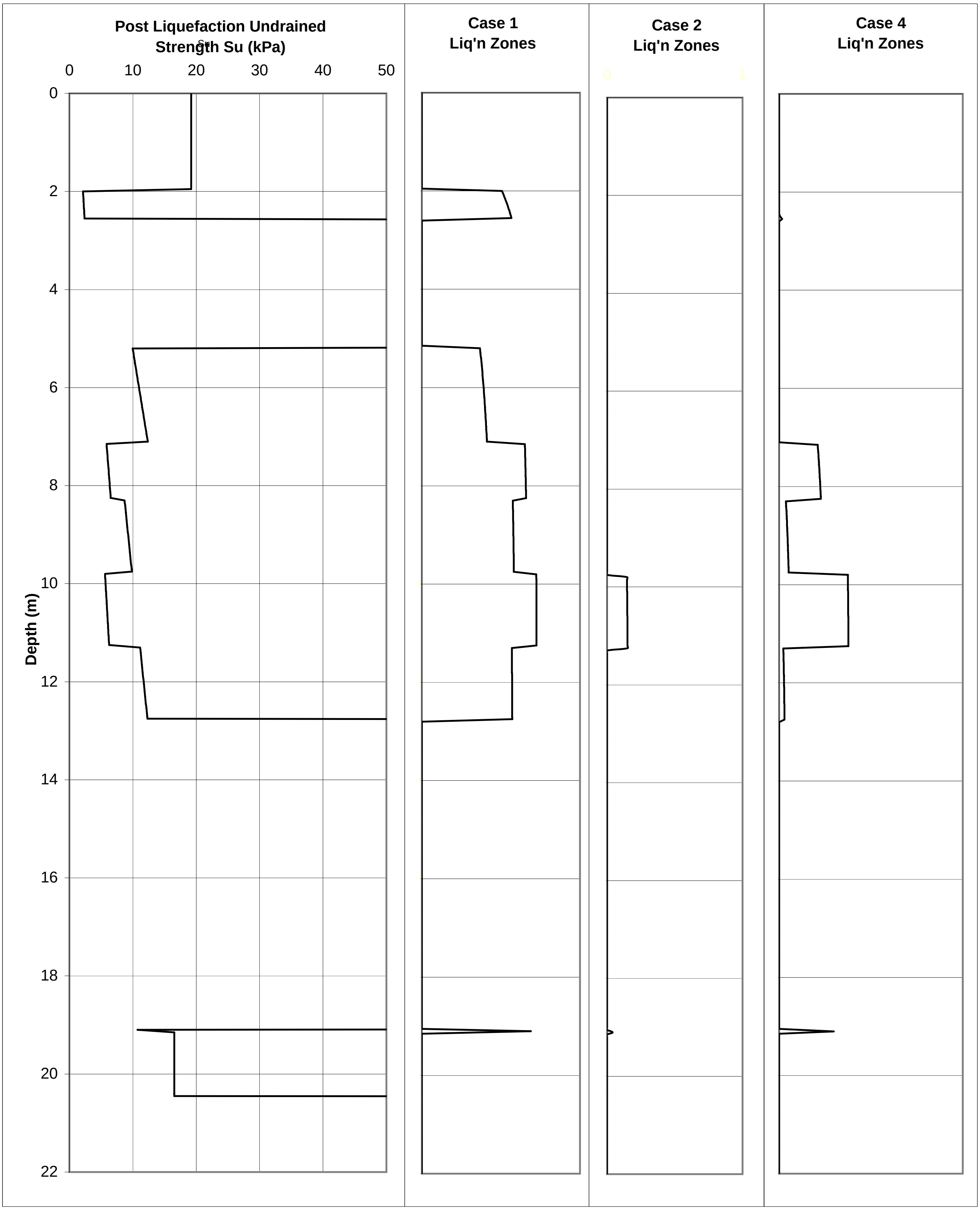
Note:	Based on Idriss & Boulanger (2006) and Ishihara & Yoshimine (1992)	**22-Feb-11**
	a(g) = 0.4	M = 6.3

	Post Liquefaction Strength			
GEOTECH	GEOTECH CONSULTING LTD			
	Project:	79 Cambridge Terrace	Hole No:	**BH1**
	Client:	Buchanan & Fletcher Ltd	Job No:	3984



Note: Based on Olson & Stark (2002)

GEOTECH	Post Liquefaction Strength			
	GEOTECH CONSULTING LTD			
	Project:	79 Cambridge Terrace	Hole No:	**BH2**
	Client:	Buchanan & Fletcher Ltd	Job No:	3984



Note:	Based on Olson & Stark (2002)

