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Design and Construction of Ground Improvement for Liquefiable Soils at Bridge Abutments for the Lincoln Road Interchange Auckland Motorway Project

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ABSTRACT

The Lincoln Road Interchange Improvements (LRII) Project encompasses approximately 2km of carriageway widening and enhancement and includes the replacement of three bridges crossing Huruhuru Creek and Henderson Creek. These works are part of an upgrade of the existing State Highway 16 motorway which forms part of the Auckland Western Ring Route. The bridge widening required approach fill embankments to be constructed over seismically liquefiable, and compressible weak alluvial deposits. Ground improvement was required to provide stability for the fill approach embankments to the bridges, minimising soil flow loading on the bridge piles under seismic loading and liquefied ground conditions, and to mitigate settlement. The ground improvement was constructed using both cement in situ Mass Stabilisation (IMS) and Deep Soil Mix (DSM) Columns. This paper describes some design aspects, initial site trials, construction, and test results while commenting on the effectiveness of both methods. Some variability was found in homogeneity and strength of the IMS and DSM, and the implications for design and construction are discussed.

Introduction

The Lincoln Road Interchange Improvements (LRII) project involves widening a section of State Highway (SH) 16. LRII includes replacement and widening of the existing bridges at Henderson Creek and Lincoln Bridge at Huruhuru Creek with reclamation embankments up to 4m high within the creeks. A new bridge was also required for the Eastbound off Ramp (EBOR). The existing motorway comprises four lanes, and the widened motorway at Henderson Creek Bridge will accommodate six traffic lanes, two bus lane shoulders and a shared cycleway/pedestrian route. The new bridge will replace the existing two lane eastbound and two lane westbound bridges. The overall width of the motorway will increase by approximately 18 metres. This paper describes practical experience with two types of ground improvement carried out to mitigate the effects of seismic liquefaction, and settlement on these high priority transportation facilities.

Geological Setting

The Lincoln Road, Henderson area is on the coastal margin of the Auckland Isthmus bordering the upper Waitemata Harbour, underlain at depth by the Miocene Waitemata Group Flysch (inter-bedded siltstone and sandstone) deposits which lie beneath most of Auckland. The eroded Waitemata Group East Coast Bays Formation (ECBF) in Henderson has been infilled with the Pleistocene aged Tauranga Group alluvial and estuarine sediments

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comprising mud, sand and gravel, and the more recent Holocene sediments estuarine muds. At Henderson and Huruhuru Creeks these deposits are typically up to 5m thick and comprise very soft high plasticity silts and clays with sand lenses and variable organic contents.

Design Considerations

Seismic loading was determined in accordance with the draft version of the Bridge Manual (NZ Transport Agency's Bridge Manual SP/M/022, Third Edition, June 2012). SH16 is identified as being an Importance Level 3 Route. For the bridge sites, slope and embankment stability analysis associated with a bridge, and liquefaction analysis were carried out with a Peak Ground Acceleration (PGA) of 0.24g. The assessment methodology used to assess liquefaction susceptibility was based on the analysis of in situ test results including CPT testing, and SPT testing in cored boreholes, and on comparison with compositional criteria from laboratory testing of particle size and Atterberg limits (NZGS, 2010).

CPT analyses including additional sensitivity analyses were carried out based on recent recommendations by Idriss & Boulanger (2008) which resulted in more conservative estimates of liquefaction potential relative to NCEER guidelines (Youd et al. 2001). These sensitivity analyses were undertaken in conjunction with the determination of fines content. SPTs provided a reasonable correlation to the susceptibility assessment using CPT data. Analysis of the CPT and SPT test results indicated that the geological units considered to be susceptible to liquefaction were the loose silty sands and low plasticity sandy silts of the Tauranga Group Alluvium, particularly those adjacent to the creeks. The more recent alluvial clays and silts are generally high plasticity indicating that these are unlikely to be susceptible to liquefaction. However these materials are low strength (average shear strength 38 kPa; 25th percentile 20 kPa) and may be subject to cyclic softening during seismic events.

The structures affected by seismic liquefaction are the approach embankments, bridges and piers for the Henderson Creek bridges, and the Lincoln bridges (crossing Huruhuru Creek). The high variability of the Tauranga Alluvium interbedded highly plastic silty clays and low plasticity sands and silts is reflected in the results of analysis of the CPT tests which indicated intermittent liquefiable zones. The analyses indicated that significant liquefaction is predicted at the eastern abutment and a lesser degree at the western abutment during the design earthquake. The older Pleistocene deposits are recognised as having a lower risk of liquefaction due to age. However, as there was no accepted way of including this in the analysis and, given the importance of these structures, design proceeded on the basis of liquefied lenses within the alluvium.

