GeoSolve Ref: 200077
8 June 2020

Gore District Council

Attention: Hashem Ramezan-zadeh

Geotechnical Peer Review – Longford Shared Path, Gore

Introduction

Gore District Council (GDC) are proposing to construction a bridge over the Mataura River as part of the Longford Shared path. DC Structures Studio Limited (DC Structures) has designed the bridge in support with the project geotechnical engineer Riley Consultants Limited (Riley).

In accordance with our Agreement dated 25 February 2020 GeoSolve Limited (GeoSolve) have undertaken a peer review of the geotechnical design input for GDC. GeoSolve has also undertaken a review of hydrological and hydraulic inputs which is covered in a separate report. This report shall be read as a whole.

Geotechnical Review

GeoSolve has reviewed the following documents:

- Report titled: Geotechnical and Hydraulic Inputs – Longford Shared Path, Gore, Revision 1.0, 5 May 2020 by Riley Consulting Ltd; and
- Email titled: RE: Longford Peer Review – Slope Stability – 14 May 2020, by Eli Maynard of Rileys Consulting Ltd.

In addition, GeoSolve has had multiple discussions and communications with Rileys as outlined in the attached review registrar in Appendix A.

No arithmetic check has been carried out of any analyses provides as we assume that this has been carried out as part of Riley's internal checking procedures. We have reviewed input data and the analysis methods to satisfy ourselves that these are appropriate and checked that results are consistent with those anticipated.

Summary and Conclusions

Based on the review and responses from Riley, in our professional opinion, not to be construed as a guarantee, the geotechnical design methodologies and analyses meets current good practice.

We understand a geotechnical Producer Statement for Design (PS1) has not been issued to date. Therefore, at this time GeoSolve has not provided a Producer Statement for Design Review (PS2) this can be issued on issuing the PS1.

Limitations

This report has been prepared for the benefit of the Gore District Council

with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

If you have any queries relating to the contents of this letter please do not hesitate to contact the undersigned.

Yours faithfully,

Tim Plunket
Senior Geotechnical Engineer (CPEng)
GeoSolve Ltd

Colin Macdiarmid
Geotechnical Team Leader (CPEng)

Appendix A – Geotechnical Review Registrar

GeoSolve Ref: 200077
8 June 2020

Project: Longford Shared Path, Cyclebridge, Mataura River, Gore

Reviewer Comments:
By: Tim Plunket and Colin Macdiarmid (GeoSolve Ltd)

Response by: Eli Maynard (Riley Consulting Ltd)

Notation:
Report – Geotechnical and Hydraulic Inputs – Longford Shared Path, Gore, Revision 1.0, 5 May 2020 by Riley Consulting Ltd
Email 15 May 2020 – RE: Longford Peer Review – Slope Stability – 14 May 2020, by Eli Maynard of Rileys Consulting Ltd

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1	Report	Site Class – Assumed Class B for structure design but Class C for liquefaction assessment. It is our understanding that based on NZS1170.5 that the Site Subsoil Class will be C not B based on the following. Therefore, we think that Class C should be used for the structural design also. • The critical definition of NZ1170.5:2004 Class B is "A surface layer of no more than 3 m depth of highly-weathered or completely-weather rock or soil (a material with a compressive strength less than 1 MPa) maybe present". It is our understanding this is taken from ground level not the founding level. • The borehole logs slightly to moderately weathered Sandstone is at approximately 2.5 to 3.1 m, apart from BH06 which shows it at a depth between 6.5 and 8.9 m depth. • The drawing "Longford Shared Path, Mataura Cyclebridge, Bridgeplan" GDC_2019/11_S100, Revision A and dated November 2019 we understand that 5.3 m of fill is proposed i.e. up to 8 m of soil.	Loads generated during an earthquake are generally small compared to the other load cases such as wind and pedestrian dynamics, even after an amplification factor for Class C is applied. Notwithstanding that, we have spoken to the bridge designer who has agreed to rerun their analysis assuming Class C conditions to confirm that this is the case.	Using Class C for structural analysis	Closed
2	Report	For the liquefaction assessment, the report says that groundwater is assumed at the ground surface but outputs in Appendix D show water at 1.3 to 2.1 m	During our analysis we undertook a sensitivity analysis to the adopted groundwater level which included the scenario of groundwater at the ground	Agreed should be required to considering design flood at	Closed

