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Dynamic replacement - Liquefaction mitigation for Hamilton section of Waikato Expressway

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ABSTRACT: This paper describes the design and construction of dynamic replacement columns (DR) to mitigate potential liquefaction of a road embankment in the Hamilton section of the Waikato Expressway in New Zealand. Liquefiable, loose silty sands extend to depths of 6 m to 7.5 m, with some localized deeper layer in the Mangaharakeke gully which is the most challenging geotechnical area of the project. To derive a Value for Money solution, the ground improvement design was carried out using a resilience in design approach, based on a tolerable embankment deformation of 250 mm under the design earthquake event. The design includes stabilization and support of 14m batter slopes, fill embankments on loose ground which is subject to high potential liquefaction. Following clearing of vegetation, a historic landslide area was discovered in the natural slope above the embankment and a Safety in Design approach was implemented to appropriately address this specific issue. During construction, the DR columns were found to refuse in a medium dense sand layer which was underlain by further liquefiable layers. Remedial works had to be carried out using geotextiles and toe berms. Nevertheless, the overall outcome is still cost-effective and achieved the desirable objective of a value for money, resilient solution. The key design methodology of the DR ground improvement, construction issues and remedial works are described in this paper.

1 INTRODUCTION

The Hamilton Section of the Waikato Expressway is a 21km highway around the eastern fringe of Hamilton City. Towards the southern end of the project, a bridge is required to be constructed over the Mangaharakeke gully. There are two gully embankments (northern and southern) and this paper is mainly concerning the work performed at the northern embankment.

The southern bridge approach embankment is situated on the side of a hill which slopes down towards the east where a river training drainage channel has been constructed along the toe of the slope.

The overall northern embankment height is about 14 m from the road surface to the toe, with a berm at the toe of the slope which is 6.5 m high. The upslope section is founded over colluvium and one section of which was found to be a historic landslide area discovered during construction. The assessment of this upslope section is presented by Dai et al (Perth, 2019). The toe berm of the embankment is founded on a deep sequence of old alluvium that is over at least 100 m thick. The groundwater level is within 1 m to 2 m from the ground surface and the upper 6 m to 7.5 m with some localized deeper zones contains loose silty sands which are susceptible to liquefaction.

The high seismicity and ground conditions posed significant challenges for the project with respect to achieving a safe, yet cost-effective solution for both the New Zealand Transport Authority and the Contractor.

An initial concept design using a lattice structure constructed using continuous flight auger (CFA) concrete columns was considered by the Contractor to be too expensive and an alternative solution was needed.

This paper describes the value for money approach adopted to develop a ground improvement solution comprising partially penetrating Dynamic Replacement (DR) columns to mitigate the liquefaction risks to acceptable levels, including validations that took place.

2 DESIGN REQUIREMENTS & VALUE FOR MONEY DESIGN APPROACH

The project design earthquake requirements are summarized in Table 1.

Table 1. Project earthquake design requirements

Return Period	Magnitude	Peak Ground Acceleration
1:500 (SLS)	5.9	0.09 g
1:1000 yr (ULS)	5.9	0.24 g

Unlike the design of the embankment immediately adjacent to the bridge abutment (i.e. structure zone) which is not the subject of this paper), the consequence of embankment failure is considered to be non-catastrophic. Therefore, a resilient and sustainable design approach is adopted and according to the project requirement, an earthquake return period of 1:1000 yr is considered to be reasonable together with the following design criteria:

- FOS ≥ 1.5 under static long-term condition
- FOS ≥ 1.0 for 100% SLS earthquake inertia and pre-earthquake soil strength
- FOS ≥ 1.1 ULS with no earthquake inertia and post-liquefied/cyclic softened soil strength
- FOS ≥ 1.0 with 65% SLS earthquake inertia and post-earthquake soil strength
- allowable lateral deformation ≤ 250 mm at critical acceleration for one design earthquake event.

It is anticipated that the embankment will be repairable following the design earthquake event should such a low probability event actually occur.

3 GROUND IMPROVEMENT DESIGN

3.1 *Dynamic replacement technique*

Based on the design approach described above, it was decided to adopt Dynamic Replacement (DR) as the ground improvement solution.

