28 March 2013

Redland City Council
Cnr Bloomfield & Middle Streets
Cleveland
Qld 4163

Our ref: 41/25756
445500
Your ref: T-1502-11/12-RCC

Attn. Mr. Rodney Powell
By email (Rodney.Powell@redland.qld.gov.au) and post

Dear Sir

Peer Review of Raby Bay Geotechnical Study

1 Introduction

We refer to our proposal to Redland City Council (RCC) dated 29 October 2012 (Ref: 41/09157/60) in relation to an Independent Review of Proposed Works (by consultants KBR) for stabilisation of canal slopes at Raby Bay, Cleveland. This letter report presents the findings of GHD's review of the provided information and confirms the advices provided to you in telecons of 11 and 17 February 2013.

It should be noted that GHD has not been referenced in any of the documents reviewed, has no corporate record of involvement in the Raby Bay development (before or after) relating to canal bank stability, and the people participating in this review have also not been involved.

GHD therefore fulfils RCC's requirement for independence.

2 Material reviewed

The reference list attached presents a bibliography of the documents provided for review. It should be noted that as the development initiated in the early 1980s and documentation has spanned some 30 years it is inevitable that further information exists which has not formed part of this review and, if it did still exist, may add greater clarity to some of the issues and uncertainties identified. GHD has therefore had to rely on the documents provided, and where these refer to other documents, on the reasonableness of the interpretations and comments therein. Further, GHD's exposure to site and ground conditions is a limited one-off site visit in January 2013. Whilst the above are limitations to this review, GHD considers that the issues identified are relevant for the purposes here. Were new information specifically targeted to the issues raised to become available, this could affect GHD's findings. Please note GHD's scope and limitations in the relevant Appendix to this report.

3 Brief history

This canal development commenced some 30 years with Stage 1 being developed in the early 1980s and Stage 15 completed in 1995/96. Sherwood Geotechnical and Research Services (SGRS) in their 1995 Report (number 95006-1) present various information on the nature and staging/timing of the development, the various design consultants involved to that time, development of canal bank failures, and specialist advice up to 1995 and this is not repeated here. From our review the following succinct summary paints a general picture of the last 30 years:

- Development of Stage 1 commenced in the early 1980s and progressed to completion of Stage 15 in late 1995/early 1996 – the residential development is largely located in a tidal foreshore.
- The canals were formed using conventional mass earthworks techniques (cut and fill) with the sea held back with bunding (understood that earthworks were executed inside the tidal zone within a bunded area) and rainfall and seepage presumably were managed by drainage to and pumping from sumps.
- Materials won from the canal excavations were used as allotment filling for creation of house lots.
- Whilst canals were designed for a typical bed level of RL-3.5 (presumably to satisfy navigation requirements) the option to deepen to RL-7.5 was also evident in the designs.
- A bathymetric survey undertaken in mid-1995 indicated that most canals have been deepened to around RL-6 to RL-7 (even reaching RL-8 in parts) – these depths have also been confirmed for later stages when individual slips in various stages were investigated.
- At some locations, localised deepening for additional borrow has been reported, sometimes backfilled with waste fill.
- Failures of canal batters occurred at the outset of the development i.e. from Stage 1 onwards and have continued through most if not all stages.
- The 1995 SGRS report categorised the various failures known at that time and reported on a predictive exercise in order for council to reach agreement with the Developer on hand-over of responsibility for the canals.
- The early failures were considered to be primarily caused by the presence of insitu clays with low strength defects (termed for consistency with prior reports as “fissured” clays) in the cut profile – the 1995 SGRS report concluded that large-scale failure caused by sheared (i.e. fissured) clay occurred during or shortly following construction and should not occur following canal filling, particularly as appropriate remedial measures were implemented during and after Stage 1 once the problem was identified. SGRS also raised and discussed the issue of fill quality leading to a variety of failure types and were of the view that research into this aspect was required.
- SGRS also advised that of the failure causes they categorised by frequency at that time, the “U”ncertain category frequencies generally outweighed by a considerable margin the frequencies of “L”ot fill and “F”issured clay occurrences.
- It is understood that on the basis of the 1995 SGRS advices RCC reached agreement with the Developer on a fixed commercial arrangement to address predicted ongoing failures.
- It should be noted that the as-constructed (as-con) drawings sighted indicate that from Stage 6 onwards an earthworks preparation detail for removal of the insitu “marine layer” was employed extending to the canal batter, and from Stage 8 onwards (Stage 7 information not sighted) a detail for dealing with “fissured clay” was depicted – the as-con drawings sighted for all stages did not show the location, details and extent of where these treatments were deployed.

- It is understood that further failures occurred from 1995 to 1998, repairs for which are understood to have reached or exceeded the available funds set aside for ongoing repairs.
- In 1998/99 SGRS were engaged for several tasks and advised that fissured clay had only been a cause of short term failure during construction, areas where there may still have been some remnant insitu fissured clay were not considered to be at risk of future instability, all fissured clay and immediate wetting-related lot fill failures had occurred and what remained was the time uncertainty of time-dependent fill failure in the lot fill ; SGRS presented sketches of types of localised failure involving the fill in the immediate vicinity of the localised splash zone rock armour (see Figure 2 attached).
- In 1999 SGRS reported their research and development activities on the fill in relation to bank stability and concluded that, beyond reasonable doubt that the weakening and failure of canal banks was directly linked to interactions of water with clay fill that was not sufficiently compacted.
- Since 1995 it is evident that failures have been ongoing in various locations and have manifested in a variety of distress ranging from differential settlement/movement induced cracking of the concrete revetment wall, failure of rip-rap, ranging through to larger scale slope failure such as at Lots 841-844 and apparently elsewhere – these have been investigated by a variety of consultants.
- From 1995 to 2012 various investigations involving drilling have been undertaken, including the installation of inclinometers and extensometers at Lot 209 in 1999, 17 inclinometers (by Soil Surveys) in Stages 4-9 and some in what appears to be Lot 809/810(?) in 2012 – extensive DCP testing and some CPT, Dilatometer and Vane Shear testing has been carried out in the materials beneath the rock armour - significant laboratory testing has also been undertaken.
- In January 2009 KBR undertook a Desktop Review of previous documents and concluded that all canal bank failures appear to be slip circle failures with failures either in fill and confined to the upper part of the batter, or through the underlying fissured clays, with stability analysis covering these postulated failure mechanisms.
- In 2012 KBR, based on the Soil Surveys investigation in Stages 4-9 together with their interpretation of inclinometer monitoring, concluded that the information suggested that the soil slip is occurring in a shallow zone (of fill) underneath the rock protection and concrete wall – they support this with revised stability analyses where the presence of insitu fissured clays has been omitted and which inevitably show that theoretically, deeper failures aren't relevant and shallow failures in fill beneath the rock armour are the issue, coupled with a suggested technique for associated slope stabilisation.
- As a result of ongoing failures and the various advices RCC has received over the years, RCC in late 2012 engaged GHD to undertake an independent review of provided information and KBR's suggested slope stabilisation works.

