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NSW GOVERNMENT

Coffs Harbour Bypass

Environmental Impact Statement

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Groundwater, flooding and hydrology assessments

Appendix N – Groundwater assessment

Appendix O – Flooding and hydrology assessment

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Appendix N

Groundwater assessment

Roads and Maritime Services

Coffs Harbour Bypass

Groundwater Assessment

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1 Introduction

1.1 The proposed project

Roads and Maritime Services (Roads and Maritime) is seeking approval for the Coffs Harbour Bypass (the project). The approval is being sought under Division 5.2 of the NSW Environmental Planning and Assessment Act 1979 (EP&A Act) as Critical State Significant Infrastructure (CSSI).

The project includes a 12 km bypass of Coffs Harbour from south of Englands Road to Korora Hill in the north and a 2 km upgrade of the existing highway between Korora Hill and Sapphire. The project would provide a four-lane divided highway that bypasses Coffs Harbour, passing through the North Boambee Valley, Roberts Hill and then traversing the foothills of the Coffs Harbour basin to the west and north to Korora Hill.

The key features of the project include:

- Four-lane divided highway from south of Englands Road roundabout to the dual carriageway highway at Sapphire
- Bypass of the Coffs Harbour urban area from south of Englands Road intersection to Korora Hill
- Upgrade of the existing Pacific Highway between Korora Hill and the dual carriageway highway at Sapphire
- Grade-separated interchanges at Englands Road, Coramba Road and Korora Hill
- A one-way local access road along the western side of the project between the southern tie-in and Englands Road, connecting properties to the road network via Englands Road
- A new service road, located east of the project, connecting Solitary Islands Way with James Small Drive and the existing Pacific Highway near Bruxner Park Road
- Three tunnels through ridges at Roberts Hill (around 190 m long), Shephards Lane (around 360 m long), and Gatelys Road (around 450 m long)
- Structures to pass over local roads and creeks as well as a bridge over the North Coast Railway
- A series of cuttings and embankments along the alignment
- Tie-ins and modifications to the local road network to enable local road connections across and around the alignment
- Pedestrian and cycling facilities, including a shared path along the service road tying into the existing shared path on Solitary Islands Way, and a new pedestrian bridge to replace the existing Luke Bowen footbridge with the name being retained

- Relocation of the Kororo Public School bus interchange
- Noise attenuation, including low noise pavement, noise barriers and at-property treatments as required
- Fauna crossing structures including glider poles, underpasses and fencing
- Ancillary work to facilitate construction and operation of the project, including:
  - Adjustment, relocation and/or protection of utilities and services
  - New or adjusted property accesses as required
  - Operational water quality measures and retention basins
  - Temporary construction facilities and work including compound and stockpile sites, concrete/asphalt batching plant, sedimentation basins and access roads (if required).

1.2 Purpose of this report

The purpose of this groundwater assessment is to assess groundwater impacts from the project construction and operation, and when required, identify feasible and reasonable mitigation measures.

This groundwater assessment has been prepared to address the Secretary's Environmental Assessment Requirements (SEARs) for the project for the purpose of seeking project approval under Division 5.2 of the EP&A Act. **Table 1** outlines the requirements relevant to this assessment and where they are addressed in the report.

Table 1: SEARS relevant to groundwater

Secretary's requirement	Where addressed in EIS
9. Soils 3. The Proponent must assess the impacts of the project on soil salinity and how it may affect groundwater resources and hydrology.	Section 2.5, Section 4.3 for groundwater sections
11. Water – Hydrology 1. The Proponent must describe (and map) the existing hydrological regime for any surface and groundwater resource (including reliance by users and for ecological purposes) likely to be impacted by the project, including stream orders, as per the FBA	Section 2.4, Section 2.7 for groundwater sections
2. The Proponent must assess (and model if appropriate) the impact of the construction and operation of the project and any ancillary facilities (both built elements and discharges) on surface and groundwater hydrology in accordance with the current guidelines, including:	Section 4
(a) natural processes within rivers, wetlands, estuaries, marine waters and floodplains that affect the health of the fluvial, riparian, estuarine or marine system and landscape health (such as modified discharge volumes, durations and velocities), aquatic connectivity and access to habitat for spawning and refuge	Section 2.4, Section 2.6 for groundwater sections

Secretary's requirement	Where addressed in EIS
(b) impacts from any permanent and temporary interruption of groundwater flow, including the extent of drawdown, barriers to flows, implications for groundwater dependent surface flows, ecosystems and species, groundwater users and the potential for settlement	Section 4
(c) changes to environmental water availability and flows, both regulated/licensed and unregulated/rules-based sources	Section 4
(f) water take (direct or passive) from all surface and groundwater sources with estimates of annual volumes during construction and operation.	Section 4.5
3. The Proponent must identify any requirements for baseline monitoring of hydrological attributes	Section 2.6, Section 5.2
4. The assessment must include details of proposed surface and groundwater monitoring.	Section 2.6, Section 5.2

1.3 Study area

The study area for this groundwater assessment was a 1km area around the project, which extends from south of Englands Road to Sapphire in the north. The area includes the construction and operational footprints of the project and an allowance for areas which could be indirectly impacted because of changes to groundwater levels. A 1km search area was chosen as a conservative distance from the project to allow for these indirect impacts and confirmed by the results of the numerical groundwater modelling (**Appendix C1 and C2**). The project alignment is presented in **Figure 1**.

1.4 Methodology

Environmental impact statement guidelines provided by the NSW Government and the SEARS have been followed in preparing this groundwater assessment. The guidelines provide guidance on factors that should be considered in the assessment of environmental impacts to meet the SEARS.

The groundwater assessment was undertaken using a methodology which comprised of characterisation of the existing groundwater regime, identification of potential receptors, evaluation of the potential impacts caused by the construction and operation of the project and identification of identified of mitigation measures to minimise impacts. The assessment process included:

- Review of existing publicly available resources comprising literature relating to the study area such as geological and environmental maps, published journal articles, government reports and geodatabases, proclaimed areas, available groundwater level, quality and flow data (within a 1 km radius of the site) and relevant water sharing plans for the study area.
- Factual and interpretative geological and hydrogeological investigation reports prepared for the project,

- Interpretation of groundwater level monitoring provided for the period between July 2017 and February 2019 and laboratory testing data undertaken at 38 groundwater monitoring sites along the project in 2017 *(RCA Australia, 2017a)*
- Identification of locations of potential receptors including groundwater supply wells, agricultural dams, groundwater dependent ecosystems and sensitive receiving environments such as wetlands,
- Development of a regional groundwater conceptual model that considers how groundwater is recharged, flows and discharges, how it is used and how it may interact with the project,
- Review of concept design documentation to evaluate elements of the project which have the potential to affect the groundwater environment,
- Identification of potential impacts on the groundwater systems with specific focus on areas of cuts and drained tunnels which extend below groundwater level, where interaction with the groundwater system is expected to be the greatest,
- Development of local scale conceptual models to highlight potential impacts on the groundwater environment,
- Qualitative assessment of potential groundwater impacts for the project during construction and operational phases,
- Development of numerical hydrogeological models for cuts (**Appendix C1**, *RCA, 2019*) and drained tunnels (**Appendix C2**, *PSM, 2019*) and quantitative assessment to evaluate the scale of groundwater table lowering and potential groundwater take for the project,
- Assessment of potential impacts against criteria set out in the NSW Aquifer Interference Policy (*DPI, 2012*) and other relevant guidelines,
- Recommendation for additional investigation or monitoring that may be implemented to supplement the existing data (**Section 5.2**) and recommendation for/discussion of proposed mitigation measures included in the concept design which are incorporated into the design to alleviate potential impacts on the groundwater environment (**Section 5.3**).

1.5 Policy context and legislative framework

State policy and guidelines relevant to this groundwater assessment are provided below.

1.5.1 NSW Water Management Act 2000

The NSW Water Management Act 2000 (WM Act) is administered by the NSW Department of Primary Industries (DPI). The WM Act is intended to ensure the sustainable and integrated management of water resources so that they are conserved and properly managed for both present and future generations. The WM Act is intended as the primary means to protect and enhance environmental qualities of river and groundwater systems and associated wetlands, floodplains and estuaries as well as providing protection of catchment conditions. It provides formal protection and enhancement of the environmental quality of waterways and instream uses.

1.5.2 NSW Water Management (General) Regulation 2018

The NSW DPI Water Management (General) Regulation 2018 is the key regulation for the implementation of the NSW Water Management Act 2000. The regulation specifies important procedural and technical matters related to the administration of the Act and specifies exemptions from licence and approval requirements under the Act.

1.5.3 NSW Water Sharing Plans

Water sharing plans are the main tool in the Water Management Act 2000 to allocate and provide water for the environmental health of rivers and groundwater systems, while also providing licence holders access to water. Water sharing plans define the rules for how water is allocated and have been developed under the WM Act for all water sources in NSW. The aims of the water sharing plans are to:

- Clarify the rights of the environment, basic landholders, town water suppliers and other licenced users,
- Define the long-term average annual extraction limit (LTAAEL) for water sources,
- Set rules to manage the impacts of extractions; and
- Facilitate the trading of water between users

Groundwater sources in the study area are regulated under two water sharing plans. These are

- *The Water Sharing Plan for the Coffs Harbour Area Unregulated and Alluvial Water Sources, 2009* and
- *The Water Sharing Plan for the North Coast Fractured and Porous Rock Groundwater Sources, 2016*

Further information relating to the water sharing plans is made in **Section 2.6.8**.

1.5.4 NSW Aquifer Interference Policy 2012

The purpose of the Aquifer Interference Policy (*DPI, 2012*) is to clarify the role and requirements of the Minister in charge of administering the WM Act in the water licensing and assessment processes for aquifer interference activities in NSW. The policy aims to clarify the requirements for licensing for aquifer interference activities as well as establishing consideration and advice structures for the potential impacts of an aquifer interference activity.

The policy applies to all aquifer interference activity but has been developed to address a range of high risk activity such as large infrastructure developments that require dewatering or ongoing drainage into excavations. An aquifer interference approval is generally required for any works that involve:

- The penetration of an aquifer
- The interference with water in an aquifer
- The obstruction of flow of water in an aquifer
- The taking of water from an aquifer in the course of carrying out mining or any other activity prescribed by the regulations
- The disposal of water from an aquifer

Sufficient access licences must be held to account for all water taken from a groundwater source as a result of an aquifer interference activity, both for the life of the activity and after the activity has ceased, until the aquifer system reaches equilibrium. Section 5.23 of the EP&A Act 1979 provides exemption for SSI projects for the need of a water use approval under section 89, a water management work approval under section 90 or an activity approval (other than an aquifer interference approval) under section 91 of the Water Management Act 2000.

The NSW Aquifer Interference Policy requires that potential impacts on groundwater sources be assessed against minimal impact considerations outlined in the policy. Minimal impact considerations depend on the groundwater source, however for less productive and fractured bedrock aquifers in which the project is sited, these criteria are summarised in **Table 2. Section 4.4** presents the results of the groundwater impact assessment against the requirements of the NSW Aquifer Interference Policy.

Table 2: Minimal impact considerations for aquifer interference activity (less productive and fractured bedrock groundwater source)

Water table	Water Pressure	Water Quality
Level 1 Less than or equal to 10% cumulative variation in the water table, allowing for typical 'post water sharing plan' variations, 40m from any: a) High priority groundwater dependent ecosystem; or b) High priority culturally significant site listed in the schedule of the relevant water sharing plan, or A maximum of a 2m decline cumulatively at a water supply work Level 2 If more than 10% cumulative variation in the water table, allowing for typical climatic 'post-water sharing plan' variations, 40m from any a) High priority groundwater dependent ecosystem; or b) High priority culturally significant site listed in the schedule of the relevant water sharing plan, If appropriate studies demonstrate to the Minister's satisfaction that the variation will not prevent the long-term viability of the dependent ecosystem or significant site. If more than a 2m decline cumulatively at any water supply work then make good provisions should apply.	Level 1 A cumulative pressure head decline of not more than a two-metre decline, at any water supply work Level 2 If the predicted pressure head decline is greater than requirement 1 then appropriate studies are required to demonstrate to the Minister's satisfaction that the decline will not prevent the long-term viability of the affected water supply works unless make good provisions apply.	Level 1 Any change in the groundwater quality should not lower the beneficial use category of the groundwater source beyond 40m from the activity. Level 2 If condition 1 is not met then appropriate studies will need to demonstrate to the Minister's satisfaction that the change in groundwater quality will not prevent the long-term viability of the dependent ecosystem, significant site or affected water supply work.

1.5.5 NSW State Groundwater Policy Framework Document 1997

The groundwater policy framework document is used to provide ecologically sustainable management guidance about groundwater resources, so they can sustain environmental, social and economic uses for the people of NSW. The policy is divided into three components, as follows:

NSW Groundwater Quantity Management Policy

The principles of this policy include:

- To maintain use of groundwater within the sustainable yield of the aquifer from which it is withdrawn,
- To ensure groundwater extraction is managed to prevent unacceptable local impacts, and

- The licencing of all groundwater extraction, which may be allowed to be transferred depending on the physical constraints of the groundwater.

NSW Groundwater Quality Protection Policy

The objectives of this policy are the ecologically sustainable management of the states groundwater resources to:

- Slow, halt or reverse any degradation in groundwater resources,
- Direct potentially polluting activities to the most appropriate local geological setting to minimise risk to groundwater,
- Establish a methodology for reviewing new developments with respect to their potential impact on water resources that will provide protection to the resource commensurate with both the threat that the development poses and the value of the resource, and
- Establish triggers for the use of more advanced groundwater protection tools such as groundwater vulnerability maps or groundwater protection zones.

NSW Groundwater Dependent Ecosystems Policy

This policy is designed to protect valuable ecosystems that rely on groundwater to survive, maintain the biophysical functions and preserve these ecosystems for the resources of future generations. Furthermore, the policy provides practical guidelines that can be used as tools to suit a specific need based on a given groundwater dependant ecosystem or environment.

1.5.6 NSW Water Extraction Monitoring Policy 2007

The objective of the Water Extraction Monitoring Policy is to increase the extent of active monitoring of water extraction with a future aim of having 90 per cent of the total volume of water in each water sharing plan being subject to active monitoring. This policy sets out the rules and guidelines for holders of groundwater extraction licenses.

1.5.7 NSW State Environmental Planning Policy 2018 – Coastal Management

The NSW State Environmental Planning Policy (Coastal Management) 2018 (Coastal Management SEPP) is focused on ecologically sustainable development that protects environmental assets of the coast, establishing a framework for land use planning to guide decision making in the coastal zone and defines coastal management areas including Coastal Wetlands as defined by the Coastal Management SEPP.

1.5.8 Risk Assessment Guidelines for Groundwater Dependent Ecosystems 2012

The risk assessment guidelines are used to manage land and water use activities that pose a potential threat to groundwater dependant ecosystems. The guidelines consist of four volumes that include the conceptual framework, worked examples, identification of high potential groundwater dependant ecosystems and their ecological value for coastal aquifers, and the risk of groundwater extraction on the coastal plains of NSW.

1.5.9 NSW Water Quality and River Flow Objectives

The NSW Water Quality and River Flow Objectives have been set-out for fresh and estuarine surface waters to identify:

- The community's values and uses of these surface waters
- Water quality indicators to assess the current condition of the waterways.

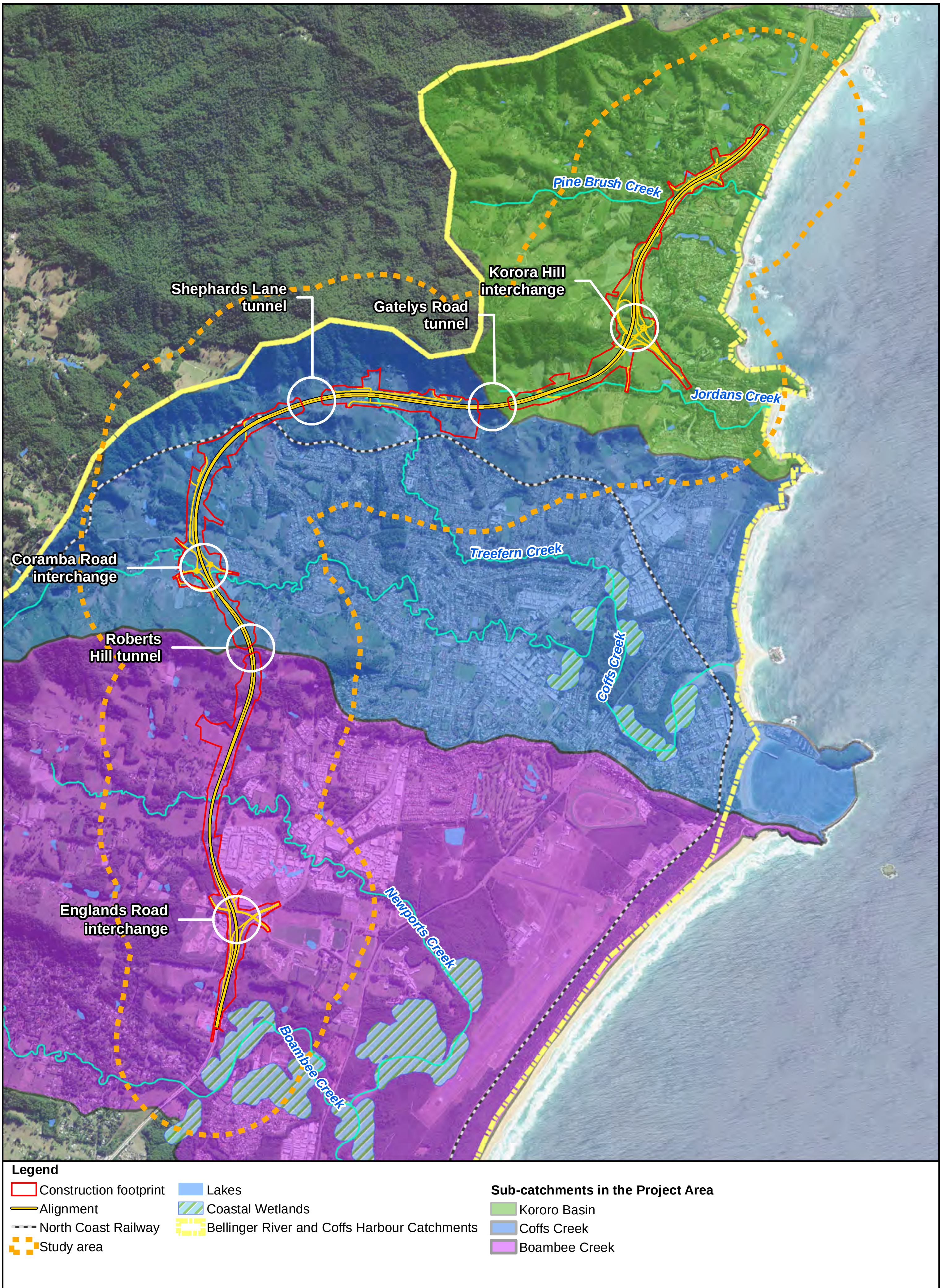
These water quality and flow objectives are consistent with the Australian and New Zealand Guidelines for Fresh and Marine Water Quality, 2000.

2 Existing environment

2.1 Topography

The project is located within six kilometres of the Coffs Harbour coastline. The Coffs Harbour region is characterised by relatively small coastal catchments located within the Bellinger River catchment, with narrow floodplains along the coastline and steep hills in the catchment headwater areas. The project extends from floodplains of the North Boambee Creek, south of Coffs Harbour, through three ridgelines (Roberts Hill, Shephards Lane and Gatelys Road) west of Coffs Harbour to an area characterised by rolling hills at Korora, to coastal plains at Sapphire, north of Coffs Harbour (**Figure 1**).

The bypass extends from relatively flat, alluvial areas in the south between about 5 – 10 m Australian height datum (AHD) to the Roberts Hill ridgeline which rises to about 85 m AHD. North of the Roberts Hill ridgeline, the project traverses through foothill areas, generally parallel to the catchment ridgelines which undulate between about 25 m AHD and 70 m AHD. The Shephards Lane and Gatelys Road ridgelines rise to 165 m AHD and 145 m AHD respectively. North of the Gatelys Road ridgeline the project topography decreases to around 40 m AHD at the current Pacific Highway alignment, past Pine Brush Creek to an elevation of about 25 m AHD.



Coffs Harbour Bypass
Site location plan and surface water features
Figure 1

2.2 Climate

Coffs Harbour is located in a warm, sub-tropical area with seasonal rainfall. Most of the region's rainfall is received during summer and autumn months and relatively drier conditions extend through winter and spring.

A review of the Bureau of Meteorology (BoM) indicates there is climatic data available for Coffs Harbour from the following weather stations:

- Coffs Harbour Airport (Station No. 059151), from August 2013 to present, 3.6 km east of the project
- Coffs Harbour MO (Station No. 059040), from February 1943 to August 2015, 3.7 km east of the project

The rainfall data for the Coffs Harbour MO and Coffs Harbour weather stations is summarised in **Table 3** and the climatic data is presented in **Figure 2**. The Coffs Harbour Airport has been excluded due to the limited amount of data available.

Table 3: Summary of rainfall statistics

Weather station	Mean annual rainfall	Highest monthly average	Lowest monthly average
Coffs Harbour MO	1699 mm	March, 235 mm	September, 60 mm
Coffs Harbour	1651 mm	March, 232 mm	September, 68 mm

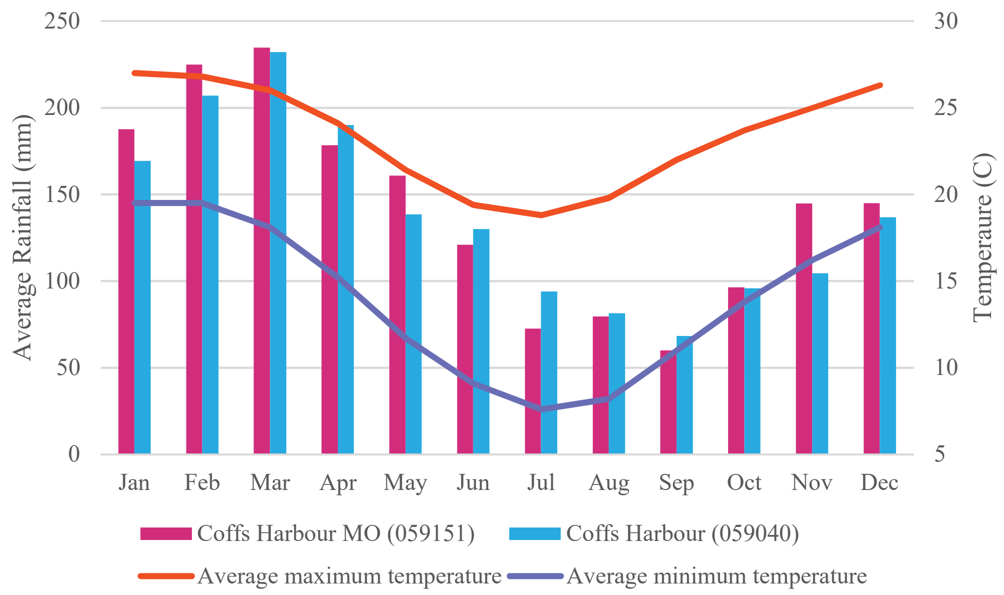


Figure 2: Average monthly rainfall and temperature at Coffs Harbour MO and Coffs Harbour weather stations

2.3 Surrounding land use and vegetation

Coffs Harbour is a regional city on the NSW mid-north coast which has an important recreation and tourism industry but regionally also supports highly productive agricultural industries (*DPE, 2017*). Much of the flat coastal plain has been cleared for urban development, where the southern and northern ends of the project tie-in to the existing Pacific Highway.

The project is located in foothills and slopes of the Great Dividing Range which have been cleared for primary industry and rural development. Primary industry activities in the region include forestry and timber milling, beef cattle production, fishing, dairying, horticulture and banana and blueberry plantations *(DWE, 2009).* These primary industries abstract groundwater for commercial purposes including irrigation and stock watering. Groundwater is also used for household supply, industrial uses and other purposes. Groundwater yields and abstraction limits are controlled by the aquifer the bore is abstracting from, which in the study area is principally fractured bedrock. **Section 2.6.9** provides additional information on groundwater use in the study area.

Vegetation is unevenly distributed along the project, which intersects areas of coastal, riparian and remnant vegetation. Some vegetation communities rely on the presence of groundwater to sustain their populations. One groundwater dependent Endangered Ecological Communities (EEC) listed in the *Biodiversity Conservation Act, 2016* (BC Act) has been identified on the floodplain surrounding the Boambee Creek and Newports Creek confluence, which supports a Freshwater Wetland ecosystem *(BoM, 2018).* Native vegetation and landscape values within the study area are discussed further in **Appendix H, Biodiversity assessment report.**

2.4 Surface water

2.4.1 Catchments

The project is located within the Bellinger catchment, which is a 1000 km^2 coastal catchment that extends from Scotts Head (50km south of Coffs Harbour) to the southern side of Yamba (95km north of Coffs Harbour). The western side of the catchment is bound by the Great Dividing Range, which is located between 0.6km to 4.8km west of the project. The catchment is located within the *Coffs Harbour Area Unregulated and Alluvial Water Sharing Plan (DWE, 2009),* however this management area does not cover the Bellinger River itself.

The Bellinger catchment comprises a relatively narrow floodplain bounded by steep mountainous areas of the Great Dividing Range, which creates a watershed for groundwater recharge to the Bellinger catchment. The steep, headwater areas of the catchment are typically well vegetated, while some of the floodplain and foothill areas have been cleared for agriculture and other uses *(Water in NSW, 2018).*

Several rivers and creeks within the catchment flow in an easterly direction, discharging into the Pacific Ocean. These creeks are fed by rainfall from within the catchment. Drainage in creek lines are predominantly surface water fed although the *Coffs Harbour Area Unregulated and Alluvial Water Sharing Plan* (*DWI, 2009*) states that base flow from up-river alluvial aquifers may have a high impact on instream values within the catchment. Baseflow from these aquifers provides the greatest impact on instream values during the drier season, when surface water run-off is less predominant.

Surface water run-off dominates in areas of steep topography within the catchment, limiting recharge to underlying groundwater, as rainfall preferentially runs off over the surface to surface water drainage channels. In the upper reaches of the catchments surface water flows are episodic in response to rainfall. In foot hill areas and further downgradient where the topography of the catchment is not as steep, rainfall recharge is likely to be greater and surface expression of the groundwater results in shallow ponds and lakes.

2.4.2 Rivers, lakes and water bodies

Several non-perennial watercourses and their tributaries intersect the study area. None of these watercourses feed directly into the Bellinger River, but instead flow directly to the coast. There are six creek lines within the study area which flow in a generally easterly direction from the foothills of the Great Dividing Range, discharging in the Pacific Ocean (**Figure 1**):

- Boambee Creek
- Newports Creek
- Coffs Creek
- Treefern Creek
- Jordans Creek
- Pine Brush Creek.

These watercourses are subject to tidal effects and estuarine processes up to their tidal limits and are connected to groundwater in alluvial deposits, particularly in up-river areas (**Section 2.5.2**).

There are numerous lakes, agricultural dams and surface water bodies within the study area (**Figure 1**), a number of which are directly intersected by the project. These bodies of water are more prevalent in topographically lower areas such as north of Korora Hill Interchange and south of Roberts Hill. They are typically often associated with and align with drainage lines which dissect the landscape. These surface water bodies are likely to be strongly surface water dominated but may be connected to the underlying groundwater where located above alluvial deposits.

In addition to these surface water bodies, there are several agricultural dams located within the project boundary and in the surrounding area. Further information on these is presented in **Section 2.6.11**.

2.4.3 Riparian areas

River bank stability is a key issue in the catchment due to land clearing for agriculture, human settlement and recreation *(Water in NSW, 2018).* Steeper areas of the catchment are under forest cover, while the floodplain and foothills have been cleared for grazing *(Water in NSW, 2018).* Riparian areas may be partially dependent upon the presence of groundwater although the level of dependency is likely to be variable. The presence of riparian vegetation can aid in minimising erosion and sedimentation within waterways. Natural drainage lines within the Coffs Harbour region are typically vegetated in a narrow riparian zone and the project intersects several of these riparian areas.

2.4.4 Wetlands

A review of the NSW Wetlands database published by the Office of Environment and Heritage (OEH) indicates there are estuarine wetland areas downstream (east of the project) that support coastal vegetation and fauna habitat. These wetland communities occur close to the coastline in lower lying areas of Boambee Creek, Newports Creek, Coffs Creek and Pine Brush Creek.

At the southern end of the project (to the west) are isolated occurrences of wetland areas associated with dams. A review of NSW State Environmental Planning Policy Coastal Management (*Coastal Management SEPP 2018*) indicates there are protected Coastal Wetland areas adjacent to the southern end of the study area as shown in **Figure 1**. These identified wetlands are not listed on the Directory of Important Wetlands in Australian *(ANCA, 1993).* No Ramsar wetlands or Nationally Important Wetlands have been mapped within the study area.

2.5 Regional geology

2.5.1 Geological units

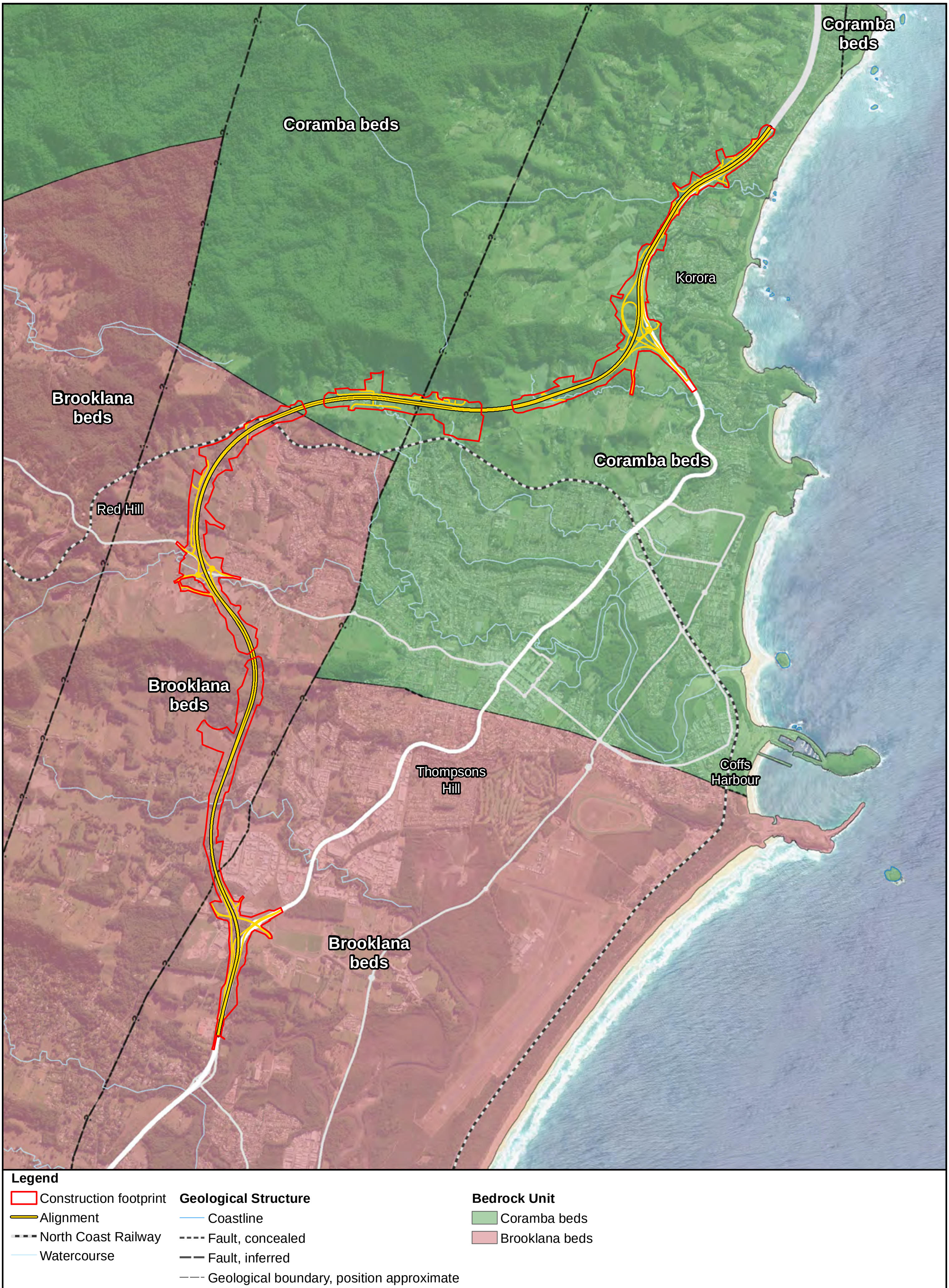
The project alignment is situated within the New England Orogen in eastern Australia. The Dorrigo – Coffs Harbour 1:250,000 scale Metallogenic Map *(Gillian et al., 1992)* indicates the project is underlain by two geological rock units, the Carboniferous aged Coramba Beds and the Brooklana Formation of the Coffs Harbour sequence (**Figure 3**). The mapped Coramba Beds extend beyond the southern end of the project to just north of the North Coast Rail Line; north of this point the project is underlain by the Brooklana Formation. Geological mapping indicates the rock units comprise of:

- Coramba Beds - lithofeldspathic wacke, minor siltstone, siliceous siltstone, mudstone, metabasalt, chert and jasper, rare calcareous material
- Brooklana Formation - thinly bedded siliceous mudstone and siltstone with rare lithofeldspathic wacke, locally chert, jasper, magnetite-bearing chert and metabasalt.

These origin of these bedrock units is interpreted to have been sedimentary turbidity currents derived from a volcanic arc source (*Korsch, 1981*). The sedimentary bedrock units have subsequently undergone two phases of metamorphism and some of the rock mass affected at higher grades has developed schistosity *(Graham and Korsch, 1985; RCA Australia, 2017a)*. Folding and faulting within the Coffs Harbour sequence has produced thrust blocks striking south-south-west to north-north-east that intersect the project (**Figure 3**). Geotechnical investigations indicate the rock mass contains multiple structures including bedding, foliation with a number of cleavage planes, cleavage, jointing, faulting, shear zones and veins *(Graham and Korsch, 1985; RCA Australia, 2017a)*. Further detailed descriptions of the geological setting along the alignment is provided in **Appendix C**.

Quaternary aged alluvial, swamp and estuarine sediments comprising sands, silts and gravels overlay the rock units in topographically low-lying areas and are generally associated with creek lines. The project intersects Quaternary alluvium which includes floodplain deposits, fan deposits, valley deposits and terrace deposits. Figure 4 shows the mapped location of alluvial deposits in the study area.

Geotechnical investigations also encountered residual and colluvial soils overlying weathered bedrock (RCA Australia, 2017a). Residual soils develop as a result of weathering of in situ bedrock whereas colluvial soils develop as a result of movement along slopes. The location and thickness of such deposits is highly variable within the study area.



Coffs Harbour Bypass
Bedrock Geology
Figure 3

Coffs Harbour Bypass
Bedrock Geology
Figure 4

2.5.2 Regional hydrogeological units

The *Water Sharing Plan for the Coffs Harbour Area Unregulated and Alluvial Water Sources* published by the NSW Department of Water and Energy (DWE) indicates there are four aquifers in the region *(DWE, 2009)*. The water sharing Plan describes the aquifers as:

- Coastal Sands Aquifer – groundwater is contained within the pore spaces of unconsolidated coastal sand sediments,
- Alluvial Aquifers – groundwater is contained in the pore spaces of unconsolidated floodplain material. The aquifer is further sub-divided into:
  - Up-river Alluvial Aquifers – coarse material in the upstream part of the catchment,
  - Coastal Floodplain Alluvial Aquifers – silts, clays and fine sands located further downstream where the floodplain flattens and widens, where the boundary between the alluvial aquifers is the tidal limit of the creek line in which the aquifer is located
- Fractured Bedrock Aquifers – groundwater mainly occurs within the fractures and joints of the rock mass.

Groundwater within the fractured bedrock aquifer is covered by the *Water Sharing Plan for the North Coast Fractured and Porous Rock Groundwater Sources* (*DPI, 2016*).

The water sharing plans recognise that groundwater and surface waters are connected, with the aim managing water as a single resource. However, the connectivity between surface water and groundwater units (i.e. the amount of interflow between the units through recharge and discharge) varies and has a direct influence on the water balance and management of water resources. Those aquifers identified in the water sharing plans have varying degrees of connection with surface waters as described in **Table 4** *(DWE, 2009)*. Alluvial aquifers have a high degree of connectivity with surface waters and are included in water sharing plans that cover surface water and its connected alluvial groundwater. Fractured bedrock aquifers conversely have low to moderate connectivity and are included in a groundwater specific plans.

The project may directly or indirectly impact upon the alluvial and fractured bedrock aquifers due to the proximity of the proposed concept design elements to these aquifers, and the type of design elements proposed. For example, design elements such as cuttings may intercept one or more of the aquifers, which typically causes groundwater to drain towards the cutting – redistributing the local flow paths. Further discussion of the impact of the design elements and the regional aquifers is provided in **Section 4**.

The project in not anticipated to affect the coastal sands aquifer due to the distance from the alignment.

Table 4: Excerpt of connectivity between aquifer types and surface water *(DWE, 2009)*

Aquifer type	Water sources	Level of connection between surface and groundwater	Level of impact on in stream values	Estimated time between groundwater and unregulated river
Coastal sands	Coffs Harbour coastal sand and all unregulated rivers[1]	Significant (tidal section only)	Low due to connection with saline water	One day to months
Up-river alluvial	All unregulated rivers[1]	Significant	High due to impact on base flows	Days to months
Coastal floodplain alluvial	Most unregulated river[1] water sources except Dirty Creek, Corindi River, Red Bank River and Arrawarra Creek	Low – moderate (tidal section only)	Low since not a major contributor and low level of connection	One season
Fractured rock	All unregulated rivers[1]	Low - moderate	Low since not a major contributor	Years to decades

Note:
1. Unregulated river applies to rivers that do not have major storages along their alignment such as dams

2.5.3 Acid sulfate soils

Acid Sulphate Soils (ASS) are sediment deposits that contain iron bearing sulfides. ASS are typically found in swamps and estuaries below 10 m AHD and where groundwater creates a reduced (oxygen deprived) environment. Left undisturbed ASS are generally harmless and considered potential acid sulfate soils (PASS). If PASS is disturbed by activities such as excavation or lowering groundwater levels, the PASS materials can oxidise rapidly to form sulfuric acid and mobilise aluminium and heavy metals within the subsurface.

The generation of acid and heavy metals can lead to contamination of, and impacts on, the groundwater environment. Deposits with PASS may oxidise due to construction activities such as dewatering, clearing or the permanent earthworks (for example, cuttings) and permanent drainage modifying the groundwater regime where PASS is located. Acid sulfate materials can also be present in bedrock units and their disturbance can also lead to generation of acid leachate.

The ASS Risk Map *(Naylor et al., 1998)* and Acid sulfate rock risk map (Roads and Maritime 2017) indicates:

- The southern end of the project intersects areas with a low probability of PASS associated with Boambee and Newports creeks and their tributaries. At this location there is a low probability of PASS at depths between two to four metres and greater than four metres below ground level.

- High ASS risk areas are located about 120m east of the southern end of the project where PASS are possible at less than one metre to two metres below ground level and the Boambee Creek bed sediments.
- The northern end of the project intersects mapped high-risk ASS near Pine Brush Creek, where the anticipated depth of PASS is greater than four metres below ground level.
- The construction footprint is located in areas of low and medium ASR risk. Medium risk areas are generally associated with the meta-sediment rock in the Coffs Harbour region

Petrographic and acid base accounting laboratory testing was completed for selected rock samples collected along the project corridor to determine the presence of ASR. Test results indicate the rock samples have sufficient acid neutralising capacity to buffer acid produced by sulfides in the rock mass and that ASR is unlikely to be a risk to the project.

Figure 5 shows the location of PASS along the project alignment.

Potential impacts and management of ASS material is discussed further in **Section 4**.

Legend

Ancillary sites
North Coast Railway
Watercourse

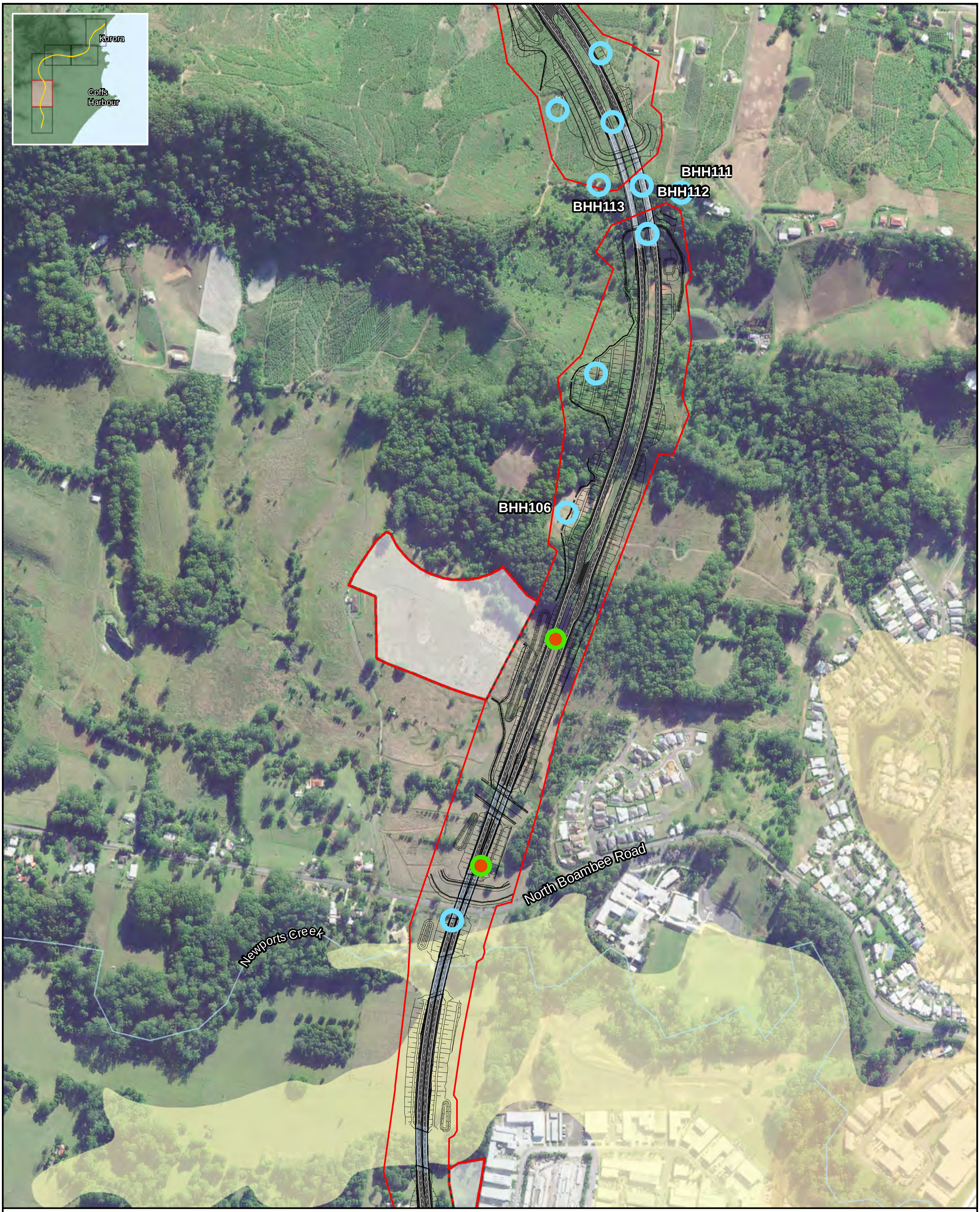
Groundwater salinity
Fresh
Brackish

Potential Acid Sulfate Soils
pH(Fox) < = 4
pH(Fox) > 4

Acid Sulfate Soil Risk
High Risk 0-1m
High Risk 1-2m
High Risk 2-4m
High Risk Sediments
Low Risk 2-4m
Low Risk above 4m
Disturbed Terrain

Coffs Harbour Bypass
Groundwater Quality and ASS Risk
Figure 5-1

0 0.1 0.2 0.3 km
Scale @A4: 1:10,000
GDA 1994 MGA Zone 56
N



Legend

Ancillary sites
North Coast Railway
Watercourse

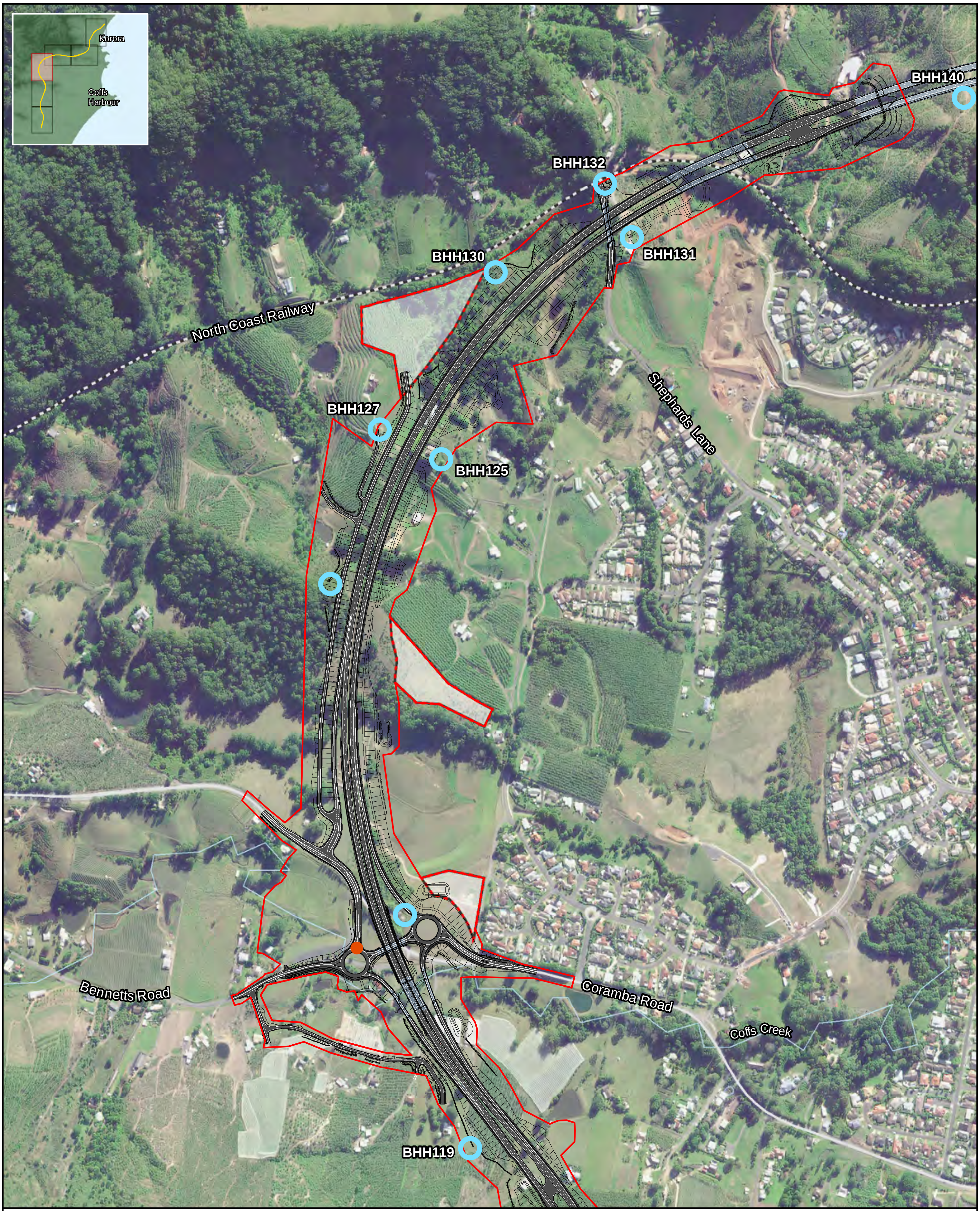
Groundwater salinity
Fresh
Brackish

Potential Acid Sulfate Soils
pH(Fox) < = 4
pH(Fox) > 4

Acid Sulfate Soil Risk
Low Risk 2-4m
Low Risk above 4m

0 0.1 0.2 0.3 km
Scale @A4: 1:10,000
GDA 1994 MGA Zone 56
N

Coffs Harbour Bypass
Groundwater Quality and ASS Risk
Figure 5-2



Legend

Ancillary sites

North Coast Railway

Watercourse

Groundwater salinity

Fresh

Brackish

Potential Acid Sulfate Soils

pH(Fox) < = 4

pH(Fox) > 4

Coffs Harbour Bypass
Groundwater Quality and ASS Risk
Figure 5-3

0 0.1 0.2 0.3 km

Scale @A4: 1:10,000
GDA 1994 MGA Zone 56

N

Legend

Ancillary sites

North Coast Railway

Watercourse

Groundwater salinity

Fresh

Brackish

Potential Acid Sulfate Soils

pH(Fox) < = 4

pH(Fox) > 4

Coffs Harbour Bypass
Groundwater Quality and ASS Risk
Figure 5-4

0 0.1 0.2 0.3 km

Scale @A4: 1:10,000
GDA 1994 MGA Zone 56

Legend

Ancillary sites
North Coast Railway
Watercourse

Groundwater salinity
Fresh
Brackish

Potential Acid Sulfate Soils
pH(Fox) < = 4
pH(Fox) > 4

Acid Sulfate Soil Risk

Disturbed Terrain

Coffs Harbour Bypass
Groundwater Quality and ASS Risk
Figure 5-5

0 0.1 0.2 0.3 km
Scale @A4: 1:10,000
GDA 1994 MGA Zone 56

Legend

Ancillary sites
North Coast Railway
Watercourse

Groundwater salinity

Fresh
Brackish

Potential Acid Sulfate Soils

pH(Fox) < = 4
pH(Fox) > 4

Acid Sulfate Soil Risk

High Risk above 4m
Low Risk Sediments
No Risk
Disturbed Terrain
Beach

Coffs Harbour Bypass
Groundwater Quality and ASS Risk
Figure 5-6

0 0.1 0.2 0.3 km
N
Scale @A4: 1:10,000
GDA 1994 MGA Zone 56

2.5.4 Salinity

The DPI *Salinity hazard report for Catchment Action Plan upgrade – Northern River CMA*, indicates the study area is underlain principally by landscapes with a very low salinity hazard potential *(Nicholson et al., 2012)*. Minor areas associated with the risk of ASS are mapped as having a very high salinity hazard potential. The two salinity landscapes in the study area are:

- Coastal Ranges Metasediments landscapes – very low hazard - corresponding to the foothills and slopes of the Coramba Beds and Brooklana Formation, which most of the bypass is located within
- Acid Sulfate Potential landscapes – very high hazard - corresponding to areas identified on ASS risk map, typically in lower lying topographical areas of the bypass and along the coastal areas of Coffs Harbour.

An excerpt of the salinity hazard landscape characteristics and potential for salinization is presented in **Table 5**.

Table 5: Salinity hazard landscape characteristics *(Nicholson et al., 2012)*

Landscape	Acid Sulfate Potential	Coastal Ranges Metasediments
Mapping area	Very high hazard	Very low hazard
Salinity hazard	Very high	Very low
Significance	Landscapes with a potential ASS risk have predominantly very high salinity hazard, high salt lad, high salt store and low water quality. Soils tend to be highly saline. Regular inundation from brackish tidal water contributes to salt store.	Steep, dissected landscape with exposed rock and shallow soils offer low storage capacity of salts.
Resilience	Salinity is primarily driven by shallow cyclic flows, estuarine and acid sulfate influences. Other salinity drivers include increased urbanisation, overuse of water, leakage of stormwater infrastructure and water delivery systems and inappropriate siting of infrastructure. Variables controlling resilience include exposure of PASS, planning, policy, siting of infrastructure, constructions methods, water use patterns and volumes, localise volume of saline substrate and extent of saline land	Drivers of salinity may include clearing of native vegetation. The likelihood of salinity development is low.
Confidence	Moderate Landscape salinity is mapped and observed.	Little salt mapped or observed.

As part of the groundwater monitoring programme, electrical conductivity testing of 30 groundwater samples from standpipes installed within the fractured bedrock units was undertaken *(RCA Australia, 2017a)*. Groundwater samples were collected from standpipes that were installed from depths between 6 metres below ground level (mbgl) and 64 mbgl across the alignment. The results (as shown in **Figure 5**) indicated the groundwater is predominantly fresh (28 of the 30 samples with an electrical conductivity of less than 0.8 dS/cm) *(NHMRC and NRMMC, 2011)*. Laboratory testing of several of the samples confirmed that the groundwater quality is generally fresh, with total dissolved solids ranging from 86 mg/l to 262 mg/l.

Two samples registered slightly higher electrical conductivity values, in the brackish range, from BHH150 and BHH169 samples, located at Gatelys Road and near the waste facility at the southern end of the alignment respectively. The groundwater sample taken from BHH150 is from one of the deepest standpipes along the alignment, screened at approximately 58 mbgl to 64 mgbl. This may be indicative of an increase in salinity with depth, possibly due to increased residence times of groundwater in the aquifer, although the overall dataset showed little correlation between sample depth and salinity. Additionally, both samples registered electrical conductivity values of approximately 1.2dS/cm indicating that the salinity of these samples is only very slightly above the freshwater/brackish threshold.

Except in areas where ASS are present, the likelihood of salinity issues being encountered is considered to be low.

2.6 Groundwater

2.6.1 Groundwater occurrence

Three groundwater bearing strata have been identified within the project which includes shallow surficial deposits, alluvial deposits and fractured bedrock. Groundwater is also present within the Coffs Harbour coastal sand aquifer however this is located outside of the study area and is not expected to be affected by the project due to its distance. The conceptual understanding of the hydrogeological setting including connectivity between aquifers and flow is further described in **Section 2.7**.

2.6.1.1 Surficial/perched groundwater

The shallow surficial deposits comprise of relatively thin colluvial and residual soils horizons, comprising clays, silts and gravels that overlie the Brooklana Formation and Coramba beds in the hill slopes and foothill areas. The distribution of surficial materials in the study area is highly variable. The unit is unlikely to act as a single groundwater body, instead presenting as a series of disconnected local perched systems.

It is anticipated that this aquifer is often unsaturated, with groundwater temporarily perching in this unit following rainfall recharge events. The perched groundwater is expected to infiltrate to the underlying fractured bedrock aquifer and/or move downgradient towards drainage lines and creeks in the surrounding topography. These deposits are not considered an aquifer in its normal sense as they are not a reliable groundwater source. The quantity of groundwater that is stored and flowing through these materials is likely to be small but may be locally important for some vegetation.

2.6.1.2 Alluvial aquifers

Alluvial aquifers in the Coffs Harbour region occur along drainage lines which interfinger topographically higher areas. The alluvial aquifer units are separated into two types; a shallow up-river alluvial aquifer and a coastal floodplain alluvial aquifer. The up-river alluvial aquifer system within Quaternary aged alluvial deposits increases in thickness east towards the coastal floodplain alluvial and coastal sand deposits. As encountered in the geotechnical investigations, the aquifer comprises interbedded silt, clay, sand and gravel with zones of lower and higher permeability.

The coastal floodplain alluvial aquifer, the boundary of which is defined by the tidal limit of the creek, typically comprises of finer grained deposits such as fine-grained sands, silts and clays. These deposits occur further downstream where the floodplain flattens and widens. Due to the finer grained nature of the deposits, connectivity to surface waters tends is reduced compared to the up-river unit, which is strongly connected (*DWE, 2009).*

Recharge to these aquifers is anticipated from two sources. The first source is from precipitation which is recharged to the aquifer through shallow surficial material and direct recharge from the rainfall into the aquifer. The second source of recharge is from direct connection (interflow) between the alluvial deposits and surface water within the creek lines. Groundwater recharge to alluvium and surface water from the underlying fractured bedrock is likely to be low; the impact on in-stream values from fractured bedrock is low according to the water sharing plans *(DPI, 2016, DoW 2009).*

The water sharing plan indicates the up-river alluvial aquifer is highly connected to creeks in the Coffs Harbour region and has a high impact on instream flow due to contribution from baseflow. The estimated travel time between groundwater and creek (period of time between recharge entering an aquifer and discharging) ranges from days to months *(DWE, 2009)*. The coastal floodplain aquifer is less connected to the creeks and therefore has a low impact on instream values.

Based on geological mapping, the project intersects alluvial deposits at several locations along the alignment (**Figure 4**). Based on the water sharing plan, these deposits are all part of the up-river alluvial deposits.

2.6.1.3 Fractured bedrock aquifer

The shallow surficial and alluvial aquifers near the Project are underlain by the fractured bedrock aquifer comprising the Brooklana Formation and Coramba beds. Groundwater in these units is in geological structures that include faults, shear zones, joint sets and cleavage planes, that in a large part have been created by regional metamorphism *(Graham and Korsch, 1985; RCA Australia, 2017a).*

The fractured bedrock aquifer forms part of a large regional groundwater source known as the New England Fold Belt Coast groundwater source (*DPI, 2016*). The regional fractured bedrock aquifer extends across a large part of the NSW coast from Nelsons Bay in the south to Woolgoolga in the north and up to the Queensland boarder further inland. From north to south the aquifer source is over 450km long and extends from the coast inland up to 150km.

The bedrock has low primary permeability (i.e. through pore spaces), except where the rock has undergone significant weathering. Groundwater storage, permeability and flow within the rock mass is principally within secondary defects (joints, fractures, faults) and weathered zones. The fractured bedrock aquifer is recharged via rainfall infiltration from the overlying surficial deposits or directly to the bedrock at outcrop in the upper reaches of the catchment which extends far to the west of the Project.

Although locally the quantity of groundwater and flow in the fractured bedrock may be low, the aquifer has a regional scale and the aquifer comprises a thick sequence of fractured rock units. Rainfall recharge to the west of the study area is likely to contribute to a deep regional groundwater flow system. Groundwater within the bedrock in the study area forms part of a shallower, more local system, where recharge and flow paths are less connected to the regional scale system. Compared to the alluvial aquifers, groundwater movement within the fractured bedrock is slow and may take years to decades or longer from the point of recharge to discharge *(DPI, 2016)*.

Groundwater flows in the shallower fractured bedrock are generally expected to follow the topographical features of the area which are broadly towards the east, except locally at ridge lines. Groundwater flow within the deeper regional bedrock is less likely to be affected by local topographic variation with flow anticipated to be eastwards towards the coast, potentially exhibiting strong vertical gradients.

2.6.2 Groundwater level monitoring data

As part of field investigations along the alignment, groundwater monitoring was conducted at 32 standpipes (25 with continuous monitoring data) and 17 vibrating wire piezometers installed within the fractured bedrock aquifer along the alignment *(RCA Australia, 2017a)*. Near continuous groundwater level monitoring data is available for the period July-17 to Feb-19 (20 months). In addition, standing water levels from 29 licensed bores from the DPI are provided in the groundwater monitoring report.

Groundwater hydrographs from monitoring locations installed during geotechnical investigations are presented in **Appendix A**. **Figure 6** to **Figure 11** present groundwater hydrographs plotted against daily rainfall from Coffs Harbour airport from monitoring locations in the following sections of the project:

- South of Roberts Hill
- Roberts Hill
- Roberts Hill to Shephards Lane
- Shephards Lane
- Gatelys Road

- North of Gatelys Road.

The position of monitoring wells and range of groundwater levels observed is presented in **Figure 12** and **Table 6**.

Across the project, groundwater levels range from between 11 mAHD to 117 mAHD. The variation in groundwater elevations along the alignment corresponds to similar changes in topography, with the highest groundwater elevations generally corresponding to the highest topographic areas around the ridgelines at Shephards Hill and Gatelys Road. Groundwater levels below ground level vary from less than 5m to approximately 43m, with the deepest groundwater generally occurring in topographically higher areas.

Groundwater levels closest to ground level were recorded at BHH114, BHH148 and BHH160. The highest groundwater levels recorded at these standpipes was within 2 – 4m of ground surface. The highest groundwater levels as recorded by one of the vibrating wire piezometers was at BHH148 which recorded groundwater pressures up to 3m above ground level.

Groundwater levels are affected by seasonal climatic variation and rainfall events. Between the wetter period between November and April, groundwater levels are generally elevated (at most) monitoring locations compared to the period May to October, when groundwater levels were generally in recession, owing to the lower rainfall.