Buchanan & Fletcher Ltd
CONSULTING STRUCTURAL ENGINEERS

DIRECTORS / PRINCIPALS
Michael R. Fletcher BE (Hons), DBA, MIPENZ
(Structural), CPEng, Int PE(NZ)
David J. Eaton BE, MIPENZ

Bradley Nuttall Bldg 79 Cambridge Terrace
PO Box 4571 Christchurch 8140 New Zealand
Phone (03) 366-0304 Fax (03) 377-4308
Email admin@bandf.co.nz

17 May 2011 4055/MRF

Columbus Property Holdings Ltd
7 Amherst Place
CHRISTCHURCH 8022

Attn: Brian Columbus

Dear Brian

BRADLEY NUTTALL HOUSE, 79 CAMBRIDGE TERRACE, CHRISTCHURCH
POST EARTHQUAKE STRUCTURAL INSPECTION – PHASE 2

1. ENGAGEMENT AND BRIEF

Buchanan & Fletcher has been engaged by you to provide structural engineering inspections and assessments following the earthquake of 22 February and to design, draw and specify any necessary repairs.

There is extensive non-structural damage to the building. That damage is not covered in this report. We recommend that an architect or architectural draughtsman be engaged to identify non-structural damage and to design and specify repairs.

The work is to be carried out in 4 phases, as set out in our Agreement for Consultant Engagement. This report covers Phase 2:

- An initial walk around with Spotless Services and a building contractor, to identify where key parts of the structure are to be exposed for inspection.
- Carry out a detailed inspection of selected parts of the structure.
- Prepare a report.
- The Phase 2 inspection was planned and executed in conjunction with Holmes Consulting Group, who have peer reviewed this report.

We carried out our initial walk around on Friday 11 March, with Scott Thompson of Spotless and Alan Dibnah building contractor. We returned to carry out our detailed inspection on Monday 14 March with Richard Seville of Holmes Consulting as well as Scott Thompson and Alan Dibnah.

2. BUILDING DESCRIPTION

Bradley Nuttall house is a 7 storey building located on the northwest side of Cambridge Terrace. For the purposes of this report it will be assumed that the building is oriented north/south, with Cambridge Terrace due south of the building. Refer to pages 18 and 19 for typical ground and upper floor plans.

2. BUILDING DESCRIPTION continued

The building was designed in 1985. Architectural design was by Richard Proko Ltd, and structural design by Alan M Reay. It has 7 main floors – the ground floor and levels 1 to 6 – plus a plant room at level 7 and a lift machine room at level 8. The main floors are approximately 22m square, excluding the external walkways.

The roof is shallow pitched and is clad with steel roofing on steel purlins and steel beams and posts. The suspended floors are Stahlton, comprising a 75mm thick mesh reinforced topping over 25mm thick timber boards and 100mm deep prestressed concrete planks. The prestressed planks span 7.2m north/south along the building. They are supported on precast concrete beams and insitu concrete columns which extend from the ground floor up to level 6.

The ground floor is a 100mm thick unreinforced concrete slab on grade. The gravity columns are founded on reinforced concrete pads. The earthquake resisting shearwalls are founded on large reinforced concrete beams. The ends of some of the foundation beams are connected to cast-in-place reinforced concrete bulb piles. The piles extend down to about 9.5m below ground floor level.

The exterior of the building is clad with precast concrete panels on all 4 sides.

Earthquake resistance is provided by reinforced concrete shearwalls around the service core at the west side of the building. There are 2 north/south walls - at the back of the liftshaft and on the west exterior wall - and 2 east/west walls located to the north and south of the core.

There are precast concrete “scissor” stairs within the core. These provide 2 means of egress from each floor.

3. DOCUMENTS PROVIDED

You have provided us with a copy of the Architectural and Structural Engineering drawings for the building.

Alan Reay Consultants Ltd inspected the building following the September 4 earthquake and produced a report dated 29 November 2010. You have provided us with a copy of this report.

4. DRAWING REVIEW

Before inspecting the building we carried out a review of the original structural drawings, to identify key structural elements and connections that needed to be checked on site.

Items we identified as needing particular attention were:

4.1 Interconnection between Foundations

The foundations to the shearwalls and columns on the west side of the building are well connected via foundation beams. However the column foundations on the east side have only minimal interconnection. These columns land on footing pads and their only restraint is by ties into the 100mm thick unreinforced ground floor slab. At grids B3, B4, C2 & C5 restraint is via a D20 tie bar cast into the slab. At the eastern corners (grids B2 & B5) restraint is by the 2 – D12 bars cast into the slab edge thickenings adjacent to each corner.

4.2 Eccentricity of Earthquake Resisting System

The earthquake resisting walls are located at the west side of the building, around the liftshaft and stairwell. For earthquake loads across the building (east/west) the resisting system is symmetrical. However for earthquake loads acting along the building (north/south) the resisting system is offset by about 7 - 8 metres from the building's centre of gravity and is therefore quite eccentric. This means that, when subjected to north/south earthquakes or earthquakes with a north/south component, the building is subjected to torsion or twisting.

