6th International Conference on Earthquake Geotechnical Engineering
1-4 November 2015
Christchurch, New Zealand

Ground Improvement by Impact Compaction for Building Founded on Liquefiable Soils

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ABSTRACT

Following the recent earthquake sequence in Canterbury, reconstruction of the Waimairi Golf Club house was required. A site investigation was conducted to assess the ground properties for foundation design of the Clubhouse. This paper presents the construction steps undertaken for densifying the underlying soils and the specifications for the engineered fill. It also presents the methodology and results of the verification programme by Cone Penetration Testing (CPTu) and the degree of improvement achieved.

Introduction

Opus was commissioned to provide architectural design, structural design, geotechnical design and construction monitoring services for the new one storey clubhouse building for client Waimairi Beach Golf Club. The new building will replace the original one storey club house that was extensively damaged by the Christchurch Earthquakes in February 2011 (MWH, 2011). The original building had to be demolished and rebuild was agreed as the best option.

Although the new building is a light-weight structure, the poor performance of the lightly to medium loaded shallow pad foundations of the original club house in the February 2011 earthquake indicated in the early preliminary design stage that shallow foundations are not a safe and realistic option. The pad foundations suffered from either the sudden loss of bearing capacity from liquefaction or from the combination of high earthquake loads with capacity reduction of the liquefied soil. As a consequence, this led to the extensive damage of the concrete superstructure and foundation. Such foundation and superstructure system is anticipated to be capable to withstand up to 50mm of total allowable settlement and 25mm of allowable differential settlement based on international practice and experience (Bowles, 1997). The structural report (MWH, 2011) on the observed ground performance states that differential settlement of up to 134mm has been recorded over the Waimairi Club House and lesser of up to 90mm has been recorded to adjacent auxiliary buildings. This level of recorded differential settlement is in agreement with the above allowable limits. From these recordings it can be inferred that the total settlement of the buildings induced by liquefaction ranged between 180mm and 270mm.

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The new building with an extensive footprint of 680 square metres and with approximate dimensions 15m by 45m, was initially scoped to be founded on piles. At the preliminary structural design stage, the vertical load demand on the piles was estimated between 500 and 700kN. In order to fulfill the Building Code requirements, driven or bored piles had to be designed with an ultimate vertical capacity between 1000kN and 1400kN. For facilitating the pile design and for discerning the ground conditions and ground risks, a deep ground investigation was conducted in November 2012 comprising of three CPTu.

Site description

Waimairi Golf Club is located at Bower Avenue, 590m west of Waimairi Beach, 8km north east of Christchurch. The location and boundary of the site is presented in Figure 1. Immediately east of the site, the residential land has been categorized TC3 and immediately east TC2 (MBIE, 2011).The general ground profile of the site is flat with some hilly landscaping designed for golf. The site is surrounded by several pre-existing CPT's and boreholes taken to the maximum depth of 47.5m, however no such pre-existing site specific investigations were available near the proposed building location. According to the Earthquake Commission (EQC, 2012) the earthquake event specific groundwater table depth is between 0 and 1m below the ground surface.

Figure 1: Location and boundary of the site. (Canterbury Geotechnical Database, 2012)

The published geological map of the area by Brown and Weeber (1992) indicates the site is underlain predominantly by young fixed dune sand and beach deposits (Quaternary, Christchurch Formation).

Level of shaking at the site

No liquefaction observations were undertaken at the time of earthquakes for the private area bounded within Waimairi Golf Club, as this area is non-urban and non-residential. However, liquefaction was recorded for the greater area surrounding the site and for the area near the original club house building. Moderate to severe liquefaction has been observed following the 22 February 2011 event at the car park and building area, based on drive-through reconnaissance

(MWH, September 2011). From aerial photography interpretation, the severity of the subsidence of the greater area surrounding the site increased from one event to the next event:

- Minor observed liquefaction was recorded from 22 June 2011 event;
- Moderate to severe observed liquefaction was recorded from 13 June 2011 event;
- Moderate to severe observed liquefaction was recorded from 24 December 2011 event.

The site-specific contours of Conditional Median PGA (Bradley and Hughes, 2012) in Canterbury Geotechnical Database (CGD5110, Conditional PGA for Liquefaction Assessment) indicate for three significant earthquakes the PGA shown in Table 1.

Table 1: Conditional Median PGA values at the site for the three significant earthquake events corrected to $M_{7.5}$ event.

