

Ground Improvement to Reduce Geotechnical Risk: Waingake Final Water Treatment Plant

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SUMMARY Site Investigations for the Waingake Final Water Treatment Plant near Gisborne in New Zealand identified poor ground conditions under the proposed retaining walls which also carry the treatment plant buildings. Due to the poor ground conditions, there was a potential for damaging settlements and foundation failure. The risk was exacerbated by the potential for strong earthquakes in the region. Ground improvement using stone columns provided a cost effective solution to mitigate the risk of damaging settlements and foundation failure. This solution also allows the retaining wall to undergo small translational movements during large seismic events with minimal disruption to the treatment facility. The stone columns were constructed using a dry compaction technique. Settlement monitoring confirmed the effectiveness of the stone columns in reducing settlements.

1. INTRODUCTION

After the 1988 Cyclone Bola, an interim treatment plant was built at Waingake to reinstate the water supply to Gisborne City in New Zealand. A final water treatment plant was subsequently designed and constructed at the Waingake site, about 50 km southwest of Gisborne City.

The final treatment plant involves the construction of additional filter tanks, a bulk lime silo, chemical dosing buildings, stores, generator and an operations building, see Figure 1. Retaining walls support the operations building, and the excavation behind the filter tanks.

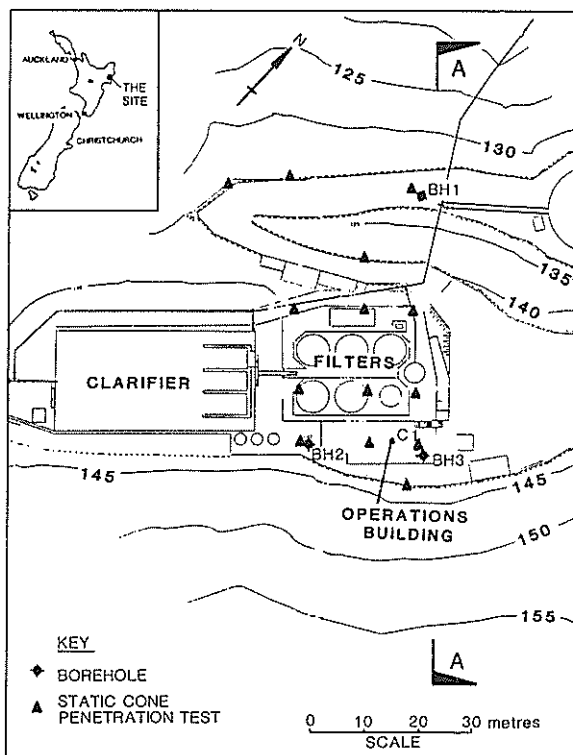


Figure 1 Location Plan

The treatment plant is located on a hillside with an average slope of 20 degrees. At the treatment plant site, colluvium overlies Miocene age massive, sandy to muddy, light blue grey siltstones, occasionally with sandstone beds. The geomorphology of the site together with information from site investigations suggests that past instability of the hillside has resulted in a thick layer of colluvium in a bedrock depression at the site, which is located between bedrock spurs.

The interim treatment plant was located partly on the bedrock spurs, and partly on the intervening colluvium, while the new construction for the final treatment plant is entirely on colluvium.

2. GROUND CONDITIONS

Site investigations comprising boreholes and Static Cone Penetration Tests (CPT) were carried out at the treatment plant site. The site investigations showed the colluvium underlying the site to be up to 25 m thick, and generally comprising soft to firm clayey silt with siltstone / sandstone clasts and some pockets of silty sand. The colluvium was found to be underlain by siltstone and sandstone bedrock. The static cone penetration tests showed the thickness and shear strength of the colluvium to vary across the site. A typical sketch geological section across the site is shown on Figure 2. The cone resistances (q_c) from the cone penetration tests were variable, but were generally less than 1 MPa up to a depth of 6 m to 10 m. Typical results from a static cone penetration test are shown on Figure 3, which illustrates the poor ground conditions under the retaining wall supporting the operations building.

A design undrained shear strength (S_u) of 30 kPa, and a coefficient of volume compressibility (m_v) of 0.34 m^2/MN to 0.13 m^2/MN is assessed from the CPT results, for the colluvium underlying the site. The groundwater level was within 2 m of the ground surface.