Ground improvement was required to achieve adequate global stability under seismic loading with lenses of liquefied soils as well as provide adequate static and seismic embankment bearing capacity. It was also required to mitigate the excessive settlement that is expected in the recent and weak alluvium that is present beneath the proposed approach embankments for all bridges. Options considered during design included jet grouting, stone columns, CFA piles, and Displacement Auger Piles. In-situ mass cement stabilisation and Deep Soil Mix columns were adopted for final design. Full scale trials were carried out to verify the adopted design parameters.

In situ mass stabilisation (IMS)

At Henderson Creek, mass stabilisation ground improvements were designed to form a ‘U” shaped zone surrounding the proposed embankments and bridge abutments as shown in Figure 1. Deeper zones extending through the alluvium were required for stability, and shallower zones generally 3m deep were used to mitigate settlement and provide construction access as shown in Figure 2. In situ soils were mass stabilised with cement binder to form a solid block of soil with a target unconfined compressive strength (UCS) of 1 MPa.

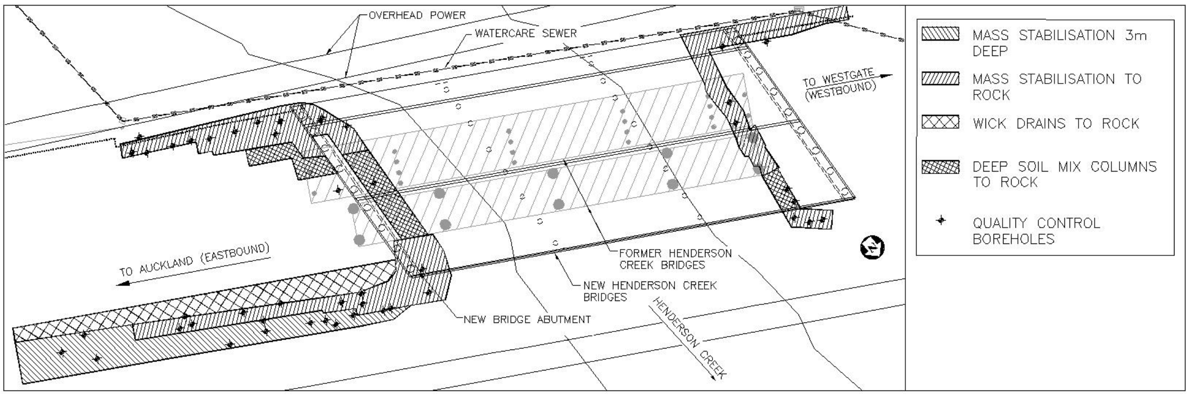

Figure 1: Plan of ground improvement at Henderson Creek Bridge.

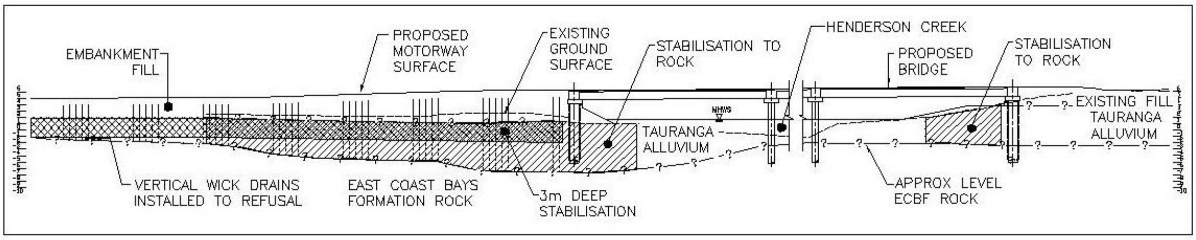

Figure 2: Geology and cross section of ground improvement at Henderson Creek Bridge.

The cost of ground improvements is significant for a project of this size. In consultation with the NZ Transport Agency, displacement criteria were adopted for seismic design of fill embankments. While a Factor Of Safety less than 1 is acceptable for stability, post seismic displacements were required to be limited to a maximum of 100mm longitudinal (and for all modes of failures affecting bridge abutments) and 500mm lateral displacement. A permanent outer sleeve casing was provided to avoid soil flow loading on the bridge piles.