Event	**Magnitude M**	**$PGA_{7.5}$**
4th September 2010	7.1	0.16g
22nd February 2011	6.2	0.27-0.33g
13th June 2011	6.0	0.12-0.14g

According to guidance from the Ministry of Business, Innovation and Employment (MBIE, December 2012) the site can be regarded as having been "sufficiently tested" as it has experienced more than 170% of design SLS in the February 2011 event. The design SLS level of shaking is explained in the next paragraph.

Site investigations and ground conditions

In November 2012 Opus undertook three cone penetration tests with pore water measurements (CPTu) to 38m depth at the new clubhouse building footprint. The groundwater has been recorded for every CPTu before and after execution and found to be within 1m of the ground surface. Based on the normalized CPT Soil Behavior Type SBTn (Robertson, 1990) the ground conditions at the proposed clubhouse were inferred and presented in Table 2. Table 2 suggests that potentially liquefiable soils extend to a depth of 29.5m. In general the site is underlain by relatively clean sand with fines content (FC) smaller than 5%, however the top 1m is indicated to have fines content of up to 26%. A groundwater level of 0.8m below ground level was considered appropriate for undertaking liquefaction analysis. Effective CPT refusal was not achieved during the site investigation. These findings are in agreement with the published geological map of the site and with pre-existing nearby site investigations.

From the site specific investigations and the thickness of the detected layers the site is classified as Soil Class D (deep or soft soils), in accordance with NZS 1170.5:2004. The importance level of the clubhouse building is IL2. Interim guidance released by the Ministry of Business, Innovation and Employment (MBIE, 2012) recommends the following design PGA for the assessment of liquefaction:

- 0.13g for SLS, M=7.5 and 1/25 annual probability of exceedance;
- 0.35g for ULS, M=7.5 and 1/500 annual probability of exceedance.

The liquefaction analysis results obtained from the CPTu using CLiq software are presented in Figure 2 and Table 3. The Boulanger and Idriss (2008) method was used for both SLS and ULS and Robertson's method (2009) for ULS.

Table 2: Interpreted ground conditions from CPTu

Layer	**Soil behavior type**	**Depth encountered (m) (from-to)**	**Average q_c (MPa) [Range]**	**Average Ic [Range]**	**Liquefiable layer against ULS event (Idriss and Boulanger, 2008)**
I	Silty Sand and sandy Silt	0.0-1.0	5.4 [0.25-9.2]	1.84 [1.61-2.42]	No (above groundwater level)
II	Sand and silty Sand	1-22.85	16.0 [6.1-26.6]	1.65 [1.41-1.95]	Yes, with non-liquefiable layers detected between 1 and 7.6m depth
III	Clay and silty clay interbedded with sand	22.85-24.90	8.2 [1.2-21.3]	2.55 [1.77-3.48]	No liquefaction
IV	Sand and silty Sand	24.9-29.5	22.8 [8.4-29.6]	1.82 [1.73-2.25]	Yes, with some non-liquefiable layers in between
III	Clay and silty clay with thin layers of silty sand/sandy silt	29.5-37.8	6.13 [1.3-21.3]	2.78 [1.81-3.54]	No liquefaction for the clay layers but yes for the sandy thin layers

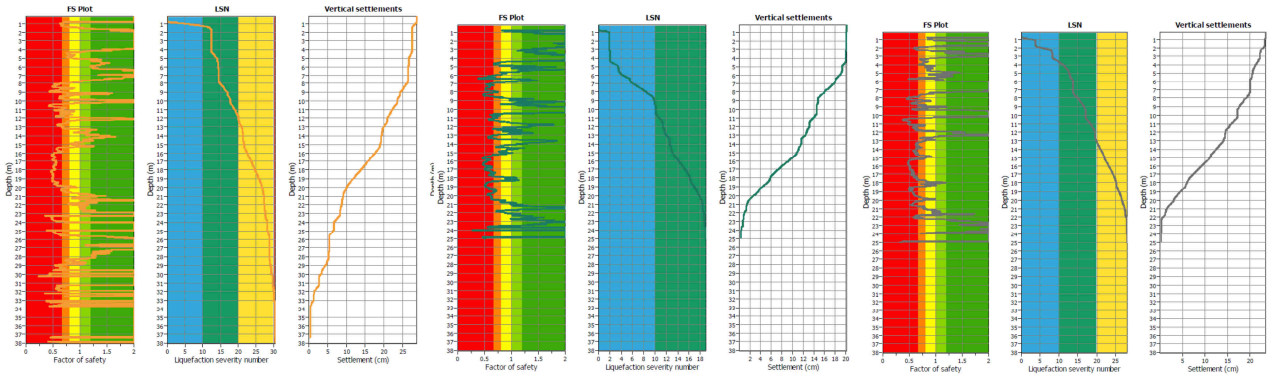

Figure 2: Liquefaction analysis results for ULS event based on Boulanger and Idriss (2008) method (CPT 1 to CPT 3 is from left to right).

