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Managing Client Expectations for Central North Island Infrastructure Designs

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ABSTRACT

As geotechnical professionals, we are often constrained by our client's expectations of timeframes, fees and scope, along with their physical works budget. Often the fees and scope don't match up. We are required to manage our client's expectations, whilst understanding their project objectives and level of knowledge. This paper looks at the challenges of designing mechanically stabilised embankments for road widening projects whilst not being able to directly interface with the client. The two example projects covered in this paper are roading projects and both based in the central North Island of New Zealand.

Case study 1 involves a 400 m section of SH1 with a maximum embankment height of 5 m. Case study 2 comprises an 80 m long embankment with a maximum height of 8 m, and a client requirement to use material excavated from site for fill. Both projects have geogrid reinforcing and subsoil drainage, and both require large undercuts for shear keys. This paper follows these projects through investigation, testing, analysis and detailed design, and highlights challenges faced by client management and design as a young geotechnical professional.

Keywords: Client Communication, Workshop, Ground Investigation.

1 INTRODUCTION

Mechanically Stabilised Embankments (MSE) are often seen as an economically viable solution for large stabilising or retaining problems. Our client's project managers required the design of MSE for both of the following road widening case studies. With the desirable outcome being the design and build of an MSE using locally sourced backfill material for a cost effective solution.

Two case studies are discussed in this paper. Both are located within the central North Island with varied natural and anthropogenic hazards, and with different geotechnical risks due to geological settings. Global stability analyses for each stage of the proposed construction were carried out for each site. In both case studies initial client communications were not direct to the designers. See Figure 1 for communication hierarchy.

The initial project understanding for Case Study 1 was detailed design of an MSE to span the entire 400 m section of State Highway 1 (SH1), with no indication of capital cost requirements expressed through the communication hierarchy to the design engineers. Detailed design was to be completed by 24 December 2015 and built within the 2015/2016 financial year.

For Case Study 2 the City Council (CC) project manager initially wanted quick, cheap and minimal ground investigation immediately preceding the detailed design on an MSE. The design engineers advised through the client relationship manager (CRM) the likely over conservative design risk and potential redesign cost implications if a thorough ground investigation was not completed. The scope changed from a detailed design to a preliminary design without ground investigation data, to be let for tender. The CC project manager wanted the ground investigation completed during the tender period, immediately followed by the detailed design of a MSE. Construction of the MSE was intended for early 2016.

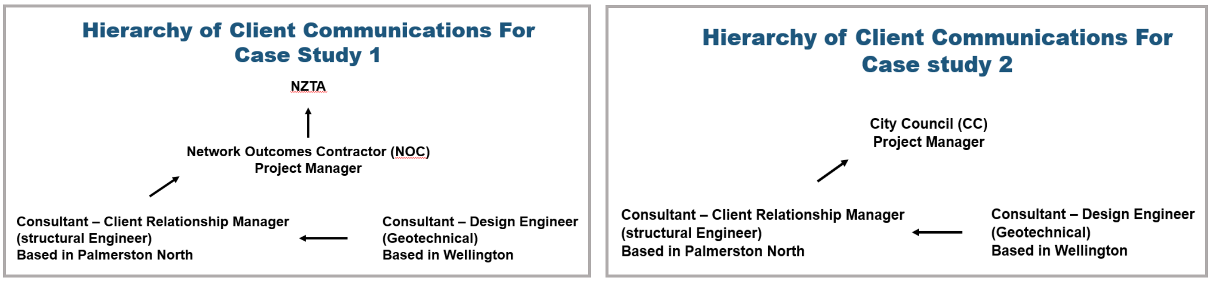

Figure 1. Hierarchy of client communications for case studies 1 and 2.

2 CASE STUDY 1 - SH1 ROAD WIDENING

It was proposed to widen SH1 by 2-4 m over a 400 m length of road. This was to be achieved by extending, steepening, and reinforcing an existing 5 m high embankment while also installing a road safety barrier. A two-week period was allowed to complete the geotechnical investigation, desktop study, slope stability assessment, detailed design of MSE, construction drawings, specification and schedule of quantities.

The existing slope was a raised fill embankment with steep slopes (max 55°) and up to 5 m in height. The existing road height was to be maintained but the slope steepened to 65° using MSE techniques.

2.1 Geology and Topography

The existing road embankment is partially located on Quaternary aged undifferentiated landslide deposits underlain by Pleistocene age river gravels and fan deposits (Townsend and Kamp 2008). The existing road is constructed on a raised fill platform in most areas along the 400 m section for stabilising which falls towards neighbouring rural properties and the local sewage treatment plant. The 400 m section of road is mostly comprised of locally sourced fill with areas in the northern section which alternates from natural ground to fill.

