PNGL Wairoa – Gisborne 296.3 - 390.5km Track Formation Repair Works Civil Construction Work Packages 30%Design & Pricing – October 2019

Date: 30th October 2019

Project: FGL 1331

Prepared by: Maurice Fraser - Fraser Geologics Ltd

Section One – Report

TABLE OF CONTENTS

1 INTRODUCTION

This report is dated 30th October 19 and outlines estimated repair costs around the track formation & drainage civil works required to reopen the PNGL railway line north of Wairoa to Gisborne. The railway line was damaged in a major storm event in late March 2012 with multiple large washouts caused by a combination of poor drainage performance and high intensity rainfall overloading installed capacity.

Previous FGL reports have been supplied in 2013 & 2014 to various parties looking at rebuilding options, based around some then broad assumptions on earthworks volumes and productivities of local contractor resources. Several earlier versions of this report have been issued to others as the works have progressed. This version is final for this tranche of the works and benefits from having 30% design quantities and costings. It also reflects a change in thinking around repairing Dropout 4 from a rail to a road / overland based operation for access and material supply with significant build cost and program time / cost savings.

For this 2019 report we have commissioned early release processing of LIDAR ground survey information from track meterage 346km to 358km (Gisborne District is expected to have the balance from 358km northwards available towards the end of the year), undertaken detailed inspections and site assessment on some 40 task items and have prepared 30% designs for eight major tasks covering the main dropouts, trackside retaining works and concrete seawall issues at Opoutama. We have set up provisional quantity schedules and a preliminary construction program taking into account various constraints and access issues around working within the local rail corridor.

We have reviewed historical construction photographs and information, particularly LINZ stereo pair aerial photograph runs from 1938 and 1942 (taken during construction) and a 1:5500 scale set flown for NZR in 1986. These all show the track formation in good detail (particularly enlargements of the 1938 / 42 sets and the 1986 NZR set) and show changes in formation and landscape over time. Parts of the 1942 set within the upper Kopuawhara Valley are particularly impressive showing significant infrastructure used to build the line as well as significant formation construction earthworks.

We have updated our information on local material and contractor resources in the Gisborne region including contractor availability and pricing; some material supply rates have significantly changed over the past few months as a result of quarry issues. Site assessment work has extended further south from the 2013/2014 assessment areas out to Nuhaka to assess coastal retaining and erosion issues between Waikokopu and Opoutama and slope stability issues from Waikokopu to Nuhaka. A 6th washout has occurred post 2014 at track meterage 347.73km due to blocked culverts and has been added to the repair list.

Based on our most recent reconnaissance, there are nominally 40 repair tasks requiring work including 35 specific sites being the original 5 (plus 1 new) major washouts, trackside / formation drainage works, rock fall / stability issues, coastal erosion works and a combined rail / road relocation issue at Blacks Beach (331.41km) being driven by the Wairoa District Council. Five generic line items such as culvert cleanouts, tunnel repairs and similar are also included.

Site works based on the 40 tasks above have been split into 3 main areas:

- North – from Dropout 4 (353.95km) to Maraetaha (365km)
- South – From Opoutama level crossing (335.700km) to Dropout 4 (353.95km)
- Coastal – from Nuhaka (324km) to Opoutama (353.95km)

Repair works for Dropouts 1 -3 utilise 2:1 sloping geogrid reinforced local site fills or vertically retained GAP80/100 hardfill to restore the track in the Beach Loop area. Dropout 4 is a hybrid fill comprising vertical retaining up to 24 metres high using imported aggregate, 1:1 sloping geogrid reinforced formation embankment using site soils and a double span short bridge at the southern end to minimise the fill footprint and allow for an uncontrolled spillway discharge in the event of extreme flood flows. Dropout 5 will be rebuilt with low height walling & GAP 80 backfill using construction plant access down Railway Road. Dropout 6 has been designed to use imported hardfill from the Nuhaka Beach quarries railed into site but could be supplied from the north depending on project timing. Dropout 4 straddles the north and south work packages in this report on the basis that it will be repaired from both ends of the project, however based on the 30% design and costing results it is likely to be repaired using the old Public Works Department (PWD) access road into the site.

There are several additional culverts requiring increased capacity to meet Q_{100} flood design requirements, significant cleanouts and repair works including some upstream inlet protection needed where adjacent logging operations are underway at Railway Road (349.32km). The seawall at Opoutama also requires some immediate work to repair washout damage and toe erosion and extend its design life for several more decades.

Required works have also been classified into the following categories:

- Immediate works required to hold the rail asset in its current state and prevent further damage (predominantly culvert & swale works, upstream inlet rail screens, vegetation spraying and clearance). This work is considered independent of a decision to reopen the rail line or not. Some critical drainage clearance works redirecting stream flows back into adjacent track drainage to prevent additional land sliding were undertaken at 356.23km as part of re-establishing rail access for this project and several other blocked culvert inlets were cleared while the excavator was tracking past.

- Track formation and drainage works to re-open the line to rail traffic

- Deferred works – there are components of some of repair items that could be deferred for 24 – 48 months if budgets are stretched but economies of scale and having access to complete them without live rail traffic as part of the reopening works package is likely to be more cost effective.

- Enabling works - multiple bridges at either end of the project require significant numbers of Peruverian sleepers to be replaced prior to running rail traffic / work trains. While these items are captured in other budget items and not repeated here we note that the majority of civil works are unable to start until heavy rail access is available from Gisborne up to Beach Loop and to a lesser extent past the Nuhaka River Bridge from the southern end.

- There is an additional enabling work task (Task 35) on Opoutama Road at meterage 331.4km where a significant road dropout caused by underlying deep seated rock mass slope instability has required the road carriageway to be temporarily shifted into the rail corridor. Remedial works have been assessed by the Wairoa District Council with road relocation needing to be pushed forward and completed as part of and partially prior to southern rail access works.

Following sections set out engineering and program design assumptions and outline design issues around the 16 key washouts and civil works items within the 40 repair tasks mentioned above. There is a second report section setting up each of the repair items with a preliminary Construction Management Plan outlining design & repair features including a significant number of photographs to illustrate site features; these tasks have been designed (along with appended schedule and relevant drawings) to be further updated and extracted from this report package to form the basis for subcontractor briefing and pricing should the project proceed.

Site & historical photos included in the appendices include a mix of photos from 1938, 1942, 1986, 2008, 2012, 2013 and 2019 and have been selected to best illustrate various features as required. Photo sources include the National Library of NZ, NZR Government reports and a range of other electronic sources. Stereopair photographs have been sourced from the Retrolens website www.retrolens.nz

This draft for final version of the report has shifted the focus onto rail operations working predominantly from the north with the main supply depot at Maraetaha station. We have considered a preliminary Rail Protection Plan as part of understanding how overall access constraints, material logistics and train movements will impact the underlying program and pricing assumptions.

We note that while each of the repair items are relatively simple in terms of design there is substantial complexity in managing the overall program and interface between all of the sites, predominantly around forward ordering of materials, stockpiling and handling of hardfill and working through all of the intricacies around rail delivery and construction operations with limited plant and road access.

An accompanying set of project drawings containing the 30 % design drawings - reference FGL 1331 dated October 2019 is also appended to this document.

2 Key Design & Pricing Assumptions

Project reporting, works schedule & pricing has been built up based on the following series of assumptions. Some of these are expected to substantially evolve as the project moves forward, additional testing and investigations are undertaken to confirm design requirements and individual subcontractors bring their own mix of equipment and expertise to the various work packages available.

2.1 General Assumptions

- Track meterages are taken from KR information as uploaded to the mobile road app (www.mobileroad.org) and either recorded using GPS on site or subsequently taken from the desktop website. Meterages are generally labelled as centre of the feature and are rounded to the nearest 10m+/-

- Site works based on 40 items and split into 3 areas:
  - North – from Dropout 4 (353.95km) to Maraetaha (365km)
  - South – From Opoutama level crossing (335.700km) to Dropout 4 (353.95km)
  - Coastal – from Nuhaka (324km) to Opoutama (353.95km)
  - Dropout 4 straddles the north and south work packages on the basis of material supply and potential construction from both ends. In practice it appears more likely this will be repaired using the old PWD / NZR access roading used to originally build the line in the late 0930s.

- Majority of rebuild is from local forestry and civil works contractors and material resources – limited requirement to accommodate out of town contractors apart from specialist rail and a small team of project management personnel.

- Local contractors are considered to have the experience and expertise to operate safely within the steep hill country across the area and are able to self-manage the majority of the construction and access risks present as well as organise supply of project sourced aggregates and other materials through local quarries and delivery firms.

- There are several additional Hawkes Bay based contractors with experience on similar terrain who are also able to undertake the works and maintain some cost competitiveness on local contractor pricing.

- Project management is run by a relatively small internal team of engineering & QS staff with design engineering embedded in the control team. Project supervision will predominantly operate on a CM3/CM4 type level depending on what element is being built with onsite field testing personnel and equipment available as required. Some elements will be CM5, particularly around establishing benching levels and matching final designs to exposed dropout ground conditions.

- Project will direct source required materials and supply to each of the sites / subcontractors as required.

- Subcontractors are expected to be broadly self managed and capable of integrating project H&S and Rail Safety requirements into their own operational procedures.

2.2 Design Assumptions

- Required repair works are designed and constructed to applicable Kiwirail & NZ design requirements including for earthworks, slope stability and seismic design. Design work for major structures requires PS1 & PS2 peer review and signoff.

- Drainage works on the 6 major dropouts are reinstated and upgraded to a Q_{100} flood design requirement.

- Additional Q_{100} upgrading is required on vulnerable embankments where overtopping flood flows would cause significant track and embankment damage. One culvert currently falls in this category at Railway Road (Task 21, 349.32km). There are several other side catchments (1.0km^2 – 1.5km^2) where relatively large base culverts are indicated on the KR culvert schedule that will require further assessment during detailed design and may require some cleanout and upgrading with rail gates or similar.

2.3 Formation Access Assumptions

- Rail access is available from both ends utilising work trains importing hardfill, drainage aggregate and geogrid reinforcement materials as required. Northern works are based at Maraetaha Station. Southern works are based either at one of the Nuhaka Beach quarries (for materials) or at Nuhaka itself. An additional material stockpile site is available at Kopuawhara siding or potentially Opoutama on railway reserve land although these are unlikely to be used.

- Southern hardfill supply can be loaded and run directly onto rail wagons from the Nuhaka supply quarry. Imported ballast from Hawkes Bay can be off loaded and transhipped at the same location for the southern work packages, or trucked over the hill to Maraetaha for the Northern. Northern ballast supply may potentially be available from FH quarries processing river gravels at Ruatoria.

- Northern hardfill stockpiling will be run from Maraetaha for materials coming from the south and also for materials from the north to limit train downtime running back into Matawhero siding or Gisborne rail yard.

- Maraetaha Station access from SH2 will need to use the new Hikurangi Forest Farms logging access constructed approx. 200m south of Maraetaha Road and operate under forestry radio channels to the Station area.

- Earthworks equipment delivered to various sites via rail are limited to nominal 20 tonne excavators, smaller compaction plant and similar. Larger plant for Dropouts 2 & 3 is able to access Beach Loop overland. Larger plant for Dropout 4 can access via the original PWD access roads built in the late 1930s.

- Overland access into Beach Loop available through the Hikurangi Forest Farms block above and to the west; the existing very steep access track at 357.8km is only suitable for tracked equipment in dry weather and a new access suitable for wheeled plant (6 wheeler dumpers & similar) is considered feasible above Dropout 2 / rock fall area at nominal 356.8km +/- (refer 1986 aerial photo in Task 7 / Dropout 2 page 38 and drone photo Task 8 page 42)

- Northern rail access into Dropout 4 through Tunnel 23 is available after dropouts 1 – 3 are repaired and rail formation and track are rebuilt along Beach Loop where required.

- The old PWD Railway construction road via the HFF block and Paritu Station requires a reasonable amount of upgrading to allow for construction plant access and hardfill delivery as well as a bridge crossing over the Tikiwhata stream.

- Access to Dropout 5 is available overland via Railway Road / JNL Forestry.

- Rail access is available from the south into Dropout 6 and from the north after dropouts 1 – 5 are repaired. In practice we expect Dropout 6 to be the last to be repaired as rail plant and equipment will be committed to repairing the dropouts further north.

- Access for the coastal section seawall repairs is available at Opoutama and Waikokopu; 60m of seawall reinstatement plus repair of wall base erosion will require low tide access from Opoutama Beach and the balance of coastal works will be from within the rail corridor above.

2.3 Material Assumptions

Available granular hardfill materials for the project include the following:

- Greywacke derived gravels and sand from Nuhaka Beach quarries, predominantly pit run material (30mm down gravel and sand) with the odd larger stone. We have looked at various processed materials in Jukes Quarry but the all in product is considered to have sufficient fines content to lock up under compaction for bulk fills. For design purposes Nuhaka materials are assumed to have parameters of 18kn/m^3 and 35 degrees internal friction angle. Confirmation of this will be required.