Figure 6: Groundwater monitoring hydrographs (south of Roberts Hill)

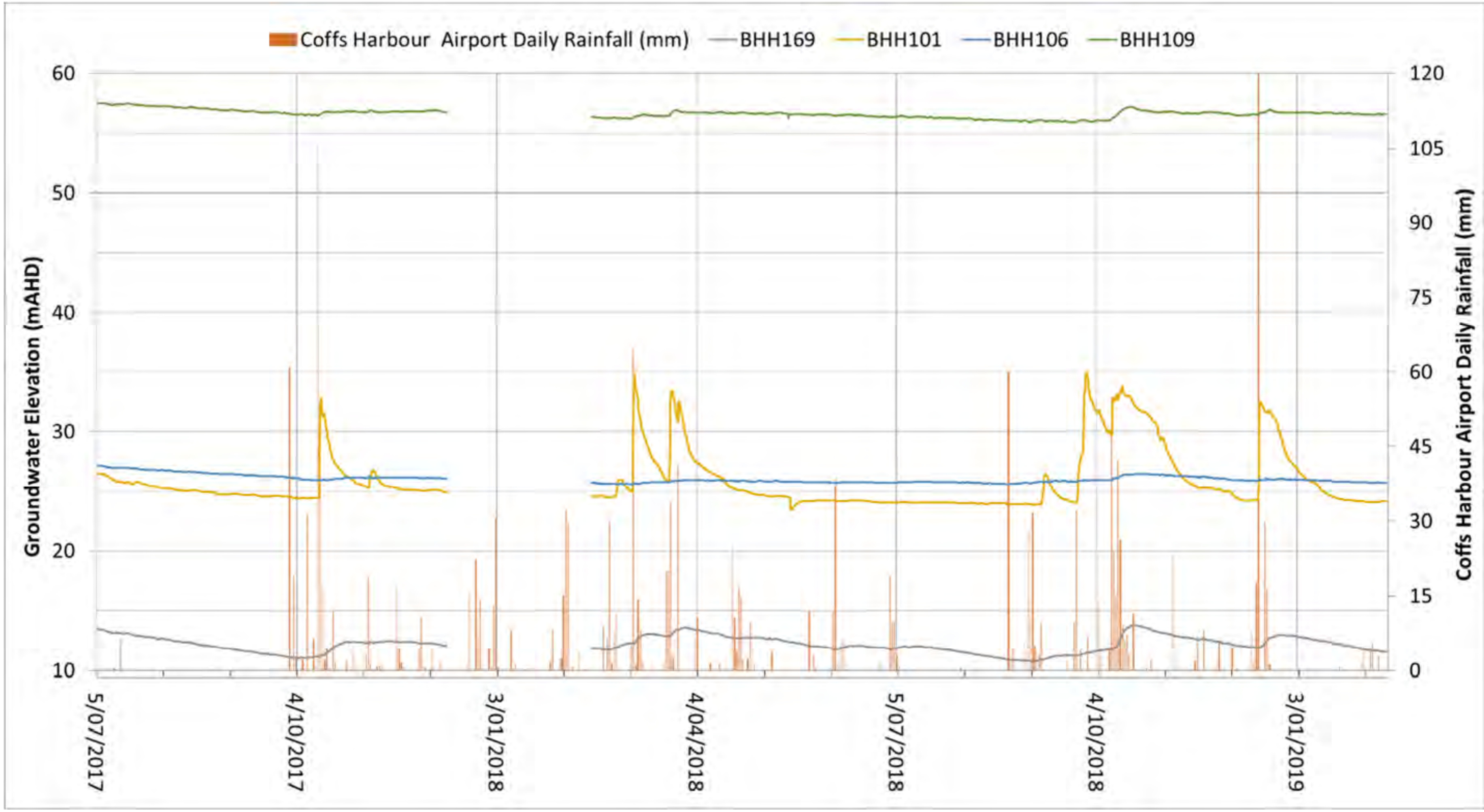


Figure 7: Groundwater monitoring hydrographs (Roberts Hill)

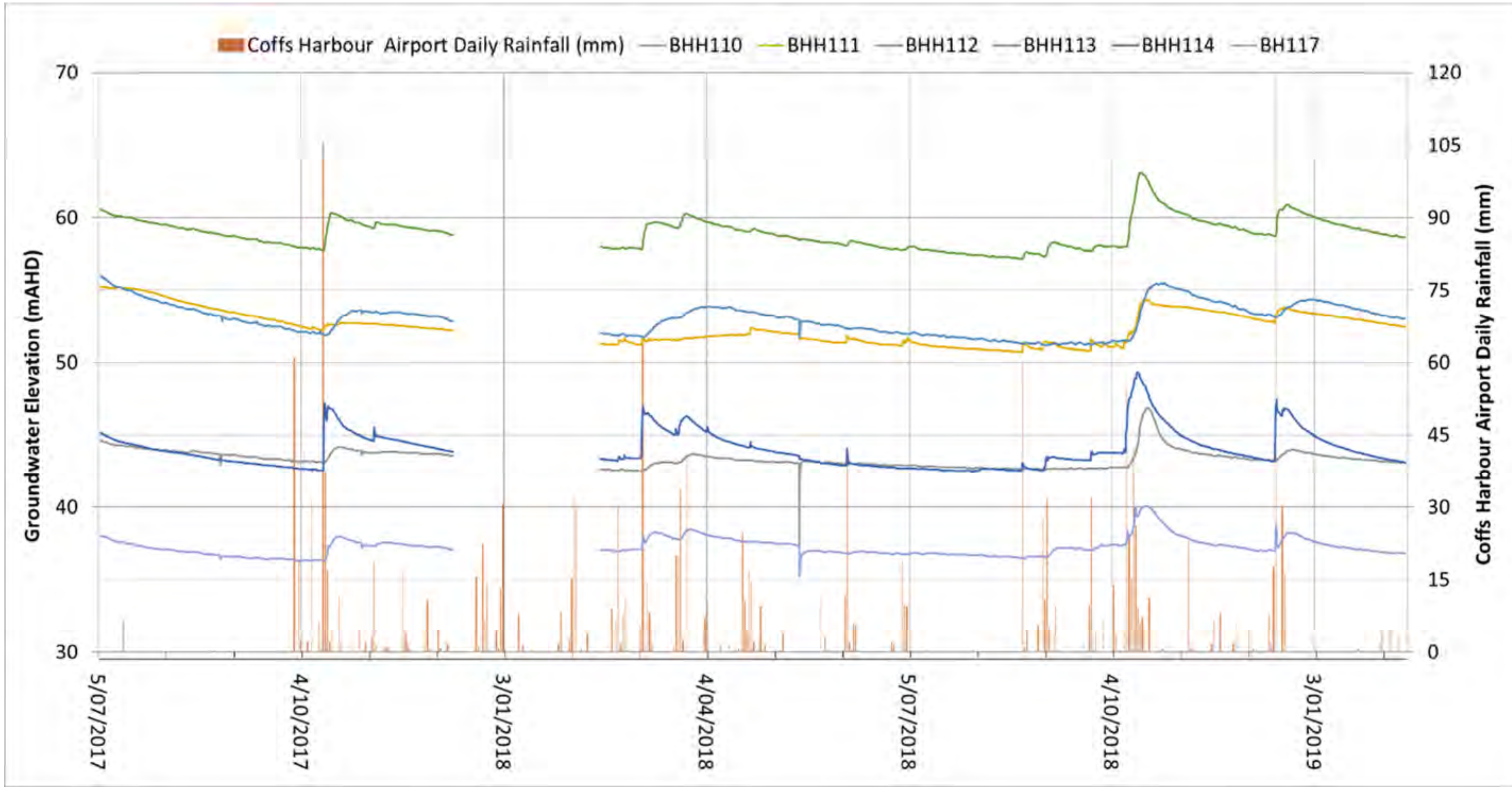


Figure 8: Groundwater monitoring hydrographs (Roberts Hill to Shephards Lane)

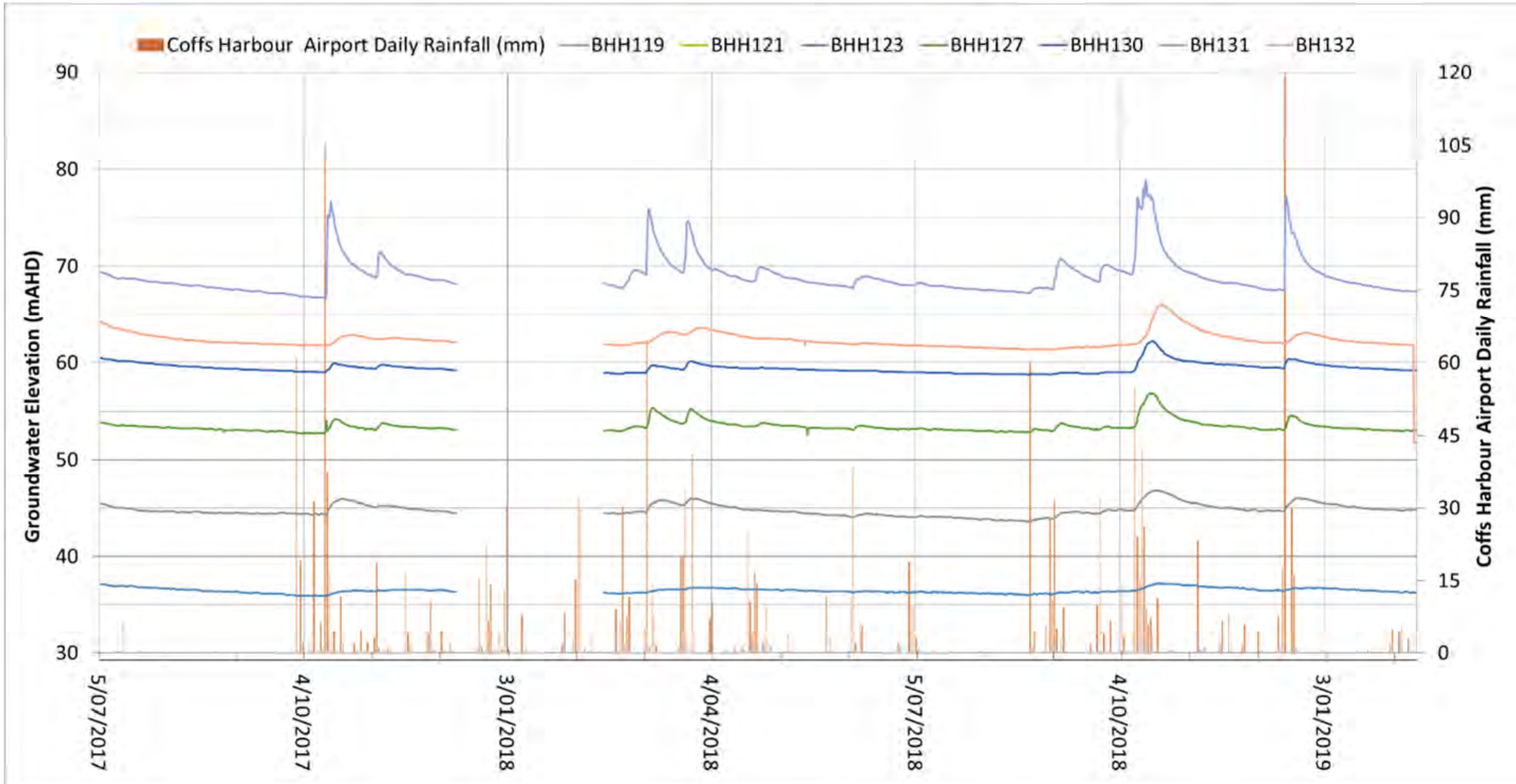


Figure 9: Groundwater monitoring hydrographs (Shephards Lane)

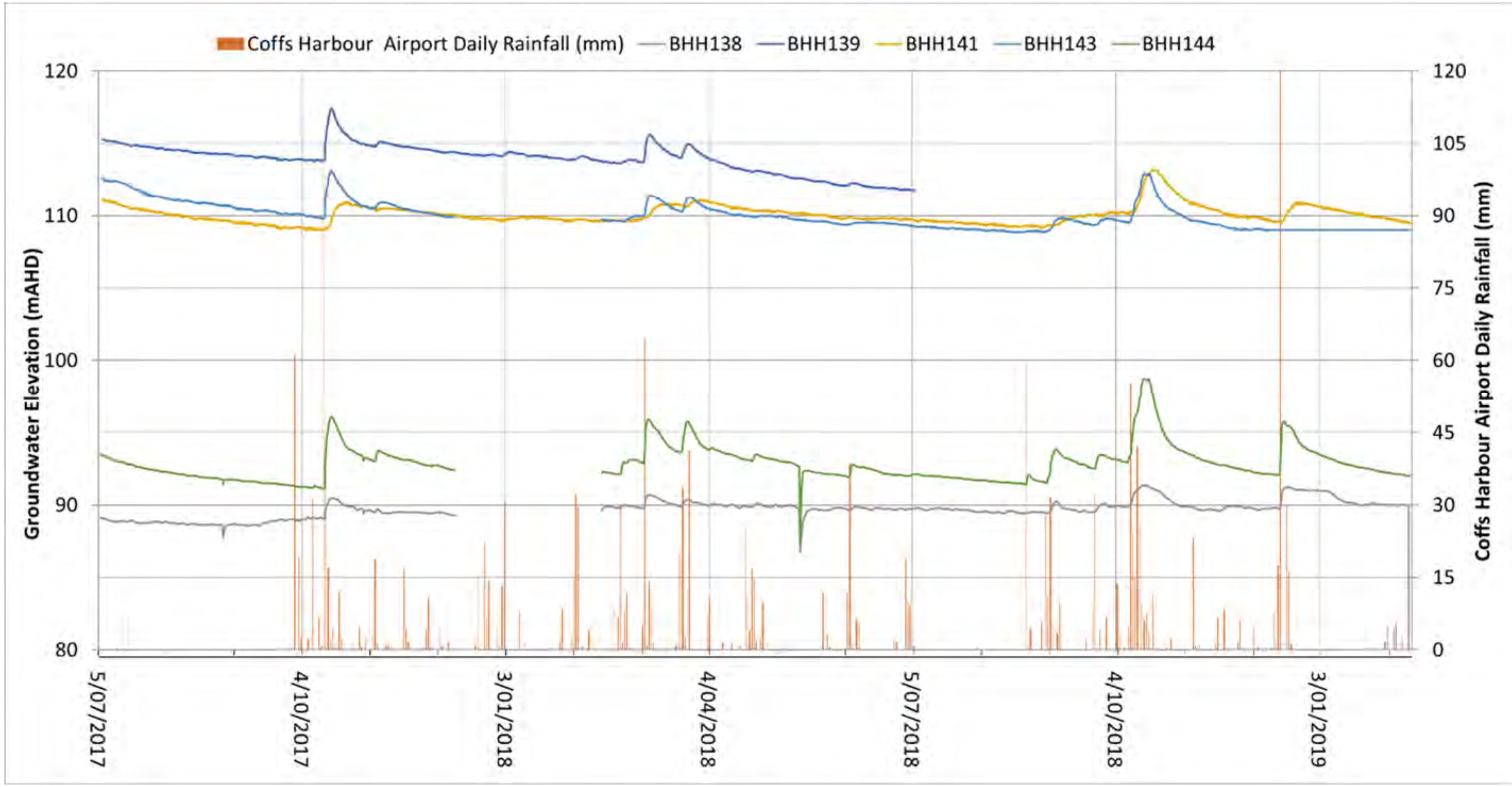


Figure 10: Groundwater monitoring hydrographs (Gatelys Road)

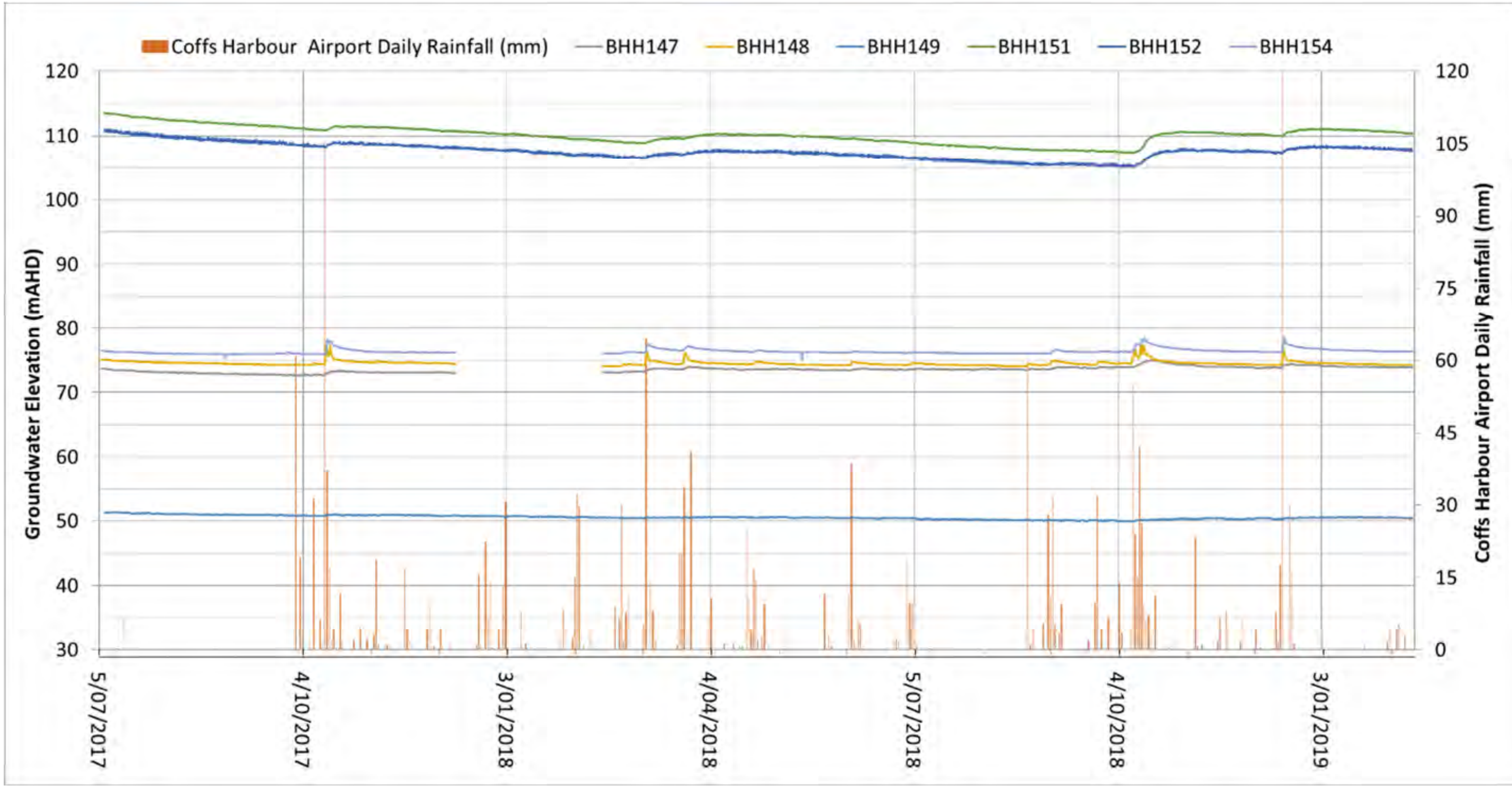


Figure 11: Groundwater monitoring hydrographs (North of Gatelys Road)

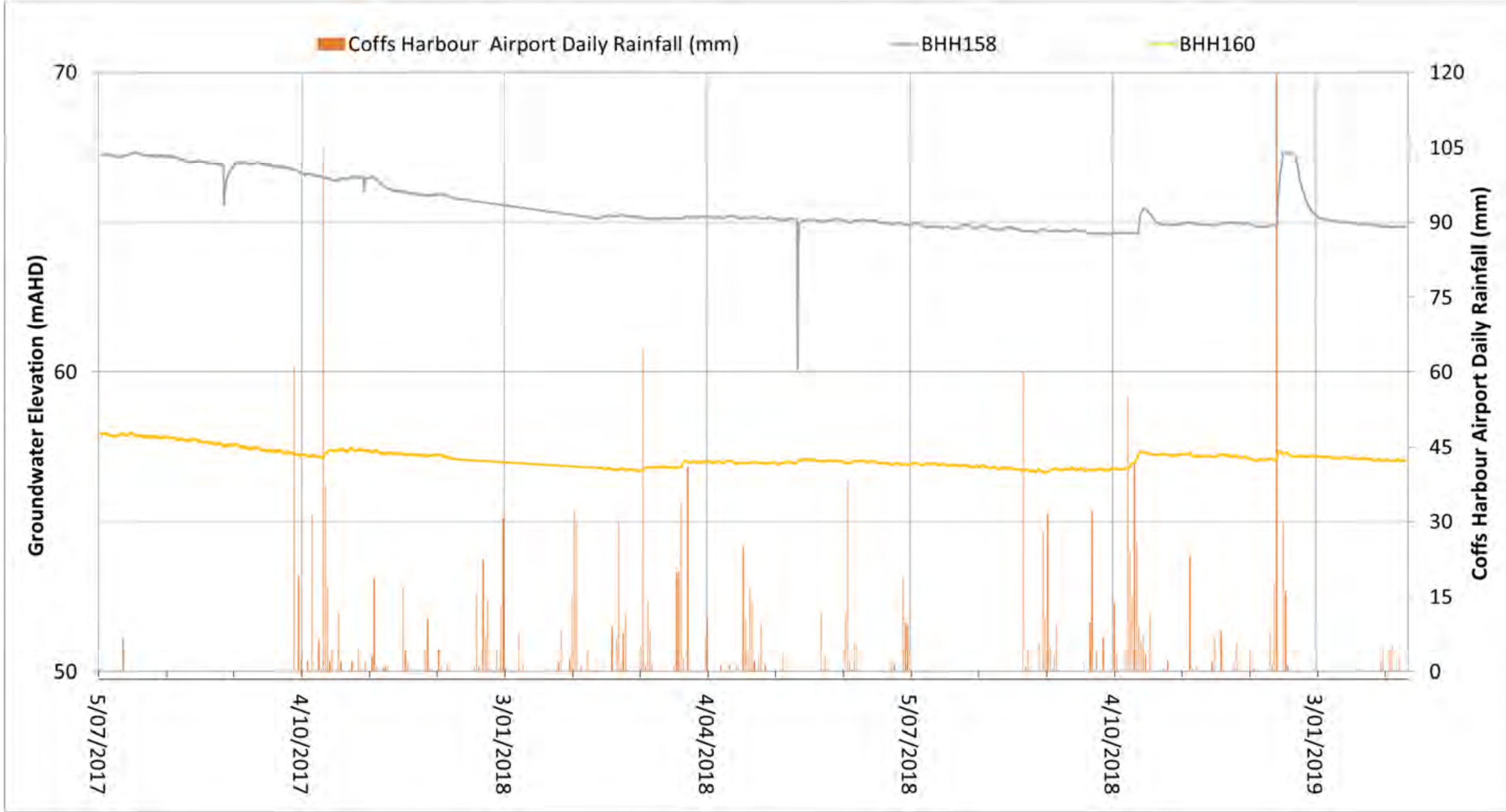


Table 6: Groundwater observations (datalogger monitoring locations)

ID	Approximate Chainage	Ground RL (mAHD)	Instrument	Groundwater level high (mbgl)		Groundwater level low (mbgl)		Range (m) [1]
				mbgl	mAHD	mbgl	mAHD	
BHH169SP	10525	25.0	Standpipe	11.1	13.9	14.2	10.8	3.1
BHH101SP	10325	42.4	Standpipe	7.7	34.7	18.5	23.9	10.8
BHH106SP	13000	38.2	Standpipe	11.7	26.5	12.6	25.6	0.9
BHH109SP	13310	70.5	Standpipe	12.9	57.6	14.6	55.9	1.7
BHH110SP	13620	52.1	Standpipe	5.3	46.8	9.5	42.6	4.2
BHH111SP	13700	83.9	Standpipe	23.3	60.6	33.2	50.7	9.9
BHH112SP	13725	84.0	Standpipe	27.4	56.6	32.8	51.2	5.4
BHH113SP	13760	83.7	Standpipe	20.7	63.0	26.6	57.1	7.2
BHH114SP	13880	51.7	Standpipe	2.3	49.4	9.2	42.5	6.9
BHH115SP	13950	72.0	Standpipe	14.4	57.6	17.8	54.2	3.4
BHH117SP	14000	53.7	Standpipe	13.7	40.0	17.4	36.3	3.8
BHH119SP	14300	52.4	Standpipe	5.6	46.8	8.9	43.5	3.3
BHH121SP	14800	34.4	Standpipe	9.6	24.8	12.6	21.8	3.1
BHH123SP	15475	57.4	Standpipe	20.2	37.2	21.6	35.8	1.4
BHH127SP	15800	76.7	Standpipe	19.9	56.8	24.0	52.7	4.1
BHH125SP	15810	60.4	Standpipe	15.0	45.4	16.2	44.2	1.2
BHH130SP	16175	86.7	Standpipe	24.5	62.2	28.0	58.7	3.5
BHH131SP	16425	95.7	Standpipe	16.9	78.8	29.0	66.7	12.1
BHH132SP	16450	94.1	Standpipe	28.2	65.9	32.8	61.3	4.6
BHH138SP	17000	98.0	Standpipe	6.7	91.3	9.5	88.5	2.8
BHH140SP	17200	161.2	Standpipe	38.3	122.9	47.0	114.2	8.7
BHH142SP	17200	158.0	Standpipe	23.5	134.5	24.8	133.2	1.3
BHH144SP	17350	105.4	Standpipe	6.7	98.7	14.2	91.2	7.6

ID	Approximate Chainage	Ground RL (mAHD)	Instrument	Groundwater level high (mbgl)		Groundwater level low (mbgl)		Range (m) [1]
				mbgl	mAHD	mbgl	mAHD	
BHH147SP	18450	97.4	Standpipe	22.4	75.0	24.8	72.6	2.4
BHH148SP	18900	79.7	Standpipe	1.8	77.9	5.6	74.1	3.8
BHH150SP	19100	154.1	Standpipe	40.7	113.4	41.9	112.2	1.2
BHH153SP	19220	136.2	Standpipe	27.8	108.4	29.5	106.7	1.7
BHH154SP	19350	87.7	Standpipe	9.0	78.7	11.7	76.0	2.7
BHH158SP	20450	83.6	Standpipe	16.2	67.4	19.0	64.6	2.8
BHH160SP	20550	62.1	Standpipe	4.1	58.0	5.5	56.6	1.4
BHH163SP	21350	59.5	Standpipe	19.0	40.5	19.6	39.9	0.6
BHH110VWP	13620	52.1	VWP	5.5	46.6	11.4	40.7	5.9
BHH111VWP	13700	83.9	VWP	VWP not functioning				
BHH112VWP	13725	84.0	VWP	30.7	53.3	35.2	48.8	4.5
BHH113VWP	13760	83.7	VWP	VWP not functioning				
BHH114VWP	13880	51.7	VWP	1.0	50.7	8.9	42.8	7.9
BHH139aVWP	17150	127.7	VWP	12.0	115.7	17.0	110.7	5.0
BHH139bVWP	17150	127.7	VWP	10.3	117.4	16.0	111.7	5.7
BHH140VWP	17200	161.2	VWP	VWP not functioning				
BHH141aVWP	17200	151.7	VWP	38.5	113.2	42.7	109.0	4.2
BHH141bVWP	17200	151.7	VWP	36.4	115.3	40.5	111.2	4.1
BHH142VWP	17200	158.0	VWP	VWP not functioning				
BHH143aVWP	17250	122.1	VWP	9.2	112.9	13.6	109.3	4.2
BHH143bVWP	17250	122.1	VWP	9.0	113.1	13.4	109.5	4.2
BHH148VWP	18900	79.7	VWP	-3.1	82.8	-0.3	80.6	2.8
BHH149aVWP	19050	79.5	VWP	28.2	51.4	29.6	50.4	1.4
BHH149bVWP	19050	79.5	VWP	28.9	50.7	31.1	49.2	2.2

ID	Approximate Chainage	Ground RL (mAHD)	Instrument	Groundwater level high (mbgl)		Groundwater level low (mbgl)		Range (m) [1]
				mbgl	mAHD	mbgl	mAHD	
BHH150VWP	19100	154.1	VWP	VWP not functioning				
BHH151aVWP	19125	150.5	VWP	36.9	113.6	43.2	108.8	6.3
BHH151bVWP	19125	150.5	VWP	36.3	114.3	42.6	109.5	6.3
BHH152aVWP	19200	139.1	VWP	28.0	111.0	33.6	106.4	5.6
BHH152bVWP	19200	139.1	VWP	27.6	111.4	33.1	107.0	5.5
BHH153VWP	19220	136.2	VWP	VWP not functioning				
BHH154VWP	19350	87.7	VWP	3.4	83.9	5.6	80.6	2.2

Notes

[1] Groundwater level ranges include the effect of short term rainfall induced increases in groundwater level

Groundwater hydrographs indicate that groundwater has a variable response to individual rainfall events. Between July-17 and late Oct-17 almost no rainfall was recorded and during this time, groundwater levels in most standpipes showed a decline in groundwater levels of between 0.5m and 4m.

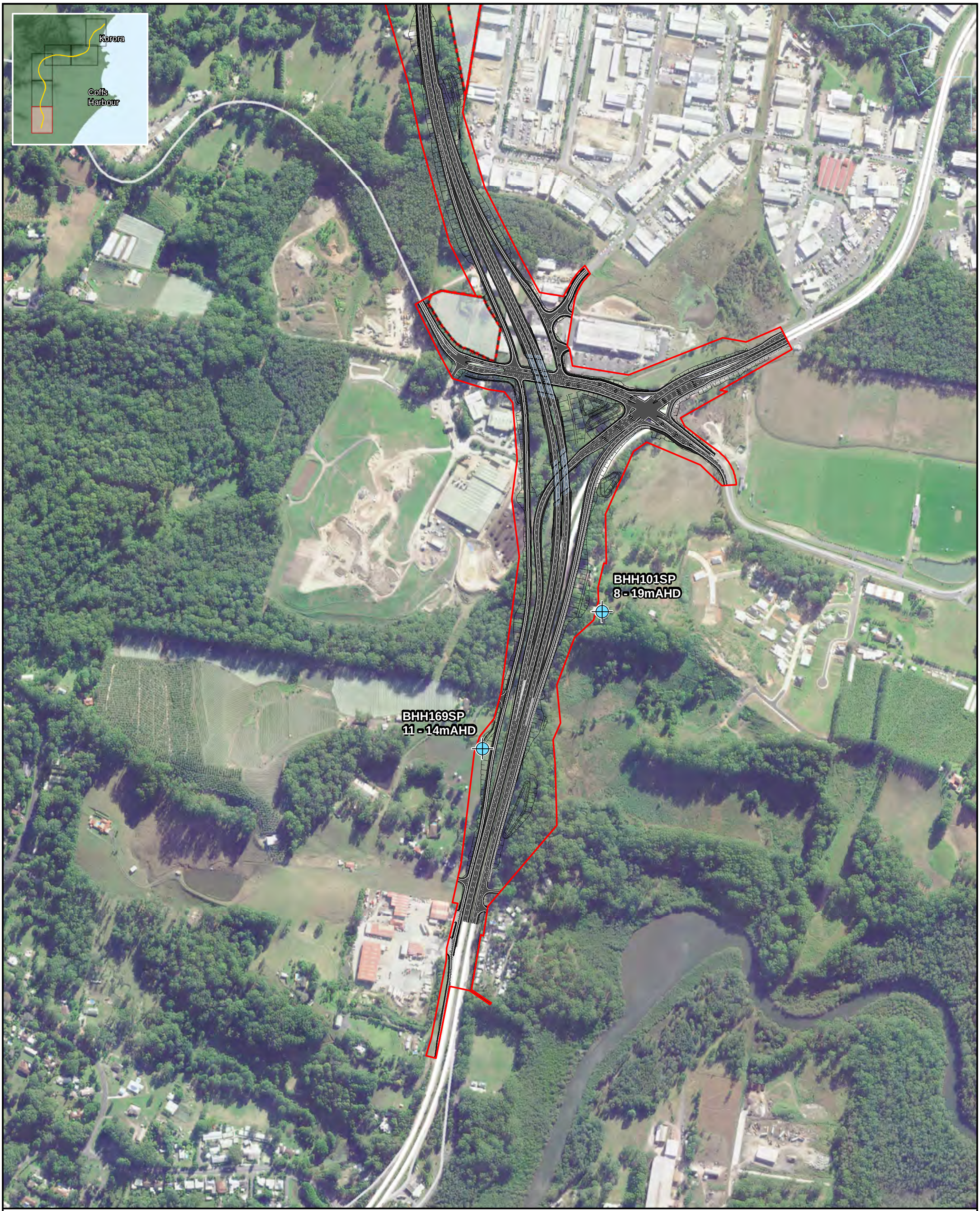
Monitoring data also suggests that groundwater levels respond relatively rapidly to large rainfall events. On the 14th Oct-17 a daily rainfall total of 105mm was recorded with a total of 174mm of rainfall in the seven days from the 12th Oct to the 18th Oct. Response to this rainfall event varied significantly between standpipe locations. Groundwater levels at BHH101 and BHH131, increased by between 9 and 10m indicating a very strong connection to rainfall recharge whereas at BHH109, BHH158 and BHH160, groundwater levels showed only minor changes (less than 0.3m). The average observed increase in water levels at all monitored standpipes was 2.2m. The data showed only a very slight correlation with screen depth indicating that the shallower groundwater may show a greater response to rainfall events.

The rate of response to rainfall events also varies between standpipe locations. The time to reach maximum groundwater level following the 14th Oct 2017 rainfall event was evaluated at each standpipe location. Response times varied from less than one day to 16 days. Generally, those standpipes which showed the greatest magnitude of response (BHH101, BHH131, BHH1144, BHH144) responded within 1 day to 4 days. All standpipes which showed a greater than 2m increase in groundwater levels responded within less than 4 days. Those with less than 2m change had significantly more variability in response time, ranging from 2 days to 16 days.

The groundwater recession period following rainfall events also varied significantly but was generally slower than the initial increase in water levels. The initial increase in groundwater levels occurs as a response to the filling of unsaturated storage within the rock via vertical infiltration. Groundwater recession is typically much slower since groundwater flow becomes lateral once it has reached the groundwater table and takes longer to discharge downgradient under natural head gradients. Additionally, it is likely that continued recharge into the system occurs during recession periods due to additional rainfall and vertical leakage from perched groundwater within residual soils overlying the fractured bedrock. In the context of the project, this response indicates that groundwater levels may increase rapidly following rainfall events and remain elevated for an extended period afterwards.

The seasonal nature of rainfall recharge to aquifers is observed in the monitoring data. Groundwater levels are generally elevated following prolonged periods of rain which occur in the wet season between November and April. Between May and October groundwater levels are generally in recession, returning to lower levels. The range of seasonal fluctuation in groundwater levels is highly variable within the study area. Accounting for the larger short-term rainfall induced groundwater level rises, the seasonal variability observed in the monitoring data varies from less than half a metre up to around 5m.

The range of responses observed in the monitoring data demonstrates the fractured rock aquifer's varied connectivity with rainfall recharge with rapid or muted/delayed responses to rainfall. The varied response is likely to be as a result of a multitude of factors including heterogeneity of the aquifer properties including hydraulic conductivity and storage, thickness and type of material of overlying the aquifer, local variation in rainfall intensity, slope and runoff, depth to standpipe measurements and fracture and discontinuity anisotropy within the fractured bedrock aquifer.



Legend

- Construction footprint
- Ancillary sites
- Pavement
- Bridge
- Tunnel
- North Coast Railway
- Watercourse
- Groundwater Monitoring Location (Jul-17 - Feb-19)

Groundwater yield (supply well)

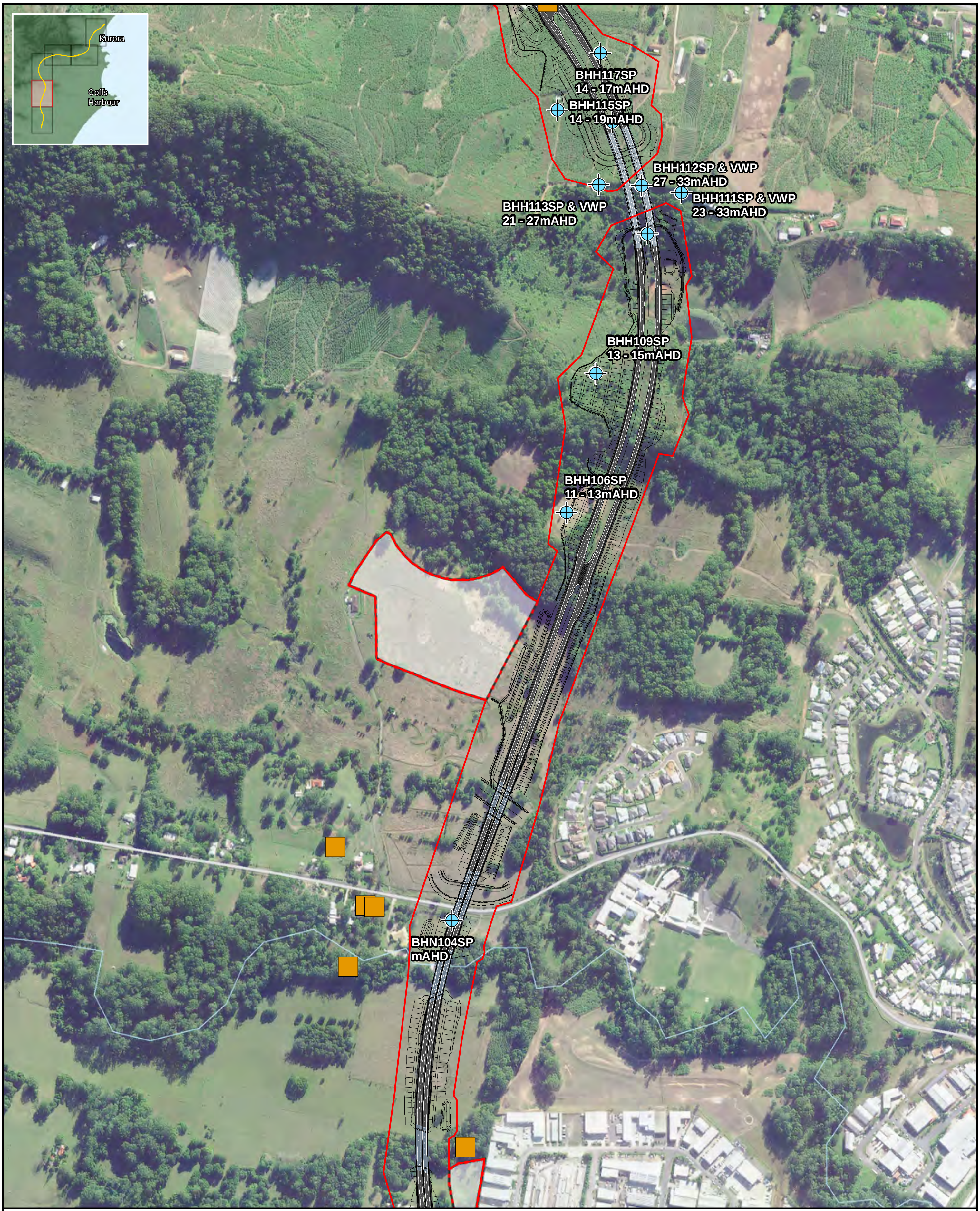
- > 432 kL/day (>5l/s)
- 86 kL/d - 432 kL/d (1 l/s - 5 l/s)
- < 86 kL/day (<1l/s)

0 0.1 0.2 0.3 km

Scale @A4: 1:10,000
GDA 1994 MGA Zone 56

N

Coffs Harbour Bypass
Groundwater Monitoring and Yield Information
Figure 12-1



Legend

- Construction footprint
- Ancillary sites
- Pavement
- Bridge
- Tunnel
- North Coast Railway
- Watercourse
- Groundwater Monitoring Location (Jul-17 - Feb-19)

Groundwater yield (supply well)

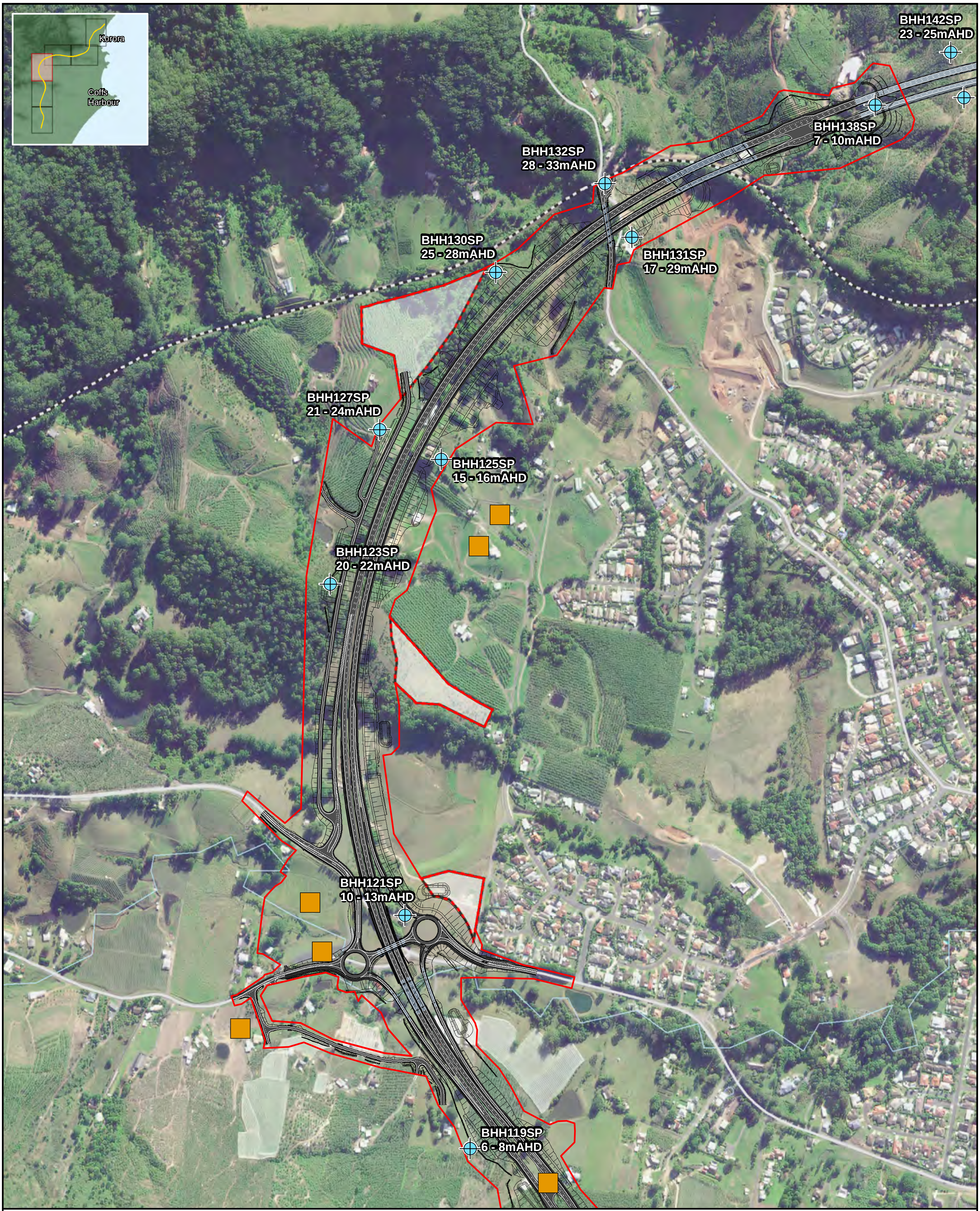
- > 432 kL/day (>5l/s)
- 86 kL/d - 432 kL/d (1 l/s - 5 l/s)
- < 86 kL/day (<1l/s)

0 0.1 0.2 0.3 km

N

Scale @A4: 1:10,000
GDA 1994 MGA Zone 56

Coffs Harbour Bypass
Groundwater Monitoring and Yield Information
Figure 12-2



Legend

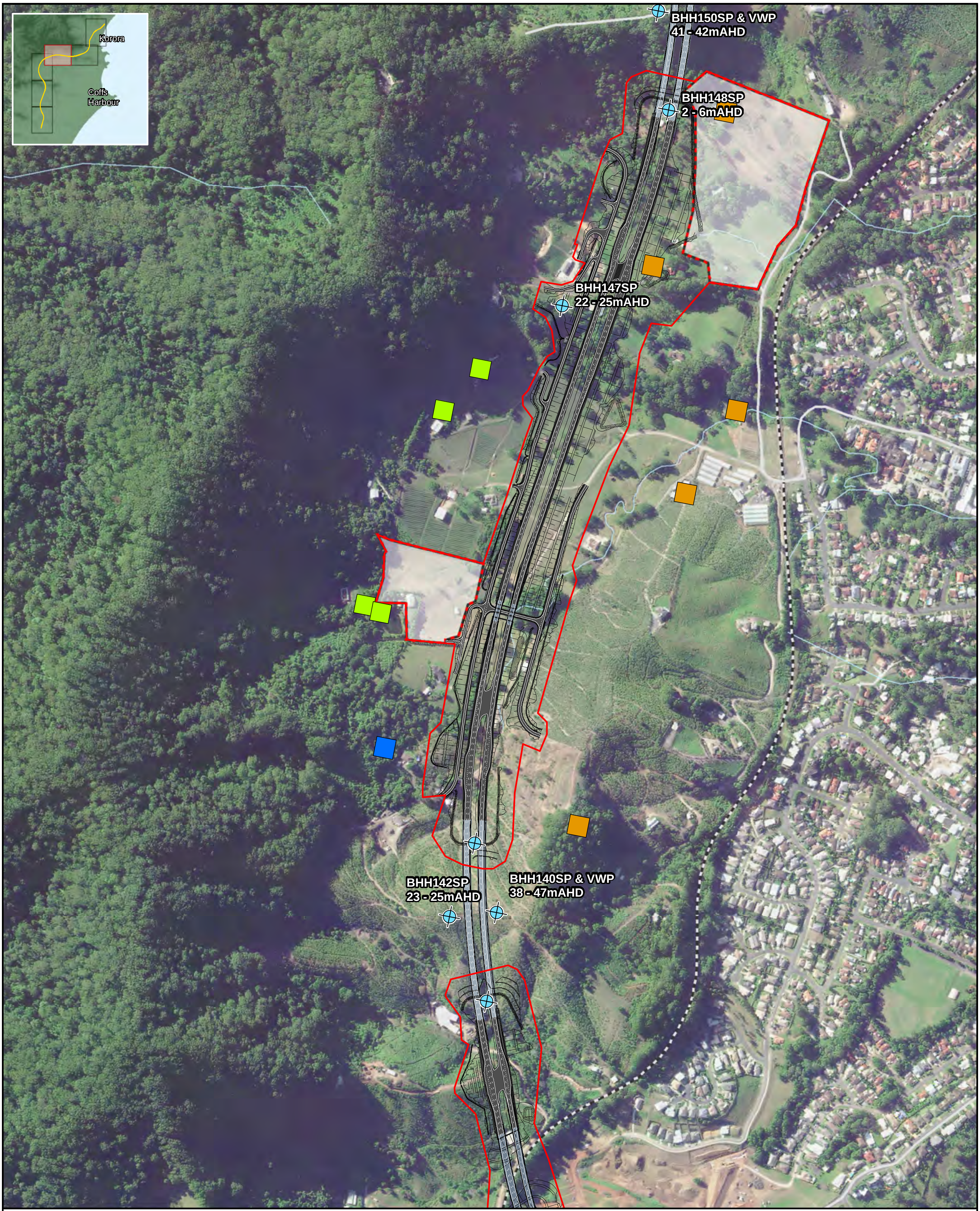
Construction footprint
Ancillary sites
Pavement
Bridge
Tunnel
North Coast Railway
Watercourse
Groundwater Monitoring Location (Jul-17 - Feb-19)

Groundwater yield (supply well)

> 432 kL/day (>5l/s)
86 kL/d - 432 kL/d (1 l/s - 5 l/s)
< 86 kL/day (<1l/s)

Coffs Harbour Bypass
Groundwater Monitoring and Yield Information
Figure 12-3

0 0.1 0.2 0.3 km
Scale @A4: 1:10,000
GDA 1994 MGA Zone 56
N



Legend

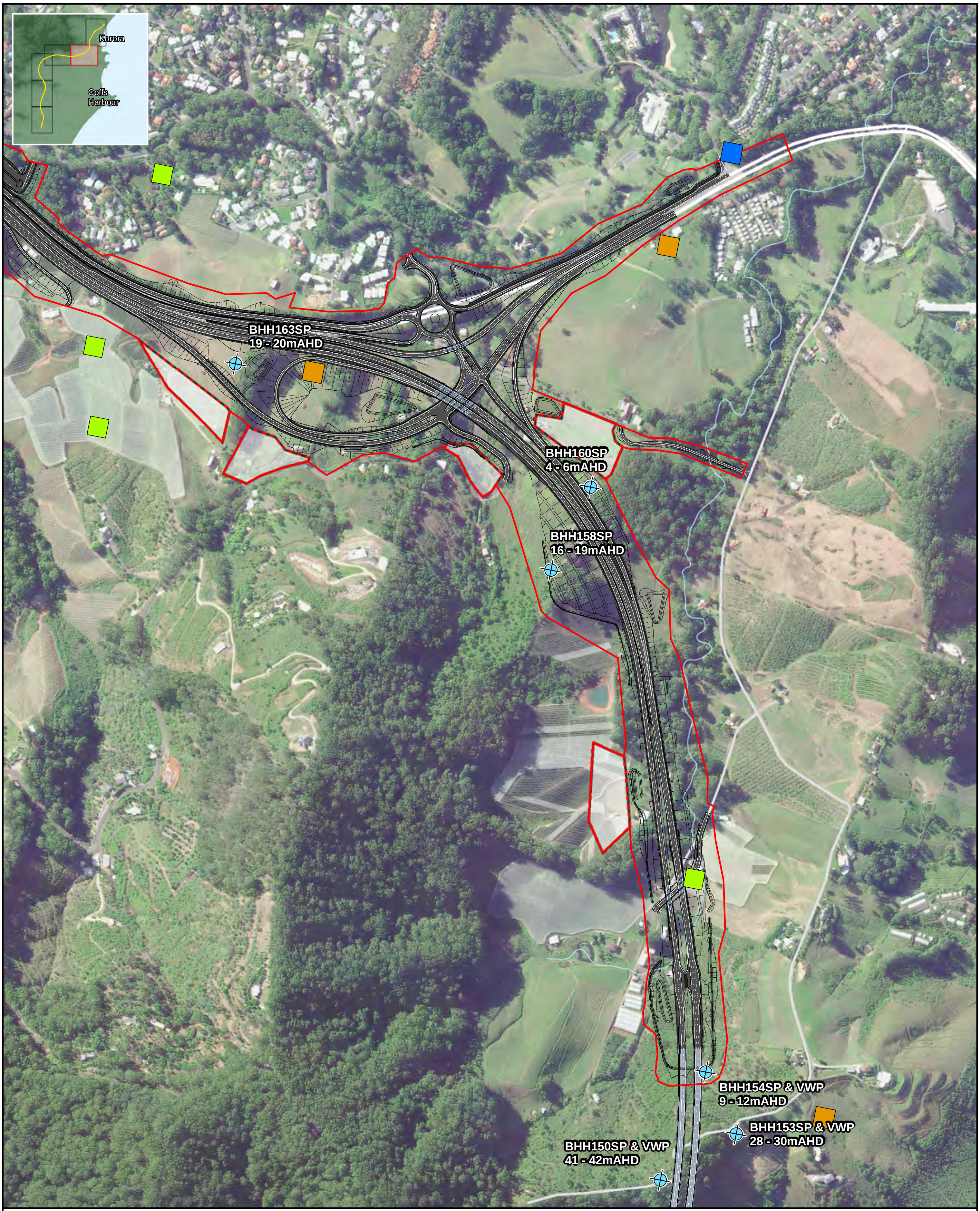
- Construction footprint
- Ancillary sites
- Pavement
- Bridge
- Tunnel
- North Coast Railway
- Watercourse
- Groundwater Monitoring Location (Jul-17 - Feb-19)

Groundwater yield (supply well)

- > 432 kL/day (>5l/s)
- 86 kL/d - 432 kL/d (1 l/s - 5 l/s)
- < 86 kL/day (<1l/s)

Coffs Harbour Bypass
Groundwater Monitoring and Yield Information
Figure 12-4

0 0.1 0.2 0.3 km
Scale @A4: 1:10,000
GDA 1994 MGA Zone 56



Legend

- Construction footprint
- Ancillary sites
- Pavement
- Bridge
- Tunnel
- North Coast Railway
- Watercourse
- Groundwater Monitoring Location (Jul-17 - Feb-19)

Groundwater yield (supply well)

- > 432 kL/day (>5l/s)
- 86 kL/d - 432 kL/d (1 l/s - 5 l/s)
- < 86 kL/day (<1l/s)

0 0.1 0.2 0.3 km

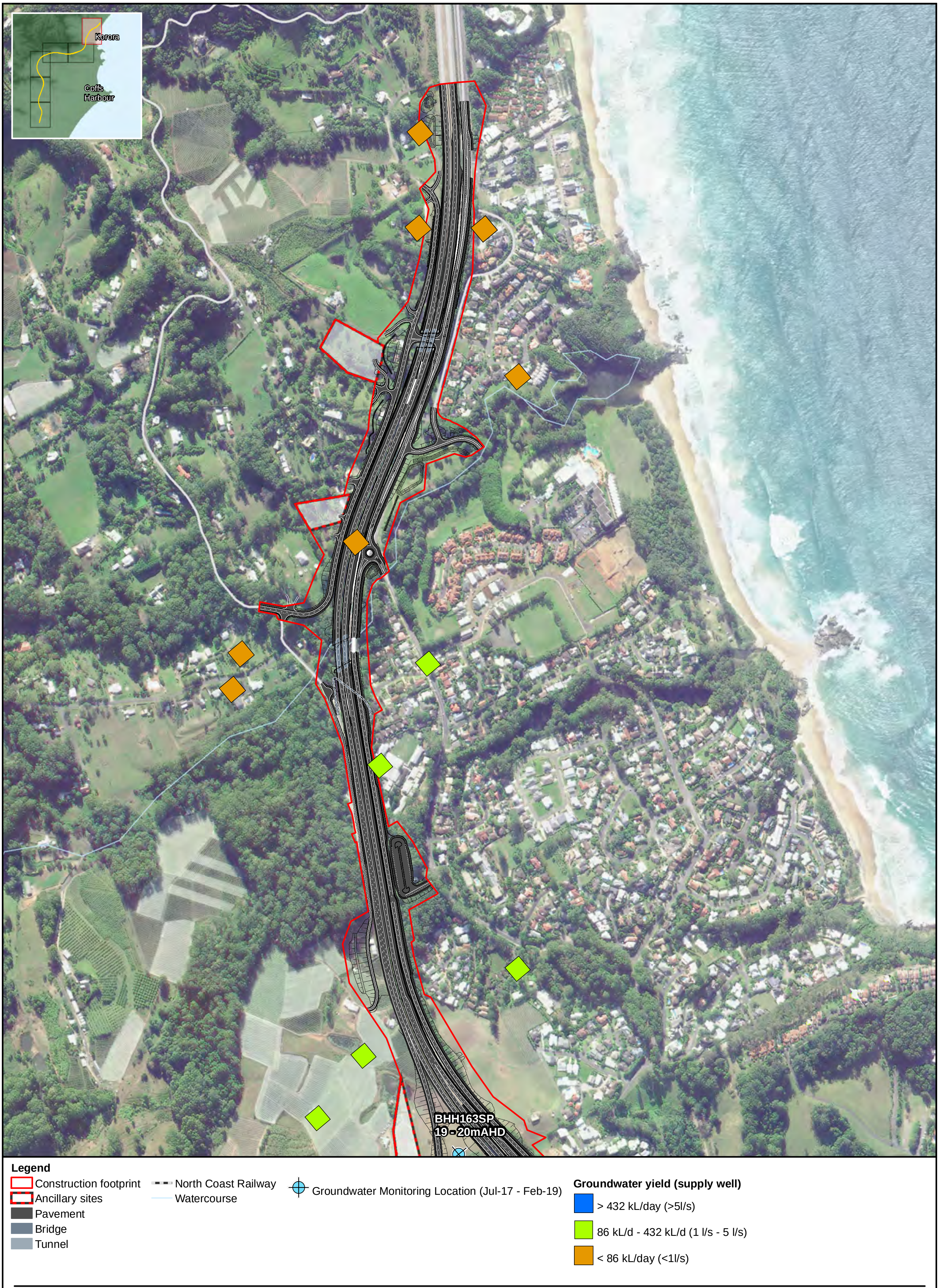
Scale @A4: 1:10,000

GDA 1994 MGA Zone 56

Coffs Harbour Bypass

Groundwater Monitoring and Yield Information

Figure 12-5



Coffs Harbour Bypass
Groundwater Monitoring and Yield Information
Figure 12-6

At locations where multiple vibrating wire piezometers and/or standpipe piezometers are installed, the data indicates the presence of some vertical gradients (i.e. at some locations the water level/pressure recorded by instruments at different levels was not the same).

Monitoring at locations BHH111, BHH1140, BHH142 recorded downward gradients which are likely to be as a result of vertical recharge. At BHH141, BHH148 and BHH154 upward gradients were observed which may be indicative of flow towards discharge zones.

Groundwater within the alluvial deposits was encountered in several test pits along the project alignment. Of those which encountered groundwater, standing groundwater levels varied from 0.9mbgl to 1.9mbgl. This indicates that groundwater is close to ground surface within alluvial deposits. Given the expected connection between creek flow and groundwater in the underlying alluvium, these shallow groundwater levels are in line with the anticipated conditions in these areas.

Groundwater level monitoring for the project is ongoing for the purposes of baselining groundwater variation and climatic response. Where required, additional groundwater investigation would be undertaken during detailed design in order to supplement the existing information and validate assumptions used in the assessment.

2.6.3 Aquifer properties

The flow of groundwater through the fractured bedrock aquifer is through defects within the rock mass (negligible flow occurs within the intact rock mass). Groundwater inflows to cuttings will be concentrated at discrete fracture, joint or fault locations and it is anticipated that the extent of complexity of the geological structures within each cut and tunnel setting may not be fully understood until excavation has proceeded.

The fractured bedrock, which is the principle groundwater bearing strata in the project area has low porosity, governed by the presence of discontinuities (secondary porosity) within the rock mass. Where the bedrock is weathered or affected by structural deformations (shear zones), the porosity is likely to be higher.

Yields from bores in the fractured bedrock aquifer are generally low, commonly less than 1l/s. Occasional supplies of up to l0l/s may be achieved where there is significant fracturing (*DPI, 2016*). Well yield information based on construction information from supply wells in the study area is shown on **Figure 12.** Data from wells in the fractured bedrock aquifer indicates that around two thirds of the wells yield less than 1l/s and 90% yield less than 5l/s. Note that not all supply wells in the study area have yield information available.

Hydraulic conductivity in the fractured bedrock units underlying the bypass has been assessed using the results from 84 borehole packer tests and nine piezometer falling head tests (**Appendix C1**).

Packer test and falling head test results are shown relative to depth below ground level in **Figure 13**. There appears to be little correlation of hydraulic conductivity with depth and the packer test results did not show a large dependency on the rock type. This is understandable because groundwater flow is principally through the defects in the rock mass. It is these defects that control the hydraulic conductivity, rather than the lithology of the rock.

Figure 14 presents a cumulative distribution of the packer test results with a log-normal distribution overlaid which shows that hydraulic conductivity results are concentrated around the mean. The log-normal distribution gives a 90 per cent confidence in the maximum mean hydraulic conductivity of $1x10^{-6}$ m/s. Numerical groundwater modelling undertaken as part of this assessment used a calibrated hydraulic conductivity value of around 10^{-7}m/s for the fractured rock aquifer, close to the mean result.