3. DESIGN PHILOSOPHY

The stability of the retaining wall supporting the operations building is critical to the final treatment plant. There was assessed to be an unacceptably high risk of failure or significant damage to retaining walls constructed with shallow foundations on existing ground, due to :

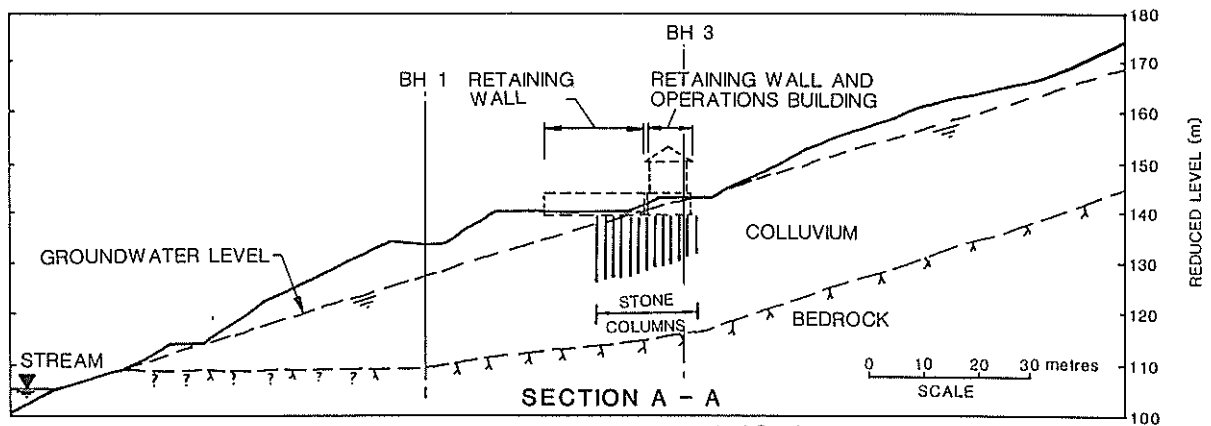


Figure 2 Typical Sketch Geological Section

- ▶ an inadequate margin of safety against bearing capacity failure under design seismic events ;
- ▶ an inadequate margin of safety against sliding failure of the retaining wall under design seismic events. The poor ground conditions and the large seismic inertia force due to the building supported by the retaining wall contribute to this instability ;
- ▶ total consolidation settlements up to 125 mm. Due to the variation of the thickness and compressibility of the colluvium across the site, the estimated differential settlements are of the order of 100 mm, and can cause significant damage.
- ▶ reduce total and differential settlements. Since the stone columns have a high permeability, they would also accelerate the consolidation settlement by providing drainage in a similar manner to sand drains ;
- ▶ accommodate small movements of the slope during extreme events, and
- ▶ allow translational displacement of the retaining wall during large seismic events, thus eliminating the need for a costly design to resist the full lateral seismic forces.

Therefore, ground improvement using a stone column foundation solution was adopted for the retaining walls at Waingake.

Deep pile foundations for the retaining walls were discounted for the following reasons :

- ▶ The piles would have had to extend to bedrock at a depth of 20 m to 25 m giving an expensive solution ;
- ▶ In view of the large lateral seismic forces on the retaining wall, the wall may be designed economically, by allowing the retaining wall to undergo small lateral displacements during large seismic events. However, if pile foundations are chosen, displacement of the wall cannot be allowed for in design, as such displacements would cause shearing at the pile-wall connection. Therefore, ground anchors would have been required to resist the lateral loads. Given the poor ground conditions and the large distance to bedrock, the ground anchors would need to be uneconomically long to achieve the required capacity;
- ▶ The slope was assessed to have marginal stability, and any small movements of the slope may impose large lateral loads on the piles, and possibly lead to failure.

Ground improvement using "stone columns" gave an acceptable foundation solution at a reasonable cost. "Stone Columns" are constructed using granular material, through poor ground to a suitable firm stratum. The stone columns were suitable for the Waingake site as they would :

- ▶ increase the bearing capacity of the ground ;
- ▶ improve the frictional resistance to sliding of the retaining wall along the wall base-ground interface ;
- ▶ improve the stability against shallow local slope failures ;