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		depth. Would you be able to confirm which water level has been adopted (e.g. ground surface or you have assumed a water RL from the hydrological information). Also if water has been taken at ground surface, the first SPT N value was at 1.5 m and appears to be in a different layer than the upper material. Therefore, have you undertaken a sensitivity analysis of what N value you need for no calculated liquefaction in the upper material?	surface (flood conditions) and the results were generally insensitive to this. The plots shown in the appendix illustrate the measured levels during drilling, which are coincident with either normal river level or as measured. We have undertaken a close review of potentially susceptible layers from the borehole descriptions and have identified pockets of fine sand with some silt, and silty fine sand. These layers are thickest in BH1, BH4 and BH5. Uncorrected SPT N values were around 10-15 where recorded. Descriptions suggest fines content in the order of 20%. Assuming the lowest SPT N=10, fines content of 20% and a contributing thickness of between 1.5m to 3.0m, and that the soils are fully saturated results in ULS free-field settlements of between 5mm and 10mm. We note that in reality these are above the normal river level and hence a flood and earthquake would need to coincide. Notwithstanding that, a conservative geotechnical pile design has assumed no contribution in skin friction from any of the soil units, and instead relies on end bearing within the rock socket.	the same time and design earthquake. Downdrag effects above liquefiable layer (including the embankment where appropriate) has been encountered by ignoring skin friction in soil component below.	
3	Report	For the pile lengths would you be able to confirm if they are from current level or filled levels as table 6 and table 7	The pile lengths are as measured from the underside of the capping beam as outlined in Table 6 and Table 7.	Noted	Closed

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		references different depths. Or what are the proposed pile depths. The target bearing stratum should be noted on the drawings.	For the avoidance of doubt, 5 no. 7m long piles will support the mast, and 2 no. 6m long piles will support the east abutment. These lengths result in an embedment of ~1.1m into the target bearing stratum (North Range group) which is greater than the minimum socket of 0.6m (1D). This aspect will be verified by inspection during construction.		
4	Report	Can you provide the calculations for the rock socket design.	Calculations are included in Appendix D: Geotechnical Analysis in 'Rock Socket Pile Design per Sinclair & Every'.	Noted	Closed
5	Report	Is ongoing settlement been considered on the eastern abutment due to the presence of potential buried topsoil?	The buried topsoil layer has been in place for at least 34 years (since the stopbank was raised in 1986) and likely much longer, and both primary and secondary consolidation due to the stopbank should therefore be complete. The net additional load imposed on this layer from the approach works will only be minor, particularly considering the bridge and abutment beam are pile-supported.	Agreed. More of a minor issue for the cycle path than the bridge	Closed
6	Report	Have you made allowance for reduction in compression skin friction when both tension and compression loads occur in piles i.e. due to cyclic loading effects. As load in both directions should decrease skin friction in both tension and compression cases.	Skin friction is estimated to contribute ~15% of the overall capacity. The average factored ULS pile load beneath the mast is 1,099kN in compression, and the maximum on a single pile 1,225kN, i.e. the bridge can be fully supported on end bearing alone. Only one pile develops in tension, at eastern abutment under flood conditions. The factored load is 85kN (tension) which is much less than the 207kN allowable geotechnical capacity in tension, noting that this was estimating ignoring all skin friction	Skin friction not required for compression loads	Closed