Figure 13: Fractured bedrock hydraulic conductivity plotted with test depth

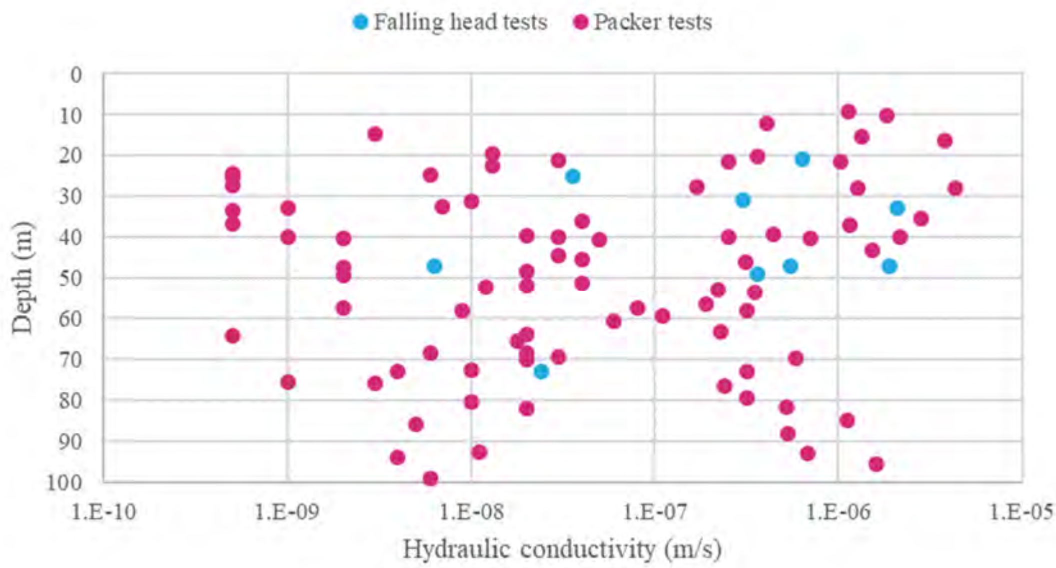
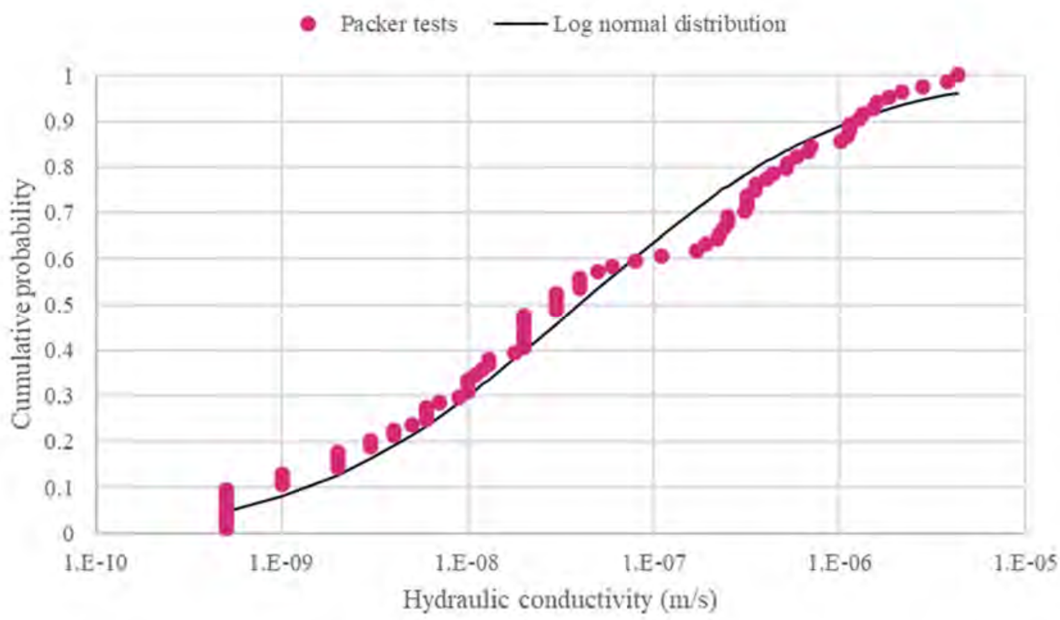


Figure 14: Cumulative distribution of fractured bedrock hydraulic conductivity



Overlying the fractured rock along the bypass are shallow areas of colluvium and residual soil (along hill slopes and foothills). Hydraulic conductivity testing of these soil layers has not been conducted; groundwater within these soils is expected to be perched and may only be present following large rainfall recharge events. Most of the soil is recorded as silty clay, residual soil. Some less frequent zones of colluvium were logged as silty to sandy clays. It is therefore likely that the hydraulic conductivity of the mainly cohesive soils along the bypass are similar to, or lower than the underlying shallow weathered rocks.

The hydraulic conductivity of the up-river alluvial aquifer is expected to be higher than the fractured bedrock, but variable owing to the mixed nature of the materials which form the aquifer. Reported yields from publicly available groundwater data (BoM, 2019) from the alluvial aquifer are generally high and typical hydraulic conductivities of sand and gravel aquifer units can vary from between 10^{-5} m/s to 10^{-3} m/s (*Domenico, and Schwartz, 1990*). Hydraulic conductivities of finer grained materials (silts and clays) within the alluvial deposits are likely to be much lower and the floodplain alluvial aquifer will be lower than that of the up-river alluvial aquifer.

Aquifer storage parameters (storativity and specific yield) also dictate how aquifers respond to recharge and stresses such as groundwater abstraction or seepage into cuttings. Numerical modelling estimated that the specific yield of the fractured rock aquifer was between 1% and 5% (**Appendix C1 and C2**). The specific yield of the alluvial aquifer is expected to be much higher; values of 20 – 35% are normal for sand and gravel aquifer units whereas finer grained silts and clays may range from less than 1% to 20% (*Morris and Johnson, 1967*).

2.6.4 Recharge

Rainfall is the main source of recharge to aquifers in the Coffs Harbour region. Precipitation falls on the ground surface and run-off generated from higher elevations recharges aquifers downstream. Vertical infiltration of rainfall is the principle mechanism of recharge to the aquifers in the region but is likely to be variable across the study area and dependent on factors such as vegetation cover, slope angle, soil and aquifer permeability and local and seasonal variations in rainfall.

Published estimates of groundwater recharge to the fractured bedrock aquifer are limited. The *Water Sharing Plan for the North Coast Fractured and Porous Rock Groundwater Sources* uses an estimate of 4% of annual rainfall to calculate extraction limits, which equates to an average recharge rate of around 65 mm/yr (*DPI, 2016*).

Recharge assessment based on the available hydrograph and rainfall data indicates a large range of potential recharge, from 2% to 19%, assuming specific yield of 1% (**Appendix C2**). Numerical groundwater modelling undertaken as part of the assessment found that recharge rates of up to 15% needed to be applied to provide reasonable match between observed and computed groundwater levels (**Appendix C1**). Wide variation in estimated recharge rates is likely to be indicative of the aquifer heterogeneity which is common in fractured rock.

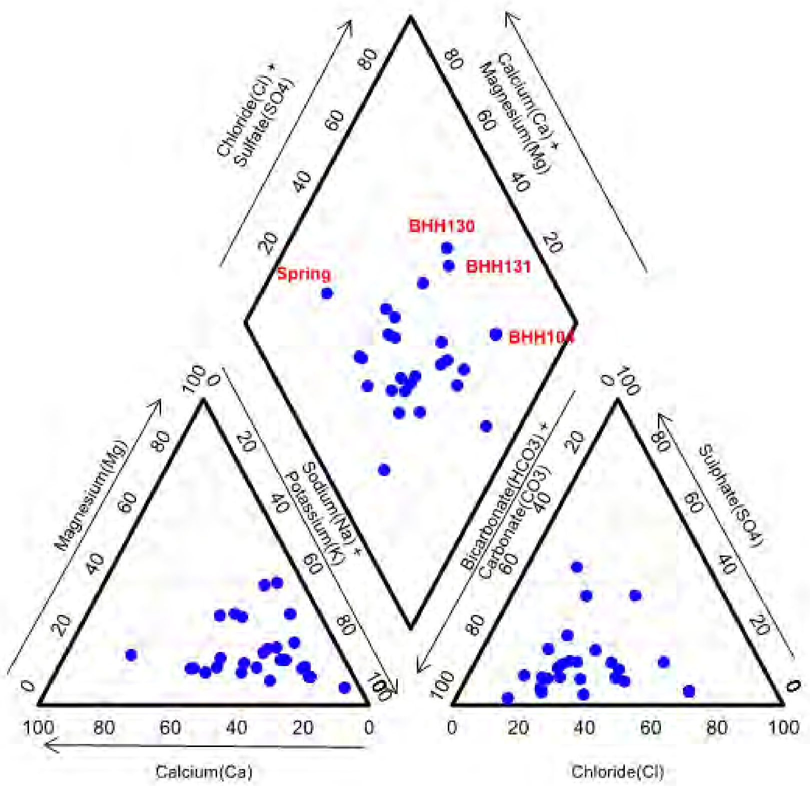
2.6.5 Groundwater quality

Groundwater quality testing has been conducted as part of ongoing groundwater monitoring programme *(RCA Australia, 2017a).* Groundwater quality data at each of the standpipes is presented in Figure 3, based on the electrical conductivity of the water (an indirect measure of groundwater salinity). Groundwater samples were collected from standpipes installed at a variety of depths ranging from 6mbgl to 68mbgl.

The total dissolved solid concentration of the groundwater is low (less than 450mg/l) and is generally of freshwater quality. Two samples registered slightly brackish electrical conductivity (BHH150 and BHH169) which are located at Gatelys Road and near the waste facility at the southern end of the alignment. Other nearby standpipes were of freshwater quality indicating that there is likely to be local variability in groundwater quality across the alignment.

The pH of the groundwater is generally slightly acidic to slightly alkaline. Dissolved oxygen was between 75% and 110% except for one sample which was lower, indicating that the water was generally oxygenated. A piper plot of the groundwater major ion chemistry is presented in **Figure 15**. The results indicate that the chemistry of the groundwater is dominated by the bicarbonate anion with a low to medium distribution of calcium and magnesium groundwater and medium to high distribution of sodium and potassium groundwater.

Figure 15: Groundwater chemistry piper plot (*RCA, 2017*)



Groundwater is considered an essential water resource for many aquatic ecosystems. The Bellinger River Catchment water quality objectives (WQO) and the Australian and New Zealand Guidelines for Fresh and Marine Water Quality *(ANZECC and ARMCANZ, 2000)* using the aquatic ecosystem protection guidelines for moderately disturbed systems have been used to evaluate the groundwater quality (**Table 7**).

Turbidity of the groundwater samples was generally low except for BHH150 which had a turbidity of 428NTU. A total of five samples had turbidity values more than the upper default trigger for lowland rivers of 50NTU.

Groundwater samples showed concentrations of trace metals above the ANZECC aquatic ecosystem guideline values. All six samples tested for zinc had concentrations slightly elevated above the guideline (a maximum of 0.05mg/l with a trigger value of 0.008mg/l). 24 out of 26 samples tested had concentrations of aluminium above the ANZECC guideline value of 0.055mg/l. There appears to be no spatial trend in the location of exceedences along the alignment. As concentrations of these analytes in the fractured bedrock do not generally correspond with any particular source, it is considered likely that the elevated concentrations of some metals are likely to be naturally occurring and indicative of regional water quality. The full set of groundwater quality testing results is provided in **Appendix B**.

Table 7: Groundwater quality summary test results

Parameter	WQO/ANZECC aquatic ecosystem guideline values	Test results		
		Range	Average	Number of results (exceedences)
pH	6.5 – 8.5 lowland rivers	5.74 – 7.87	6.8	29 (11)
Total dissolved solids (mg/L)	-	86 – 437	238.7	6
Conductivity (μS/cm)	125 – 2200 lowland rivers	168 – 1174	465.7	29
Turbidity (NTU)	6 – 50 lowland rivers	< 1 – 428	37.0	29 (5)
Hydroxide Alkalinity as $CaCO_3$ (mg $CaCO_3$/L)	-	-	< 1	29
Carbonate Alkalinity as $CaCO_3$ (mg $CaCO_3$/L)	-	< 1 – 11	< 1	29
Bicarbonate Alkalinity as $CaCO_3$ (mg $CaCO_3$/L)	-	8 – 571	122.0	29
Total Alkalinity as $CaCO_3$ (mg $CaCO_3$/L)	-	8 – 571	122.3	29
Sulfate as SO4 (mg/L)	-	8 – 236	43.9	29
Sulfite as SO2 (mg/L)	-	-	< 2	6
Dissolved Oxygen (mg/L)	85 – 110% lowland rivers	2.5 – 9.9mg/l (30 – 110%)*	7.8	22 (7)
Salinity (%)	-	0 – 0.33	0.03	22
Chloride (mg/L)	-	12 – 130	37.3	26
Total Calcium (mg/L)	-	1.2 – 89	26.1	29
Total Magnesium (mg/L)	-	2.3 – 88	11.7	26
Total Sodium (mg/L)	-	9.8 – 150	37.1	26
Total Potassium (mg/L)	-	0.7 – 36	4.7	26
Total Zinc (mg/L)	0.008	0.01 – 0.05	0.03	6 (6)
Total Manganese (mg/L)	1.9	0.006 – 3.6	0.7	6 (1)
Total Aluminium (mg/L)	0.055 if pH > 6.5	0.02 – 2.1	0.4	26 (24)
Total Iron (mg/L)	-	0.02 – 2.8	0.8	26
*Calculated based on elevation of 0mAHD and 20ºC				

A search of groundwater quality data from the NSW groundwater borehole database yielded no hydrochemical groundwater data in the Coffs Harbour region (*NGIS, 2019*). Groundwater salinity data (total dissolved solids) from a total of 28 borehole records in the study area indicated that groundwater is fresh, with a salinity ranging from 60mg/l to 800mg/l (with an average of less than 200mg/l).

Groundwater in alluvial aquifers has a short residence time and strong connection with surface waters as described in the *Water Sharing Plan for the Coffs Harbour Area Unregulated and Alluvial Water Sources, 2009* (*DoW, 2009*). Connection between the up-river alluvium and creek flow is such that the water sharing plan considers water in the alluvium and creek to be the same source. The quality of the alluvial groundwater is therefore expected to be similar to that of the surface water within the creeks.

Water quality sampling of creek water indicated that water quality was fresh and consistent with the guidelines for aquatic ecosystem protection, with the exception of turbidity (which was low) and dissolved oxygen (DO) which was also low for a number of the sites. Most metals were below the relevant trigger levels for all sites except for zinc (total) at one location which was just over the ANZECC guideline.

Surface water had lower concentrations of heavy metals that were observed in the fractured bedrock groundwater however creek water was found to have high pre-existing nitrogen and nutrient concentrations indicative of disturbed water courses within heavy agricultural land usage. Differences in quality between the fractured bedrock groundwater and creek water indicates that the contribution of water to the creek systems is limited, as the creek water would likely have similarly elevated concentrations of heavy metals if it was a major contributor of water.

2.6.6 Groundwater dependent ecosystems

Groundwater dependent ecosystems (GDEs) require access to groundwater on a permanent or intermittent basis to maintain their communities of flora and fauna, ecological and ecosystem processes. There are three types of GDEs based on the type of groundwater reliance. These are:

- Aquatic GDEs dependent on surface expression of groundwater and includes surface water systems which may have a groundwater component (i.e. groundwater fed wetlands or river baseflow ecosystems),
- Terrestrial GDEs dependent on subsurface expression of groundwater (i.e. terrestrial and riparian vegetation), and
- Subterranean GDEs dependent on subterranean presence of groundwater (i.e. karst and cave ecosystems)

Assessment of the potential for the study area to support groundwater dependent ecosystems (GDEs) was assessed using the Australian Government's Bureau of Meteorology (BoM) Groundwater Dependent Ecosystems Atlas and Statewide GDE mapping (*DPI 2016b*). GDE Atlas mapping is from two broad sources:

- National scale assessment based on a set of rules that describe potential for groundwater/ecosystem interaction from available GIS data, and
- Regional scale assessment which includes studies undertaken by States and/or regional agencies using approaches including fieldwork, satellite imagery analysis and application of conceptual models

The identification of potential GDEs in the Atlas does not necessarily confirm that a particular ecosystem is groundwater dependent.

Figure 16 shows the location of potential GDEs and mapped native vegetation communities within the study area. Nine PCTs, one a groundwater dependent wetland community and eight groundwater dependent vegetation communities, all identified as 'High Probability GDEs' (from national assessments), and reliant on subsurface expression of groundwater.

The potential GDEs identified from the GDE Atlas are mostly terrestrial which could rely upon the subsurface expression of groundwater to support the ecological community. These terrestrial GDEs support a variety of vegetation ecosystems and protected areas including for Indigenous use.

The GDE Atlas illustrates that PCT 1064 Paperbark swamp forest vegetation present in the vicinity of the Newports Creek floodplain, south of Englands Road, to be the only area of High Potential GDE (from regional studies). **Table 8** summarises the potential GDE vegetation communities present within the study area.

Table 8: GDEs recorded within the study area

GDE Name	PCT Details	Landscape position
Groundwater Dependent Wetland Communities – High Probability GDE		
Paperbark	PCT 1064 Paperbark swamp forest of the coastal lowlands of the NSW North Coast Bioregion and Sydney Basin Bioregion (NR217)	PCT 1064 occurs in the southern and central parts of the study area east of Englands Road and west of Highlander Drive along and adjacent to tributaries of Newports Creek in the North Boambee Valley. PCT 1064 occurs on low lying, typically waterlogged ground within the study area and across the Coffs Harbour LGA this vegetation community is associated with low-lying inundated areas on alluvial floodplains and back-swamps. Areas of PCT 1064 present within the study area are considered to be ground water dependent vegetation, reliant on subsurface expression of groundwater.
Groundwater Dependent Vegetation Communities – High Probability GDE		
Sub-Tropical Rainforest	PCT 670 Black Booyong - Rosewood - Yellow Carabeen subtropical rainforest of the NSW North Coast Bioregion (NR111)	PCT 670 occurs in well sheltered gullies and slopes at low altitudes, with only one occurrence of the PCT present within the study area north of Mackays Road.
Wet Sclerophyll Shrub Forests	PCT 692 Blackbutt - Tallowwood moist ferny open forest of the coastal ranges of the NSW North Coast Bioregion (NR120)	PCT 692 is broadly located on foothills and ranges from the Manning Valley north to the Corindi River and within the study area commonly occurs towards the northern and southern end of the project.
Wet Sclerophyll Shrub Forests	PCT 695 Blackbutt - Turpentine - Tallowwood shrubby open forest of the coastal foothills of the central NSW North Coast Bioregion (NR122)	The PCT is known to occur on the ranges of the great escarpment from Dingo Tops north to Chandlers Creek. Within the study areas its occurrences include multiple locations throughout the centre and north of the alignment

GDE Name	PCT Details	Landscape position
		with the largest location adjacent to Jordans Creek.
Wet Sclerophyll Shrub Forests	PCT 747 Brush Box - Tallowwood - Sydney Blue Gum tall moist forest of the ranges of the central NSW North Coast Bioregion (NR138)	Distributed in near coastal valleys and foothills from the Nambucca Valley north to the Corindi River, the PCTs occurrence within the study area is generally associated with creeks and drainage line through the centre of the alignment.
Central Mid Elevation Sydney Blue Gum	PCT 1244 Sydney Blue Gum open forest on coastal foothills and escarpment of the North Coast (NR258)	The PCT is generally known to exist as a tall wet forest with an over storey dominated by Sydney Blue Gum (*Eucalyptus saligna*). Two occurrences of the PCT occur within the study area to the north of the Kororo Nature Reserve and to the south of North Boambee Road.
Dry Grassy Tallowwood-Grey Gum	PCT 1262 Tallowwood - Small-fruited Grey Gum dry grassy open forest of the foothills of the NSW North Coast (NR263)	Distributed throughout the coastal lowlands and foothills of the midnorth coast from the Manning Valley north to the Corindi River, this PCT exists as two patches in one location in Korara within the study area.
Open Coastal Brushbox	PCT 1285 Turpentine moist open forest of the coastal hills and ranges of the NSW North Coast Bioregion (NR274)	Generally located on coastal lowlands and foothills from the Manning Valley north to the Corindi River, PCT 1285 occurs in two locations at the northern end of the study area adjacent to Kororo Nature Reserve, and adjacent to the existing Pacific Highway alignment near Charlesworth Bay.
Lowland Rainforest on Floodplain	PCT 1302 White Booyong - Fig subtropical rainforest of the NSW North Coast Bioregion (NR280)	Located on the floodplains in the North Coast region, three occurrences of this PCT were recorded within the study area. these include adjacent to the Coffs Creek tributary north of Coramba Road, immediately west of Treefern Creek, and near an unnamed watercourse near Bruxner Park Road.

The water sharing plans which cover the Coffs Harbour area indicate that there are no high priority groundwater dependent ecosystems or high priority karst environment GDEs identified at the commencement of the plans (*DoW, 2009, DPI, 2016*).

2.6.7 Springs

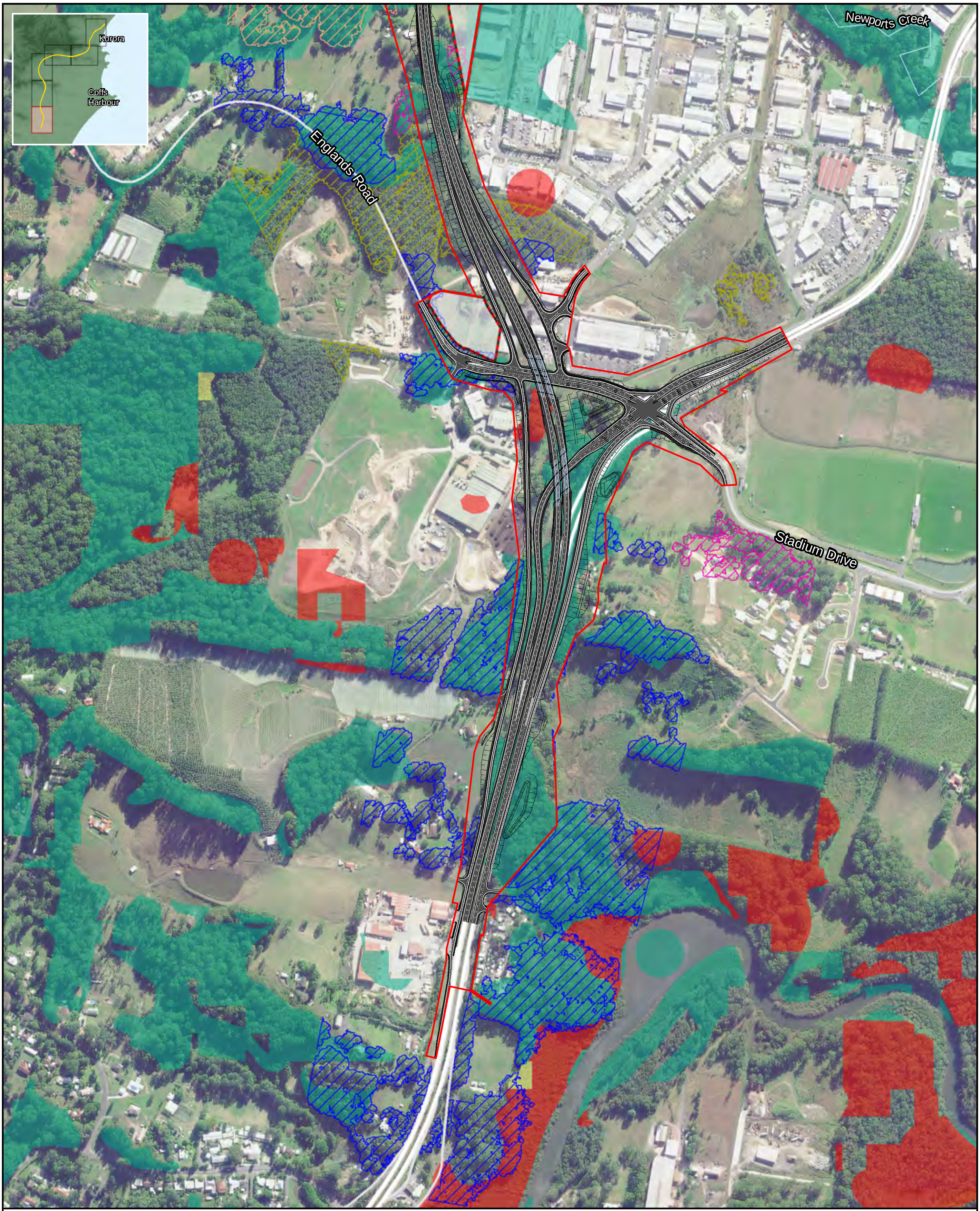
Springs represent the surface expression of the groundwater table, where groundwater discharges or ponds. Springs form in a variety of settings and may represent point (from discrete fractures) or diffuse discharges. They can occur at changes in stratigraphy, breaks in slope, or simply as a result of groundwater levels reaching the surface following prolonged periods of recharge.

NSW Hydrographic mapping (*NSW, 2016*) indicates no mapped springs in the study area. A spring was noted during drilling for BHH138 at Shephards Lane (RCA, 2018), other anecdotal evidence and conceptual understanding of the hydrogeology indicates that springs are likely occur in the region and may be a source of water for creek flows, vegetation and agricultural dams for local landowners.

Springs are most likely to occur during and following the wet season when groundwater levels are highest. They may also occur in areas of steep topographic variation such as the three main ridge lines. The nature of spring emergence will be affected by topographic variation, underlying geological profile and recharge dynamics. The presence of springs is likely to be both spatially and seasonably.

At Roberts Hill, the owner of an agricultural dam indicated that it is partly groundwater fed (the agricultural dam is shown on **Figure 17**). Several other ponds/lakes located around the 30 m AHD topographic contour suggest that this could be a location of spring discharge from the fractured bedrock, where the groundwater table in the fractured bedrock intersects the ground surface.

At Gatelys Road tunnel, several ponds and agricultural dams are located between 45 m AHD and 70 m AHD which may also be naturally fed by groundwater springs or in connection with the underlying groundwater.



Legend

Construction footprint
Ancillary sites
Pavement
Bridge
Tunnel
North Coast Railway
Watercourse

Vegetation Communities
686
692
695
826
1064
1244

Terrestrial GDEs (from regional studies)
High potential terrestrial GDE
Moderate potential terrestrial GDE
Low potential terrestrial GDE

Coffs Harbour Bypass
Groundwater Dependent Ecosystems and Native Vegetation
Figure 16-1

0 0.1 0.2 0.3 km
Scale @A4: 1:10,000
GDA 1994 MGA Zone 56
N

Legend

- Construction footprint
- Ancillary sites
- Pavement
- Bridge
- Tunnel
- North Coast Railway
- Watercourse

Vegetation Communities

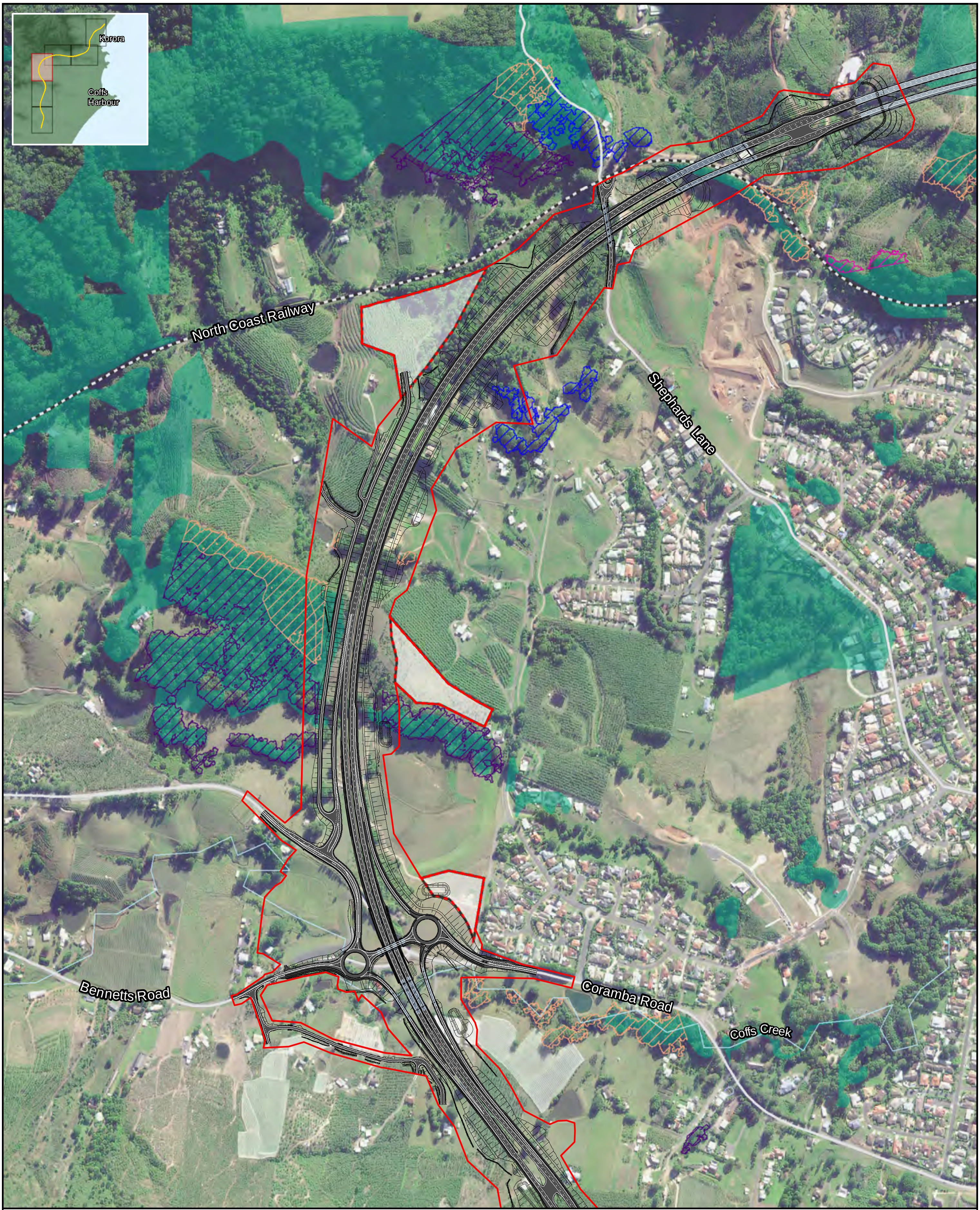
- 686
- 692
- 695
- 826
- 1064
- 1244

Terrestrial GDEs (from regional studies)

- High potential terrestrial GDE
- Moderate potential terrestrial GDE
- Low potential terrestrial GDE

Coffs Harbour Bypass
Groundwater Dependent Ecosystems and Native Vegetation
Figure 16-2

0 0.1 0.2 0.3 km
Scale @A4: 1:10,000
GDA 1994 MGA Zone 56
N



Legend

Construction footprint
Ancillary sites
Pavement
Bridge
Tunnel
North Coast Railway
Watercourse

Vegetation Communities

686
692
695
826
1064
1244

Terrestrial GDEs (from regional studies)

High potential terrestrial GDE
Moderate potential terrestrial GDE
Low potential terrestrial GDE

0 0.1 0.2 0.3 km

Scale @A4: 1:10,000
GDA 1994 MGA Zone 56

N

Coffs Harbour Bypass
Groundwater Dependent Ecosystems and Native Vegetation
Figure 16-3

Legend

Construction footprint
Ancillary sites
Pavement
Bridge
Tunnel
North Coast Railway
Watercourse

Vegetation Communities
686
692
695
826
1064
1244

Terrestrial GDEs (from regional studies)
High potential terrestrial GDE
Moderate potential terrestrial GDE
Low potential terrestrial GDE

Coffs Harbour Bypass
Groundwater Dependent Ecosystems and Native Vegetation
Figure 16-4

0 0.1 0.2 0.3 km
Scale @A4: 1:10,000
GDA 1994 MGA Zone 56

Legend

Construction footprint
Ancillary sites
Pavement
Bridge
Tunnel
North Coast Railway
Watercourse

Vegetation Communities
686
692
695
826
1064
1244

Terrestrial GDEs (from regional studies)
High potential terrestrial GDE
Moderate potential terrestrial GDE
Low potential terrestrial GDE

Coffs Harbour Bypass
Groundwater Dependent Ecosystems and Native Vegetation
Figure 16-5

0 0.1 0.2 0.3 km
Scale @A4: 1:10,000
GDA 1994 MGA Zone 56

Legend

- Construction footprint
- Ancillary sites
- Pavement
- Bridge
- Tunnel
- North Coast Railway
- Watercourse

Vegetation Communities

- 686
- 692
- 695
- 826
- 1064
- 1244

Terrestrial GDEs (from regional studies)

- High potential terrestrial GDE
- Moderate potential terrestrial GDE
- Low potential terrestrial GDE

0 0.1 0.2 0.3 km
Scale @A4: 1:10,000
GDA 1994 MGA Zone 56
N

Coffs Harbour Bypass
Groundwater Dependent Ecosystems and Native Vegetation
Figure 16-6

2.6.8 Water sharing plans

The project is located in the area covered by the *Water Sharing Plan for the Coffs Harbour Area Unregulated and Alluvial Water Sources, 2009,* which encompasses surface water and alluvial groundwater systems *(DWE, 2009),* and *the Water Sharing Plan for the North Coast Fractured and Porous Rock Groundwater Sources 2016 (DPI 2016)* which covers the fractured bedrock aquifer.

The unregulated and alluvial rivers water sharing plan area is divided into 13 Extraction Management Units (EMU), which correspond to the coastal catchment areas created by creeks and estuaries in the area that discharge into the ocean. The project intersects the Boambee Creek, Coffs Creek and Korora Basin EMUs, which are highly relied upon for the economic benefits they provide to the horticulture industry for irrigation and environmental benefits to estuarine areas that rely on natural flows for their water supply. Due to the high connectivity between the alluvial aquifers and creek discharge, the water sharing plan recognises that in up-river alluvial reaches, the surface and groundwater is a single water source.

A summary of each EMU's catchment characteristics and licensed water access licenses (WAL) for the Unregulated and Alluvial Water Sources is presented in **Table 9**.

Table 9: Excerpt summary of EMU catchments and licensed entitlements

EMU	Catchment area	Total licensed entitlement	Total alluvial groundwater entitlement
Boambee Creek water source	51.2 km^2 (37% forested)	637 ML/year 43 WALs (91% for irrigation, 6% for farming)	30 ML/year 2 WALs (100% for irrigation)
Coffs Creek water source	26.6 km^2 (23% forested)	443 ML/year 28 WALs (97% for irrigation, 1% for farming)	19 ML/year 2 WALs (74% for irrigation, 36% for farming)
Korora Basin water source	14.8 km^2 (30% forested)	444 ML/year 39 WALs (84% for irrigation, 6% for farming)	N/A

The fractured bedrock aquifer is covered by *the Water Sharing Plan for the North Coast Fractured and Porous Rock Groundwater Sources 2016 (DPI 2016).* The fractured bedrock in the Coffs Harbour Region is part of the New England Fold Belt Coast groundwater source. This groundwater source is part of a large regional unit on the mid-north coast of NSW. The groundwater source extends from Port Stephens in the south to the NSW-QLD border in the north, and east of Moree. North of Coffs Harbour the groundwater source is overlain by volcanic units which represent a separate groundwater source.

The Long-Term Average Annual Extraction Limit (LTAAEL) for the New England Fold Belt Coast Groundwater Source is calculated as the current entitlement plus the estimated future water requirements for the term of the water sharing plan. This is set as 60,000 ML/yr which is equal to 16% of the Upper Extraction Limit (UEL) of the groundwater source of 375,000 ML/yr. The existing entitlement at the start of the water sharing plan in 2016 was 35,468 ML/yr.

2.6.9 Groundwater users

The *Water Sharing Plan for the Coffs Harbour Area Unregulated and Alluvial Water Sources, 2009* indicates groundwater is primarily used for irrigation and farming purposes. The Korora Basin was identified as having high instream value, so water trading will be limited for this water source with no future increase in water entitlement.

The *Water Sharing Plan for the North Coast Fractured and Porous Rock Groundwater Sources 2016 (DPI 2016)* indicates that water requirements from the New England Fold Belt Coast groundwater source at the commencement of the plan was 35,468 ML/yr. The LTAAEL for the groundwater source is set at 60,000 ML/yr whereas the UEL is 365,000 ML/yr. This indicates that there is a large water availability from the groundwater source and water licences are likely to be available.

The Department of Primary Industry Water database indicated that there are numerous licenced groundwater wells within close proximity to the alignment. Details of these groundwater bores including user type are summarised in **Table 10** and presented on **Figure 17** (note that this figure also shows the location of wells recorded as monitoring wells in the NGIS database).

The search results indicate that most groundwater wells in the study area draw water from the fractured bedrock aquifer. A total of four water access licences from alluvial water sources were active at the start of the water sharing plan in 2009 in the Boambee Creek and Coffs Creek EMUs.

Table 10: DPI Water Groundwater Wells (from NGIS)

Groundwater Well ID	Depth to Groundwater (m)	Approximate Chainage (m)	Aquifer Type	Bore Type
GW302024	Not Recorded	10380	Silty Clay	Monitoring Bore
GW050939	Not Recorded	11680	Rock	Stock/Domestic
GW305778	Not Recorded	12000	Not Recorded	Stock/Domestic
GW052382	Not Recorded	12020	Clay and Rock	Stock/Domestic
GW304993	10	12140	Rock	Domestic
GW049234	Not Recorded	12140	Rock	Domestic
GW055122	Not Recorded	12240	Clay and Rock	Stock/Domestic
GW300335	5	14140	Rock	Commercial
GW303892	3	14520	Rock	Domestic/Farming/Stock
GW302315	2	14740	Rock	Irrigation/Stock/Farming/ Domestic

Groundwater Well ID	Depth to Groundwater (m)	Approximate Chainage (m)	Aquifer Type	Bore Type
GW053093	Not Recorded	14860	Clay/ Gravel (alluvium)	Stock/Irrigation/Domestic
GW303298	6.5	15620	Bedrock	Stock/Domestic
GW304429	6	15680	Bedrock	Domestic
GW303812	24	17400	Bedrock	Domestic
GW304578	34	17529	Bedrock	Domestic
GW303467	12	17760	Bedrock	Domestic
GW303672	9	17760	Bedrock	Domestic
GW300311	6	18160	Bedrock	Domestic
GW303424	12	18200	Bedrock	Domestic
GW304148	18	18280	Bedrock	Domestic
GW303960	24	18340	Bedrock	Domestic
GW301578	9	18600	Bedrock	Stock/Domestic
GW072693	27	18900	Bedrock	Domestic
GW068986	9	19260	Bedrock	Domestic
GW063664	Not Recorded	19630	Bedrock	Domestic
GW302679	3	19720	Bedrock	Domestic
GW068806	9.3	21160	Bedrock	Stock/Domestic
GW066175	18	21540	Bedrock	Stock
GW063728	Not Recorded	21560	Bedrock	Domestic/Irrigation
GW304356	Not Recorded	21600	Bedrock	Domestic
GW065993	9	22160	Bedrock	Recreation
GW071387	9	22360	Bedrock	Domestic
GW033998	0.9	22410	Alluvium	Domestic
GW304542	26	22360	Bedrock	Domestic
GW303098	5	22420	Bedrock	Domestic
GW305196	15.2	22640	Not Recorded	Domestic
GW059711	Not Recorded	22650	Bedrock	Domestic
GW305344	18	22960	Bedrock	Domestic
GW073178	6	23070	Bedrock	Domestic
GW064687	20	23350	Bedrock	Stock/Domestic
GW064686	Not Recorded	23360	Bedrock	Stock/Domestic
GW064174	Not Recorded	23350	Bedrock	Domestic
GW064368	Not Recorded	23560	Bedrock	Domestic
GW033417	Not Recorded	23630	Bedrock	Domestic/Recreation
GW065857	14.6	1140[1]	Bedrock	Monitoring Bore

Groundwater Well ID	Depth to Groundwater (m)	Approximate Chainage (m)	Aquifer Type	Bore Type
GW063912	Not Recorded	1369(1)	Bedrock	Domestic

Note: 1. Southbound off-ramp at Korora

2.6.10 Water access licenses

Within the sub-catchment areas which the project is in, a total of 110 water access licenses (WALs) have been granted, with four WALs for alluvial groundwater entitlements for *Coffs Harbour Area Unregulated and Alluvial Water Sources (DWE, 2009).* The WAL entitlements over the three sub-catchments cumulate to 1524 ML/year in total and 49 ML/year in alluvial groundwater.

Within the New England Fold Belt fractured bedrock groundwater source, the total water entitlements for the source totalled 35,468 ML/yr (DPI, 2016).

2.6.11 Agricultural dams

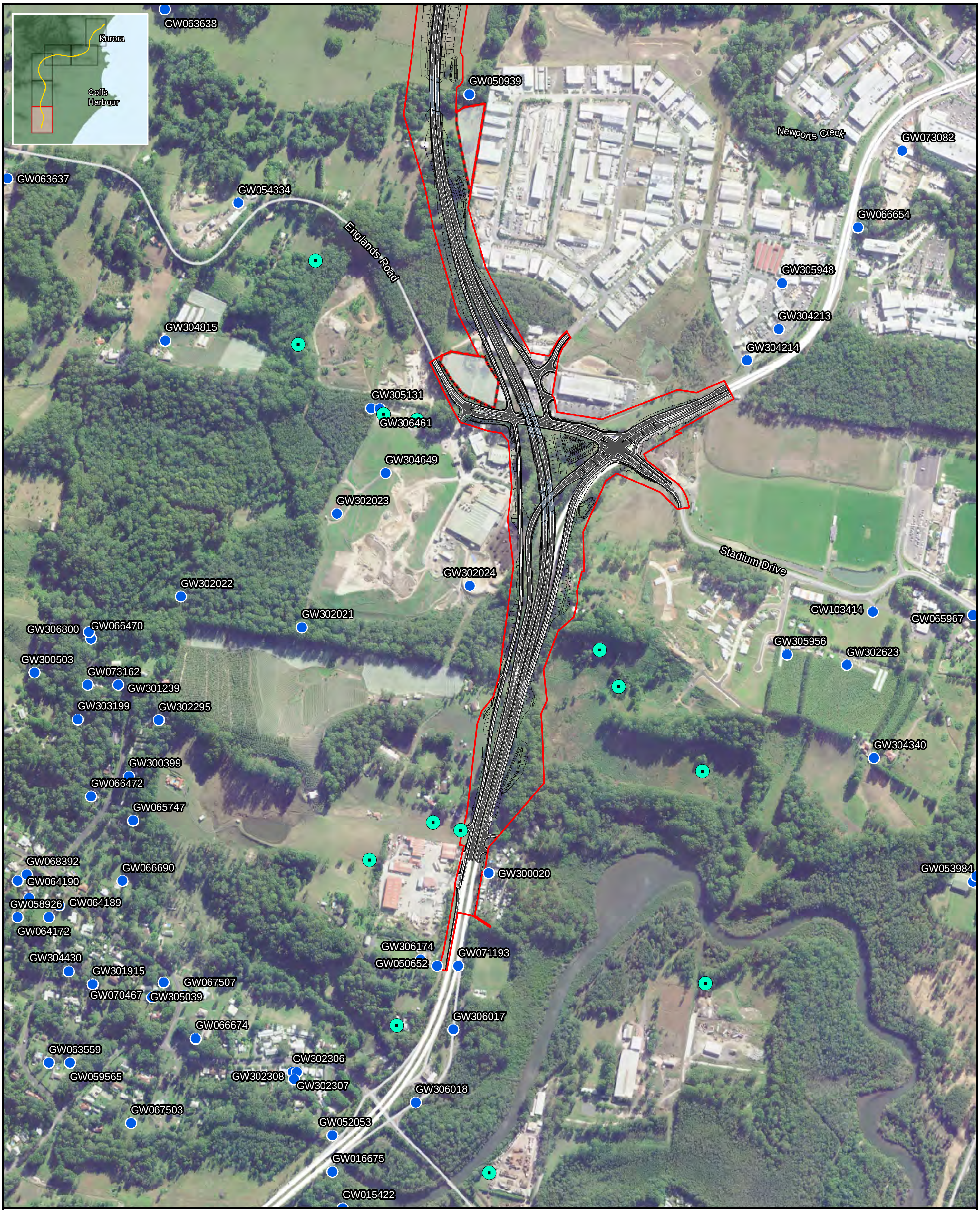
NSW Hydrographic mapping (*NSW, 2016*) indicates that there are a total of 71 agricultural dams within 1km of the alignment. The location of these agricultural dams and other surface water bodies in the study area are presented in **Figure 17**.

The source of water for these agricultural dams is not known however it is typical that they are maintained by a combination of surface run off, top up from nearby creeks or groundwater fed (through processes of spring discharge and or direct connection with the underlying water table). Groundwater fed dams may be from perched water within surficial deposits or from discharge from the underlying fractured bedrock aquifer.

Several agricultural dams and ponds are located close to the project construction footprint. A number located downgradient of cuttings or drained tunnels could be affected by changes to groundwater flow or level. It is likely that some may be directly or indirectly groundwater fed.

As discussed in **Section 2.6.7 Springs**, a number of ponds/agricultural dams located around the ridgelines at Shephards Lane, Gatelys Road and Roberts Hill may be groundwater fed. A number of other dam locations near to the project, particularly around cuts 11 and 12 (chainage 15600 to 16200), 16 (chainage 18000 to 18600) and drained tunnels at Roberts Hill, Shephards Lane and Gatelys Road ridgelines could also potentially be linked to groundwater. At this stage however, the exact hydrological dynamics of each dam/pond is not known and further ground truthing and field investigations are required to clarify this.

The location of known agricultural dams and other surface water features is used in the impact assessment as a preliminary means to evaluate where potential impacts could result due to changes in the groundwater environment.



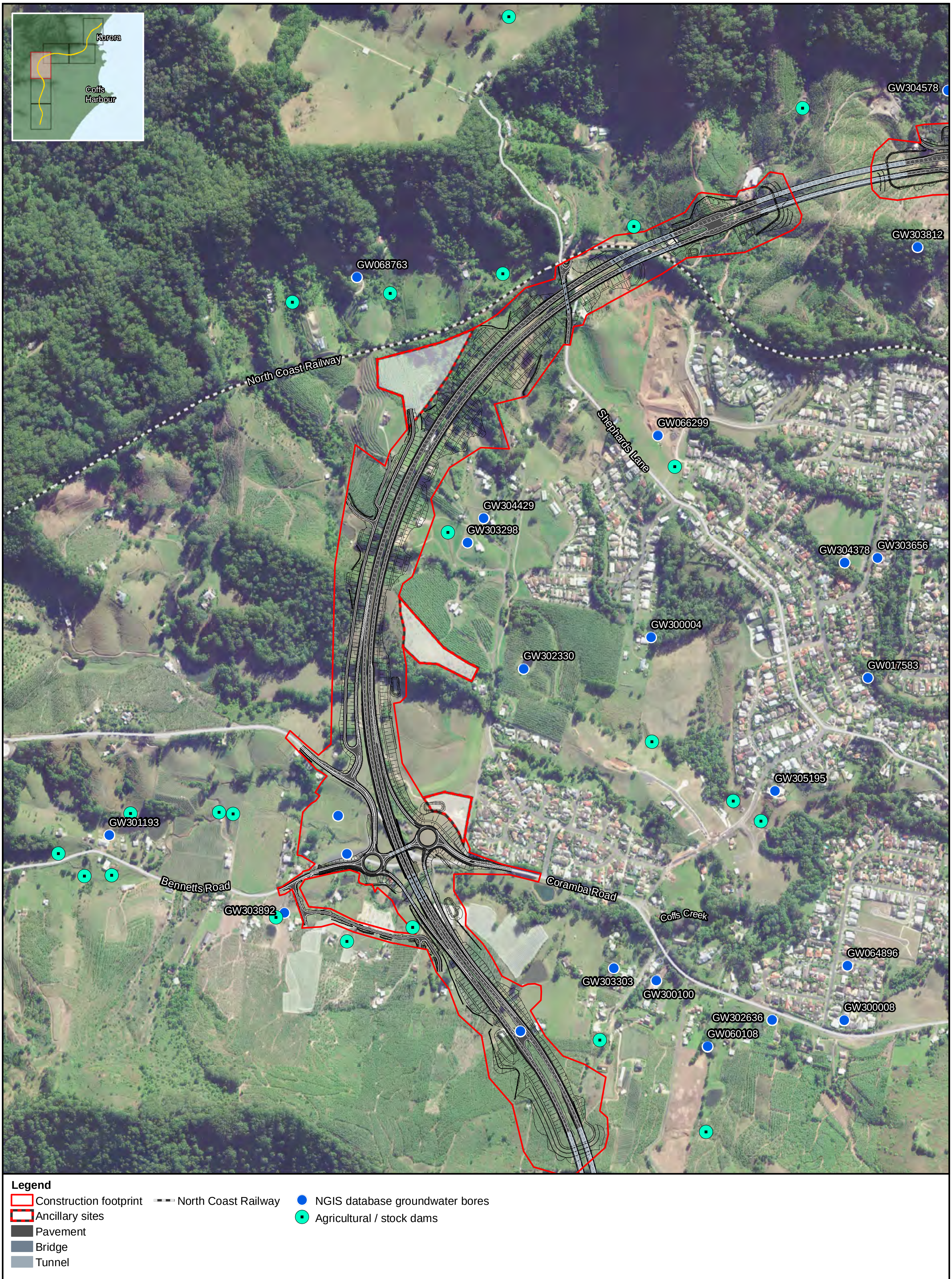
Legend

- Construction footprint
- Ancillary sites
- Pavement
- Bridge
- Tunnel
- North Coast Railway
- NGIS database groundwater bores
- Agricultural / stock dams

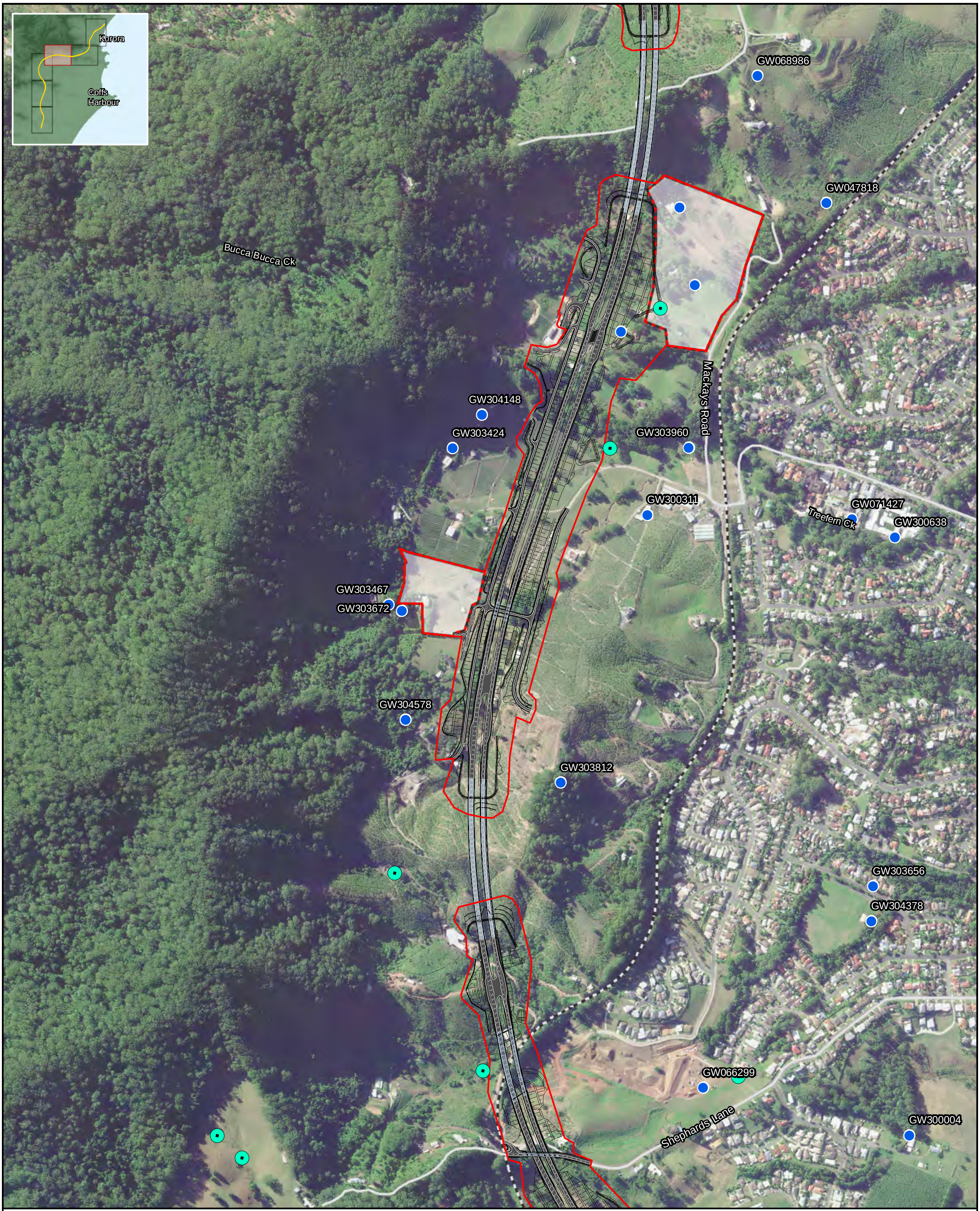
Coffs Harbour Bypass
Groundwater Bores and Agricultural Dams
Figure 17-1

0 0.1 0.2 0.3 km
Scale @A4: 1:12,390
GDA 1994 MGA Zone 56
N

Coffs Harbour Bypass
Groundwater Bores and Agricultural Dams
Figure 17-2



Coffs Harbour Bypass
Groundwater Bores and Agricultural Dams
Figure 17-3



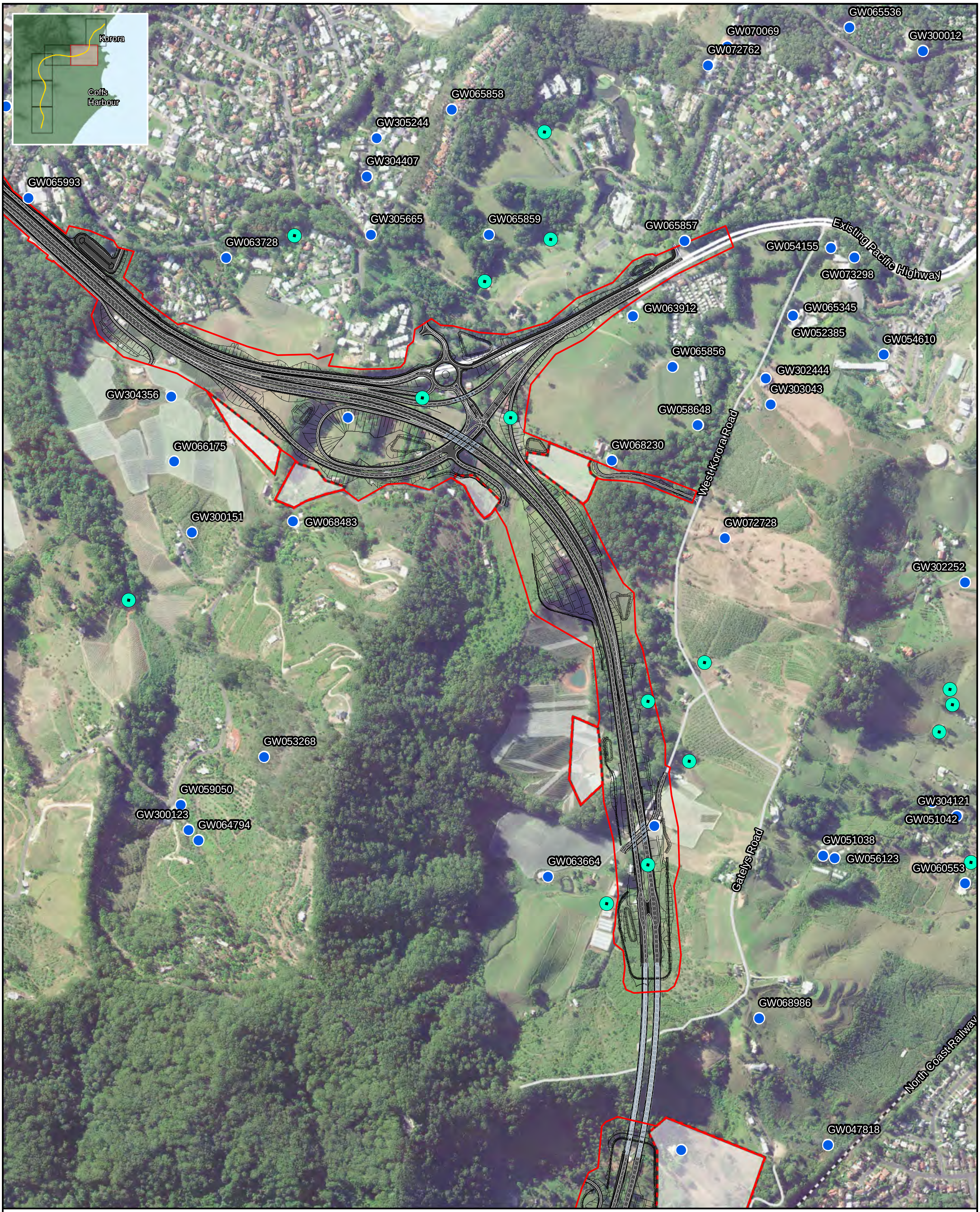
Legend

- Construction footprint
- Ancillary sites
- Pavement
- Bridge
- Tunnel
- North Coast Railway
- NGIS database groundwater bores
- Agricultural / stock dams

Coffs Harbour Bypass
Groundwater Bores and Agricultural Dams
Figure 17-4

0 0.1 0.2 0.3 km
Scale @A4: 1:12,410
GDA 1994 MGA Zone 56

N



Legend

- Construction footprint
- Ancillary sites
- Pavement
- Bridge
- Tunnel
- North Coast Railway
- NGIS database groundwater bores
- Agricultural / stock dams

Coffs Harbour Bypass
Groundwater Bores and Agricultural Dams
Figure 17-5

0 0.1 0.2 0.3 km
N
Scale @A4: 1:12,420
GDA 1994 MGA Zone 56

Legend

- Construction footprint
- Ancillary sites
- Pavement
- Bridge
- Tunnel
- North Coast Railway
- NGIS database groundwater bores
- Agricultural / stock dams

Coffs Harbour Bypass
Groundwater Bores and Agricultural Dams
Figure 17-6

0 0.1 0.2 0.3 km
Scale @A4: 1:12,410
GDA 1994 MGA Zone 56
N

2.7 Conceptual hydrogeological model

Conceptual models simplify and describe how complex systems work and how components of those systems interact which each other. A conceptual hydrogeological model for the Coffs Harbour regional area is presented in Figure 18. Additional local scale conceptual models describing potential impacts from the project are presented in **Section 3.3 Groundwater assessment**.

Four systems have been considered in the hydrogeological conceptual model for the study area: surface water (including creeks, lakes and wetlands), surficial deposits, and two relevant aquifers; an alluvial aquifer and a regional fractured bedrock aquifer. A third aquifer is also shown on the conceptual model, the coastal sands aquifer, however in terms of impacts the project is sufficiently far away to be of relevance.

- Surficial deposits made up of colluvial and residual soil horizons comprising a variable mixture of clays, silts, sands and gravel contain localised, perched groundwater. These deposits are highly variable in thickness and distribution but are likely to be present across much of the study area. Groundwater within these is strongly affected by seasonal rainfall and climatic factors and contribute to vertical recharge into the underlying fractured bedrock. Groundwater levels are likely to be highly variable but of limited thickness and may be unsaturated during dry periods. Lateral movement of groundwater in these deposits may occur at the interface between units of differing hydraulic conductivity and may locally lead to ponding or minor seepages at the surface.
- An alluvial aquifer which is split into an up-river and a floodplain alluvial aquifer comprising of interbedded silt, clay, sand and gravel lenses. These deposits are confined to creek lines and their floodplains, having been deposited by riverine processes. The boundary between the up-river alluvium and floodplain alluvium is defined by the tidal limit of the creek however there is unlikely to be a distinct boundary between these aquifers. The aquifer becomes vertically and laterally more persistent towards the east of the project. Groundwater levels are close to ground surface within this aquifer, broadly at the same level as the creek water.
- A regional fractured bedrock aquifer (comprising the Brooklana and Coramba Beds) which forms the undulating foothills and ridgelines in the Coffs Harbour area and underlies the entire region. Groundwater in the aquifer is contained within discontinuities comprising faults, joint sets and shear zones. Groundwater flow is slow and comprises a deep regional flow pattern and a shallower one which is affected by local topographic variation and recharge. Groundwater levels in this aquifer vary significantly across the alignment from a few metres below ground surface in topographically lower areas to more than 50m at ridgelines. Where groundwater levels are close to the surface during wet periods spring emergence is likely to occur, particularly at large breaks in slope.

Recharge to all of the units is from rainfall which occurs across the catchment. Vertical infiltration through residual soils and directly at outcrop can lead to rapid changes in groundwater level. Recharge to the alluvial aquifers is from direct precipitation and recharge from creek flow during wet periods. During drier periods the alluvial aquifers contribute base flow to the creeks. Creek flow discharge is predominantly driven by surface water run-off with some contribution from the underlying alluvial aquifer.

Groundwater flow directions in the shallow fractured bedrock aquifer is principally towards the coast, generally being acting as subdued version of the topography, however the rock structure (shear zones and faults) will locally affect the groundwater flow patterns.