4 Comments on site and marine earthworks

Original design documentation (plans and specifications) were not available for review – available as-con plans appear to be design drawings with minor or no alterations, signed-off as-constructed.

In the early Stages 1-5, the canal bank was typically shown as predominantly in cut although fill was required to reach finished allotment level of around RL+3.5. The change in designer from Cardno & Davies to Sinclair Knight in Stage 6 brought greater detail around surface preparation prior to placement of canal bank filling over natural ground and from the Stage 8 drawings (Stage 7 not sighted) included specific treatment for fissured clays in the foundation. With an indicated design requirement for removal of near surface marine sediments to a maximum of 1.5 m depth from Stage 8 onwards, and an unknown amount from Stage 1-7, coupled with a natural surface typically in the range of RL+1.5 to RL-1.0, it is unlikely that the prepared surface for fill placement would have been deeper than RL-2.5 and typically expected to be higher. Coupled with a canal floor of around RL-6 to -7 or so at many locations, it can be reasonably expected that there is 3.5+ m of exposed insitu cut face at many locations, excepting when remove and replace repairs were executed during construction, wherever these might have occurred.

Whilst in 1995 SGRS present their Figure 17 showing mass replacement of cut material where ordered, and also qualify this as being generic and varying widely at specific sites, it is unclear what the typical geometry of excavation and replacement was actually used prior to Stage 8. For example, Coffey in 1984 when dealing with a deeper failure at the easternmost finger in Stage 1, required backfilling of the canal from RL-6 to RL-3.5 and excavation of the head of the slip to near the bottom of the stone pitching i.e. about RL-1.4 and replacement with compacted fill. On the face of it around 2 m of slipped material remained untouched, whereas repair of Lots 81/82 indicated significant rockfill replacement to RL-4.50.

Based on GHD's experience with geotechnics and earthworks in both terrestrial and marine environments, the following comments are relevant:

- The insitu stiff and residual clays at this site are likely over-consolidated with significant locked-in horizontal stresses which are relieved on excavation.
- It is common practice to ignore for engineering purposes the near surface 0.5-1 m of soil like materials permanently submerged due to unrestrained swelling and softening.
- To ensure compaction to the full outside sloping edge of placed fill would require placement over-width and cutting back the lesser compacted edge material where plant won't fully traverse for safety.
- In order to achieve full width fill density where fill overlies a cut batter and the fill material is won from forming the cuts (where borrow is short), a well-planned and coordinated earthworks operation is required.
- It is conceivable, for reasons of cost and profitability pressures associated with development activities, that identification, excavation and replacement of intact fissured materials, repairs of failures, and exacting control of earthworks operations could have been managed such that only what was needed at the immediate time (or thought to be needed) was implemented and no more. Noting that control of the earthworks operation was in the hands of the contractor and likely that the designer's input and specialist consultants were only required to assist when called for.

There is good evidence presented that the fills near surface within the canal banks are low strength. This is to be expected to a limited depth for submerged and unprotected earthen materials, and/or to a greater extent if there wasn't careful attention paid to construction sequencing to ensure proper full width compaction at the outset.

It is also clear that, aside from repaired areas or areas where fissured clays were positively identified during construction of Stages 8-15 and treated as per the design, there is a substantial exposed cut batter forming the canal banks and supporting the placed fill at many locations. Even for the shallow canals in Stages 8, 12 and 13 (bed RL~-3.7 to -4), some exposure of insitu material in cut batters can be expected. The likelihood of exposed cut batters is further supported by the Earthtech 1997 investigation of the Tasman and Magellan Canal slips where some of the developed cross sections indicated bare cut slopes and where fill was thought to be present, a thin veneer paralleling the slopes is inferred.

The as-con drawings however lack the necessary detail (which would have been reasonably expected to have been included) to identify where fissured clays were identified and treated in the canal floor and cut batters, which is a significant shortcoming in managing the asset now.

Indeed, more recent failures have identified fissured clays which clearly were not identified and treated during the original construction which indicates that the method of identification was not as comprehensive as necessary and contrasting with SGRS's 1995 conclusion that appropriate remedial measures had been introduced (perhaps in design from Stage 8 onwards but not necessarily executed in construction) and that the risk of future sheared clay failure was not material.

5 Nature of the reported failures

In the information reviewed there does not appear to be one location where the actual failure surface has been investigated and positively identified. Typically, failure surfaces have been postulated based on (often) circular failure surfaces generated from stability analyses. Whist SGRS in 1995 categorised the failures as caused by "L"ot fill, "F"issured clays or "U"ncertain, the information on actual proven failure surfaces was not available in the information reviewed by GHD. Therefore, the robustness of the information leading to the SGRS categories could not be determined.

Further, the contribution of fissured clays to the more recent failures is virtually impossible to determine from the information reviewed. Due to their nature and occurrence, unless an investigation was specifically targeted at identifying fissures, it is the author's experience that they can be easily missed. Limiting geotechnical investigation budgets, constraints due the investigation technique (e.g. excavation and mapping not feasible under water and therefore rely on small scale drilling), sparseness of sampling and inexperienced practitioners can all conspire against finding fissures and hence leading to the conclusion that they are not present. Experience informs that fissures can be difficult to find with small size boreholes, even more so with infrequent sampling spacing. With sampling spacing often up to 1.5 m a zone of fissures could easily be missed. Another way of looking at this issue is, if fissures are positively found recognising the constraints above, then it is likely that they are prevalent notwithstanding material variability. Table 1 below summarises some of the investigation locations post 1995 where fissures have been found in the zone relevant and/or considered by the consultants to be significant with respect to bank stability.