DR is similar to stone columns, but its construction does not involve specialized and costly vibro-replacement machinery. DR comprises the repeat dropping of a heavy pounder at each column location which is progressively backfilled with gravel or rock to extend the columns into the ground. A schematic sketch of the DR installation process is shown in Figure 1.

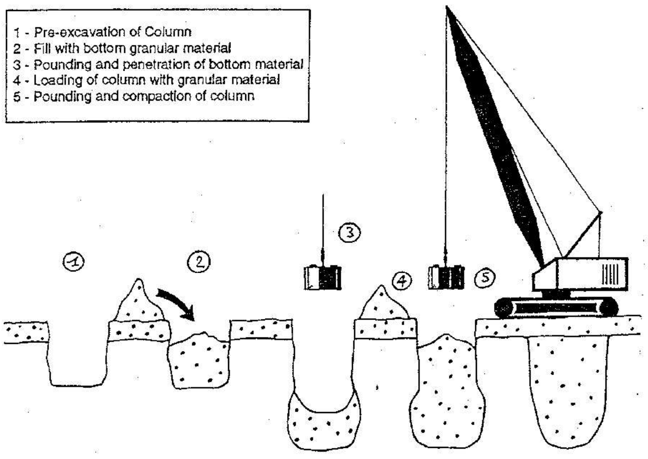

Figure 1. Typical DR installation procedure

The advantage of DR versus stone columns is that the equipment required has lower cost, and the columns produced have greater diameter so that for the same replacement ratio, larger spacing (therefore fewer columns) can be adopted to reduce construction time. The biggest disadvantage for DR, however, is its limitation on the depth of penetration which is typically 5 to 6 m maximum depending on the size and shape of the pounding, and soil type.

For this project, the liquefiable soils extend up to about 7.5 m depth. Pre-excavation below the ground surface to 1.5 m was planned to enable full penetration of the DR columns if a 6 m penetration depth is achieved. If 6 m penetration could not be achieved, it there is a potential that some liquefiable soil layers may remain below the bottom of DR columns. The design of the DR ground improvement is therefore subject to validation testing and design review during and following the DR work as described in Section 4 below.

3.2 *Beneficial effects of DR in mitigating liquefaction*

Similar to stone columns, DR has proven effective in improving the strength of soft or loose ground. The presence of granular columns in sandy soils that are susceptible to liquefaction are described below

3.2.1 *Increase cyclic resistance ratio (CRR):*

- Increase Soil Density (only applicable to granular soils);
- Increase Lateral Stress / Lateral Confinement of Soil (applicable to granular soils but may also cause some lateral stress increase with time following dissipation of excess pore pressures during DR column installation); and
- Provide Mechanism for Rapid Dissipation of Excess Pore Pressures (applicable for both fine and coarse grained soils). However, efficiency in fine grained soils may be reduced significantly due to smearing effects around the perimeter zone of the DR column).

3.2.2 *Decrease cyclic stress ratio (CSR):*

- Soil Reinforcement/Shear Stress Redistribution (applicable for both fine and coarse grained soils).
- The installation of DR significantly increases the relative density of surrounding loose granular soils, and hence reduce the potential for liquefaction. The installation of stone columns will also often increase the strength of silty sands and some fine grained soils, although several months may be required before the beneficial effect is observed, or when primary consolidation of the soft clay occurs after construction of the overlying embankment.

3.3 *Design methodology*

Technical report MCEER-06-009 prepared by Thevanayagam et al. (2006) for FHWA provides design guidelines for liquefaction remediation using stone columns. The method presented in that report considered some of the beneficial effects of stone columns discussed above and provided design charts to assess the improved $(N1)_{60cs}$ in relation to the permeability (k) of the soil, area replacement ratio (a_r) where $(N1)_{60cs}$ is the normalized SPT blow counts for clean sand. The chart for the case without wick drains for an area replacement ratio of 22.5% (highest reported area ratio) is reproduced in Figure 2.