- Quarried greywacke materials are available from Matawai Quarry (Fulton Hogan) including AP65 / 80 & 100 size materials. Similar size range materials are also available from Ruatoria although with longer transport distances/ cost. For design purposes the northern quarry materials are assumed to have parameters of 20kn/m^3 and 38 degrees internal friction angle. Confirmation of this will be required.

- Processed limestone is also available from quarries to the north (e.g. Downers at Tiniroto) or a private farm quarry on Mangaone Road inland from Nuhaka to the south. General contractor comment is that limestone is preferred for forestry roading as it packs down with sufficient fines to lock up under forestry traffic. The potential to use this as MSE reinforced fill will need to be assessed during detailed design.

- Processed subsoil drainage materials can be supplied from either north or south quarries with a preference for Nuhaka supply subject to pricing. Processed river gravels back loaded from Ruatoria may also be suitable.

- We understand railway ballast is available from Hawkes Bay. Fulton Hogan advise they have materials that could meet much of the current Kiwirail spec with some processing and this will be assessed further in detailed design. Given the rail tonnages expected on the reopened line some pragmatism around ballast specifications would offer significant savings. We note the current railway ballast on the track is predominately rounded beach gravels sourced from Nuhaka and is nowhere near the current KR specified standard.

- Onsite soils are a fill option for much of the required earthworks although their use will depend on what time of the year the works are undertaken. Geogrid reinforced site fills require a greater volume of material per m^2 of reinstatement embankment front face area due to poorer engineering properties compared with hard fill and also require hard fill shear keys for seismic sliding along the base which significantly adds to site fill costs.

- Drainage culverts are either standard size concrete pipe or utilise a 1.75m diameter arched concrete box culvert designed for the project with culverts fabricated locally in Gisborne. We have priced conventional box culverts as part of 30% design optioneering but supply and transport costs are prohibitive compared with local arch manufacture, even with taking into account the cost of new formwork.

- Project will direct source required materials such as geogrids, retaining wall materials, aggregates, concrete supply and similar and make these available to subcontractors as required.

2.4 Construction Program Assumptions

- The provisional construction program assumes the following: (please note there are significant changes from earlier reporting in these program assumptions)
  - Repair works requiring rail support will be progressed predominantly from the north (essentially this requires just one train set instead of two, albeit requiring transport of locomotives and rolling stock into Gisborne by road from Nuhaka),
  - A separate overland construction operation will run for Dropout 4 with rail support available after Dropouts 1 – 3 are repaired,
  - Culverting & Dropout 5 works south of Tikiwhata Tunnel will be accessed from Railway Road,
  - Dropout 6 will be repaired once Dropout 4 is complete and the train set can run through to the south – note a temporary fix may be used to get the train south of the dropout to allow it to work backwards from Nuhaka for additional material supply or support at Blacks Beach operations,
  - Seawall works at Opoutama will run as a separate operation (with local batched concrete),
  - Blacks Beach repair works will run a separate operation, although with the potential to use rail to transport large rock and spoil for coastal protection and buttressing works to just north of Waikokopu (Task 33).
- Construction works assume the Beach Loop dropout repairs / northern section is constructed in summer to maximise local site cohesive fill placement with Dropouts 4 to 6 (southern and coastal sections) able to operate in wetter parts of the year due to hardfill backfill.
- We note the accompanying pricing estimate has Dropouts 2 & 3 as currently being constructed out of hardfill & retaining wall blocks as this is the cheapest option based on design assumptions to date; this allows for some flexibility in timing of the construction works depending on when project decisions are made.
- Construction operations are based on having 20 to 30 tonne excavators, 6 wheeler dump trucks, compaction equipment and the usual range of small construction plant available at each dropout location.
- Onsite soils will be used for the dropout fill repairs where these are cost effective, with drainage aggregates, rail ballast and concrete aggregate imported via multiple rail mounted wagons, generally with side tip facilities.
- Concrete works for drainage works, wall / bridge / structure foundations, coastal works and general purposes will be mixed on site and operating under ready mix plant certification and test regimes from Firth Concrete in Gisborne.

2.5 Land Access & Consent Assumptions

- Pricing and works scheduling is on the basis that access to all areas is straightforward and earthworks / tracking consent conditions are standard / non notified.

- Some access points and potential cut areas for filling around Beach Loop appears to sit outside KR boundaries – this is assumed to be resolved and at $0 project cost.

- We have provisional verbal agreements to access the rail corridor through Paritu Station including upgrading the old PWD road where required.

- Discussions with JNL / Hikurangi Forest Farm representatives indicate access should be straightforward through the land they control, provided we operate within their access procedures where live logging operations are underway.

- Based on our recent experience with Gisborne District Council for retaining wall design on the Lower Logyard we have assumed that embankment fill solutions, bridging and drainage structures can be exempted from the Building Consent process provided independent PS1 & PS2 certification is supplied as part of the works package.

- We have assumed building consent works with Wairoa District Council can be obtained on a similar exempt basis with PS1 & PS2 supplied as above.

- We have assumed that coastal consents for the work at Opoutama can be obtained on a non-notified basis, albeit with additional (limited) environmental assessment reporting.

3 PNGL Task List

A preliminary list of railway repair tasks is outlined below. These have been substantially expanded in the appendices.

Table 1 – Repair Task List, PNGL

Task no	Name	track meterage	Issue / required works	Scope / scale /volume / quantities / other	Comments
1	**Embankment Slip 1 @ 363.72**	363.72	Short washout on embankment edge approx. 150m south of bridge 274 (over SH2)	Driven H pile repair wall with timber lagging nominal 10m long. Allow driven 250UC73 @ 800 centres, 9m deep with 200*50 H5 timber lagging installed behind rear flange. Nominal trackside retained height 1.5m with sloping embankment toe.	Design TBC - steel section sizing to be confirmed. Driven pile design adopted on basis of driving into embankment filling and avoiding drilling works and concrete supply into corridor. Alternative drill & concrete (600mm with temporary casing) as well as whalers & tiebacks if required.
2	**Embankment Slip 2 @361.11**	361.11	Short washout on embankment edge approx. 300m south of logging road crossing off Wharewhatas from SH2	Driven H pile repair wall with timber lagging nominal 15m long. Allow driven 250UC73 @ 800 centres, 9m deep with 200*50 H5 timber lagging installed behind rear flange. Nominal trackside retained height 2.5m with sloping embankment toe.	Design TBC - steel section sizing to be confirmed. Driven pile design adopted on basis of driving into embankment filling and avoiding drilling works and concrete supply into corridor. Alternative drill & concrete (600mm with temporary casing) as well as whalers & tiebacks if required.
3	**Wharekakaho Tunnel 26 North Portal**	359.93	Tunnel Portal Drainage Works	Excavate out and reinstate swales coming out from tunnel portal. Nominal 60m of swale deepening & widening Allow new secondary culvert where stream crosses under track - allow new 1200mm diameter culvert nominal 10m long TBC Extend and deepen bypass and overflow swales down both sides of formation. Widen stream channel on eastern side and dig back into slope to provide more sediment catch area.	Inspect ground above tunnel - there is around 1.5km^2 of area to west of tunnel and not clear on site where the drainage for this runs - possibly over top of tunnel or approx. 50m north. May require access track to be constructed up onto western side of tunnel and clean out of areas. Existing culvert partially blocks with material coming down stream channel, larger / additional culvert required.
4	**Wharekakaho Tunnel lining cracking**	nominal 359.2km	Tunnel Lining Assessment	Allow for baseline lazer scan survey through tunnel (Woods Consultants) as part of track reopening works	Assess concrete lining cracking - has been in place for nearly 80 years, in good condition for age.
5	**Wharekakaho Tunnel 26 South Portal**	358.48	Drainage works	Excavate out and reinstate swales coming out from tunnel portal - nominal 80m of swale deepening and widening	Deeper swales and cutoffs required.

6	**Dropout 1 Tunnel 26**	358.3	Downstream embankment - allow to reinstate buttress support using Paragrid reinforced MSE slope up to 25m high and reinstate emergency overflow drainage works. Install new 1.75m arch culvert as additional Q_{100} flood capacity Install new high level drainage access onto existing arch culvert under embankment.	Rebuild buttress fill against pinnacle embankment using Paragrid reinforced MSE slopes. Setup area as spoil dump site for surplus materials from Beach Loop and material excavated from northern end of Wharekakaho tunnel - use DBM side tip wagons. Allow to slew track westward (1-3m) to ease tight radius curve on top of current embankment. Allow for new high level secondary inlet on existing tunnel drainage under ridge line (vertical manhole drilled down onto tunnel?) Allow for repair of emergency overflow works -recover pipes in downstream washout and reinstate with additional rip rap outlet protection. Allow to excavate cutting slopes on southern end of site to allow for track slew and supply buttress fill material.	KR completed significant inlet works in 2014 / 2015 including construction of rail protection inlet screens For detailed MSE design - look at rockfill buttress and design requirements for Paragrid reinforcement. Consider subsoil drainage under buttress fill - mix of aggregate, geotextile protection and megaflo Ultra 300 high strength pipe. Consider keying depth at base of buttress toe key - may need undercut down into / through fill materials present in base of fill.
7	**Dropout 2 - Beach Loop**	357.14	Rebuild track formation using either 2:1 (V:H) Paragrid reinforced MSE slopes faced with hybrid gabion baskets backfilled with local site soils, Or Paraweb reinforced compacted GAP 80 / 100 faced with vertically faced Stonestrong concrete blocks.	Scale & volume TBC Either option requires significant benching into existing scour washout sideways & down to lock new fill into underlying slope. Install upgraded drainage inlet on underlying primary culvert system. Clean out existing swale drainage and reinstate. Deepen swale next to siding - concrete wall required? Repair culvert outfall - consider COPED units for energy dissipation (requires additional inspection)	MSE design requirements - rock toe for keying into slope, drainage design, geotextile protection, fill slopes may need to extend out past the adjacent ground to give sufficient sliding width on geogrid block steps. Use 2:1 gabion faced structure to limit “tail chasing” down the slope; note obtaining sufficient Paragrid length for sliding design under seismic is significant cost and currently making site sourced fills more expensive than imported hardfill option. Consider design with nominal Ru value for wet soils Consider subsoil drainage design, drainage aggregate, geotextile protection, megaflo Ultra 300 high strength pipe, benching works into underlying SST/ZST - rock breaker required? Consider COPED units for energy dissipation at base of upgraded drainage works
8	**Cutting excavation & drainage reinstatement @356.83**	356.83	Excavate out slip material blocking track and reinstate stream drainage.	Scale & volume TBC Excavated material to go to dropout 2 as MSE fill Install new 1.75m culvert drainage outlet from stream - deepen swale next to track - install additional culverts under track to take stream overflows.	Establish where slope debris has come from and consider any uphill works required – allow to excavate access track up slope and undertake benching or drainage redirection works above site. Existing culvert(s) may be buried under fill, current culvert and deep swale drainage leading north looks impractical - concrete wall required under track formation to better direct water to north? New 1.75 arch culvert allowed for in costings
9	**Rockfall batter slope@356.76**	356.76	Regrade slope, reinstate excavated benches	Excavate out failing material and large boulders - use boulders as rock toe for Dropout 1 and 2 with backup fill material for Dropout 3 Reinstate existing benches and tidy up internal slope drainage. Excavate out swale at bottom of slope adjacent rail, improve rock catching ability, rework culvert inlets and outlets Consider loss of ground on downstream side of site - any precautionary H piles needed for track support? Drilled or driven pile design? (not allowed for in current pricing, assumed sufficiently stable)	Rock slope looks unstable but has generally performed well since 2013. Some material failing into swale at southern end, will require additional assessment.

