GEOGRID REINFORCED LIGHT WEIGHT EMBANKMENT ON STONE COLUMNS

Ka-Ching Cheung
Connell Wagner Ltd, Auckland
New Zealand

SUMMARY

As part of the South Eastern Arterial Road project in Auckland, a 2m to 4m high geogrid reinforced embankment with steep side slopes was built as an alternative to a section of the bridge structure to span the soft ground for the Carbine Road section of the route. The embankment is underlain by soft ground and is in close proximity to three adjacent structures. Stone columns were installed along the edges of the embankment to strengthen its foundations and reduce the influence of ground movement on the adjacent buildings. Light weight sand fill was used in the embankment to reduce the ground settlement. Extensive stone column loading tests and ground monitoring were undertaken. Ground level measurements confirmed that the design successfully eliminated the influence of the embankment settlement on the adjacent buildings.

1 INTRODUCTION

Auckland City, the Principal, called for design-build tenders for the South Eastern Arterial Road project. Connell Wagner was engaged by Downer Construction Ltd as its design consultant for the two major bridge structures; the 475m long Sylvia Park Viaduct and the Southern Motorway Underpass. In design-build projects, designers are required to develop cost effective design solutions to assist the contractor to win the contract. At the eastern end of the Sylvia Park Viaduct, Connell Wagner proposed to strengthen the soft ground with stone columns and construct a geogrid reinforced embankment as an alternative to using the bridge to span the soft ground. The proposal shortened the viaduct structure by two spans and resulted in substantially saving in the construction cost of the route. Three existing structures are situated between 2m and 10m from the edges of the embankment.

The embankment makes use of the following techniques (i) four rows of stone columns around the perimeter of the embankment to strengthen the ground, (ii) light weight sand fill to reduce the imposed load and hence ground settlement, (iii) reinforcement of the 1 :4(horizontal:vertical) slopes with geogrids (iv) geotextile basal reinforcement to increase the short term stability and the seismic resistance of the embankment and (v) wick drains within the central unreinforced zone of the embankment to increase the consolidation rate. A high frequency vertical vibration compaction technique was developed for compacting the stone placed in the pre~bored stone column holes to minimise the installation effects on nearby structures. This paper presents the design philosophy, methodology, requirements and monitoring results for the embankment.

2 GROUND CONDITIONS

The site is underlain by a 22m thick layer of alluvium which slightly increases in strength with depth, and is interbedded with a 2m to 3m thick layer of basaltic lava at a depth of 6m to 7m below the ground surface level. The ground surface layer is a 15m thick firm to stiff volcanic ash. Below the ash layer and above the basalt material, a 4m to 5m thick layer of soft to firm alluvial clayey silt/silty clay is present. The upper alluvial layers are generally under-consolidated and occasionally thinly interbedded with peat and sand layers. Typical vane shear strengths of 20kPa to 35kPa were measured in the clayey silt/silty clay layer. Cone penetration tests (mechanical cone) indicated the cone tip resistance is generally below 0.6MPa in this material. The groundwater level is close to the ground surface during the winter.

Below the basalt layer there is 10m to 12m of firm to stiff alluvial silty clay with typical standard penetration test N values in the order of 5 but values of 10 to 15 are occasional encountered. Consolidation test results generally indicated that this lower silty clay layer is over-consolidated with an over-consolidation ratio of 3-4. The alluvium is underlain by the so-called Waitemata formation comprising interbedded layers of mudstone, siltstone and sandstone. The weathering of the Waitemata formation decreases rapidly with depth.

3 DESIGN PHILOSOPHY

It was estimated that the ultimate bearing capacity of the upper alluvium under the 23m wide embankment is in the order of 100kPa to 150kPa. However, analyses showed that there was an insufficient margin of safety against rotational failure for the proposed 1:4(Horizontal:Vertical) side slopes for an embankment higher than 25m. Because the lower silty clay layer is over-consolidated the induced settlement of this layer is likely to be small provided the embankment load transferred to it is less than its pre-consolidation pressure. Conservative analyses showed that the settlement of this layer would be less than 20mm for an embankment height of less than 4m. In consequence strengthening the upper layer would be sufficient to ensure the stability of the embankment and eliminate the influence of the embankment settlement on the adjacent structures. A typical cross section of the embankment is shown in Figure 1. Based on these considerations, the combination of the following techniques were employed to improve the performance of the embankment.