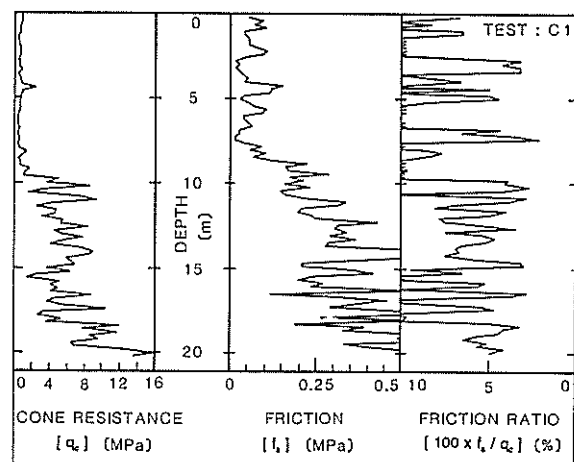


Figure 3 Typical Cone Penetration Test Results

4. RETAINING WALL DESIGN

Various forms of retaining walls were considered for the retaining wall supporting the operations building. To limit the bearing pressures under the retaining wall and provide sufficient resistance against sliding, a rigid reinforced concrete box type retaining structure with a large base slab and counterfort walls was adopted. The retaining wall box structure was designed to be partially filled with soil. Allowance was made in the design to permit small controlled translational movements of the retaining wall during large earthquake events without causing significant disruption to the treatment plant. Calculations indicated that displacements of the order of 60 mm would occur during a design seismic event with a peak ground acceleration of 0.32 g.

5. STONE COLUMN DESIGN

The ultimate load capacity of stone columns was calculated in accordance with the method proposed by Hughes et al (1975). The stress distribution between the granular columns and the soil, and hence the settlement reduction due to construction of the stone columns was derived based on elastic theory (Mitchell, 1981).

Based on these calculations, a design configuration of 1.0 m nominal diameter stone columns at a triangular centre to centre spacing of 1.8 m was chosen. The stone columns were designed to extend between 5 m and 12 m through the soft deposits to a stiffer stratum within the colluvium. The chosen stone column configuration gave a settlement reduction ratio (β) of 0.47, where β is the ratio between the calculated settlements with and without the stone columns. However, since the stone columns do not extend through the full thickness of compressible soils, the overall settlement reduction ratio is smaller than estimated above. The calculations indicated that the consolidation settlements with ground improvement using stone columns would be up to 70 mm. For a flexible foundation, differential settlements would be of the order of 60 mm. However, the retaining wall foundations are rigid, and therefore the actual differential settlements would be much smaller than the 60 mm calculated for flexible foundations.

The stone column layout chosen for the Waingake site is shown on Figure 4. One row of stone columns extends beyond the foundation base for the retaining walls. These columns were sealed at the surface with bentonite-cement grout to prevent ingress of surface water, to avoid any possibility of an increase in the groundwater pressures which could reduce the stability of the overall slope.

6. SLOPE STABILISATION MEASURES

Slope stability analyses indicated that the slope on which the treatment plant is located is marginally stable. There is a potential for slope instability with a failure surface within the colluvium or along the colluvium-bedrock interface, especially following periods of high rainfall.

In view of the assessed marginal slope stability of the Waingake site, and the risk of slope movements following periods of high rainfall, it was considered prudent to undertake remedial measures to improve the stability of the site.

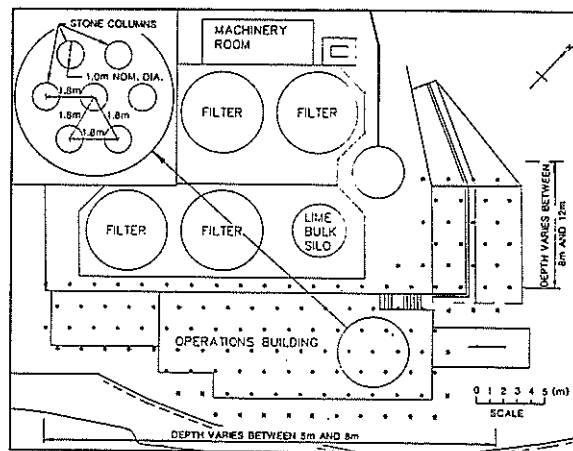


Figure 4 Layout of Stone Columns

Slope stability analyses indicated that lowering the high groundwater levels at the site would provide reasonable improvements in the stability of the slope at a moderate cost. Therefore, surface drains and sub-horizontal drainage holes were incorporated into the scheme to reduce or control the groundwater levels.

7. CONSTRUCTION TECHNIQUES

7.1 Stone Columns

In view of the potential instability of the site, the wet vibroflot process commonly used in Europe to construct stone columns was considered unsuitable for the Waingake site. The wet vibroflot technique would inject large quantities of water into the ground, and may reduce the factor of safety of the marginally stable slope. This consideration coupled with the general unavailability of vibroflot equipment in New Zealand led to the specification of a dry compaction method of stone column construction. To the author's knowledge, this is the first time such a technique has been used in New Zealand. Similar techniques have been used in countries such as India (Datye, 1982) and Australia (Waterton and Foulsham, 1984).

The method of construction adopted is shown in Figure 5. A 920 mm diameter hole to the required depth was drilled using an auger, and cased with a 910 mm outer diameter steel casing.