Table 1 Investigation locations since 1995 where fissured clays identified

Date	Stage	Lot #/Canal	BH #	Source
September 1997	3-7	Tasman and Magellan	BHs 105, 109, 111, 112	Earthtech
September 1997	3/4?	226	BH 1	Douglas
November 1998	15	837 to 839	TP 101	Earthtech
October 2000	6	362 to 364	Dwg #3	Earthtech
September 2006	15	841 to 844	BH 787	Golder
June 2007	8	340 and 447	BH 1	Morrison

Figure 1 attached presents a simple and practical depiction of the broad subsurface profile at the development and is a useful contextual reference when considering further comments relating to ground conditions. Of course, variability (both natural and man-made) cannot be depicted in such a simplified model, and it has been noted in the literature reviewed that variability can be extreme and over very short distances.

In April 2012 Soil Surveys report on the installation of 17 inclinometers spread across Stages 4 to 9, with a view to measuring movements of the bank. Monitoring results to end of August 2012 were also reported by Soil Surveys with various trends identified. GHD has reviewed the monitoring data and is unable to identify any reliable trends since:

- Most movements are small and near the accuracy of reading for a high quality installation.
- What appear to be outward bank movement trends are often reversed.
- Movements along the alignment of the banks is often the same order of magnitude or larger than bank movements towards the canal.
- Most movements are in the upper metre or so which raises the question of the security of the inclinometer casing installation through the armour rock.
- Some movements are into the bank.

Given some 30 years of investigations into the slips at Raby Bay it is surprising that none of the failure investigations available for this review definitively identified the actual insitu failure surface. Many of the stability assessments have however thought it appropriate to look at deeper seated failures passing through natural materials likely weakened by fissures. The presence of near surface fill material in a low strength state would also be a contributor as would softening of overconsolidated insitu clayey materials either through unrestrained swelling and/or shear strain localisation on stress relief and pore pressure equilibration with time. Most recent failures appear reasonably large scale implying deeper failures into natural material.

In reference to the scale of slips, GHD's use of the terms "large", "deep" and their derivatives refer to failures that of their observable scale and/or from the factual and/or anecdotal evidence provided either in reports or advised by RCC, indicate that failure is not specifically constrained to a small zone of near surface fill and extends to the contact with or more likely into natural (potentially if not likely fissured/sheared/weakened) clays. That is, the natural clays play a role. GHD is aware of anecdotal evidence that KBR have, in their limited exposure to the site (understood to be 2008 to present), considered failures exclusively in fill to be the cause. As subsequently discussed, factual information was not presented in sufficient detail for GHD to meaningfully independently review.

Where the scale of "failure" is lesser e.g. unacceptable differential or total settlements that have not progressed to collapse, there are a number of possible scenarios including creep of fill, creep of softened and /or fissured clays, etc. as it is to be expected that the canal banks have a low Factor of Safety (FoS) discussed as follows:

- Unloading (by excavation) of overconsolidated fissured clays results in depressed pore pressures in saturated materials, time-dependent recovery of these followed by strain localisation and strain softening to at or near the fully softened condition (or known as critical state in contemporary soil mechanics) – for the high plasticity clays here this would be a long term effective frictional strength component phi' of low 20°'s and negligible cohesive intercept. Poorly compacted fills prone to collapse on inundation and possible swelling could also be expected to have a frictional component in the low 20°'s.
- With a slope of circa 18° (1V on 3H) an approximate long term FoS in the range of 1.1 to 1.4 could be expected for translational slips in material with phi' of 20°-25°.
- It is well understood that at these FoS values there will be portions of a "failure" surface at limiting conditions (i.e. FoS ~1) and therefore ongoing creep and strain softening can be expected where conditions prevail.
- For collapse to occur (with hydrostatic pore pressures) a frictional strength component of less than 18° is required indicating that in some part of the failure surface, residual or near residual conditions must have been reached, either movement induced and/or the presence and interaction of low strength fissures.

Such a model provides one plausible explanation for the deep seated failure at Lot #843 (verified by hydrographical survey), which occurred more than 10 years after completion. It also provides one plausible explanation where RCC have seen failures/movement continuing below structural repairs effected near the revetment wall (at Lot # 812) where RCC advise that stabilisation in 2010 of the fill above the natural materials did not fully arrest lower bank movements pointing to deeper issues likely in the natural materials.

This is also consistent with RCC's comments that often, once a failure initiates, it progresses along the bank affecting other properties i.e. reflecting a low FoS situation where, once lateral restraint is reduced by a failure, adjacent areas are triggered.

Whilst the above points to the insitu fissured clays being a key contributor to distress, this doesn't remove the issue of poorly compacted reactive fill playing a role, nor the absence of a sacrificial surficial zone of submerged material that would normally be allowed for submerged and unprotected soil-like materials. The ultimate difficulty for this review was the absence of definitive and objective information on actual failure surfaces and mechanisms. Rather, most of the assessments undertaken post SGRS were influenced in their assessments and choice of parameters for analyses by the earlier reports and seemed to rely on their experience and theoretical analyses to justify the failures observed and remediation design.

6 KBR assessment

In January 2009 KBR reported on their Desktop Review (of prior information) and Pre-feasibility Study and concluded that all canal bank failures appear to be slip circle failures. As GHD could not find one example of where the actual failure surface was physically identified and shape defined, this appears to be a speculative conclusion, even though it may reflect almost all of the historical theoretical analyses.

The parameters adopted for stability analyses are largely derived from the previous work reviewed and, whilst further investigation and testing work was recommended, the interpretations of others have largely been relied upon. There are a number of issues with the KBR stability analysis as follows:

- It is not clear where the ground model is derived from and how it reflects the changing approach to dealing with fissured clays as the development progressed.
- The analytical models do not name or show the properties of each of the typically 6 ground profile layers making it difficult to know exactly what has been analysed.
- The analyses seem to use residual strengths for the natural and fill clay layers above the canal floor – residual properties apply post failure, not prior, and it is inconceivable that residual properties apply to all parts of the failure surface in these layers given the structural orientation and surficial properties of fissures.
- The analyses adopt the residual (post failure) strength for fissured clays, but applies it to fills – this is considered excessively conservative for fill materials as they are unlikely to be extensively pre-sheared insitu to the extent of being anywhere near residual – further most of the fill material is a mixture with silts and sands – regardless of composition a critical state strength would be more appropriate.
- Whilst Section 3.1 of the KBR report identifies two forms of failure (traversing fissured clays and failures confined solely to placed fill) the summary concludes that most canal slope failures are limited to the engineered clay fill – there is no stated substantiation for such a definitive conclusion which is presumably influenced by the perceptions of others and/or KBR's stability analyses.
- Although KBR consider most canal slope failures are limited to the engineered clay fill, their Table 3.1, para 3.7 and Figure 2(a) (Appendix A) demonstrates low FoS<1 (0.82) for failure surfaces where the toe clearly traverses natural/fissured clays, demonstrating that deeper seated failures are equally if not more likely to occur if suitable conditions prevail, such as at Lots 809/810 and 843.
- The report concludes by theoretically assessing the potential effect of introducing various slope support measures at the pre-feasibility level of assessment.