As the building twists in an earthquake, gravity columns will be displaced laterally. The columns furthest from the shear core (ie those near the eastern corners) will be displaced the furthest. Columns subjected to a lot of lateral movement can fail unless they are designed and constructed to accommodate such movements.

4.3 Interconnection between Upper Floor Gravity Beams

The upper floor gravity beams run east/west across the building on lines 2, 3, 4 & 5. There are no tie beams running perpendicular to these beams, so their only interconnection is via the 75mm thick slab topping which is reinforced with 665 mesh.

4.4 Ties between Upper Floors and Shearwalls

The upper floors of the building act as diaphragms, transferring earthquake loads into the shearwalls. The floor diaphragms are connected to the walls via reinforcing bars. These run across the walls on lines 3, 4 & E and are turned out of the west wall on line F.

The tie reinforcing appears to be minimal, particularly at the north/south walls on lines E & F.

5. PREPARATION FOR INSPECTION

Prior to our main inspection we visited the building on 10 March with Scott Thompson of Spotless and Alan Dibnah, builder, to point out the key parts of the structure that were to be exposed for inspection. These parts were:

- On ground floor lift carpet at columns B2 – B5 to check for gaps between columns and floor slab. Check these columns for verticality using a spirit level and mark.
- 2 ground floor block walls at top and bottom.
- Tops and bottoms of columns B2, B3, B4, B5, C3 and C4 at levels G–1, 2–3 and 5–6.
- Slab to shearwall connections from underside at 3 levels.
- Beam to end of shearwall connections at the junctions of line D/E with lines 3 and 4, at the undersides of levels 1, 3 and 6.
- Stair units at tops and bottoms of flights at 3 levels.
- Clean coating off soffits of 2 stair units that had bowed down.

In addition it was agreed that Spotless / Alan Dibnah would:

- Take levels on a 3.6 metre grid over the ground floor – ie at every column and midway between columns.
- Arrange for lift servicemen to attend the main inspection on 14 March so the inside of liftshaft could be inspected.

During this visit we accessed the Lift Motor Room on level 8 and were able to check the steel beams that support the lift equipment. We saw no signs of movement or damage to these beams.

6. VISUAL INSPECTION ON 14 MARCH

As noted above we carried out a detailed inspection on 14 March. Personnel present were:

- Scott Thompson, Spotless Services Ltd
- Alan Dibnah, Builder
- Richard Seville, Holmes Consulting Group
- Mike Fletcher, Buchanan & Fletcher Ltd
- 2 Otis Elevators staff for the first part of the inspection.

- *Photos A to I on pages 15 – 17 show cracking and other damage.*

Our observations were as follows:

6.1 Ground Floor

Floor levels had been taken at some of the columns, and were marked on the columns. Differences in levels were relatively small, and there did not appear to have been any significant settlement of the ground floor.

Where the carpet had been lifted in front of the columns on Line B, there were no signs of movement between the columns and the surrounding ground floor slab. During our inspection, carpet was removed for 2 metres in front of one of the line B columns. This allowed us to check for cracking parallel to line B, at the point where diagonal tie bars cast into the ground floor slab terminate. No cracking was found.

Two block walls around the main lobby had been exposed for our inspection. These walls appear to cantilever from the ground floor. They stop short of, and are not fixed to, the floor above. The walls showed no signs of damage. Block walls across the front of the liftshaft (near line D) are fixed to both the ground and first floors. There is no visible damage to these walls, but there is some separation between them and the precast concrete walls that form the north and south sides of the liftshaft.

6.2 Concrete Columns

At ground floor level some of the columns had been checked for verticality, and the results marked on the columns:

- Column B2 has cracked cover concrete at the top and leans 25mm to the north. There is no damage at the base of the column.
- Column B3 is undamaged.
- Column B4 has cracked cover concrete at the top and leans 20mm to the west. There is no damage at the base of the column.
- Column B5 has cracked cover concrete at the top which extends into the line 5 beam above. One of the prestressed Stahlton planks has cracked at the beam.
- Column C3 has minor cracking to cover concrete at its corners at both top and bottom.
- Column C4 has no obvious damage.

- *See Photo A for typical cracking at top of column.*

6.2 Concrete Columns continued

At level 3:

- Column B2 has cracked cover concrete at the top. There is no damage at the base of the column.
- Column B3 has no obvious damage.
- Column B4 has 2 cracked corners at the top, but no damage at the bottom.
- Column B5 has 1 cracked corner at the top, but no damage at the bottom.
- Columns C3 & C4 have no obvious damage.