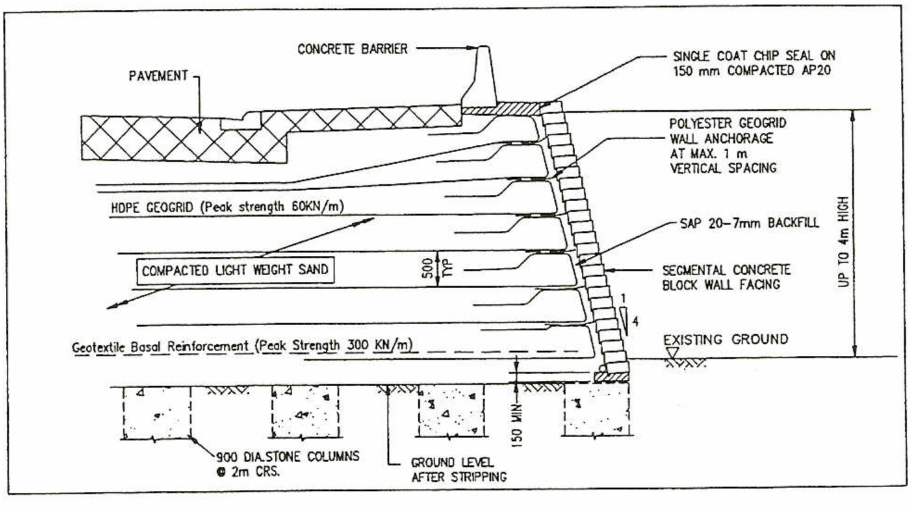

FIGURE 1 Typical Cross Section of Embankment

Embankment side slope geogrid reinforcement - The width available for embankment was tightly constrained by the site boundaries necessitating the use of 1:4 batter slopes to the embankment. The internal stability of the embankment could be ensured by using HDPE geogrid reinforcement with a peak tensile strength of over 55kN/m at a vertical spacing of 500mm. Segmental concrete block wall (Keystone) facing was tied by high modulus polyester geogrid layers extending from the embankment for the protection of the HDPE geogrids. Because of the expected consolidation of the embankment during the construction stage, the segmental block wall facing was installed after completion of the consolidation settlement of the embankment.

Stone Columns and Basal Geotextile Reinforcement - The design requires that the stone columns on their own will ensure the side slope stability of the embankment after full consolidation. A layer of woven geotextile with a 300kN/m peak tensile strength was placed at the base of the embankment as basal reinforcement to ensure an acceptable margin of short term stability during the two stage construction with a minimum holding period between stages. The basal reinforcement layer will become redundant after consolidation under normal loading. Under extreme loading conditions such as during a seismic event, the basal geotextile reinforcement will come into effect again to maintain the stability of the embankment. Failure of stone columns under seismic loads is unlikely. It has been found that stone columns exhibit excellent capacity to resist horizontal shear in simulated seismic ground acceleration experiments, Engelhardt and Golding (1975).

Stone columns of 900mm diameter were arranged in four rows at 2m centres around the perimeter of the 2 to 4m high emban1anent No ground improvement was carried out where embankment height was less than 2m. Stone columns were also designed as vertical drains to shorten the time required for porewater pressure dissipation of the surrounding soil.

Temporary sheet pile walls - The stone columns and basal geotextile reinforcement were designed to reduce the ground deformation. Steel sheet pile walls were employed as a precautionary measure to isolate the three adjacent structures from the effects of the construction of the stone columns and the settlement of the embankment.

Wick Drains - In the central part of the embankment where stone columns are not installed porewater pressure may have built up under the embankment loading therefore wick drains were installed to reduce the time for the porewater pressure dissipation.

4 DESIGN REQUIREMENTS

Because of the limited construction time available for the entire project, it was necessary to design the embankment to have a satisfactory factor of safety during embankment construction which had the minimum holding periods between loading stages. In consequence the stone columns were designed to maintain the side slope stability with a minimum FOS> 1.3 for the short term stability and a FOS> 1.5 for the long term. The spacing and the diameters of the stone columns were designed to reduce the predicted ground settlement to negligible amounts at the adjacent structures. Basal geotextile reinforcement was found to be required if the FOS for the short term stability and the seismic resistance of the embankment were less than 1.3. The spacing of the stone columns and the wick drains was also designed to ensure that the 90% of the primary consolidation was complete within one year of imposing the fill load.

5 DESIGN METHODS

Design of Stone Column Group in Clay

For stone column groups in clay under an embankment, the stress conditions within the clay-stone columns are very different from that of a single column. Shear failure of individual columns is unlikely to occur and field observations and centrifuge testing by Osborne (1994) and Stewart and Fahey (1994) indicate that stone column groups performed satisfactorily under a 20m high iron ore stockpile and failure of individual columns was unlikely. However, when columns are placed under the side slopes of an embankment, gross bearing capacity failure is likely to be the critical condition. Slope stability analyses may be carried out using a conventional method considering the properties of the composite material of the foundation.