2.2 Geotechnical Investigation

The ground investigation was heavily constrained by the tight timeframe and budget. The investigation points were determined where changes in height, slope angle, and signs of instability were present. A series of nine hand augers and associated Scala Penetrometer tests (scalas) were conducted with an additional five scalas being carried out independent of the hand augers. The general stratigraphy across the embankment can be described as fill consisting of silt with trace sand and silty sand. Organic silty material was encountered at the toe of the embankment slope in a number of places.

2.3 Geotechnical Analysis

As no laboratory testing was carried out the material geotechnical properties were assumed based on experience and published values for similar materials. To assess the stability of the existing and proposed embankment, stability analysis was carried out using Slope/W software on a series of cross sections. The Morgenstern-Price method was adopted due to its applicability to the failure mechanisms expected from the ground model (Fedlund and Krahn 1977).

The asset (new embankment) was assigned an Importance Level (IL) of 3 in accordance with NZS 1170.0:2004 and the New Zealand Transport Agency's (NZTA) Bridge Manual SP/M/022, 3rd Edition. The peak horizontal ground acceleration (PGA) was taken as 0.63 g, calculated using both NZS 1170.0:2004 and the NZTA Bridge Manual (3rd Edition). A reduction factor of 0.5 was used to reduce the PGA to 0.32 due to both the existing embankment and proposed MSE being classed as non-rigid/flexible structures where the reduction factor levels off the over-conservative and overestimated PGA calculations for (rigid) retaining structures (NCHRP Report 611 2008).

Due to the shallow very limited ground investigation and lack of reliable in situ geotechnical data, a conservative design approach was employed to minimise the resultant risk. The proposed MSE was assessed for global stability for drained and undrained seismic and static load cases similar to the pre-excavation analysis. The results of the stability analysis indicate that the existing slope would fail under seismic conditions.

The stability analysis was carried out on five cross sections chosen to assess the more geologically challenging sections while providing coverage of the site. The existing embankment cross sections were analysed under static conditions with 12 kPa traffic surcharge, and seismic conditions with the applied PGA. The factor of safety (FoS) results were below equilibrium (1.0) and would be subject to some degree of failure during a seismic event under the modelled conditions. The failure in seismic conditions and the requirement for additional width justified the reinforcement of the carriageway verge to stabilise the slope.

2.4 Embankment Design

The 400 m chainage site area was broken up into three sections (A, B, C) based on the pre-excavation analysis, different slope geometry and soil parameters. The designs varied based on face height, available space, soil conditions and material encountered during the ground investigation.

An undercut / shear key was required due to poor founding conditions, and to reduce the length of geogrid in tight areas. Subsoil drains installed parallel to the back cut would effectively drain the subgrade material with the pipes draining the flow past the toe of the MSE to reduce the risk of erosion or subgrade softening. The proposed MSE increased the existing FoS to above the design threshold of >1.0 under seismic conditions, and >1.5 under static conditions for a slope in equilibrium.

2.5 Design Outcomes

Detailed design for the proposed MSE was completed within the extremely tight timeframes as required by the client. Given the limited timeframe and geotechnical investigation, the client accepted the residual risks relating to deep seated instability, temporary works stability, culvert design, guardrail design, amendments to the design as a result of utility company requirements, and lack of settlement and liquefaction analysis. A number of these issues could have been addressed with a more comprehensive investigation phase and a longer analysis and design timeframe.

Ideally in addition to a more reasonable timeframe, a workshop between the design engineer, the CRM, and the Network Outcomes Contractor (NOC) project manager would have been very beneficial as other types of retaining structures could have proved suitable. Also geotechnical risks could have been explained directly to the client, and the client's risk profile could be better understood by the design engineer. However, the request for a workshop was declined. The MSE design met the client's requirements and design criteria, but it was later deemed by the CRM to be too expensive, specifically the volume of backfill for the shear key undercut.

Communication between the NOC project manager, design engineers and CRM opened up following the completion of the MSE detailed design. A new design was requested, that of a retaining wall, which would be more in line with the client's budget. The proposed design life was discussed with a view to be more flexibility. It was intended by the CRM and the NOC project manager that reducing the design life and subsequently the PGA would allow for the design of a less robust structure at a higher risk profile and hence a reduced construction cost. The CRM pushed for the lower of the design life, resulting in the design engineers request for written approval from the asset owner (NZTA) that reduction in design life was acceptable.

