Deep soil mixing in liquefiable pumiceous sand

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ABSTRACT

This paper presents a case study of the design and construction of deep soil mixing columns in liquefiable pumiceous sand ground conditions. A new extension to a wastewater treatment plant was constructed at a waterfront site. Assessment of Cone Penetration Test (CPT) data indicated that there was a significant potential of liquefaction during the design earthquake event. Deep soil mixing (DSM) columns were selected as the preferred mitigation measure as this option satisfied project requirements both in terms of constructability and required depth of treatment. Cement soil columns were constructed in a cell pattern beneath the foundation of the wastewater plant extension to confine liquefiable deposits. The design of the cement column cells was carried out using both a conventional limit equilibrium approach and finite element analyses. A stringent quality control testing was implemented to ensure consistency in column strength and mixing quality throughout the entire cell structure in tidal groundwater conditions. The cement soil columns were successfully designed and constructed within the project timeframe. Laboratory testing results showed the unconfined compressive strength of column samples achieved relatively high design strength in pumiceous sand deposits.

1 INTRODUCTION

Liquefiable alluvial and airfall ash deposits occur in central Tauranga, New Zealand. The risk of liquefaction is real in this region during seismic events. During the 1987 Edgecumbe earthquake, liquefaction related damage was observed in the nearby town of Whakatane (Berrill et al. 2001). Mitigation of the liquefaction potential was a key issue in the design and construction of a wetwell extension structure at Chapel Street Wastewater Treatment Plant, Tauranga. The wetwell extension site comprises variable, pumiceous and sensitive soils, which affect the selection of appropriate mitigation measures. This paper presents a case study of the inaugural use of deep soil mixing for liquefaction mitigation in New Zealand.

2 LIQUEFACTION POTENTIAL

2.1 Site Description

The wastewater plant extension site topography is relatively level. The proposed wetwell is a reinforced concrete structure that will house two effluent tanks and a final effluent pump. The structure was constructed on a raft embedded 3.1 m into the ground, and has a footprint of approximately 10.2 m by 5.8 m. The ground treatment site is located adjacent to Tauranga Harbour. A photograph of the site is shown in Photo 1.

2.2 Geology

Healy et al. (1974) indicates that the geology of the site predominantly comprises Holocene undifferentiated alluvium. The general ground conditions of the site are reported to vary, and include estuary and airfall ash deposits and lahar colluvium.

Photo 1: Ground improvement site.

2.3 Ground Conditions

The subsurface conditions predominantly comprise pumiceous sand and weakly cemented silt, which generally densify with depth. Interpreted general ground conditions are shown in relation to the wetwell extension structure in Figure 1.