10	**Stream works @ 356.23**	356.23	Excavate out slip material blocking swale drainage - clean out concrete headwall swale back to splash wall.	Slip partially excavated out by DBM in week of 15 June as emergency works Additional excavation works and removal of spoil to Dropout 1 or local disposal required.	Additional protection where stream enters top of debris fan down to swale drain should be considered. Additional excavation and removal of fan drainage to be assessed further.
11	**Old Tunnel 24 site - two slow moving slip areas**	355.970 & 355.825	Slow moving ground movement - ongoing KR maintenance and track reinstatement issue	Allow to undercut and repack track over both areas as interim measure Allow for geotechnical investigations and machine holes on both slip areas - establish depth of movement as well as ground models	Northern slip appears to be material failing down dip - possibly bedding plane controlled movement. Southern slip appears to be old fill movement, likely in material placed from removal of tunnel 24 in mid 1950s.
12	**Rock scaling areas on western side of formation between 355.42 and 355.79**	355.42-355.79	Cliff face areas require scaling to remove significant rock fall hazard.	Allow to scale cliff face Allow to put 20-30 tonne excavator up on bench and remove collected rocks - push out onto formation bench below. Remove track and sleeper sets prior.	Require expert assistance to define scale and scope of problem - prelim budget 10-12 days of actual abseil work on site.
13	**Dropout 3 @ 355.57**	355.57	Rebuild track formation using either 2:1 (V:H) Paragrid reinforced MSE slopes faced with hybrid gabion baskets backfilled with local site soils, Or Paraweb reinforced compacted GAP 80 / 100 faced with vertically faced Stonestrong concrete blocks.	Scale & volume TBC Either option requires significant benching into existing scour washout sideways & down to lock new fill into underlying slope. Clean out existing swale drainage and reinstate. Replace culvert outfall - extend outfall out and down to better discharge position. Current discharge is destabilising slope below rail formation.	MSE design requirements - rock toe for keying into slope, drainage design, geotextile protection, fill slopes may need to extend out past the adjacent ground to give sufficient sliding width on geogrid block steps. Use 2:1 gabion faced structure to limit “tail chasing” down the slope; note obtaining sufficient Paragrid length for sliding design under seismic is significant cost and currently making site sourced fills more expensive than imported hardfill option. Consider design with nominal Ru value for wet soils Consider subsoil drainage design, drainage aggregate, geotextile protection, megaflo Ultra 300 high strength pipe, benching works into underlying SST/ZST - rock breaker required?
14a	**Tunnel 23 Northern Portal**	355.354	Drainage works @ northern portal	Clean out swale and outfall drainage at northern portal Establish where significant sound of water coming from at nominal 355.28km - google images suggest deeply scoured narrow gully on slope above tunnel centreline, is water getting down in behind tunnel lining?	Tunnel drainage in generally good condition – clean out outfall, track down exterior lining inflow. Note tunnel is acting as drainage under slope to south, inflow unlikely to be an issue provided tunnel lining is not exposed
14b	**Tunnel 23 Southern Portal**	345.41	Drainage works @ Southern Portal	Clean out swale drainage - extend and deepen towards south. Install new 900mm diameter culvert to south of existing culvert as additional bypass capacity	Evidence of water and silt flowing back into tunnel for 50 to 100m – additional outlet capacity required plus swale cleanouts.
15	**Rock scaling areas on western side of formation between 354.0 & 354.2**	354-354.2	Cliff face areas require scaling to remove significant rock fall hazard.	Allow to scale cliff face - significantly less work than around dropout 3. Rockmass dipping into slope, cut face appears to be relatively stable with minimal rock in adjacent swale. Allow for any minor scaling as part of scaling works further north	Require expert assistance to define scale and scope of problem - prelim budget 2 -3 days of actual abseil work on site.

16	**Dropout 4 @ 353.95**	353.95	Complex site with 34m of vertical rebuild height over 90m of track formation centreline. Two options currently assessed: Option one 75,000m3 bulk fill of onsite soils with under drainage and chimney drainage to meet dam design requirements, 12m high toe retaining wall, 90 m of 1.75m arch culvert secondary bypass drainage, overflow erosion protection, Option two 24m high vertical retaining wall using GAP 100 backfill reinforced with Paraweb, 10m high site soil rail embankment on top reinforced with Paragrid, upstream cohesive fill shoulder as dam design cut off, 35m of 1.75m arch culvert as secondary bypass drainage, two second hand 12.2m rail spans as short bridge at southern end to minimise overall fill volumes and allow for emergency spillway at RL 140. Option 2 allowed for in pricing schedule.	Vertical wall use imported GAP 80 / 100 hardfill ex Matawai quarry, Stonestrong blocks and Paraweb reinforcement to build 24 metre high stepping batter slopes. Install new bridge 268A – use two 12.2m spans ex Hamilton yard to minimise fill volumes in backfill and allow for emergency spillway. Upstream of rail centreline use local cohesive materials as permeability cut off (embankment needs to be designed like a dam in several respects) Allow to excavate out and reinstate tunnel drainage under adjacent northern ridge line - include for new driven rail protection units around inlet as well as additional emergency inlet in concrete headwalls Allow to install spillway @ RL140 bypass	MSE design requirements - Stonestrong blocks required to arch and key into base and side of slope, drainage design, geotextile protection, battered fill slopes Significant logistic exercise to construct - overland access considered best option for construction.
17	**Tikiwhata short tunnels 20-21-22**	353.39 - 353.78	Add walkways to bridge 267A & 268 as part of hardfill import option (required for rail operators & access works if southern supply used.)	Add walkways to bridge 267A & 268 as part of hardfill import option. Clean out drainage @ end of tunnel 19 - inlet area is choked with debris Assess ground above tunnel 19 portal - minor rockfall on track	Current walkway requirements likely to be superseded if overland construction used for Dropout 4
19	**Tikiwhata tunnel 19 Southern portal**	350.4	Drainage works	Clean out swales at southern tunnel portal	
18	**Tikiwhata Tunnel lining cracking**	nominal 352km	Tunnel lining	Additional review of inspectors report on tunnel lining cracks and the need or otherwise for additional works on lining support.	Consider baseline lazer scan survey through tunnel (Woods) to assess any wall closure or cracking issues. No physical works considered required at this stage
20	**Tikiwhata tunnel southern bridges**	350.25	Erosion protection on Bridge 266	Allow to use large rock to provide additional erosion protection on northern bridge abutment 266	
21	**Railway Road Culvert works**	349.32	Culvert Works	Clean out upstream outlet area; install new railway iron protection works around inlet and additional rail iron screen upstream. Install all-weather track into culvert headwall & screen areas so emergency access is available 24/7 from Railway Road for timber slash clearance. Install new 1.75 arch culvert for Q100 flood design requirements.	Current culvert size is 1.3m wide by 2.0m high, 20m long, 10m deep @ 90 skew (width / height TBC) Upstream catchment is nominal 2.2km2 +/- (Waipawa Stream. Downstream outlet is undermining back into bank – scour protection required (large rock / concrete)

22	**Dropout 5**	349	Formation repair, track drainage upgrade, possible 1 – 2m track slew uphill to unload adjacent railway iron retaining walls.	Repair scour washout under track and reinstate formation with concrete block walls on downstream edge. Assess swale drainage and upgrade water movement on either side of washout - likely blocked culverts, may require additional culverts or upgrading.	
23	**Dropout 6**	347.73	Rebuild track formation using Paragrid reinforced MSE stepping 45 degree slopes and imported hardfill ex Nuhaka Jukes Quarry.	Note this work is additional track damage post March 2012 event – failed sometime in 2014? Repair scour washout under track and reinstate formation with steep reinforced MSE slope on downstream edge, local cohesive fill on upslope side. Use imported Nuhaka Quarry fill for MSE block or alternative GAP80/ 100 loaded on at Railway Road. Cleanout current culvert to act as low level bypass and install new higher level 1.75m arch culvert as primary drainage system.	Alternative consider site soil filling – summer weather constraints on this option. Check erosion on adjacent Kopuawhara river at gully outlet.
24	**Culverts Tunnels 14 - 17**	345-347.8	Culvert cleanouts and repairs	Multiple culvert & swale issues extending from Kopuawhara viaduct through to Tunnel 17 – culvert cleanout, outlet protection, scour protection, excavation / cleanout and reshaping of swale drains	
25	**Slip south of tunnel 13**	344.73	Counterfort drainage assessment	Current visual geotechnical assessment is that the site area is stable as it currently stands; allow for further investigation work if project proceeds.	Some counterfort drainage may be beneficial above and below track from 344.7 back to tunnel 14 Entrance – history of slope movement in area, assess current performance of installed stabilisation drainage works.
26	**Rock cuttings south of Kopuawhara Viaduct**	341.3-344.5	Clean up rockfall debris around formation	Multiple size rock fall events within rock cuttings. Overall cuttings look to be relatively stable and minor additional slope scaling work expected.	Consider excavating out downslope cutting banks and using excavated hard rock as coastal protection rock at Opoutama
27	**Bridge 262 Opoutama Abutment**	335.43	Repair abutment erosion and reinstate abutment support on northern end of Bridge 262	Abutment retaining design, require coastal design input. Check structure issues with Novare.	Combination concrete block walling and hard rock spoils to control coastal erosion at head of the beach.
28	**House Slip @ 355.05**	335.05	Slip failing in front of house above track	Assess failure causes. Consider if proposed earthworks will worsen upslope stability in vicinity of house? Are there any EQC reports available on house site?	Slip appears to be reactivation on previous slip area – multiple ongoing events visible in aerial photos over time.
29	**Seawall reinstatement and erosion protection**	335.26	Seawall reinstatement over nominal 60m	Existing seawall requires repair / extension – has been ongoing design issue since late 1960s. Original 1938 seawall was butted up against rock outcrop that has eroded away. Remedial design either big bags concrete filled, large rock spoils or Stonestrong seawall.	Consent issues will require early consideration. Options available to remove / relocate old railway wagons to remove visual issue at end of beach and replace with wall to improve.
30	**Seawall under erosion & overtopping A**	335.1-335.2	Overtopping protection on top of existing seawall	Multiple sites on southeast facing wall sections – require additional 1.2 – 1.8m height plus repair of significant erosion under existing 1937 / 1938 seawall.	Walls were mostly cast against rockmass outcrops apart from some minor gully fill areas. Repair of scouring pumped concrete bags plus additional toe protection? For overtop protection – geotechnical capacity of concrete wall likely to be limited in terms of supporting additional fill surcharge, particularly where fill imposes a lateral wedge load at top of wall . Consider self drilling grouted anchors to provide stabilising force at top of wall where new height is installed.

31	**Seawall under erosion & over topping B**	335.26-335.34	Overtopping protection on top of existing seawall	Multiple sites on southeast facing wall sections – require additional 1.2 – 1.8m height plus repair of significant erosion under existing 1937 / 1938 seawall.	Walls were mostly cast against rockmass outcrops apart from some minor gully fill areas. Repair of scouring pumped concrete bags plus additional toe protection? For overtop protection – geotechnical capacity of concrete wall likely to be limited in terms of supporting additional fill surcharge, particularly where fill imposes a lateral wedge load at top of wall . Consider self drilling grouted anchors to provide stabilising force at top of wall where new height is installed.
32	**Seawall under erosion & over topping C**	334.91	Overtopping protection on top of existing seawall	Multiple sites on southeast facing wall sections – require additional 1.2 – 1.8m height plus repair of significant erosion under existing 1937 / 1938 seawall.	Designer comments – walls were mostly cast against rockmass outcrops apart from some minor gully fill areas. Repair of scouring pumped concrete bags plus additional toe protection? For overtop protection – geotechnical capacity of concrete wall likely to be limited in terms of supporting additional fill surcharge, particularly where fill imposes a lateral wedge load at top of wall . Consider self drilling grouted anchors to provide stabilising force at top of wall where new height is installed.
33	**Rock protection behind railway wagons**	334.5-334.65	Buttress rockfill on slip area behind railway wagons - weight on slip toe to preserve global stability as well as minimise removal of soil by wave action.	Area is currently protected by wagons at mid tide level but overtopping waves are eroding away slip debris and material behind wagons – needs some rock fall armouring to prevent loss of soil.	Check global stability – wagons and new rock fall expected to be required to provide for adequate FOS on upslope movement.
34	**Rock scaling areas on western side of formation between 334-334.38**	334-334.38	Cliff face areas require scaling to remove significant rock fall hazard.	Allow to scale cliff face - significantly less work than around Dropout 3. Rockmass dipping into slope, cut face appears to be relatively stable with minimal rock in adjacent swale. Allow for any minor scaling as part of scaling works further north	Require expert assistance to define scale and scope of problem - prelim budget 10 days of actual abseil work on site.
35	**Road Dropout @ Blacks Beach**	331.4	Road has failed - track will require movement back into hill to reinstate double carriageway	Significant work undertaken by Wairoa District Council, including geotechnical & geological assessment and proposed realignment works, discussions with KR including land ownership and entry permits.	Ground models require review but solution proposed by WDC looks reasonable and a good medium term solution to significant underlying rock mass failure and coastal erosion issues affecting the road and indirectly the railway line.
36	**Generic Culvert cleanout & reinstatement**		Secondary culvert inlet/ outlet cleanouts and flushing	Multiple culvert cleanouts and swale improvements required along formation from 324km to 364km area.	
37	**Generic bridge erosion protection and scour / protection issues**		Any minor civil works required as part of bridging packages	Majority of bridge foundations are in good condition – known areas flagged in item 20. Additional checking required as part of detail works.	
38	**Generic Tunnel issues**		Any minor civil works required as part of additional tunnel packages	Majority of tunnels foundations are in good condition – known cracking areas flagged in items 4 & 18. Some additional minor lining required for water ingress and some lining requires refixing.	
39	**Generic rail formation issues**		Any minor civil works required as part of additional formation engineering requirements	Formation issues place holder – formation in generally good condition, significant ballast shoulder work required.	
40	**Generic Vegetation clearance**		Outside civils - in rail budget?	Significant vegetation and spraying required along formation and around tunnel portals.	