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			in the soil, includes a reduction of 2/3 of the positive skin friction in the rock socket, and then includes a SRF=0.5.		
7	Report	What is the spacing from the centre of the piles to the crest of the fill slope just so we can confirm that there is enough width to gain lateral capacities.	Refer supporting information below regarding slope stability and generation of lateral capacities. Normally, we would assume that full lateral capacities can be developed if 8D of soil is available, i.e. the same applied to trailing piles in laterally loaded pile groups. However, for this project, we have taken a conservative view by: 1. Considering a soil spring range of 1/3 (soft spring) to 3x (hard spring) from the calculated value. 2. On the western embankment, limiting the maximum lateral force to the value at 3m (500kN), below which the passive wedge intersects the embankment surface. 3. For the eastern abutment, switching off (removing) the 1m lateral spring (at 3mbgl) in the model.	Noted	Closed
8	Report	There are a significant lateral forces given in Table 7, which are larger than vertical capacities given. Should a lower limit be applied to these?	Note above. A maximum of 500kN for piles at the western embankment and 135kN on the eastern abutment have been applied.	Limit of lateral loads applied	Closed
9	Report	For lateral loads what factor has been adopted for pile groups for both sides of the bridge.	As outlined in Section 8.1.2, interaction factors have been applied where piles are spaced less than 8D to account for effects such as shielding/trailing pile interaction. A	As discussed the springs have been used for a displacement analysis of the	Closed

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			linear value from 100% at 8D down to 25% at 3D was applied.	structure with sensitivity studies but see 9a	
9a		Can you confirm that these factors have been used for piles in all directions especially when 3D apart with reference to Cubrinovski et al (2009) referenced in your report.	Pile shadowing effects were considered by including a factor ranging between 1 (at 8D) and 0.25 (3D) in the structural model for loading in all directions. This is conservative in view that Cubrinovski recommends ignoring such effects for spacings of 6D and greater. We also note that we have adopted a greater range of soil springs from best-estimates (i.e. 33% and 300% vs. the recommended 50% and 200%).	Noted	Closed
10	Report	Ground profile and pile embedment appears to be different on both sides of the water. How critical is differential pile response across the bridge and/or has a sensitivity analysis been undertaken.	Piles on both sides of the bridge are bearing the same depth (c.1.1m) into the underlying North Range Group. Applied loads on the eastern approach are small compared to calculated capacities (16%), and calculated settlements on both side of the bridge within acceptable limits.	Check undertaken by Riley to confirm differential settlement in both cases accounting for different ground conditions.	Closed
11	Report	Typically anchors bond lengths should not be more than 10 m long so that the entire fixed length can be moblised. Knowing that the anchor lengths will be updated after the trial anchor testing. If higher capacities are not met then may have to consider a larger diameter hole.	We agree, hence 7m anchor bond lengths with a 4m unbonded section were selected. Agreed, either lengthening the anchor to 10m or increasing the hole diameter can be considered should the test anchor not achieve required capacities.	Noted	Closed
12	Report	In the last paragraph of Section 8.2 of the report it says that sliding can be achieved across the base of the concrete surround. Is there any chance of this losing contact with the ground due to the anchors?	Further analysis has been undertaken on this aspect since the report has been issued. The concrete surround is resisted laterally by 1.4m deep keyways and self-weight. The intention is to install and lock	Noted	Closed

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		Also in regards to passive pressure has anchor deflection been considered.	off the anchors prior to constructing the concrete surround, and for a 150mm HDPE duct to be installed to allow for these to act independently. Yes, anchor deflection (elongation) has been considered in the stressing analysis and tolerances.		
13	Report	Is there a filling specification? Also can you confirm gravel hardfill will be used?	An earthworks specification will be provided prior to construction. We confirm that approved granular hardfill is proposed for the western embankment.	Noted. The spec has not been reviewed.	Closed
13a		Based on assumed 38 degree friction angle angular hardfill is recommended (not pit run/river run) compacted to 95% of maximum dry density based on Test 4.1.3 of NZS4402:1986	38° has been assigned for the in situ sandy gravel (SPT N=40 typical, range from 30 to 50). We have assumed 35° friction angle for engineered fill, but agree that high quality angular hardfill compacted to a target of 95% MDD is desirable.	Noted	Closed
14	Report	The Insight Engineering report attached says there is some instability noticed especially around the western bank. How have you considered this especially given that the new embankment will be constructed adjacent to the river bank.	We note that a blanket of large-diameter riprap will be placed surrounding the toe. Preparations for constructing the western embankment will include tidying up local instability features that may impact the embankment. It is considered that these can be inspected and the extent of remediation/removal specified during construction.	Noted but should include assessment of eastern embankment as well	Closed
15	Report/Email 15 May 2020	Beca has raised the question if slope stability is an issue and what effect if any has on the piles. Would it be possible to provide more information about the proposed batter angles and materials used similar to point 13.	The embankment will have maximum slope batters of 2H:1V and be constructed of imported granular fill that is compacted to a high standard. Note discussion on slope stability below.	See Point 22	Closed