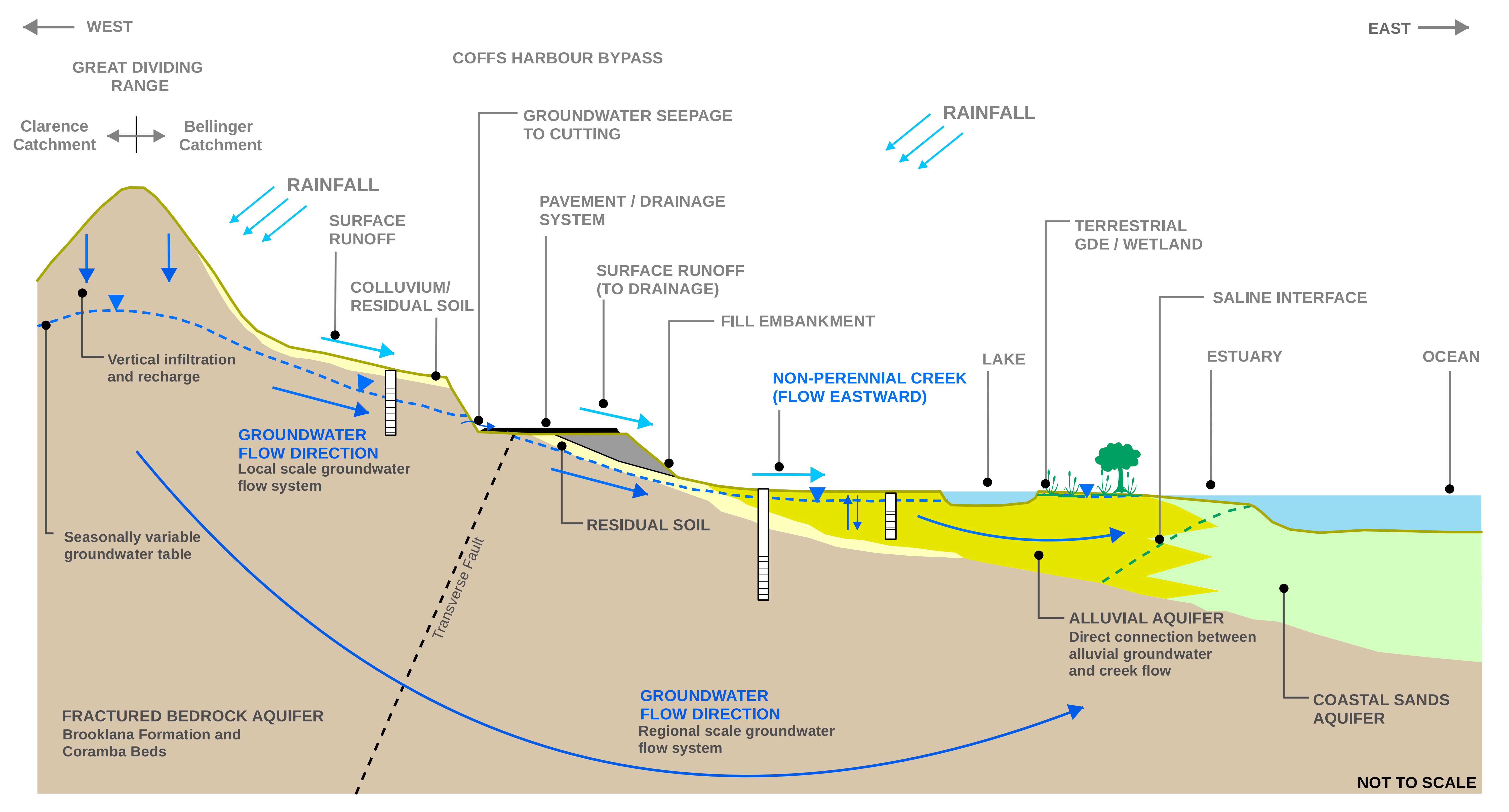
Groundwater flow direction in the alluvial aquifers is locally dictated by the extent of the alluvial deposits but is also broadly to the east, following the flow direction of the creek. The up-river alluvium is strongly connected to creek flow and groundwater flow rates are likely to be rapid. Floodplain alluvium is less connected with creek flows and groundwater flow is slower. Discharge from the alluvial aquifers occurs as creek flows or ultimately discharges at the sea.

The connectivity between alluvial aquifers and fractured bedrock aquifers is understood to be low but variable. Vertical gradients are likely to exist between the aquifer units. Connectivity between the aquifers will be dictated by the alluvial material type and discontinuity distribution within the fractured rock however the overall quantity of groundwater discharge from the fractured bedrock aquifer is low.

Losses from the groundwater systems include discharge to surface at springs or creeks, evapotranspirational losses from GDEs and native vegetation utilising groundwater, discharge to the ocean and groundwater utilisation. **Table 12** provides a summary of information relating to the aquifers in the study area. **Figure 18** presents the regional scale hydrogeological conceptual model. Local scale conceptual models are presented in **Section 3.3** which further detail the hydrogeological processes at a project scale context.

Table 11: Project groundwater summary

Property	Residual soil/colluvial	Alluvial	Fractured bedrock
Hydraulic conductivity	Low to high	High where sand and gravel dominated, low where clay and silt dominated	Low
Storage / specific yield	High (up to 35%)	High (up to 35%)	Low (1% - 5%)
Recharge	Recharge from rainfall at surface	Recharge from rainfall at surface, creek flow, residual soil, likely limited input from fractured bedrock aquifer	Recharge from overlying residual soils and directly at surface where outcrop
Groundwater levels	Perched groundwater, likely to be variable in response to rainfall events and of limited thickness. May be unsaturated during dry periods	Variable – levels close to surface (within a few metres) along main drainage channel lines. Water levels expected to respond rapidly to rainfall events	Large variation in groundwater levels along alignment from 2mbgl to 50mbgl. Water level fluctuation at individual bores varies from <1m to ~10m due to rainfall and recharge.
Flow direction	Expected to be locally variable. Vertical flow contributing to recharge, lateral flow where contrast in hydraulic conductivity of units	Generally, towards east but influenced by creek line geometry and alluvial deposit extent	Regional flow towards coast, shallow groundwater flow affected by local variations in topography and recharge.
Groundwater quality	Fresh	Fresh	Slightly acidic to slightly alkaline, generally fresh quality
Groundwater users	Unlikely to be significant contribution	Water supply wells, water supply taken directly from creek	Water supply wells, inflow to agricultural dams from springs
GDEs / baseflow	Limited, may provide some water locally to native vegetation if present	Significant contribution to baseflow to creeks/rivers and GDEs/vegetation	Unlikely to provide significant baseflow to creeks and/or GDEs
Estimated travel time in aquifer	Unknown, likely days to months	Days to months	Years to decades



COMMERCIAL IN CONFIDENCE

Legend

- Saline Interface
- Fault
- Groundwater Table
- Fill
- Water
- Surficial Deposits
- Alluvial Aquifer
- Coastal Sands Aquifer
- Fractured Bedrock Aquifer
- Groundwater supply bore

Client
Roads and Maritime Services

Job Title
Coffs Harbour Bypass Concept Design EIS

Figure Title
Hydrogeological Conceptual Model (Regional Scale)

D1	15/07/2019	CJ	TR	JL
Issue	Date	By	Chkd	Appd

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Scale at A3	Figure Status
NTS	Issue

Coordinate System
GDA 1994 MGA Zone 56

Job No	Figure No
248379	18

\\global.arup.com\australasia\BNE\Projects\248000\248379-00 Coffs Harbour Bypass\Work\Internal\GIS\MXD\Geotech\Map\GW Technical Appendix\Alternative Design\Figure 18 Regional_Conceptual_Hydro_Model.mxd

3 Impact assessment approach

3.1 Project elements

The project has major construction elements which have the potential to impact on the hydrogeological environment. These include:

- Three short drained tunnels through ridges at Roberts Hill (around 190 m long), Shephards Lane (around 360 m long), and Gatelys Road (around 450 m long)
- A series of drained cuttings and embankments along the alignment
- Structures to pass over local roads and creeks as well as a bridge over the North Coast Railway

3.2 Potential groundwater issues

Groundwater impacts considered as part of this assessment include risks to a variety of receptors which include groundwater users (groundwater that is extracted by bore for stock and domestic supplies, irrigation needs or municipal reserves) and the natural environments, which require sufficient groundwater supply and quality to maintain function. The potential risks that are considered as part of this assessment are described below:

- Risks to water supply quantity as a result of interruption of groundwater flow or changes in groundwater level (either through changes in permeability or impedance in flow or due to changes in supply or discharge as a result of groundwater interception for instance, in cuts and tunnels).
- Risks to water supply quality from anthropogenic (contamination) or natural sources (salinity and acid sulfate materials) which may be altered or affected by the project.
- Risks to water supply and quality to the natural environment (for instance baseflow to creeks, streams and rivers) occurring as a result of ponding in areas of fill, or drainage in areas of cut or tunnels.
- Risk to groundwater dependent ecosystems and vegetation which may be dependent on groundwater supply
- Risks due to settlement caused by changes in groundwater levels caused by interception of groundwater flow.

The assessment of impacts on the groundwater environment has been undertaken using both quantitative and qualitative approaches. For parts of the scheme which are anticipated to cause greater impacts on the groundwater environment (cuttings and tunnels which extend below the groundwater table), a numerical groundwater modelling approach has been used. For other elements of the scheme a qualitative assessment of the potential impacts has been undertaken. Detailed focus was given to the elements of the scheme anticipated to have the greatest impact on the groundwater environment. These elements (cuts and tunnels) directly interact with the groundwater environment and their impact can be predicted using local scale (i.e. non-regional) numerical models. Although other elements of the scheme (such as embankments) may indirectly affect groundwater due to changes in groundwater recharge, the effects are likely to be lesser and more localised.

Potential groundwater impacts to receptors are evaluated against relevant legislation requirements such as the NSW Aquifer Interference Policy and requirements of relevant water sharing plans (see **Section 4** for additional information).

3.3 Groundwater assessment

Cuttings and drained tunnels have the potential to impact groundwater levels where they extend below the existing groundwater surface. The major tunnel elements and cuttings are located within the foothills and ridgelines of the Great Dividing Range to the west of Coffs Harbour. The three tunnels in particular cross below prominent ridges lines (Roberts Hill, Shephards Lane and Gatelys Road) which form local-scale catchment divides.

Where seepage from groundwater into excavations occurs during construction, and into permanent drainage systems during operation, groundwater levels will be lowered (drawdown) in the area surrounding the cuttings and tunnels. Seepage which enters the cuttings and tunnels is also captured, and without remedial measures will be prevented from flowing along its natural course. The extent of drawdown and seepage rates into the cuttings/tunnels will depend on the depth below groundwater levels which the elements extend, the length over which seepage occurs and the local hydrogeological conditions at each of the cuttings.

To evaluate the potential impacts associated with each cutting and tunnel along the alignment the proposed mainline elevations were compared to groundwater level information obtained from geotechnical investigations and publicly available information.

The range of groundwater levels at each cutting and tunnel was evaluated, with an average level determined over the period of monitoring. The average groundwater level was compared to the mainline elevation to assess the maximum potential drawdown which could occur at each cutting/tunnel.

Based on this assessment, each of the cuttings and tunnels were classified into three types based on the following:

Type A (moderate to high impact)

Where the design level of the cutting or tunnel is predicted to be below the groundwater table. This could lead to localised lowering of water levels around the cutting sides which may:

- Affect groundwater flow rates and discharges downgradient potentially affecting GDEs or other groundwater users, if present within the zone of influence of the cutting.

- Have more than a minimal impact on nearby water supply works as defined by the NSW Aquifer Interference Policy.
- Cause engineering mitigations to be implemented during construction or operation of the road system e.g. drainage blankets beneath pavement, pressure reduction drainage in cut batters.

Type B (negligible to low impact)

Where the design level of the cutting or tunnel is within 5 m of the groundwater table where there is not expected to be an adverse impact to the groundwater regime and engineering mitigation measures are not expected to be required. Fluctuation in groundwater levels may lead to interception of the water table during wet periods although this is likely to be for short periods only. Type B cuts may impact on design and construction but are unlikely to affect nearby waterworks or GDEs if any (for example where the groundwater level rises to the grade level after large rainfall events).

Type C (no impact)

Where groundwater levels are greater than 5m below the design cut level with no anticipated impact

Where adjacent to one another, the side road/access ramp cuttings and the mainline cutting are grouped together, with the lowest design level assumed as a conservative estimate of impact. Only those side road/access ramp cuttings which are not grouped are presented in **Table 12**.

To further explain the types of issues which may occur due to construction of the cuttings and tunnels, a series of local scale conceptual models are presented in **Figure 19** to **Figure 22**. The idealised local baseline hydrogeological conceptual model is presented in **Figure 19** which provides a simplified model of the local scale hydrogeological setting of the project.

Conceptual models highlighting the potential impacts for Type A cuttings, Type B cuttings and drained tunnels are presented in **Figure 20, Figure 21** and **Figure 22** respectively.

These conceptual models highlight the range of potential impacts which could occur as a result of construction of the tunnels and cuttings; it is noted that not all impacts will occur at each location. Further assessment of the potential impacts at each individual cutting and tunnel is provided in **Section 4 Assessment of potential impacts**.

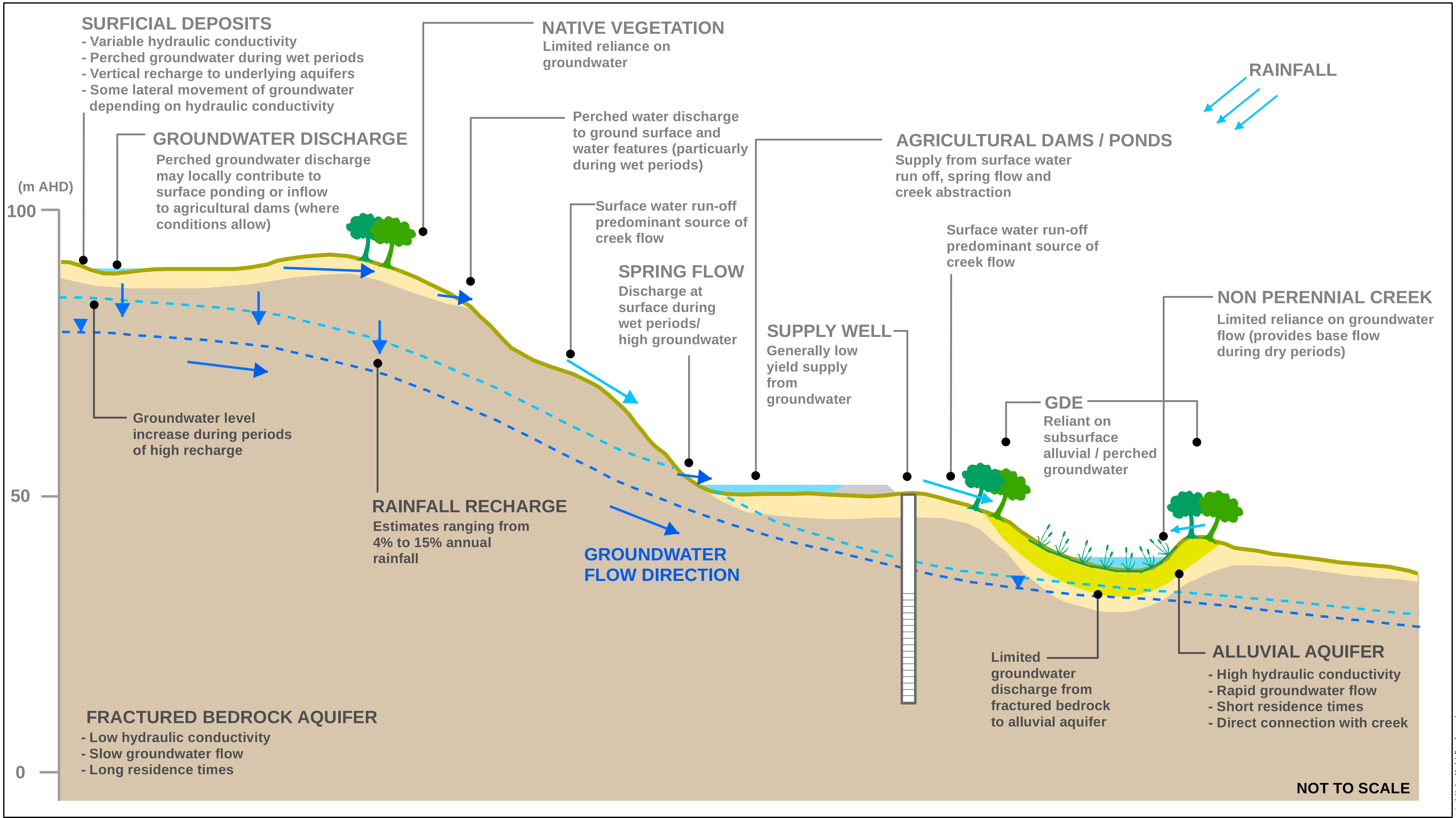
The results of this assessment indicate that there are:

- Seven type A cuttings
- Three Type A tunnels
- Twelve type B cuttings
- Five type C cuttings

For Type B cuttings, where the average groundwater level is anticipated to be below the base of the cutting, the impacts are anticipated to be minor. Inflow to these cuttings is only likely to occur during wet periods when groundwater levels are highest and although there may be capture of some throughflow, the time period over which this is expected to occur will be limited (groundwater data indicates generally rapid and peaky response to rainfall events). The list of cuttings and tunnels included in the project along with the assessment of type is provided in **Table 12**. The location of each cutting along the project is presented in **Figure 23**.

The proposed tunnels and cuttings along the alignment are principally sited within the fractured bedrock aquifer. Minor alluvial deposits may be locally impacted in certain cut areas (highlighted in **Table 12**) however there is generally very limited overlap between areas of cutting and mapped alluvial deposits; the direct impact (from excavation activity) to the alluvial deposits is therefore likely to be limited. Impacts due to reduction in groundwater flow are discussed in **Section 4**.

Numerical groundwater modelling was undertaken at all mainline Type A cuttings/tunnels. The approach to the numerical modelling is described in **Section 3.4**.The results of the numerical modelling were then used to evaluate where potential impacts on the groundwater environment may occur.



Legend

Ground Surface	Fill	Alluvium
Water table (high)	Water	Residual Soil / Colluvium
Water table		Fractured Bedrock

Groundwater supply bore

Client
Roads and Maritime Services

Job Title
Coffs Harbour Bypass Concept Design EIS

Figure Title
Idealised Local Hydrogeological Scale Conceptual Model

Issue	Date	By	Chkd	Appd
D1	30/04/2019	CJ	TR	JL

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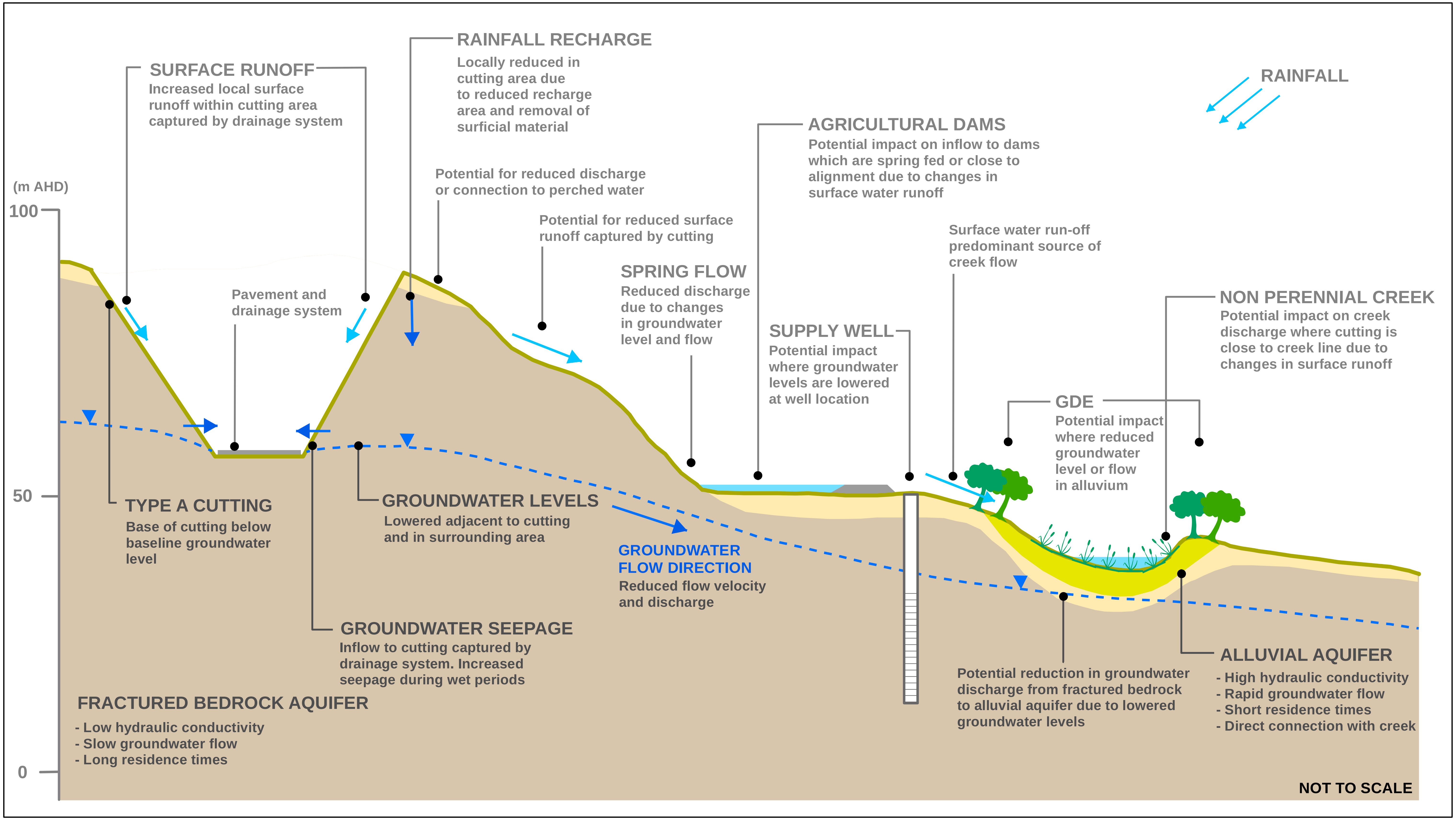
Scale at A3: **NTS**	Figure Status: **Issue**
Coordinate System: **GDA 1994 MGA Zone 56**	
Job No: **248379**	Figure No: **19**



Legend

- Ground Surface
- Water table
- Water
- Fill
- Alluvium
- Residual Soil / Colluvium
- Fractured Bedrock
- Groundwater supply bore

Client
Roads and Maritime Services

Job Title
Coffs Harbour Bypass
Concept Design EIS

Figure Title
Local Scale Hydrogeological Conceptual Model (Type A cutting)

Issue	Date	By	Chkd	Appd
D1	17/04/2019	CJ	TR	JL

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Scale at A3: **NTS**
Figure Status: **Issue**
Coordinate System: **GDA 1994 MGA Zone 56**
Job No: **248379**
Figure No: **20**

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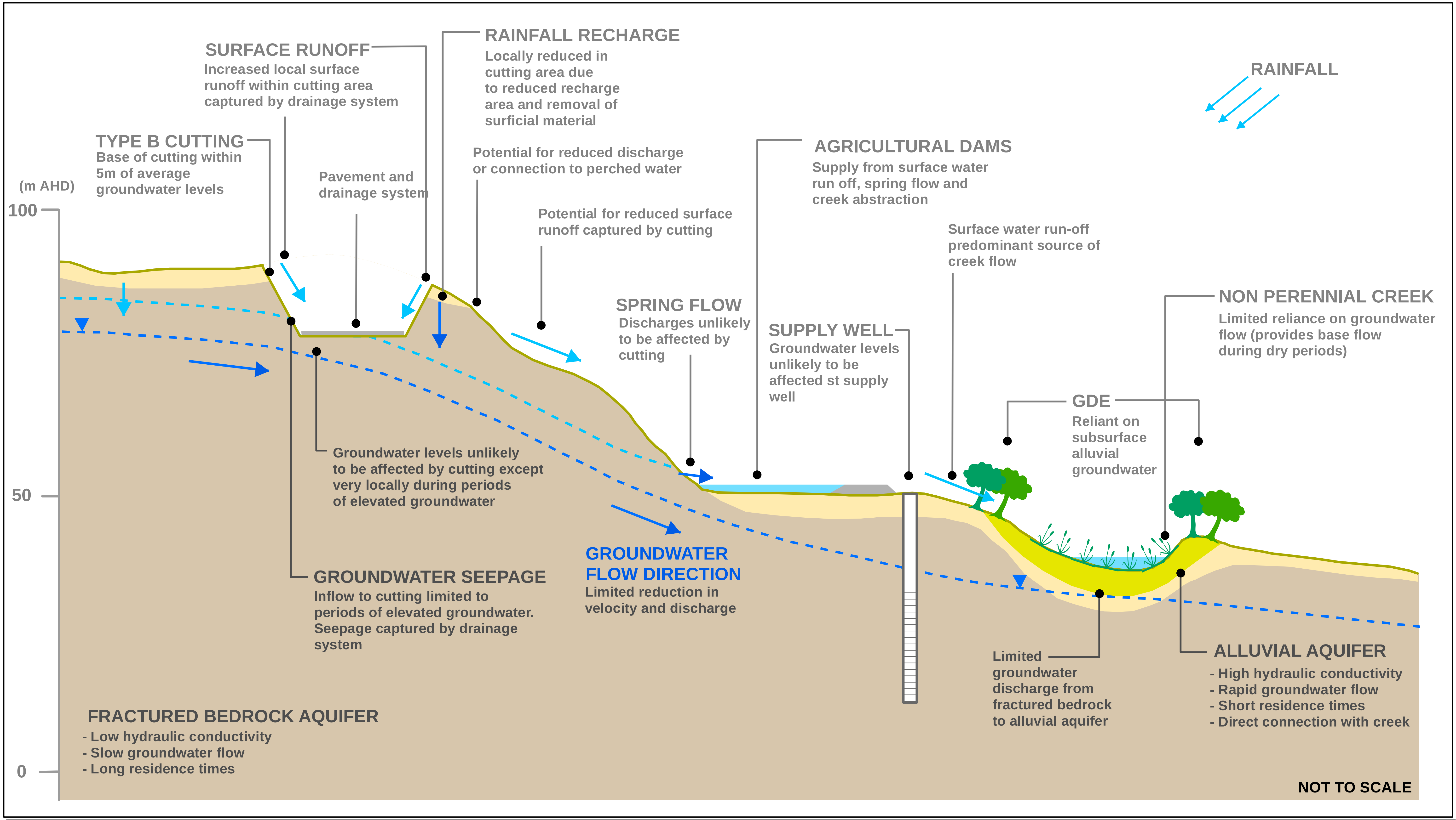

Legend

- Surface
- Water table (high)
- Water table
- Fill
- Water
- Alluvium
- Residual Soil / Colluvium
- Fractured Bedrock
- Groundwater supply bore

Client	Roads and Maritime Services
Job Title	Coffs Harbour Bypass Concept Design EIS
Figure Title	Local Scale Hydrogeological Conceptual Model (Type B cutting)

Issue	Date	By	Chkd	Appd
D1	17/04/2019	CJ	TR	JL

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Scale at A3	NTS
Figure Status	Issue
Coordinate System	GDA 1994 MGA Zone 56
Job No	248379
Figure No	21

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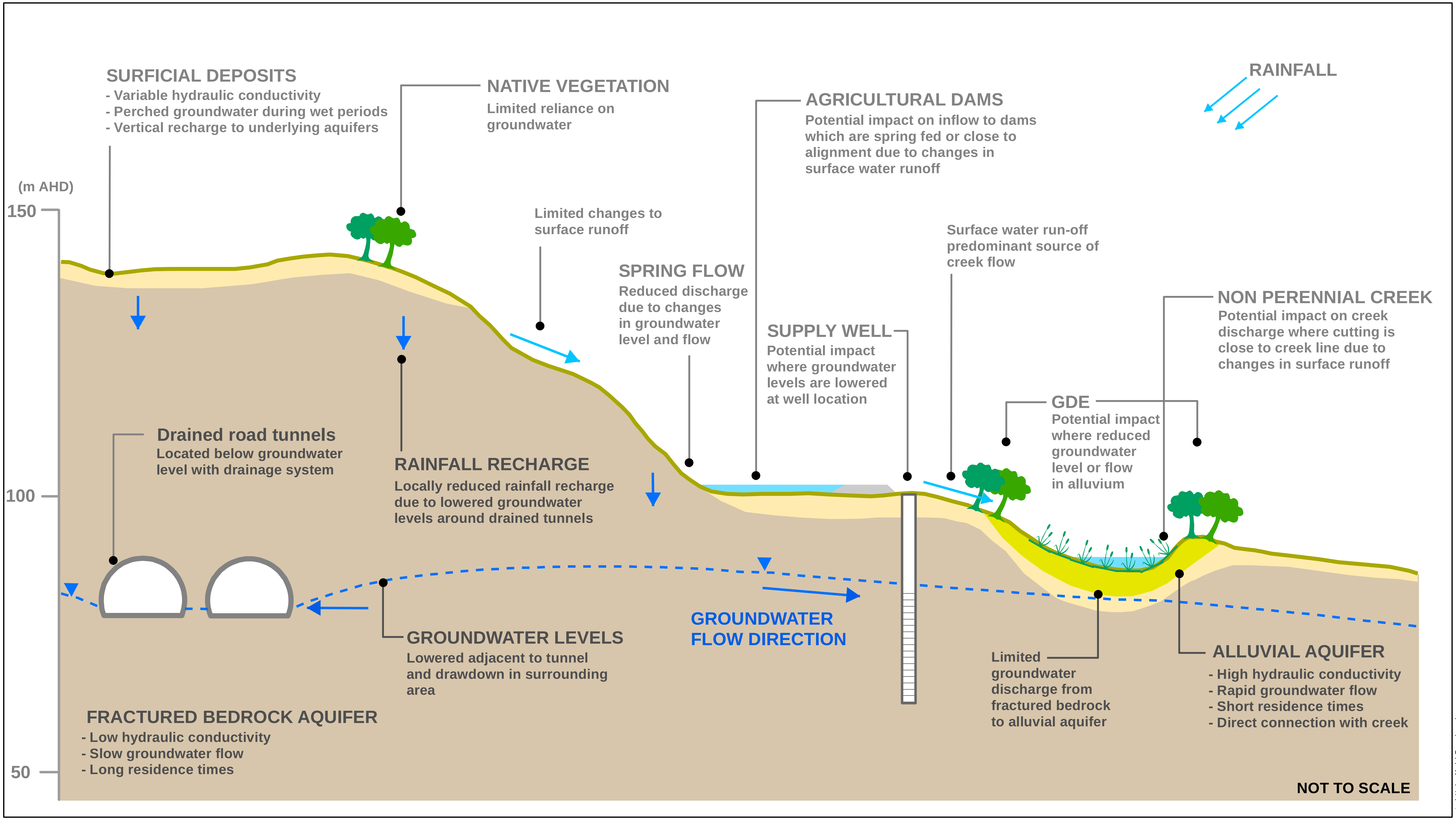

Legend

- Ground Surface
- Water table
- Tunnel
- Fill
- Water
- Alluvium
- Residual Soil / Colluvium
- Fractured Bedrock
- Groundwater supply bore

Client
Roads and Maritime Services

Job Title
Coffs Harbour Bypass
Concept Design EIS

Figure Title
Local Scale Hydrogeological Conceptual Model (Drained Tunnels)

Issue	Date	By	Chkd	Appd
D1	30/04/2019	CJ	TR	JL

ARUP
Level 10, 201 Kent Street
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Scale at A3: **NTS**
Figure Status: **Issue**
Coordinate System: **GDA 1994 MGA Zone 56**
Job No: **248379**
Figure No: **22**

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Table 12: Summary of project cuttings and tunnels

Cutting No. / Tunnel	Mainline Chainage from	Mainline Chainage to	Cutting Length (m)	Geological unit	Estimated average groundwater level (m AHD)	Design cut level (m AHD)	Potential drawdown (m)	Cutting type
C1	10025	10275	250	Fractured Bedrock	12	15	0	B
C2	10450	10540	90	Fractured Bedrock	12	24	0	C
C3	11460	11580	120	Fractured Bedrock	8	12	0	B
C4	12860	13425	565	Fractured Bedrock	43	30	13	A
Roberts Hill tunnel and portals	13600	13825	20	Fractured Bedrock	54	41	13	A
C8	13825	14400	575	Fractured Bedrock	56	43	13	A
C9	14660	14925	265	Fractured Bedrock	23	23	0	B
C10	15360	15490	130	Fractured Bedrock	37	37	0	B
C11	15560	15835	275	Fractured Bedrock/alluvial	54	49	5	A
C12	15975	16200	225	Fractured Bedrock	60	57	3	A
C13	16375	16475	100	Fractured Bedrock	72	72	0	B
C14	16780	16900	120	Fractured Bedrock	90	85	5	A
Shephards Lane tunnel and portals	17000	17375	375	Fractured Bedrock	118	88	30	A
C16	17975	18575	600	Fractured Bedrock	73	64	9	A
Gatelys Road tunnel and portals	18900	19350	450	Fractured Bedrock	112	73	39	A
C18	20250	20635	385	Fractured Bedrock	58	44	14	A
C19	21235	21950	715	Fractured Bedrock	40	51	0	C

Cutting No. / Tunnel	Mainline Chainage from	Mainline Chainage to	Cutting Length (m)	Geological unit	Estimated average groundwater level (m AHD)	Design cut level (m AHD)	Potential drawdown (m)	Cutting type
C20	22900	22970	70	Fractured Bedrock/alluvial	12 [6]	16	0	B
C21	23215	23415	200	Fractured Bedrock/alluvial	12 [6]	27	0	C
C9r[1]	Ramp at Englands Road Interchange		50	Fractured Bedrock	ID	2mbgl	2	B [5]
C10r[1]	Ramp at Englands Road Interchange		75	Fractured Bedrock	ID	1.5mbgl	1.5	B [5]
C11r[1]	Ramp at Englands Road Interchange		40	Fractured Bedrock/alluvial	ID	2mbgl	2	B [5]
C18r[1]	Ramp at Korora Hill Interchange		275	Fractured Bedrock	31	37	3	A
C19r[1]	Ramp at Korora Hill Interchange		65	Fractured Bedrock	14 [3]	27	0	C
C20r[1]	Ramp at Korora Hill Interchange		100	Fractured Bedrock	14 [3]	27	0	C
C5a[2]	Seaview Close side road		175	Fractured Bedrock/alluvial	12 [4]	14	0	B
C7a[2]	Seaview Close side road		125	Fractured Bedrock/alluvial	12 [4]	16	0	B

Notes:

ID – Insufficient nearby groundwater level data

[1] Ramp cutting

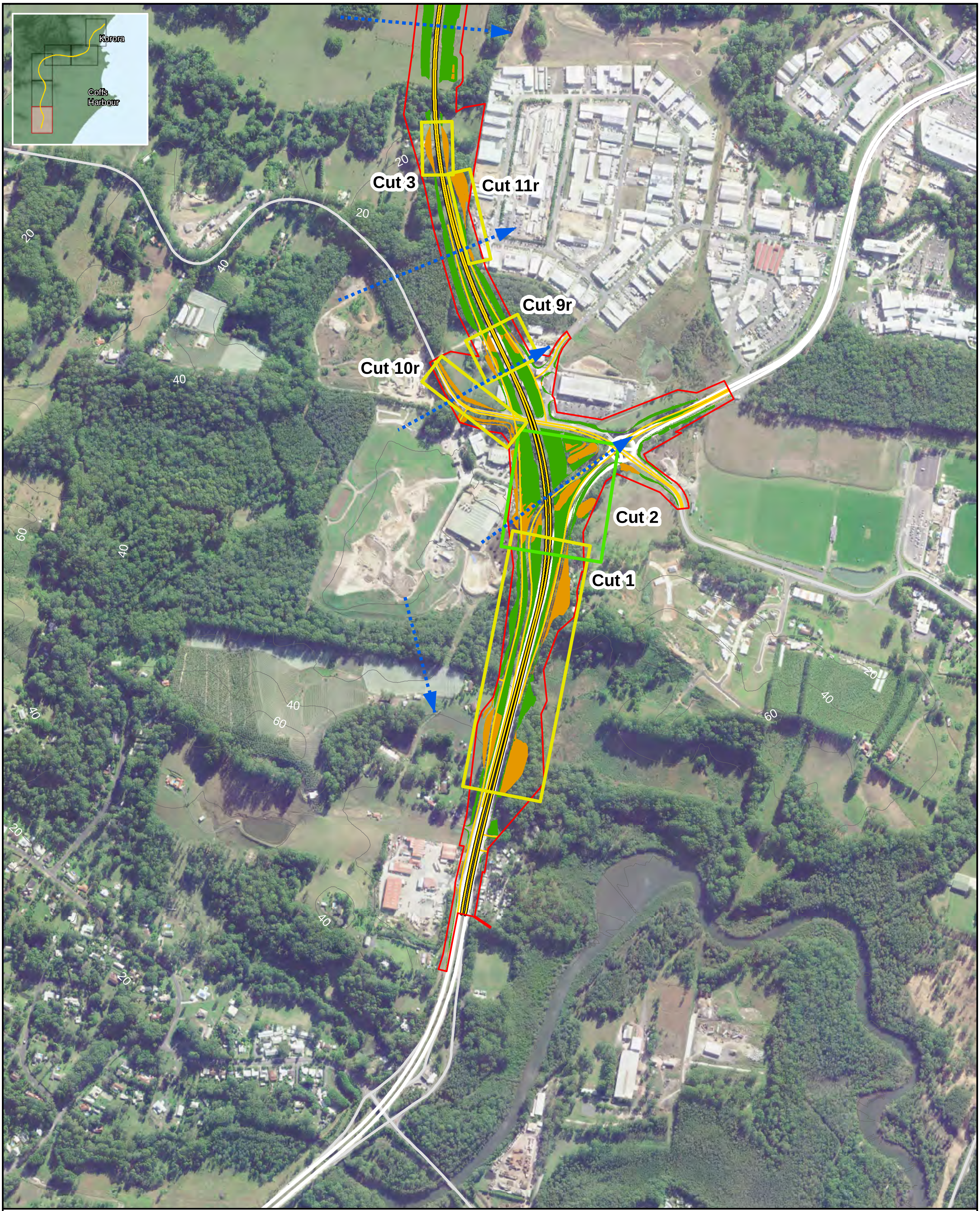
[2] Side road cutting

[3] Trial pit water strike used to infer a minimum groundwater level

[4] Publicly available groundwater level information from existing groundwater bores used to infer groundwater level

[5] Denotes minor cut (less than 2m deep where impact is anticipated to be negligible)

Highlighted rows indicate cuttings with numerical modelled undertake (**Appendix C1** and **Appendix C2**)



Legend

Construction footprint
Alignment
Tunnel
North Coast Railway

Cut and Fill

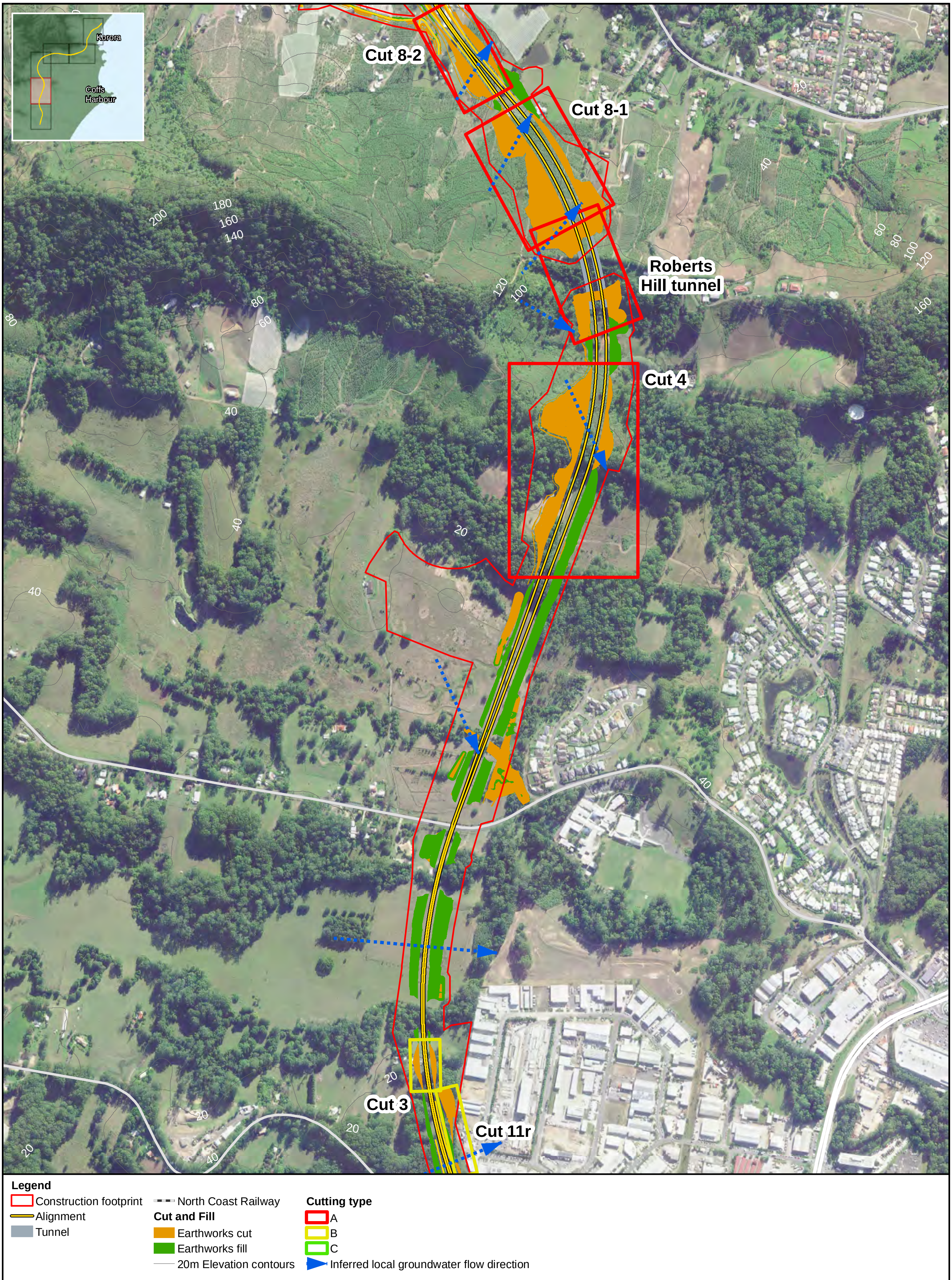
Earthworks cut
Earthworks fill
20m Elevation contours

Cutting type

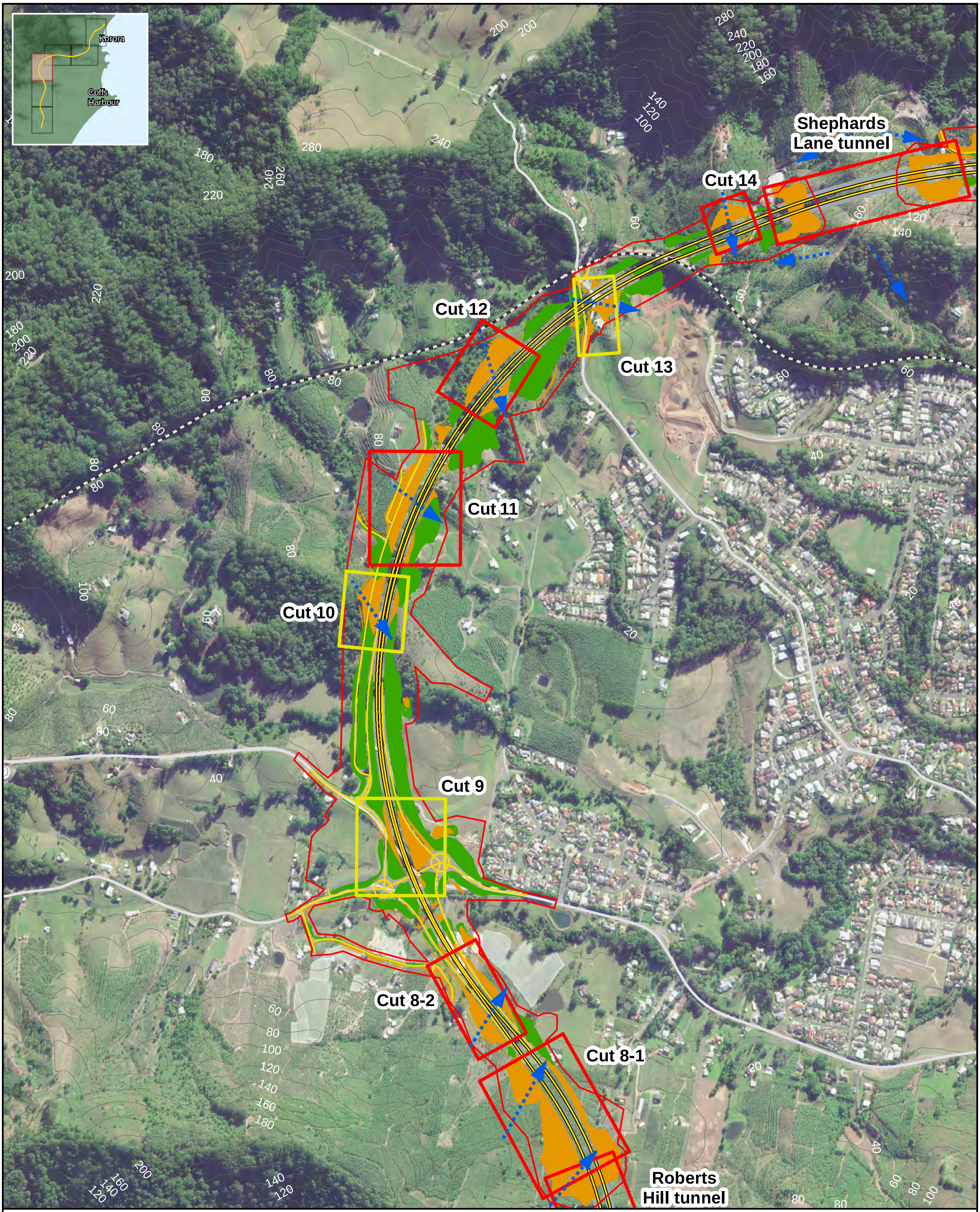
A
B
C
Inferred local groundwater flow direction

Coffs Harbour Bypass
Cutting Location and Type
Figure 23-1

0 0.1 0.2 0.3 km
Scale @A4: 1:12,390
GDA 1994 MGA Zone 56
N



Coffs Harbour Bypass
Cutting Location and Type
Figure 23-2



Legend

- Construction footprint
- Alignment
- Tunnel
- North Coast Railway

Cut and Fill

- Earthworks cut
- Earthworks fill
- 20m Elevation contours

Cutting type

- A
- B
- C
- Inferred local groundwater flow direction

0 0.1 0.2 0.3 km

Scale @A4: 1:12,650

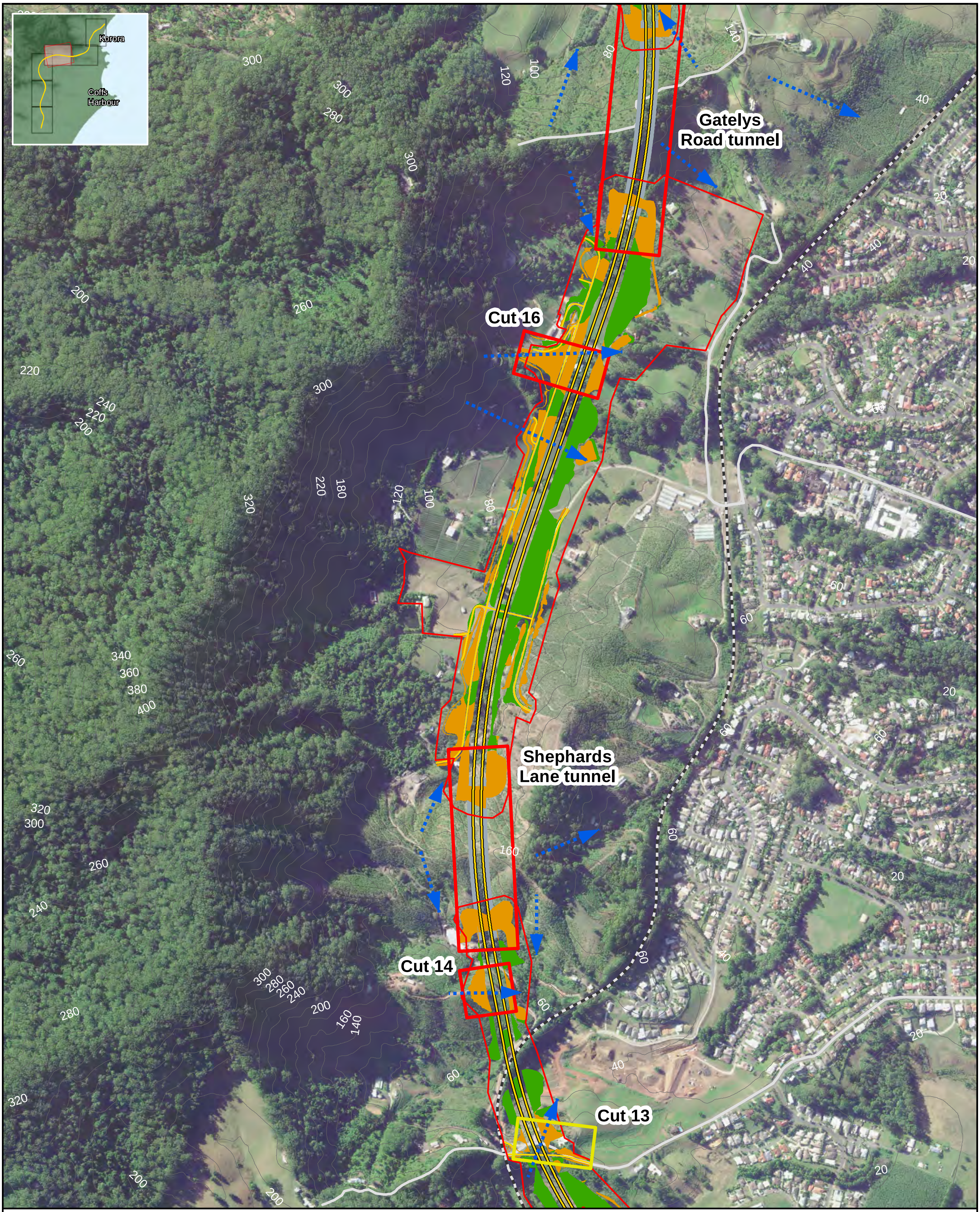
GDA 1994 MGA Zone 56

N

Coffs Harbour Bypass

Cutting Location and Type

Figure 23-3



Legend

Construction footprint
Alignment
Tunnel

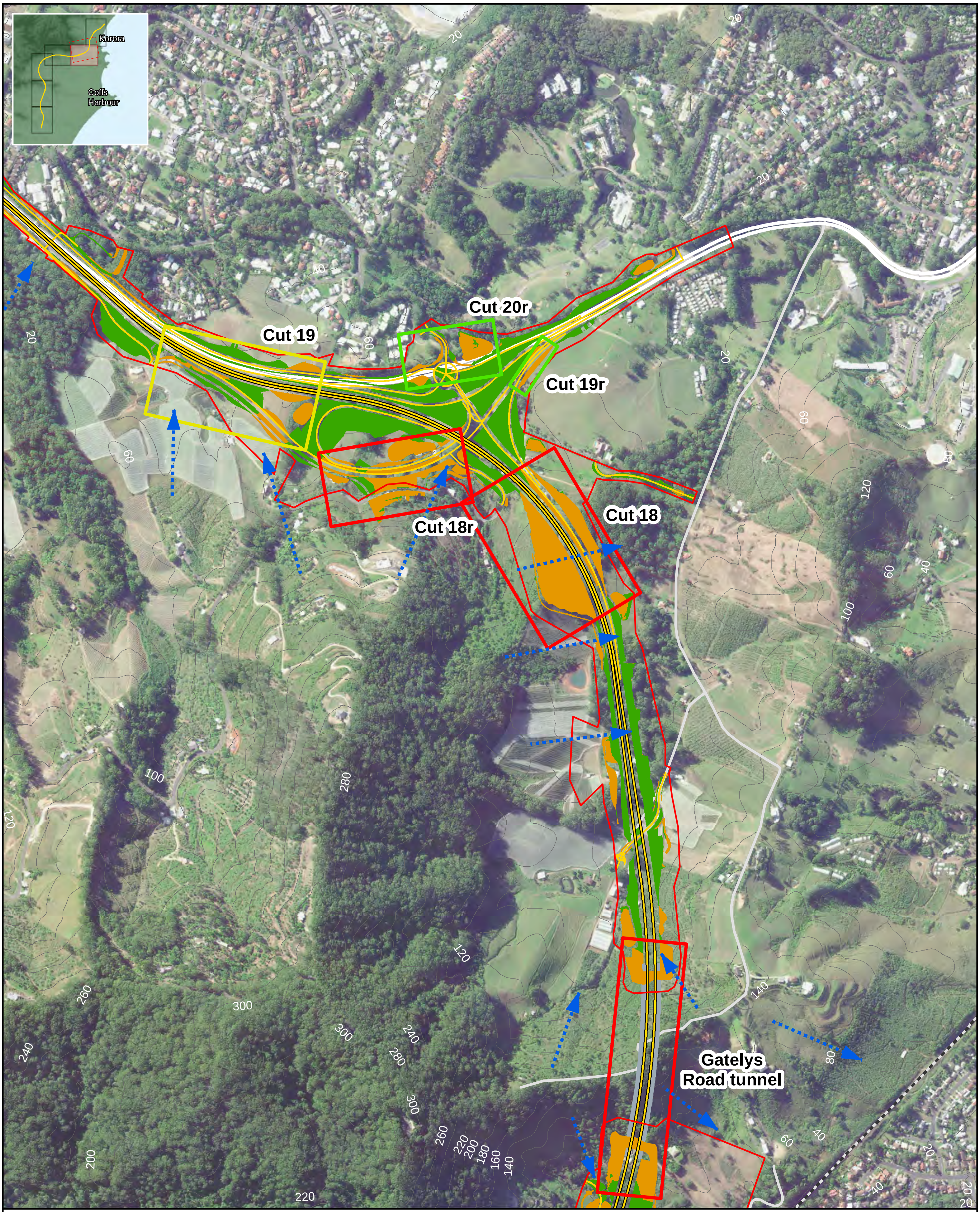
North Coast Railway

Cut and Fill
Earthworks cut
Earthworks fill
20m Elevation contours

Cutting type
A
B
C
Inferred local groundwater flow direction

Coffs Harbour Bypass
Cutting Location and Type
Figure 23-4

0 0.1 0.2 0.3 km
N
Scale @A4: 1:12,410
GDA 1994 MGA Zone 56



Legend

- Construction footprint
- Alignment
- Tunnel
- North Coast Railway

Cut and Fill

- Earthworks cut
- Earthworks fill
- 20m Elevation contours

Cutting type

- A
- B
- C
- Inferred local groundwater flow direction

0 0.1 0.2 0.3 km

Scale @A4: 1:12,410
GDA 1994 MGA Zone 56

Coffs Harbour Bypass
Cutting Location and Type
Figure 23-5

Coffs Harbour Bypass
Cutting Location and Type
Figure 23-6

3.4 Summary of groundwater numerical modelling

Based on the results of the assessment outlined in **Section 3.3 Groundwater assessment,** numerical groundwater modelling was undertaken at all Type A mainline cuttings and tunnels in order to evaluate the seepage inflow rates, potential drawdown at the cutting/tunnels and drawdown in the wider hydrogeological environment *(***Appendix C1, Appendix C2***)*. This section provides details of the methodology and approach taken towards the numerical modelling. Numerical modelling was undertaken with reference to the Australian groundwater modelling guidelines (*Barnett et al, 2012*).

3.4.1 Methodology

Numerical modelling was undertaken using 2D finite element groundwater modelling package 2D RS2 finite element software for the cuttings (**Appendix C1**) and FEFLOW 7.0 for the three tunnel sections (**Appendix C2**). At each of the location, a 2D section was created using topographic data and design drawings. The models were vertically oriented and aligned broadly with the interpreted groundwater flow lines.

The impact on the groundwater environment was predicted by inserting the cutting and running the model in steady state, to evaluate the long-term impact. The modelling was used to estimate the following:

- The distance over which there is a lowering (drawdown) of groundwater levels caused by the cuttings and tunnels
- Groundwater inflow rates to the cuttings and tunnels

The three tunnel models were also run as a transient assessment to evaluate the initial flush inflows and time taken to reach steady state conditions.

The following cuttings and tunnels were modelled as part of the impact assessment. These Type A cuttings all extend below groundwater levels measured from nearby monitoring wells as described in **Section 3.3**. The chainage at which the 2D section modelling was undertaken is also provided below.

- Cut 4 (chainage 13325)
- Roberts Hill tunnel and portals
- Cut 8 (split into two models – 8-1, chainage 13925 and 8-2, chainage 14300)
- Cut 11 (chainage 15750)
- Cut 12 (chainage 16075)
- Cut 14 (chainage 16850)
- Shephards Lane tunnel and portals
- Cut 16 (chainage 18450)
- Gatelys Road tunnel and portals
- Cut 18 (chainage 12400).

Geological units were assigned to each of the models based on the available nearby geotechnical investigation data. Each hydrogeological unit was modelled as a uniform porous medium with adopted hydraulic conductivities based on the results of borehole packer and piezometer falling head tests.

Rainfall recharge was applied to the ground surface as a percentage of annual rainfall. Fixed head boundary conditions were applied at the horizontal extents of the modelling domains to represent flow to the downgradient receptors. Boundary conditions in each of the models were set at a lateral distance which was sufficiently far from the cutting as to minimise the effect of the boundary conditions on the analysis. The downgradient boundary condition was generally set at a topographical low point where groundwater discharge is likely.

The modelling reports (**Appendix C1 and C2**) indicated that the cuttings and tunnels were assumed to have a negligible effect on the regional groundwater system and that the purpose of the modelling was to investigate the effects on the groundwater system in the locality of the cuts.