In 2010 KBR prepared a Geotechnical Investigation Options report detailing a 60 borehole investigation from land and water covering all stages at an initial cost of $1.5-$1.6M (understood to be deferred due to excessive tendered sums at the time). The approach included a number of insitu tests together with recovering samples for a suite of laboratory testing. Much of the testing proposed had previously been undertaken at a variety of locations, but it was not clear how the investigation was tailored around and to complement the existing information. The purpose of the report primarily appears to be, as indicated in KBR's Summary, to present and justify the basis for the investigation proposed on economic and safety factors following the abandoned call for tenders in mid-2009.

Subsequently Soil Surveys were engaged in 2012 and undertook a broad investigation within Stages 4-9 including the installation of 21 inclinometers at 17 locations (Stages 4-9) and some inclinometers in a (then) developing failure at Piermont Place (Stage 15) as noted in KBR's 2012 report (thought by GHD to be at Lots 809/810). GHD's view of the inclinometer measurements from Stages 4-9 is presented in Section 5, suffice to say that no reliable trends were able to be conclusively identified.

In 2012 KBR presented their Geotech Analysis Report detailing their views on, and concept design addressing the canal stability issues concluding that:

- Inclinometers indicate failures are confined to the fill materials.
- Stabilisation of the fill supporting the rock armour could be achieved by grout injection into uncompacted fill.
- Optimisation of their concept design was required through field trials and numerical (finite element) modelling.

Whilst GHD are of the view that reliable trends were not evident from the inclinometers in Stages 4 to 9, KBR have interpreted these, and the additional inclinometers installed at Piermont Place at the time, as showing that slip is occurring in a shallow zone underneath the rock protection and concrete wall with most of the movement above the toe of the rock protection.

This led to KBR undertaking stability analyses with revised strength parameters, most notably changing the strength of the natural stiff and very stiff clays (where fissured zones have generally been identified) by significantly increasing the strengths from residual used in their 2009 analyses to peak strengths (see KBR Table 3). This has the inevitable effect of forcing the critical failure surfaces to be localised in the foundation fill beneath the rock armour. The 2012 stability analyses therefore don't provide any other insight into the failures other than to mirror KBR's view that failures are localised to the fill and insitu fissured or softened natural clays are not relevant to either the stability of the existing slopes nor the design of stabilisation measures

In this context, it is pertinent to consider the inclinometer data and subsurface profile prepared (and relied on more broadly) by KBR for Piermont Place – Section 2 from their 2012 report. The section has been annotated by GHD and this version is presented in Figure 3 attached. GHD considers this the most useful and reliable inclinometer data reviewed as it was located in a known moving mass and there are consistent movement trends identified. This clearly indicates a translational slide developing with movements occurring at depth in natural stiff clayey materials (not only in fill as postulated by KBR) well downslope of the toe of the rock armour. This contrasts with KBR's view that all movements are largely in fill and exiting at/near the toe of the rock armour. The location of greatest movement at around RL-4.5 is consistent with the understood construction practice i.e. foundation preparation for placement of fill would not have extended to this depth according to the design and hence insitu material forms much of the bank as a cut face profile.

For the failure at Piermont Place to have occurred, the only reasonable conclusion from the material reviewed is that natural material with a low strength likely caused by fissuring and/or softening existed at some depth and combined with many other factors, was the main contributor to the collapse. Lot # 843 is across the canal from where GHD understand this failure occurred, and from the available information likely suffered a similar fate.

The 2012 KBR report does not address this mode of failure and neither do the stabilisation measures proposed. GHD considers this a notable omission.

GHD understands that there is anecdotal evidence underpinning KBR's belief that the failures are high level and localised to the fill beneath the wall and revetment. Unfortunately, in the context of past life of this estate, KBR has only recently been involved (understood to be since 2008) and therefore hasn't seen any of the prior failures dating back. Aspects of the failures KBR have observed are not presented in sufficient detail for GHD to meaningfully independently review. Further, it is unclear how KBR have rationalised the previous reports which present contrasting information indicating the occurrence of larger/deeper failures and fissured clays, particularly in respect of the works KBR propose to resolve the stability issues across the estate.

7 Response to the brief

GHD's brief is given in Section 2 of our proposal and involved review of KBR's findings including:

1. The data used to construct ground models.
2. Analysis methodology.
3. Remedial measures proposed and associated risks.
4. By necessity, review of the plethora of background documents spanning over some 30 years.
5. Reporting of the above including discussion on risks and opportunities associated with the work.

7.1 Data used to construct ground models

It is understood that KBR's literature review led to the typical ground model geometry adopted. Unfortunately the absence of detailed as-con drawings documenting exactly what was done and where do not exist and this is a fundamental drawback to any model proposed. As a simple example, the contact between fill and prepared natural surface (if prepared) is simply not known aside from interpreting point data from drill holes. Further, given the variability at this site one model does not suit all. Also, some of the details of the ground model used in the 2009 KBR analyses are unclear.

The most significant change in the model from the 2009 to the final 2012 analyses is the consideration of insitu clays in the zone of site wide identified fissuring. In 2009 KBR's assessment of residual strength was used, whilst in 2012 substantially higher peak strengths were adopted. No reason has been given for this, other than the changed focus to failures being totally contained in the fill supporting the rock armour. GHD considers that the model in this regard does not fully reflect the likely reality and attendant risks.