Deep soil mix (DSM) columns

Due to site constraints, a portion of the ground improvements was constructed with DSM columns instead of mass stabilised soils as the DSM columns were able to penetrate through granular hardfills and to be constructed from the existing ground level to avoid extensive temporary works. The 800mm diameter DSM columns were constructed using grout injected under pressure and mechanically mixed in an overlapping secant grid pattern at 600mm centres with repeating grid patterns 2.4m x 3m. The specified UCS of the DSM was 3 MPa.

It was considered likely that there would be variability in strength and homogeneity of the cement stabilised mass. Hence, the stabilised soils were conservatively modelled using an undrained shear strength of 200 kPa to allow for variability, as the specification for IMS required a UCS of 1 MPa. DSM columns design was based on an area replacement ratio approach. Similar ground improvements were designed for the Lincoln Bridge and EBOR bridges. The two lane EBOR includes a fill embankment up to 4m high and approximately 120m long that is constructed entirely over the mangrove estuary. The contractor submitted an acceptable alternative to the IMS design using DSM columns overlain by a geogrid reinforced load transfer platform for a section of the EBOR.

A zone of the widened embankment needed to be constructed over the existing embankment slopes. Vertical wick drains were used to accelerate settlement and ensure that post-construction differential settlement of the carriageway spanning between the slope area and existing embankment would be within tolerable limits.

Construction and Quality Control

Mass stabilisation was undertaken by injecting a dry cement binder, through a rotary mixing head. The area to be stabilised was divided into cells approximately 2m x 1m that were given unique alpha numeric codes. The mixing tool was inserted twice into each cell and the quality control record included the kg/m^3 dosage per metre depth per cell.

Two field trials were undertaken at Henderson Creek (4m x 12m x up to 6.5m deep) and Huruhuru Creek (33m x 5m x up to 7m deep) to assess stabilisation of the variable soils. The soils at Henderson Creek were more sandy, but at Huruhuru Creek ground improvements were required for stabilisation of the very soft recent alluvium with undrained shear strengths as low as 13 kPa. The trials were carried out to both establish what binder (typically cement / lime) ratio was required to achieve the specified UCS of 1 MPa but also to determine the effectiveness of the mixing method, particularly in achieving homogeneity at depth. Initial samples were collected from both creeks and blending trials undertaken with laboratory testing to establish a blend mix for an in situ trial. A number of cement binder contents ranging from 130 to 170kg/m^3 were used. It was found that a mix of 140kg/m^3 of cement only generally provided an adequate binder addition to achieve the 1 MPa UCS at 28 days.

Wet grab samples were used for early strength testing only. During the trial phase, manual testing with a Scala Penetrometer was carried out to determine whether an early indication of strength and homogeneity could be gained from doing early penetrometer testing through the partially cured stabilised soils. One benefit of this would be that if poor stabilisation was encountered the soils could be excavated easily. However, it was found that after 24 hours the Scala Penetrometer could not penetrate the IMS, and before this time the material was too weak to give any realistic evaluation of consistency. Following mixing, the IMS surface was probed with a rod at 1m intervals to check for soft spots. Minor shallow soft spots were re-excavated and replaced with compacted granular hard fill. Any soft spots exceeding 0.5m deep were excavated and replaced with 10 MPa UCS flowable fill.

The most effective quality control testing was to carry out fully cored boreholes to the full depth of stabilisation and penetrating beyond the depth of mixing into the dense underlying soils. The boreholes were drilled at a minimum of 28 days following mixing. From these boreholes it was possible to determine whether a homogenous mix of cement stabilised soils had been achieved, and to check whether the mass stabilisation had achieved the depth required. The strength of the stabilised soils was determined from observation of the core and from laboratory unconfined compressive strength testing of samples at regular intervals over the full depth of the borehole. SPT testing was found to create significant disturbance in the core and was of limited use in determining if the ground improved material met the specified strengths. Accordingly, SPT tests were discontinued in the stabilised materials and carried out in the in-situ soils below only. A continuous core without SPTs gave a better indication of the homogeneity and UCS testing gave a better indication of strength.