An example of each of model setup and geometry (hydrogeological units, water levels and cut/tunnel geometries) for one of the cuttings and tunnel models are presented in **Figure 24** and **Figure 25**. Full details of all of the models including cross section figures are presented in **Appendix C1 and C2.**

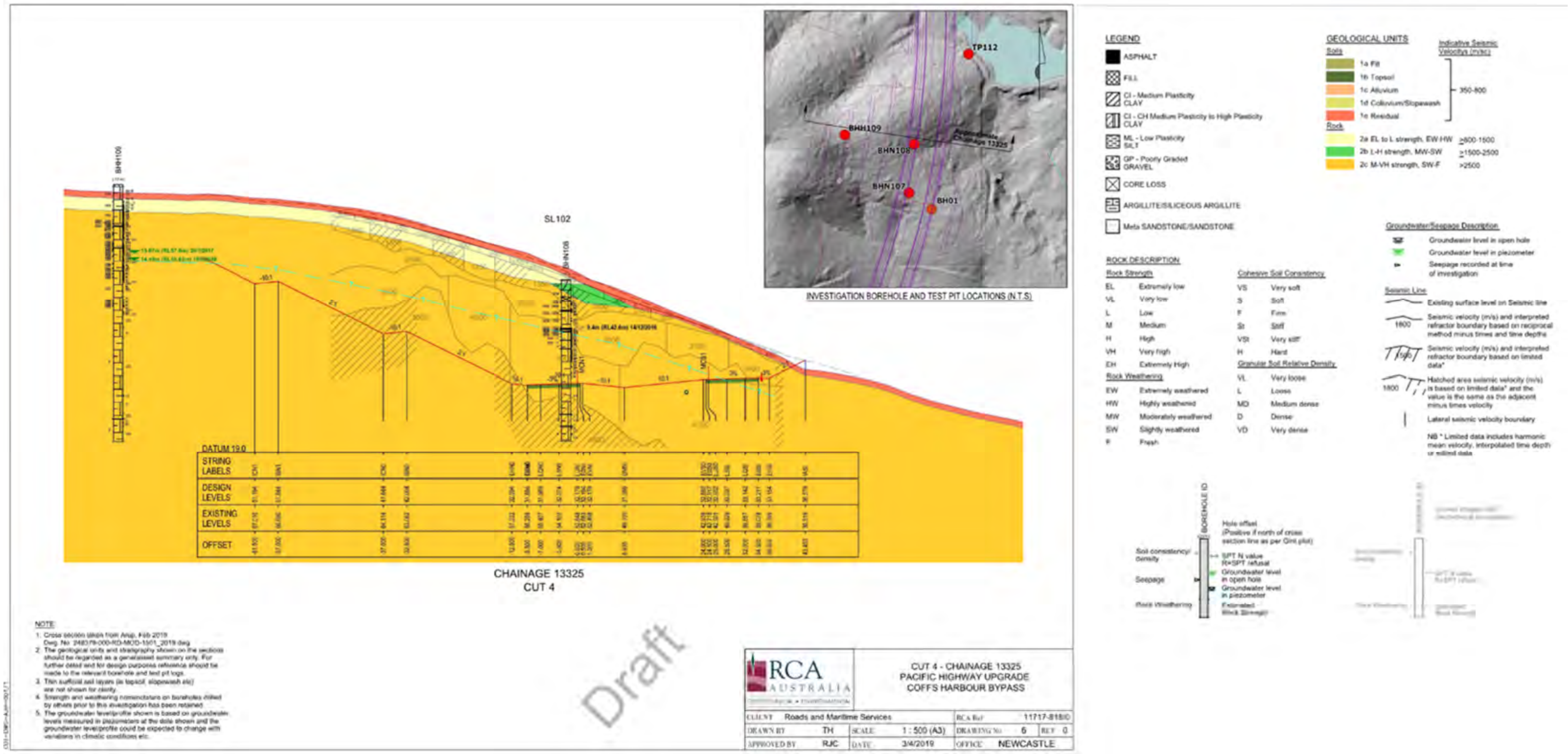


Figure 24: Cut 4 (chainage 13325) – 2D geometry, geology and groundwater levels for model setup

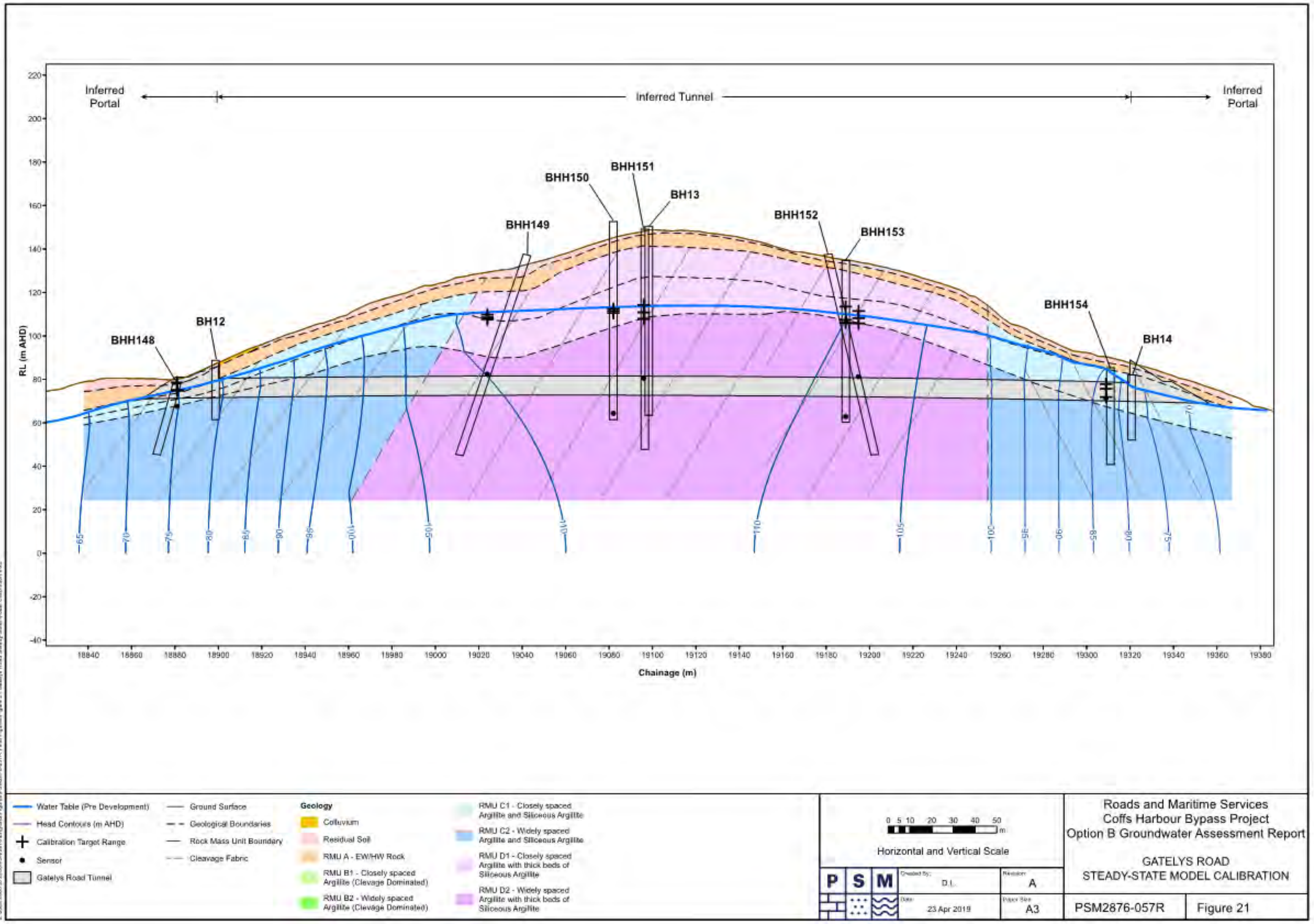


Figure 25: Gatelys Road steady state model set up and calibration results

3.4.2 Calibration

Steady state calibration was undertaken to provide an estimate of rainfall recharge (as a percentage of annual rainfall) and hydraulic conductivity by matching the simulated hydraulic heads to observed heads at monitoring locations. The hydraulic conductivity values were constrained by the range observed from packer and falling head testing undertaken during investigations. As the ratio of hydraulic conductivity to recharge is non-unique and many different combinations can lead to the reasonable simulation of existing heads, a variety of scenarios were modelled over the range of potential recharge rates. The recharge values (and the position of the downgradient constant head boundary) were varied until a satisfactory calibration was achieved.

Based on the geometric mean hydraulic conductivities, a recharge rate of 15% was required to achieve satisfactory calibration for the tunnel models. The cutting models found that adequate calibrations were achieved within the range of tested hydraulic conductivities at recharge rates ranging from between 5% and 5% of annual rainfall.

The range of hydraulic conductivity values used in the modelling at a recharge rate of 15% is presented in Table 13**.** A nominal anisotropy ratio of 1:2 horizontal to vertical hydraulic conductivity was adopted for the bedrock, and isotropic for the colluvium and residual soils. Sub-vertical cleavage planes were identified as the dominant defect set within the fresh bedrock indicating that vertical hydraulic conductivity is likely to be enhanced compared to the horizontal. Further information on the rock mass structure is provided in **Appendix C1** and **Appendix C2**.

Relatively high recharge rates were needed in some of the models to provide a reasonable calibration. This is broadly in line with assessment of the groundwater hydrograph data which indicated comparatively high recharge and high vertical hydraulic conductivity.

Table 13: Hydraulic conductivity values used for numerical modelling based on 15% recharge scenario

Geotechnical Unit	Material	Hydraulic Conductivity range (m/s)		Specific yield
		Cutting assessment	Tunnel assessment	
Soil	Residual soil and colluvium	1 $\times10^{-6}$ to 2 $\times10^{-6}$	2 $\times10^{-6}$ to 4 $\times10^{-6}$	0.05
Rock – 2A	Extremely weathered to highly weathered Argillite	1 $\times10^{-6}$ to 2 $\times10^{-6}$	2 $\times10^{-6}$	0.05
Rock – 2B	Moderately weathered Argillite	1 $\times10^{-7}$ to 2 $\times10^{-6}$	1 $\times10^{-6}$ to 2 $\times10^{-6}$	0.02

Geotechnical Unit	Material	Hydraulic Conductivity range (m/s)		Specific yield
		Cutting assessment	Tunnel assessment	
Rock – 2C	Slightly weathered to fresh Argillite	6 x10^{-9} to 1 x10^{-6}	7 x10^{-9} to 3 x10^{-7}	0.01 – 0.02

Transient calibration was also undertaken for the three tunnel models. Transient simulations were undertaken for a range of parameter combinations with recharge ranging from 2% to 20% of the recorded daily rainfall depths from the period October 2016 to March 2019.

Simulation results were compared to groundwater hydrographs from monitoring wells at each of the model locations. The results indicated that a recharge rate of at least 10 to 15% was required to replicate the observed trends in groundwater levels. Specific yield parameters for the soil and rock mass units based on a 15% recharge rate are provided in **Table 13.**

The combination of recharge, hydraulic conductivity and boundary conditions used for each model may not represent a unique solution. It is plausible that other combinations of the three parameters could also produce a similar approximation to the steady state conditions. The derived parameters for each model are included in **Appendix C1 and C2,** along with additional detail on the calibration and modelling process.

3.4.3 Predictive modelling

The calibrated cross-sectional groundwater flow models were then used to predict groundwater inflow and evaluate the lateral extents of drawdown away from the cuttings and drained tunnels.

For cut scenarios, the base of the cut was modelled with a fixed head and seepage faces were applied to the side walls of the cut. The fixed head boundary was assigned at the level of the subgrade drainage blanket. The batter slopes were assigned a reduced recharge rate to simulate the enhanced runoff along the batters. The total calculated flow rates into each individual cutting was estimated by taking the per metre inflow from the model and multiplying it across the length of the cutting below ground level, and proportionally based on the depth below water table. Predictive modelling was undertaken under recharge scenarios of 5% and 15% to evaluate the sensitivity to the parameter.

The tunnel scenarios were modelled using seepage face boundary conditions which were applied from the design road surface to the roof of the tunnel. Construction and operation of the drained tunnels will induce groundwater flow from the direction perpendicular to the tunnel alignment. To simulate lateral flow, a simplified 3D geometry was adopted whereby the model was extruded uniformly along the ridgeline which allowed for the simulation of lateral flow paths but did not accurately account for the 3D geometry of the ridgelines. For the purposes of the assessment, this is considered to be a reasonable simplification.

Drained tunnel models were undertaken at a baseline recharge rate of 15%. A sensitivity scenario was also undertaken at a recharge rate of 2%.

3.4.4 Modelling class

The numerical modelling undertaken as part of the impact assessment is based on available groundwater monitoring data collected along the alignment since 2017 and hydrogeological testing (from packer tests and slug test data). Most of the cuttings and tunnel sections have a reasonable spread of monitoring locations and nearly 2 years-worth of continuous groundwater monitoring data, providing a good baseline of hydrogeological conditions for the impact assessment.

Model calibration and sensitivity analyses led to a reasonable match between observed and model results and the results are considered to provide reasonable predictions for the purposes of the impact assessment.

Based on the Australian groundwater modelling guidelines (*Barnett et al, 2012*) the modelling classification is considered to be between Class 1 and Class 2, for the specific attributes below

- Data – Class 1/2 – Groundwater head observations are available but may not provide adequate coverage throughout the domain. In this case there is limited data available near to potentially sensitive receivers (i.e. agricultural dams, springs and creek lines/alluvial aquifers)
- Calibration – Class 1/2 – a reasonable steady state calibration to the available data was achieved for all models and transient calibration for the tunnel models
- Prediction – Class 1/2 – the timeframes and magnitudes of stresses for the predictive scenarios are such that the Class 2 indicators are generally satisfied.

3.4.5 Results

The results of the modelling in relation to predicted water take and drawdown are discussed below. The impacts on the groundwater environment and potential receptors are discussed in **Section 4**.

3.4.5.1 Water take

Table 14 presents the predicted steady state flow and estimated construction inflow rates into each of the modelled cuttings and tunnels. The maximum construction inflow for the tunnels has been predicted through transient modelling. The maximum construction inflow from the cuttings is estimated based on the tunnel transient analysis since modelling for the cuttings is only undertaken in steady state.

Table 14: Estimated water take from cuttings and tunnels

Cut / Tunnel	Estimated maximum steady state water take (kL/d)[1]	Estimated maximum construction inflow (kL/d)[3]
Cut 4	7.5	37.5
Roberts Hill tunnels and portals	10.6	98.0
Cut 8-1	17.3	86.7
Cut 8-2	12.1	60.3
Cut 11	0.5	2.5
Cut 12	0.1	0.2
Cut 14	13.6	68.0
Shephards Lane tunnels and portals	22.1	94.0
Cut 16	7.9	39.5
Gatelys Road tunnels and portals	55.0	305.0
Cut 18	7.6	37.8
C18r[2]	1.4	3.4
Total	156 kL/d 57 ML/yr	

Notes

[1] Maximum steady state water take uses 15% model recharge scenario

[2] Estimated using modelling results from adjacent cuts and anticipated drawdown at cutting

[3] Estimated maximum construction inflow rate based on five times maximum steady state inflow, except for tunnels where this has been individually modelled using transient analysis.

Groundwater discharge during construction will principally be drawn locally from storage within the bedrock as the cutting or tunnels are excavated into the groundwater table. Seepage rates may initially be higher than the predicted steady state inflows but are likely to reduce rapidly as groundwater levels in the surrounding bedrock are lowered.

Predicted steady state inflow rates into the cuttings and tunnels are quite variable, owing to the generally low permeability of the fractured bedrock aquifer, distribution of structural features, anisotropy, variability in groundwater levels and cut and tunnel depths. The largest anticipated flow rates are at Shephards Lane tunnels, Gatelys Road tunnels and Cut 8. The long term steady state inflow rates at these locations range from 22 kL/d (thousand litres per day) to 55 kL/d (0.3 L/s to 0.6 L/s). Inflow to these three sections accounts for approximately 70% of the predicted inflow across the project. The total predicted steady state inflow to the cuttings and tunnels is 158 kL/d (57 ML/yr).

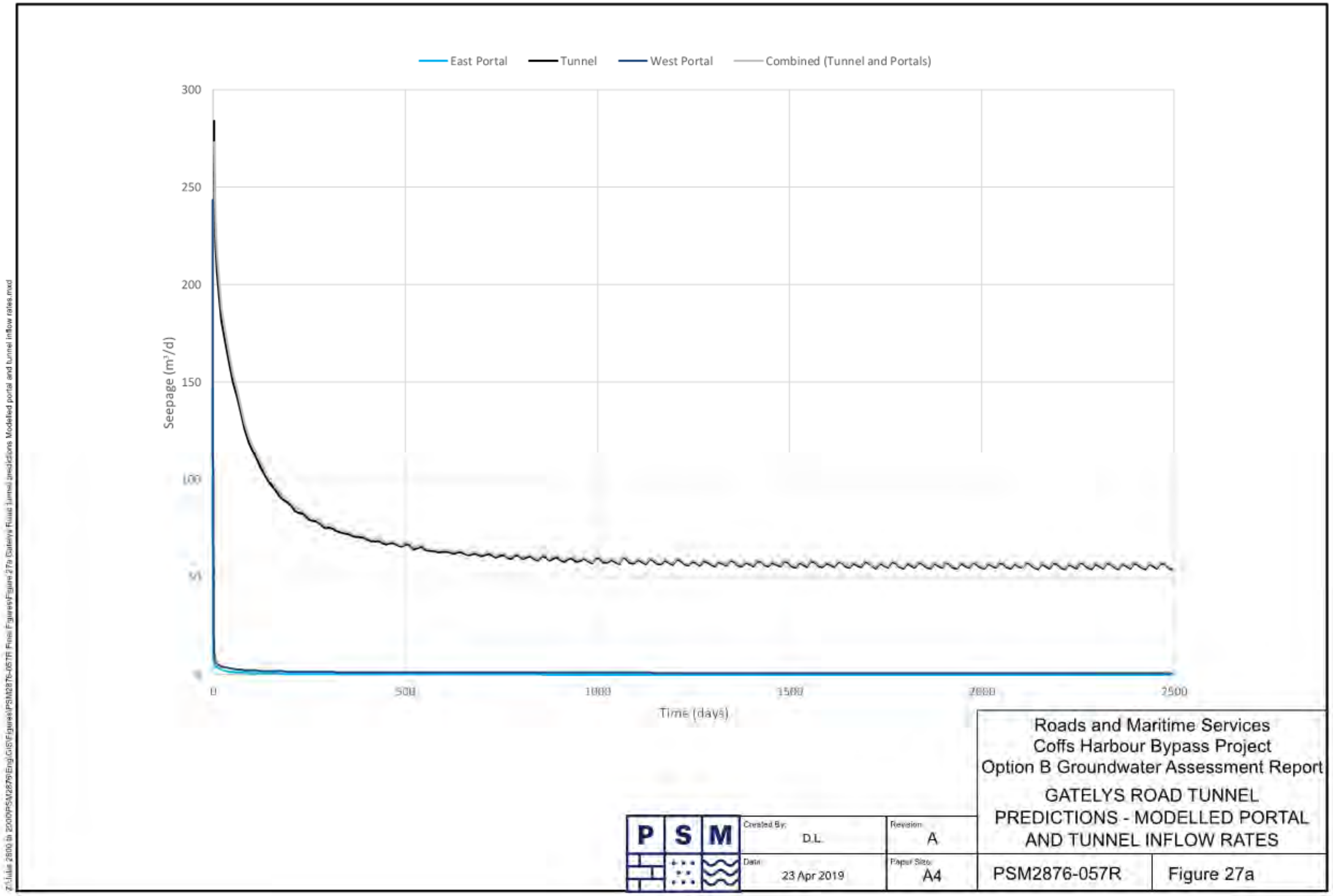
Numerical modelling of the drained tunnels indicates that the initial construction inflow rate may be between 4 and 9 times higher than the steady state inflow but that these very high flush inflows are only maintained for a short period of weeks rather than months or years. Modelling of Roberts Hill, Shephards Lane and Gatelys Road tunnels for instance indicates that construction inflows reduce to approximately twice the steady state inflow rates within around 50 days, 60 days and 100 days respectively. The time taken to develop steady state conditions and inflow ranges from around 3 years to 4 years. **Figure 26** presents an example of the predicted inflow changes over time at Gatelys Road. **Appendix C1 and C2** presents the numerical modelling results at other cuts and tunnels.

It is noted that the initial high inflow rates occur as a result of an assumption in the modelling that the tunnels and cuttings appear instantaneously across their entire length. This is a highly conservative assumption since it assumes that the tunnel and cuttings drain under maximum groundwater pressures across their entire length. The actual construction inflow rates at individual cuttings and tunnels will depend on the rate at which excavation or tunnelling takes place as this will control the area over which inflow occurs at any given time, and the resulting groundwater head conditions which control inflow rates.

The cumulative construction inflows for the entire project will also be dependent on the construction program and where excavations are being undertaken concurrently. A worst-case estimate of construction groundwater inflows would be to assume all that the maximum predicted inflows at each cutting and tunnels occur at the same time. However, given the rate at which the predicted initial construction flow rates are predicted to reduce and that the maximum flush construction inflow rates are likely to be an overestimate (due to the above assumption), this is considered to be unrealistic.

An estimate of the total construction water take for the project at this stage is highly uncertain. A conservative estimate may be to assume total of twice the maximum predicted steady state inflow rate at each cutting or drained tunnel which would produce an estimated construction inflow rate of around 314 kL/d. Further assessment of construction water take would be undertaken during detailed design, once construction programs have been refined. This will be undertaken to better quantify the likely volumes of water take during construction and to refine proposed mitigation approaches.

Figure 26: Predicted inflow rates at Gatelys Road Tunnel (*PSM, 2019*)



Groundwater inflow is likely to be concentrated at joints and fractures intercepted by the cuttings and tunnels. Estimated steady state inflow rates along the batter faces of cuts are likely to be lower than the average daily pan evaporation rate (between 2.3mm/day to 6.2mm/day) and therefore may be lost to evaporation. However, most of groundwater take at cuttings is anticipated to be at or close to the pavement drainage level and is therefore likely to be captured prior to evaporation occurring. Likewise, inflow into tunnels is unlikely to be lost to evaporation since seepage will be inside drainage systems within the tunnel. During construction, inflow at excavation batter sides is likely to make up a larger proportion of inflow to excavations prior to groundwater levels lowering in the surrounding area.

The impact on regional groundwater resources within the fractured bedrock aquifer is discussed in **Section 4.5**.

3.4.5.2 Groundwater levels and drawdown

The impact on groundwater levels upgradient and downgradient has been predicted using groundwater numerical models. At the cuttings and tunnels, groundwater levels will be drawn down close to the pavement level (or level of the permanent drainage system). The distance over which groundwater lowering (drawdown) occurs in the surrounding aquifer is dependent on the hydrogeological properties of the aquifer, recharge to the aquifer system and depth below the groundwater level of the cut or tunnel. An example of the predicted groundwater drawdown along the ridgeline at Gatelys Road due to construction is presented in **Figure 27**. **Appendix C1 and C2** presents the numerical modelling results at other cuts and tunnels.

Figure 28 shows the zone of drawdown around each of the cuttings and tunnels**.** This zone is based on the distance upgradient and downgradient to the 1m drawdown contour, as presented in **Table 15**. Drawdown will continue to extend beyond the 1m contour however 1m is commonly used as a value to delimit zone of impact. In comparison the NSW Aquifer Interference Policy uses 2m drawdown as the basis of an impact to groundwater supply wells. For the purposes of evaluating potential impacts, the locations of potential receptors were evaluated against the zone of drawdown as described in **Section 4.2.3**.

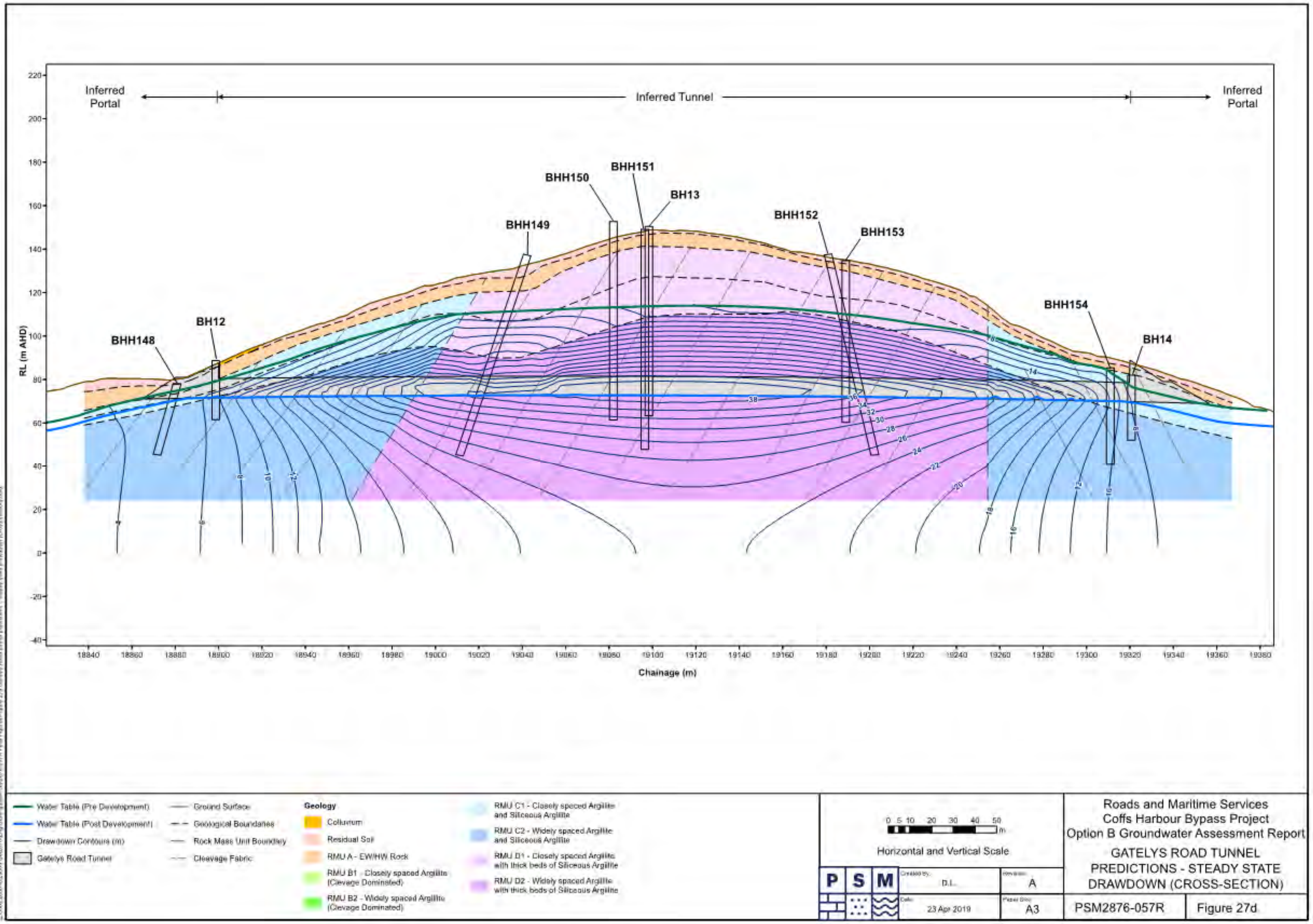
Table 15: Estimated zone of influence for modelled cuts and tunnels

Cut / Tunnel	Maximum predicted distance to upgradient 1m drawdown (m)	Maximum predicted distance to down gradient 1m drawdown (m)
Cut 4	223	50
Roberts Hill tunnels and portals	143	143
Cut 8-1	99	37
Cut 8-2	100	203
Cut 11	195	154
Cut 12	63	71
Cut 14	185	59
Shephards Lane tunnels and portals	197	197
Cut 16	114	95
Gatelys Road tunnels and portals	355	355
Cut 18 / CH20425	191	125
Cut 18r [1]	50	50
[1] Numerical modelling results from Cut 18 have been used to estimate the steady state drawdown for Cut 18r. This cut extends locally to a few metres into the water table and the resultant drawdown is predicted to be localised within the construction footprint.		

The zone of influence shown on **Figure 28** is likely to be an overestimate as it assumes that drawdown occurs uniformly across the entire length of the cutting or tunnel. The predicted drawdown however is modelled at the deepest part of the cut/tunnel and as such is likely to be a conservative approach to delimiting the potential area of impact.

The distances presented in **Table 15** are the long term steady state averages. During construction, the zone of drawdown will propagate away from the cutting and tunnels as excavation proceeds. The rate at which this develops will be dictated by the rate of excavation and the hydrogeological properties of the ground. Changes to groundwater levels, gradients and flow directions will develop over the construction period as groundwater discharges into each of the cuts eventually reaching new equilibrium steady state water levels.

Figure 27: Steady state drawdown predictions along Gatelys Road tunnel alignment



4 Assessment of potential impacts

4.1 Background

This section discusses the potential impact of the project on the groundwater environment and groundwater receptors.

4.2 Construction impacts

The main impacts to groundwater during the project construction phase are likely to be associated with groundwater ingress at excavation areas. Groundwater that is intercepted during the formation of cuts will initially drain into the excavation at a higher rate than over the longer term as groundwater pressures are decreased.

The main risks to groundwater during construction are expected to be:

- Changes to groundwater flows, surface flows and connectivity due to lowering of the groundwater level as a result of cuttings and tunnels being below groundwater level.
- Construction of large fill embankments which may concentrate runoff and recharge to groundwater systems.
- Impact to GDEs, water supply wells, agricultural dams and creeks from changes to groundwater levels and throughflow along the project.
- Groundwater contamination, which may occur during construction if construction activities are not adequately managed.
- Changes to groundwater quality due to the oxidation of acid sulfate soils and rock, caused either by exposure of due to construction excavation activity or lowering of groundwater levels.
- Changes in groundwater quality due to exposure and leaching of saline soils along the alignment.

4.2.1 Cuttings and tunnels

The project has a total of 18 mainline cuttings, seven side road and 27 access ramp cuttings. The cutting depths vary from less than 2 m to approximately 35 m with a total of eight in excess of 20 m. Additionally, three mainline drained tunnel sections are proposed to cut through ridgelines at Roberts Hill, Shephards Lane and Gatelys Road. These tunnels reach maximum depths of between 40mbgl and 80mbgl (at the ridgeline).

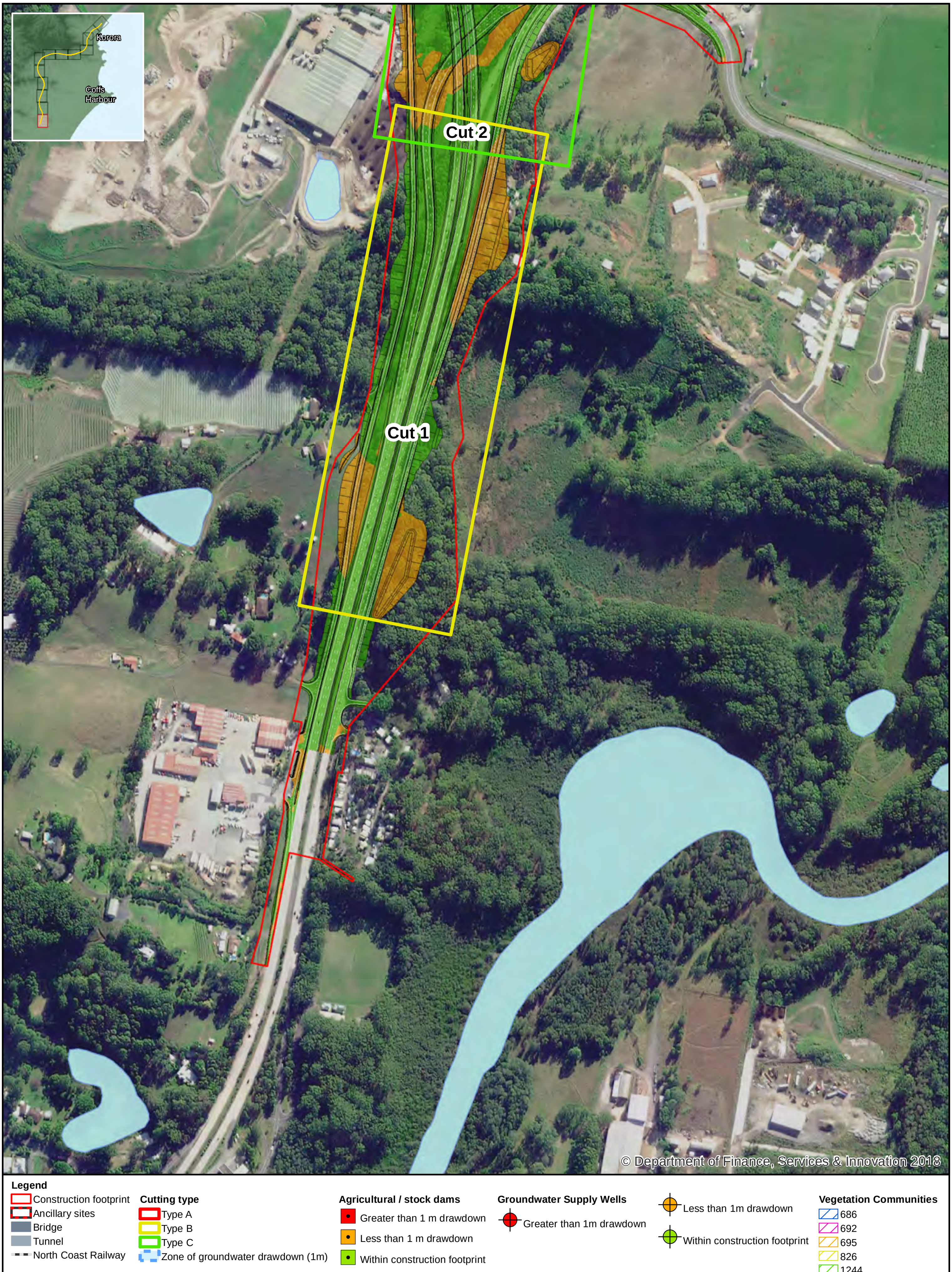
A number of these cuttings and tunnels will intersect and affect the existing groundwater flow regime (as presented in **Table 12**).

The construction of cuttings below the groundwater table will lead to lowering of groundwater levels during construction as cuttings are excavated vertically and tunnels laterally through the subsurface. Where the surface of cutting and tunnel excavations is below the water table, groundwater will seep into the excavation. Typical practice is to capture this seepage in the temporary construction drainage network, which may be reused on-site or treated and disposed of. Seepage of groundwater into excavations during construction will reduce groundwater pressures in the surrounding area leading to a lowering of groundwater levels and local reduction of groundwater throughflow.

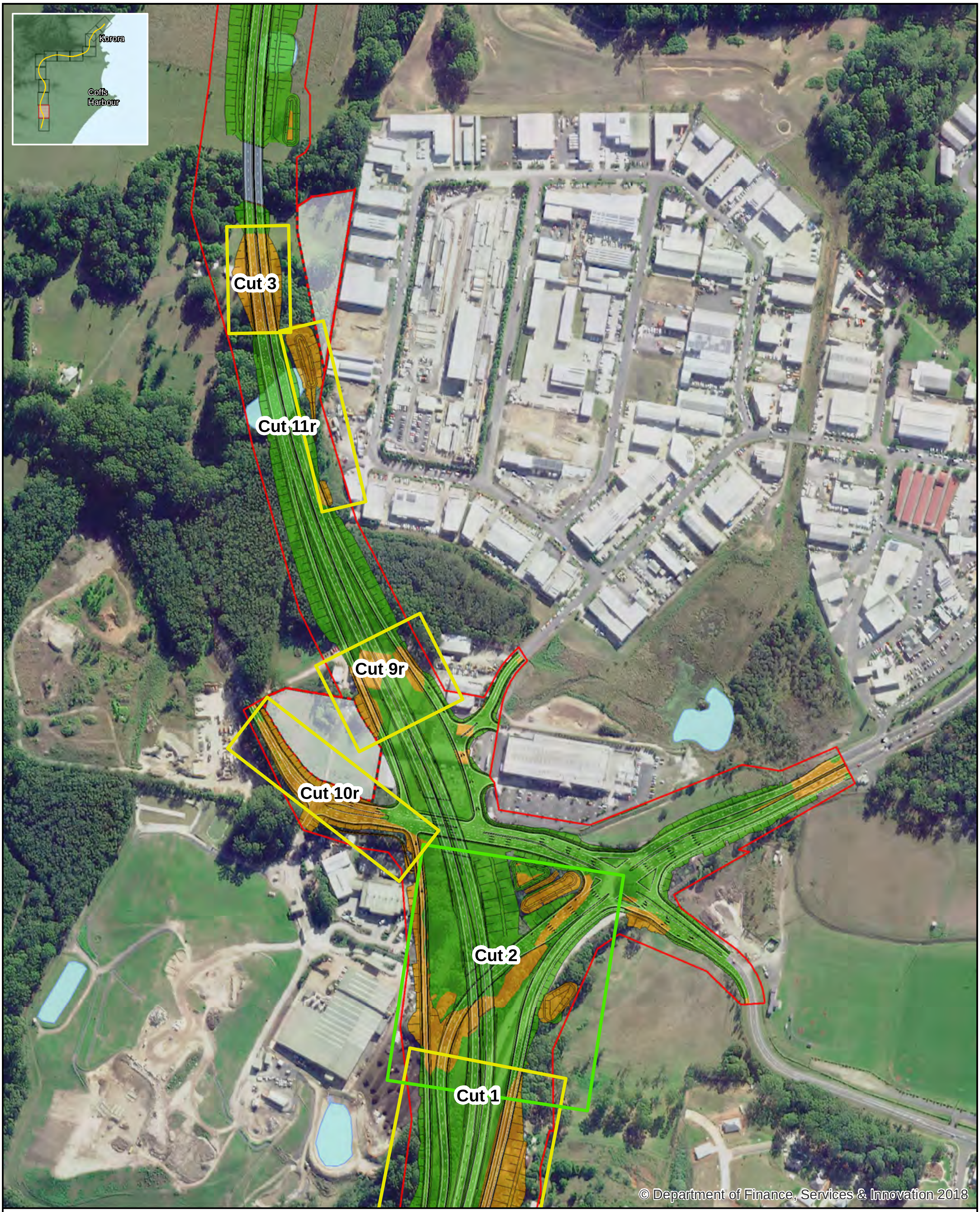
To evaluate the potential impact from cuttings and tunnels along the alignment the results of the numerical groundwater modelling were used (**Appendix C1**, **Appendix C2***)* as described in **Section 3.3 Groundwater assessment**. For the purposes of assessing the potential impacts on the groundwater environment, where multiple scenarios were modelled (i.e. using different recharge parameters), the scenario which produced the largest zone of drawdown was used at each of the cuttings and tunnels. This was not consistently the higher or lower recharge scenario due to the variability of the local hydrogeological and topographic conditions at each of the cuttings. However, using the largest zone of drawdown is a conservative assumption which aims to evaluate receptors which may potentially be affected.

As discussed in **Section 2.7**, it is anticipated that only the shallow bedrock groundwater will be impacted by the cuttings and tunnels. The deeper regional flow fields are unlikely to be affected as the cuttings and tunnels are largely restricted to topographically higher areas and ridgelines.

Potential impacts to receptors at each cutting and tunnel are discussed in **Section 4.2.3**.



Coffs Harbour Bypass
Groundwater Drawdown Extent (Long Term) and Potential Receptors
Figure 28-1



Legend

Construction footprint
Ancillary sites
Bridge
Tunnel
North Coast Railway

Cutting type
Type A
Type B
Type C
Zone of groundwater drawdown (1m)

Agricultural / stock dams
Greater than 1 m drawdown
Less than 1 m drawdown
Within construction footprint

Groundwater Supply Wells
Greater than 1m drawdown
Less than 1m drawdown
Within construction footprint

Vegetation Communities
686
692
695
826
1244

Coffs Harbour Bypass
Groundwater Drawdown Extent (Long Term) and Potential Receptors
Figure 28-2

0 0.06 0.12 0.18 km
Scale @A4: 1:6,160
GDA 1994 MGA Zone 56
N

Legend

Construction footprint
Ancillary sites
Bridge
Tunnel
North Coast Railway

Cutting type

Type A
Type B
Type C
Zone of groundwater drawdown (1m)

Agricultural / stock dams

Greater than 1 m drawdown
Less than 1 m drawdown
Within construction footprint

Groundwater Supply Wells

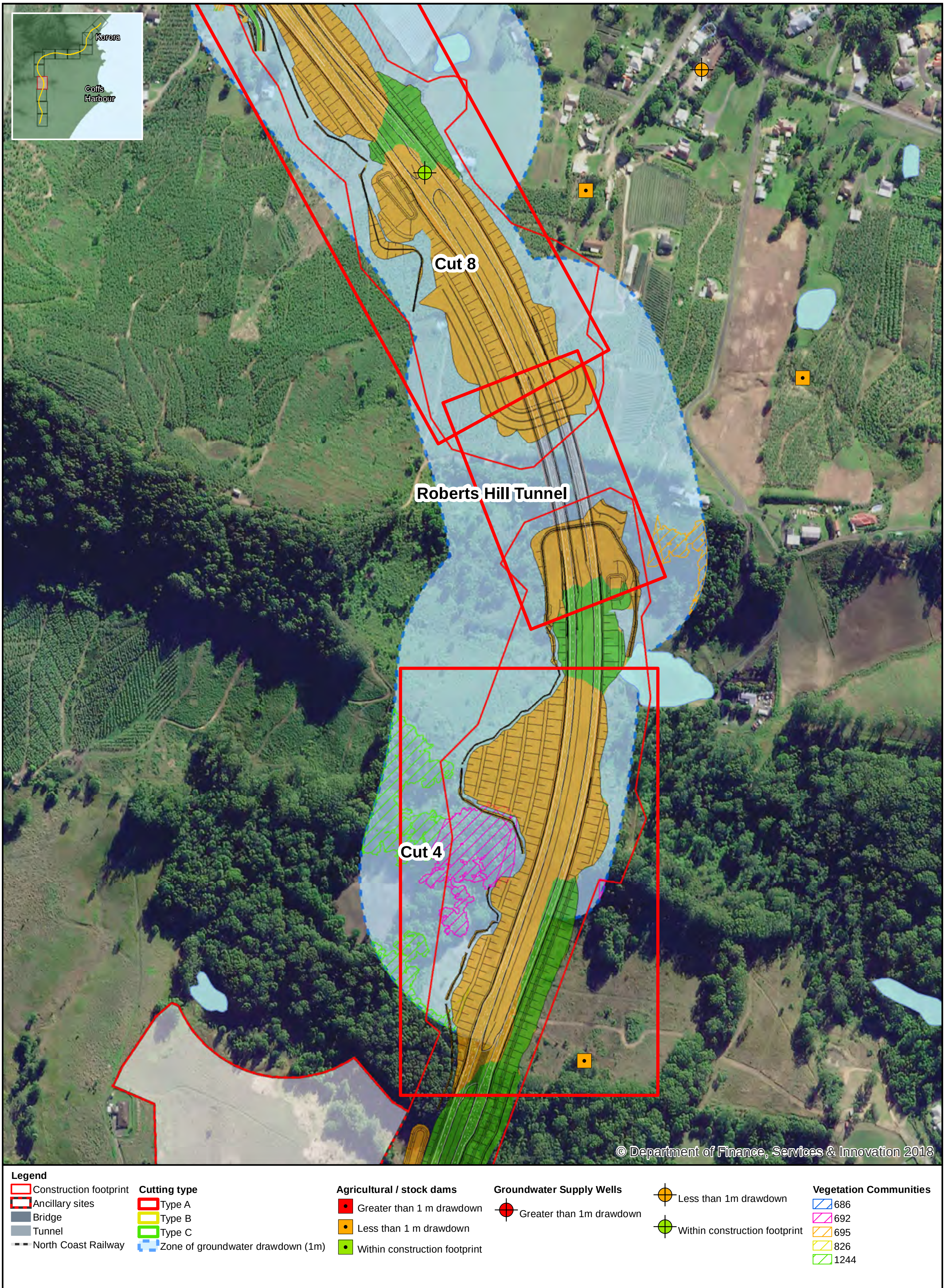
Greater than 1m drawdown
Less than 1m drawdown
Within construction footprint

Vegetation Communities

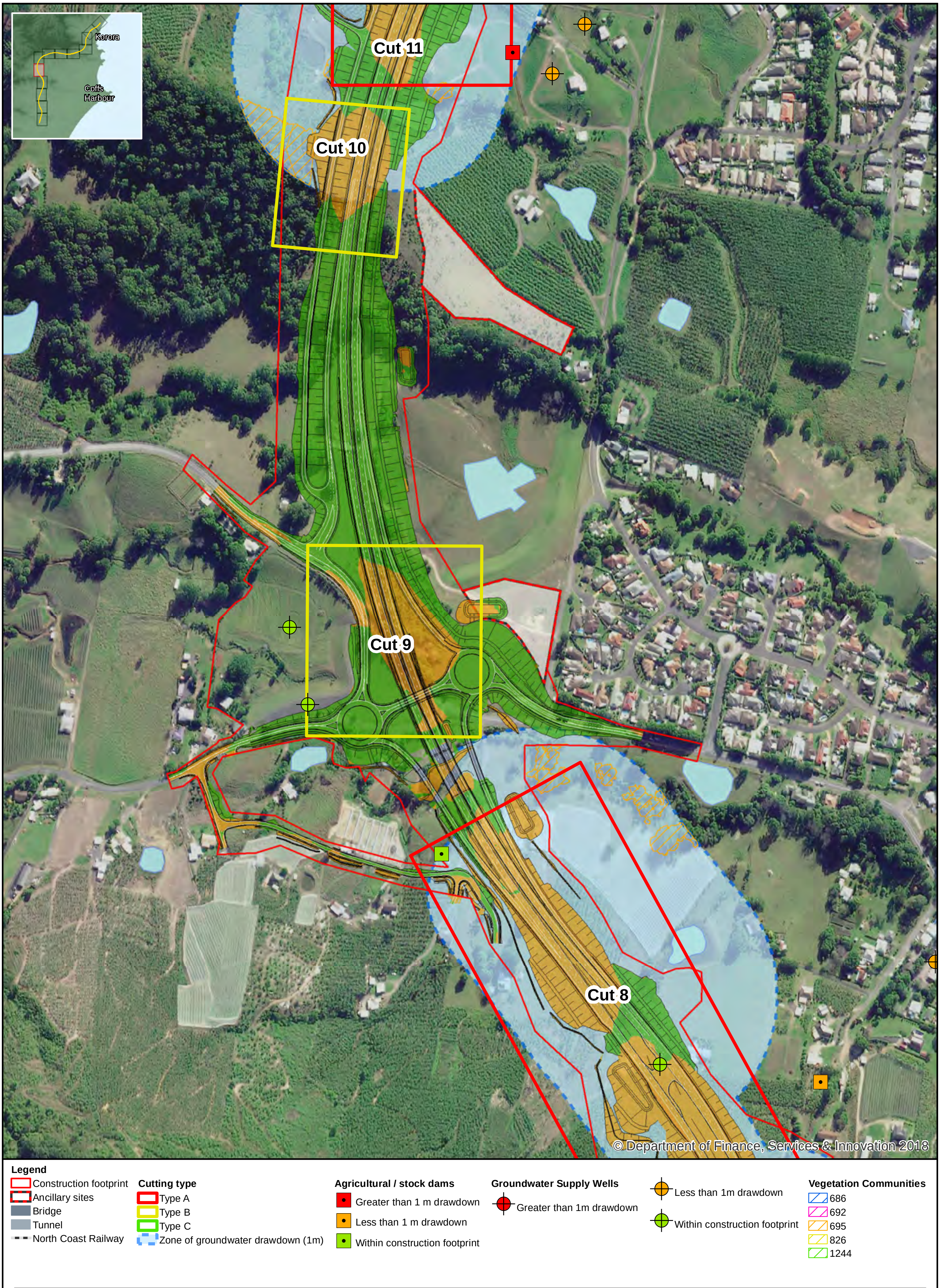
686
692
695
826
1244

Coffs Harbour Bypass
Groundwater Drawdown Extent (Long Term) and Potential Receptors
Figure 28-3

0 0.06 0.12 0.18 km
Scale @A4: 1:6,160
GDA 1994 MGA Zone 56
N

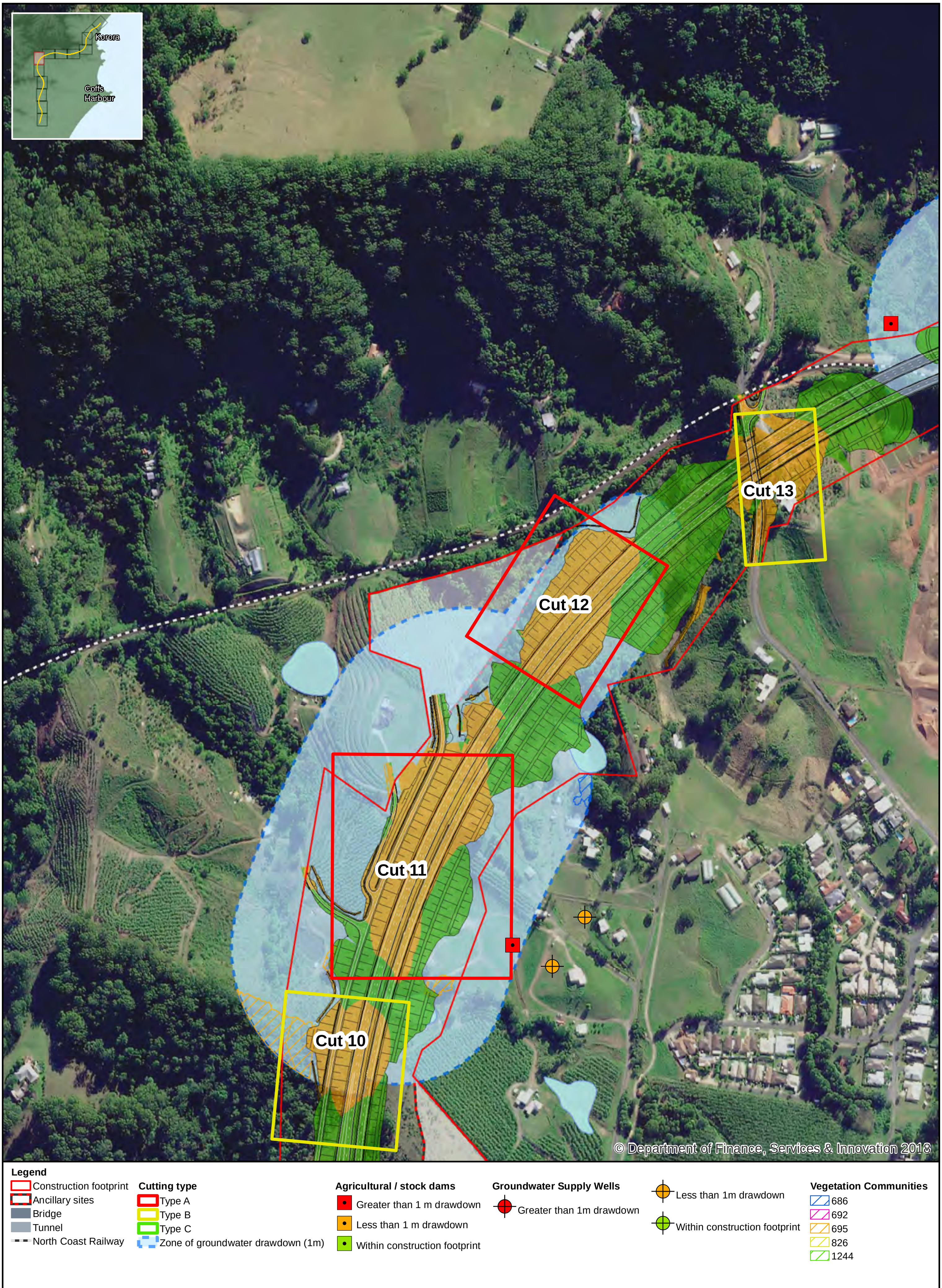


Coffs Harbour Bypass
Groundwater Drawdown Extent (Long Term) and Potential Receptors
Figure 24-4

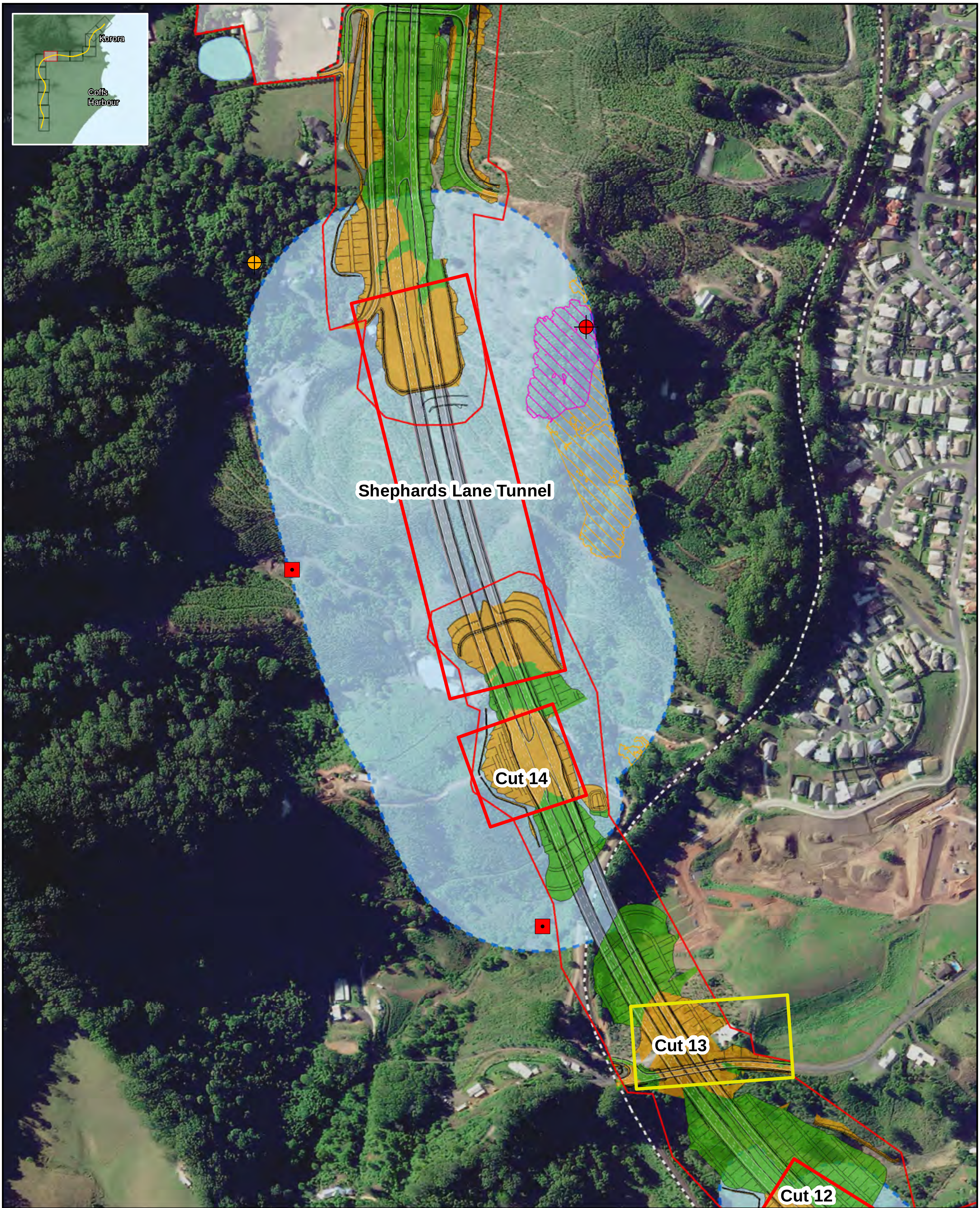


Coffs Harbour Bypass
Groundwater Drawdown Extent (Long Term) and Potential Receptors
Figure 28-5

Scale @A4: 1:6,160
GDA 1994 MGA Zone 56



Coffs Harbour Bypass
Groundwater Drawdown Extent (Long Term) and Potential Receptors
Figure 28-6



Legend

- Construction footprint
- Ancillary sites
- Bridge
- Tunnel
- North Coast Railway

Cutting type

- Type A
- Type B
- Type C
- Zone of groundwater drawdown (1m)

Agricultural / stock dams

- Greater than 1 m drawdown
- Less than 1 m drawdown
- Within construction footprint

Groundwater Supply Wells

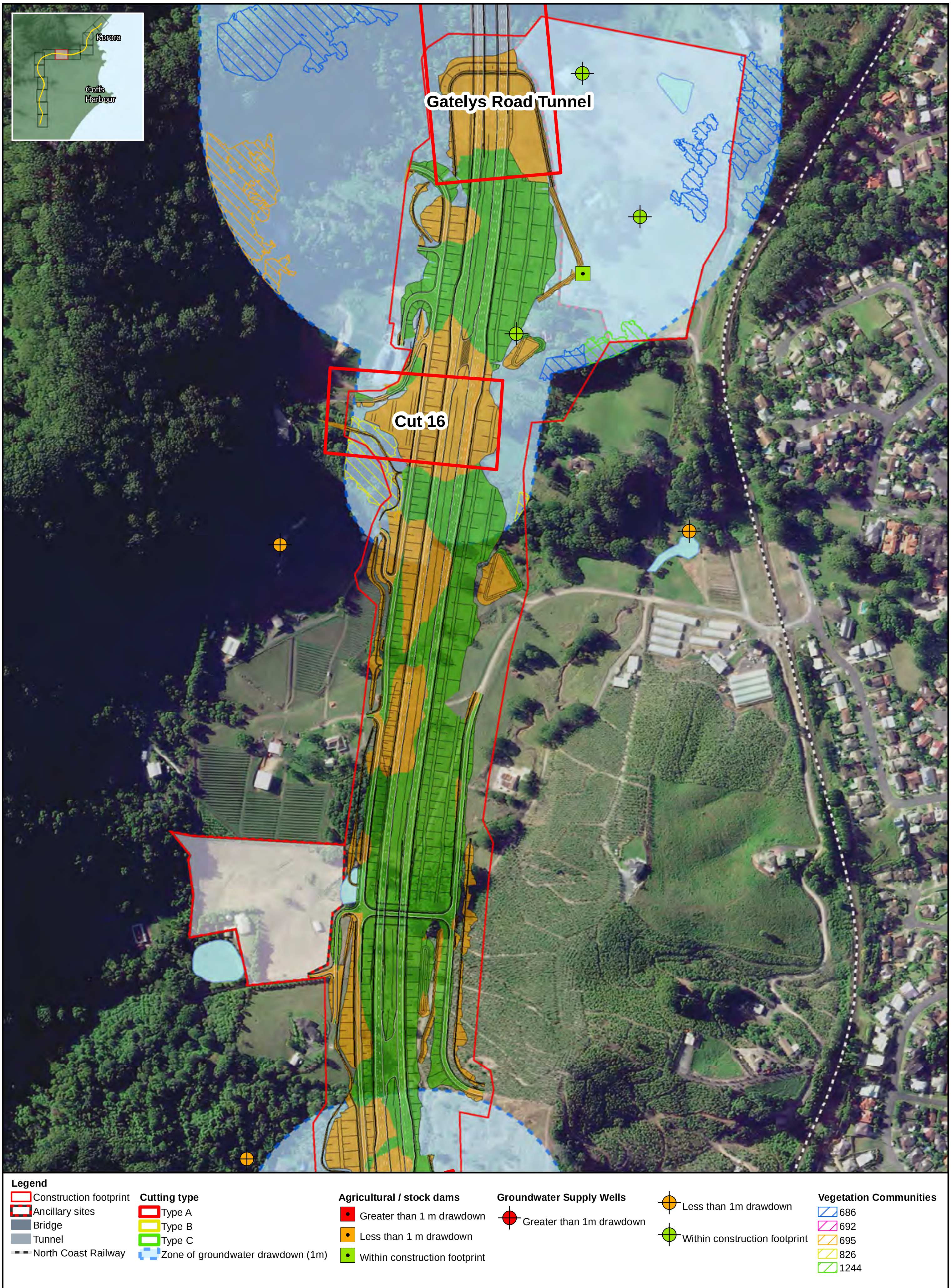
- Greater than 1m drawdown
- Less than 1m drawdown
- Within construction footprint

Vegetation Communities

- 686
- 692
- 695
- 826
- 1244

Coffs Harbour Bypass
Groundwater Drawdown Extent (Long Term) and Potential Receptors
Figure 28-7

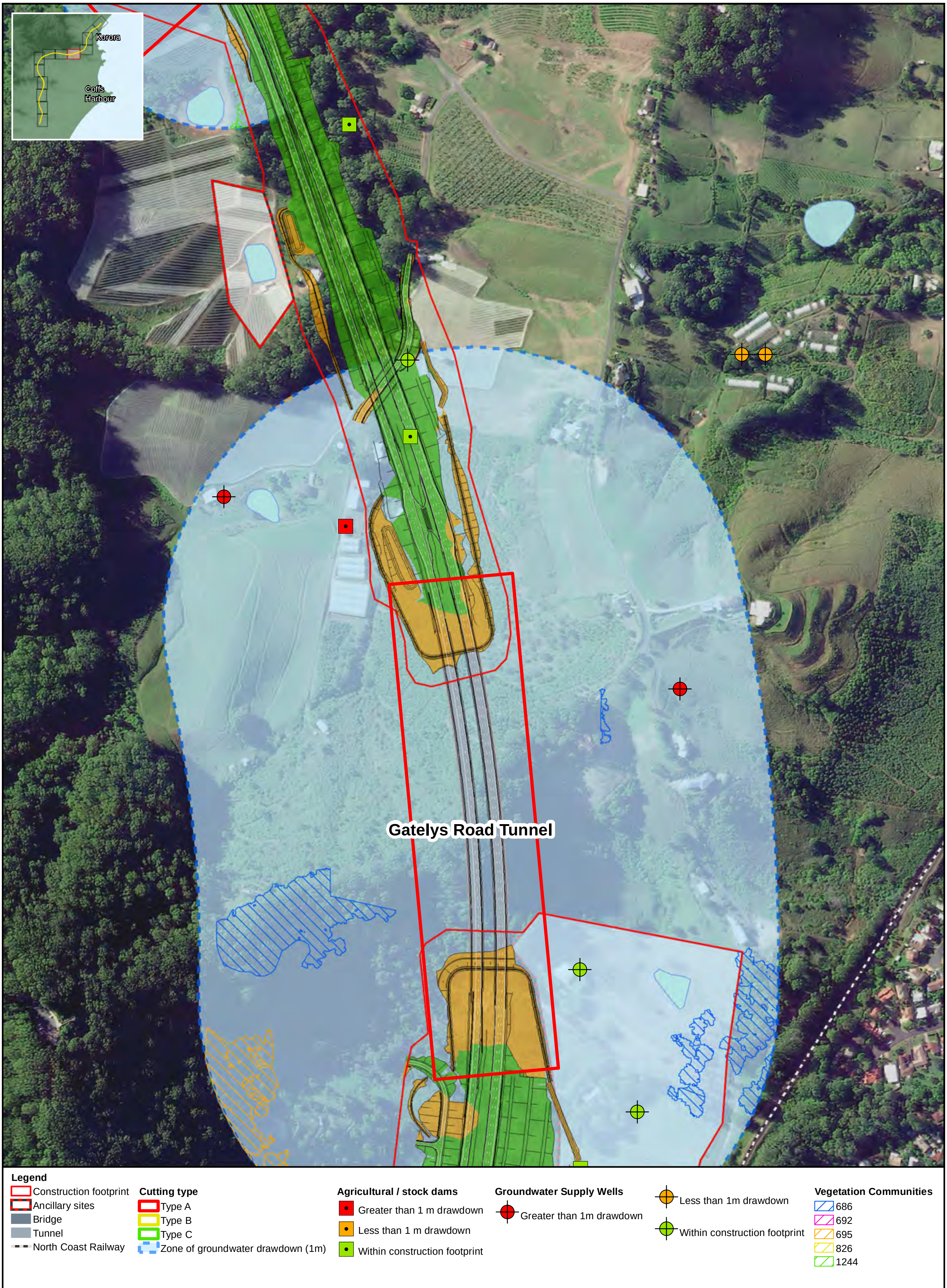
0 0.06 0.12 0.18 km
N
Scale @A4: 1:6,160
GDA 1994 MGA Zone 56