The "equivalent soil-stone column block" and "equivalent trench" approaches can be used for the modelling of soil-column foundations. The "equivalent soil-column block" method proposed by Almedia and Parry (1985) assumes that the reinforced zone can be represented by an equivalent soil block with strength properties based on the contribution of the foundation soil and the columns. The properties of the block are influenced by the relative stiffness of the in-situ soil and the column materials. The ratio of the vertical stress in the stone column σ_s to that in the clay σ_c is defined by the stress concentration ratio n as defined below:

$$n = \sigma_s / \sigma_c \tag{1}$$

Mitchell and Katti (1981) suggest that n is in the range of 2 to 6 and usually has a value of 3 to 4 for stone columns in clay. The replacement ratio as representing the area of the clay foundation replaced by a stone column is given by:

$$a_s = A_s / (A_s + A_c) \tag{2}$$

where A_s denotes the cross sectional area of the stone column and A_c the plan area of the clay foundation per column. Under an average applied pressure σ from the embankment, the stress taken by the clay foundation and stone column is given by:

$$\sigma_s = \beta_s \sigma \text{ and } \sigma_c = \beta_c \sigma \tag{3}$$

where $\beta_c = [1 + (n\text{-}l)\, a_s]^{-1}$ (4a)

and $\beta_s = n\, \beta_c$ (4b)

in which σ_s and σ_c represent the stresses taken up by the stone column and the clay foundation respectively. Based on the parameters obtained from equations (1) to (4), the equivalent block properties as given by Almedia and Parry (1985) are:

$$\tan \text{Ø}'_e = m \tan \text{Ø}'_s + (1 - m) \tan \text{Ø}'_c \quad (5a)$$

$$c'_e = (1 - m)\, c'_c \quad (5b)$$

where

$$m = a_s\, \beta_s \quad (6)$$

Ø' and c' denote the effective friction angle and cohesion, respectively. Subscripts, e, s and c denote the parameters for the equivalent clay-column block, stone column and clay foundation respectively.

In the equivalent trench method, Stewart (1995) proposed that the individual columns are defined in the slope stability analysis with their dimensions modified to convert the three dimensional layout to an equivalent two dimensional geometry. For example, the centre to centre spacing of the columns should remain the same, but the column width should be reduced to equivalent trenches in the analysis. The equivalent trench width is given by:

$$W = A_s / S \quad (7)$$

where S is the column spacing as above. To account for the stress concentration in the columns under the embankment load in the analysis, the friction angle of the stone columns is factored up by the β_s value in Eq. (4b) as given below:

$$\tan \text{Ø}'_w = \beta_s \tan \text{Ø}'_s \quad (8)$$

Settlement Prediction

The elastoplastic finite element method for the estimation of the settlement of stone column groups proposed by Balaam N P and Booker J R (1985) was used in the design. In the method, the elastic settlement of the stone columns was first calculated from a set of analytical equations. The estimated elastic settlement was then corrected using design charts to account for the plastic deformation of the column material for the calculation of the total settlement. The stiffness of the stone columns is dependent on that of the surrounding soil. Typically, published values of the Young's modulus for stone columns are in the range of 20 to 40 times that of the surrounding clay. The stone column load versus settlement relationship can be calculated from the method. A two dimensional finite different analysis using the computer program FLAC was also carried out to evaluate the ground deformation outside the embankment.

Stone Column Materials

Uniformly graded materials with a particle size in the range of 20mm to 60mm are typically used for stone columns however, the ineffectiveness of stone columns in providing drainage and reducing settlement because of clay infiltration from the surrounding soil into the stone column has previously been reported, Garga and Medeiros (1995). In consequence, in this project, gap graded minus 65mm greywacke crushed rock with less than 20% of material passing 4.75mm sieve was used. By adding some finer materials to reduce the open voids of the stone column material, the amount of clay infiltration into the stone columns was reduced. The grading was designed to ensure good drainage and mobilisation of the column strength after a relatively small settlement.

Consolidation Time

The stone columns installed under the edges of the embankment served as vertical drains to accelerate the consolidation process. The coefficient of consolidation of the soil has been assumed to be in the range of 1 to 3 m^2/year. Based on the analytic solution proposed by Barron (1948) for radial drainage, the time required for the soil around the stone columns to reach 90% of consolidation would have been in the range of 6 months to 1 years after applying the fill load.

Embankment Stability and Basal Geotextile Reinforcement

Analyses of both the internal and external stability of the embankment supported on the stone column reinforced foundation was carried out using the slope stability analysis program SLOPE. Geogrid reinforcement for maintaining the stability of the side slopes (internal stability) and basal geotextile reinforcement were modelled. Both equivalent soil-column composite block and equivalent trench methods were used for comparison in the design.