The project specification required that mixing be controlled using GPS positioning and cement discharge be recorded on an "1m vertical x 1m horizontal" plan to achieve the required dosage rate per m^3. It was found that, as expected, variability was encountered in both strength and homogeneity of the stabilised soils. The boreholes were logged as 'well cemented' (where stabilised soils were assessed to have UCS of greater than 1 MPa), 'weakly cemented' (where cement was visible in the core which comprised firm to stiff soils, but clearly not the required 1 MPa strength), and 'uncemented' where there was no visible cementation. A graphical representation of the borehole logging of the degree of cementation in each borehole is presented in Figure 3 below.

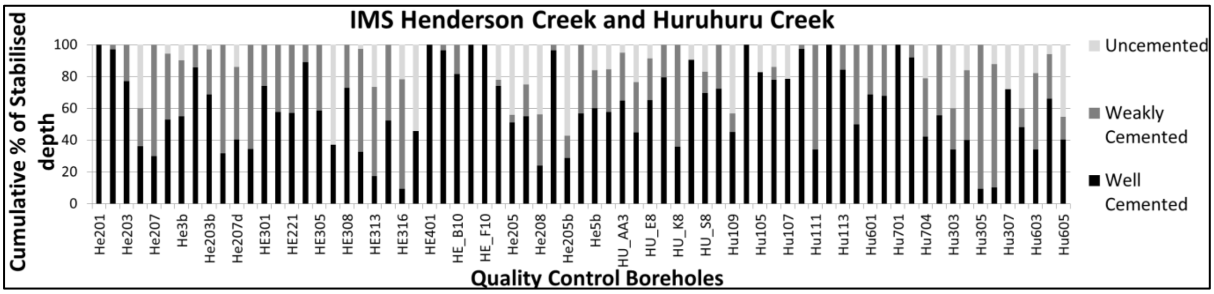

Figure 3: Graphical log of quality control boreholes in IMS ground.

Quality control boreholes were required at 10m intervals. When a significant depth of uncemented or weakly cemented material was encountered, an additional borehole was drilled either side to determine whether the zone was continuous and would need remediation. On two occasions, the quality control boreholes indicated a significant zone of 'weakly cemented' materials. The cement was visible in the core which comprised firm to stiff soils (but clearly not the required 1 MPa strength). Test pits were excavated into the zone. The material that was logged as 'weakly cemented' material was found to be a mix of very stiff materials with hard clasts within the mix. The overall strength of the material was assessed to be in excess of the 200 kPa shear strength which had been conservatively assumed for design to allow for the expected variability in strength and homogeneity. In addition, the material excavated was warm indicating that the cementation process and increase in strength was continuing well after the 28 days. The test pits were backfilled with 10 MPa flowable concrete fill. It is noted that overseas experience (Bruce, 2001) has indicated that the 60 day UCS of stabilised soils can have up to 1.5 times the 28 day UCS. A much greater consistency was observed in the DSM as shown in Figure 4 below.

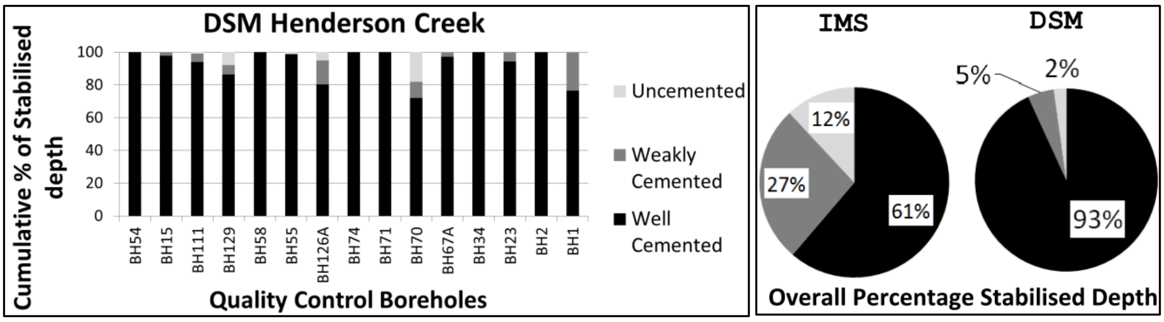

Figure 4: Graphical log of boreholes in DSM soil. Overall % stabilised depth IMS and DSM.

The 28 day UCS of the IMS soils with target strength 1 MPa varied from less than 100 kPa to 8.1 MPa with an average of 1.8 MPa. The UCS values of DSM with a target strength 3 MPa were more consistent, which we believe was a reflection of the more controlled piling equipment and methodology (as well as the higher cement dosage rate of 350kg/m^3). The 28-day UCS of DSM varied from 1.1 MPa to 12 MPa with an average of 4.5MPa. UCS results are presented in Figure 5 below.