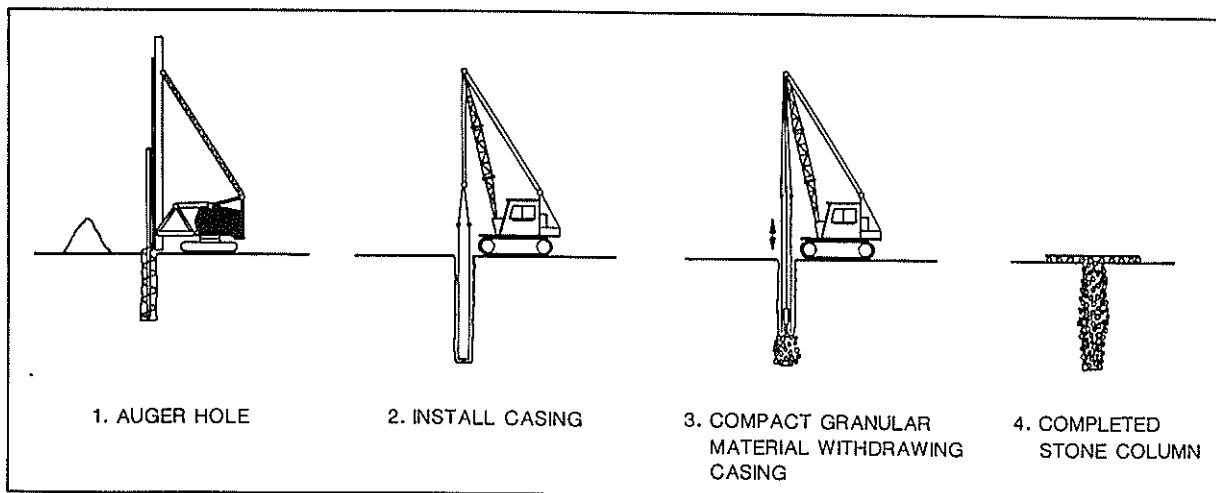


Figure 5 Construction of Stone Columns

Granular material was placed in the cased hole in small lifts, and the casing was withdrawn maintaining the bottom of the casing below the surface of the granular material. The granular material was compacted using a 2000 kg hammer falling through a height of 1.5 m, until a total set of 15 mm for 10 drops of the hammer was achieved. The procedure was repeated till a compacted column is formed to the ground surface. The stone columns were constructed using conventional piling equipment.

7.2 Drainage Measures

Sub-horizontal drainage holes of 86 mm diameter were drilled and lined with 40 mm internal diameter, partially slotted PVC lining tubes. Water flow was observed from most of the drains, and significant flow from some drains, indicating that the sub-horizontal drains were effective in reducing the groundwater pressures in the slope.

Surface drains were installed at the head of the marginally stable slope, and immediately above the treatment plant area, to minimise the surface water run-off entering the slope.

8. MONITORING

A programme of settlement and movement monitoring was implemented to monitor the settlement of the treatment plant structures, and to detect any movement of the slope. Settlement pins were also installed on the existing interim treatment plant structures, to detect any settlement due to lowering of the groundwater levels as a result of the slope drainage measures. Standpipe piezometers were installed to monitor the groundwater levels.

The initial settlement monitoring results indicate very small settlements of the retaining walls founded on stone columns, indicating that the ground improvement has been very effective in reducing settlements.

9. CONCLUSIONS

Site investigations comprising boreholes and static cone penetration tests showed poor ground conditions at the Waingake Treatment Plant site. A geotechnical appraisal identified potential foundation problems and the marginal slope stability of the site.

Shallow foundations for the retaining wall presented a high risk of significant damage due to large differential settlements, and bearing capacity or sliding failure of the walls during design seismic events. Conventional pile foundations together with ground anchors would have given an expensive solution with a risk of pile failure due to small slope movements.

Ground improvement of the soils under the retaining walls was carried out using stone columns to minimise the risk of foundation failure at an acceptable cost. The stone column solution would allow controlled movement of the retaining walls during large seismic events, and would accommodate small slope movements. The stone columns were constructed using a dry compaction technique with conventional piling machinery available in New Zealand.

Surface drains and sub-horizontal drainage holes were incorporated in the scheme to control the groundwater levels in the slope and mitigate the risk of slope failure.

The monitoring programme has shown that the stone columns were effective in reducing the settlement of the retaining walls.

This case study demonstrates the usefulness of site investigations and geotechnical engineering to identify the geotechnical and seismic risks, and to develop appropriate and cost effective solutions to mitigate the risks.

10. ACKNOWLEDGEMENTS

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