6 DESIGN PARAMETERS

In the analyses, the following material parameters were used:

Embankment

Density of light weight sand fill	= 14kN/m^3
Ø' of sand fill	=30°
Surcharge on top of embankment	= l0kPa

Clay Foundation

Undrained shear strength of clay	= 20kPa
Young's Modulus	= 1.5MPa
Ø' of clay	= 18°

Woven Basal Geotextile Reinforcement

Peak tensile strength	= 300kN/m

Stone Column

Column Diameter	=0.9m
Column Spacing	=2.0m
Ø' of column material	= 35°
Stress concentration ratio, n	=3

HDPE Geogrid Side Slope Reinforcement

Peak tensile strength	=60kN/m
Long term design strength	=18kN/m

7 CONSTRUCTION

The stone columns were installed in 900mm diameter pre-bored holes taken down to the basalt rock at about 6m depth. After boring the hole a steel casing was lowered into it to maintain the stability of the hole walls. The stone column material was poured into the hole in 1m lifts. The casing was then withdrawn to the top of the loose material. The material was compacted for four minutes under a 800mm diameter hammer head connected to a vibrating hammer above the ground via a steel I beam. The vibrating hammer had an eccentric moment of 0.05kN-m at 50Hz. It was found that the loose material was compacted to the range of 40-70% of its loose lift height. During the stone column construction, very little ground vibration was felt beyond about 10m from the column under construction.

The embankment was constructed in two stages. The first stage brought the fill height up to 2m high at a rate of 1m per week Embankment settlement and porewater pressure responses were monitored on a weekly basis. After a 1 month holding period, the embankment was constructed to its full height at the rate of 1m per week One year after the embankment construction was completed measurement showed that the embankment settlement was complete. The segmental concrete block wall facing was then constructed.

8 RESULTS

Plate bearing tests with up to 250kN of axial loads were carried out on 20 stone columns. Measured stone column settlements under the test load were generally less than 15mm. No indication of failure of stone column was observed. Back analysed Young's modulus values of the stone columns were in the order of 160MPa. This is about 100 times of that of the surrounding clay and is significantly higher than the published values of 20 to 40 times.

Measurements of settlement gauges installed in the unreinforced zone of the embankment indicated ground settlements of 100mm under 2m of the light weight fill. This was in good agreement with the predicted settlement. The embankment settlement rate became very small 6 months after construction. Within the stone column reinforced zone, ground settlements of 40mm to 70mm were measured under the 4m high embankment load. At a distance of about 2m outside the embankment area, no detectible ground deformation was recorded in the deformation surveys.

9 CONCLUSIONS

This paper presents the design philosophy, methodology, requirements and monitored results for a geogrid reinforced light weight sand embankment with steep side slopes on a stone column strengthened foundation. Uniformly graded 20 to 60mm gravels are commonly used for stone column construction however, because of clay infiltration into stone columns, ineffective drainage and settlement reductions were previously reported. To overcome these shortcomings, finer materials were used to reduce the open voids in the stone column material. Load testing of stone columns indicated that the stone column settlement was generally less than 15mm under a 250kN test load. Back calculated Young's modulus values for the stone columns were in the order of 160MPa and were about 100 times that of the surrounding clay.

The monitored results indicated that the stone columns successfully strengthened the soft clay site enabling the construction of an embankment of 2m to 4m high. It was founded that the embankment settlement had a negligible influence on the adjacent structures although the structures were within 2m to 10m from the edge of the embankment. Furthermore the adoption of a vibrating hammer to compact the stone columns virtually eliminated nuisance from vibration during the stone column construction.

10 REFERENCES

[1] Almedia M S S., and Parry R H G., (1985) Centrifuge studies of embankment foundations strengthened with granular columns", Third Int. Geot. Seminar, Soil Improvement Methods.

[2] Balaam N P and Booker J R (1985) "Effect of stone columns yield on settlement of rigid foundations in stabilisation clay", Int. J. Num. Anal. Meth. in Geomechanics, Vol. 9, pp.331-351.

[3] Barron R A (1948) "Consolidation of fine-grained soils by drain wells", Trans. ASCE Vol.113.

[4] Engelhardt, K. & Golding H. C. (1975) "Field Testing to evaluate stone column performance in a seismic area". Geotechnique.

[5] Garga V K and Medeiros LV (1995) "Field performance of the port of Sepetiba test fills", Can. Geotech. J. Vol.32, pp106-121.

[6] Hughes J MO., Withers, N. 1., and Greenwood, D. A. (1975) "A field trial of the reinforcing effect of a stone column in soil", Geotechnique Vol. 25, pp 31-44.

[7] Osborne T R (1994) "Design and construction of stone columns for Nelson Point project", Ground Improvement Seminar.

[8] Mitchell J K and Katti R K (1981) "Soil improvement state-of-the-art report. Proc 10 Int. Conf. SMFE, Stockholm Vol 4.

[9] Stewart D P and Fahey M (1994) "Centrifuge modelling of a stone column foundation system", Ground Improvement Seminar.

[10] Stewart D P (1995) Personal communications, University of Western Australia.