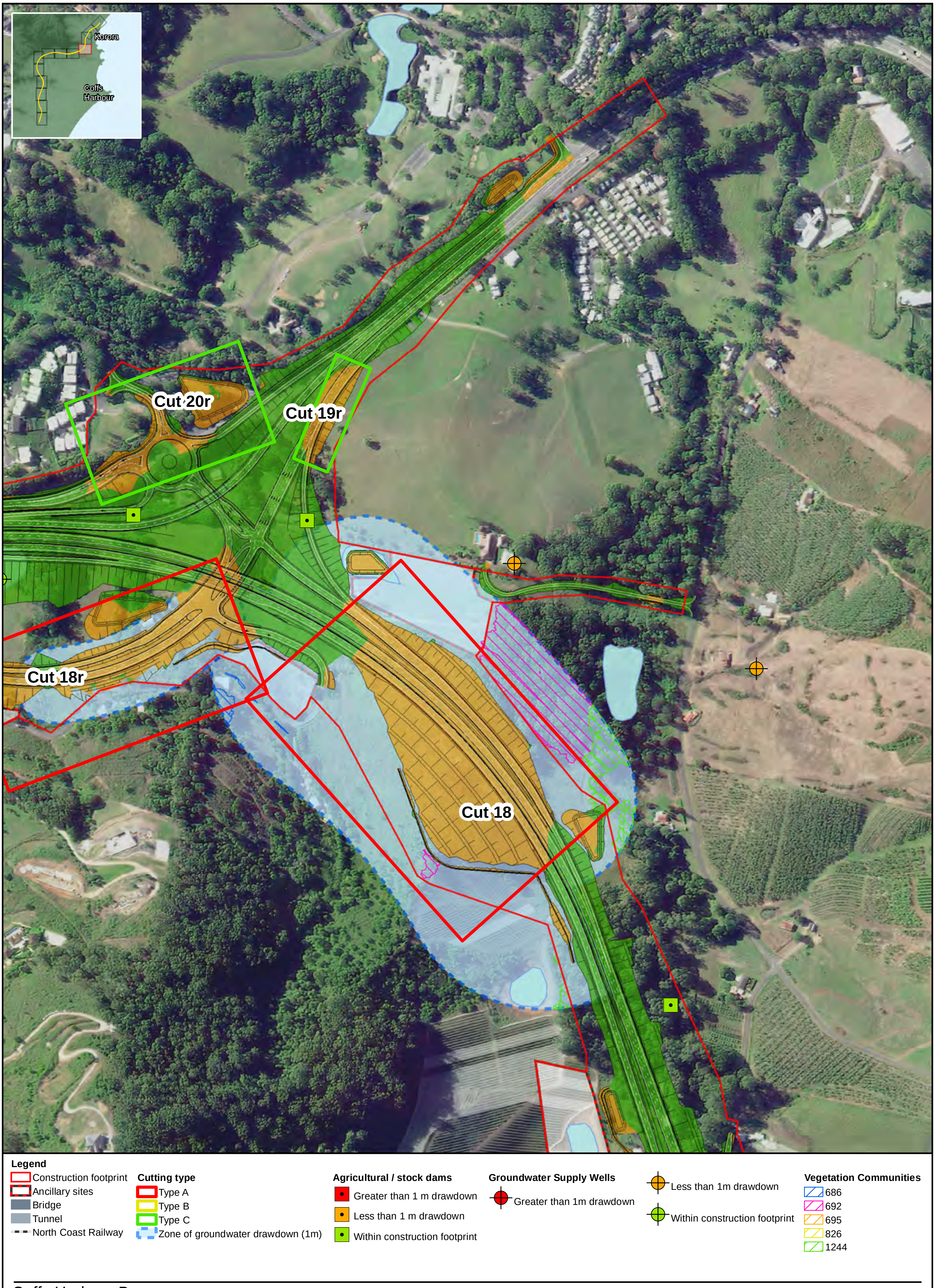
Coffs Harbour Bypass

Groundwater Drawdown Extent (Long Term) and Potential Receptors

Figure 28-8



Coffs Harbour Bypass
Groundwater Drawdown Extent (Long Term) and Potential Receptors
Figure 28-9

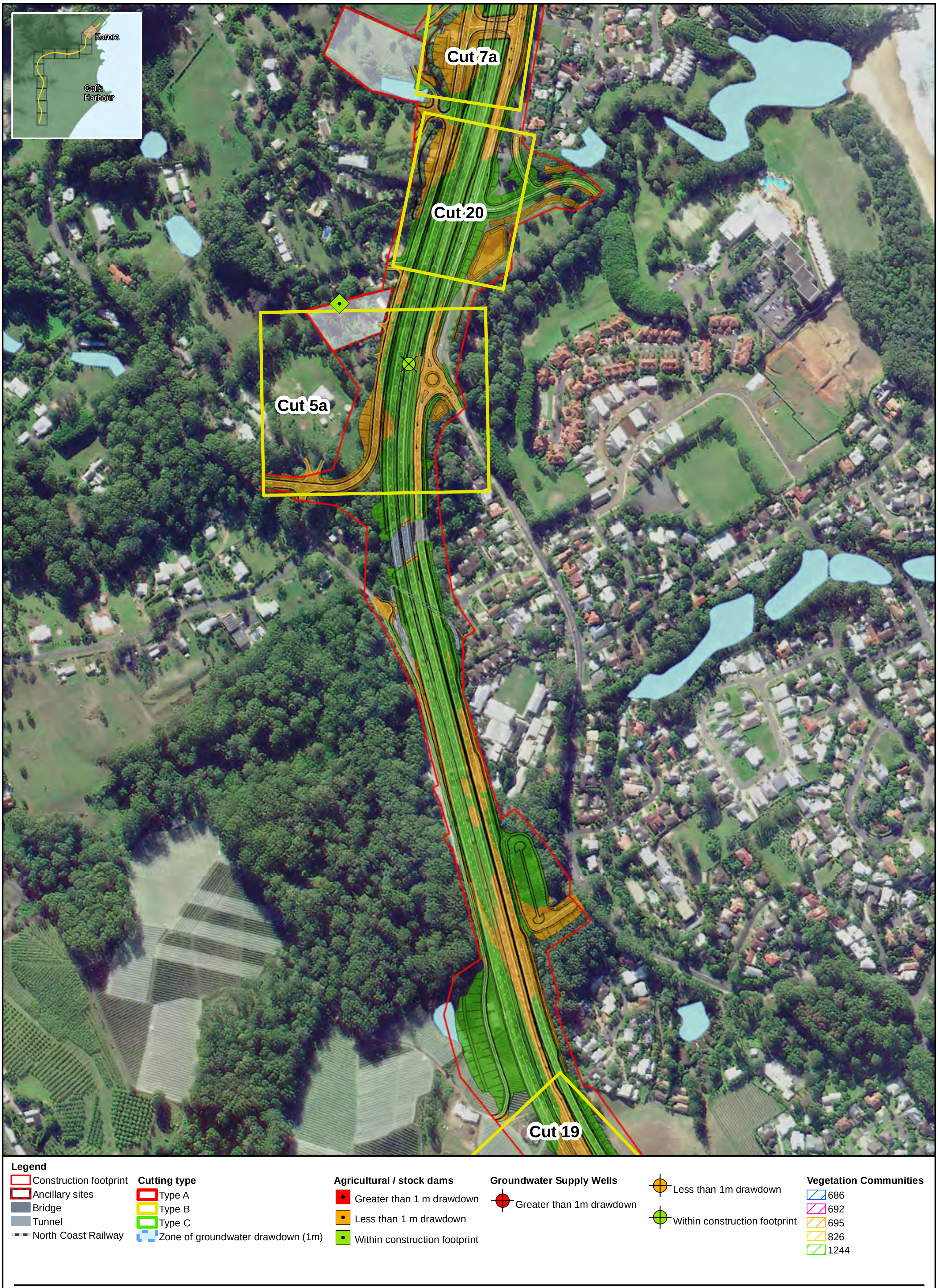


Coffs Harbour Bypass
Groundwater Drawdown Extent (Long Term) and Potential Receptors
Figure 28-10

Coffs Harbour Bypass

Groundwater Drawdown Extent (Long Term) and Potential Receptors

Figure 28-11



Coffs Harbour Bypass

Groundwater Drawdown Extent (Long Term) and Potential Receptors

Figure 28-12

Legend

Construction footprint
Ancillary sites
Bridge
Tunnel
North Coast Railway

Cutting type
Type A
Type B
Type C
Zone of groundwater drawdown (1m)

Agricultural / stock dams
Greater than 1 m drawdown
Less than 1 m drawdown
Within construction footprint

Groundwater Supply Wells
Greater than 1m drawdown
Less than 1m drawdown
Within construction footprint

Vegetation Communities
686
692
695
826
1244

Coffs Harbour Bypass
Groundwater Drawdown Extent (Long Term) and Potential Receptors
Figure 28-13

0 0.06 0.12 0.18 km
Scale @A4: 1:6,160
GDA 1994 MGA Zone 56
N

4.2.2 Embankments

The preparation of fill foundations will include compaction of the surficial materials that may comprise suitable existing in-situ material or imported engineered fill to replace unsuitable in-situ material. This foundation preparation is likely to create areas of lower permeability relative to the existing subsurface, which may reduce the infiltration of surface runoff to soils and surficial deposits and subsequently to underlying aquifers. Areas of fill may also cause temporary ponding upgradient of the embankment, and temporary drying down gradient of the embankment, particularly where groundwater is within a few metres of the surface.

Fill embankments vary in maximum height from between 2 m and 30 m with total lengths ranging from 20 m to around 750 m. The location of areas of fill embankments along the alignment is presented in **Figure 23**. Embankment fill is present across most of the alignment to some degree however areas of substantial fill include;

- Englands Road interchange
- Between Englands Road interchange and Roberts Hill tunnel along the floodplain of Newports Creek
- Coramba Road interchange
- Between Shephards Lane tunnel and Gatelys Road tunnel infilling existing drainage lines which bisect the alignment
- North of Gatelys Road tunnel
- Korora Hill interchange

The impact from fill embankments is expected to be greatest around the largest embankments and where groundwater levels are closest to the surface. The impact on groundwater from embankments will be reduced by the preparation of suitable temporary drainage systems to prevent ponding because of rainfall events during construction.

Permanent systems including drainage layers at the base of embankments and adequate surface drainage to prevent the build-up of water behind them will also be included to mitigate the effects following construction. Additional surface runoff (and reduced recharge) caused by construction of the embankments is unlikely to have a large impact on the groundwater flow system due to the localised nature of the embankments compared to the catchment of the fractured bedrock groundwater system.

4.2.3 Potential impacts to receptors

Changes in groundwater levels and flows due to the construction and operation of the project has the potential to impact a variety of receptors. These include environmental receptors such as GDEs and groundwater users such as groundwater supply wells and agricultural dams (where the source of water is from spring flow). Changes to the hydrogeological environment may impact on receptors through a variety of mechanisms which include lowered groundwater levels, reduced groundwater throughflow, reduced discharge at surface (at springs) or changes to recharge or surface run off.

The greatest impact on the groundwater environment is anticipated where deep cuttings and tunnels extend below the groundwater table (i.e. Type A cuttings and tunnels, as described previously). **Table 16** provides a summary of those receptors and constraints (such as PASS) which are located within the zone of 1m predicted drawdown along the alignment. **Figure 28** shows the location of each of these potentially affected receptors. The potential impact at receptors will progressively diminish with distance away from the cuttings of tunnels as the drawdown decreases from a maximum close to the cutting or tunnel, to zero some distance away. For this reason, the figure also shows supply wells and agricultural dams outside of the 1m drawdown contour, because drawdown continues to extend beyond this zone (less than 1m drawdown).

The extent of impact on receptors included in the table below is likely to vary owing to local hydrogeological conditions and level of connection with the fractured bedrock aquifer, which is where drawdown and changes to flow will be experienced. It is likely that those receptors which are reliant on groundwater from other hydrogeological units (such as the alluvial aquifer, or perched water within the residual soils) will experience limited impact due to the drawdown in the fractured bedrock aquifer. The impacts at each of the potential receptors are described in further detail in sections below.

Table 16: Assessment of drawdown from cuttings and potential receptors located in areas predicted to exceed 1m of groundwater drawdown

Cutting/ Tunnel	**Geology/ Aquifer**	**Predicted distance to up gradient / down gradient 1m drawdown (m)**	**Potential receptors and constraints**							
			GDEs (from regional studies	**Mapped vegetation community**	**Alluvial aquifers**	**Creeks**	**Groundwater supply wells**	**Agricultural Dams**	**Lakes**	**PASS**
Cut 4	FB	223 / 50	Terrestrial, low potential	PCT 692 PCT 1244	Yes	-	0	0	1 [1]	>500m
Roberts Hill tunnels and portals	FB	143 / 143	Terrestrial, low potential	PCT 695	-	-	0	0	0	>500m
Cut 8-1	FB	99 / 37	-	-	-	-	0	0	0	>500m
Cut 8-2	FB	100 / 203	-	PCT 695	Yes	Coffs Creek	0	1	1	>500m
Cut 11	FB/A	195 / 154	Terrestrial, low potential	PCT 695 PCT 686	Yes	-	0	1	1	>500m
Cut 12	FB	63 / 53	-	PCT 686	-	-	0	0	0	>500m
Cut 14	FB	185 / 142	Terrestrial, low potential	PCT 695	Yes	-	0	1	0	>500m
Shephards Lane tunnels and portals	FB/A	197 / 197	Terrestrial, low potential	PCT 695 PCT 692	Yes	-	GW303812	1	0	>500m
Cut 16	FB	115 / 95	Terrestrial, low potential	PCT826 PCT686	Yes	-	GW301578 [2]	0	0	>500m
Gatelys Road tunnels and portals	FB	355 / 355	Terrestrial, low potential	PCT 695 PCT 686 PCT 692	Yes	Jordans Creek	GW306794 [2] GW072693 [2] GW068986 GW063664 GW302679 [2]	3	2	>500m

Cutting/ Tunnel	Geology/ Aquifer	Predicted distance to up gradient / down gradient 1m drawdown (m)	Potential receptors and constraints							
			GDEs (from regional studies	Mapped vegetation community	Alluvial aquifers	Creeks	Groundwater supply wells	Agricultural Dams	Lakes	PASS
Cut 18	FB	191 / 121	Terrestrial, low potential	PCT 692 PCT 1244	Yes	Jordans Creek	0	0	2	>500m
Cut 18r	FB	50 / 50	-	-	-		0	0	0	>500m

Notes:

FB – fractured rock aquifer A – Alluvial aquifer

[1] located between cut 4 and Gatelys Road

- Geology / aquifer – the geology or aquifer in which the cutting or tunnel is directly constructed within or through. All cuttings and tunnels are constructed within the fractured bedrock aquifer however at a few locations minor areas of alluvial deposits may also be affected, at the upper parts of creek lines.
- Distance to 1m drawdown contour – based on long term steady state modelling predictions (**Table 15**),
- GDEs – based on regionally mapped data from the BoM GDE Atlas,
- Mapped native vegetation communities – based on field mapping undertaken within the study area,
- Alluvial aquifer – from published geological mapping where alluvial material is located within the zone of drawdown. Groundwater drawdown is within the fractured bedrock aquifer and as such the extent of impact on alluvial aquifers will be dependent on the connectivity between the two aquifers,
- Creeks – Creek lines that are located within the zone of drawdown,
- Groundwater supply wells – location of groundwater supply wells within the 1m drawdown contour based on the NGIS groundwater database,
- Agricultural dams – agricultural or stock dams that are located within the zone of drawdown, based on NSW hydrographic mapping data,
- Lakes – lakes or surface water bodies that are located within the zone of drawdown, based on NSW hydrographic mapping data,
- PASS – although not specifically a receptor, this is included as a constraint to highlight where PASS could potentially be affected by groundwater drawdown and potentially present a risk to the groundwater environment.

4.2.3.1 Groundwater dependent ecosystems

GDEs may be affected by lowering of groundwater levels caused by the excavation of cuttings and tunnels which intercept and drain groundwater from the fractured bedrock aquifer. Most GDEs within the study area is likely to draw groundwater from shallow surficial deposits or alluvial groundwater which is within a few metres of the surface. GDEs are unlikely to be dependent directly on groundwater from the fractured bedrock aquifer except where it is close to the ground surface, for instance at spring locations.

Where seepage occurs at excavations during construction, water will be captured and redirected to temporary construction sediment basins. GDEs have the potential to be impacted if this seepage is diverted away from a downstream GDE that is reliant on it. Vegetation supported by groundwater could also be affected if there is a significant reduction in water levels, where lowered from close to the ground surface. Changes to surface water run-off may also locally affect GDEs due to changing distribution of recharge to surficial deposits and flows to alluvial aquifers and creek lines.

A review of the location of potential GDEs and mapped native vegetation communities within the zone of potential drawdown from cuttings and tunnels. **Table 16** provides a summary of the potential GDEs from regional studies and mapped vegetation communities which may be located within the zone of drawdown from cuttings and drained tunnels. These are also presented in **Figure 28.**

The alignment intercepts several low potential GDEs and native vegetation communities, which may be intermittently groundwater dependent. The anticipated zone of drawdown from Type A cuttings and tunnels also extends to some low potential GDEs outside of the project boundary. No moderate or high potential GDEs are anticipated to be within the zone of drawdown. There are no mapped Coastal Management SEPP wetlands within the expected long-term zone of drawdown around any of the cuttings or drained tunnels.

There are several native vegetation communities which are present within the zone of drawdown. These comprise of vegetation communities occurring on creek lines which may be reliant on shallow groundwater within alluvium and more broadly those which may draw water from local perched systems and soils. As previously discussed, changes to groundwater flow and levels will predominantly occur within the fractured bedrock aquifer system. The effect on perched groundwater systems and alluvial aquifers is anticipated to be small as these systems are reliant on surface water runoff and local recharge, rather than connection with the fractured bedrock aquifer (even if there is connection, the contribution of flow from the fractured bedrock is small). Across most the alignment, groundwater levels in the fractured bedrock aquifer are deep There is unlikely to be an impact on the native vegetation communities as a result of drawdown in the fractured bedrock aquifer.

The native vegetation communities anticipated to be within the zone of drawdown from Type A cuttings and tunnels predominantly comprise of sclerophyll forest including:

- Blackbutt – Tallowwood (PCT 692), Turpentine (PCT 695) and Pink Bloodwood (PCT 686)
- Sydney Blue Gum (PCT 1244)
- Flooded Gum (PCT 826)

Since groundwater inflows captured by the project are from the fractured bedrock aquifer, the potential impact on GDEs and native vegetation communities is expected to be limited. Where native vegetation communities are groundwater dependent, it is likely that they are reliant on water within alluvial aquifers (and perched water within surficial soils), which are predominantly surface water dependent. Groundwater from the fractured bedrock has a low impact on creek instream values and flow into alluvial aquifers, and as such is only likely to have an impact where surface discharge occurs, say at spring locations which is discussed further below.

4.2.3.2 Groundwater supply wells (water works)

A review of the DPIE (Water) groundwater bores database from the NGIS was undertaken to evaluate those which might be impacted by groundwater drawdown caused by cuttings and tunnels (**Figure 28**) and those within the construction footprint which will need to be acquired. At the time of land acquisition, RMS also acquire the water access licence associated with the bores and subsequently become the owner of the licence.

Table 17 lists the supply wells which are located within zone of influence of cuttings and tunnels (including where drawdown is less than 1m) and those within the construction footprint. The breakdown of supply wells affected is as follows:

- 10 supply wells located within the construction footprint (5 of which are also within the zone of groundwater drawdown) which are not considered further as they will be removed during construction
- 12 supply wells located within the anticipated zone of groundwater drawdown. Of these
  - 8 are expected to have a drawdown of less than 1m,
  - 3 are expected to have a drawdown of between 1m and 2m
  - 1 is expected to have a drawdown of around 4.3m, in excess of the Aquifer Interference Policy Minimal requirements

All of the wells predicted to be impacted by groundwater drawdown are installed within the less productive fractured bedrock aquifer. No groundwater well sources in alluvial aquifers within the study area are anticipated to be affected by groundwater drawdown.

The *NSW Aquifer Interference Policy* states that the minimal impact consideration for aquifer interference is a cumulative pressure head decline of not more than two metres at any water supply works. This assessment indicates that a total of four supply wells could be affected by more than the minimal impact consideration all of which are near to Gatelys Road and Cut 16. Three of the wells however are located within the construction footprint of the project and are not considered further since they are expected to be acquired as part of land acquisition.

The well outside of the construction footprint anticipated to be affected by more than the minimal impact consideration is GW068986, which is predicted to have a drawdown of approximately 4.3m.

Information relating to GW068986 indicates that it is used for domestic water supply and was drilled in 1991 to a depth of 27mbgl. One recorded groundwater level reading was measured at 9mbgl. The operational status of this well will be confirmed prior to construction during the detailed design phase of the project.

Where a landholder bore is destroyed or impacted beyond the minimal impact criteria as set out in the aquifer interference policy, make good measures will be considered in negotiation with the bore owner. These measures are discussed further in **Section 4.4**.

Table 17: Groundwater borehole sources located within zone of influence of drawdown or within construction footprint

ID	Closest cut/tunnel	Easting	Northing	Purpose	Well Depth (m)	Groundwater level (mAHD)	Yield (l/s)	Distance to mainline (m)	Anticipated drawdown (m)	Notes
GW305778	N/A	507775	6646906	Stock/domestic	N/A	N/A	N/A	0m	N/A	Well within construction footprint
GW300335	Cut 8-1	508045	6648924	Commercial and industrial	72	36	0.38	0m	N/A	Well within construction footprint
GW300100	Cut 8-2	508408	6649059	Household supply	35	N/A	N/A	320m NE	<1m	
GW053093	Cut 9	307557	6649499	Stock/irrigation/domestic	7	N/A	0.65	150m W	N/A	Well within construction footprint
GW303298	Cut 11	507903	6650229	Household supply	61	29	0.50	190m SE	<1m	
GW304429	Cut 11	507946	6650294	Household supply	18	24	0.63	210m SE	<1m	
GW303812	Shephards Lane tunnel	509106	6651018	Household supply	67	126	0.38	200m S	1.3m	
GW304578	Shephards Lane tunnel	509187	6651438	Household supply	63	90	5.0	185m NE	<1m	
GW304148	Cut 16	509987	6651394	Domestic	61	95	0.51	195m N	<1m	
GW303960	Cut 16	510004	6650861	Domestic	67	23	1.01	300m S	<1m	
GW301578	Cut 16	510262	6651087	Stock/domestic	42	50	0.25	60m S	2.5m	Well within construction footprint
GW306794	Gatelys Road tunnel	510413	6650924	Household supply	60	N/A	N/A	190mS	2.5m	Well within construction footprint
GW072693	Gatelys Road tunnel	510601	6651001	Domestic	73	52	0.06	110m S	3.9m	Well within construction footprint
GW068986	Gatelys Road tunnel	510968	6650870	Domestic	27	118	0.38	270m S	4.3m	Drawdown anticipated to exceed minimal requirements of AIP
GW063664	Gatelys Road tunnel	511217	6651465	Domestic	45	N/A	N/A	230m NW	1.3m	
GW302679	Gatelys Road tunnel	511398	6651226	Domestic	36	44	1.9	35m SE	1.1m	Well within construction footprint

ID	Closest cut/tunnel	Easting	Northing	Purpose	Well Depth (m)	Groundwater level (mAHD)	Yield (l/s)	Distance to mainline (m)	Anticipated drawdown (m)	Notes
GW056123	Gatelys Road tunnel	511407	6650759	Domestic	32	N/A	N/A	430m S	<1m	
GW068230	Cut 18	512291	6651509	Unknown	52	N/A	N/A	210m SE	<1m	
GW072728	Cut 18	512150	6651189	Domestic	19	N/A	N/A	330m S	<1m	
GW068806	Cut 18r	512270	6652191	Stock/domestic	31	25	0.10	0m	<1m	Well within construction footprint
GW059711	Cut 5a	512931	6653496	Domestic	24	N/A	0.13	0m	N/A	Well within construction footprint
GW064174	Cut 21	513466	6653926	Domestic	30	NA	0.30	60m NW	N/A	Well within construction footprint
Notes										

4.2.3.3 Alluvial aquifers

Table 16 describes where alluvial deposits are located within the anticipated zone of drawdown of each cutting. The location of mapped alluvial deposits in relation to the project is presented in **Figure 3**. The assessment indicates that some alluvial deposits are located within the zone of drawdown in the fractured bedrock aquifer at most Type A cuttings and tunnels. Where drawdown in the fractured rock aquifer occurs below overlying alluvial material, there is a small chance that groundwater levels in the alluvial aquifer may also be impacted due to hydraulic connectivity between the units. This impact will be dependent on the water levels in the two aquifers, the degree of aquifer connection and differences between the hydraulic characteristics of the aquifers.

The connectivity between fractured rock aquifers and alluvial aquifers within the study area is understood to be limited based on information presented in water sharing plans for the region; groundwater flow within the fractured bedrock has a low impact on alluvial and creek flow (*DoW, 2009, DPI, 2016*). The amount of water transmitted by the fractured bedrock to the alluvium aquifer is small in comparison to the contribution from surface runoff due to the low permeability and limited storage of the aquifer system.

Changes to water levels in the fractured bedrock in the surrounding aquifer due to construction of the tunnels and cuttings could potentially promote vertical drainage from the alluvial aquifer into the underlying aquifer. This will only occur where the relative water levels between the units become lower in the fractured bedrock than in the alluvial deposits due to drawdown (i.e. the alluvial groundwater becomes losing to the fractured bedrock). The extent of any potential drawdown impact in the alluvium will be dependent on the rate of water loss into the fractured bedrock (determined by the gradient between the units and the hydraulic conductivity of units). If the rate of water loss from the alluvium is matched by rate of recharge (say from creek flow) then the drawdown impact in the alluvium will be negligible.

Generally, drawdown impacts do not extend significantly into areas of mapped alluvial deposits. Where it does, it is noted that the elevation of the alluvial deposits are generally lower than the design RL of the cutting or tunnel causing drawdown and is considered unlikely that the water level drawdown in the fractured bedrock will lead to substantial vertical gradients between the alluvium and bedrock. Furthermore, given the low permeability of the bedrock, the volume of water loss from the alluvium is likely to be comparatively small compared to the flow within the alluvium and connected creek.

Changes to surface water runoff due to construction of cuttings, paved surfaces and embankments may locally affect recharge to alluvial aquifers. These impacts would be managed by implementation of appropriate drainage measures during detailed design.

4.2.3.4 Creeks and wetlands

Creeks located near to major cuts or tunnels could be affected by changes in groundwater flow, water levels or surface water runoff. Cuttings located near to creek lines may cause lowering of groundwater levels below the creek line (within the bedrock aquifer). The assessment indicates that Coffs Creek and Jordans Creek are those which are most likely to be affected due to their proximity to cutting 8 and Gatelys Road tunnel respectively.

The extent of impact, as with the alluvial aquifers, is likely to be small. The connectivity between the alluvial and fractured bedrock aquifer is likely to be low, and the changes to water levels within the fractured bedrock below the creek lines is unlikely to be great or extend over a substantial area of creek line. The quantity of flow within the alluvial aquifer and creek lines is expected to be substantially higher than the quantity that may move into the bedrock aquifer as a result of lowered groundwater levels.

Changes to the bedrock groundwater flow system therefore are not anticipated to have a large impact on the creek flows. Changes to the emergence of spring flows due to groundwater drawdown may locally affect creek flow volumes. However, spring flow is likely to occur during wetter periods (i.e. following sustained rainfall) when creek flow is also likely to be highest due to increased surface runoff. The volume of water discharging from springs is therefore unlikely to be a significant contributor to creek flows, and the impact is likely to be limited.

Additionally, the impact at creeks along the alignment is likely to be limited since cuttings and tunnels are located at the upper reaches of the creek lines. The catchment of each creek increases significantly to the east of the alignment. As a result, the reduction in groundwater throughput in the fractured bedrock caused by the cuttings is likely to represent a very small fraction of the total flow supplied to the creeks compared to that from surface water runoff and alluvial aquifer baseflow. The Coastal Management SEPP wetlands associated with Boambee Creek and Newports Creek are not anticipated to be affected by changes to the groundwater system because of the project. The nearest cutting with the potential to lower groundwater levels is located approximately 1km from the wetlands. The impacts on these wetlands are therefore anticipated to be negligible.

Changes to surface water runoff due to construction of cuttings, paved surfaces and embankments may locally affect runoff and creek discharges. These impacts will be managed by design and implementation of appropriate drainage measures during the detailed design phase of the project. Adequate road drainage and discharge of captured groundwater and surface water downgradient (within the same catchment/creek) will reduce the impact on creeks and water bodies by replacing captured throughput back into the groundwater environment.

4.2.3.5 Agricultural dams and lakes

There are seven mapped agricultural dams which could potentially be affected by changes in groundwater levels caused by the project (located within the 1 m drawdown contour). Three of these locations are located within the construction footprint area, along with a further seven locations across the study area (**Table**

18). Several other agricultural dam locations are located downgradient of the proposed tunnels and cuttings highlighted on **Figure 28**. These are located outside of the 1m drawdown contour but may potentially be affected due to reduction in throughput or changes to spring emergence upgradient. Those dams which are spring fed from the fractured bedrock aquifer are likely to be most at risk of impact from changes in the groundwater environment as a result of construction and long-term changes to groundwater levels. Local changes to surface water flows may also affect nearby dams.

Examples of how spring fed dams and lakes / surface water bodies might function along the alignment are shown conceptually at Roberts Hill, Shephards Lane and Gatelys Road in **Figure 29**, **Figure 30** and **Figure 31** respectively. **Figure 31** at Gatelys Road shows an example of a pond/dam which may be fed by, or in connection with the underlying groundwater in the fractured bedrock.

The surface water may be due to discharge to the surface from the fractured bedrock at springs or as a direct connection with the underlying fractured bedrock groundwater table. **Figure 30** at Shephards Lane is an example of a pond which could be in connection with perched groundwater water well above the groundwater level in the fractured bedrock. Although these features could be reliant on perched groundwater, they may also be reliant on surface water run-off.

Changes to groundwater levels or through flow down gradient of drained tunnels and cuttings could have a direct impact on those agricultural dams or lakes which are partially reliant on the underlying groundwater. There is currently no direct information relating to the exact source of water for those agricultural dams or lakes highlighted in **Table 18**, which means that it is not currently possible to accurately predict the actual impact at these locations. Due to the complexity of the local hydrogeological regime and the limited amount of local information at each site, it is likely that some of the agricultural dams and lakes area be reliant on multiple sources of water for supply, with spring discharge or direct connection with the fractured bedrock likely making up some contribution along with surface run-off (but not necessarily at every location). For the purposes of the assessment, a conservative assumption is made that agricultural dams within the zone of drawdown of the cuttings could be impacted by a reduction in groundwater flow into the dams

The information in **Table 18** and **Figure 28** also provides an indication of those dams and/or surface water ponds/lakes outside of the construction footprint which potentially could be at risk of impact due to changes in the fractured bedrock aquifer as a result or changes to groundwater throughflow, and those that are located within the construction footprint. The implications of agricultural dams that are located within the construction footprint is discussed further in **Appendix K2, Agricultural assessment report**.

It is noted that the current assessment may not include all agricultural dams within the surrounding area; those included are from data from the NSW hydrometric database. Further field-based investigations would be undertaken during detailed design stage to fully ascertain positions of agricultural dams and lakes / surface water bodies which may be impacted, and to evaluate the likelihood of impacts at them.

Figure 29: Conceptual model at Roberts Hill showing potential groundwater connection to nearby surface water features (*PSM, 2019*)

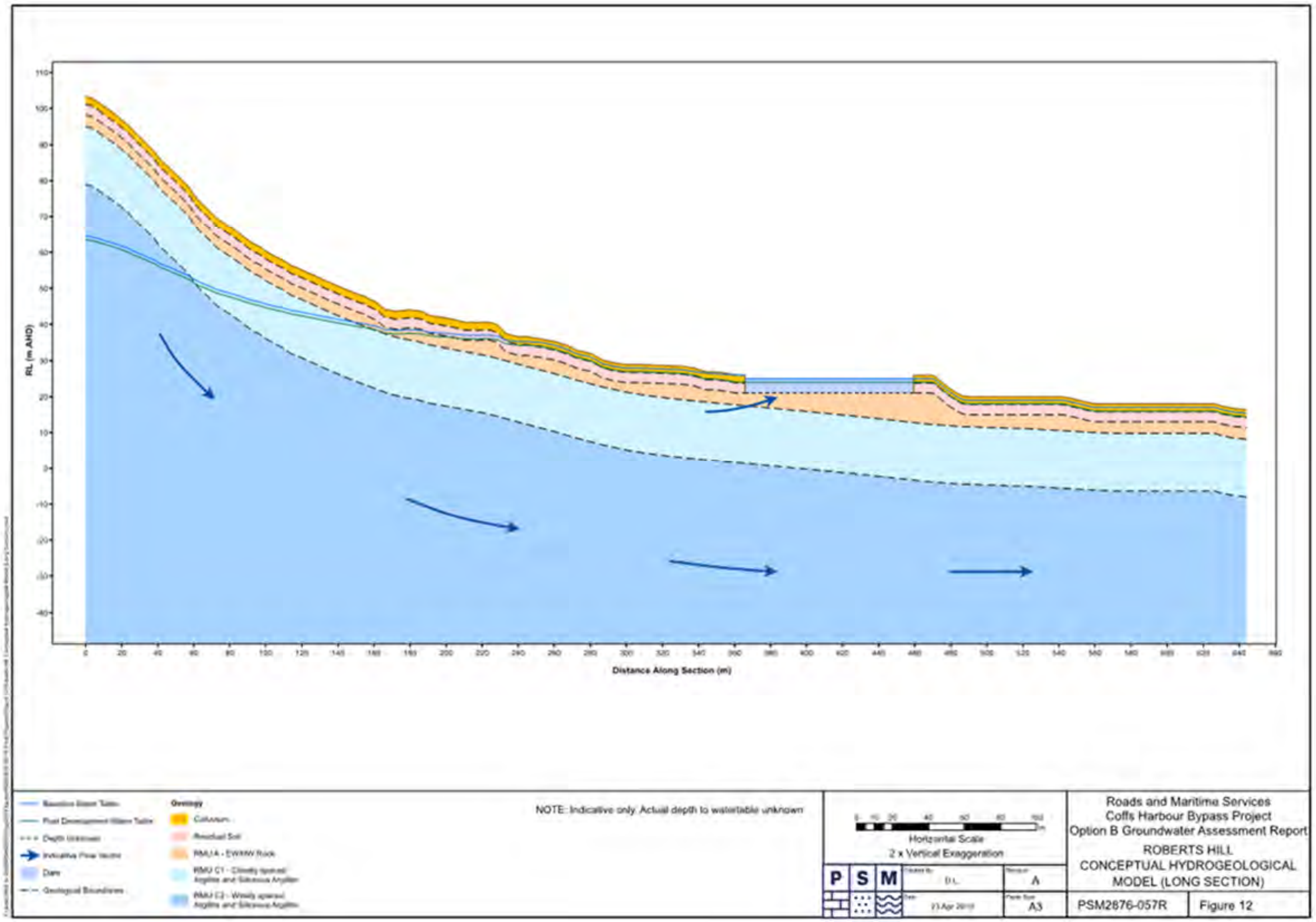


Figure 30: Conceptual model at Shephards Lane showing potential groundwater connection to nearby surface water features (*PSM, 2019*)

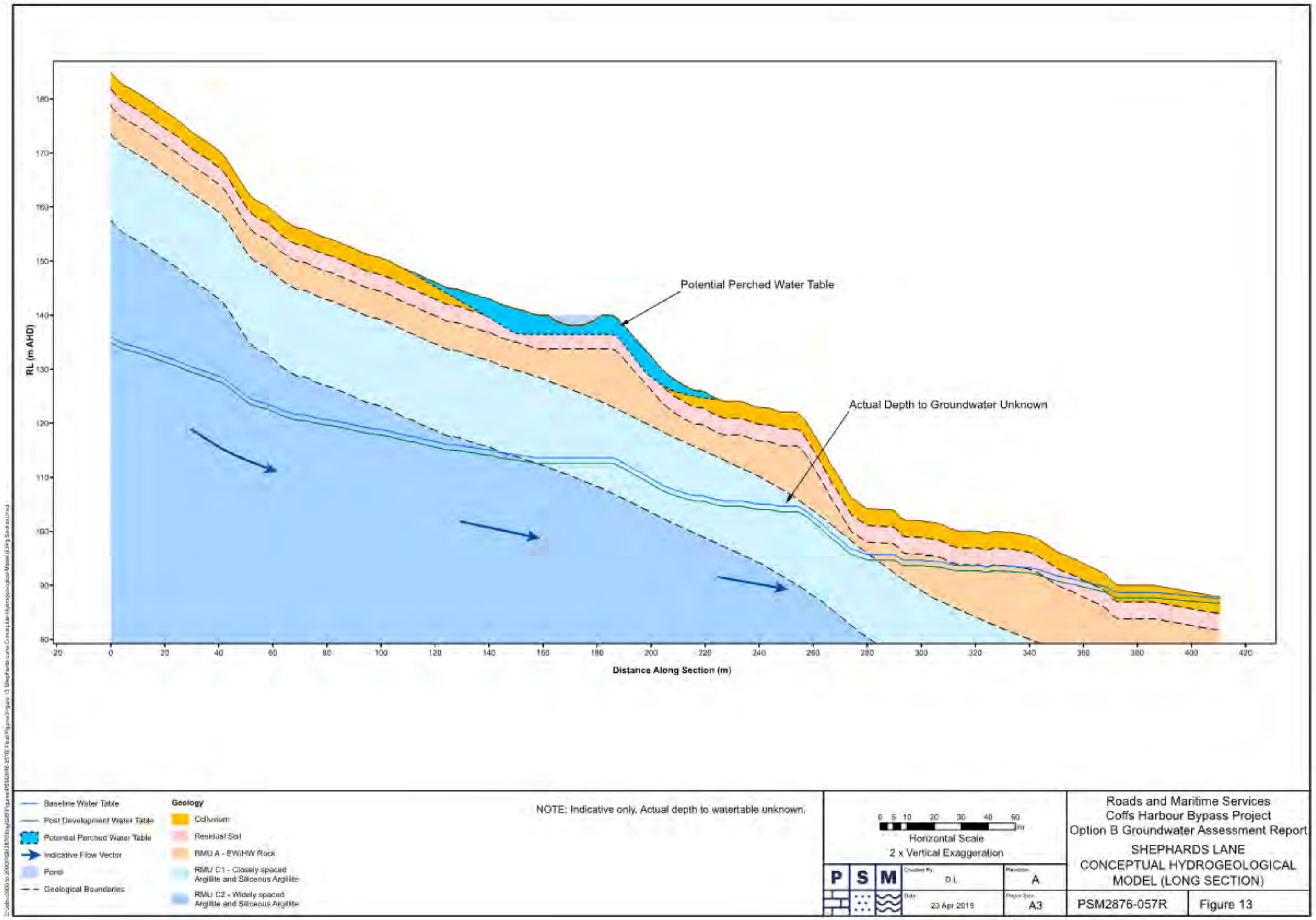


Figure 31: Conceptual model at Gatelys Road showing potential groundwater connection to nearby surface water features (*PSM, 2019*)

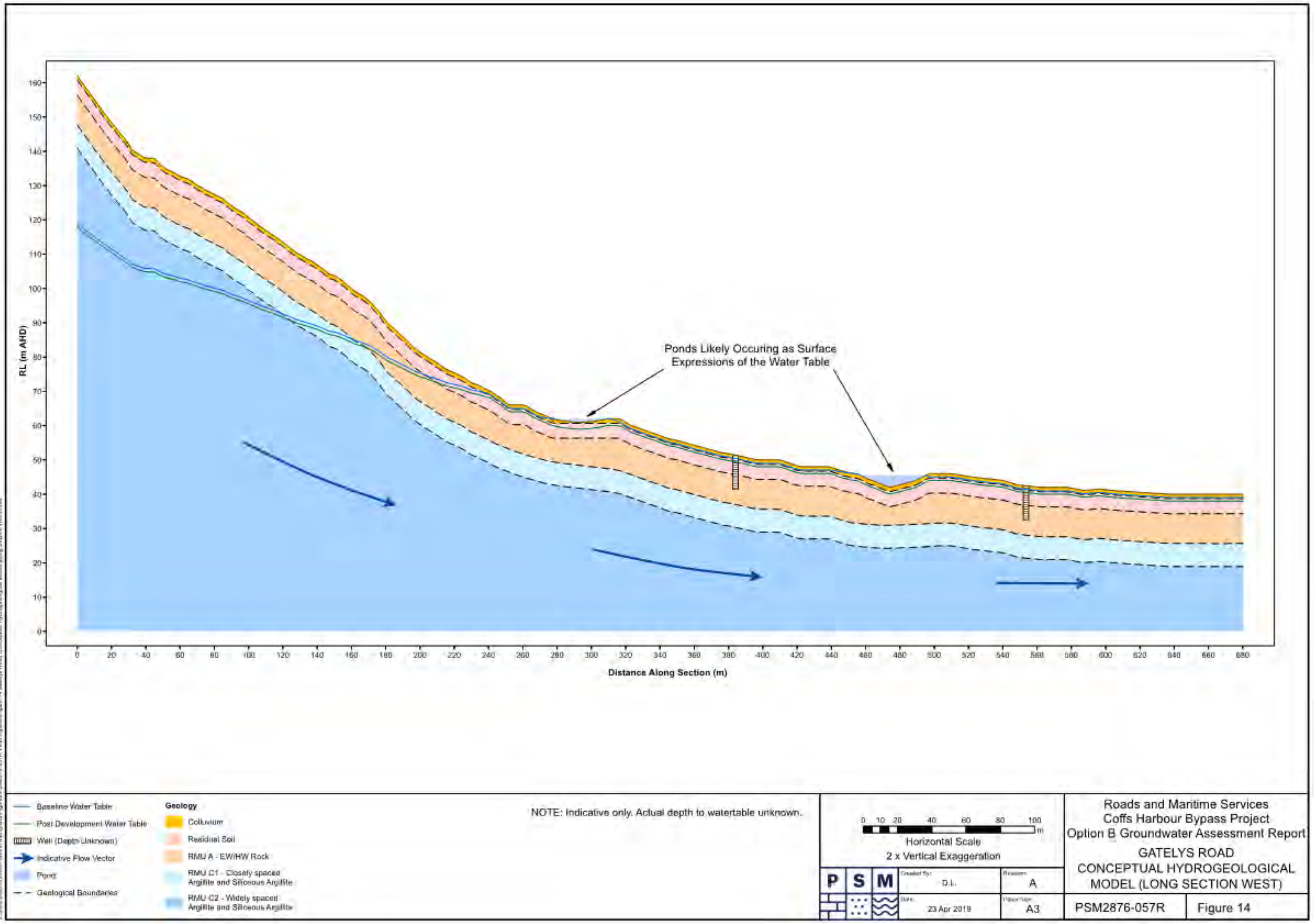


Table 18: Known agricultural dam locations within the zone of drawdown or construction footprint

Cut / tunnel	Feature	Easting	Northing	Notes
None	Agricultural dam	507860	6647035	Within construction footprint
4	Agricultural dam	508255	6647760	Downgradient of alignment, outside of footprint and 1m drawdown
4	Lake / surface water	508350	6648250	Partly within construction footprint [1]
Roberts Hill	Agricultural dam	508540	6648655	300m downgradient of alignment, outside 1m drawdown
8	Agricultural dam	508250	6648890	150m downgradient of alignment, outside 1m drawdown
	Agricultural dam	507760	6649200	Within construction footprint [1]
	Lake / surface water	508080	6649085	[1]
	Lake / surface water	508110	6649280	250m downgradient of alignment, outside 1m drawdown
11	Agricultural dam	507850	6650255	[1]
	Lake / surface water	507950	6650505	Within construction footprint [1]
	Lake / surface water	507935	6650055	250m downgradient of alignment, outside 1m drawdown
14	Agricultural dam	508350	6651075	[1]
Shephards Lane	Agricultural dam	508800	6651390	[1]
None	Lake / surface water	509535	6651300	Within Construction footprint
16	Agricultural dam	509965	6651060	Within Construction footprint
	Lake / surface water	509980	6650870	300m downgradient of alignment, outside 1m drawdown
Gatelys Road	Agricultural dam	510340	6651000	Within Construction footprint [1]
	Agricultural dam	511180	6651305	[1]
	Agricultural dam	511300	6651220	Within Construction footprint [1]
	Lake / surface water	510579	6650880	Within Construction footprint [1]
	Lake / surface water	511208	6651412	[1]

Cut / tunnel	Feature	Easting	Northing	Notes
None	Lake / surface water	511525	6651420	Within Construction footprint
18	Agricultural dam	511705	6651300	Within Construction footprint
	Agricultural dam	512350	6651785	Within Construction footprint
	Lake / surface water	511725	6651485	[1]
	Lake / surface water	512280	6651700	Within Construction footprint [1]
	Lake / surface water	512145	6651365	200m downgradient of alignment, outside 1m drawdown
18r	Agricultural dam	512355	6652015	Within Construction footprint
5a	Agricultural dam	512920	6653615	Within Construction footprint
21	Agricultural dam	513605	6654065	Within Construction footprint
Notes [1] Located within 1m zone of drawdown (corresponding to Table 16)				

4.2.3.6 Settlement

Lowering of groundwater levels within soils and rocks can lead to ground settlement to changes in the stresses of the material. Drawdown of groundwater levels along the construction footprint is principally within the fractured bedrock aquifer, with the greatest drawdown occurring adjacent to all Type A cuttings and tunnels. The stiffness of bedrock is very high, although it is reduced in the presence of major geological features. The extent and magnitude of settlement occurring within the rock mass surrounding cuttings and tunnels due to groundwater drawdown is anticipated to be small given the high stiffness of the bedrock.

Groundwater levels in alluvial aquifers may be locally affected, although the extent and magnitude of any change is likely to be small. The risk associated with settlement of unconsolidated alluvial material is expected to be low. Upper reach alluvial deposits (those which could be impacted by changes in water level) comprise largely of non-cohesive material which is less prone to settlement risk. Although it is unlikely to be necessary, mitigation of groundwater level drawdown in the alluvial deposits can be achieved by recharge of captured groundwater. Additional monitoring of water levels in the alluvial deposits would be undertaken as part of detailed design to provide supplemental baseline groundwater level information in these areas.

4.2.4 Impacts on groundwater quality

Potential risks to groundwater quality during construction include:

- Infiltration of contaminated surface water runoff,

- Infiltration of captured groundwater from excavations during construction,
- Hydrocarbon contamination from potential fuel and chemical spills during construction activities including drill and blast activity, leading to contamination of groundwater
- Exposure of acid sulfate soils during excavation or lowering of groundwater levels within the soils, leading to generation of acid leachate into groundwater, and
- Leaching of saline water into groundwater following disturbance of saline soils during operation and soil salinisation at cuttings due to evaporation of groundwater seepage.

4.2.4.1 Groundwater discharge quality and contamination

Infiltration into the ground is generally effective at filtering contamination and pollutants bound to particulate matter. Those contaminants such as hydrocarbons and solvents which are not bound to particulate matter are therefore at greater risk of polluting groundwater. Water quality controls including spill basins, proper storage of chemicals and having appropriate spill management plans in operation are standard mitigation approaches to reduce the risk to groundwater during construction.

Groundwater quality testing undertaken on the fractured bedrock aquifer indicated minor exceedences above the 95% percentile ANZECC aquatic ecosystem guideline values for a small number of heavy metal analytes. These exceedences were observed in samples collected from most monitoring locations sampled, indicating that groundwater is likely to be naturally elevated with respect to these heavy metals rather than occurring from a particular source.

Tunnelling will require use of construction water treatment plants to manage groundwater inflow into the tunnelling sites. The captured groundwater will be treated and discharged in accordance with criteria established in consultation with EPA and DPIE (Water). Processes will be established to allow for groundwater recharge back into the underlying aquifers or creeks to mitigate impacts. For cuttings, water will be directed to nearby sediment basins which would be discharged into local creeks/waterways/drainage lines in accordance with EPL requirements.

Management of turbid groundwater discharge during construction will be managed through use of sedimentation basins to prevent turbid water reaching water courses. Natural infiltration of discharged groundwater back into the aquifer system will provide natural filtering of suspended material within the water. Groundwater captured by cuttings and tunnels will be returned into the aquifer down gradient and within the same catchment from where it was intercepted where reasonable and feasible

4.2.4.2 Acid sulfate materials

Figure 5 shows the mapped location of PASS sites along within the study area and the results of samples collected during geotechnical investigations. If ASS are exposed or affected by groundwater drawdown, it may lead to oxidation and cause acid leachate formation. This may occur in situ or in excavated stockpiles during construction. Acid leachate may contain elevated heavy metals and can be transferred through groundwater and surface water and directly impact on aquatic life, water supply quality and construction materials. Acid leachate generation may cause corrosion of material such as concrete, iron, steel, and some aluminium alloys.

Several of the soil samples tested as part of the geotechnical investigations indicated presence of residual chromium reducible sulfur, low pH values and high total actual acidity levels. Potential acid sulfate soil was confirmed near Englands Road, North Boambee Road and Coramba Road.

The only areas of the project which are anticipated to extend into areas of mapped ASS are north of the Korora Hill interchange (some small areas of cut around C5a/C20). PASS testing in these areas indicated a pH(fox) of greater than 4, indicating that the risk is likely to be limited. The risk to groundwater caused by contamination from acid sulfate soils is considered to be low. However, because some of the soil samples tested indicated the presence of residual chromium reducible sulphur, low pH values and high total actual acidity (TAA) levels, an Acid Sulfate Materials Management Plan will to be required for the project to appropriately mitigate the potential risk to groundwater from ASS.

The risk of ASS disturbance due to lowering of groundwater levels is considered to be negligible. Predicted long term drawdown is unlikely to extend into areas of mapped ASS.

The risk of ASR along the project is generally considered to be negligible. Whilst tested rock samples contained pyrite, sufficient acid neutralising capacity was present to neutralise the samples and were determined to be non-acid forming. Further testing should be undertaken during detailed design investigations to confirm that the risk from ASR is negligible along the project and at all areas of cut.

4.2.4.3 Salinity

Based on groundwater quality testing and observed from publicly available information, there is unlikely to be an impact on groundwater from changes to/in salinity during the construction of the project. Groundwater quality testing indicated fresh to weakly brackish groundwater present within the fractured bedrock aquifer. Saline water is likely to be associated with estuarine and coastal aquifers. Deeper cuts are associated with soil landscapes further inland which are not saline.

Salinisation due to discharging groundwaters is not known to occur within the study area. The Coffs Harbour region is a high rainfall area and regular flushing of the road surface and salt accumulation is unlikely to occur at cuttings due to evaporation of groundwater seepage. Salinity issues are not anticipated to occur because of construction however groundwater monitoring during pre-construction and construction is recommended to assess changes in water quality.

4.3 Operational impacts

Once construction is complete, there will still be potential impacts associated with the operational phase of the project. During the operational phase, the impacts are mostly associated with the groundwater system reaching an equilibrium with the new topographic surface. In areas of fill there will be limited to no impact on groundwater. In areas of cut and drained tunnels, groundwater levels will be redistributed up and downgradient due to changes in discharge of groundwater into the cuttings.

The main operational phase of the project may impact upon aspects of groundwater including:

- Changes in groundwater levels, flow direction and throughput of groundwater due to potential redistributed flow paths,
- Changes to groundwater quality from pollution caused by spills and leakage of road user vehicles or drainage maintenance issues, and
- Changes to groundwater from longer-term acid sulphate generation caused by exposure or reduction in groundwater levels

4.3.1 Groundwater levels and flow

All cuttings and tunnels associated with the project will be fully drained, allowing ongoing seepage during the operational phase. Groundwater will be collected by drainage blankets installed below the base of the road and trench drains installed down the length of cutting sides.

All three tunnels have separate drainage systems to capture and recharge groundwater, and to manage stormwater ingress and water from the fire suppression (deluge) system. Captured groundwater in the Roberts Hill tunnel would drain through a longitudinal pit and pipe network to the southern portals before being recharged via infiltration pits or basins. Captured groundwater in the Shephards Lane and Gatelys Road tunnels would drain through a longitudinal pit and pipe network to both the southern and northern portals before being recharged via infiltration pits or basins.

Ongoing seepage is expected to continue to cuttings and tunnels during the operational lifetime of the project (see **Section 4.5** for details of the predicted water take).

The changes to groundwater level and flow within the fractured bedrock aquifer observed during construction will reach a new baseline equilibrium which will continue through the operational phase of the project. Additional impacts to those discussed in **Section 4.2** as a result of changes to groundwater level and flow are not anticipated during the operational phase of the project.

Appropriate management of captured groundwater will be undertaken to mitigate the downgradient impact. Permanent drainage systems will be installed to manage groundwater and will include transfer to absorption trenches, infiltration galleries or swales which will allow water to slowly infiltrate vertically into the underlying aquifers or discharge of groundwater to water quality ponds before being discharged into a downstream drainage channel. Further discussion of proposed mitigation measures is presented in **Section 5.3**.

Groundwater modelling indicates that adjacent to cuttings and drained tunnels, groundwater levels are likely to experience drawdown to near to the design level of the cut or base of the tunnel. Away from the cutting, the amount of drawdown will decrease to a level determined by the aquifer parameters and recharge to the aquifer. Seasonal variation in groundwater levels due to rainfall events is not captured by the modelling and will vary throughout the year.

Recharge to the aquifer systems is still anticipated to be primarily due to rainfall with a slight reduction in infiltration to the surficial and regional aquifers due to the increase in impermeable area (road surface, engineered fill). However, given that most of the project is within bedrock aquifer which is generally of low permeability, the overall impact is expected to be limited.

Flow directions in the fractured rock aquifer are likely to be locally permanently affected by the cuttings. Cuts in the ridges on the foot slopes of topographically higher areas is expected to capture local groundwater that infiltrates relatively quickly. The regional groundwater flow in the Coffs region is expected to flow at depth below the shallow recharge groundwater systems associated with each of the ridgelines.

On a regional scale, the flow direction of the aquifers is unlikely to be affected since the area of cut into the aquifer remains relatively small compared to the regional groundwater catchment.

As discussed in **Section 4.2** groundwater well sources are not anticipated to be significantly impacted; this is expected to remain the case for the operational phase of the project.

4.3.2 Groundwater quality

There are not anticipated to be any direct impacts on groundwater quality during operation of the project. Rather there may be outstanding residual impacts associated with the construction of the project. These may include:

- Infiltration of contaminated surface water runoff from unpaved surfaces. It should be noted during the operational phase of the project all drainage infrastructure will have been installed so opportunity for further contamination of groundwater sources should be significantly reduced.
- Hydrocarbon contamination from fuel and chemical spills during construction activities. There remains a potential for impacts associated with chemical spills associated with vehicle crashes however risk to groundwater systems is reduced as all surface drainage will remain in place.

The risk from salinity outlined in **Section 4.2** is expected to be the same for the operational phase of the project.

The risk from ASS during operational phases of the project is also considered to be negligible. Exposure of ASS is not expected to occur during operation and any existing ASS encountered during construction are expected to have been treated or disposed of in accordance with the ASS management plan. Areas of ASS are generally located in valleys, away from major cuttings where the largest drawdown of water levels will occur. The impact of groundwater drawdown in areas of ASS unlikely, as such the potential for generation of ASS leachate during operation is also considered to be unlikely to occur.

4.4 Assessment against NSW AIP

Potential impacts on the hydrogeological environment are compared to the requirements of the NSW Aquifer Interference Policy (2012) in **Table 19**. For the purposes of the assessment, the fractured bedrock is considered to be a less productive groundwater source. This is defined as:

- A groundwater source having total dissolved solids greater than 1500mg/l or
- A groundwater source that does not contain water supply works that can yield water at a rate greater than 5l/s.

The NSW Aquifer Interference Policy requires that potential impacts on groundwater sources, including their users and groundwater dependent ecosystems, be assessed against the minimal impact considerations. If the predicted impacts are less than the Level 1 minimal considerations (as outlined in **Table 19**) then the impacts of the project are acceptable.

Table 19: Impact assessment compared to the requirements of the NSW Aquifer Interference Policy

	Water table	Water Pressure	Water Quality
Less productive groundwater sources – porous and fractured rock	<u>Level 1</u> Less than or equal to 10% cumulative variation in the water table, allowing for typical 'post water sharing plan' variations, 40m from any: a) High priority groundwater dependent ecosystem; or b) High priority culturally significant site listed in the schedule of the relevant water sharing plan, or A maximum of a 2m decline cumulatively at a water supply work <u>Level 2</u> If more than 10% cumulative variation in the water table, allowing for typical climatic 'post-water sharing plan' variations, 40m from any c) High priority groundwater dependent ecosystem; or d) High priority culturally significant site listed in the schedule of the relevant water sharing plan, If appropriate studies demonstrate to the Minister's satisfaction that the variation will not prevent the long-term viability of the dependent ecosystem or significant site. If more than a 2m decline cumulatively at any water supply work then make good provisions should apply.	<u>Level 1</u> A cumulative pressure head decline of not more than a two-metre decline, at any water supply work <u>Level 2</u> If the predicted pressure head decline is greater than requirement 1 then appropriate studies are required to demonstrate to the Minister's satisfaction that the decline will not prevent the long-term viability of the affected water supply works unless make good provisions apply.	<u>Level 1</u> Any change in the groundwater quality should not lower the beneficial use category of the groundwater source beyond 40m from the activity. <u>Level 2</u> If condition 1 is not met then appropriate studies will need to demonstrate to the Minister's satisfaction that the change in groundwater quality will not prevent the long-term viability of the dependent ecosystem, significant site or affected water supply work.
Comment	No high priority GDEs or culturally significant sites within *Water Sharing Plans for the Coffs Harbour Area Unregulated and Alluvial Water Sources*, 2009 or the *North Coast Fractured and Porous Rock Groundwater Sources*, 2016 are listed in the study area. The project would not result in impacts to a culturally significant site or high priority GDE.	Predictive modelling indicates that most of the project meets the minimal impact consideration of less than 2m pressure head decline at any water supply work. The exception to this is at Gatelys Road tunnel where predictions indicate one groundwater supply well may experience a pressure head decline of more than 2 m. GW068986 has a predicted drawdown of around 4 m.	Groundwater inflows to cuttings will be captured and discharged via water quality basins or absorption trenches, infiltration pits or swales. Captured water during tunnelling will be treated using construction water treatment plants and discharged in accordance with EPA and DPIE (water) requirements. The risk of contamination on site and potential for discharge of pollutants will be managed on site using standard construction management procedures. The project is therefore not anticipated to change the beneficial use category of the groundwater source beyond 40 m from the activity.

The Aquifer inference policy states that where the predicted pressure head decline is greater than Level 1 minimal impact requirements then appropriate studies are required to demonstrate to the Minister's satisfaction that the decline will not prevent the long-term viability of the affected water supply. As described in **Table 19** the impact at one well is predicted to exceed the minimal impact criteria, however three of these are located within the construction footprint and will be acquired as part of the project.

Further assessment of the impact at GW068986 which has a predicted pressure head decline of approximately 4m will be undertaken as part of the detailed design process. Investigations will be undertaken at this well location to supplement existing information and to evaluate the potential impact on the long-term viability of the source.

Information to be collected at this stage should include operational status, supply well construction information, usage requirements, operational groundwater level data from the supply well and water quality. Should these investigations indicate that groundwater level drawdown is likely to impact the long-term viability of the groundwater supply well, additional monitoring, mitigation or remediation (make good provisions) will be required. Make good provisions may include:

- Provision of an alternate water supply/well
- Changing the bore pump so that it is better suited to the decreased water level in the bore
- Deepening the bore to allow it to draw water from a greater length of the aquifer
- Reconditioning of the water bore to improve its hydraulic efficiency
- Increased monitoring of the bore water levels to provide a level of confidence to the landholder that the impacts are managed appropriately

4.5 Assessment of water take

The estimated annual water take for the project during construction phase is 115 Megalitres pre year (ML/yr) and 57 ML/yr during the operation phase (**Table 20**).