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16	Report	How thick is the fill to be placed and will this cause any settlement issues?	Fill is up to 5.5m, and given the granular nature of the underlying sand and gravel, settlements will generally occur as immediate settlement during construction.	Noted	Closed
17	Factual geotechnical report	The boreholes are terminated at relatively shallow depths into the rock, which is typically described as having closely spaced joints, although this could be drilling induced. The design requires 1 pile diameter into competent rock. Has the level of "competent rock" been established by the investigations? Is there a risk that piles will need to be significantly deeper than noted?	A check of geotechnical pile capacities was made applying dense gravel parameters and the method of B1/VM4 to assess the implications on bearing in weathered rock rather than as a socket. This provided a factored ultimate capacity of 1,225kN which gave us confidence that piles would not need to be considerable deeper. We note that no lateral load is generated at this depth (depth of fixity has been found to be ~5D or 3m for this bridge) and hence loads are vertical and would therefore not require heavy reinforcement. We will discuss the risks of needing to go slightly deeper (say 1m) with the contractor to ensure that this can be managed by inspection during construction.	Noted	Closed
18	Factual geotechnical report	There has been no rock coring or rock testing carried out. Has additional investigation been considered to confirm the deeper rock profile and to assess if the fracturing noted on the logs is drilling induced by the sonic drilling process. This would also confirm the likely rock types for the anchors which are noted to extend to 14 m depth (given the sedimentary rock is there a risk that weaker rocks are present at depth)?	The core recovered was transported to the laboratory but unfortunately was unable to be tested (for UCS or point-load). We have therefore applied somewhat conservative values estimated in Roclab based on our experience with similar rock. The North Range Group (Murihiku Terrane) rock in this location is very old (c.230Ma) and hence its strength tends to be governed by fracturing/joint sets and other discontinuities rather than weak rock types. We note that a trial anchor will be tested to	Closed. See comment 20 for additional risk mitigation for the test anchor	Closed

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			failure, and each anchor locked off to above the working load to confirm installed capacities.		
19	Drawings 106	The target compaction is 92% the compaction test this refers to should be stated. Also is site won fill is to be used then an alternative target may be appropriate (e.g. 95% of MDD from a standard compaction test).	Imported granular fill will be used, and hence this target will be established based on the material proposed by the Contractor. At this stage the source is currently unknown but will be subject to approval prior to use.	Noted	Closed
20	Drawings	The prelim drawings in the report mentioned a test anchor, but I couldn't see these in the latest drawing. The test anchor is to only have a 3 m bonded length so that it can be tested to failure. Given the lack of deeper rock data how can you be sure that the 3 m section is going to be representative of the fixed length.	The 3m bonded length was selected as it should represent the weakest/more weathered rock and represents about 50% of the proposed production anchor fixed length (indicated as 7m, but could be 5m as indicated by FHWA). Alternatively, this could be installed somewhere in the middle (say bonded over 2-5m where 0m is the start of the bonded zone). We will discuss this with the	To reduce risk the test anchor could be drilled to the full depth and the bonded area then selected	Closed
21	Drawings	A larger bar will be required for the test anchor	Ideally, the test bar would be the same as the production bar so that the installation and testing method can be verified. We have noted that the drawings indicate a 42mm bar, which is an error and should be 46mm. We note that a 46mm Williams 150KSI bar has a yield strength of 1,240kN, which is double the load generated from an assumed bond strength of 400kPa for a 3m long, 0.15m diameter anchor.	Noted	Closed
22			Confirmation of stability analysis and outcomes provided in email 28/05/2020.	Noted	Closed
23	Anchors	Can you confirm how anchor group effects have been considered?	Calculations based on Eq. 5-17 of FHWA NHI-05-039 for uplift capacity of micropile groups in cohesionless soil indicate a total	Noted	Closed

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			soil/rock weight of 2,914kN (971kN per anchor) compared to a maximum single cable load of 560kN. The uplifted blocks between the deadman anchor groups are spaced 12m apart, which means that uplifted blocks do not intersect.		