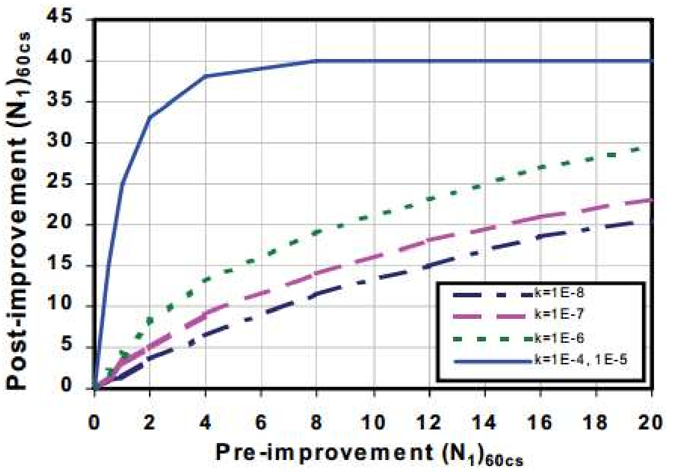

Figure 2. Pre and post improvement (N1) 60cs for a_r = 22.5% (after Thevanayagam et al, 2006)

The available investigation data indicate the SPT blow counts N for the soil layers varies between 0 and 7 with typical values being around 1. The permeability of the silty sand can be expected to be in the order of 1x10^{-5} m/s which from Figure 2, a pre-improved normalized SPT blow counts $(N1)_{60cs}$ of 1 could result in an improved $(N1)_{60cs}$ of around 25 for an a_r=22.5%.

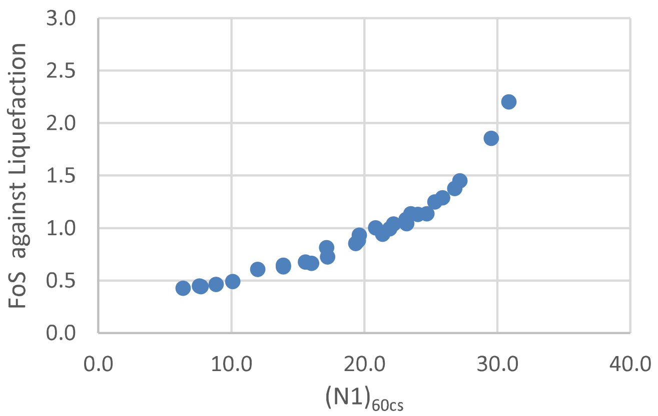

Figure 3. Assessed required SPT $(N1)_{60cs}$ value to achieve factor of safety of 1 against Liquefaction

The assessment as presented in Figure 3 indicates an improved $(N1)_{60cs}$ of about 22 would be sufficient to mitigate liquefaction risk in the silty sand layers.

However, the project site is more complex in that it contains soft clay layers interbedded with the loose silty sand layers. Therefore, whilst the above chart was used as a guide to the design, the actual design methodology adopted was by treating the DR columns as a partial ground replacement. It was assumed that the soil between the DR columns and away from the treated block will not be improved (i.e. they will still potentially undergo liquefaction/cyclic softening under the design earthquake conditions. In other words, the DR column reinforced soil block will have higher averaged strength and stiffness compared to the soil mass without any soil improvement. The design analysis carried out was based on the pseudo-static stability analysis approach, with averaged composite strengths to describe the soil and DR columns. Any strength increase of the soil between the DR columns was conservatively ignored. The DR design was generally carried out in accordance with the methodology presented by Barksdale and Bachus (1983), using the following procedure:

The stress carried by the columns, σ_{column}' and soil, σ_{soil}' were assessed based on the area ratio (a_r) and 'n' according to equations (1) to (4).

$$\sigma'_{column} = \mu_{col}\sigma_a \tag{1}$$

$$\sigma'_{soil} = \mu_{soil}\sigma_a \tag{2}$$

$$\mu_{soil} = \frac{1}{1+(n-1)a_r} \tag{3}$$

$$\mu_{col} = \frac{n}{1+(n-1)a_r} \tag{4}$$

where σ_a = applied stress from the embankment; μ_{col} and μ_{soil} = the stress ratios of the column and soil respectively; a_r = area replacement ratio, and n = the stress concentration factor.