4 Pricing Basis & Risk Management

4.1 Pricing Basis

Pricing has been worked up two ways, as follows:

- Using day works rates with a range of assumed and advised productivity from several local forestry and earthworks contractors
- Using measure and value rates based on preliminary design volumes and items.
- Pricing items have been checked against local contractor rates as at August 2019 with additional allowances based on operating in a constrained access environment (many of the works are similar to forestry roading and maintenance type projects where access is relatively long and stringy and significant health & safety constraints exist).

4.2 Risk Management

Risk is managed as follows:

- Risk allocated to principal / project on basis that is the best place to manage it from - limited lump sum and risk pushed onto individual contractors

- Land ownership access and earthworks risk is assumed to be resolved at minimal cost prior to the project commencing.

- Drainage design risk is managed with relatively conservative Q_{100} estimates; these appear justified based on the over topping failure events that have occurred. We have allowed for multiple lines of drainage defence however a comprehensive maintenance regime is critical for the track going forward.

- Contractual risk needs to be discussed further; we have assumed a relatively small team of design and project management personnel as well as on site testing equipment / technician(s) to run the project. This needs to be discussed further.

- Physical construction risks are considered best managed by employing subcontractors who work in this terrain every day and have good local knowledge of ground conditions and behaviour.

- There are some cost risks around what time of the year the project is built - it is somewhat cheaper project to construct in summer than winter

- We have allowed for some design creep risk in the design work and pricing to date, but there are significant unknowns with consenting and any consultation works required.

5 Specific Major Repair Tasks

Following sections outline design thinking around some 20 of the 35 major task items outlined above and in the appendices. These are the major issues either requiring 30% design as part of pricing or are a substantial cost item requiring additional explanation and assessment. Additional design comment is also in the appendices.

5.1 Tasks 1 & 2, H Pile Walls

The two driven H pile embankment repair sites are at track meterages 363.72km and 361.11km respectively. Both appear to have been caused by over topping surface water from blocked swale drainage or some toe erosion from streams at the embankment base. Both sites consist of several metres depth of end tipped and relatively uncompacted embankment fills formed from local cohesive soils and weathered rockmass taken from surrounding cuttings and borrow areas.

Proposed retaining solutions involve relatively close centred driven H piles (250UC 73 @ 800mm centres) embedded nominal 9 metres depth across the top edge of both drop out areas. Steel sections are able to cantilever retain up to 3m of soil and train load provided the wall centreline is located a minimum of 1200mm past the sleeper edge. 200* 50 H5B timber lagging backfilled with GAP65 will be used to make up any exposed formation height but the primarily track load support mechanism is the driven steel sections with soil arching in between. The proposed repair solution is essentially the traditional Kiwirail driven rail solution but updated to use heavier steel sections

Drilling and concreting the H piles in 500/600mm diameter pile holes is an alternative installation method however there are significant cost issues around drilling equipment and concrete supply into the corridor as well as significant collapse issues from loose embankment fill and drilling refusal on larger boulders that will affect drilling performance.

Retaining wall capacity can be improved with whalers and anchors or tiebacks if necessary; we will extend the current 30% design with this option if ground conditions indicate additional capacity is required.

5.2 Task 6 – Dropout 1 @ Wharekakaho Tunnel.

Dropout one straddles a deeply incised gully approximately 150 m south of the Wharekakaho Tunnel (Tunnel 26) southern portal at track meterage 358.300km. Dropout dimensions are in the order of 40m wide by 60m long and nominally 20 to 30 metres deep depending on how the washout is measured. Upstream catchment is some 1.83km^2 with a base concrete arch culvert of nominal 1.2*1.8m dimensions and a 900mm diameter, high level emergency overflow steel pipe culvert. Top of rail level (TOR) is nominally RL 130m with the base culvert invert at nominal RL 110m and emergency overflow set at nominally RL 119m. The formation fill embankment is constructed from side and bottom tipped tunnel spoil excavated from the adjacent Wharekakaho Tunnel - refer Task 6 in the appendices for additional photos and more detailed information. Base of the dropout is around RL100m +/-, some 60 to 70m downstream of rail centreline.

Existing formation damage is due to the base culvert blocking with flood debris and storm water subsequently overtopping the track formation and washing out the downstream facing as well as erosion from the emergency bypass set approx. 11m below track level.

Repair works required at Dropout 1 are as follows:

- Existing slope stability FOS values on the current pinnacle embankment are considered unacceptably low, particularly under flood loading and/or seismic shaking and reinstatement of the downstream fill buttressing is required for long term rail operations. Adequate short term geotechnical capacity is considered available for work trains passing the dropout at 10km/hr.

- Buttress fill is designed as Paragrid reinforced local soils derived from the dropout itself, as well as material recovered from the track cutting to the south and additional materials brought into the site via side tipping rail wagons.

- Paragrid MSE will need to be toe keyed at the base (potentially requiring 3 – 5m of localised excavation down to the inferred level of underlying competent rockmass and backfilled with imported hardfill for seismic sliding performance) and then designed to take surcharge loads from end tipped and track rolled filling placed on top. The site will essentially be set up to act as a fill site for civil works with sufficient material added from the cutting slopes at the end of the project to provide adequate long term existing formation buttressing.

- Subsoil drainage comprising megaflo Ultra 300, Bidim geotextile and imported granular aggregates is required to maintain the internal stability of the MSE block as well as the external stability of the overall embankment.

- The cutting excavations to the south will be designed to allow some easing of the track curve on top of the embankment between Tunnel 26 and Bridge 273 - currently 150m right radius through Bridge 273 northwards, transitioning onto 300m right radius into the tunnel. In practical terms this will require a 3 – 4m widening of the cut +/- with a nominal track slew of 1 – 3 metres dependant on final track centreline design.

- 30% drawings and cross sections are included in the drawing set, ref 1331. Indicative MSE fill volumes are in the order of 23,000m^3

- KR undertook significant drainage works on the underlying arch culvert in 2014 / 2015, installing a rail protection structure to keep logs out of the culvert as well as an additional rail screen further upstream. Deepening and realignment of the stream channel was also undertaken around the inlet. These works require some minor clearance of vegetation and sediment from around the rail base but are otherwise in very good condition.

- NIWA Q_{100} runoff from the nominal 1.83km^2 catchment is modelled as 33m^3/s. The existing concrete arch culvert is 1.2m wide by 1.8m high and capable of passing 23m^3 / sec based on a surcharge head of 9m (surcharge up to the level of the emergency bypass culvert – note this is a significant flood volume stored behind the embankment, up to nominal RL 119m) .

- The secondary / emergency bypass culvert comprises a 900 mm diameter steel pipe able to provide additional flood flows of 4m^3/s under a 3m high surcharge. There is approx. 1000m^3 of flood storage on the upstream side of the embankment which does somewhat buffer the flood routing past the embankment but to meet the Q_{100} flood design without embankment overtopping an additional 8m^3/s capacity is required.

- We note that the above capacity calculations do not take into account any partial blockages on / around the rail protection gate structures that would occur in a Q_{100} flood event - there will be significant shallow landslides and earthflow type failures in residual soils dumping spoil and vegetation into the catchment and this will cause significant blockages on the current intake structures.

- Based on the above, we have allowed for the following:
    - Installation of a high level emergency inlet on the RL 110 arch culvert beneath the ridge line (essentially a 1500/1800 mm diameter rising manhole or similar with a significantly engineered scruffy dome on top to allow for stormwater flow into the base culvert in the situation where the rail gates become significantly blocked with flood debris)
    - Installation of a single 1.7m dimensioned arch box culvert with the inlet set nominally 6m – 8m under TOR level, discharging down the southern edge of the downstream MSE fill. This location is the shortest distance from side to side and there is room along the side of the MSE fill to install large rock riprap and provide a flood channel.
    - There is an option to locate the arch culvert at the northern end of the rail embankment where the spillway could be cut into hard rockmass on the ridgeline down the side of the MSE fill; this will be looked at further in detailed design.
    - Arch box units are a specific design to accommodate the high (18m plus) fill surcharge loadings on Dropout 4 (see following design sections) and using them here helps to further lower their unit cost for the project.
    - Design discharge volumes for the 1.75m arch culvert are $10.5m^3/s$ at 2m head above invert and $18.5m^3/s$ with 5m flood surcharge head.
- Detailed design checking will relook at flood routing through the base culvert, the effect of upstream ponding on required bypass volumes, engineering issues around the new vertical intake transitioning into the horizontal arch culvert and value engineering around 1 or 2 arch culverts under the track. At this stage budget costs have allowed for one arch culvert and a spillway down the southern edge of the MSE fill.

5.3 TASK 7 – DROPOUT 2 @ BEACH LOOP.

Dropout two straddles a backfilled incised gully at nominal track meterage 357.14km. Dropout dimensions are in the order of 25m wide by 70m long by 10m deep depending on how the washout is measured. The dropout is at the southern end of the Beach Passing Loop and sits more or less directly underneath the turnout.

Upstream catchment is some $0.3km^2$ with a concrete arch culvert of nominal 1.0*1.6m dimensions set nominally 5m below TOR level. Secondary/ emergency drainage is via a swale drain heading north on the inland side of the passing loop. The formation fill comprises excavated soil and rockmass from the adjacent track formation cuttings and is in the order of 15 to 20m depth based on the surrounding slope geomorphology. TOR level is in the order of RL134 +/- with the base of the dropout nominally RL100, some 70m downstream of rail centreline.

Material exposed in the washout is predominately excavated cut materials comprising a mix of silty and sandy residual soils and gravel to boulder size siltstone and sandstone clasts with no intact rockmass visible, although it is likely to be present at 3 – 4m depth in places down the slope. There is onsite evidence of at least one washout having occurred previously with an older, cemented stacked stone retaining wall and backfill present immediately south of the current washout.

Required repair works are as follows:

- Reshaping and significant benching out the washout area prior to installing either a 2V:1H (63 degree) sloping Paragrid reinforced MSE slope faced off with hybrid 0.5m high gabion baskets filled with local site soils or a vertically faced retaining wall structure backfilled with Paraweb reinforced GAP 80/ 100 greywacke derived hardfill.

- Both options have advantages and disadvantages; the retaining wall option has minimum benching and imported fill volume requirements to meet design requirements, using more expensive albeit substantially better performing materials. The Paragrid / gabion option has cheaper on site fill costs but requires substantially more excavation and fill volumes due to poorer engineering parameters and is more expensive in the current design iteration used for pricing.

- Previous reporting proposed a series of nominally 45 degree faced, 5 – 8m high reinforced blocks stepping up the facing with 2 – 4m wide level benches in between. These have been superseded as a result of better survey information and design modelling.

- Either option will need to be thoroughly keyed into underlying ground at the toe, rear and sides. Surplus excavated materials will be placed downstream of the MSE toe to provide additional toe support.

- Subsoil drainage comprising megaflo Ultra 300, Bidim geotextile and imported granular aggregates is required to maintain the internal stability of the MSE block as well as the external stability of the overall facing.

- NIWA Q_{100} runoff from the nominal 0.3km^2 catchment is modelled as 5.4m^3/ sec. The existing concrete arch culvert is 1.0m wide by 1.6m high at the inlet and capable of passing 8.5m^3 / sec based on a surcharge head of 3m above the invert. The arch culvert is tunnelled through intact rockmass along the northern side of the dropout and discharges out onto the Beach Loop facing north of the dropout.

- The secondary / emergency bypass system for this area comprises a sideways cutting into the concrete faced swale drain at the back of the passing loop to the north. Modelled swale inlet capacity is in the order of 2-3m^3/s dependant on how wide and deep the swale entrance can be profile cut during repair works, although this will be constrained by narrower swale sections further north.

- The next three discharge culverts running down the swale to the north are 600mm diameter @ 357.308km & 357.452km with a 900mm diameter culvert at 357.549km. Combined these three culverts have somewhat less capacity than the underlying arch culvert - total of 3.4m^3/s made up of 0.7m^3/s from each of the 600mm culverts and 2.0m^3/s from the 900mm culvert assuming a driving head of 2 metres (essentially the adjacent swale drain full to overflowing).

- Given the history of over topping failures in the immediate area from what is a relatively small catchment we have allowed for the following:

    - Additional excavation and rail inlet protection around the existing culvert inlet and installation of an additional upstream rail gate to prevent it being blocked with trees and vegetation.