7.2 Analysis methodology

The analysis methodology used by KBR is limited to circular and non-circular failure surfaces assessed using simple limiting equilibrium analyses. This type of analysis does not take into account strain compatibility of elements (e.g. hard inclusions such as grout injected (under pressure) columns displaced into low strength materials), nor the effect of ground movements (e.g. bending of unreinforced columns with little moment capacity) nor the effect on overall stability of displacing ground at a likely already low FoS (e.g. disturbance to and/or potential for mobilisation of fissured/softened clay zones marginally stable). GHD considers the analyses suitable for concept assessment only and would urge caution in implementing the treatment without further consideration of the issues above (and any others which arise) and their impact on the viability of the technique proposed. KBR do recommend numerical modelling to refine the design but it is not clear if the above issues have been contemplated.

A particularly critical issue is that the final analyses are predicated on the failure mechanism being solely confined to the fill supporting the rock armour and not contributed to or caused by the natural insitu materials. There is sufficient evidence to suggest the failure surface at a number of the failures exists below the rock armour and the likelihood of fissured/softened insitu clays contributing is almost certain. In this case, the proposed stabilisation can be expected to be inadequate.

7.3 Proposed stabilisation measure

(600 mm diameter, 3m long grout injected piles at 1m spacing along shoreline)

The stabilisation measure suggested is grout injection into the “uncompacted” fill (i.e. fill supporting the rock armour) to stabilise this zone exclusively with soil cement piles (or pins). Whilst deeper seated failure surfaces have been analysed using simple limiting equilibrium techniques, it has been assumed that weakened zones of insitu material due to fissuring and/or strain softening are absent. From the material reviewed, GHD does not support this contention and sees this amongst the other issues raised in 7.2 as significant risks to the effectiveness of this treatment.

In this situation, where failure surfaces and mechanisms have not been well defined, it would be prudent to err on the side of caution, as many consultants have done in the past i.e. use of robust structural solutions making some allowance for the likelihood of failures being contributed to by lower strength insitu materials. From the material reviewed, GHD cannot find any adequately substantiated reason(s) to change from this approach at this time.

7.4 Document review

Within the budget and time constraints for this review, GHD has perused the documents provided but it was clearly impractical to delve deeply into all reports nor reprocess data or undertake numerical stability analyses. Further, there are other documents referred in the documents reviewed which were unavailable. It would also have been very desirable to actually view soil samples recovered from the site to better understand the nature of the materials, but this was outside of the scope and samples may not have been retained over the years regardless.

Notwithstanding these constraints, the following is evident:

- Ground conditions at Raby Bay are complex and variable.
- As-con records sighted provide little to no information on actual ground conditions encountered, repairs implemented and inherent site variability.
- There is no doubt that a significant cut face of natural material is exposed in the majority of the canals, especially where excavation for borrow often extended down to RL-6 to RL-7.
- There is little doubt that ground preparation, fill conditioning, placement and compaction practices appeared wanting, including the need to compact over-width and cut back to well compacted.
- The original design did not appear to recognise the typical outer-skin softening which occurs with submerged soil like materials, cut or fill.
- Failures have continued at a rate and magnitude not expected by RCC nor originally contemplated in terms of ongoing repair costs.
- Many consultants have been involved over the 30 year life acting for various parties and each bringing their own views to the problem.
- Many of the failure assessments and/or stabilisation designs have been influenced by the extensive work of SGRS and the soil properties they have considered continue to be used.
- Although previously considered to be unlikely post-inundation, there is sufficient evidence from failures since 1995 to point to fissured clays likely playing a role in the larger failures that have occurred.
- Inclinometers installed in 2012 in Stages 4 to 9 have been interpreted by Soil Surveys and KBR as providing some movement trends – GHD could not conclusively identify these trends and consider interpreted trends from the limited readings taken as unproven.
- KBR recognised the role of fissured clays in their 2009 review but took a different view in their 2012 work, where they considered slips to be confined to a wedge of uncompacted fill under the rock armour and developed a concept design to locally stabilise this fill solely– Figure 3 attached depicts inclinometers in a mobile area of Piermont Place which presents a different picture and confirms the likelihood of larger scale failures extending into the natural (likely fissured in places) clays.

7.5 Reporting – summary

It is understood that RCC's preference is to determine and implement a cost effective and practical solution globally across the estate that solves the stability issues.

On the basis of the information reviewed GHD is not able to recommend that the KBR solution (which is at concept level only) would solve the Raby Bay canal failure potential nor that it should be implemented as a broad coverage fit and forget solution. GHD has raised a number of reservations and the key issue is that even if implemented, instability on a larger scale involving insitu natural clays is a real and significant risk. The concept treatment may also not totally arrest creep movement of fills such that some degree of distress to the revetment wall and upper canal bank may still occur.

In terms of concepts for stabilising the banks, there are many factors (known and unknown) impacting on choice. It is understood that, to date, reasonably robust designs involving structural piled retainment have been deployed and that these have generally catered for (in practice if not design) the effects of lower strength insitu materials such as softened and/or fissured clays. It is understood that this has mostly been proven to be effective but costly and is executed on a case by case reactive basis usually requiring some time (1-2 years) to implement. It is understood such a treatment would be cost prohibitive if it were implemented pre-emptively on a widespread basis, and on at least one recent occasion advised by RCC has not totally arrested movements.

At a conceptual level it should be possible to install more rows of grout piles lower in the bank profile or at the toe to reinforce for both localised and larger scale failures extending into the insitu clays. This would require consideration of the various risks already raised before being further considered.

A combination treatment where toe loading the canal (sand/rock fill) could be considered for larger failures and grout piles for the localised fill issue. To be effective this would likely require a change to the useable waterway depth/profile which may not be acceptable. This would also prove costly as the majority of canals were deepened to RL-6 to RL-7, even up RL-8 apparently for borrow.

It is unfortunate that reliable and definitive information on failure surfaces and their location, implemented construction details, pore pressures within the bank etc. are not available, as these are key inputs into any selection and optimisation process for ground stabilisation.

We also note that KBR are of the view (section 4.2.3 of their 2012 report) that the apparent issue with the canal slopes has been identified, no further geotechnical investigations are required and simple survey and probing ahead of remedial work is all that is required for the way forward. From this review and the risks and uncertainties identified GHD cannot accord with this definitive view on such a complex issue.