The stress concentration factor, n, is dependent on the stiffness ratio of the soil and column, and whether yielding of the column could occur. If the soil is assumed to have very low stiffness following liquefaction, very high n values are calculated from conventional arching theory. However, observed n values in the field are typically limited to 2 to 5 for single columns used to support column loads. For column groups beneath embankments, the confinement of the soil around the columns reduces the limit of column yielding, and a maximum n value of up to 10 is recommended by Barksdale and Bachus (1983). In the design for this project, an initial n value was computed from arching theory depending on soil and column modulus ratios and column spacing then adjusted for column yielding depending on embankment height and confining pressure with depth. The maximum n value was limited to 10 thus the adopted range of n values in the design ranged from 2 to 10.

Figure 4 shows the mechanism governing the maximum load a stone column group could carry, q_{ult} where the limiting lateral yield stress σ_3 can be computed using Visic (1972) based on cavity expansion theory. The angle, β, is given by Equation (5) where

ϕ'_{eq} is the equivalent friction angle of the DR improved block as given by Equation (6):

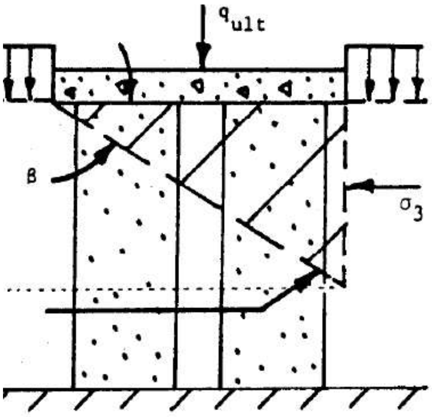

Figure 4. Mechanism of failure of stone column group

$$\beta = 45 + \frac{\emptyset'_{eq}}{2} \quad (5)$$

The equivalent strength properties of the DR block were derived as according to equations (6) and (7):

$$\phi'_{eq} = \tan^{-1}[\mu_{col} a_r \tan(\phi'_{col}) + \mu_{soil}(1-a_r) \tan(\phi'_{soil})] \quad (6)$$

$$c'_{eq} = c'_{col} a_r + c'_{soil}(1-a_r) \quad (7)$$

where ϕ'_{eq} = equivalent friction angle of DR block; c'_{eq} = equivalent cohesion of DR block; ϕ'_{col} = friction angle of DR column material; ϕ'_{soil} = Friction angle of soil; c'_{col} = cohesion of DR column material and c'_{soil} = cohesion of soil.

In order to consider the undrained behaviour of the liquefied/cyclic softened soil following a seismic event, the assessed residual strength τ/σ_o' was converted to an equivalent undrained shear strength (S_u) of soil based on the insitu effective stress of the soil without embankment loading, and the DR block equivalent strength parameters were assessed with c'_{soil} substituted by S_u.

The following parameters were adopted for the DR columns which comprised high strength minus 300 mm Greywacke rock:

- Unit weight, γ_{col} = 20 kN/m3
- Cohesion of DR column material, c_{col} = 0 kPa
- Effective friction angle, ϕ'_{col} = 42 degrees (after pounding)
- Young's Modulus of DR column material, E_{col} = 50 MPa (after pounding)

Based on the above assessment procedure, adopted DR column parameters, and soil parameters assessed from the geotechnical investigations, the required area replacement ratio was assessed to be 0.25. For robustness of design with allowance for some uncertainties in the application of the method for seismic design, an a_r ratio of 0.35 was adopted for design, using an initial DR diameter of 2.5 m at 4.0 m centres arranged in a triangular grid pattern. The assessed equivalent DR improved soil block parameters are summarised in Tables 2 and 3 for the static and earthquake cases, respectively.

Table 2. Equivalent parameters (static stability)

Soil Unit	n	γ_{eq} (kN/m^3)	E_{eq}' (MPa)	c_{eq}' (kPa)	ϕ_{eq}' (degrees)
Soft Silty Clay	10	17	4.1	0	39.6
Loose Silty Sand	2	17	7.8	0	36.4
Loose Silty Sand	3	17.8	7.8	2.3	36.4

Table 3. Equivalent parameters (seismic stability)

Soil Unit	n	γ_{eq} (kN/m^3)	E_{eq}' (MPa)	$c_{eq}'(S_u)$ (kPa)	ϕ_{eq}' (degrees)
Soft Silty Clay	9	17	3.2	1.5	34.0
Loose Silty Sand	6.8	17	3.2	0.4	32.0
Loose Silty Sand	10	17.8	3.2	3.8	34.7

The equivalent static stability parameters shown in Table 3 are long-term drained parameters, whereas the equivalent seismic parameters include the undrained behaviour of the liquefied soils. In both cases, the influence of the DR columns dominates the equivalent friction angle of the DR block.