    - Installation of a high level culvert inlet (nominal 1500/1800mm manhole riser with a scruffy dome top) set several metres downstream to provide a high level inlet into the base culvert in the event the main culvert entrance becomes blocked with flood debris. This can be set closer to the rail formation and tied in with swale inlet works depending on final design requirements.

  - Excavation of a pondage area to provide for some future spoil volume from periodic culvert maintenance and cleanout (excavated material will be used as part of the MSE fill or toe buttress)

  - Substantially improved entrance into the swale drain including cutting back of the ridgeline and some short culverting to ensure adequate access is available to the swale when the main drain becomes blocked.

  - We have considered the option around installation additional culverts at relatively shallow depth under the area to act as an emergency bypass system. These would be configured to start operating when the swale drain is half fill, and will discharge on top of the existing culvert drainage outfall.

  - Detailed design of the new high level inlet onto the base culvert and further swale assessments may negate the requirement for additional culvert capacity (E.g. it may be feasible to widen out the swale drain and install larger culvert capacity to the north). However the history of the site indicates that significant flood events do occur with periodic washouts happening and this round of repairs will have to comprehensively address the hydraulic conditions present.

- 30% drawings and cross sections are included in the drawing set, ref 1331.

- Indicative MSE fill volumes are in the order of 5600m^3 for the retaining wall option and nominally 16000m^3 for the site fill option. The main difference is engineered fill performance under seismic shaking with the onsite soils having relatively low friction angles requiring longer reinforcement and hence more excavation and backfill volume.

- Some repair work is required where the existing drainage discharges onto the slope. Access is complicated due to its location and repairs will need to be sorted as part of detailed design.

5.4 Tasks 8 & 9 – Cutting & Rock fall areas.

While separated out in this report for scoping repair costs, these two areas run together on site and are part of a single uphill slope failure affecting the formation between track meterages 356.73km to 356.85km. There are additional slope failures some 150m to 200m uphill of the site and much of the debris blocking the cutting appears to have come from this upstream area as an earthflow type failure washing down the slope during the 2012 event. (Refer photos in appendices).

These upstream debris flow failures are very hard to mitigate against and at best can be managed through deep channels and culverts able to bypass a lot of the movement spoil which will require detailed cleanouts after major events. We will look to install some driven rail gates further up the stream channel although by the time a failure event hits these it will travelling at several m/s and heavily fluidised with rainfall and channel flow and are likely to be ineffective at best.

Review of 1942 & 1986 stereo pairs indicate this area has had an ongoing history of slope movement since formation construction with adjacent slope movement visible in the 1942 set.

The rockfall site was significantly battered back in 2011 after a failure event and while currently considered metastable due to much of the failure material being removed, additional benching and rock scaling work is required to improve risk from predominantly individual rock falls off the slope above.

- Materials recovered from this site will be used to either build the MSE reinforced fill at Dropout 2 if the Paragrid / gabion option is used or taken through to Dropout 1.

- NIWA Q_{100} runoff from the nominal 0.2km^2 catchment is modelled as 3.6m^3/ s. The upstream area is nominal 0.2km^2 and not that much smaller than the area above Dropout 2 & 3 which have a significantly larger drainage outlets.

- Two culverts are recorded on the KR culvert log, being a 650* 1150 box culvert at 356.773km (Q= 2.2m^3/s with1.8m head) and a 760*760 box culvert at 356.847km (Q=1.5m$^{3/}$s with 2m head).

- Technically there is enough drainage capacity for the Q_{100} event with the available culverts however they are offset from the main drainage channel and prone to significant blockage and over topping from debris being carried down from above.

- We have provisionally allowed for a new single set of 1.7m arch culverts at relatively shallow level under the track formation more or less opposite the main drainage channel down into the site. The exact location for this will need to be determined on site and the proposed system subjected to more rigorous detailed design assessment.

5.5 Task 11 – Former Tunnel 24.

Tunnel 24 was day lighted in 1956 / 57 on account of significant distress in the tunnel lining caused by ongoing downslope movement in the seaward side of the then ridgeline. The track was diverted slightly to the east around the outside of the then tunnel centreline. Large volumes of excavated spoil were bulldozed and pushed southward in front of large rock bluffs and to the north. Several historical photographs of the daylighting works are included under Tasks 11, 12 & 13 in section 2.

There has been ongoing slow settlement in the realigned track formation since daylighting that has required periodic track realignment and packing for level and line to address twist faults and settlement.

Two slope movement areas are visible on site and marked up on a drone photo in the appendices. Measured deformation is in the order of nominally 300mm vertical and 100mm horizontal over 7 years, translating to annual movement in the order of 40mm and 15mm respectively at both locations.

Movement at the northern end of the site appears to be the same movement that caused the original tunnel deformation and is inferred to be ongoing slow movement running down underlying bedding dip to the north, failing on a bedding plane shear surface at depth. The inner edge of slope movement is directly visible under the track sets however it is difficult to locate its eastern (coastal) extent.

Movement at the southern end appears to be some ongoing down slope creep in a mix of shallow residual soils and the tunnel daylighting spoil placed on top. Movement magnitudes are similar to the northern side of the site but the direction is downslope to the east (refer appendices).

Ground conditions are further complicated by large scale slope erosion & steep failures extending well over 100m high from the beach on the eastern side of the site. In the medium to long term (or somewhat shorter with a significant seismic event), we expect coastal erosion will significantly impact the area adjacent to the track formation.

We have allowed for geotechnical investigations and stability modelling of the two areas as part of detailed design works. While the current solution of periodically lifting and packing the track is expected to be the ongoing solution and to continue, the underlying risks need to be better quantified going forward.

We have allowed to excavate out behind the remnant Tunnel 24 on the uphill side for additional backfill materials for Dropout 3 repair works (if required) as a start on any future track realignment in the area.

5.6 TASK 13 – DROPOUT 3 @ BEACH LOOP.

Dropout 3 is another deep infilled gully set between two significant rock bluffs, approximately 200 m north of Tunnel 23 at track meterage 355.57km. Dropout dimensions are in the order of 45m wide by 90m long and nominally 15 metres deep on the outer edge of track formation. Upstream catchment is some 0.3km^2 (similar to Dropout 2) with a relatively complicated and ineffective drainage system comprising a 1200mm culvert from the upper part of the gully infill fill leading down to a 0.9m*1.2m concrete arch culvert crossing the track at meterage 355.516km. Discharges from this culvert are progressively destabilising the immediate downhill formation slope and there is potential for a significant landslide event to occur somewhat greater than the current dropout volume. Current discharge flows are being directed into this culvert through a swale system built up in windrowed landslip & stream washout debris. There is no secondary or emergency drainage line apart from the windrow / sideways swale and this still uses the same culvert crossing under the track.

Top of rail level (TOR) is nominally RL 134m +/-. The formation fill is constructed from side and end tipped tunnel spoil excavated from the adjacent Tunnel 23 as well as from significant material excavated off the rock bluffs on either side. Base of the dropout on the outer formation edge is around RL120m +/- with the toe of the dropout at nominal RL90, some 60m to 70m downstream of rail centreline.

Material exposed in the washout is predominately excavated materials similar to the material described in Dropout 2 with competent, intact rockmass bedding visible on the northern facing.

The incipient landslide area mentioned above movement extends approx. 50 to 60m south of the dropout centreline and butt up against the southern edge of the replacement embankment. Ground movement is being driven by uncontrolled surface water discharging from the 355.516km culvert crossing the rail. The area is marked up in Task 13 in the section 2 (pages72 & 73) and is a combination of residual soils and side cast filling/ Tunnel 23 spoil being lubricated by water discharging from the culvert.

We have included photos from Dec 2013 and June 2019 of the culvert area. The culvert was inspected in 2010 by the report author as part of inspecting some landslip movement above the northern portal on Tunnel 13 during a general inspection at that time and looked similar to the 2019 photo, showing a nominal drop of 400mm to 600mm mm below the culvert with water discharging into the slope. To the best of our knowledge this was repaired by the ganger team relatively soon after the 2010 inspection and matches the works shown in the December 2013 photo. The 2019 photo indicates an additional 400mm plus movement from 2013 and is evidence of ongoing slow downstream creep caused by water build up in the slope soils.

The current FOS on this part of the slope is unacceptably low and the primary drainage system through the site needs to be reconfigured to avoid wetting up the fill slope and causing ongoing stability issues adjacent to and under the formation.

Required repair works are essentially the same as Dropout 2 and described as follows:

- Reshaping and significant benching out the washout area prior to installing either a 2V:1H (63 degree) sloping Paragrid reinforced MSE slope faced off with hybrid 0.5m high gabion baskets filled with local site soils or a vertically faced retaining wall structure backfilled with Paraweb reinforced GAP 80/ 100 greywacke derived hardfill.

- As with Dropout 2 both options have advantages and disadvantages; the retaining wall option has minimum benching and imported fill volume requirements to meet design requirements, using more expensive albeit substantially better performing materials. The Paragrid / gabion option has cheaper on site fill costs but requires substantially more excavation and fill volumes due to poorer engineering parameters and is more expensive in the current design iteration used for pricing.

- Both engineering solutions need to address incipient instability on the southern end – we have allowed to excavate & bench out at least 10m depth of insitu material and recompact this back into place with Paragrid reinforcement.

- Either option will need to be thoroughly keyed into underlying ground at the toe, rear and sides. Surplus excavated materials will be placed downstream of the MSE toe to provide additional toe support.

- Subsoil drainage comprising megaflo Ultra 300, Bidim geotextile and imported granular aggregates is required to maintain the internal stability of the MSE block as well as the external stability of the overall facing.

- NIWA Q_{100} runoff from the nominal 0.3km^2 catchment is modelled as 5.4m^3/ sec. The existing concrete arch culvert is 1.2m wide by 0.9m high and only capable of passing 2.1m^3/s based on a surcharge head of 1.2m above the invert. There are no secondary drainage outlets or swale drains to discharge additional flow into.

- Given the history of over topping failures in the immediate area from what is a relatively small catchment plus the incipient slope movement caused by culvert discharges, we have allowed for the following:

  - Reconfiguration of the primary drainage system from the current culvert to a new 1.75Ø arch culvert system, running at around 5m invert depth and discharging down the northern side of the MSE fill.

  - Water from this will be piped or further flumed down the slope.

  - The culvert will be benched into the intact rockmass on the northern side of the drop out and concrete haunched to lock it in place.

  - Excavation of a pondage area to provide for some future spoil volume from periodic culvert maintenance and cleanout; excavated material will be used as part of the Dropout 3 MSE fill or used to fill in the channel below the structure as toe buttress material.

  - The existing culvert system at 355.516km will be reconfigured to act as emergency bypass system with some consideration for additional culverting under the rail formation. Additional concrete channelling / fluming will be installed downstream of the culvert discharge to prevent water getting into the slope.

- The above replacement drainage scenario has been priced in the accompanying engineers estimate. We will relook at this area in detailed design; it appears feasible and more cost effective to use a steeply sloping smaller diameter PE line to get the required discharge volume provided we can configure the inlet to make maximum use of the smaller pipe volume, thread it downslope through the MSE fill construction and work out a way to mitigate significant water velocity at the discharge point.
- 30% drawings and cross sections are included in the drawing set, ref 1331. Indicative MSE fill volumes are in the order of 6500m^3 for the retaining option and 15,00m^3 for the Paragrid / site soils option.

5.7 Task 16 – Dropout 4 @ 353.95km

Dropout Four is further south in the upper reaches of the Tikiwhata Stream catchment at track meterage 353.95km. The dropout straddles a deeply incised gully (up to 30 metres deep on track centreline and some 90 plus metres wide at formation level) with an estimated embankment volume of some 75,000 - 80,000m^3 washed out in the 2012 event. Track level is at nominal RL 150m +/- with the base of the dropout at nominal RL 120 on track centreline and some 20 metres lower at the downstream end of the former embankment fill area.

The site sits towards the base of a deeply incised and moderately well vegetated upstream catchment having a nominal area of some 0.95 km^2 and a Q_{100} discharge of (rounded up) 18m^3/s. Dropout 4 is the largest failure requiring repair between Wairoa and Gisborne and as a general comment is significantly more difficult to engineer solutions for on account of the failure/ rebuild scale and difficult access for plant and materials using either road or rail.

The majority of previous embankment filling was excavated tunnel spoil from the adjacent series of Tikiwhata Tunnels to the south (Tunnels 19 to 22) along with some material from cuttings to the north and Tunnel 23. Additional volumes of tunnel spoil are visible further south in 1942 stereopair photos as an embankment fill between Tunnel 19 & 20 but this appears to have failed in a similar washout type event to Dropout 4 and was replaced by Bridge 267A at some date prior to 1967 aerial photographs.