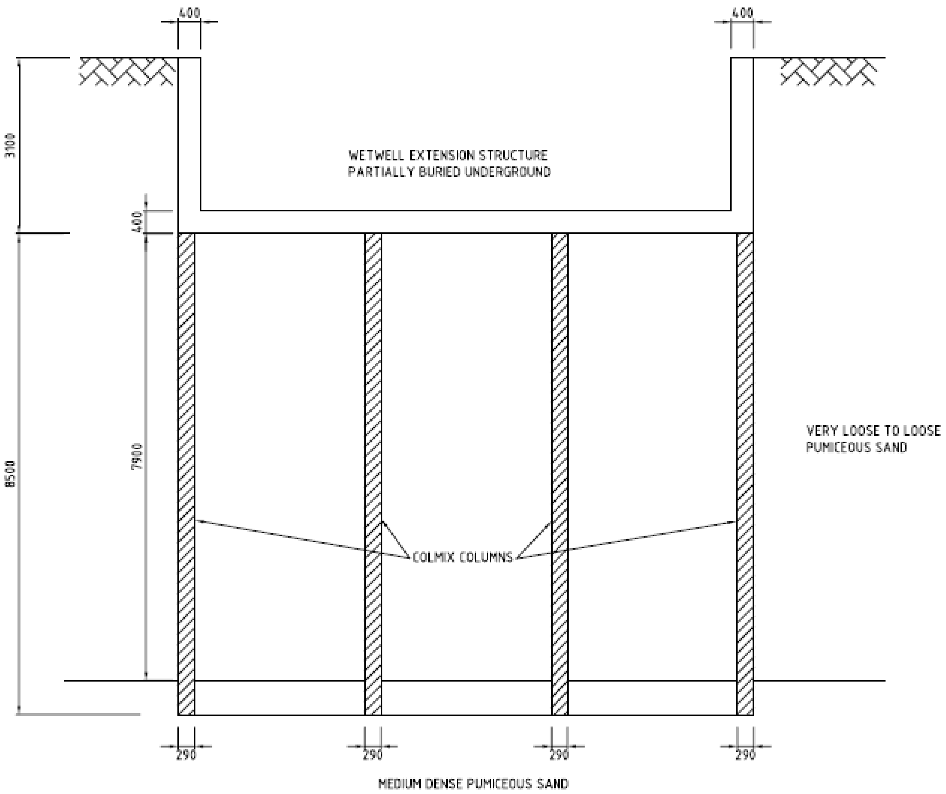

Figure 1: DSM columns and interpreted ground conditions

The pumiceous sand is very loose to loose down to 11 m depth, with Cone Penetrometer Test (CPT) cone resistance of 1 - 4 MPa. Medium dense sand generally occurs below 11 m depth, and the cone resistance increases to 2 – 10 MPa. Minor organic content was identified in some places. The site is on the fringe of a tidal estuary. Groundwater levels vary with tidal conditions.

2.4 Liquefaction Potential Assessment

Liquefaction could result in lateral spreading of the liquefied soil towards the estuary. The lateral spreading will cause permanent ground deformation, and undermine the plant facility operations.

A CPT based assessment (Youd et al. 1997) suggests that there is a liquefaction potential in the very loose to loose sand and silt from the ground level to about 11 m depth, for the design peak ground acceleration of 0.18 g. Therefore ground improvement works were carried out from the base of the wetwell structure to 11.6 m below the original ground level (Figure 1). The conventional CPT based assessment could be somewhat conservative. Wesley et al. (1998) suggests that CPT's tend to underestimate the effect of confining pressures in pumiceous sand.

3 REMEDIAL OPTION

Deep soil mixing (DSM) was identified as the preferred ground improvement option. In a DSM process, the soil is firstly drilled and loosened with a series of augers. As the augers are advanced through the ground, the soil is mixed with a binder, usually cement or lime, which reacts with the soil to improve the soil characteristics. Once the target mixing depth is reached, the augers are reversed and withdrawn. This process compacts the soil-grout mixture.

The DSM method was adopted for the following reasons:

- The existing facilities significantly limited available working space. The excavator-mounted rig used was capable of working with difficult site constraints.
- The treated area is in close proximity to existing wastewater plant facilities. DSM generates negligible construction noise and vibration, compared to other methods such as stone columns and vibro compaction.
- Liquefiable material extended to some 11 m depth. Shallow treatment methods such as dynamic compaction and replacement would not feasible.
- The in-situ estuary and airfall deposits are reported to be sensitive in some places, and disturbance could have had a detrimental effect.

4 DESIGN CONSIDERATIONS

The project specification included the following design criteria:

- A minimum replacement ratio of 27.7% in the raft footprint is to be provided.
- Minimum column length is 8.50 m measured from the raft base level (i.e. 3.1 m below existing ground level).
- Maximum allowable settlement under static conditions is 10.5mm within the mass stabilised block from a uniform net loading pressure of 55.7kPa.
- Minimum UCS of DSM column is 2.5MPa.
- DSM columns prevent liquefaction potential by carrying shear stresses during the design earthquake.
- A cellular DSM layout was specified to confine the liquefiable soil (Figure 2). The columns are to overlap to form continuous cell walls.

Figure 2 shows the layout of the DSM columns, which was developed to meet the above design criteria. The replacement of the treated area is 29.2%.