Three design options were presented to the client:

i. 100-year design life for the majority of the site;
ii. 25-year design life for the majority of the site: and,
iii. 50–year design life for critical areas were remediation and upgrade where most required.

Option (iii), the critical areas option, was chosen by the client as it addressed the high risk areas and was constructible within the available budget. The NOC project manager had the opinion that a letter allowing the design life reduction would not be attainable from the NZTA. Had this conversation been able to be had between the designers and the client earlier in the project, the subsequent redesign could have been avoided.

2.6 Tied Back Retaining Wall

The critical areas consisted of three retaining walls totalling 50 m in length at areas where shallow seated failures had occurred previously. The tied back retaining wall maximum face height was 1.50 m.

The critical sections were re-analysed using the same soil parameters with changes in the geometry and design life. The Importance Level was brought down to IL 2 due to the reduction in area and height of the structures. This allowed for a lower PGA of 0.32, and with an applied 0.5 reduction factor the PGA was reduced to 0.18 g. Slope/W was used to check the global stability of the proposed retaining wall design. WALLAP software was used to design the retaining walls. The model was analysed using undrained (long term) and drained (short term) seismic conditions for all of the sections.

2.7 Case Evaluation

The design brief provided to the design engineers via the CRM changed significantly from the start of the project to the end tied back retaining wall design. The second design allowed for direct contact through workshops between the NOC project manager and the design engineers to establish the design parameters, tie in the re design scope of works, total capital cost, and new timeline.

Construction of the critical areas tied back retaining walls was completed within the financial year and well within the available budget of the NOC project manager. The tied-back retaining wall construction fees were 60% less than the initial MSE design, plus there was the addition of re-design fees.

3 CASE STUDY 2 – LOCAL ROAD WIDENING

Case study 2 was located in an area undergoing a transformation from a rural setting to more urbanised. The road widening was required for a new footpaths and cycleway. Several retaining structures including an 80 m section were required along with an existing unreinforced embankment with height varying between 8 m to 2.5 m.

The scope of the work was preliminary design and detailed design, followed by tender. The preliminary design was completed prior to any ground investigation, due to the client's desire to go to tender so that works could be completed within the financial year. The detailed design phase incorporated the geotechnical investigation and allowed for a refined ground model and robust design.

3.1 Site Description

The site is situated on an old river terrace 1.7 km north of the Manawatu River in a rural setting outside the flood risk zone. The terrace edge riser crosses the design area, resulting in predominantly flat land on either side of the steep terrace slope. A drainage pond and ephemeral stream were present at the base of the slope. At the toe of the embankment was moderately to well sorted alluvial flood plain gravel, commonly overlaid by over bank deposits and, poorly to moderately sorted gravel with minor sand or silt sometimes weathered, underlying a terraced surface, and/or overlying loess/paleosol couplets (Lee and Begg 2002). Gravelly silts were exposed in the terrace slope. The slope steeply dips between 25° - 40°, towards the alluvial flood plain deposits. The southern section of the widening area was underlain by fill placed at the time of the road construction.

3.2 Preliminary Analysis

The design soil parameters were determined (in absence of field and laboratory test data) by utilising the strength parameters for similar geological units based on experience, local knowledge and published values for similar materials.

The slope stability analysis was carried out using Slope/W. The importance level for this structure was determined to be Level 2 due to its intended road user volume and in accordance with NZS 1170.0:2004 and NZTA's Bridge Manual (3rd Edition). A traffic surcharge of 12 kPa was applied on the top of the embankment. For seismic conditions a PGA of 0.35g was applied to calculate the embankment's FoS against global stability. The PGA was further reduced by 0.5 to 0.18 g as the embankment was classed as a non-rigid/flexible structure (NCHRP 2008).

The existing unreinforced embankment global stability results were found to be >1.5 FoS under static conditions, however FoS under seismic conditions were <1.0 FoS warranting the design and construction of a slope retention measure for the widening. The proposed preliminary MSE design with a shallowing in slope angle, benched shear key undercut and reinforced geogrid with site sourced material as fill was required to achieve >1.0 FoS under seismic conditions.

3.3 Preliminary Design

The client's timeline required a preliminary MSE design to be completed prior to a ground investigation being carried out. The preliminary ground model for this was generated using desk study, aerial photographs, site photos and information gathered by the CRM local consultancy staff. Material outcrops observed along the terrace and around the site were also used in the absence of intrusive geotechnical data. The main slope stability problem centred on the existing uncontrolled fill at the top of the embankment on the southern section, poor existing stormwater infrastructure off the road above where the embankment was proposed, along with steep slope gradients along the entire existing embankment.