The Dropout 4 failure appears to be caused by a large volume of ponded storm water overtopping the track formation and washing out the embankment filling; it is also probable that a significant component of saturated mass movement also occurred during the failure event. Deposits of outwash material seen in 2013 inspections below the site indicates the material was essentially liquefied during failure with substantial material "flow" deposited on the opposite downstream bank well above stream level. We consider that the ongoing partial culvert blockage and elevated inlet levels will have contributed to creating a significant groundwater profile through the embankment over multiple decades and this would have been a significant factor in contributing to the likely failure mechanisms seen.

The existing tunnel drainage sits some 30 plus metres under the ridgeline to the north and comprises an upstream 29m length of arched top box culvert 1.95m high by 1.22m wide with a nominal 70m length of 2.5m dimensioned tunnel excavated through solid rock mass on the downstream end. The culvert has a nominal discharge capacity of 23m^3/s at 9m head which is above the Q_{100} discharge required – 18m^3/s capacity is around 6m of head. The culvert system is in good condition for its age and apart from inlet works is able to be directly reused.

The upstream inlet appears to have been buried by 3-4 of metres of stream debris over time based on 1986 photos with a steel pipe riser visible at the upstream end during site inspections. The steel riser within the inlet is further blocked by an additional 3 – 4 metres of landslide debris which we infer occurred early on during the 2012 event causing the water to build up behind the embankment and cause its eventual failure. Total burial depth is in the order of 6 – 8 metres at the upstream end, working off a nominal intake level of RL 127m +/- (Outlet is around RL126m)

We have put considerable thought into repair options for Dropout 4, ranging across multiple design iterations containing bridging solutions and multiple embankment type constructions including vertical retaining and sloping MSE solutions, imported and local backfills and variations and combinations of the same. Delivery options around using rail and overland access have also been considered.

Constraining factors around producing the optimal solution for repairing Dropout 4 include the following:

- Rail access is complicated by dropouts on either side as well as the need to rebuild several hundred metres of track within the Beach Loop area to the north prior to accessing the site by rail.

- Access from the south is relatively straightforward once Dropout 5 & 6 are repaired but there is limited laydown areas between Tunnels 22 & 23 and direct access into the site is through Tunnel 22 and down a 30m high sub vertical facing.

- Access from the north requires Dropouts 1 – 3 to be repaired and new rail to be installed as well as having limited laydown areas available and complicated by equipment access across three rail bridges between Tunnel 23 and Dropout 4 (Bridges 269, 270, 271).

- Overland access is available into the site through the HFF Forestry Block to the north and along the old PWD access road to Tikiwhata Camp and down into the southern end of Tunnel 23 through Paritu Station.

- Additional tracking is required to get earthworks equipment down to and across Tikiwhata stream and into the base of the site. A HNHO72 design standard bridge is also required to get materials into the site.

- There is access available through Paritu Station to the top of the ridgeline directly above Tunnel 22 – there is an old location on the ridgeline where concrete was skipped down to the tunnels below. There is additional quad bike type access down to the Tunnel 23 portal but the track is steep and would require significant upgrading works to use (and the exit point is still 30m above the base of the dropout).

- Various bridging options have been discussed with Novare Design looking at multiple longer SPG spans and a through truss or base truss type design to extend pier centres. Both options are complicated by the need to build multiple centre piers up to 30 meters high with limited crane and concrete access as well as the constraints around installing steel work at height. Steel spans are considered preferable to concrete ballast deck due to crane lifting and general access difficulties. A bridge <u>could</u> be fully built across the dropout but our considered engineering assessment is that a full bridging solution is slower and more expensive than the embankment options outlined below and has ongoing maintenance issues.

- For full embankment options the design structure needs to be sized as an earth dam in the event of additional / future drainage blockage and a build up of water on the upstream face. The embankment also needs to be able to handle a future over topping event without failing catastrophically as has currently occurred.

- A 1.75Ø arch culvert emergency bypass drainage line some 90m to 120m long is required for any embankment options to cover future blockage events; substantially shorter culvert lengths are required for part embankment options (see below). Variable culvert lengths are related to installation level and where it is discharged – either at the base of the embankment filling or discharging out into the cut back ridgeline immediately to the northeast of the dropout (refer plans). This culvert is designed to handle the high fill loads present within the depth of embankment filling and has a nominal 18.5m^3/s capacity at 5m head. The 1.75Ø culvert design was undertaken specifically for this dropout site but has been used elsewhere along the formation to get the unit cost lower.

- We have looked at knocking down the ridge line above Tunnel 22 and pushing it into the top part of the dropout and then reworking the material into either a bulk unreinforced fill solution or a MSE fill reinforced with Paragrid geogrids on the downstream portion of the site. The issues with this are getting the local site soils to behave under compaction, especially if there is a wet summer; there are limited options to dry out fill materials at the base of the incised gully. The cohesive fill option will require significant internal chimney and side cut off aggregate drainage to control pore water build up as part of the MSE "dam" design which adds complication and cost into the build. Indicative volumes are in the order of 75,000m^3 of local site soils and some additional 9000m^3 of imported aggregates and drainage materials.

- We note that there substantially better ability to use local soils for dropouts 1 – 3 as these site materials can be conditioned in cut as well as blending to meet design requirements. These sites are well exposed to the weather allowing for better drying back after rainfall.

- The second embankment option is to construct a hybrid solution involving a vertical faced MSE wall some 24m high (from RL 116 to RL 140 +/-) built from imported GAP80/ 100 aggregate reinforced with Paraweb topped off with a 10 m high 1:1 Paragrid reinforced site soil embankment directly under the rail formation. Some 35 – 36m of the reinstatement length at the southern end will include a short section of low height anchored block walls and two second hand 12.2m railway bridge spans spanning across to the embankment to minimise fill volumes. This option would have a much shorter 1.75m arch culvert length in the order of 40 metres long.

- Local cohesive soils will be compacted on the upstream face to act as a seepage cut off for any seepage under flood surcharge loadings. The downstream aggregate will act as its own underdrainage system and vertically facing the MSE allows for much better efficiency in terms of geogrid performance and limits the amount of imported granular material required.

Based on the assessment constraints and significant optioneering around construction costs and construction programs, the recommended repair works at Dropout 4 is the hybrid embankment solution of two short bridge spans and a MSE reinforced embankment as outlined above. Additional comment on this option includes:

- The two, 12.2m second hand steel SPG bridge spans at the southern end of the dropout start about 12 – 15 metres out from the Tunnel 22 portal edge and extend some 24.4 metres north to just past the RL140 contour on the southern dropout slope.

- There are currently 10 of these spans sitting in the Hamilton Yard with all of them in a reasonable state of repair. Spans will be founded on shallow concrete foundations anchored down onto the underlying rock mass with the northern abutment founded on a nominal 10m high MSE retaining wall built on the end of the 1:1 Paragrid reinforced slope formation embankment.

- Using these two spans avoids having to extend the embankment filling an additional 50 or so metres further down the dropout base to provide a formation edge where the track alignment runs immediately north of Tunnel 22

- We have looked at building an anchored retaining wall as an alternative to the two bridge spans; the wall would be up to 10 metres high with multiple rock anchors to hold it in place and need to be designed to significant static and seismic loading. Our cost estimates indicate the bridge spans are going to be substantially cheaper than retaining, particularly with the access and block handling costs around constructing a 10m high wall starting some 20 metres up a very steep rock slope.

- Using the two bridge spans also allows for a third emergency bypass drainage system in that an overflow spillway with huge flood capacity can be set at nominal RL138 – 140 underneath the bridge itself.

- Having the spillway option also substantially simplifies the embankment design – there is less requirement to consider dam design requirements under extreme events which in turn has savings on embankment sizing / volumes and similar.

- There is a cost / time trade off using reinforced vertical walls vs steep reinforced slopes or flatter unreinforced slopes. The biggest constraint on the site is the ability to transport sufficient aggregate into the site to form a well drained structural embankment on the downstream face; using wall elements minimises the volumes needed with lower reinforcement quantities and the ability to get the short bridge / spillway as a viable construction solution at least cost and construction risk.

- The MSE wall will be significantly keyed into the base and side slopes and arched around a nominal 30m radius in plan view to better lock into the surrounding rockmass. Paraweb reinforced aggregate backfill will also be benched out into a wider wedge shape to further lock the structure into the dropout area.

- Subsoil drainage comprising megaflo Ultra 300, Bidim geotextile and imported granular aggregates is required to maintain the internal stability of the MSE reinforced block as well as the external stability of the overall embankment.

- Downstream MSE filling will comprise imported GAP 80/ 100 aggregate from Matawai Quarry brought into the area via the upgraded PWD road and delivered into the construction site with 30 tonne articulated dump trucks.

- The RL 140 to 150, 1:1 formation embankment and upstream filling will be cohesive residual soils and rockmass taken from the trackside ridgeline immediately to the north above the stream. There is approx. 20,000m^3 of material in this area, sufficient for the upstream embankment section.

- Prior reporting indicated walkways on Bridge 267A & 268 (between Tunnels 19 & 20 and 20 & 21) would need to be installed to facilitate safe access for rail operating personnel using rail supply options. To some extent this has been superseded with the change to predominately overland delivery however they have been left in current construction estimates as the new bridge (268A) will have walkways installed and it make sense to upgrade these at the same time.

- 5 – 8 metres of site excavation and reconfiguration of the upstream channel will be required to reinstate the existing culvert inlet and installation of multiple upstream rail gates are required to limit it being blocked with trees and vegetation.

- Installation of a high level culvert inlet (nominal 1500/1800mm manhole riser with a scruffy dome top) or similar is required to provide a high level inlet in the event the culvert becomes partially blocked with flood debris building up on the rail gate protection.

- Excavation /creation of a pondage area to provide for some future spoil volume from periodic culvert maintenance and cleanout.

30% drawings and cross sections are included in the drawing set, ref 1331. Indicative MSE fill volumes are in the order of 20,000m^3 of imported aggregates and some 13,000m^3 of local site soils.

5.8 Task 21 – Railway Road Culvert @ 349.32km

Significant logging and forestry works are underway in the 2.05km^2 catchment upstream of the Railway Road Culvert at track meterage 349.32km. The KR culvert log advises there are two culverts at this location, being a 20m long base level concrete arch culvert at nominal 10m below TOR with dimensions of 1.3m wide by 2.0m high and a second 1050 diameter culvert pipe set 3m below TOR. The base culvert is in generally good condition although the outfall has scoured back into the slope by a couple of metres since construction in the late 1930s with a significant scour hole on the outlet area and progressive undermining still taking place. Aerial photos and site inspections indicate the culvert was tunnelled through the ridgeline to the north of the then stream with the majority of the gully backfill sitting under Railway Road to the south (refer photographs in appendices).

Inlet structures on the base culvert are in a poor state of repair with no secondary upstream rail screen to further protect the culvert from logging slash damage and blockage. There is also significant vegetation growing in the stream channel and access to the culvert face is poor. Given the adjacent forestry operations we consider there is a significant risk profile around this culvert with the potential for it to be blocked by logging slash and a major washout event to occur.

The upstream catchment area of 2.05 km^2 has a nominal NIWA Q_{100} flood estimate in the order of 37m^3/s. Capacity of the 1.3m wide base arch culvert is in the order of 28.5m^3/s with 8m of surcharge head. An additional 4m^3/s capacity is available within the 1050Ø pipe at 3m head.

Arguably there is just about enough drainage capacity for the 100 year design requirement however we expect a major logging slash event will substantially block the underlying culvert (to say 50% of capacity even with rail protection) and the following is recommended:

- Excavation and inlet protection around the existing base arch culvert inlet and installation of an upstream rail gate to prevent it being blocked with trees and vegetation plus significant upstream vegetation clearance.

- Installation of a high level culvert inlet (nominal 1500/1800mm manhole riser with a scruffy dome top) or similar box structure to provide a high level inlet in the event the culvert becomes partially or fully blocked with flood debris.

- Installation of a single or double set of 1.7m dimensioned arch box culverts set nominally 5 m below TOR to act as an emergency overflow as well as provide additional Q_{100} flood capacity. The arch culvert has nominal 18.5m3/s capacity at 5m head which is around 50% of Q_{100} flows. This will be excavated in rockmass following the line of the tunnelled culvert below, will replace the current 1050Ø pipeline and will discharge over the rockmass slope above the existing outlet.

- At this stage one set of arch culverts have been allowed for in attached estimates. This will be revisited in detailed design as we get access into the site and assess the current state of the asset – assumed to be in good condition.

5.9 Task 22 – Dropout 5 @ 349km

Dropout 5 appears to mostly be scour failure and shallow formation washout due to blocked swale drainage behind the track. Approximate extents are 15m long by 3m wide by 1m high. A gravity / MSE concrete block wall designed for the track surcharge load is proposed, founded on competent fill visible in the base of the washout. Additional swale drainage works are covered under Item 36 in this area.