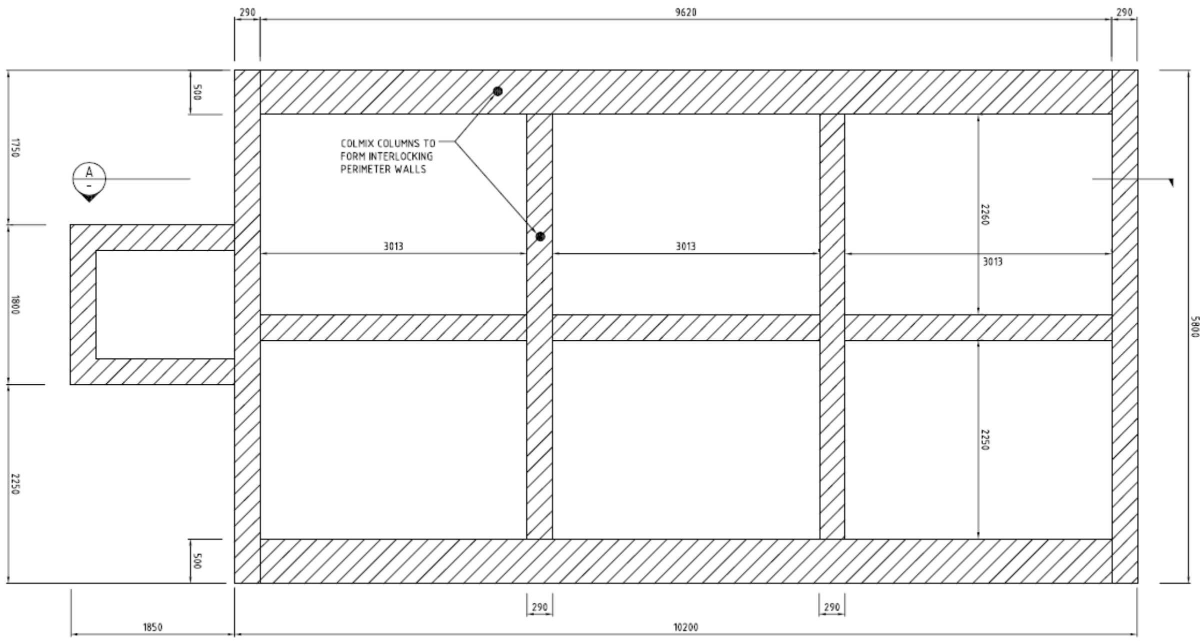

FIGURE 2: Cellular DSM layout.

5 ANALYSES

5.1 Cellular configuration

Cellular or lattice type DSM improves the shear resistance of the ground, and thus mitigates the liquefaction potential of a liquefiable site (Porbaha et al. 1998). The cellular type DSM was designed in two parts; external stability and seismic shear resistance.

5.2 Stability analysis

The external stability of the improved soil mass was checked using limit equilibrium analysis of the seismic and hydrodynamic forces acting on the cellular DSM body, which was assumed to be a rigid structure. The factors of safety for sliding and overturning mechanisms were computed. The seismic stability of the improved soil body under the design seismic loading was analysed for two cases. The first case assumes no liquefaction occurs in the surrounding soil whereas the second case allows for the loss of soil strength due to liquefaction. The results are summarised in Table 1.

Table 1: External stability of DSM treated block

No Liquefaction		Liquefaction	
FOS against overturning	FOS against sliding	FOS against overturning	FOS against sliding
2.9	4.5	1.8	1.3

5.3 Dynamic analysis

The performance of the cellular DSM structure in terms of liquefaction mitigation was evaluated using a 2D finite element analysis. The effectiveness of the ground treatment design was measured by comparing the cyclic shear stresses prior to and after the DSM installation. The liquefaction potential was evaluated using the Modified Robertson method (Youd et al. 1997). This method calculates the cyclic resistance ratio (CRR) based on CPT test data and compares it with the cyclic stress ratio (CSR) induced in the soil during a seismic event. The induced shear stress was simulated using the finite element model. A summary of the liquefaction potential before and after ground improvement is shown in Table 2. The DSM columns significantly reduced maximum shear stresses.

Table 2: Liquefaction potential – Unimproved and improved ground

Depth below EGL (m)	Baseline Conditions – no ground treatment		DSM Ground Treatment	
	Max Shear Stress (kPa)	FOS against liquefaction (1)	Max Shear Stress (kPa) (2)	FOS against liquefaction (1)
4	10.4	0.75	3.3	2.28
6	14.3	0.75	6.6	1.64
8	18.9	0.68	7.9	1.62
10	26.0	1.34	7.9	4.40

Note (1) Factor of safety = CSR/CRR.
(2) Maximum shear stress induced in in-situ soil confined by soil-cement walls.

6 CONSTRUCTION

A total of 159 columns were constructed at the Chapel Street site in 9 working days. The DSM columns were designed as continuous cells to confine the liquefiable soil. Overlapping of columns was one of the key issues. Overall, the overlapping of the DSM columns was successful, providing continuous cell walls. The Project Engineer, MWH, identified several discontinuities in the top 2 – 3 m of the completed cell walls. Additional columns were installed to close discontinuities, but at two locations approximately 100 mm wide gaps remained. An additional finite element analysis was carried out to assess the effect of the discontinuities. The results indicated the effects of these discontinuities were inconsequential to the overall performance of the structure and met with the client's prescribed performance criteria.