6.3 Shearwalls

6.3.1 East/West Shearwalls – Lines 3 and 4

The east/west shearwalls show an extensive pattern of diagonal cracking at the lower levels with cracks in each direction at about 300mm centres. There is also cracking at the upper levels, but this is more widely spaced. Where checked, the cracks run right through the walls.

Generally damage to the line 3 wall is more significant than damage to the line 4 wall.

Crack widths are difficult to estimate because of the plaster coating on the walls, but we estimate them to be up to 0.5mm at the lower levels reducing to 0.2mm near the top of the building.

- *See Photo B showing diagonal cracks in the line 3 wall.*

Damage on the ground floor near grid F3 is described in 6.3.3 below.

6.3.2 North/South Shearwall – Line F

This is the west exterior wall of the building. From the outside, multiple horizontal cracks are visible for at least 3 levels above ground. These appear to be regularly spaced, at about 300mm centres. Because this wall is extensively lined on the inside, it wasn't possible to check whether the cracks run right through the wall. There is a minor diagonal crack visible on the inside of the wall from within a cleaner's cupboard on level 5.

- *See Photo C showing horizontal cracks from outside.*

Damage on the ground floor near grid F3 is described in 6.3.3 below.

6.3.3 Junction of Line F and Line 3 Shearwalls at Ground Floor Level

At this location there is a door opening in the line F wall adjacent to line 3. There is significant horizontal cracking in the line 3 wall, which connects to the short length of line F wall to the north of the door opening. Cover concrete has spalled off the north end of the line F wall at its base.

Cracks in the line 3 wall here are horizontal rather than diagonal.

Crack widths in this location are up to 0.7mm.

- *Refer Photo D.*

6.3.4 North/South Shearwall – Line E

This is the wall at the back of the liftshaft. It is connected to the walls on lines 3 and 4 by precast reinforced concrete coupling beams cast into the line E, 3 and 4 walls.

No cracks were noted in the line E wall itself, but there are vertical cracks visible at the joint between the coupling beams and the main wall at most levels. These vary in width from an estimated 1.0mm at level 1 down to 0.1mm at level 6. At grid D3 on level 3 we noted diagonal cracking in the coupling beam itself as well as a vertical crack at the beam to wall junction.

- *Refer Photo E showing a typical vertical crack.*

6.4 Connections between Floor Beams on Lines 3 & 4 and Shearwalls

The concrete floor beams on lines 3 and 4 transfer earthquake loads into the east/west shearwalls. The condition of these connections was checked at grids 3D/E and 4D/E at several levels. The floor levels noted here are those of the beam, which was always viewed from below.

At level 1:

- At 3D/E there is a horizontal crack across the column that forms the east end of the shearwall.
- At 4D/E there is minor cracking at the beam to wall junction.

At level 2:

- At 3D/E there is moderate damage to the beam to shearwall junction with some spalling of the end of the shearwall. There are hairline vertical cracks in the beam itself.
- At 4D/E there is cracking in both the beam and the column, but no spalling.

- *Refer Photos F and G.*

At level 4:

- At 4D/E a corner of the column has cracked where the beam seats. There are vertical cracks in the beam away from the shearwall.

At level 6:

- At 4D/E there are very minor cracks in the beam to wall junction and vertical hairline cracks visible from the north side.

6.5 Connections between Shearwalls and Adjacent Floor Slabs

These were checked from the underside at a number of locations. No signs of damage or movement were noted.

6.6 Liftshaft

We examined the inside of the liftshaft from ground floor level, where we were able to stand inside the over-run pit; and at level 6, through the lift doors.

The bottom precast panel on line 4 (the south side of the shaft) looked as though it may have tipped to the east. However, as the lift guides are straight, this situation must predate the earthquakes.

At level 6 the precast panel on line 3 (north side of the shaft) is badly cracked. The cracking predates both the September and February earthquakes, but looks to have worsened since we last saw it in September.

As noted in Section 4 we accessed the Lift Motor Room on level 8 during our visit on 10 March. We saw no signs of movement or damage to the steel beams that support the lift equipment.

6.7 Stairs

Each floor level is served by a pair of stairs which sit side by side in a “scissor” arrangement. The stair flights are constructed in precast reinforced concrete, and are built into the floor slab at the top and bottom of each flight.

Several stair flights have bowed downwards, with the maximum bow estimated at 20mm. Where the “Whisper” soffit lining has been removed from beneath 2 stair flights, cracks are visible in the stair soffits. The cracks run across the stair flights.

We saw no evidence of damage at the support points at the tops and bottoms of the stair flights.

- *Refer Photo H showing a bowed down stair unit.*

6.8 Precast Cladding Panels

All 4 sides of the building are clad with precast cladding panels with an exposed aggregate finish. We examined panels on the south side of the building at level 6, and panels on the west side at either side of the shearwall from both the ground floor and level 2.