The *Water Sharing Plan for the North Coast Fractured and Porous Rock Groundwater Sources* (DPI, 2016) provides rules for granting access licences, managing access licences, water supply works approvals and access licence dealings. The estimated project water take (**Table 20**) is compared to the available water in the New England Fold Belt Coast Groundwater Source outlined in the water sharing plan *(DPI 2016)*. The maximum estimated water take is approximately 0.2% of the Long-Term Average Annual Extraction Limit and 0.03% of the Upper Extraction Limit, representing a small proportion of the total water availability in the groundwater source

Total recharge to the fractured bedrock across the three river sub-catchments (Boambee creek, Coffs Creek and Korora Basin, see **Figure 1**) which the project crosses is estimated to be approximately 7 GL/yr. This is based on a net recharge of 5% of annual rainfall of 1651 mm across the three sub-catchments (43.7 km^2, 26.5 km^2 and 14.8 km^2 respectively). The total predicted steady state discharge of groundwater into all cuttings and tunnels is 57 ML/yr, or approximately 0.8% of the total annual estimated recharge into the fractured bedrock within the three sub-catchments.

This assessment does not necessarily take into account the true recharge catchment of the fractured bedrock aquifer which is unlikely to align with that of the surface water creeks and is likely to be much larger. Neither does it take into account local variation in recharge across the catchments. Even so, it indicates that total water take is likely to represent a small proportion of the total recharge into the fractured bedrock aquifer.

Table 20: Project water take and available water

Max estimated construction phase water take (ML/yr)	**Estimated water take – operation phase (ML/yr)**	**New England Fold Belt Coast groundwater source**	
		LTAAEL (ML/yr)	**UEL (ML/yr)**
115	57	60,000	375,000

Section 5.23 of the EP&A Act 1979 provides exemption for SSI projects for the need of a water use approval under section 89 of the Water Management Act 2000. If required, the project will need to ensure an aquifer interference approval has been granted for the proposed works.

5 Mitigation and management

The impact on groundwater systems will vary during the phases of the project, as described above. The management strategy needs to be prepared in advance of the construction phase and carried through to the operational phase of the project. The assessment indicates that there are seven mainline cuts and three drained tunnels which are likely to extend below the estimated average groundwater.

Groundwater modelling undertaken as part of the concept design indicates that there may be impacts on groundwater users and other receptors because of the construction of the scheme, however the impacts are not anticipated to be major. Groundwater inflow is predicted at all Type A cuttings and tunnels which extend below the water table however anticipated groundwater inflow from cut 18, Shephards Lane and Gatelys Road is predicted to make nearly 70% of the water take for the project.

Regardless of the water take at each, captured throughput into all cuttings and tunnels will need to be managed during construction and operational phases in order to mitigate impacts on groundwater levels, aquifer throughput or baseflow to downgradient receptors. Options for mitigation measures to manage this are discussed in **Section 5.3.**

5.1 Management strategy

The proposed management approach to address the issues described in this report as discussed in more detail in subsequent sections, is as follows:

- Pre-construction investigations and ongoing groundwater monitoring – additional geotechnical investigations to supplement existing information in particular at cuts or tunnel sections where additional baseline groundwater level information may improve modelling predictions, and longer-term groundwater monitoring information from alluvial deposits close to Type A cuts and tunnels. Additional acid sulfate soil testing and establishment of supplemental investigations and construction of groundwater monitoring at sensitive receptors,
- In combination with additional groundwater information obtained from the investigations, revision of existing numerical models may be undertaken in order to improve certainty around the predictions and outcomes. Revisions to the modelling would also be based on the project detailed design and additional hydrogeological data to supplement the current conceptual understanding of the system,
- Construction and operational monitoring to assess whether the impact assessment predictions are accurate and to aid early intervention should outcomes deviate from those predictions.
- Mitigation – design and implementation of environmental and engineering management measures where necessary to minimise the impacts on the groundwater environment and receptors.

5.2 Monitoring

The current groundwater monitoring program targets the fractured bedrock aquifer along the project alignment, at areas of proposed cutting and tunnels. Continuous monitoring has been on-going since the 5th May 2017 (reported up to February 2019 in this report). The groundwater monitoring network comprises:

- 19 standpipes, with data loggers installed at 11 of these locations
- 11 standpipes with data loggers and a vibrating wire piezometer (VWP) grouted below the screen level connected to a data logger
- 6 nested VWP locations where 2 VWPs are installed in each location, approximately 20 m apart in depth, and are connected to data loggers

Hourly readings are taken by the data loggers for both standpipes and VWPs. The VWPs record a frequency which is calibrated to the height of water above the VWP. The standpipe loggers record temperature and absolute pressure above logger, which is corrected for barometric pressure effects.

Prior to construction, monitoring of groundwater level and groundwater quality is proposed to provide continued assessment of baseline groundwater conditions. During construction and operation, continued monitoring will be undertaken to verify modelling predictions and to ensure engineered mitigation measures are effective. The proposed monitoring program would comprise of the following:

- Continued groundwater monitoring at monitoring standpipes and VWPs installed as part of previous geotechnical investigations,
- Groundwater quality sampling,
- Installation of additional groundwater monitoring wells to supplement and expand on existing datasets. The location of additional investigation would be targeted in areas where additional monitoring would serve to improve certainty around modelling predictions.

The objectives of groundwater monitoring for each of the three phases of the project are as follows:

- Preconstruction phase:
  - Supplement and confirm baseline groundwater conditions,
  - Identify parameters for monitoring during construction.
- Construction phase
  - Demonstrate compliance with approvals and other monitoring requirements for the project,
  - Verification of modelling predictions, assessment of groundwater impacts and effectiveness of engineering mitigation measures.
- Operation phase
  - Evaluation of impacts and effectiveness of engineering mitigation measures;

  - Evaluation of site stabilisation and determination of new groundwater conditions
  - Groundwater monitoring during this phase would be undertaken for a period of three years, or before if it can be proved that no impact has occurred.

Additional hydrogeological investigation and monitoring would be undertaken to supplement the current monitoring network and assist in the mitigation of groundwater impacts during the construction and operation of the project. The proposed additional groundwater investigation scope includes:

- Installation of groundwater monitoring standpipes at cutting locations where there less available data (these are generally at smaller cuttings) or from cuttings and drained tunnels where supplementary groundwater information may improve the certainty in modelling predictions and outcomes,
- Installation of groundwater monitoring standpipes in alluvial deposits along the project which have the potential to be affected by changes in groundwater level from cuttings or tunnels,
- Ground truthing of potentially affected agricultural dams and environmental receptors to supplement information in the current assessment. This may include additional investigation to monitor groundwater levels and quality,
- Investigations (ground truthing and consultation with landholders) to evaluate the potential impacts at those supply wells identified in this assessment where the predicted impacts exceed the minimal impact considerations of the NSW Aquifer Interference Policy. These investigations will be undertaken during detailed design and may include operational status, supply well construction information, usage requirements, operational groundwater level data and water quality, to evaluate the long-term viability of the supply wells as a result of the predicted impacts.

Where possible, existing groundwater monitoring locations will be used for construction and operational monitoring and all new locations will be positioned to enable, where possible, ongoing monitoring (i.e. outside of areas directly affected by earthworks or construction activity).

5.3 Mitigation measures

Groundwater impacts have been identified for the construction and operational phases of the project. To minimise the potential impacts on the groundwater environment, engineering mitigation measures are likely to be required.

5.3.1 Mitigation measures to reduce impact on groundwater flow and quantity

The major impacts to groundwater flow, levels and quantity are anticipated to be due to construction of cuttings and drained tunnels, and capture of groundwater seepage. Mitigation measures to reduce the impact of these would typically be expected to comprise of one of the following:

- Engineered measures that transfer seepage water downgradient. Standard practice would be collect seepage at the cut face and from drainage blankets which would be diverted into water quality ponds before being discharged into a downstream drainage channel, or
- Engineered mitigation measures that transfer seepage water back into the groundwater system downgradient of the cut or embankment. Captured water would be transferred to grassed swales or absorption trenches which allow water to slowly infiltrate vertically into the underlying aquifers.

From the perspective of reducing drawdown impacts from the major cuttings, the second option is likely to be favourable as it reduces the net extraction of groundwater from the aquifer, however this option relies on their being sufficient vertical permeability to allow recharge back into the aquifer. The concept design prioritises discharge to grassed swales where possible however due to space constraints, where there is insufficient swale length for treatment, sediment basins with downstream discharge will be used instead.

At the three drained tunnels, the current concept design incorporates drainage systems that will be installed within the tunnels to manage stormwater ingress, water from the fire suppression system and groundwater seepage. Captured water will be conveyed to holding tanks located near the tunnel portals with outlets connecting to downstream basins.

Captured groundwater in the Roberts Hill tunnel is designed to drain through a longitudinal pit and pipe network to the southern portals before being recharged via infiltration pits or sediment basins. Captured groundwater in the Shephards Lane and Gatelys Road tunnels is designed to drain through a longitudinal pit and pipe network to both the western and eastern portals before being recharged via infiltration pits or sediment basins. The use of infiltration pits and basins will reduce the impact of captured groundwater throughput by returning it back into the same catchments from which it was captured.

Surface water runoff at Roberts Hill tunnel is designed to drain towards the southern portals where it would be captured in an operational water quality basin adjacent the tunnel portal. Both Shephards Lane and Gatelys Road tunnels are designed with crests to allow surface water runoff to drain to both the western and eastern portals where it would be captured in an operational water quality basin adjacent the tunnel portals.

5.3.2 Mitigation measures to reduce impact on groundwater quality

To reduce potential impacts on groundwater quality from construction activity, standard mitigation approaches are expected to be implemented. These would include:

- Stockpiles, washdown areas, refuelling and chemical storage will be located away from sensitive areas and where groundwater levels are close to the surface. If necessary, these areas will be lined to prevent potential contamination entering the shallow groundwater environment

- Chemical and fuel storage will be appropriately stored in bunded areas
- Management of construction runoff from the site to prevent mixing with groundwater seepage, where possible. Treatment of poor quality or mixed groundwater in sedimentation basins before being discharged,
- Development of an acid sulfate management plan and appropriate handling of acid sulfate materials.

5.4 Management approach

Table 21 provides a summary of the expected impacts, environmental management measures, responsibility and timing for the project.

Table 21: Groundwater impacts and mitigation measures.

Impact	Mitigation measure	Responsibility	Timing
Management of groundwater during construction	A Soil and Water Management Plan (SWMP) will be prepared in accordance with Landcom (Blue Book) Erosion and Sediment Control Principles and Procedures (Landcom 2004) and Erosion and Sediment Management Report: Coffs Harbour Bypass (SEEC, 2019). The plan will identify all reasonably foreseeable risks relating to groundwater quality and describe how these risks will be managed and minimised during construction.	Contractor	Prior to and during construction
	Groundwater seepage into excavations will be managed in line with Roads and Maritime Technical Guidelines Environmental Management of Construction Site Dewatering (RTA, 2011), and in accordance with any licence conditions.	Contractor	Construction
Acid sulfate materials	An ASS Management plan will be implemented as part of the SWMP. The plan will be prepared in accordance with the Guidelines for the Management of Acid Sulfate Materials (RTA 2005) and will include provision for stockpiles containing PASS will be lined and bunded in accordance with relevant guidelines to prevent leachate contaminating groundwater ASS to undergo appropriate treatment and materials handling in line with the plan.	Contractor	Prior to and during construction
Management of groundwater interception	Further geotechnical ground investigation will be undertaken including installation of groundwater monitoring standpipes. Water quality testing and groundwater level monitoring will be undertaken at these sites (See **Section 5.2** for further detail of proposed strategy for determining	Roads and Maritime	Prior to construction

Impact	Mitigation measure	Responsibility	Timing
	locations where additional investigation may be beneficial).		
	Supplemental numerical modelling will be undertaken, if appropriate, where additional groundwater monitoring information can be used to improve calibration and confidence of the results of existing models, or where detailed design is materially different to the concept design. Additional modelling is likely to comprise of updates to existing numerical modelling based on supplemental information or design changes, or where required, construction of additional 2D cross section models.	Roads and Maritime	Prior to construction
	Where groundwater is captured by cuttings, it will be returned into the aquifer down gradient and within the same catchment from where it was intercepted. During construction, this can be facilitated by discharging water into grassed swales and temporary recharge basins for infiltration. The swales can also be used to divert water around the construction site to ensure water does not mix with construction water. Engineering measures for long term management of groundwater inflow to cuttings will be designed and constructed to ensure that where possible, groundwater is recharged downgradient of the cutting from where it is captured, and within the same catchment. This will be facilitated by discharge basins, infiltration pits and grassed swales.	Contractor	Prior to and during construction
	Where this is not possible due to space constraints, measures will be designed and implemented that transfer seepage water downstream via sediment basins before being discharged into a downstream drainage channel or creek, within the same catchment.	Contractor	Prior to and during construction and operation
	Where space is insufficient to return groundwater via temporary recharge basins, groundwater may be collected via water quality control ponds prior to being discharged into natural waterways down slope of the cutting but within the same catchment from where the water was intercepted. Due to the potential for changes to groundwater level and flow directions,	Contractor	Prior to and during construction

Impact	Mitigation measure	Responsibility	Timing
	groundwater monitoring will need to be undertaken to evaluate the extent of groundwater drawdown and impact on the aquifer.		
Prevention of groundwater impacts from cuttings and embankments	Additional ground truthing and site inspections to evaluate receptors which could be impacted because of changes to the groundwater environment. This would include potentially impacted supply wells and agricultural dams within the surrounding area, with the aim of providing additional information to evaluate potential impacts. Make good provisions where appropriate.	Roads and Maritime / Contractor	Prior to construction
	Ongoing monitoring of existing groundwater monitoring boreholes along the alignment. Water level and water quality monitoring will be undertaken to confirm that impacts are in line with predictions. Monitoring of seepage into cuttings will be evaluated against the predictions of the numerical modelling. Environmental and engineering management measures will be implemented where predictions and/or modelling and monitoring suggest that these are required to minimise impacts on groundwater.	Roads and Maritime / Contractor	Prior to and during construction and operation
	Groundwater monitoring of Type B cuttings should continue during construction to identify any unforeseen impacts and the need to implement any mitigation measures during construction.	Contractor	Prior to and during construction
	Major embankments will be designed to enable distributed flow of surface water to prevent ponding using appropriate drainage systems	Roads and Maritime	Detailed design

6 Conclusions

Evaluation of the potential impacts of the Coffs Harbour Bypass on groundwater has been undertaken to address the Secretary's Environmental Assessment Requirements (SEARs).

The hydrogeological setting of the project comprises of a regional fractured bedrock groundwater system and local alluvial aquifer systems found within creek line deposits. In their upper reaches, the alluvial aquifers are highly connected to surface water within creeks, which are strongly dependent on rainfall and surface water run-off. Groundwater flow within the alluvial system is broadly to the east but is locally constrained to the creek lines and floodplains.

The fractured bedrock aquifer is a regional system in which groundwater flows through secondary features (fractures, joints, faults, shear zones) within the rock mass, in an easterly direction towards the coast. The fractured bedrock is generally of low hydraulic conductivity and groundwater movement is slow. The project alignment flanks the foothills of the Great Diving Range and traverses several ridgelines into which cuttings and tunnels are proposed. These are all within the upper parts of the regional fractured bedrock groundwater system.

The principle impacts to the groundwater system from the project are:

- Changes to groundwater level and flow direction due to cuttings and drained tunnels intercepting the fractured bedrock groundwater table. Seepage of groundwater will locally draw water levels down in the surrounding aquifer, potentially affecting nearby environmental receptors, and groundwater users utilising the fractured bedrock aquifer; and
- Deterioration of groundwater quality due to construction activity, thorough spillage of chemicals into groundwater, exposure and leaching of acid sulphate material and saline soils.

Numerical groundwater modelling has been undertaken to evaluate the impact from cuttings and drained tunnels which extend below the groundwater table. The results of the modelling predict a total steady state inflow to cuttings and tunnels of 57ML/yr. Construction inflows are predicted to initially be higher, however the cumulative water take will be determined by the construction programme.

Based on an assumed maximum total groundwater inflow of 311 kL/d (115 ML/yr) may be captured by the project during construction. Although the project is exempt from need for a water use licence, the existing water allocations from the regional fractured bedrock aquifer indicates there is likely to sufficient water available. The total water take is predicted to be less than 1% of the annual recharge to the fractured bedrock within the Boambee, Coffs and Korora sub-catchment areas.

Potential groundwater impacts have been assessed against the impact criteria specified in the NSW Aquifer Interference Policy (*DTI, 2012*) for less productive groundwater sources. The groundwater impacts have been assessed to be below the Level 1 minimal impact considerations for water table and water quality impacts but above the minimal impact consideration was water pressure at several groundwater supply wells surrounding the project alignment.

Several groundwater supply wells have been identified in the study area. The predicted drawdown indicates that a single supply wells outside of the project footprint (near to Gatelys Road tunnel) will experience a decline of groundwater level of around 4.3m. As required by the AIP, further investigations will be undertaken at detailed design to evaluate the potential impact at this supply well in order to evaluate the long-term viability of the supply well as a result of the predicted impacts. Should these investigations indicate that groundwater level drawdown is likely to impact the long-term viability of the groundwater supply well, additional monitoring, mitigation or remediation (make good provisions) will be required.

Agricultural dams and surface water bodies have been identified in the area surrounding the project alignment. Conceptually, it is possible that several of these features are at least partially spring fed or connected to the underlying aquifer. Changes to groundwater level or local throughput in the fractured bedrock may impact on the availability of water recharging the agricultural dams. Further investigations at detailed design will be undertaken to evaluate the potential impact on those dams and surface water features highlighted in this assessment.

There are no potential impacts to high priority GDEs, culturally significant sites or protected wetlands. Potential impacts to low priority GDEs and native vegetation down gradient of cuts and drained tunnels are unlikely to be significant since these are unlikely to be reliant on groundwater from the fractured bedrock. Changes to surface water run off may affect perched groundwater systems within surficial deposits however the effect is likely to be localised.

Mitigation measures will be implemented during construction and operation to manage groundwater seepage and minimise impacts from drawdown, reduction in aquifer throughput and impacts on groundwater quality. These are expected to comprise of:

- Capture of seepage inflow at cut sites and transfer of the water downgradient within the same catchment. Where possible, groundwater will be recharged back to the aquifer system in grassed swales to reduce groundwater level drawdown.
- Implementation of construction management plans including acid sulphate materials, stockpile management and control of construction chemicals and other potential groundwater pollutant including appropriate handling of excavated materials, chemicals and fuels on site. Temporary water treatment plants will also be used during the construction phase to ensure water meets discharge quality criteria

Ongoing monitoring of groundwater levels at boreholes along the project is currently being undertaken to supplement the existing baseline understanding of the system. The assessment has highlighted other areas of the project where additional groundwater information may be collected to beneficially supplement the existing data. Installation of additional groundwater monitoring instrumentation is recommended as part of detailed design, prior to construction activity. Groundwater level and water quality monitoring will be undertaken to confirm that impacts on the environment are in line with those predicted in this assessment and any additional update.

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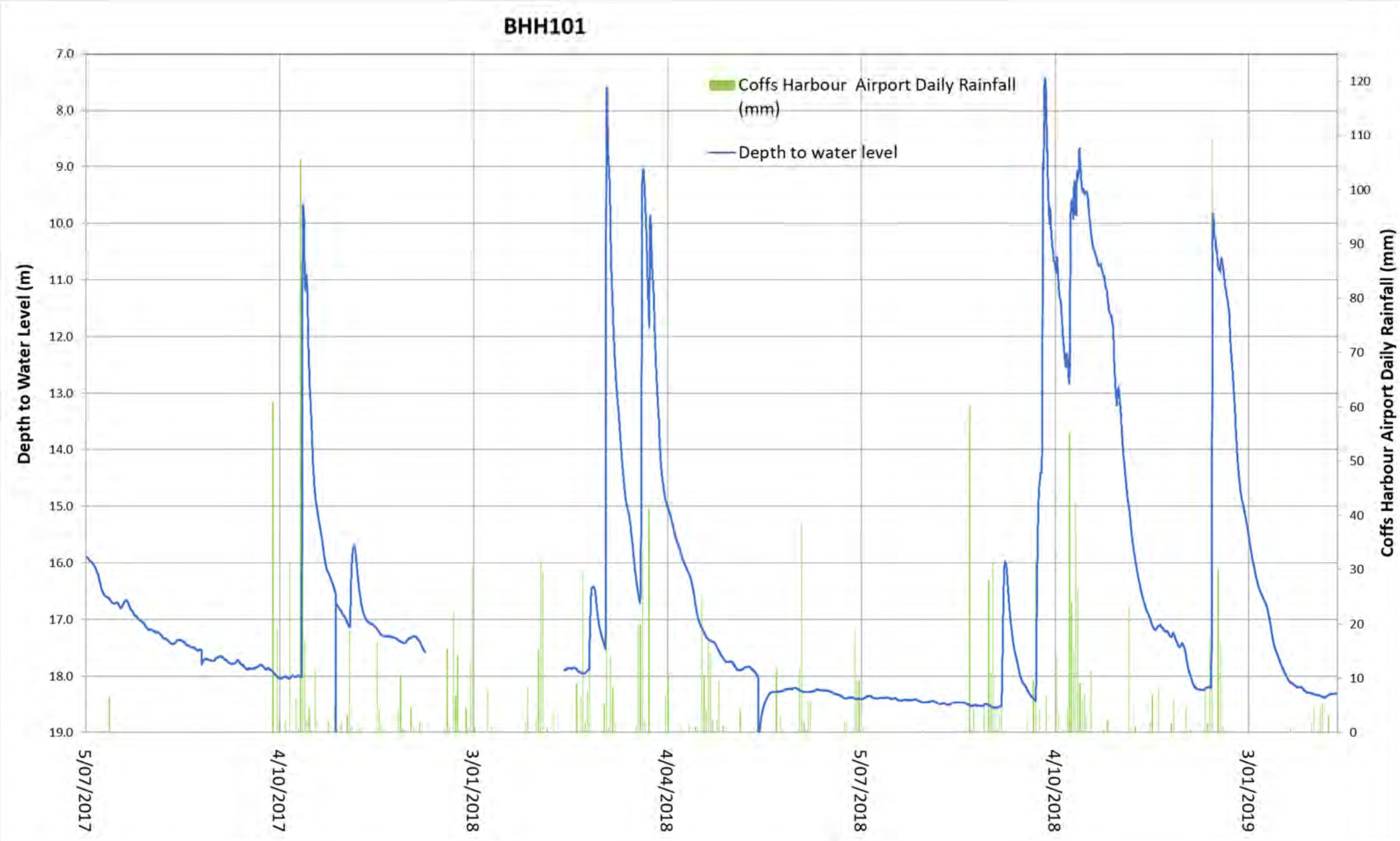
Appendix A

Groundwater Level Monitoring Data



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Depth to water level
Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
15.0
16.0
17.0
18.0
19.0
120
110
100
90
80
70
60
50
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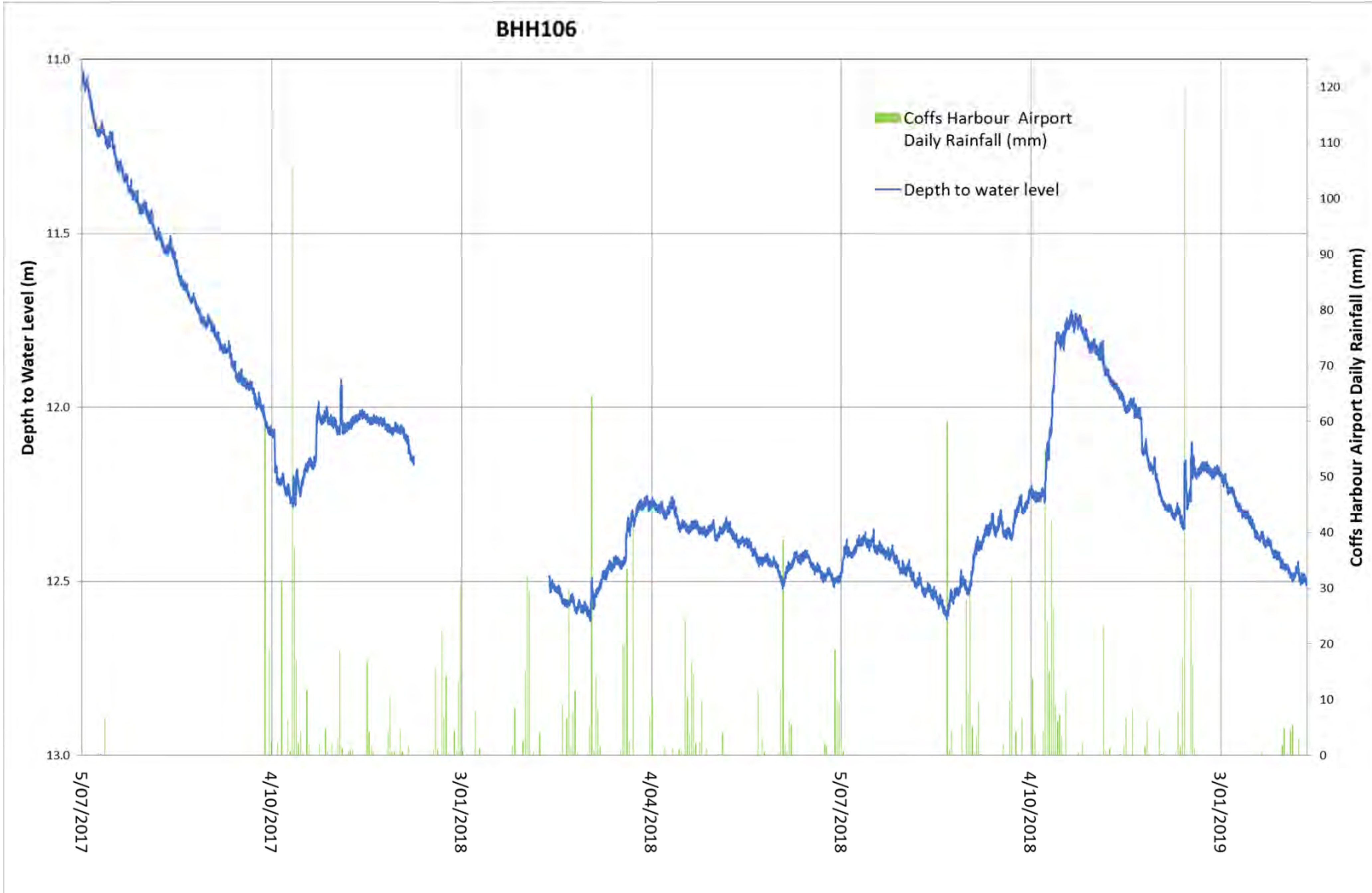
4/10/2018
3/01/2019


BHH106
Coffs Harbour Airport Daily Rainfall (mm)
Depth to water level
Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
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11.5
12.0
12.5
13.0
120
110
100
90
80
70
60
50
40
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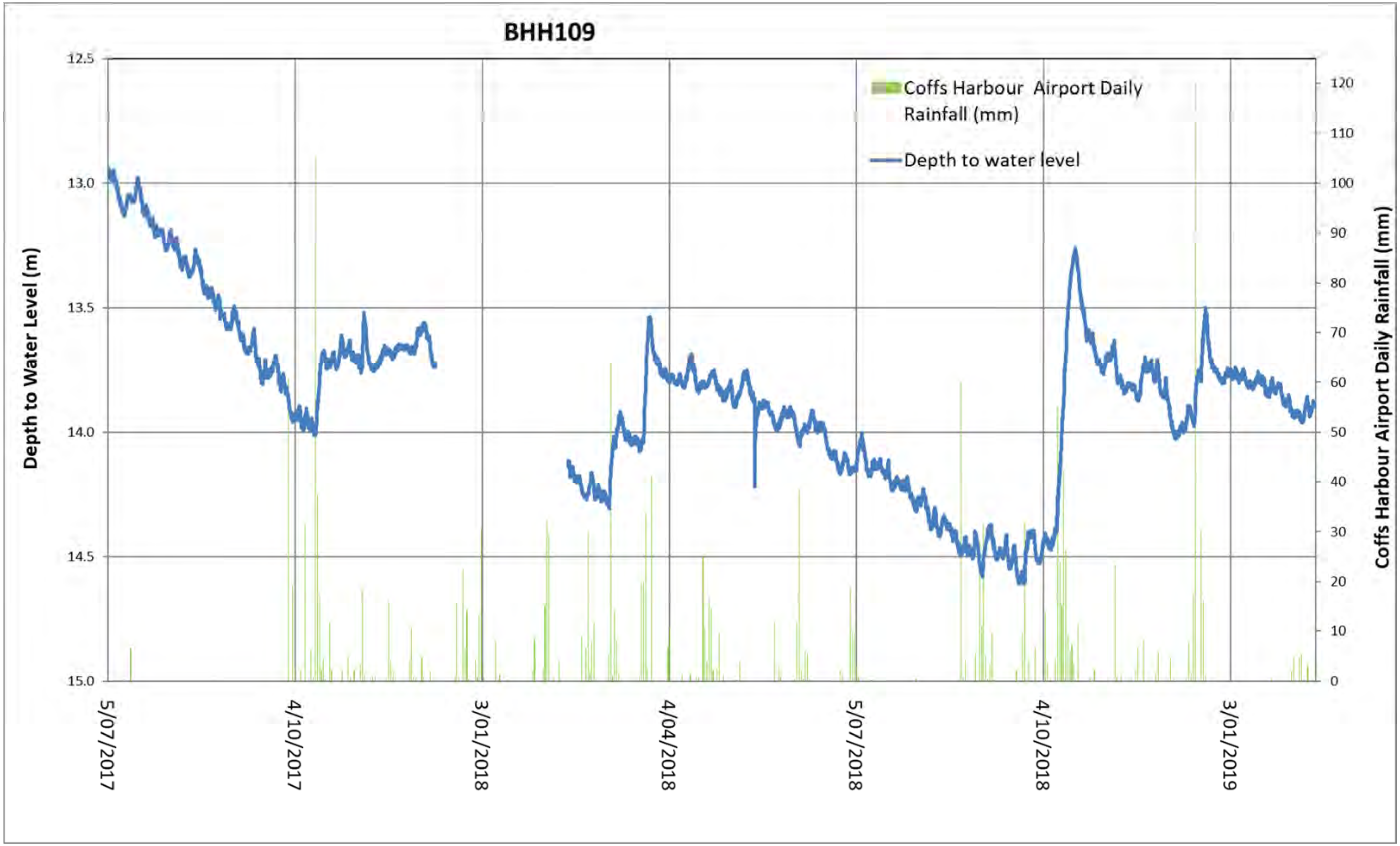
4/10/2018
3/01/2019



BHH109
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Depth to water level
Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
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14.0
14.5
15.0
120
110
100
90
80
70
60
50
40
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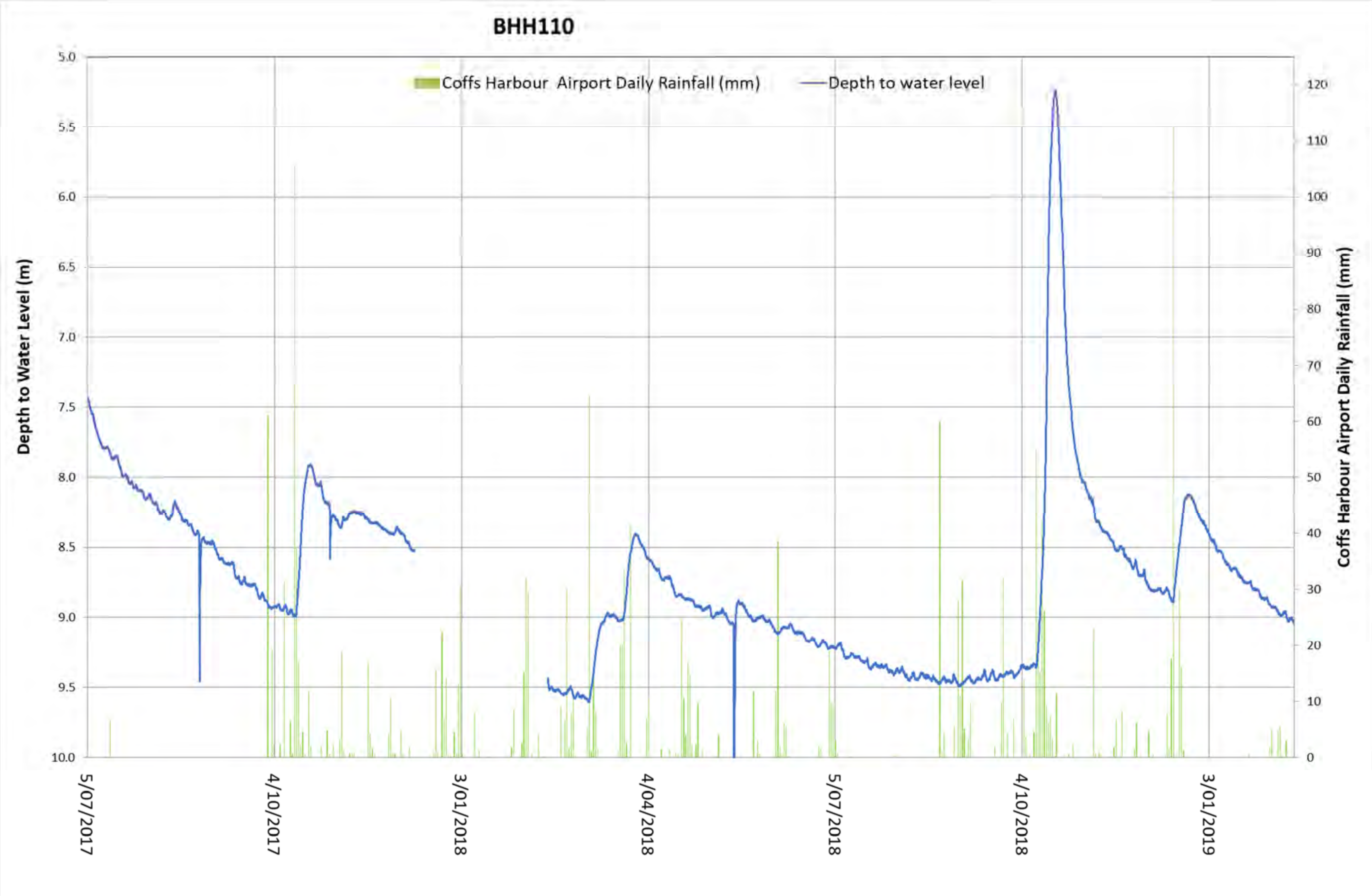
4/10/2018
3/01/2019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Coffs Harbour Airport Daily Rainfall (mm)
Depth to water level
Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
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6.5
7.0
7.5
8.0
8.5
9.0
9.5
10.0
120
110
100
90
80
70
60
50
40
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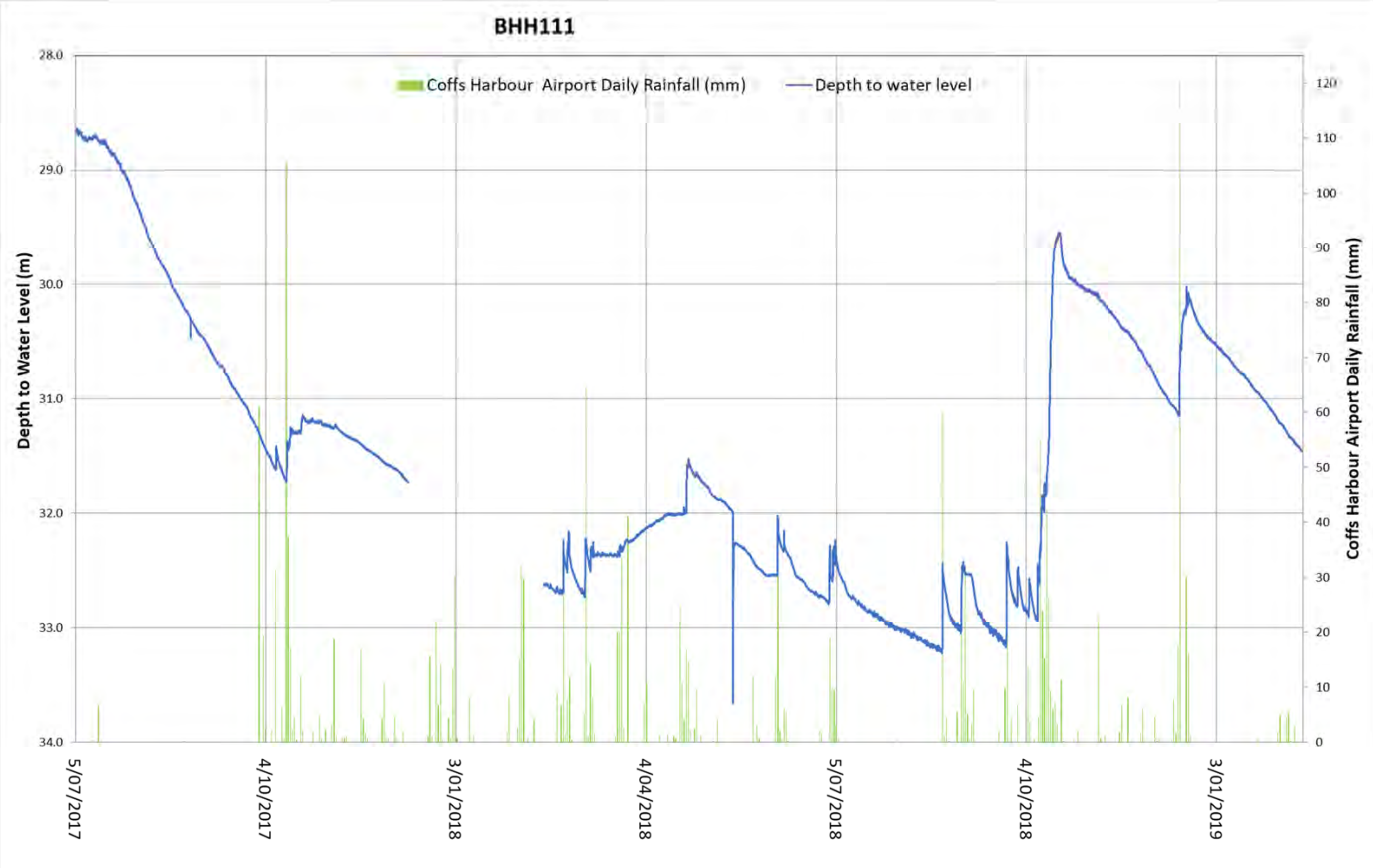
4/10/2018
3/01/2019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Depth to water level
Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
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31.0
32.0
33.0
34.0
120
110
100
90
80
70
60
50
40
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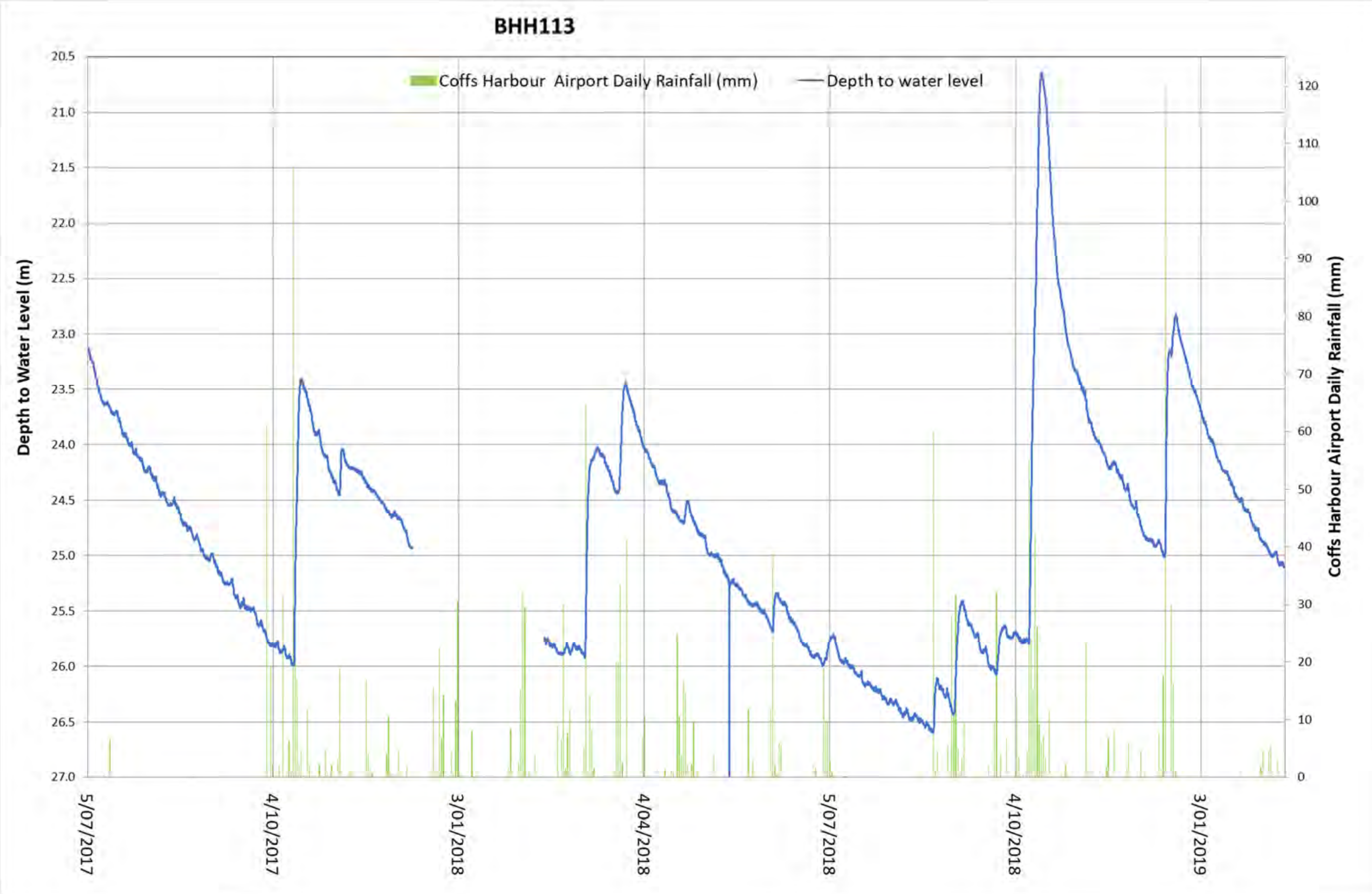
4/10/2018
3/01/2019


BHH113
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Depth to water level
Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
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21.0
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22.0
22.5
23.0
23.5
24.0
24.5
25.0
25.5
26.0
26.5
27.0
120
110
100
90
80
70
60
50
40
30
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10
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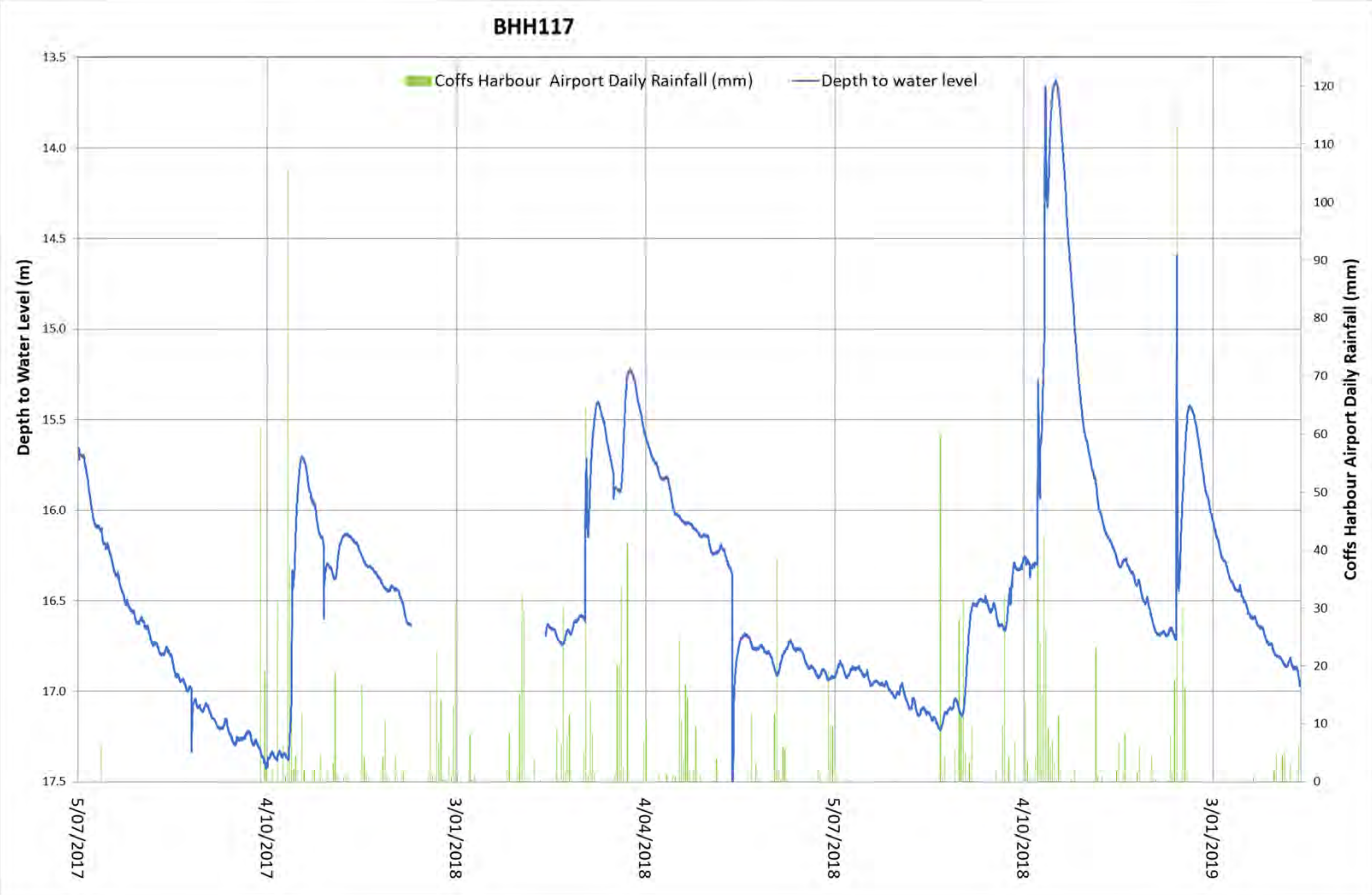
4/10/2017
3/01/2018
4/04/2018
5/07/2018
4/10/2018
3/01/2019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Depth to water level
Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
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14.5
15.0
15.5
16.0
16.5
17.0
17.5
120
110
100
90
80
70
60
50
40
30
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4/10/2017
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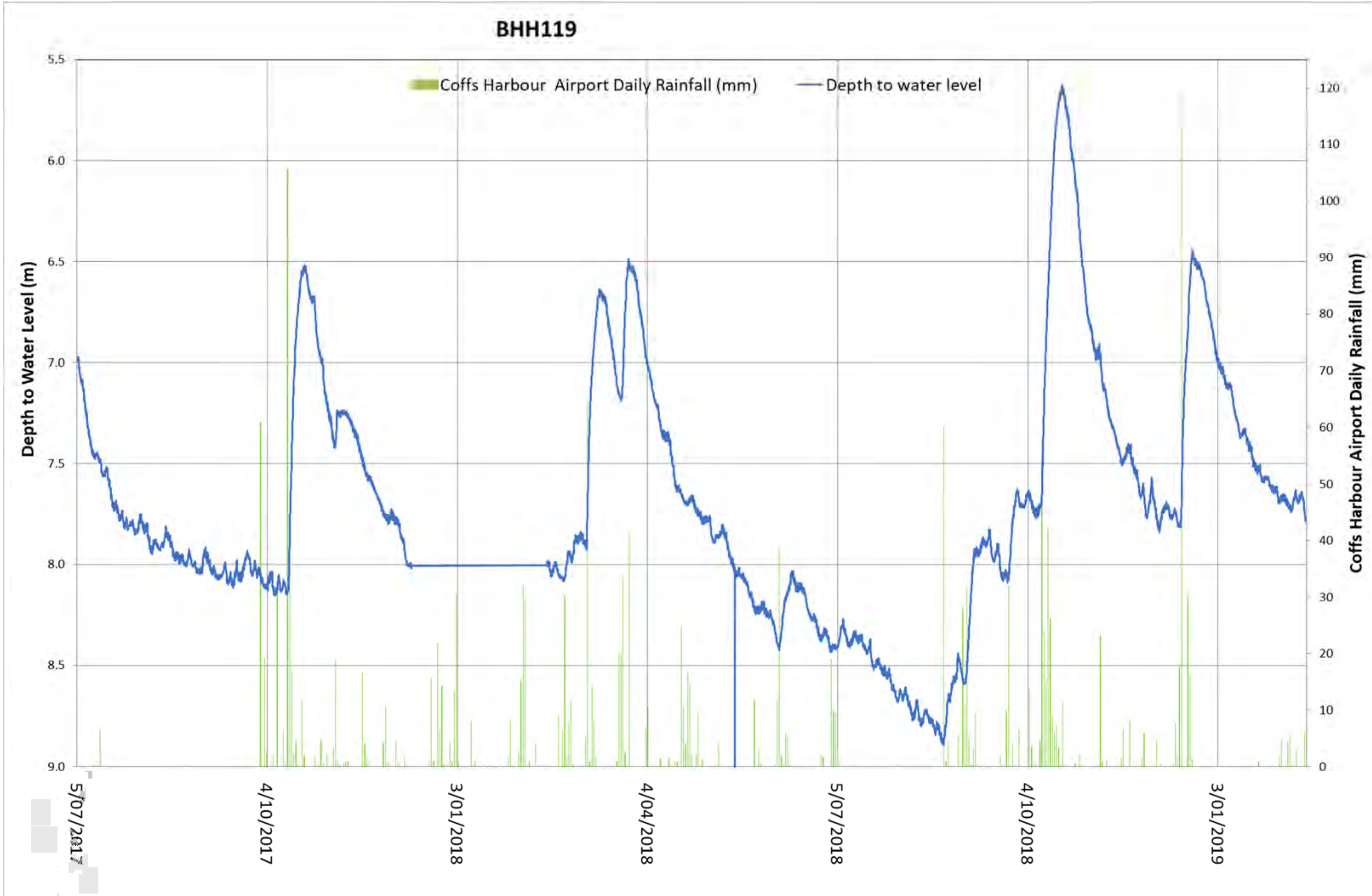
5/07/2018
4/10/2018
3/01/2019



BHH119
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Depth to water level
Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
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6.0
6.5
7.0
7.5
8.0
8.5
9.0
120
110
100
90
80
70
60
50
40
30
20
10
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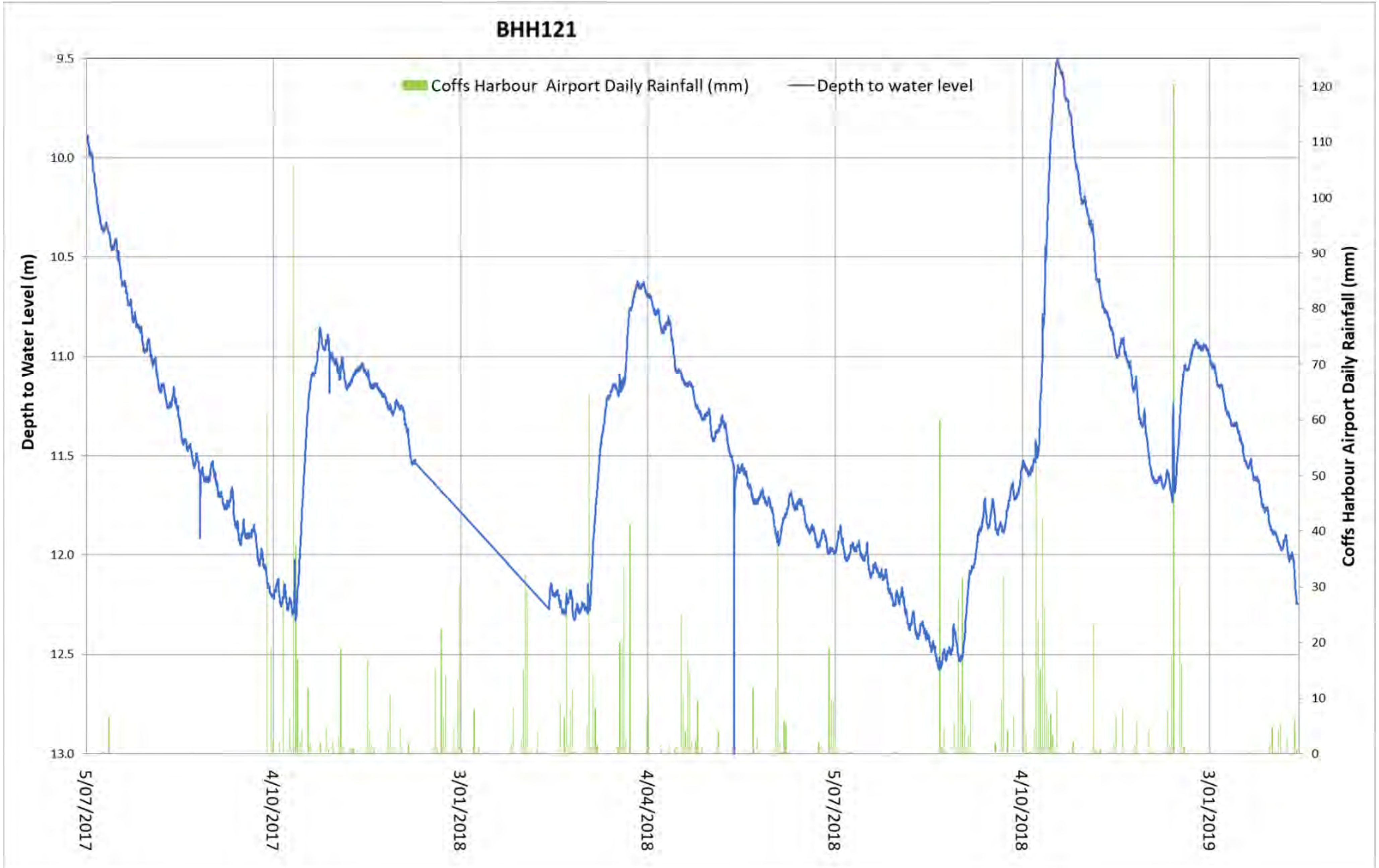
4/10/2017
3/01/2018
4/04/2018
5/07/2018
4/10/2018
3/01/2019



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Coffs Harbour Airport Daily Rainfall (mm)
Depth to water level
Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
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10.0
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11.0
11.5
12.0
12.5
13.0
120
110
100
90
80
70
60
50
40
30
20
10
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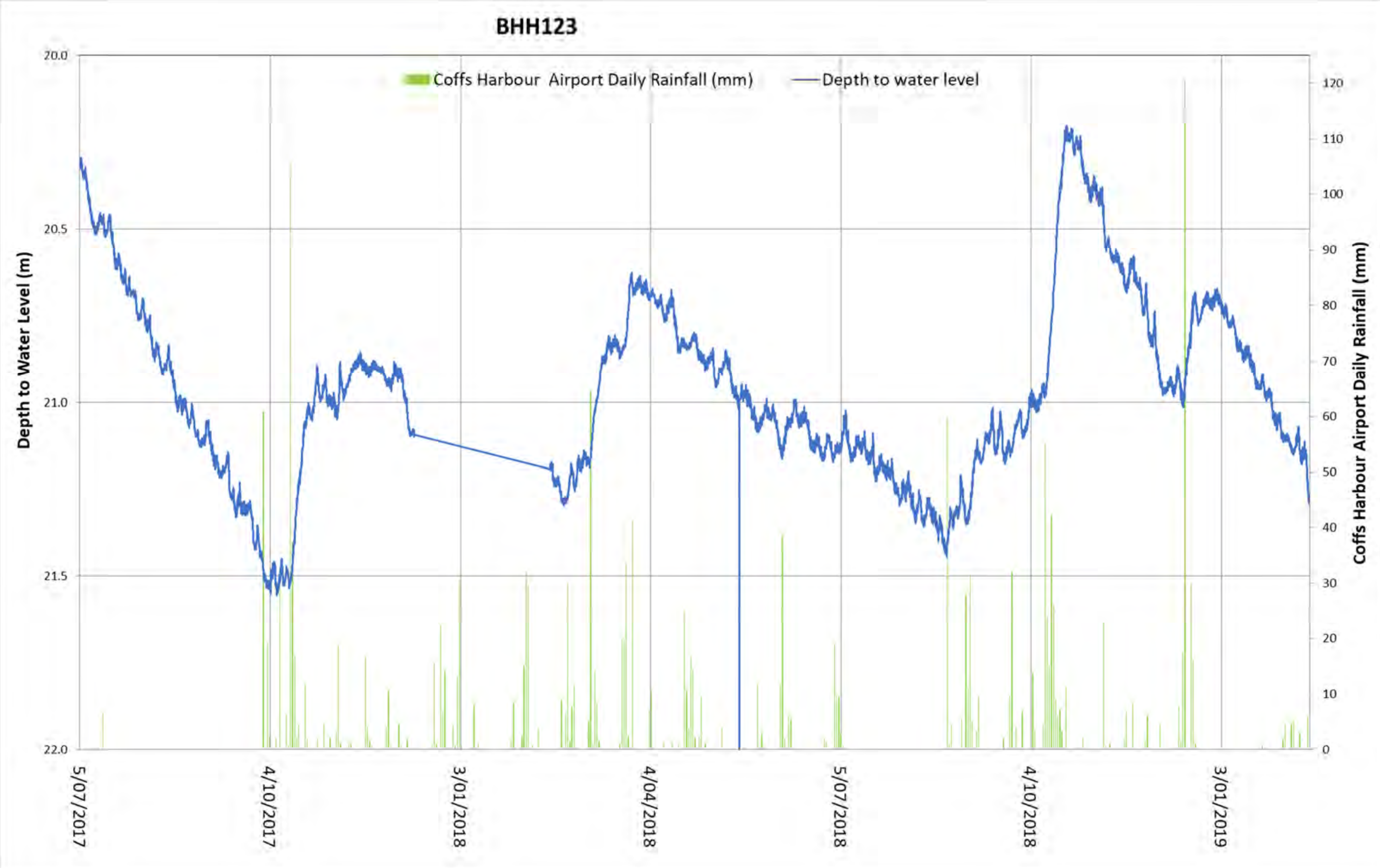
4/10/2017
3/01/2018
4/04/2018
5/07/2018
4/10/2018
3/01/2019



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Depth to Water Level (m)
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21.5
22.0
120
110
100
90
80
70
60
50
40
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10
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4/04/2018
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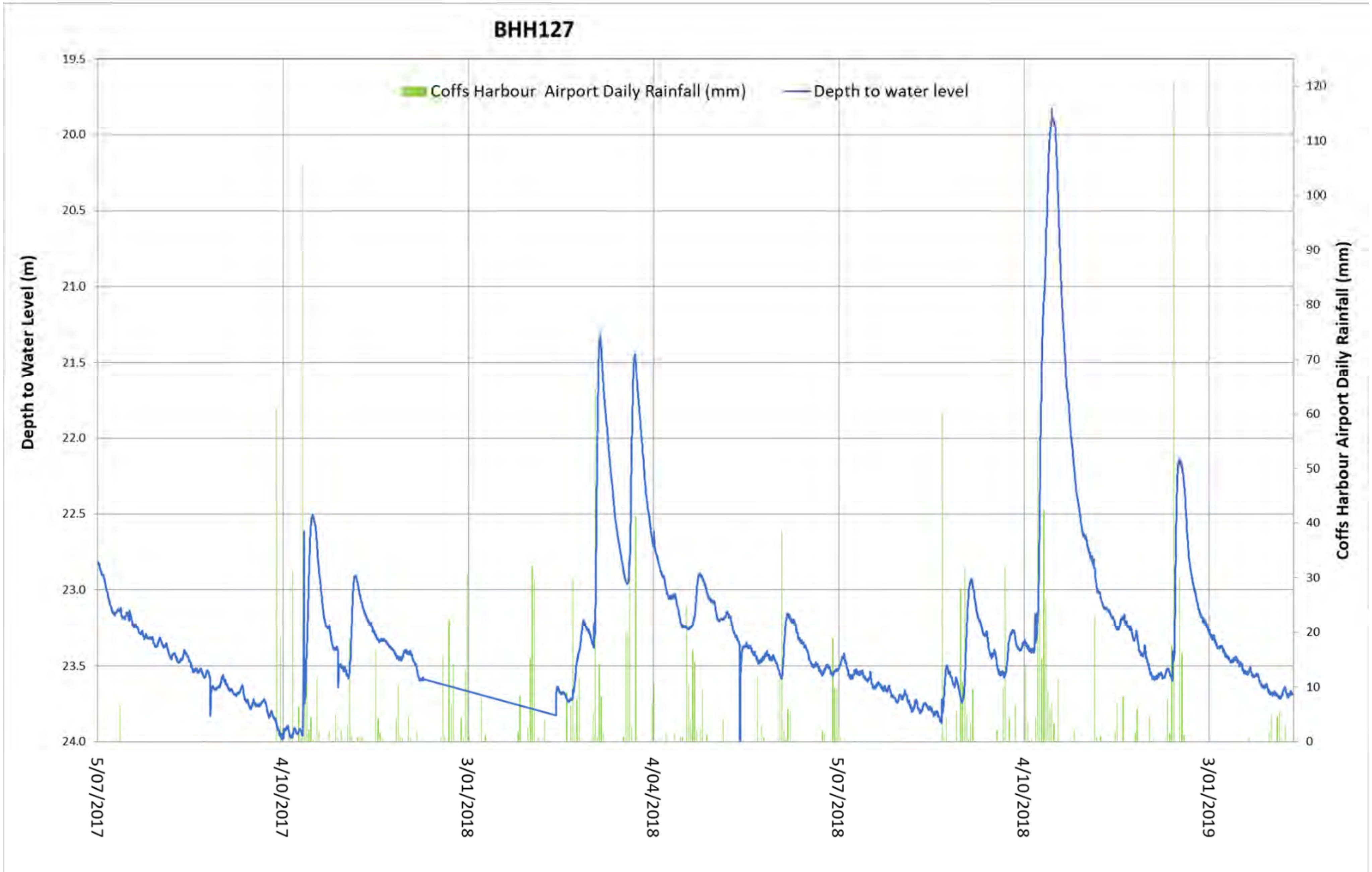
3/01/2019