Based on these equivalent parameters, the design criteria stated in Section 2 were achieved.

4 VIBRATION ISSUES

The assessment of potential vibration to nearby residence is an important element to the choice of equipment and success of the proposed DR ground improvement. The project specification requires the peak particle velocity inside the buildings to be less than 5 mm/sec.

An initial assessment was made on the required energy for the DR using Equation (8).

$$D = 0.4 \sqrt{(WH)} \quad (8)$$

where D = effective penetration depth, W = weight of pounder and H = height of drop.

To achieve a depth of penetration of 6 m, the required energy (WH) is assessed to be 225 tonne.metres or a compaction energy W_o of 2.25 x 10^6 Joules.

Hackney (2011) provides a relationship of peak particle velocity and energy potentially imposed by DR as shown in Equation (9):

$$ppv = 0.18 (\sqrt{W_o})/S \text{ (mm/sec)} \quad (9)$$

where ppv = peak particle velocity, W_o = energy of compaction in Joules, and S = distance to the point of impact

The nearest residential properties are located about 50 m from the site and using the above equation, a potential ppv of 5.4 mm/sec was assessed. This is consistent with Figure 11 of Moyles and Airey (2014) which gives recorded ppv for Dynamic Replacement of 4 mm/sec to 10 mm/sec at S = 50 m.

5 CONSTRUCTION VALIDATION

5.1 *Preliminaries*

The ground improvement contractor adopted a pounder having a diameter of 1.8 m and a weight of 13 tonnes. Initially, a 120 tonne crane was used to drop the pounder from a limited height of 10 m over concerns of vibration to nearby residence.

Following initial trials, a larger 200 tonne crane was mobilized to increase the drop height to 20 m in order to increase productivity and obtain higher depth penetration.

Based on the initial trials conducted and finding that a much large column diameter was achieved, the design pattern of the DR ground improvement was changed to 3.6 m diameter columns at 5.8 m centres on a triangular grid to give the same design a_r ratio of 0.35.

5.2 *Validation regime*

The DR validation regime comprised the following:

- Measurement of quantities of rock used, blow counts per theoretical metre depth of penetration.
- Excavation following column installation to check column diameter achieved at different depths within practical depths of excavation.
- Drilling through the centre and near the perimeter of columns to check the depth of penetration achieved and assessment of column toe diameter.
- Observe the angle of repose of excavated rock.
- Conduct plate bearing test on exposed column head to check Young's modulus of DR column.
- Conduct Cone Penetration Tests (CPT) with or without shear wave velocity measurement beyond the toe of the columns to check for thickness of remaining liquefiable soils, and whether any strength improvement occurred below the toe of the columns.
- Conduct CPT with or without shear wave velocity measurement between columns to check if there has been any improvement of the soil due to lateral compaction from the DR installation. Note, however, that potential improvement of the soil was not relied on in the design.
- Conduct MASW survey through a selected section to assess overall improvement of the DR zone.
- Measurement of ground level after DR installation to check ground heave.
- Vibration Monitoring at nearby residence at the top of the hill, at slope distances of about 60 m from the DR area.

5.3 *Validation testing results*

The following results were revealed from the validation testing:

- Typical blow counts to install each DR column ranged approximately 60 to 300 blows.
- Most of the columns terminated on top of a medium dense sand layer at a depth ranging from 4 m to 4.5 m. This probably explains the larger diameter achieved as the pounding energy caused lateral spreading of the columns rather than deeper penetration.
- The exposed column diameter ranged up to 3.6 m diameter.
- An average angle of repose of 44^o was observed for the rock used for the DR columns (The lowest value measured 42° and the highest value was 45°), confirming the adopted design value of 42^o to be appropriate.
- Plate bearing tests gave E_{col} ranging from 48 MPa to 55 MPa, confirming the adopted design value of 50 MPa after pounding to be appropriate.
- CPT testing below the toe of the columns and the base of the medium dense sand layer and MASW survey revealed the presence of liquefiable layers up to about 7.5 m depth generally, and in some cases thin (up to 1 m thick) liquefiable soils exists down to about 12 m depth.
- CPT testing before and after DR showed some improvement of the soils below columns, as shown in Figure 5.
- Measured vibrations were generally below the project specified limit of 5 mm/sec.