We saw no obvious damage to panels or fixings on the south side at level 6.

On the west side (line G), to the south of the line F shearwall, 2 precast units at level 1 appear to have pulled away from the wall by 20 – 30mm. The units involved are small end units – a beam unit about 1.2m square and a hand rail unit about 1.2m long x 190mm deep. They are not the main units supporting the external walkway.

- *Photo I shows the separation of the cladding units from the Line F wall.*

6.9 Site and Surrounds

The south side of the building is about 25 metres from the Avon River.

There is slumping in the ramp leading down to the basement carpark of the adjacent building to the west, and some signs of liquefaction in the basement itself. We saw no evidence of liquefaction or settlement on the other 3 sides of the building, or between the building and the river

7. DISCUSSION

7.1 Earthquake Loadings

The building was designed in 1985. The earthquake loads used for the design would have been those of the then loadings standard NZS 4203:1984. These loads are very similar to those of the current loadings standard NZS 1170.5:2004, which would have been used for the earthquake design of a building in Christchurch prior to the February 22 earthquake.

Buildings are designed for a level of earthquake shaking that is expected to occur once every 500 years. At this level of shaking the building is expected to sustain structural damage, but not to collapse. The February 22 earthquake caused severe shaking at the site. Earthquake loads in excess of 100% of design loads were recorded in some locations around the central business district for a short period (approximately 10 seconds).

As a result of the September and February earthquakes, and new knowledge of faults, it is felt that there may be increased earthquake activity in Canterbury for up to 50 years or more. This could include another earthquake similar to that of 22 February. The Structural Engineering Society of New Zealand (SESOC) has recommended that earthquake design loads be increased to allow for this possibility.

SESOC have recommended an increase in design earthquake loads of 36% for buildings with a period of less than 1.5 seconds. This would apply to your building which has a period of approximately 0.7 second. We expect that this increased loading will be adopted by the Department of Building and Housing and the Christchurch City Council, and will become mandatory for all new buildings in Christchurch.

7.2 Damage and Repairs

Damage and possible repair options for various elements of the building structure are discussed in sections 7.3 to 7.10 below. Note that, apart from the discussion in section 7.9 regarding improving the performance of the stairs, the repairs suggested here would only restore the building to a condition similar to that it was in prior to the earthquakes.

7.3 Ground Floor

Where floor levels had been marked on the columns differences in levels were small, indicating that the floor has probably not settled. However this information was not complete and further levels are needed over the entire floor.

The absence of cracking or movement in the ground floor slab adjacent to the line B columns, and in the wider area in front of one of the columns, suggests that it unlikely that these columns have moved outwards (ie to the east).

The block walls around the entry lobby do not form part of the earthquake resisting system. Most of these walls cantilever from the ground floor and are not connected to the first floor. We found no damage in the 2 walls which were exposed for our inspection.

The walls at the front of the liftshaft, near line D, are connected to the first floor but are too short to have any significant effect on earthquake resistance. There is some separation between these walls and the precast panels that form the sides of the liftshaft. This damage is cosmetic.

7.4 Concrete Columns

From the measurements taken by Alan Dibnah, several of the ground floor columns are on a lean. However the directions of the leans are not consistent, and damage to the columns does not correspond well with the measured leans. This suggests that the leans may pre-date the earthquake. More information is needed, so all of the reasonably accessible ground floor columns should be checked for verticality in both the north/south and east/west directions. Refer section 9.1.2.

Cracking observed at the tops and bottoms of columns is earthquake induced damage and repairs will be required. When repairs are carried out, all columns throughout the building will need to be checked. Repairs will probably involve removal of cracked cover concrete and building up of the damaged areas with new micro-concrete.

The damage to the beam and Stahlton unit above column B5 can probably be repaired by epoxy injection.

7.5 Shearwalls

7.5.1 Cracking and Damage

Diagonal cracks in the shearwalls on lines 3 and 4 are not wide – typically 0.5mm or less – but they are numerous. At the lower 2 levels the cracks are at about 300mm centres, and extend the full height and length of the walls. The cracks are at about 45 degrees in both directions, suggesting that these walls have been worked hard in both directions by the earthquake.

Cracks in the line F wall are horizontal and appear narrower than those in the line 3 and 4 walls. However there are a high number of cracks, and some walls on other buildings with similar narrow cracks have been found to have fractured reinforcing bars. These cracks will need to be investigated further.

Cracking and damage at the ground floor junction of the line F and line 3 shearwalls is significant. Repairs will involve breaking away and re-casting of damaged concrete.

The line E wall is a coupled shearwall. It utilises the beams over the stairwell doors and their connections to the line 3 & 4 walls to increase its earthquake resistance. These beams were precast complete with reinforcing and cast into the line 3, 4 & E walls as the building was constructed. We saw no damage to the wall itself, but the vertical cracks between the coupling beams and the main wall suggest that the diagonal reinforcing in the coupling beams has yielded in the earthquake.