There is ongoing scour on the outer dropout face. Longer term erosion solutions will need to be addressed as part of detailed design.

Access into the site will be overland, via Railway Road.

5.10 Task 23 – Dropout 6 @ 347.73km

Drop out six straddles a moderately incised gully with a nominal upstream catchment area of some $0.3km^2$, similar in size to dropouts 2 & 3. Washout dimensions are in the order of 30m wide by 40m long by 10 – 12 metres deep.

The site is known to have had several washout events with a recent repair incorporating twin 600mm culverts sighted by the report author during the mid 2010 Tunnel 13 slip inspection visit outlined in section 5.6 above. This appears to have failed in late 2014 / 2015 as access was available across the fill embankment during our 2013 & 2014 reconnaissance visits.

Immediately downstream of the dropout the railway line curves around a large rock bluff. This bluff is failing down dip out towards the river but appears to be relatively stable at track level based on geological observations to date. The upper part of the bluff appears to have been accessed a couple of times to provide material to backfill the dropout area; there is significant potential for additional movement upslope of the borrow area to move downwards and excavating additional material out for backfilling will need to be assessed during detailed design. More details are show on marked up drone photographs in the appendices.

The existing blocked and buried on site base culvert is not recorded on the KR culvert log and indicative dimensions are not available due to the outlet being buried. This culvert is set relatively low at both ends and may be influenced by back water effects if the adjacent Kopuawhara River is in large flood. The base culvert is considered suitable for a low level bypass during construction but given the multiple failure history of the site, we have allowed for a single set of 1.75Ø arch culverts to be installed at a higher level as permanent drainage for the Q_{100} event.

The upstream catchment area of 0.3 km^2 has a nominal NIWA Q_{100} flood estimate in the order of 5.4m^3/s. The proposed 1.7 arch box culvert has a capacity of 5.4m^3/s at 1.2m head depth above invert.

Required works at dropout 6 are as follows:

- Construction of a nominal 12m high Paragrid reinforced MSE slope fill on the downstream face using Nuhaka quarry aggregates with local site soils used on the upstream face to minimise imported fill volumes.
- No rockmass is likely to be exposed at the toe – fill will need to be keyed a couple of metres deeper for scour protection from the adjacent Kopuawhara river. Some reworking of larger rock in the Dropout 6 washout materials within the Kopuawhara riverbed may be used for upstream river / erosion protection.
- Subsoil drainage comprising megaflo Ultra 300, Bidim geotextile and imported granular aggregates is required to maintain the internal stability of the MSE block
- Imported aggregate from the Nuhaka quarry supply will be brought into the stockpile area and discharged from side dump wagons into stockpile. This material will be further placed with a loader into the dropout and compacted in place.
- Deep excavation and rail inlet protection is required around the existing culvert inlet and installation of an upstream rail gate to prevent it being blocked with trees and vegetation. The stream channel upstream of the existing base culvert will need to also be excavated down to restore the original levels from 1940 +/-.
- Installation of a single set of 1.7m dimensioned arch box culverts to act as an emergency overflow as well as provide additional Q_{100} flood capacity.
- 30% drawings and cross sections are included in the drawing set, ref 1331. Indicative MSE aggregate fill volumes are in the order of 4000m^3 with a nominal volume of 3000m^3 for cohesive materials placed on the upstream shoulder.

Detailed design will relook at the adjacent bluff stability and reconsider if sufficient material is available to build the embankment out of local site soils while minimising imported fill requirements.

5.11 TASKS 27 TO 34 – WAIKOKOPU – OPOUTAMA SEAWALL AREA 334.0KM – 335.5KM.

Required works along the Waikokopu – Opoutama coastal section include repairs to the northern abutment of Bridge 262 (Task 27, 335.43km), remedial works on a landslide falling down into the formation area (Task 29, 335.05km), numerous seawall repairs (Tasks 28 and 30 -32) some landslide / coastal erosion works (Task 33, 334.55km) and some rock slope scaling work (Task 34) from 334km to 334.38km to mitigate existing rock fall hazards at the Waikokopu end of the site.

The original 1.5km of coastal works between Waikokopu and Opoutama were constructed between 1937 and 1939 and involved construction of multiple concrete seawalls, significant earthworks, benching and cutting back of rock faces to form track formation areas and upslope stabilisation works as well as building two ballast deck bridges – Bridge 261 at Waikokopu and Bridge 262 at Opoutama.

Based on a review of aerial photographs and historical information the majority of the seawalls were directly cast against sandstone / siltstone rockmass exposed along the foreshore with a couple of minor areas where walls were constructed across inlets and have more extensive wall backfilling derived from local soils and rockmass. Construction photographs indicate the structures are predominantly mass concrete with relatively light steel reinforcement used to form the rolled lip along the top wall edge.

Concrete walls were founded on nominally embedded shallow concrete footings excavated into the underlying rockmass. Typical wall dimensions are in the order of 400mm to1100mm wide with heights of between 1.6 and 3 plus metres.

Substantial additional coastal protection works were carried out in the 1960s & 1970s involving the placement of hundreds of concrete backfilled type L and LA 4 wheel railway wagons to protect additional areas of the coastline from wave erosion. Typical "wagon" concrete block sizing is in the order of 4.8m long by 2.2 m wide by 1.1m high – nominally 11.5m^3 of concrete with a mass of some 27 tonnes +/-. There are also significant numbers of "half wagon blocks" at nominally 2.4m*2.2m *1.1m with 5.8m^3 of concrete / 14 tonne capacity +/-. Both of these blocks appear to be stable under the existing wave environment with little evidence of block settlement / movement apart from that caused by under scouring or adjacent erosion.

While the relatively elderly concrete walls themselves are in quite good condition for their age, at least in terms of concrete performance, there are significant areas of wall undermining (up to 1 – 1.2m deep under multiple wall sections) where the rockmass has scoured out over the past 80 years.

There is also evidence of some increasing overtopping erosion on south east facing wall sections. Aerial photos from 1938 onwards show variable movement with the boulder banks in front of the walls (majority of these are tucked in behind wall sections protecting them from direct southerly wave activity). There is also some variability in the amount of exposed rockmass on the foreshore platform over the past 80 years although much of this is likely to be an artefact of tide times in various photograph sets.

The walls themselves appear to have limited geotechnical capacity to accept additional fill surcharges from large rock or similar placed on top of them; there is a couple of 6m long wall panel failures at meterage 335.220km where rock placed for scour protection has pushed the panels outward although this is complicated by evidence of toe erosion at the base and overtopping wave activity already having scoured out the wall back face prior to rock placement.

Based on our visual appraisal of the current situation, our engineering assessment is as follows:

- The existing seawalls have sufficient concrete capacity for several more decades (30 to 50 years), contingent on the toe erosion and base scour being repaired as a matter of urgency with some additional wall height increases to limit overtopping and scour behind the walls.

- Base scouring can be repaired with a combination of cast insitu toe beams anchored down into the rockmass using grouted fibreglass anchors and large geotextile bags placed under the walls and pumped full of concrete and similar such measures.

- Between 800mm and 1.5m of additional retained height is required in a couple of relatively short wall areas, primarily on outside wall points directly facing to the south east to direct overtopping wave sets. Given the limited geotechnical surcharge capacity within the existing walls, precast blocks should be used, concreted to the underlying seawall with epoxied fibreglass starters and tied back into the ground behind using hollow bar full cement grouted anchors.

- The extensive areas of railway wagon block seawalls are also in good condition for their age and while they are somewhat unsightly, the concrete blocks are doing a good job of coastal protection.

- We estimate these wagon block structures will have a similar future life in them of nominally 30 to 50 years. Areas of steel will continue to corrode and be washed away however the concrete is expected to remain.

- There are a couple of areas where sideways erosion is working in behind the end of the wagon block walls. Several large geotextile bags of concrete should be used to infill these areas to limit further wave penetration.

- There is an area of land slippage between track meterage 334.48 and 334.56km (Task 33 in the task list). The track is kinked in this general location and the area is known as an ongoing maintenance area, requiring periodic tamping and track realignment. The toe of the landslide is at or below the low tide level with the upper slip extent extending up to 200m inland and over 30m in elevation.

- The outer extent of the landslide is protected by 2 to 3 rows of wagon blocks however ongoing slope movement has pushed these down and seawards. Waves are able to break over these at mid to high tide and the area behind is eroding, further destabilising the landslide toe.

- The toe of the landslide in behind the wagons requires buttressing with large rock to prevent further erosion as well as provide some additional resisting mass to improve landslide FOS values. The wagon blocks provide a reasonable outer edge to build from.

- Some geotechnical investigation is required to assess groundwater levels, better model the slope movement and adequately size the rock toe volume and well as consider if any improvement would be obtained from counterfort drainage. Several additional wagon blocks would also be helpful, although we understand no old wagons are currently available.

- The landslide at 335.05km (Task 29) is failing in front of a house on Opoutama Road and appears to be driven by road culvert discharges. Repair works will need to take the upper slip area in front of the house into account as there is potential for triggering additional slope movement adjacent to the property as part of remedial rail works.

- The northern abutment on Bridge 262 requires some short areas of concrete block walling and large rock to address some ongoing scour issues.

5.12 Task 35 – Blacks Beach Road Subsidence 331.4km

There is significant distress on the Opoutama Road alignment at track meterage 331.4km with the road reduced to a single lane and encroaching well into the railway corridor for vehicles to get past. This area will need to be remedied to allow the line to reopen.

The road site has a known history of geotechnical issues stretching back over 30 years and has been the subject of multiple investigations and repair efforts over that time. Road works in the immediate area include construction of a significant MSE retaining structure in the early 1990s subsequently anchored and restrained with a piled / anchored ground beam in the early 2000s.

The most recent movement occurred in mid-2017 and has been subject to additional reports and engineering solutions commissioned by the Wairoa District Council (WDC). The preferred solution is to excavate some 25,000m^3 of material off the slope uphill of the railway formation and move the road and rail corridor nominally 8 metres back into the hill.

WDC has had numerous discussions with KR and has agreement in principle with KR and affected landowners. There is currently an outstanding peer review from KR undertaken by Holmes Consulting Group that needs to be resolved with project stakeholders.

We have reviewed the available geotechnical reports and KR peer review document. The main outstanding issues are:

- confirmation of the underlying ground model causing failure in the road (we note the failure is in underlying rockmass complicated by faulting and high groundwater)
- How far back into the slope the movement surface is in relation to available stable ground and the relocated road and rail formation positions.
- Consideration of the long term impact of coastal erosion at the toe and what sort of design life is available before the road and rail require additional relocation; it may be better to allow for additional retreat works as part of this project to ensure a robust long term solution is available.

To move the road relocation project forward as part of rail reopening works we recommend that a series of machine boreholes are drilled to confirm the underlying ground model and then the design can be finalised based on these results.

We confirm that moving the rail and road back into the slope as proposed by WDC is considered the best option under the current circumstances. We note that there is a more complicated option of dropping the rail alignment into a box cut slot with the relocated road running over the top on a series of backfilled culvert arches which would either minimise the upslope excavation volume or allow for an additional 5m+ of sideways road relocation retreat depending on the ground model results. Dropping the rail into a 5 – 6m deep slot would also allow for some grade easement on track formation coming from the Waikokopu direction.

We also note that there is some opportunity to select larger rock fill out of the nominal 25,0000m^3 of slope retreat excavation required as part of the WDC solution which could be used as backfill for the coastal slip buttressing in Task 33 between track meterage 334.5 and 334.65km.

We recommend that these works are considered as part of the overall works package to reinstate the rail formation between Wairoa and Gisborne.

6 GEOTECHNICAL GROUND MODELS

6.1 30% GROUND MODELS

Engineering geology ground models are based on the following:

- Ground & dropout repair model development utilising historical site information (where available) and engineering / geological observations from the immediate area including preliminary rockmass logging, strength assessment and preliminary back analysis around existing slope movement & Dropout features.

- Underlying geology for the Waikokopu – Opoutama Beach track section, upper Kopuawhara Valley / Tikiwhata stream / Beach Loop coastal sections & intervening hill country sites is Mid Miocene age, Tunanui Formation rockmass comprising alternating sandstones & siltstone/mudstones. Residual soils are a mix of sandy silts with broken/ weathered rockmass & colluvium forming a substantial component of the residual soils veneer.

- Geology within the lower Kopuawhara Valley comprises outwash silts, sands, gravels and boulders derived from the hill country geology described above. There is a significant river gradient change opposite nominal track meterage 342.5km where the river channel becomes incised into a series of finer grained outwash materials and the boulder content of the river bed drops off. The lower 7 – 8km of the track formation towards Opoutama traverses a deep profile of alluvial and estuarine soils including sand dunes, buried beach deposits and organic soils.