Sincerely
GHD Pty Ltd

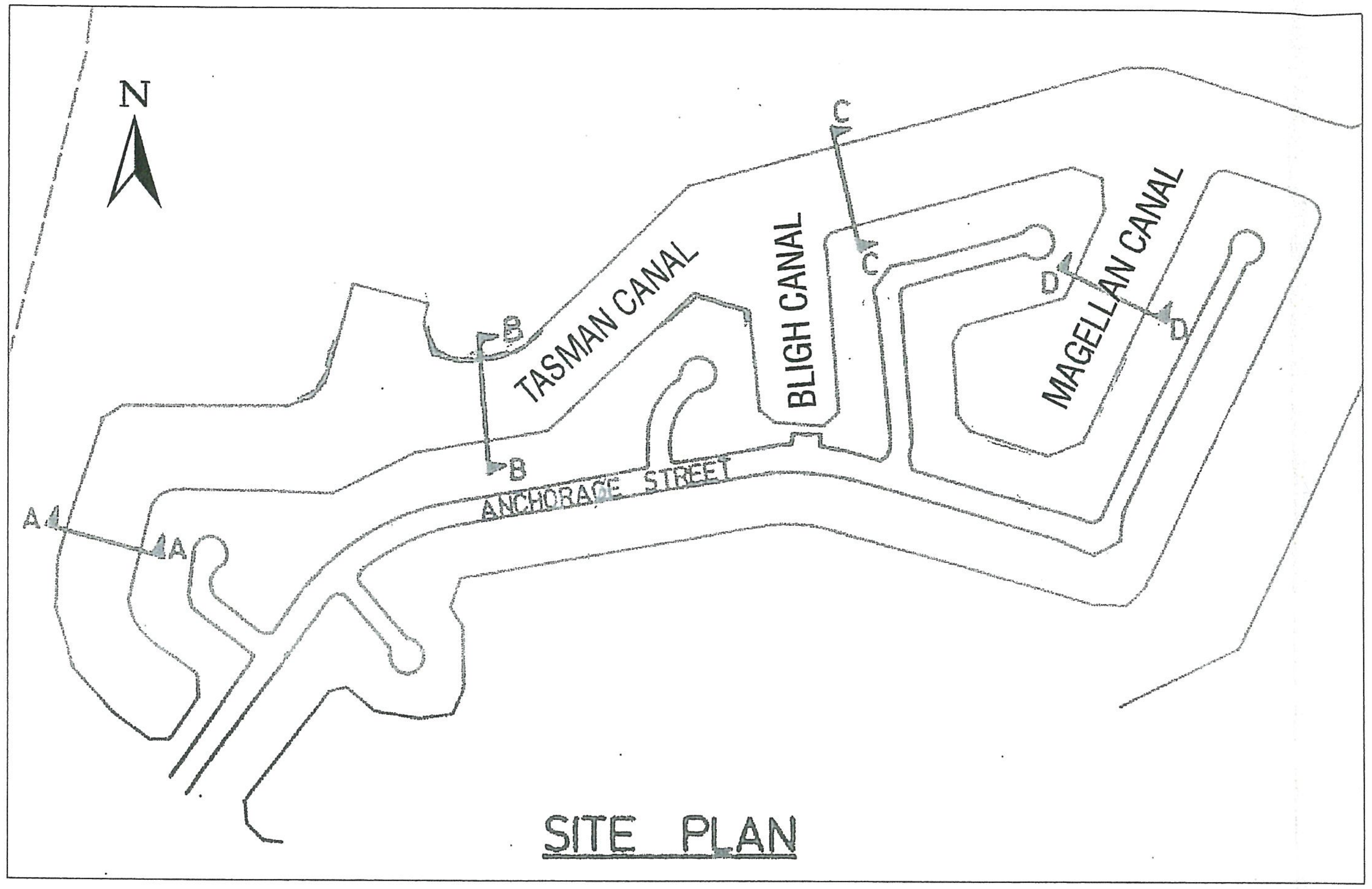
Alex Litwinowicz

07 3316 3582

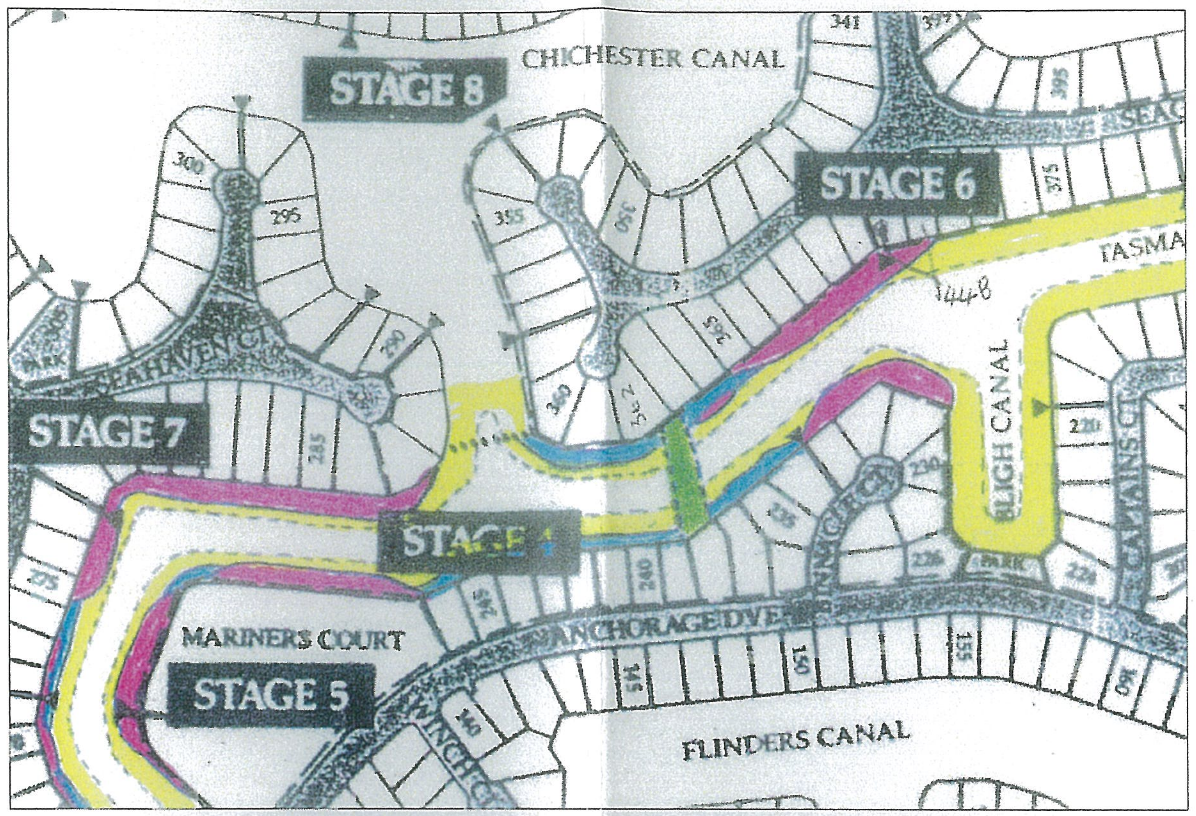
Attachments:

Figures
Bibliography
Scope and Limitations

Figures



SITE PLAN



LEGEND

RIP RAP

ZONE OF NATURAL MATERIAL REMOVED AND ENGINEERED FILL REPLACEMENT (NOTE : Area outlines based on Leighton survey data but modified in parts for consistency with site observations)

APPROXIMATE TO INFERRED GEOLOGICAL BOUNDARY

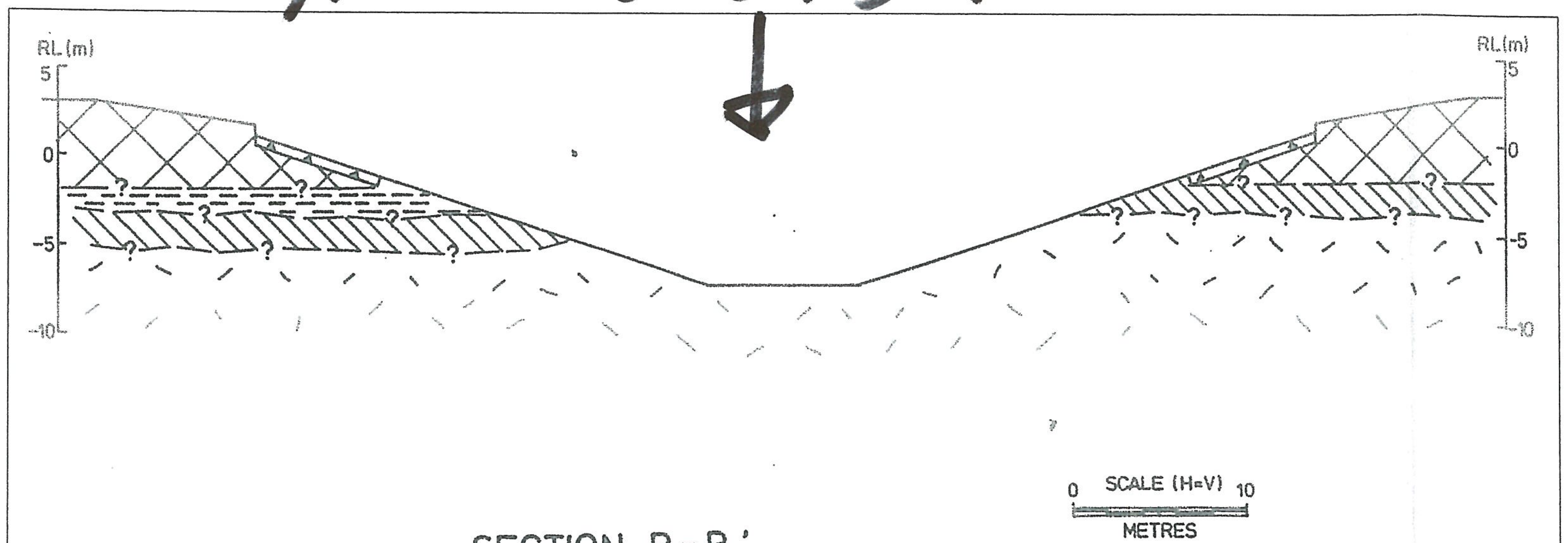
CLAY, high plasticity, red-brown.

CLAY, high plasticity, mottled green to grey and red-brown, occasionally fissured with some polished surfaces.

BASALT, extremely to highly weathered, mottled green / red-brown and orange-brown with localised ferruginous concentrations; very low to low rock substance strength.

BASALT, highly to moderately weathered, dark brown-grey medium to high rock substance strength.

TYPICAL GROUND MODEL



SECTION B-B'

FIGURE 1

EARTHTECH CONSULTANTS
Geotechnical & Environmental Engineering

DRAWN MRS	REDLAND SHIRE COUNCIL
CHECKED ALM	RABY BAY CANAL ESTATES LOTS 362 TO 364 - TASMAN CANAL
DATE OCTOBER 2000	SCHEMATIC CROSS-SECTION FROM COFFEY & PARTNERS REPORT, NOV '86
JOB NO MF1621	SCALE As Shown — Drawing No. 3