Table 3: Liquefaction analysis results

SI	Predicted subsidence (mm)			Non-liquefiable layers at ULS (from-to) (m)
	Idriss and Boulanger (2008)		Robertson (1998)	
	SLS	ULS	ULS	
CPT 01	10	290	220	1.6-4.0 and 5.1-7.6
CPT 02	5	210	110	1.0-4.35
CPT 03	5	240	125	1.1-1.8 and 2.4-3.1

One of the main problems detected from the liquefaction analysis (refer Figure 2) is shallow potentially liquefiable layers of loose sand. These ground conditions create serious problems for placing directly on the ground any type of conventional shallow foundation. Both methods used produce significant liquefaction subsidence, with Robertson's method consistently estimating thicker non-liquefiable zones than Idriss and Boulanger's method (see Figure 3). These results are comparable with the observed total settlement of up to 270mm, with Boulanger and Idriss method to provide an estimation closer to the observed settlement. In any case both methods come to the same conclusion that shallow foundation is not a viable solution against a ULS event as both methods estimate subsidence in excess of the total allowable settlement of 50mm. However, the analysis indicates liquefaction induced subsidence for the serviceability event of less than 25mm. This amount of settlement is considered as tolerable for shallow foundations.

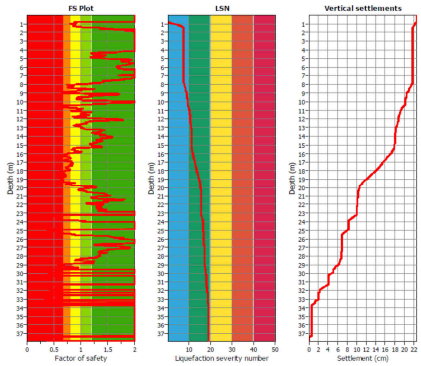

Figure 3: Robertson's method indicates a non-liquefiable crust between 1.5m and 8m depth

Thus, site specific measures needed to be incorporated such as piles or ground improvement.
As the site investigation did not encounter founding layer for the initially proposed piles, it was evident that an alternative foundation concept was required with some form of ground improvement.

Adopted foundation solution

A revision of the foundation options was undertaken considering the results of the site investigation. A stiff reinforced concrete raft foundation solution was considered which proved to be producing high levels of elastic settlement at very low working foundation stress levels due to the large raft width of 15m. Strip narrower footings seemed to be a better option if the ground was having a higher soil modulus that could be the result of ground improvement. A possible problem and risk was detected to be the increased fines content in the top 1m of the excavation which could inhibit the level of improvement. This material was then identified on site during excavation to be thicker than 1m and it was non-plastic dilative silt.

After discussions with the project manager, the structural engineers, the client and the contractor, the adopted foundation solution comprised a thickened reinforced concrete floor slab on perimeter strip footings formed on the top of 1.75m thick engineered fill above the ground level reinforced with biaxial geogrid. The use of the reinforced fill deemed as necessary due to the risk of the silty material to inhibit the level of ground improvement. The underlying in-situ ground was to be treated by impact compaction to mitigate liquefaction. The static bearing stress under the 500 to 700mm wide footing is in the order of 15 to 30kPa.

The concept behind the adopted solution is to provide a non-liquefiable crust below ground level that is composed of two distinct layers:

- the soil treated by impact compaction from the surface to 2.5m depth
- the non-liquefiable layers detected by the liquefaction analysis

Thus, a non-liquefiable crust with a total thickness between 2.5m and 10 meters was to be provided and was deemed as an economically viable solution. Furthermore, the 1.75m thick engineered fill above ground provides a stiff jacking-up platform for the strip footings in case the footings undergo liquefaction subsidence from a future earthquake event. The approach by NAVFAC DM-7.2 (1982) for the given strip footings on the top of this crust suggests no deterioration of bearing capacity when liquefaction occurs at depths in excess of 5.5 meters below the foundation.