7.5.3 Torsion and Eccentricity

As discussed in section 4.2, the building's earthquake resisting system is located against the west side. The building is therefore eccentric for north/south earthquakes, or earthquakes with a north/south component, and subject to torsion or twisting.

7.5.4 Significance of Cracking

The cracks in the shearwalls show that they have been subjected to high earthquake loads and that the reinforcing has been stretched. Two consequences of this are:

- Reinforcing may have yielded (ie permanently stretched) across the cracks, particularly the wider ones. There is a limit to how far reinforcing steel can be stretched before it fails, and also a limit to the number of earthquake cycles involving yielding to which a reinforced concrete wall can be subjected. The reinforcing in these shearwalls is high yield. This is stronger than the alternative mild steel reinforcing, but also less resilient.
- In their cracked state the walls are less stiff than when they were uncracked. This means that they, and the building as a whole, will move more in an earthquake than they would have before. Because the building's earthquake resisting system is eccentric in one direction, it will now twist more in an earthquake with a north/south component. In an extreme event, the increased movement and twisting could cause severe damage to or even collapse of columns remote from the shear core.

Hence before designing any repair for the shearwalls it must be established that the reinforcing has not been yielded to the point where it would perform poorly in a design earthquake, and that the cracks can be repaired so as to restore the stiffness of the shearwalls to their original levels.

A full earthquake analysis will be needed to help answer these questions. Refer section 9.1.1

7.6 Connections between Floor Beams on Lines 3 & 4 and Shearwalls

There was damage to the beam to shearwall connection at every location we inspected, indicating that these junctions have been subjected to large earthquake forces.

When repairs are carried out, all of the beam to shearwall connections throughout the building will need to be checked. Repairs are likely to involve removal of cracked cover concrete from columns and shearwalls and building up of the damaged areas with new micro-concrete, plus epoxy injection of cracks in the beams.

7.7 Connections between Shearwalls and Adjacent Floor Slabs

We found no evidence of damage or movement between shearwalls and the adjacent floor slabs. However more checks should be made as repair work is carried out.

7.8 Liftshaft

Both lifts are operational, and the only damage we noted was to the precast wall panel on line 3 at level 6, which is badly cracked. The cracks extend right through the panel. This damage appears to predate all of the earthquakes, but in our opinion has worsened since we saw it in September.

The panel supports lift guides, which also provide braking for the lift car in the event of an emergency. The cracks should be repaired to restore the panel to its intended strength.

7.9 Stairs

Because the stair units are fixed to the floor slab at the top and bottom of each flight they act as props, transferring earthquake forces between levels. If the propping forces are large enough, the stair units can bow and possibly buckle and fail. In newer buildings this problem is avoided by fixing the stair flights at one floor only and allowing them to slide on the other floor.

In this building, the top and bottom fixings appear undamaged. However several stair units have bowed downwards and there are cracks in the two soffits that were exposed for our inspection. The bowing and cracking indicate that the main stair reinforcing has probably yielded.

Repairs will be needed. As a minimum these would return the stairs to their pre-earthquake condition, but it would be preferable to modify them for improved earthquake performance.

Repairs could involve grinding off the soffit linings and fixing carbon fibre strips to the undersides of the stairs to supplement the existing reinforcing. Not all the stair units have bowed down, but it would be advisable to carry out the same repair on all units.

Options for improving the stairs' performance include, in increasing order of difficulty:

- Fitting 1 or 2 steel cross beams under each stair unit so that it cannot drop in the event of a failure.
- Modifying the support detail at top or bottom of each unit to allow it to slide.
- Replacing the stair units with new units, probably framed in steel, designed to slide at one end.

7.10 Precast Cladding Panels

There was no obvious damage to the precast cladding panels we inspected, or to their fixings except as noted in Section 6.8.

The 2 units that may have pulled away from the line F shearwall have a simple dowel fixing into the wall. The beam unit (indentified as PF6 on the original drawings) does not rely on the dowel for support. However the handrail unit (PF 8) does rely on the dowel. Although it is not clear whether the separation of these units has been caused by the earthquakes, supplementary fixings should be put in place to prevent the panels from moving further.

As part of the repair operations, all precast panels and their fixings must be thoroughly checked and any damage or movement reported.

7.11 Site and Surrounds

This part of Cambridge Terrace has been identified by a University of Canterbury geotechnical engineer as having sustained low to moderate liquefaction in the February 22 earthquake. The slumping and possible liquefaction we observed to the west of the building tends to confirm this.