The completed design was required to widen the road by up to 10 m with a face slope angle of 30°. A benched undercut up to 5.2 m deep with 300 mm compacted GAP 65 was designed with geogrid layers. A subsoil drainage network to drain the back cut of the embankment out to the ephemeral stream was designed. Conservative parameters were adopted in lieu of real investigation data.

The use of site fill was not included in the preliminary design due to the risk involved with the unknown properties of an untested material only sparsely seen on site. The CC project manager through the CRM expressed hope for the available site fill to be utilised in the embankment following confirmation during detailed design.

3.4 Ground Investigation

Once the preliminary design had been issued for tender, there was time to complete the ground investigation. This consisted of a total of five cored machine boreholes with Standard Penetration Tests (SPTs) to update the existing ground model. Following the ground investigation, laboratory testing was carried out (Atterberg limits, Particle Size Distribution (PSD), and Standard Compaction Test. The laboratory testing enabled adjustment of phi and cohesion soil parameters previously adopted and indicated that gravelly silt terrace material could be utilised as fill in the proposed MSE.

3.5 Detailed Design

The stability analyses were run again with the refined soil parameters. Stability analysis results indicated that the proposed embankment configuration satisfied the global stability requirements under all load case previously analysed during the preliminary design phase.

Internal stability analyses were also conducted during the detailed design phase to assess the internal stability of the embankment for the proposed 14 m length geogrid configuration and check for pull out and sliding of the structures under static and seismic conditions.

A liquefaction assessment indicated that for liquefaction to occur, both a seismic and storm event would need to coincide. However, accepted industry standard and convention applies the use of one or the other and not a combination of seismic and 100-year storm event. Therefore, the potential for liquefaction under design criteria is considered negligible.

Finite element (FE) analysis was used to model soil behaviour. A staged construction sequence was modelled to capture realistic deformation behaviour of the embankment. Both static and seismic conditions were analysed using the refined soil parameters. In addition, an FoS using *phi - c* reduction technique was calculated (Khabbaz et al. 2012). The analysis results of the proposed MSE indicated that the deformation under static and seismic condition could be approximately 25 mm and 90 mm respectively. Most of the deformation under static condition is expected to occur during construction.

3.6 Case Evaluation

The communication hierarchy allowed for the early clarification change in the scope and timeline requirements before the design phase had commenced. Ongoing communication between the CRM and CC project manager during the preliminary and detailed design phases allowed information to filter through to design engineers to comprehend the client's risk profile, design parameters and objectives, and budgetary requirements. The detailed design was completed early 2016. Tender was awarded within the financial year, however the construction for the MSE has been delayed until summer 2017.

4 DISCUSSION AND CONCLUSION

Case Study 1's low cost, extremely restricted ground investigation only provided for a limited shallow ground model and a conservative MSE design which also resulted in a design that exceeded the available budget for construction. Case Study 2's lack of preliminary ground investigation, resulted in the design features the CC project manager initially wanted, but at much higher construction cost than initially anticipated, mainly due to the limited geotechnical data used to formulate a detailed ground model. Had sufficient investigations and adequate ground models been able to be produced before design in both cases, this may have saved costs on the design and construction phases of the project.

The client's project manager's design expectations in both case studies were received second hand through the CRM from a non-geotechnical discipline for a large portion of the projects. The level of technical understanding of the CRM, CC and NOC project managers were unknown until well into the design process when conference calls with all parties involved in the designs were approved to clarify design queries including risk profile, design life, importance level, budget, and type of retaining structure.

The CC project manager for Case Study 2 was made fully aware (by the CRM) of the additional cost involved in producing a preliminary design for tender to be updated with a detailed design during the tender period. The intended haste to complete preliminary design in line with letting an early tender to save time, may not have paid off considering the project did not start construction within the originally intended timeline.

The MSE and eventual tied back retaining wall design for Case Study 1 also required additional design fees. The tight timeframe and instructions through indirect communication ultimately led to a design for the MSE with recommendations stating that other retaining structures would offer a cheaper solution (construction), and that construction costs (of the embankment) would likely overrun any reasonable budget. However, the project was successful in completing the construction of the tied-back walls within the financial year as planned and within the available budget.

A direct line of communication with the client was missing in both case studies until well into the design phase, despite being requested several times. A simple meeting between the designers and the end client at the start of the project and at designated hold points, would have provided the clarification required to successfully deliver to the client's expectations. such meetings will be non-negotiable moving forward and will be used to confirm the scope, client's risk profile, design parameters and objectives, and importantly the client's design and construction budget to make sure the resultant design meets the client's expectations.

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