7 QUALITY CONTROL TESTING

A relatively high compressive strength of 2.5 MPa for the columns was specified. There was also uncertainty as to the quality of deep soil mixing columns constructed in pumiceous loose alluvial and ash deposits, which could be sensitive to disturbance. Furthermore the DSM columns were constructed in high tidal groundwater conditions. A rigorous testing regime was implemented at Chapel Street to ensure that the columns achieved the required quality. The results of 7 day unconfined compressive strength (UCS) testing of grout soil mixture are shown in Table 3.

Table 3: 7-day unconfined compressive strength results

Sample ID	Water Content (%)	Dry Density (t/m3)	Max Stress (kPa)	Strain at Failure (%)	Young's Modulus (MPa)
A	48.2	1.09	2626	0.5	928
B	38.5	1.23	5291	0.7	1521
C	32.4	1.34	5072	0.7	1343
D	47.2	1.11	3228	0.7	744
E	15.7	1.37	1990	0.7	453
F	13.5	1.53	2864	0.6	663
G	36.3	1.25	4342	0.7	1140
H	39.2	1.23	3901	0.6	1311

The wet grab samples of the grout-soil mixture achieved strengths significantly higher than the design values at a curing time of only 7 days. One of the samples did not achieve the design strength after 7 days, but after the nominal 28 day curing period the design strength was exceeded.

A second test involved a series of cored samples obtained from three cured columns at varying depths. The density and strength of the cored samples are recorded in Table 4, where nearly all of the cores achieved a compressive strength that was at least double the design value when crushed at 21 days. This is indicative of a significant strength gain between 7 day and 21 day curing periods. This also suggests that the 7 day UCS underestimates the long term strength of the DSM if used as a quality assessment parameter. The long term DSM column strength should be estimated from 21 day or preferably after an even longer period UCS.

Table 4: 21 day unconfined compressive strength of cored samples

Column	Depth (m)	Density (kg/m3)	Strength (MPa)
A	1.0	1840	4.5
A	1.5	1710	4.0
B	0.5 a	1730	11.5
B	0.5 b	1720	10.0
B	1.0	1730	8.0
B	1.5	1780	9.5
D	0.5	1750	6.0

8 CONCLUSIONS

DSM columns were constructed in a pumiceous sand site to mitigate liquefaction potential. The stability and effectiveness of the cellular DSM were demonstrated using both limit equilibrium and finite element analyses.

The DSM columns were successfully constructed, providing continuous overlapping of columns and encapsulating each cell. One of the key conclusions is the exceptional quality of the DSM columns that were constructed in loose, pumiceous and potentially sensitive alluvial and airfall deposits. The laboratory testing of DSM samples indicated that the strength of the constructed columns exceeded the design value. The quality of the DSM columns was good despite the high tidal groundwater conditions.

9 ACKNOWLEDGEMENT

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REFERENCES

Berrill, J.B., Christensen, S.A., Keenan, R.P., Okada, W. and Pettinga, J.R. (2001). *Case study of lateral spreading forces on a piled foundation.* Geotechnique, Vol. GE51, Issue 06.

Healy, J., Schofield, J.C., Thompson, B.N., (1974). *Geological map of New Zealand, Sheet 5 Rotorua*, NZ Geological Survey.

Japanese Geotechnical Society (1998). *Remeidal Measures against soil liquefaction - From investigation and design to implementation.* A.A. Balkema Publishers.

Porbaha,A., Tanaka, H., Kobayashi, M. (1998*). State of the art in deep soil mixing technology: part II. Applications.* Ground Improvement (1998) 2, 125 – 139.

Porbaha, A., Zen K. and Kobayashi, M. (1999). "Deep mixing technology for liquefaction mitigation." J Infrastructure Systems, Vol. 5(1), ASCE, 21 - 33.

Siddharthan, R.V., El-Desouky, M. and Porbaha, A. (2004). "Simplified Procedure for Seismic Analysis of Sites with Deep Soil Mixing." Geosupport 2004, ASCE, 862 - 874.

Wesley, L.D., Meyer, V., Pender, M.J. (1998). *Cone penetrometer tests in pumice sand.* New Zealand Geomechanics News, June 1998.

Youd, T.L. et al. (2001). *Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/ NSF workshops on evaluation of liquefaction resistance of soils.* J Geotech. and Geoenvironemental Engrg., Vol. 127(10), ASCE, 817 - 833.

Youd, T.L., Idriss, I.M. (1997). *Proceedings of the NCEER workshop on evaluation of liquefaction resistance of soils*, NCEER.