Construction

At the time of construction and after some rainfall the water was ponded on the ground surface partially due to site topography. Furthermore, the silt content within the soil in the upper 0.5m and the shallow groundwater level at 0.5m rendered the site difficult to drain without the use of dewatering. To begin with, the upper 0.5m of ground was dewatered and then removed. Well points were installed by considering the permeability coefficients estimated by CPT correlations. Their arrangement was such that their influence radius covered the total excavation of approximately 30m by 25m. The depth of the well points was 2.5m and complete dewatering of the area was achieved in 48 hours. The CPT data with an average permeability K of 1.2 to $9X10^{-5}$ m/s proved to provide a reliable order of magnitude for the design of an effective dewatering scheme.

The impact compaction took place by using a tractor towed 13ton impact compactor of square cross section with 1m height. The input energy rating for this impact compactor was 31kJ per blow. The complete technical data of the compactor are presented in Table 4.

Table 4: Technical data of impact compactor

Total weight	13,150kg	Blows per second	2 at max recommended speed
Drum weight	9,070kg	Breaking force	217.67 MPa
Max recommended speed	13km/h	Compaction energy	31.39KJ
Effective compaction width including wheel roll			2.13m

The impact compactor densifies soils by dropping a weight from a height at a regular rate. The compaction was carried out for three consecutive days with 500 to 1000 passes. Grid survey of levels with accuracy of ±5mm indicates that after the impact compaction was carried out, the site was lowered by 200mm.The engineered fill was reinforced with four layers of biaxial geogrid at 400mm vertical spacing. A bi-axial E-Grid 30/30 geogrid was used with AP65 material. The achieved relative compaction as measured by NDM testing was found to be greater than 95% in all measurements, at all recommended points inspected. The field NDM testing was undertaken according NZS 4407:1991 Test 4.2.2 and the determination of the dry density/water content relationship according NZS 4402 Test 4.1.2.For the placement of the fill the approach recommended by the Specification for Highway Works (Barnes, 1995) was implemented.

Figure 4: Impact compactor in action.

Table 5: Post treatment CPT results and improvement

	Pre	Post
Maximum improvement depth	-	2.5 m
Average q_c range within the treated depth of 2.5m (MPa)	0.25-13.6	0.26-17.04
Average q_c (MPa)	9.4	12.7
Maximum q_c attained after treatment (MPa)	-	20.65 (CPT 2)
Settlements at SLS to 10m depth	4-10 mm	4-20 mm
Settlement at ULS to 10m depth	50-70 mm	37-65 mm
Minimum depth of non-liquefiable crust at ULS	1 m	4.4 m

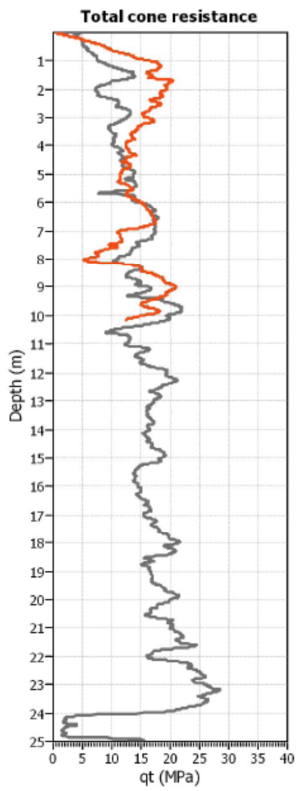

Figure 5: Total cone resistance increased (orange colour) after the impact compaction treatment to the depth of 2m.

Conclusions

In order to measure the achieved degree of improvement, three CPTu were carried at post treatment of the site at the same initial CPTu locations. The average results are summarized in Table 5 and Figure 5. In general, the impact compaction is not very effective when saturated loose silty material is present near the compactor. For this reason the engineered fill was adopted in the design for mitigating the risk from the silty material to inhibit the ground improvement from impact compaction.

The impact compaction has increased q_c by an average of 30% to about 2m depth. The influence of the impact compaction below this depth is diminishing with depth as was expected. In general the liquefaction induced subsidence at SLS remained at the same levels as before the treatment. The main difference from the treatment is observed at ULS where the subsidence was reduced between 5mm and 35mm, depending on the analysed CPTu for the top 10m. It is believed that the silty material present in the top 0.5m of the treated soil was dilating during impact compaction, as this dilation was monitored by surveying. This means that the level of improvement may have been higher if the silt content was lower. The building was constructed on time and within the budget.

Acknowledgments

We wish to thank the Waimairi Golf Club Directorate for their kind permission to Opus to use the relevant information from the given site.

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