MF1621-003A

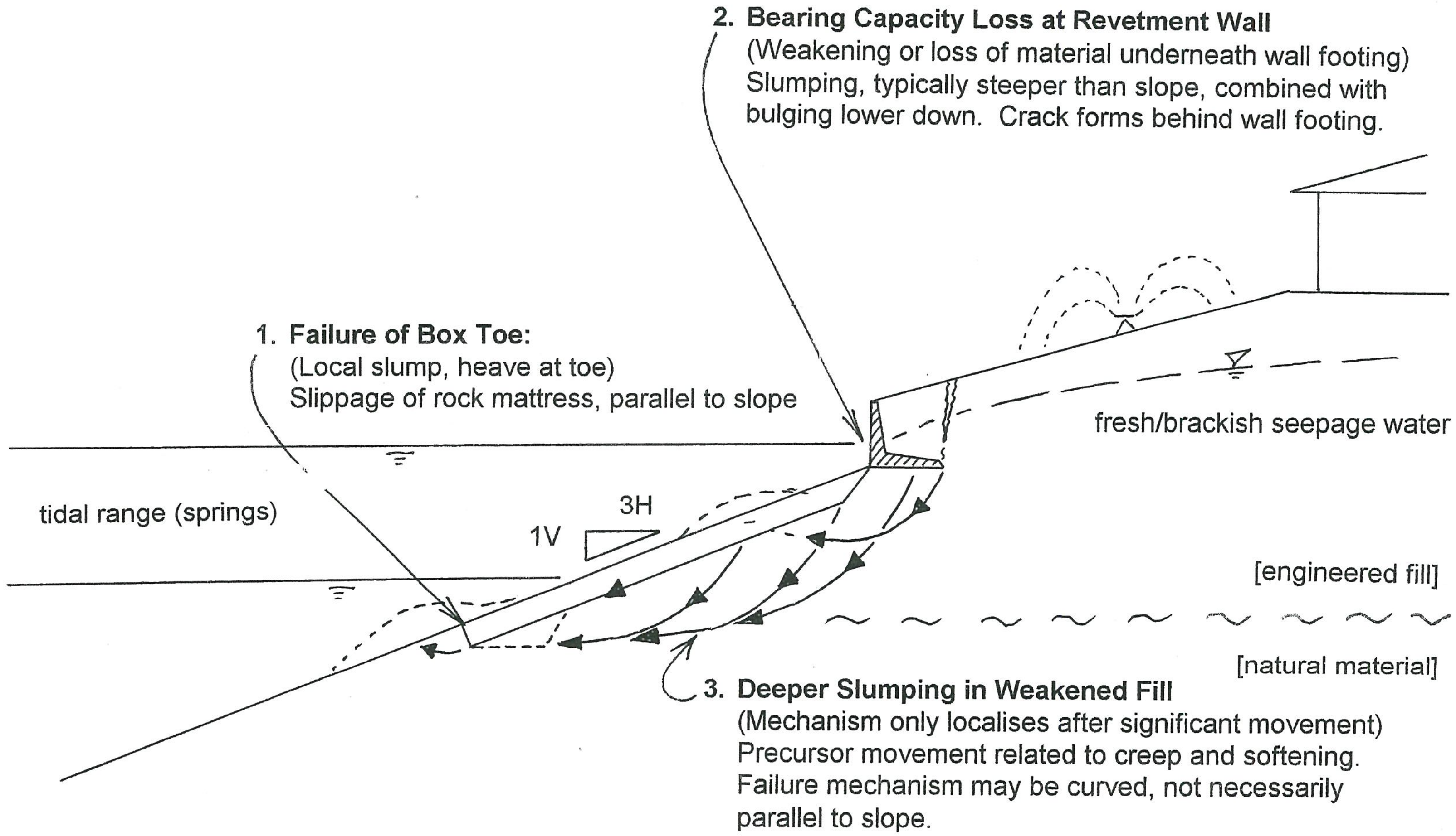
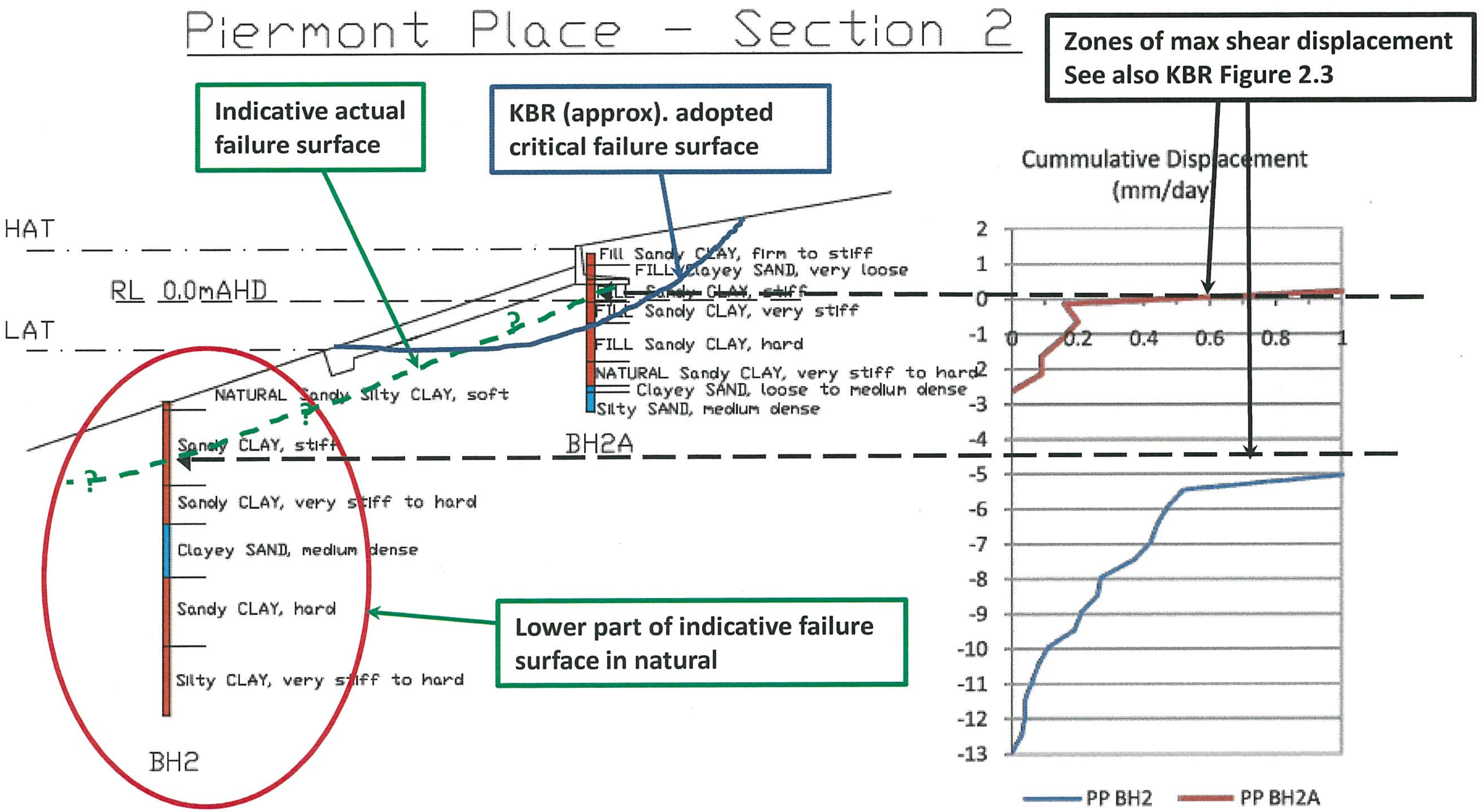


Figure 2 Explanation for Slip-Related Movement Directions

FIGURE 2

N.B. Annotations by GHD



Record Length: PP BH2 = 6 days

PP BH2A = 14 days

FIGURE 3

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