Note that the post-DR CPT between DR columns also had to be predrilled because of the lateral extent of rockfill. In the example given in Figure 5, the depth of the DR column is at approximately 4 m depth. It can be seen from this figure that the presence of the medium dense sand layer with cone tip resistance of about 3.5 to 4 MPa prevented penetration of the DR column. It can also be seen from the post-DR CPT results that some of the sand layers below the DR toe have been improved. However, there are silty or clayey layers in the example shown and in other CPTs that were not improved below the DR column toes likely due to excess pore pressures generated during the DR process. From about 1-2 m depth below the DR there is practically no improvement.

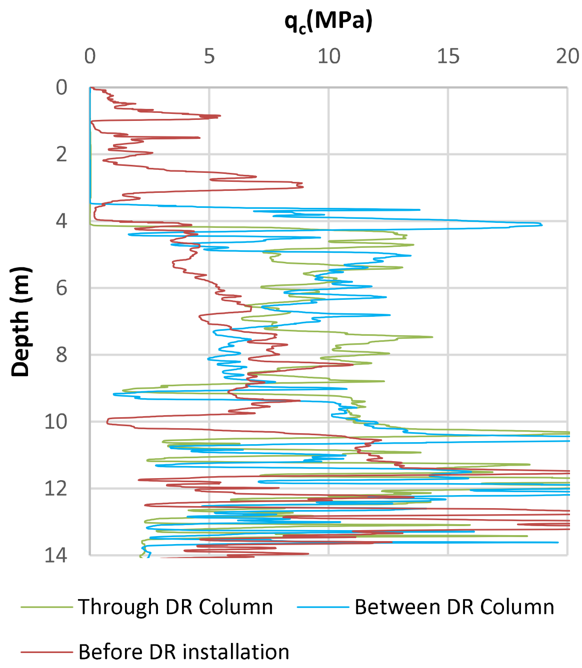

Figure 5. CPT cone tip resistance before DR installation and after DR installation between and through columns

5.4 *Post DR stability analyses*

Following the installation of DR and verification testing, stability of the embankment was reassessed. The extent of penetration was modified using the as-built records. Since the post DR CPT test results show insignificant improvement below the DR column, no improvement was considered for the soil layers below the DR in the reassessment. With additional stability measures such as increased strength of geotextile tensile fabric, couple layers of geotextile, toe berm, and a modified methodology in assessing the yield PGA which is presented in a separate paper by Dai et al (Rome 2019) , the design criteria were met the Minimum requirement by NZTA for the northern embankment.

6 CONCLUSION

Dynamic Replacement ground improvement was adopted to mitigate the potential liquefaction under a road embankment. The design was carried out with the aim of improving the liquefiable soil up to a depth of 7.5 m, with localized pre-excavation and replacement up to 1.5 m to allow for limited depth of DR column installation to 6.0 m. Dynamic replacement trial was carried out to assess constructability and optimize construction methodology such as drop height of pounder and column diameter to achieve required area replacement ratio and to limit vibration and noise levels. A comprehensive verification test procedure was adopted to assess the outcome of the DR production works such as the adopted material properties, column diameter, depth of DR penetration and improvement of soil below and between DR columns.

Due to the restrictions such as available size of pounder, project specific vibration limit and presence of medium dense interbedded layers, depth of DR column penetration was restricted to typically 4.0 m to 4.5 m. Some improvement in the liquefiable soil below the columns was observed in general from the CPT testing carried out following the DR installation. Although, the targeted depth of treatment was not achieved, the reassessment of the embankment stability and deformation based on verification test results and revised models indicated the design intent can be achieved by employing some remedial works such as additional geotextile and toe berms for the northern embankment. The overall outcome using DR ground improvement at northern embankment was cost-effective and achieved the desirable objective of a value for money, resilient solution, compared to the initially considered mass concrete lattice structure using continuous flight auger (CFA) techniques.

7 ACKNOWLEDGEMENT

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