- Railway embankment & sidling fills predominantly comprise a mix of residual soils and excavated rockmass materials excavated from adjacent cuts and tunnel excavations from the surrounding Tunanui formation rockmass as described above. Broken rockmass content is reasonably high and this tends to push up design parameters by 3 – 4 degrees above adjacent residual soils. Embankment fills are generally loose and uncompacted and were usually placed as side or end tipped materials.

- Within the Kopuawhara Valley floor from 335.3 to 341.2km the low height railway formation & embankments appear to be predominantly constructed from a mix of Nuhaka quarry gravels and deep ballast lifts. Some sand fill may also have been used. We expect formation to have been placed over timber fascines to provide bridging over softer and organic soils where required in this area.

6.2 DESIGN PARAMETERS – 30% ENGINEERING DESIGN

Provisional engineering properties used in the MSE retaining wall and RESSA retained slope designs are as follows:

Table 2–Engineering Design Parameters

Summary of Soil / Rock Engineering Design Parameters				
Engineering Unit	**ϒ (kN/m³)**	**c' (kPa)***	**Ø' (deg)**	**Su**
Underlying Tunanui formation rockmass –cemented sandstone and siltstone	20	40	40	>>200
Transitional zone stiff / rubbly soils	18	10	34	>100

Residual soils	17	5	30	100
Alluvium – AZ & ZA (Kopuawhara Valley floor)	17	3	28	>40
Opoutama seawall backfill (predominantly beach gravels and cohesive cut materials from adjacent rail formation works)	17	3	30	>60
Railway embankment filling – predominately loosely compacted or uncompacted end tipped material derived from tunnels and cuttings in Tunanui formation materials (note higher friction angle derived from larger rock in places)	17	3	34	>60
Site derived local cohesive and weathered / excavated rockmass soils for MSE Structures	17	3*	30	>100
***	Note MSE designs using site derived fills include a Ru pore pressure coefficient of 0.2 to model the effect of groundwater pore pressures during placement, compaction and long term performance.			
Imported Nuhaka Quarry bulk quarry run gravel & sand filling for MSE structures	18	0	35	N/A
Imported GAP 65 / 80 /100 from Matawai and / or Ruatoria quarry sources	20	0	38	N/A
Notes: Seismic soil class taken as B. Engineering values interpolated from available regional data Ƴ = unit weight; c' = effective cohesion; Ø' = effective friction angle; *Cohesion in bulk gravel and reinforced block ignored in MSE retaining wall & reinforced slope design				

6.3 SEISMIC DESIGN – STRUCTURES & REINFORCED SLOPES

The project is located on the East Coast of the North Island, approximately 50 kilometres east of the Hikurangi Fault margin, the largest and one of the most active fault zones in the NZ region with the potential for large magnitude earthquakes up to 8.9 and extended periods of ground shaking. We are aware of significant work being undertaken on fault movement and ground shaking/ tsunami risks (The East Coast LAB project being run in conjunction with GNS Science) which has good data around anticipated ground shaking results but somewhat less information around expected PGA and other relevant design parameters.

For the purposes of this report, seismic design is based on NZS 1170 (2016), NZTA Bridge Manual requirements (3rd Edition 2016) and MBIE / NZGS Earthquake Geotechnical Engineering Practice Module one requirements (2016).

We have allowed for the following in terms of structure seismic design:

For the 6 major dropouts and Opoutama Seawall repairs supporting track & formation structures sitting on or close to Tunanui group rockmass:

- NZBM – Table 2.1 Importance Level 3 (level 3 pushes the ULS design requirements from 1:1000 to 1:2500 year return period event) which we consider is a more realistic design acceleration for modelling ground shaking from the adjacent Hikurangi fault margin
- NZS 1170 - class B soil conditions due to underlying rockmass
- 1:2500 year ULS design 0.55g, 1:1000 year 0.4g. 1:100 year SLS design 0.1g. (In practice the ULS design parameters govern MSE design)

For the balance of structures including drainage design and H pile retaining walls:

- NZBM – Table 2.1 Importance Level 2 (these structures not critical to track formation and can be repaired relatively easily)
- NZS 1170 - class C soil conditions due to underlying embankment fills
- 1:1000 year 0.4g. 1:100 SLS design 0.1g.

Review of multiple reports around Hikurangi Fault earthquake shaking scenarios including liquefaction studies undertaken for Hawkes Bay Regional Council and relationships between anticipated MMI shaking and PGA generally supports a nominal PGA of 0.5 - 0.6g for high magnitude earthquake shaking. We consider the 0.55g for ULS design above to be a realistic design number, albeit somewhat higher than currently required within available design codes.

6.4 Load Factors MSE & RESSA Design

MSE reinforcement design for Paraweb reinforced retaining wall design options using imported aggregate fill on Dropouts 2, 3, 4 & 6 is undertaken using MSEW (3.14) software which follows the design guidelines of FHWA-NHI-00-043(2001) and AASHTO 2007-2010 for LFRD design. The LRFD design follows the AASHTO LRFD Bridge Design Specification (5th Edition, 2010).

Paragrid reinforced local soil slope options on Dropouts 1, 2, 3, 4 & 6 are designed with MacStars software supplied by Maccaferri Ltd and cross checked by RESSA software following the same design guidelines as MSEW. Both MSEW & RESSA software packages are developed by Adama Engineering LLC in the USA and are used globally for the design of reinforced soil structures and walls.

MSEW wall designs have been further checked with RESSA software to check slope & structure stability (particularly under seismic) and reinforcement lengths adjusted where necessary to comply with both MSEW design requirements and RESSA stability analyses.

Design of MSE Walls & RSS slopes has been undertaken on the basis of the following:

- Railway traffic loading of 80 kPa acting over the horizontal sleeper distance at the base of ballast on a 2:1 V: H load spread into the embankment - in practice the static and seismic embankment fill loads govern the design with the live railway loads on top relatively small in comparison.

- Retaining wall & reinforced slope design under ULS Case 1A & 3A assumes fully drained ground conditions within the MSE and reinforced soil block.

- For embankments built with local cohesive soil a nominal Ru of 0.2 is applied.
- Paraweb 2D50 geostrap reinforcement & Paragrid geogrid reinforcement designed on the basis of a durability reduction factor RFd = 1.08; installation damage reduction factor RFid = 1.20 to allow for larger sized GAP 80/ GAP 100 aggregate, and creep reduction factor RFc = 1.38.
- ULS (1/2500yr) acceleration of 0.55g
- SLS (1/100yr) acceleration of 0.1g (not checked directly as ULS design governs)
- Maximum horizontal deflection under ULS acceleration of up to 100mm is allowable per section 6.6.9 of BM3.
- Maximum vertical deflection is limited to 40mm in section 6.6.9; Vertical deflection is not expected to be a significant design issue in terms of embankment fills however some track settlement will occur.

For Dropout 4 - Paraweb reinforced block walls:

- Stack / running bonded retaining wall units used to minimise the downstream toe extent of Dropout 4 and take up as much of the 32m high retaining height as practicable.
- Paraweb 2D50 reinforcement strips are structurally connected to the rear of the Stone Strong retaining wall units and extend back into the aggregate compacted backfill zone. The Paraweb strips provide a long term design strength value of 32.8kN per individual strip and 65.6kN for each 2D50 block connection.
- Paraweb connections are between 6 and 8 connections per block.
- Static and seismic lateral loads on the wall is resisted by frictional resistance between the Paraweb reinforcement and select angular granular backfill, such that strip pullout, strip rupture and MSE block direct sliding failure mechanisms are designed to exceed the maximum earthquake lateral force from the analyses.
- The Paraweb strip to soil pullout capacity, strip tensile capacity (rupture) and the capacity of the Paraweb connection to the Stone Strong block have all been designed to exceed the ULS earthquake load to ensure that the failure mode is by block sliding or global instability where less than 100mm of permanent displacement is predicted to occur.

6.5 Durability & Design Life

All reinforced slope and wall components have a design life of 120 years, based on BBA certification supplied by the manufacturers. Blocks are rated to a minimum 100 year design life in terms of NZS 3101 but are expected to exceed the 120 year design life of the adjacent reinforcement.

7 SITE HYDROLOGY & CULVERT DRAINAGE DESIGN

Runoff flood modelling has been taken from the flood frequency tool on the NIWA flood modelling website (https://niwa.co.nz/natural-hazards/hazards/floods). The website provides results from three different modelling approaches including H-C18, HC_{se} and the Rational Method using regional flood rainfall / runoff models calibrated with actual flood records, empirically derived modelling approaches and similar.

There is reasonable agreement between the three analysis models at larger catchment sizes (generally greater than $2km^2$) while results from smaller catchments ($0.3km^2$) vary quite significantly. For the purposes of this report we have combined the modelling approaches for some 9 catchments straddling the rebuild area and averaged out the Q_{100} results to smooth out the variability seen in the smaller catchment results.

Catchment areas have been cross checked against the NZS260 series of 1:50,000 topographic maps with the areas used from these in preference to the NIWA catchment models.

Existing culvert capacity has been derived from tables and methodologies outlined in the Concrete Pipe Association of Australasia (CPAA)" Hydraulics of Precast Concrete Conduits Manual" (2012). New and replacement culverts have also been sized using the same document.

We have costed supply of box culverts into Gisborne as part of the rebuild exercise. It is substantially cheaper to cast box culverts locally even with the cost of formwork put into the project and a 1.75Ø arch culvert shape capable of handling 30 metres of fill surcharge was developed as part of earlier iterations on Dropout 4.

In terms of design we have allowed for the following:

- We have modelled the Q_{100} runoff as $18m^3/s$ per km^2 of catchment. The nett result generally lifts the smaller catchment discharges, which, given the failures needing rebuilding is considered appropriate.

- We have allowed for the Q_{100} event to be passed through or under the various repair features using a combination of new and existing drainage. We have considered partial blockage on some of the underlying culverts taking into account what is happening in the upstream catchments (e.g. logging at Railway Road) and adjusted the additional capacity estimates required.

- All existing base culverts have upgraded inlets to provide multiple ways to get stormwater flows into the underlying drainage system. These have been cost modelled as larger diameter manholes and insitu concrete work however these will be revised at the detailed design stage to consider if other options are appropriate.

- The drainage system at Dropout 3 needs a complete rebuild and new location to avoid discharging water into incipient landslide movement below the track culvert at 355.515km.

8 APPLICABILITY & CLOSURE

Site reporting and the preliminary 30% designs outlined in this report are based on our analysis of engineering geological and geotechnical site assessment data and site observations from previous & current work in the rail corridor and to the best of our knowledge are based on a reasonable interpretation of the general conditions of the site. Any changes in ground conditions discovered as a result of ground works should be referred back to us for additional comment and design review as required.

This report has been prepared for the benefit of BERL Consultants in relation to the specific proposals outlined herein 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 written agreement.

We trust that this satisfactorily confirms our understanding of your requirements. We will be pleased to discuss any aspect or to supply additional information if this is required. Please refer any further enquiries or correspondence to Maurice Fraser - mobile 021 378 399 or email frasergeologics@xtra.co.nz.

Yours sincerely

FRASER GEOLOGICS LIMITED

Maurice Fraser
MASCE, CMEngNZ, IntPE (NZ)
Managing Director

Z:\Fraser_Geologics\Fraser Geologics\1331PNGLFixup\2019Costings\DesignReports\10Oct19 PNGL RepairWorksDraftWordsPart1.docx
16 October 2019

9 REFERENCES

MSEW Design

- AASHTO, 2010, LRFD Bridge Design Manual 5th Edition.
- AS 4678 (2002) Earth Retaining Structures. Standards Australia.
- Federal Highway Works Administration, 1998, *Geotechnical Earthquake Engineering Reference Manual*, NHI Training Course in Geotechnical and Foundation Engineering, NHI Course 13239- Module 9, Publication FHWA HI-99-012.
- NZTA Bridge Manual 3rd Edition (SP/M/022) (2016)
- Transportation Research Board 2010: Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankment, NCHRP Report 611, Washington D.C.

Seismic Design

- NZS 1170:2016 – Structural Design Actions (Part 5)
- NZGS (2016) Geotechnical Earthquake Engineering Practice - module 1, identification assessment and mitigation of liquefaction hazards.
- TRB 611- Transportation Research Board 2010: Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankment, NCHRP Report 611, Washington D.C.

Site Specific Reporting

- Historical Stereopair Aerial Photographs: www.retrolens.nz
- Historical Photographs - National Library of NZ: https://natlib.govt.nz/ , https://paperspast.natlib.govt.nz/parliamentary (NZR reports 1937 – 1941), www.mtghawkesbay.com (various historical construction images) www.photonews.org/Gisborne (various historical photos & photo essays on the East Coast Railway)

Geology

- Mazengarb, C; Speden I.G. (compliers) 2000: Geology of the Raukumara area. Institute of Geological and Nuclear Sciences 1:250,000 geological map 6.