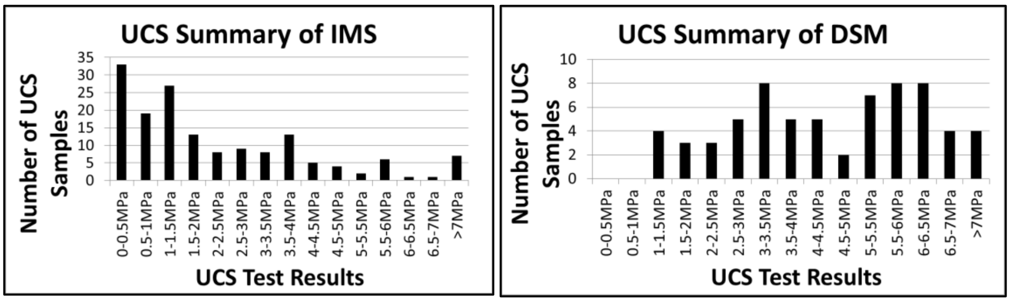

Figure 5: 28 day Unconfined Compressive strengths tested from IMS and DSM Cores.

At 28 days the stabilised soils were relatively weak and some lower measured strengths may be due to sample disturbance. The average strengths were well above the specified requirements. It is noted that while the average laboratory tested strengths may tend to be an over-prediction as the test samples were collected from more competent cores, the stabilised soils continue to cure well past 28 days and higher long term strengths would be expected.

Mass stabilisation is relatively easy to mobilise on weaker ground and was the Contractors preferred methodology. The strength of the material that the IMS tool could penetrate was limited to firm to stiff soils and the surface of the area to be treated was typically pre-cleared of roots, and granular fill. The mixing tool is fixed to the boom of a track mounted excavator, can reach out to mix soils that are about 5m beyond the machine. It does not require as stable a platform as the DSM rig, which must be located over a DSM column. The IMS rig has low head room capability and could be used beneath overhead power. The IMS process forms a block which is trafficable after about 24 hours. The IMS process was considerably cheaper (less than half the cost) than the DSM secant grid options per lineal meter of stabilised embankment. The DSM process was able to be carried out through granular hardfill and to be carried out to depths greater than 5m depth.

Conclusions

The designer should allow for the variability of both the stabilised soil strengths and mixing homogeneity in selecting design parameters for both IMS and DSM ground improvement. The DSM turbojet process appeared to give a more homogeneous product mix with more consistent strength than the IMS.

Adequate ground investigation is needed not only for design, but also for constructability, in particular to determine that mixing can be carried out to the depth required. Although the initial site trial successfully stabilised to 6m depth, in practical terms successful stabilisation to this depth is difficult. Homogeneous stabilisation to 3m depth appears reasonably easy to achieve. Stabilisation to 5m depth has more quality control issues but is also achievable. However, the most effective way found to stabilise any deeper was to create benches within the stabilised area to limit actual mixing depth to 5m (or to use DSM). Both methods were used to stabilise to the base of the old channels in Henderson Creek. In situ trials are needed not only to confirm the binder mix ratios for the proposed strengths but also to ensure that the contractor has robust quality control. It is critical to ensure that the methodology can achieve and record discharge of the binder on a per m^3 basis plan location and depth. At LRII even with this level of quality control, variability occurred in the IMS. Once mixing was in progress, the ground surface was obliterated, and GPS wass the most effective location tool to ensure the correct dosage of binder.

Cored boreholes through the stabilised soils were found to be the most effective method of determining strength and homogeneity. The programme must allow for at least 28 days for curing of the IMS (preferably longer), in order to achieve meaningful results from the boreholes and avoid excessive drilling disturbance of the core which would necessitate repeat boreholes. Remediation of IMS would be difficult to achieve due to the strength of the stabilised mass that has been successfully mixed, and the time for curing and further quality control drilling that would need to be accommodated in the works programme should major remediation be required. Establishing a robust methodology to ensure such remediation is not required is considered essential to this method of ground improvement.

IMS was found to be an effective method for improving the engineering characteristics of the very soft and loose alluvial sediments in an estuarine environment. However, consistency of mixing to achieve a homogenous mass was the greatest challenge and design of such ground improvement measures needs to allow for variability in the stabilised soils.

Acknowledgments

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