If liquefaction has occurred then it is possible that the foundations and/or piles have been compromised. More investigation is needed. In the first instance accurate levels should be taken on the ground and first floors to determine whether the ground floor itself, or any of the main structural elements, have settled. Following this, geotechnical advice should be sought. This may lead to further investigations.

8. VISUAL INSPECTION ON 2 MAY

We carried out a further brief visual inspection on Monday 2 May, following a significant aftershock on 16 April. There had been some changes to the damage we noted on our detailed inspection of 14 March:

- Concrete columns appeared unchanged.
- Cracks are now visible in the line 3 and 4 shearwalls up to an including level 5. These may have been present before, but weren't specifically noted.
- Generally cracking in the line 3 and 4 walls now appears more extensive at the lower levels.
- Cracking in the line F shearwall appears unchanged.
- On line E (back wall of the liftshaft), vertical cracks at the coupling beam joints now appear wider. A level 1 crack previously noted as 1.0mm wide now measures 1.4mm. There are now diagonal cracks in the coupling beams at all levels up to level 4, and at both ends of line E (previously only noted at the north end at level 3).

9. RECOMMENDATIONS

Before proceeding to the design and specification of detailed repairs for the building, more investigation and analysis is needed. In particular a full earthquake analysis needs to be carried out on the building to determine whether and how the cracked shearwalls can be effectively repaired. Until this question is answered, there is little point in beginning work on the other repairs.

9.1 Immediate Investigation work

We recommend that the investigation work set out below should proceed immediately. The results of this work will enable a decision to be made on the future of the building.

9.1.1 Seismic Analysis

In our procedure for structural inspections and review dated 6 March we recommended that a full earthquake analysis be carried out on the building to assess its resistance to collapse in a major earthquake. This was to be carried out as Phase 4 of the procedure, at the same time as repairs were designed.

However, following our inspection and having seen the extent of cracking in the shearwalls, we now recommend that the earthquake analysis be carried out immediately, before the repairs are designed. This will help determine whether and how the shearwalls can be repaired.

Points to be addressed by the analysis include:

- How did the building's earthquake resistance, assuming uncracked shearwalls, measure up against the current loadings standard NZS 1170.5:2004 (as it stood prior to 22 February)?
- How will the cracked shearwalls affect the building's performance? In particular how will any increased torsional movement (twisting) affect the performance of gravity columns remote from the shear core?
- Is the existing column reinforcing adequate?
- Are the connections from beams to shearwalls and from floor slabs to shearwalls adequate?
- Are there any weak points in the earthquake resisting structure itself?
- How will the proposed 36% increase in the design earthquake load affect all of the above points?

9.1.2 Column Verticality at Ground Floor Level

All of the reasonably accessible columns on the ground floor should be checked for verticality in both the east/west and north/south directions. The results should be marked up on a floor plan and forwarded to us. The columns involved are:

- All 4 columns on line B.
- 3 columns on line C (excluding C5).
- Column D2.
- Columns F2 and F5.

9.1.3 Levels at Ground and First Floors

Accurate floor levels should be taken on both the ground and first floors. Levels should be taken on a grid of no more than 3.6 metres in each direction and the results plotted onto a floor plan. The levels should be adjusted to allow for the thicknesses of any floor coverings.

We recommend that this work is carried out by a surveyor.

9.1.4 Geotechnical Report

A report should be commissioned from a geotechnical engineer. This should cover:

- Ground conditions beneath the building
- The possibility of the piles or foundations having been damaged by the February 22 earthquake.
- The potential for liquefaction damage in a future ultimate limit state design earthquake.
- The adequacy of the existing foundation system and piles.

The information on column verticality and floor levels described in 9.1.2 and 9.1.3 would need to be made available to the geotechnical engineer.

9.2 Future Investigation work

Once a decision has been made to proceed with repairing the building, further investigations will be required. The scope of these may be influenced by the investigations described in 9.1. Future investigation work will include, but will not be limited to:

9.2.1 Columns

All columns throughout the building will need to be checked at top and bottom for cracking or other damage. The beams that land on each column will also need to be checked.

9.2.2 Connections of Floor Slabs and Beams to Shearwalls

Although no damage was found where floor slabs connect to shearwalls, this was only checked at a limited number of locations. These connections should be checked in several locations at each floor level. Floor coverings will need to be lifted so that these connections can be inspected from above.

We found damage to the beam to shearwall connection on lines 3 and 4 at every location we checked. Hence the beam to shearwall connections on lines 3 and 4 must be opened up for inspection at every level.

9.2.3 Precast Cladding Panels

All of the precast cladding panels and their fixings must be thoroughly checked and any damage or movement reported.

9.3 Design of Repairs

The design and specifying of repairs to the building will follow on after the investigation work is complete.

9.4 Occupation of the Building

We recommend that the building remains un-occupied until all investigations are complete and repairs have been carried out.

10. SCOPE AND LIMITATIONS OF REPORT

10.1 Restriction of Use

This report has been prepared solely for the benefit of Columbus Property Holdings Ltd as our client with respect to the brief. The reliance by other parties on the information or opinions contained in the report shall, without our prior review and agreement in writing, be at such parties' sole risk.

10.2 Limitations of Report

The brief is as set out in Section 1 of this report. This report has been prepared for the purpose of the meeting the requirements of the brief and the report shall not be relied on for any other purpose.

Our investigation has extended to visual inspections of the building. The inspections were limited to selected parts of the structure building that were readily accessible or had been exposed for our inspection. No testing of strength of materials was carried out.

We did not carry out any structural engineering calculations or formal design work in the preparation of this report.

Please contact the writer if you have any questions or concerns.

Yours faithfully

M R Fletcher
DIRECTOR
BUCHANAN & FLETCHER LTD

PHOTO A
Minor cracking at top of ground floor column

PHOTO B
Diagonal cracking in line 3 wall at level 5

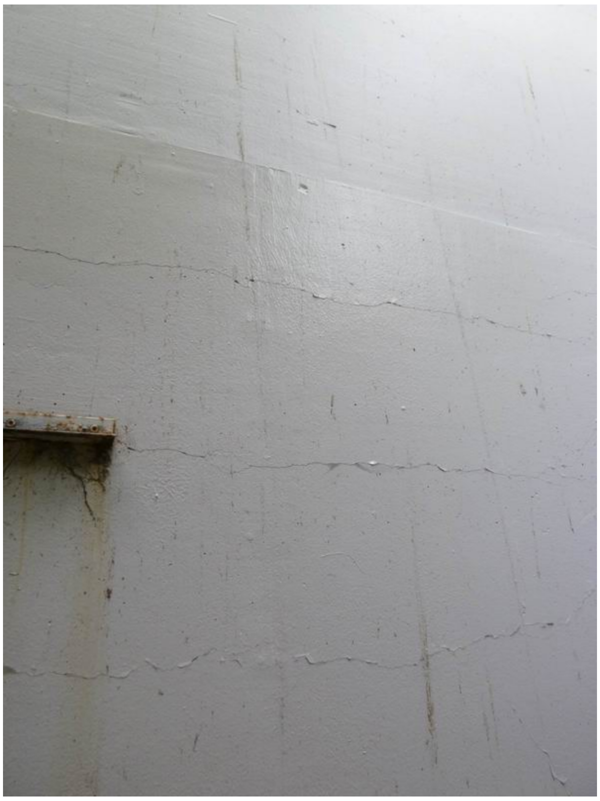


PHOTO C
Horizontal cracking in line F shearwall between ground and first floors

PHOTO D
Damage at junction of line 3 and F shearwalls, ground floor

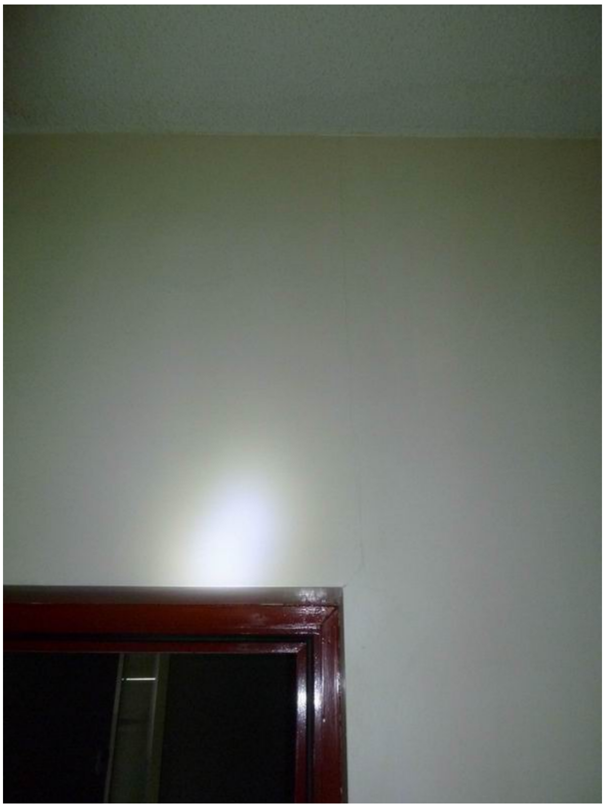


PHOTO E

Vertical crack between precast coupling beam and main shearwall on line F

PHOTO F

Damage to top of column and beam above at junction of line 3 beam and end of line 3 shearwall, at level 2 floor

PHOTO G

Close-up of damage to beam in Photo F

PHOTO H

Bowed down stair unit

PHOTO I

Separation of precast cladding panels from line F shearwall, at first floor level