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22.0
22.5
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23.5
24.0
120
110
100
90
80
70
60
50
40
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3/01/2018
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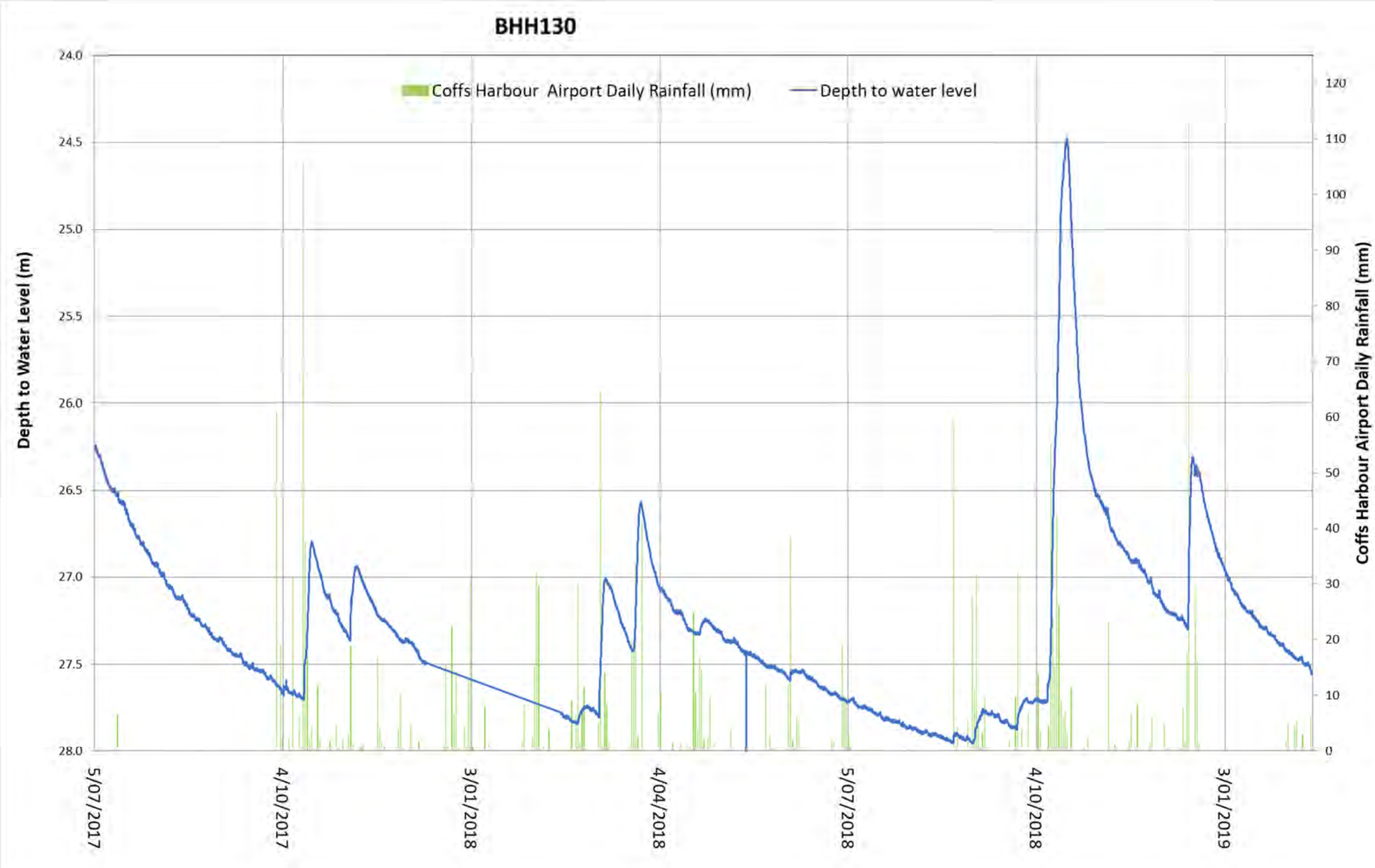
4/10/2018
3/01/2019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Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
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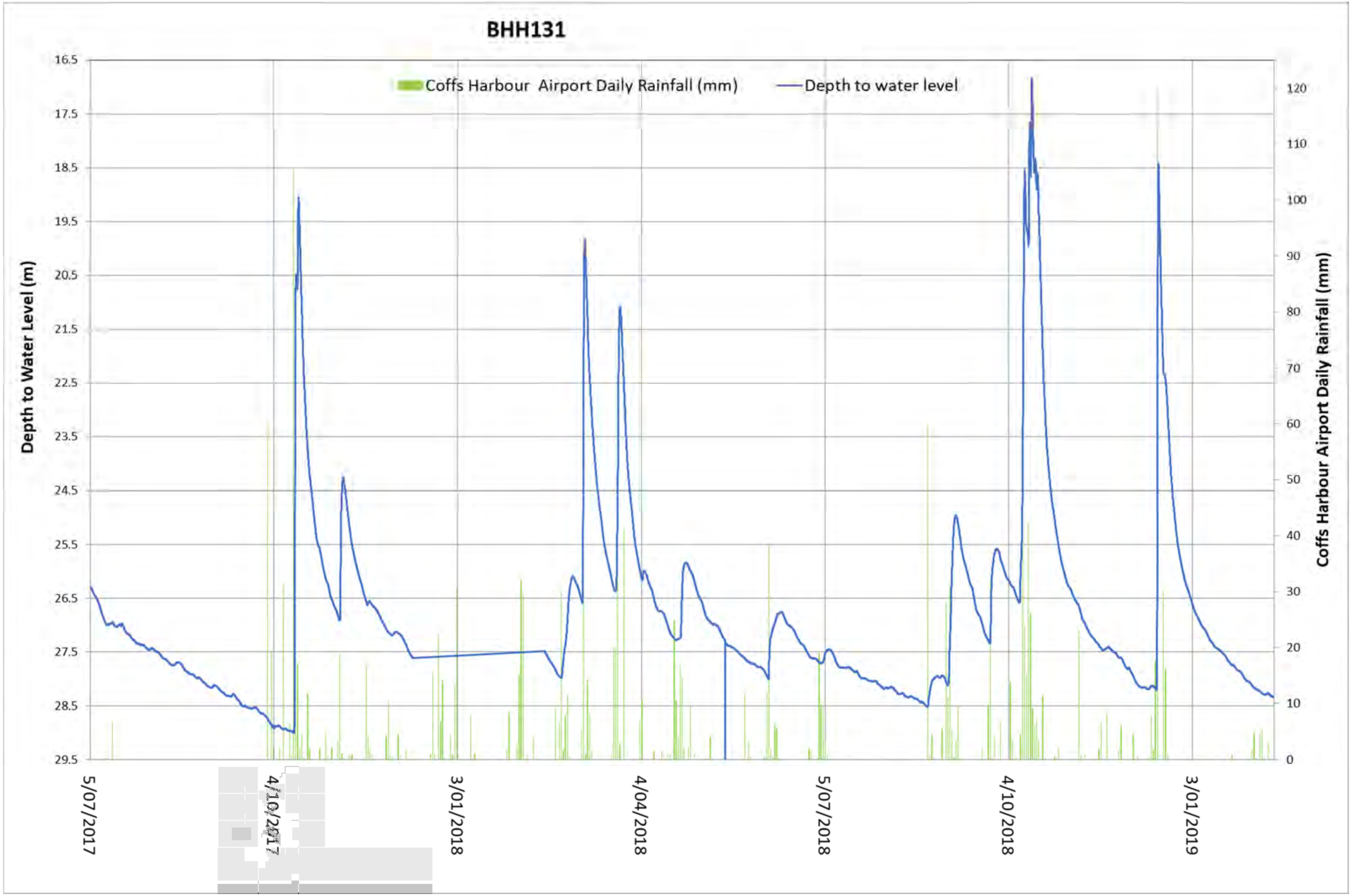
3/01/2019


BHH131
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Depth to Water Level (m)
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19.5
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24.5
25.5
26.5
27.5
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70
60
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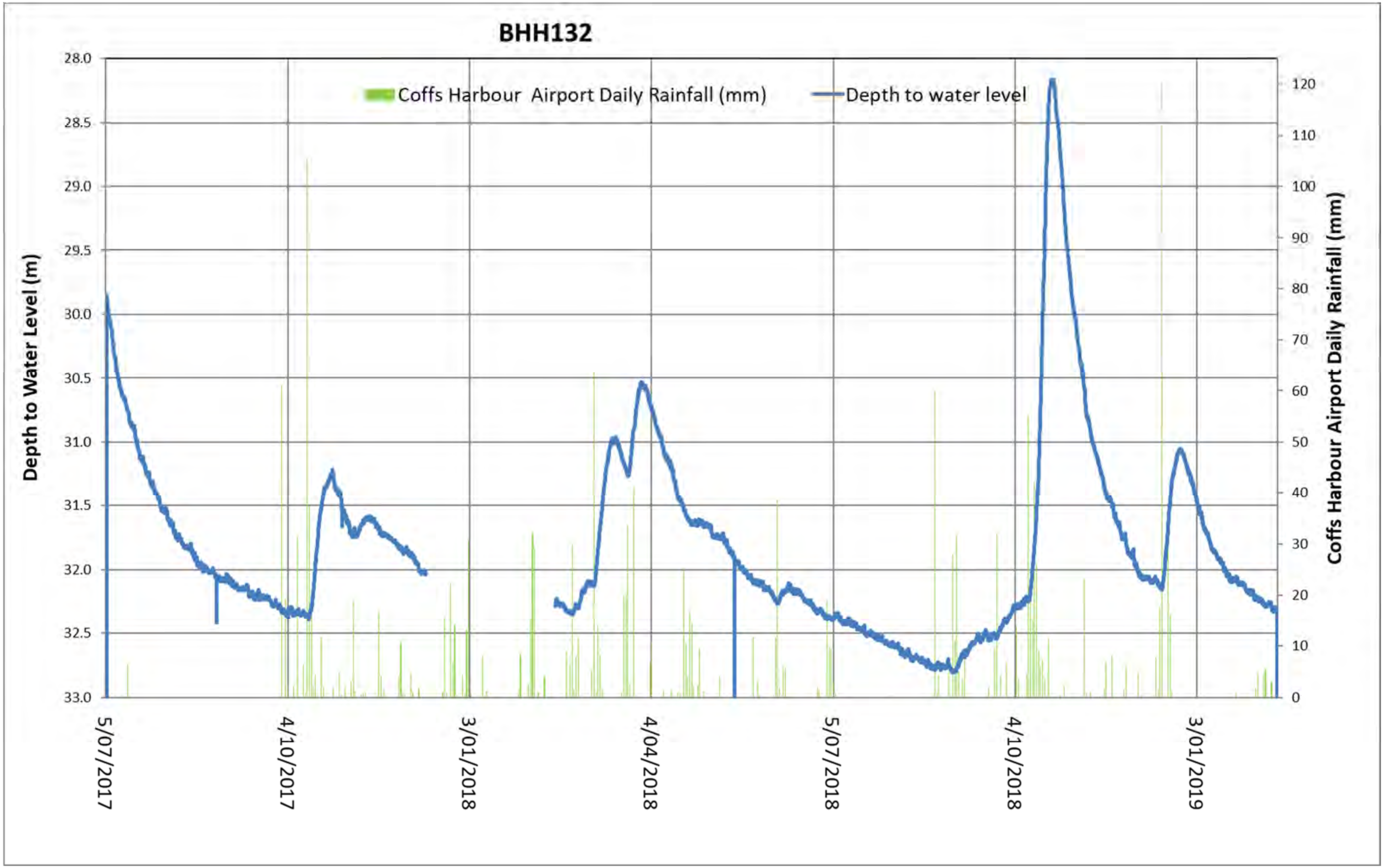
3/01/2019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Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
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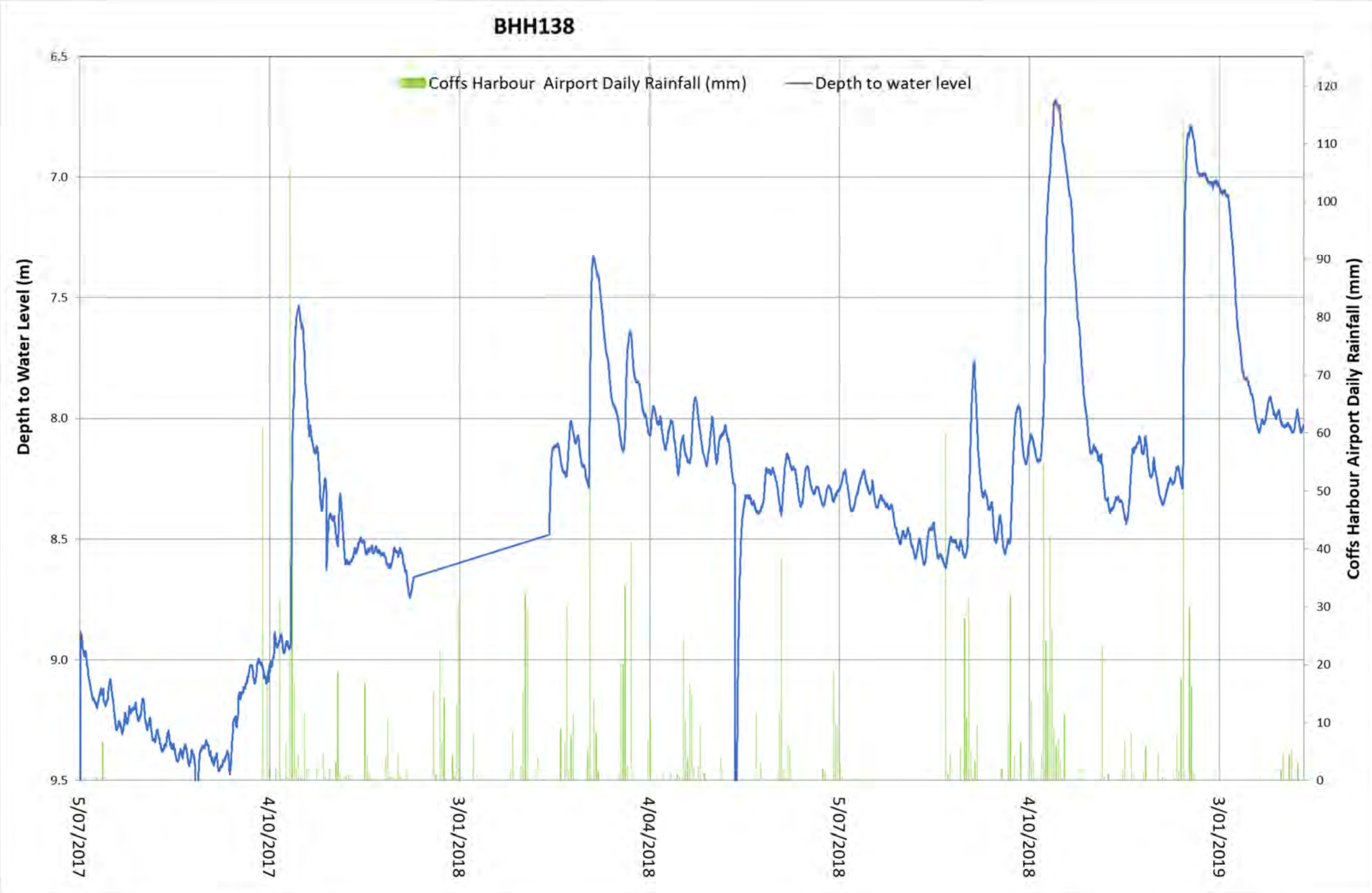
3/01/2019



BHH138
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Depth to water level
Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
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8.5
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9.5
0
10
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70
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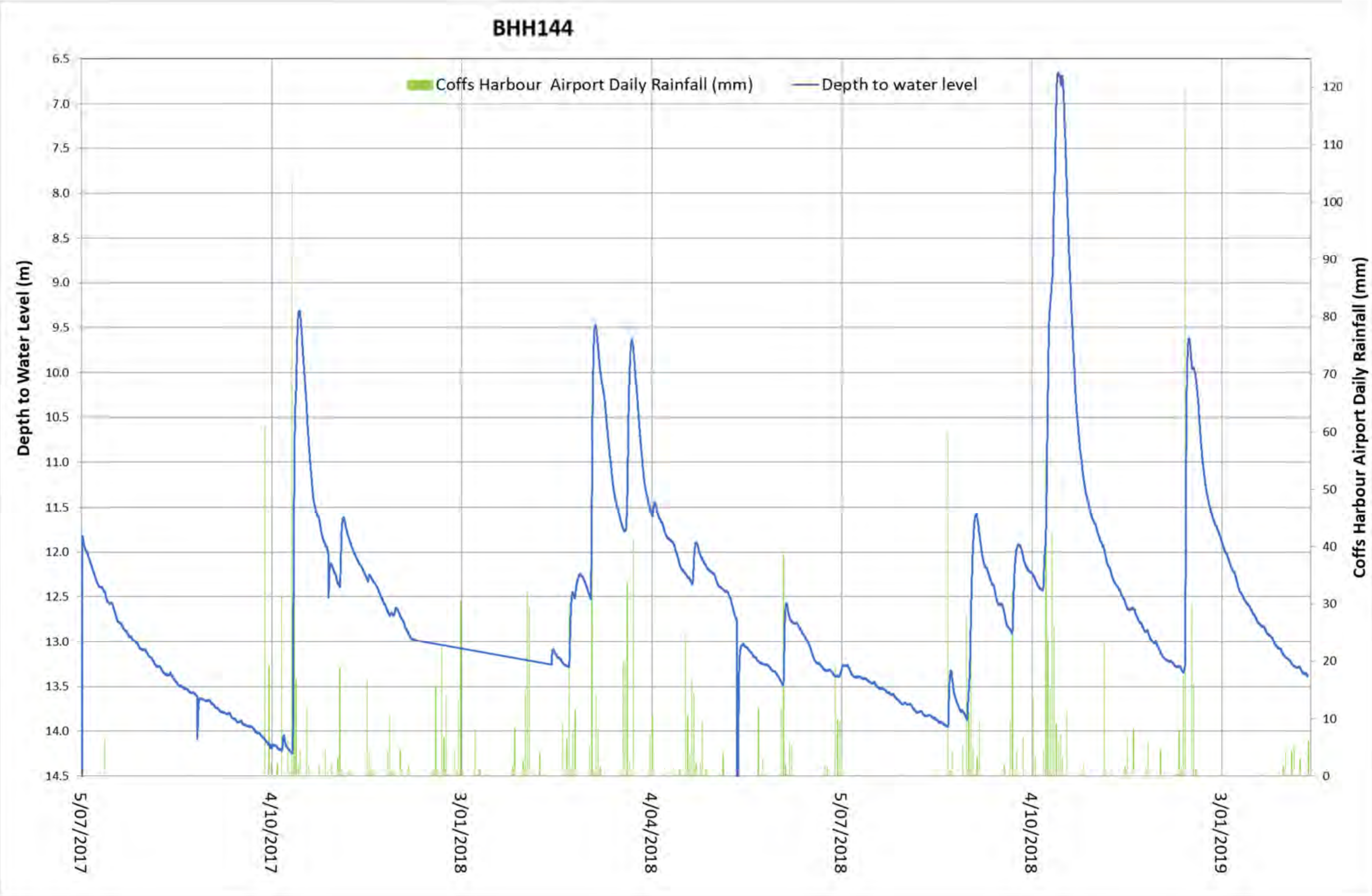
3/01/2019



BHH144
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Depth to water level
Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
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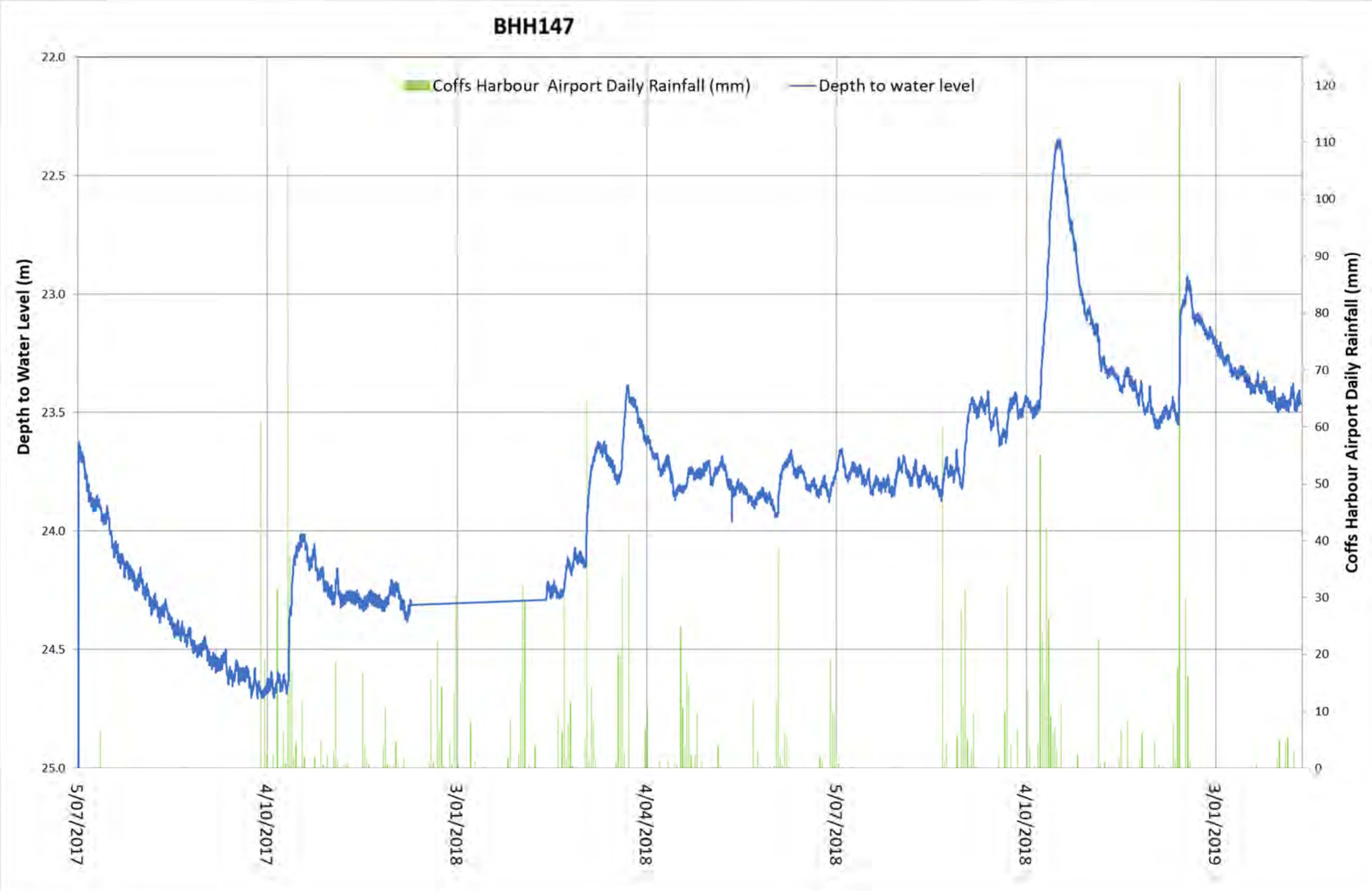
4/04/2018
5/07/2018
4/10/2018
3/01/2019



BHH147
Coffs Harbour Airport Daily Rainfall (mm)
Depth to water level
Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
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24.0
24.5
25.0
120
110
100
90
80
70
60
50
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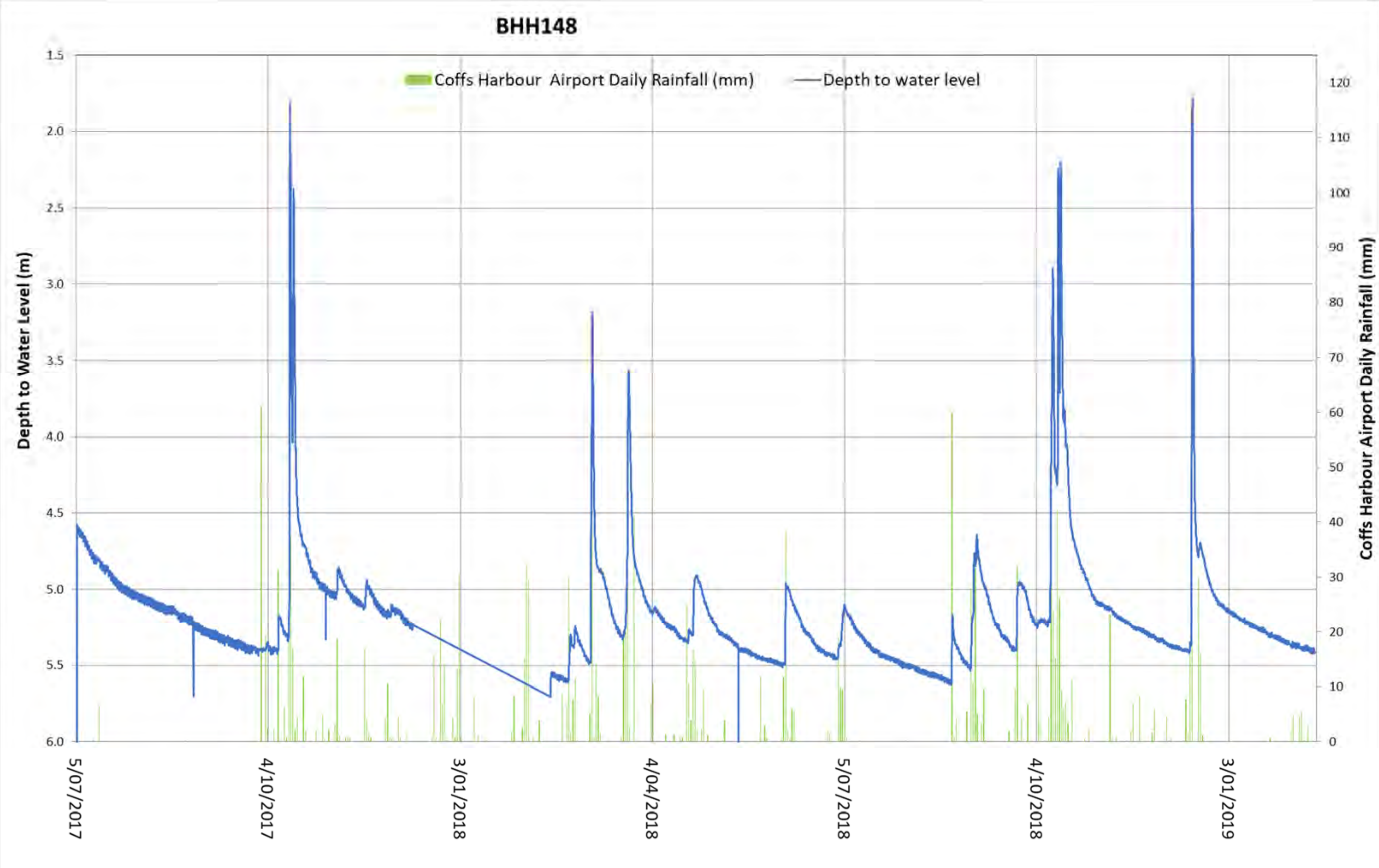
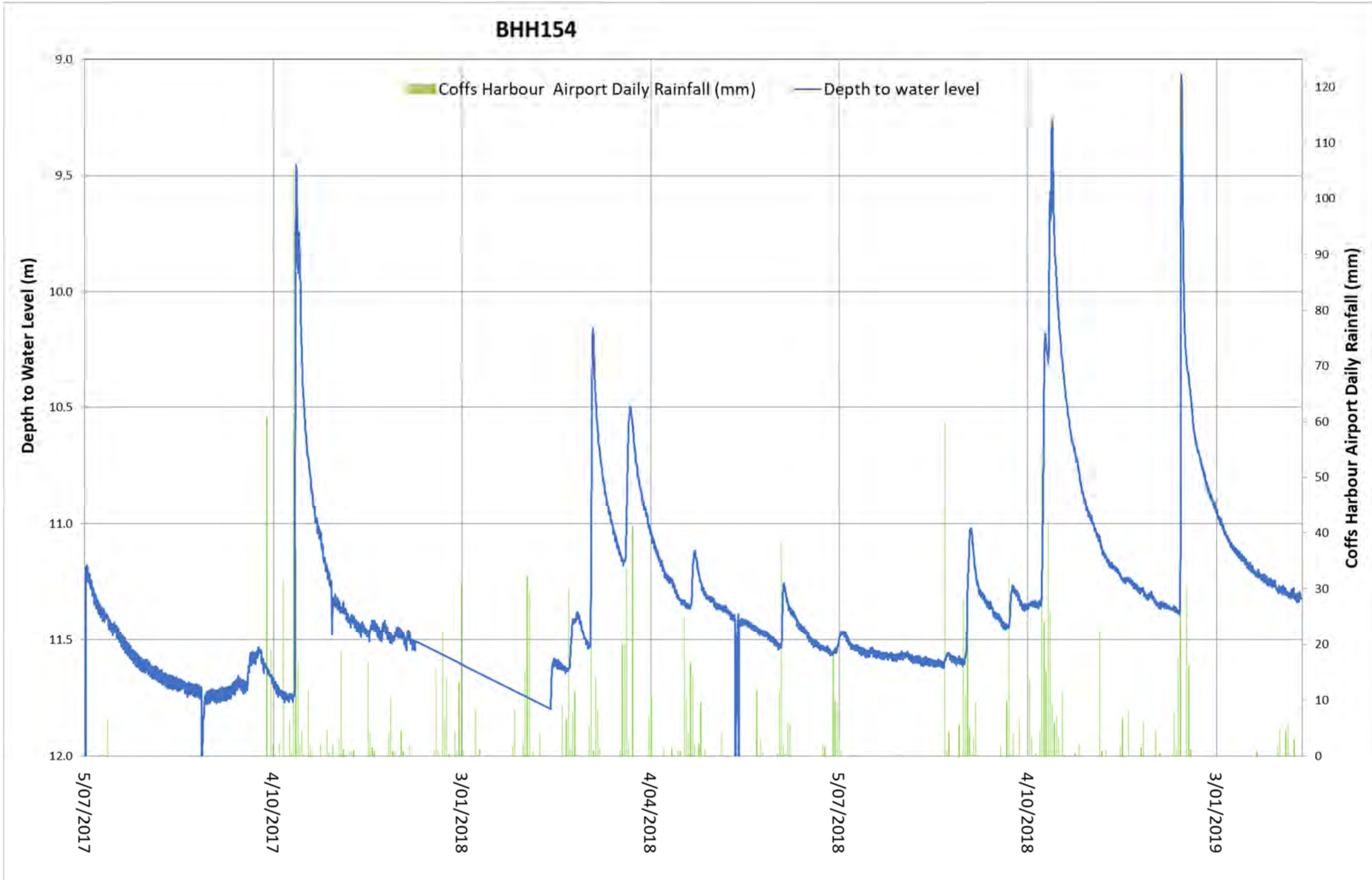
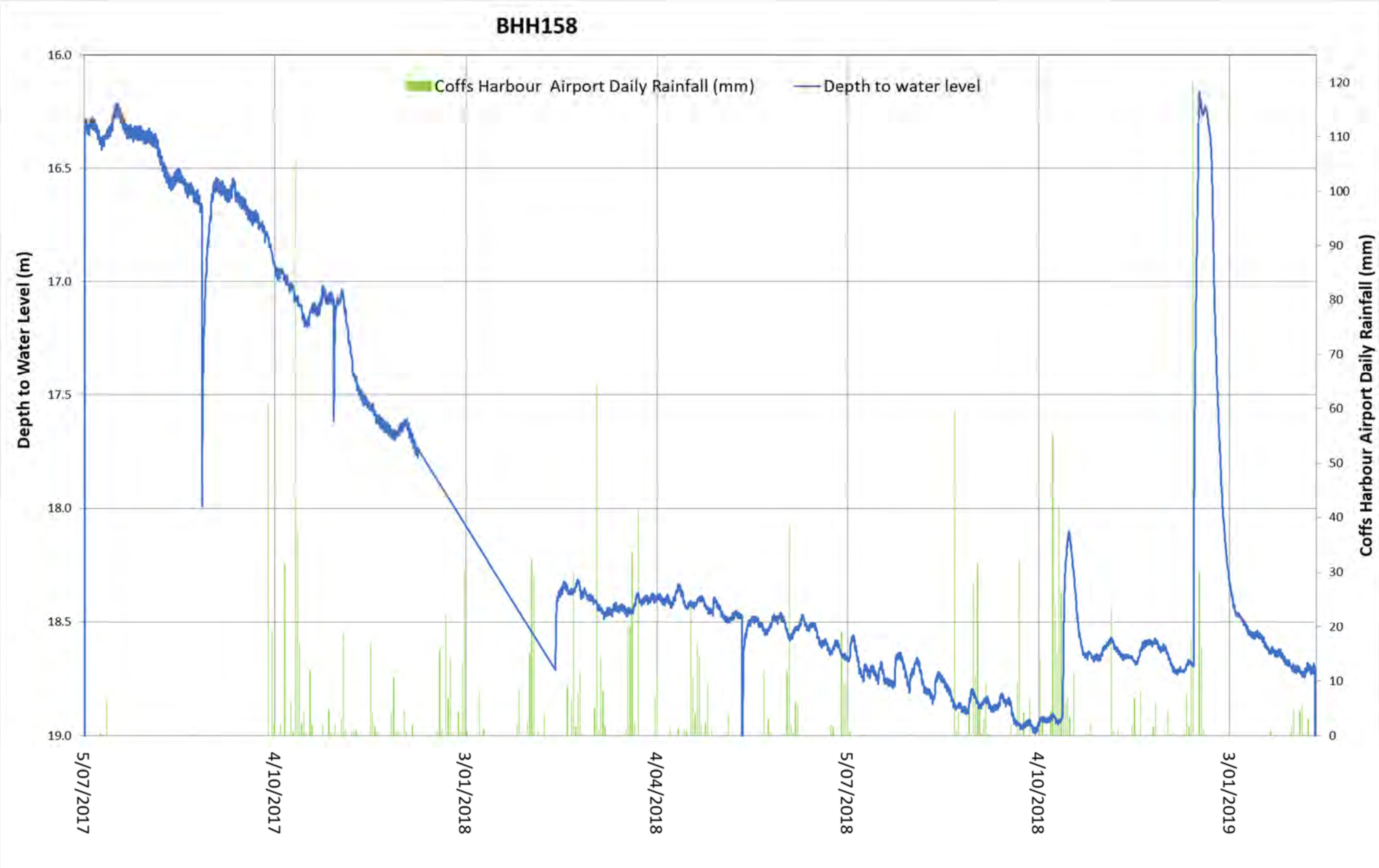
4/10/2018
3/01/2019

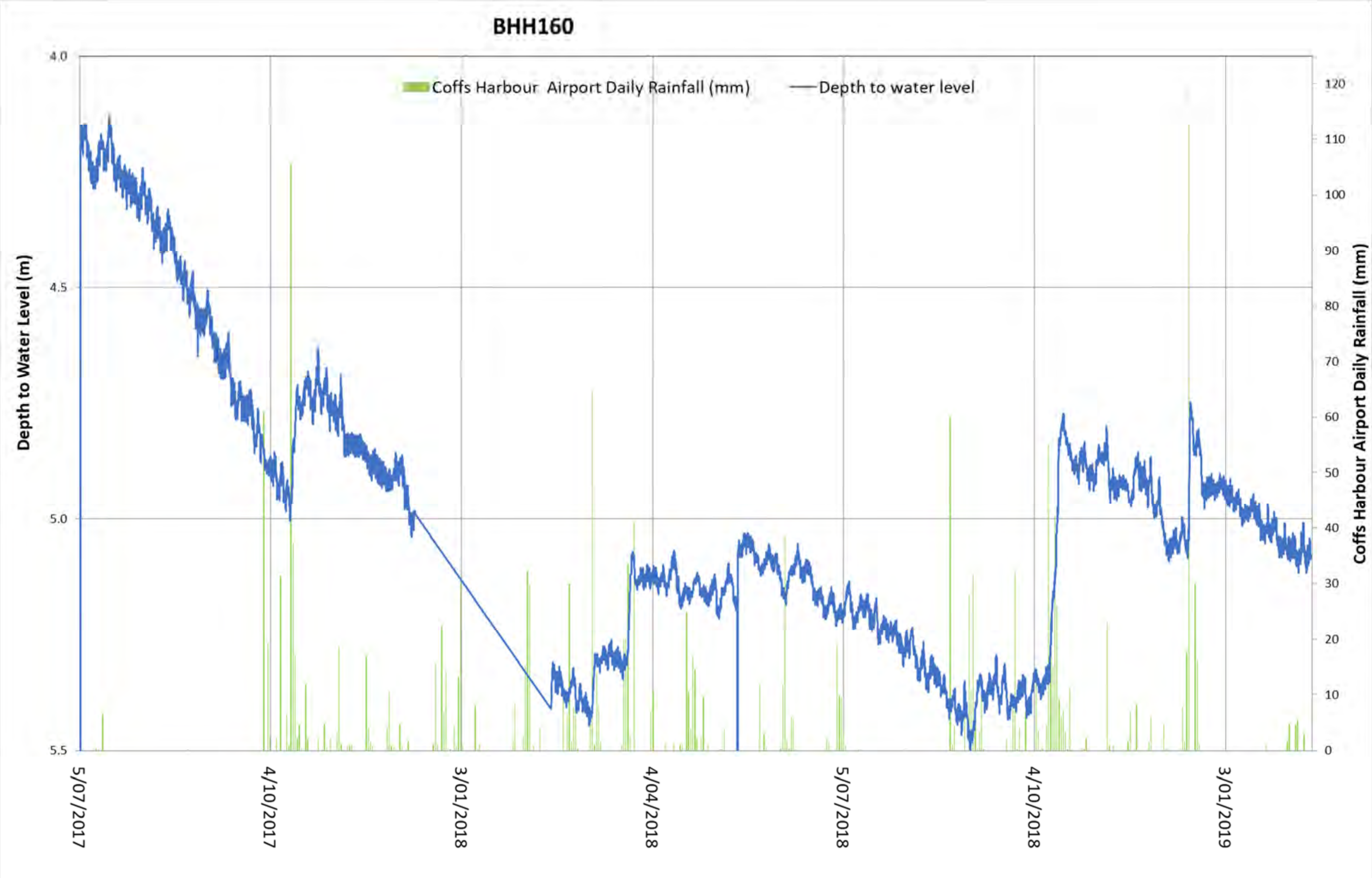


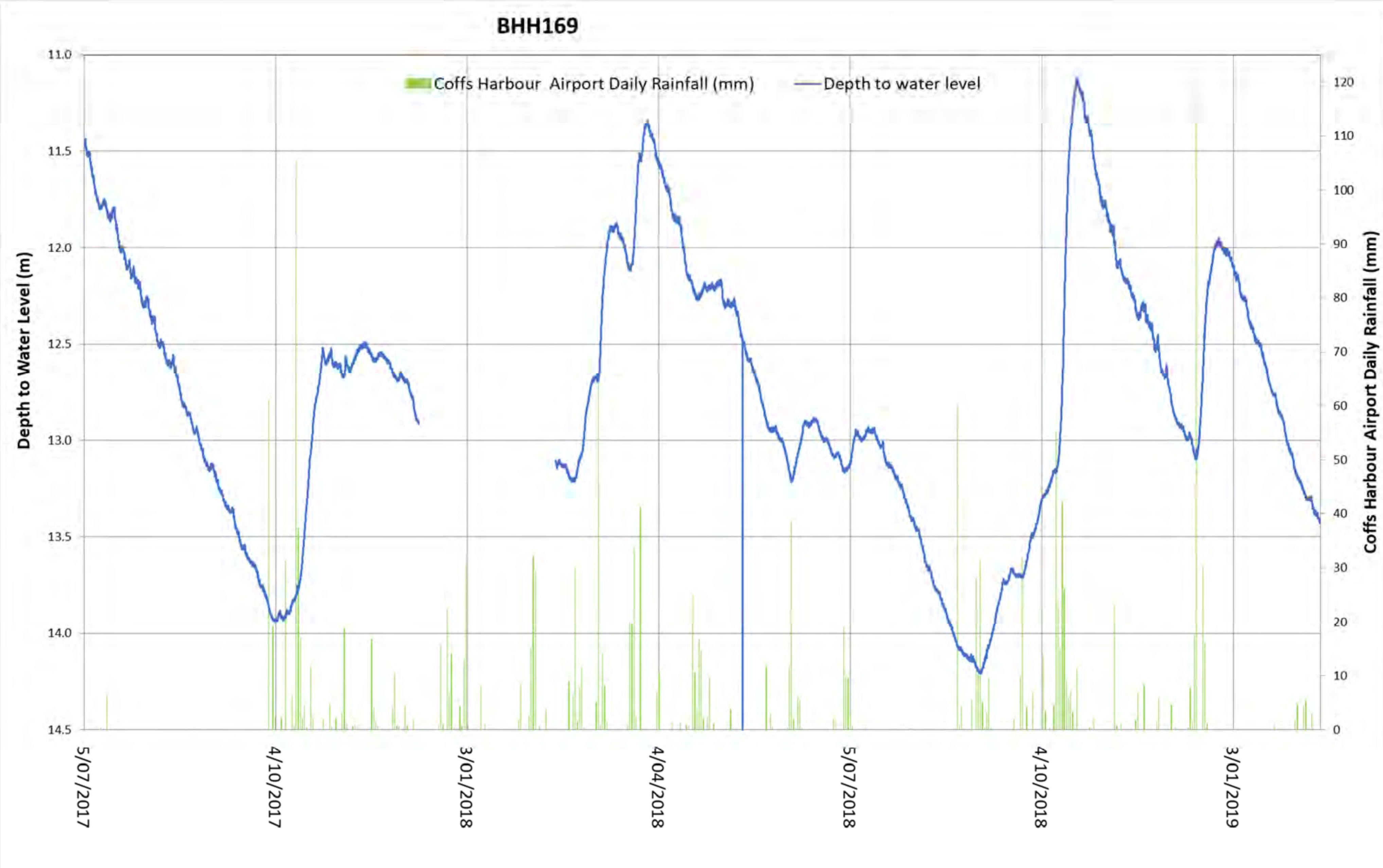
BHH148
Coffs Harbour Airport Daily Rainfall (mm)
Depth to water level
Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
6.0
120
110
100
90
80
70
60
50
40
30
20
10
0
5/07/2017
4/10/2017
3/01/2018
4/04/2018
5/07/2018
4/10/2018
3/01/2019


BHH154
Coffs Harbour Airport Daily Rainfall (mm)
Depth to water level
Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
9.0
9.5
10.0
10.5
11.0
11.5
12.0
120
110
100
90
80
70
60
50
40
30
20
10
0
5/07/2017
4/10/2017
3/01/2018
4/04/2018
5/07/2018
4/10/2018
3/01/2019



BHH158
Coffs Harbour Airport Daily Rainfall (mm)
Depth to water level
Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
16.0
16.5
17.0
17.5
18.0
18.5
19.0
120
110
100
90
80
70
60
50
40
30
20
10
0
5/07/2017
4/10/2017
3/01/2018
4/04/2018
5/07/2018
4/10/2018
3/01/2019


BHH160
Coffs Harbour Airport Daily Rainfall (mm)
Depth to water level
Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
4.0
4.5
5.0
5.5
120
110
100
90
80
70
60
50
40
30
20
10
0
5/07/2017
4/10/2017
3/01/2018
4/04/2018
5/07/2018
4/10/2018
3/01/2019



BHH169
Coffs Harbour Airport Daily Rainfall (mm)
Depth to water level
Depth to Water Level (m)
Coffs Harbour Airport Daily Rainfall (mm)
11.0
11.5
12.0
12.5
13.0
13.5
14.0
14.5
120
110
100
90
80
70
60
50
40
30
20
10
0
5/07/2017
4/10/2017
3/01/2018
4/04/2018
5/07/2018
4/10/2018
3/01/2019

Appendix B

Groundwater Quality Data

Sample Identification	PQL	BHH101	BHN104	BHH106	BHH109	BHH110	BHH110	BHH111	BHH112	BHH112	BHH112	BHH113	BHH114	BHH115	BHH117	BHH119
Location		Cut	North Boambee Overpass	Cut off Lakes Drive	Cut	Roberts Hill	Roberts Hill	Roberts Hill	Roberts Hill	Roberts Hill	Roberts Hill	Roberts Hill	Roberts Hill	Cut	Cut	Cut
SWL in open hole		--	--	24.84	56.21	43.31	43.31	54.7	51.34	51.34	51.34	59.74	43.04	54.85	37.89	--
SWL in piezo		28.86	7.39	26.34	56.88	43.12	43.12	60.55	52.03	52.03	52.03	64.3	49.35	59.09	39.48	44.85
SWL at time of sampling (m)		25.13	7.21	26.91	56.76	44.12	43.59	56.67	54.13	54.13	53.83	62.09	46.29	56.48	37.09	44.7
Volume Removed Before Sampling* (L)		80	300	260	320	160	315	345	250	330	490	190	195	280	130	300
Date of sampling		10/5/17	27/4/17	10/5/17	10/5/17	20/4/17	17/5/17	17/5/17	5/5/17	18/5/17	30/5/17	10/5/17	20/4/17	3/5/17	3/5/17	3/5/17
Sample collected by		RCA - JH	RCA - JH	RCA - JH	RCA - JH	RCA - JH Sample Rejected due to field pH	RCA - JH	RCA - JH	Not Sampled	Not Sampled	RCA - JH	RCA - JH	RCA - JH	RCA - JH	RCA - JH	RCA - JH
Laboratory - RCA																
pH (pH unit)	--	7.04	6.42	6.62	6.61	--	7.03	6.76	--	--	6.89	6.18	6.37	7.04	6.75	7.47
Total Dissolved Solids	5	--	437	--	--	--	238	--	--	--	--	--	199	--	--	--
Conductivity (μS/cm)	1	534	707	299	383	--	565	770	--	--	490	251	259	516	388	363
Turbidity (NTU)	1	14	<1	11	12	--	19	11	--	--	6	2	2	11	75	14
Hydroxide Alkalinity as $CaCO_3$	1	<1	<1	<1	<1	--	<1	<1	--	--	<1	<1	<1	<1	<1	<1
Carbonate Alkalinity as $CaCO_3$	1	<1	<1	<1	<1	--	<1	<1	--	--	<1	<1	<1	<1	<1	<1
Bicarbonate Alkalinity as $CaCO_3$	1	240	84	85	93	--	96	117	--	--	111	49	45	162	128	189
Total Alkalinity as $CaCO_3$	1	240	84	85	93	--	96	117	--	--	111	49	45	162	128	189
Sulphate as SO_4	1	15	40	41	23	--	24	236	--	--	50	16	35	28	82	17
Dissolved Oxygen	1	6.2	--	6.6	6.9	--	--	--	--	--	9.3	7.1	--	8.6	7.6	7.5
Salinity (%)	--	0.03	--	0.01	0.02	--	--	--	--	--	0.02	0.01	--	0.01	0.01	0.01
Laboratory - SGS																
Chloride	0.05	--	130	--	--	--	25	26	--	--	32	28	19	57	24	21
Sulphate as SO_4	1	--	23	--	--	--	22	210	--	--	44	15	27	38	60	8.2
Sulphite as SO_2	2	--	<2	--	--	--	<2	--	--	--	--	--	<2	--	--	--
Total Calcium	0.1	--	26	--	--	--	18	41	--	--	23	7.8	13	52	20	31
Total Magnesium	0.1	--	11	--	--	--	8	30	--	--	12	4.5	6	9.8	10	5.8
Total Sodium	0.1	--	86	--	--	--	24	42	--	--	35	24	24	34	41	41
Total Potassium	0.2	--	2	--	--	--	3.8	7.7	--	--	3.5	2	2.3	3.1	3.9	7.3
Total Zinc	5	--	52	--	--	--	26	--	--	--	--	--	19	--	--	--
Total Manganese	1	--	490	--	--	--	3600	--	--	--	--	--	51	--	--	--
Total Aluminium	5	--	130	--	--	--	210	340	--	--	600	73	37	360	380	720
Total Iron	5	--	600	--	--	--	360	240	--	--	480	17	26	790	2000	1400
Field																
pH	n/a	7.05	6.56	6.99	6.78	9.58[#]	6.91	6.47	11.1[#]	~8	7.21	6.5	6.34	7.07	6.69	7.71
Electrical Conductivity (mS/cm)	n/a	0.544	0.703	0.277	0.389	0.256	0.317	0.653	0.591	--	6.43	0.234	0.269	0.558	0.415	0.439
Turbidity (NTU)	n/a	11	15	12	20	12	6	5	95	--	7	5	6	3	19	17
Dissolved Oxygen	n/a	2.47	2.31	1.21	3.2	6	3.75	2.85	5.3	--	3.62	1.58	4.45	2.05	1.62	1.7
Temperature (^{o}C)	n/a	20.3	22	19.6	19.1	19.6	18.9	19.2	18.9	--	18.7	18.7	21	19.3	19.5	20.5

All results are in units of mg/L unless otherwise stated

Blank Cell indicates no criterion available

PQL = Practical Quantitation Limit

* Where more than one sampling event, volume is accumulative

[#] Field results included to show effect of grout, in combination with low rock permeability and limited submergence of piezometer (i.e. available groundwater for sampling). Values are not considered representative of groundwater quality.

Sample Identification	PQL	BHH121	BHH123	BHH125	BHH127	BHH130	BHH131	BHH132	BHH138	BHH138	BHH138	Spring	BHH140	BHH142	BHH142	BHH144	BHH147
Location		Cut	Cut	Cut	Cut	Cut	Cut	Cut	Shephards Lane	Shephards Lane	Shephards Lane	Shephards Lane	Shephards Lane	Shephards Lane	Shephards Lane	Shephards Lane	Cut
SWL in open hole		22.11	37.97	44.14	56.12	57.6	69.96	61.89	94.74	94.74	94.74	NA	118.18	132.98	132.98	93.84	72.67
SWL in piezo		24.19	38.77	45.23	52.42	59.84	69.69	62.76	91.09	91.09	91.09	NA	122.86	134.42	134.42	94.26	73.86
SWL at time of sampling (m)		23.65	36.52	44.59	52.64	58.78	69.38	62.99	87.31	87.5	not measured	NA	118.65	130.51	132.14	93.6	73.47
Volume Removed Before Sampling* (L)		155	340	160	305	350	340	170	42	92	122	NA	180	220	510	110	270
Date of sampling		3/5/17	3/5/17	3/5/17	3/5/17	3/5/17	3/5/17	10/5/17	27/4/17	17/5/17	30/5/17	17/5/17	27/4/17	17/5/17	30/5/17	27/4/17	27/4/17
Sample collected by		RCA - JH	RCA - JH	RCA - JH	RCA - JH	RCA - JH	RCA - JH	RCA - JH	RCA - JH	RCA - JH Sample rejected due to field pH	Not Sampled	RCA - JH	RCA - JH	Not Sampled	Not Sampled	RCA - JH	RCA - JH
Laboratory - RCA																	
pH (pH unit)	--	6.29	6.38	6.18	6.83	6.19	5.74	7.03	--	--	--	6.3	7.17	--	--	7.87	6.37
Total Dissolved Solids	5	--	--	--	--	--	--	--	--	--	--	86	--	--	--	210	--
Conductivity (µS/cm)	1	226	490	227	313	178	168	514	--	--	--	262	370	--	--	279	410
Turbidity (NTU)	1	13	138	32	182	11	428	4	--	--	--	2	9	--	--	<1	<1
Hydroxide Alkalinity as $CaCO_3$	1	<1	<1	<1	<1	<1	<1	<1	--	--	--	<1	<1	--	--	<1	<1
Carbonate Alkalinity as $CaCO_3$	1	<1	<1	<1	<1	<1	<1	<1	--	--	--	<1	<1	--	--	11	<1
Bicarbonate Alkalinity as $CaCO_3$	1	50	86	55	89	18	18	157	--	--	--	8	121	--	--	63	69
Total Alkalinity as $CaCO_3$	1	50	86	55	89	18	18	157	--	--	--	8	121	--	--	74	69
Sulphate as SO_4	1	23	32	32	35	33	16	17	--	--	--	8	27	--	--	26	121
Dissolved Oxygen	1	7.9	8	7.6	7.5	9.9	8.9	8.8	--	--	--	--	9.6	--	--	--	2.5
Salinity (%)	--	0	0.01	0	0.01	0	0	0.02	--	--	--	--	0.33	--	--	--	0.02
Laboratory - SGS																	
Chloride	0.05	32	46	14	29	15	21	31	--	--	--	12	26	--	--	19	22
Sulphate as SO_4	1	14	42	37	36	39	14	20	--	--	--	8.2	25	--	--	26	94
Sulphite as SO_2	2	--	--	--	--	--	--	--	--	--	--	<2	--	--	--	<2	--
Total Calcium	0.1	3.1	13	6.8	12	2	1.7	50	--	--	--	1.2	29	--	--	20	15
Total Magnesium	0.1	2.3	13	4.8	7	4	5.3	7.8	--	--	--	3	7.8	--	--	5.4	7.6
Total Sodium	0.1	34	37	27	33	19	18	24	--	--	--	9.8	28	--	--	23	43
Total Potassium	0.2	0.7	5.8	3.3	3.6	1.2	2.1	2	--	--	--	3.3	3.1	--	--	2.9	2.1
Total Zinc	5	--	--	--	--	--	--	--	--	--	--	11	--	--	--	43	--
Total Manganese	1	--	--	--	--	--	--	--	--	--	--	6	--	--	--	53	--
Total Aluminium	5	190	1100	600	1800	69	2100	61	--	--	--	56	300	--	--	550	290
Total Iron	5	270	1000	790	2800	100	2800	130	--	--	--	42	180	--	--	280	2400
Field																	
pH	n/a	6.09	6.36	6.11	6.61	5.68	5.63	7.12	12.51[#]	11.17[#]	12.88[#]	6.25	6.82	12.55[#]	12.97[#]	7.9	6.28
Electrical Conductivity (mS/cm)	n/a	0.247	0.441	0.248	0.348	0.186	0.191	0.449	5.11	1.54	4.47	0.135	0.377	7.33	6.18	0.24	0.427
Turbidity (NTU)	n/a	5	140	86	250	9	500	2	30	20	14	0	64	9	7	15	25
Dissolved Oxygen	n/a	1.2	2.75	4.05	4.3	4.5	4.6	2.11	2.03	5.7	3.61	6.18	2.7	1.6	4.72	3.45	1.05
Temperature (°C)	n/a	19.9	19.7	19.8	19.4	19	19.7	19.1	20.8	19.9	19.1	17	19.7	17.7	18.7	20.6	18.7

All results are in units of mg/L unless otherwise stated

Blank Cell indicates no criterion available

PQL = Practical Quantitation Limit

* Where more than one sampling event, volume is accumulative

[#] Field results included to show effect of grout, in combination with low rock permeability and limited submergence of piezometer (i.e. available groundwater for sampling). Values are not considered representative of groundwater quality.

Sample Identification	PQL	BHH148	BHH148	BHH150	BHH153	BHH154	BHH154[$]	BHH158	BHH160	BHH163	BHH169
Location		Gatelys Road	Gatelys Road	Gatelys Road	Gatelys Road	Gatelys Road	Gatelys Road	Cut	Cut	Cut	Waste Facility at Southern End
SWL in open hole		79.7	79.7	113.99	113.66	83.09	83.09	63.9	56.32	39.61	15.34
SWL in piezo		76.01	76.01	113.5	108.42	75.82	75.82	68.85	56.77	39.53	16.87
SWL at time of sampling (m)		75.88	75.5	133.1	108.7	75.25	75.25	66.63	57.4	39.57	16.42
Volume Removed Before Sampling* (L)		100	465	220	200	70	230	110	250	NA	170
Date of sampling		27/4/17	Not Sampled	24/5/17	24/5/17	27/4/17	1/6/17	10/5/17	10/5/17	Not Sampled	20/4/17
Sample collected by		RCA - JH Sample rejected due to field pH	RCA - JH	RCA - JH	RCA - JH	RCA - JH Sample rejected due to field pH	RCA - JH	RCA - JH	RCA - JH	RCA - JH	RCA - JH
Laboratory - RCA											
pH (pH unit)	--	--	--	7.45	7.53	--	7.57	6.49	7.14	--	6.56
Total Dissolved Solids	5	--	--	--	--	--	262	--	--	--	--
Conductivity (μS/cm)	1	--	--	1167	543	--	446	433	781	--	1174
Turbidity (NTU)	1	--	--	15	12	--	6	12	28	--	<1
Hydroxide Alkalinity as $CaCO_3$	1	--	--	<1	<1	--	<1	<1	<1	--	<1
Carbonate Alkalinity as $CaCO_3$	1	--	--	<1	<1	--	<1	<1	<1	--	<1
Bicarbonate Alkalinity as $CaCO_3$	1	--	--	186	201	--	119	62	265	--	571
Total Alkalinity as $CaCO_3$	1	--	--	186	201	--	119	62	265	--	571
Sulphate as SO_4	1	--	--	51	19	--	58	22	23	--	122
Dissolved Oxygen	1	--	--	9.1	9.1	--	--	7.6	8.8	--	6.5
Salinity (%)	--	--	--	0.03	0.03	--	--	0.02	0.04	--	0.08
Laboratory - SGS											
Chloride	0.05	--	--	25	41	--	29	37	100	--	110
Sulphate as SO_4	1	--	--	39	19	--	37	26	25	--	120
Sulphite as SO_2	2	--	--	--	--	--	<2	--	--	--	--
Total Calcium	0.1	--	--	48	64	--	35	14	89	--	44
Total Magnesium	0.1	--	--	12	9.7	--	6.8	8.6	13	--	88
Total Sodium	0.1	--	--	32	29	--	30	25	51	--	150
Total Potassium	0.2	--	--	5.2	3	--	6.7	3.4	2.3	--	36
Total Zinc	5	--	--	--	--	--	25	--	--	--	--
Total Manganese	1	--	--	--	--	--	74	--	--	--	--
Total Aluminium	5	--	--	210	140	--	420	150	430	--	24
Total Iron	5	--	--	670	320	--	510	180	1200	--	29
Field											
pH	n/a	11.21[#]	9.64	7.39	7.35	10.83[#]	7.7	6.5	7.02	--	6.5
Electrical Conductivity (mS/cm)	n/a	0.513	0.352	0.517	0.554	0.509	0.439	0.315	0.854	--	1.66
Turbidity (NTU)	n/a	65	15	16	26	60	24	3	43	--	2
Dissolved Oxygen	n/a	3.88	4.2	1.7	2.46	3.1	2.57	4.87	2.3	--	1.17
Temperature (^oC)	n/a	19.7	19.9	19.8	19.4	20	20	19.5	20.7	--	22.6

All results are in units of mg/L unless otherwise stated

Blank Cell indicates no criterion available

PQL = Practical Quantitation Limit

* Where more than one sampling event, volume is accumulative

[$] Sample was incorrectly identified as BHH112 to external laboratory

[#] Field results included to show effect of grout, in combination with low rock permeability and limited submergence of piezometer (i.e. available groundwater for sampling). Values are not considered representative of groundwater quality.

Appendix C

Groundwater Modelling Reports

C1 RCA Modelling Report

GROUNDWATER MODELLING of MAJOR CUTS

PACIFIC HIGHWAY UPGRADE – COFFS HARBOUR BYPASS

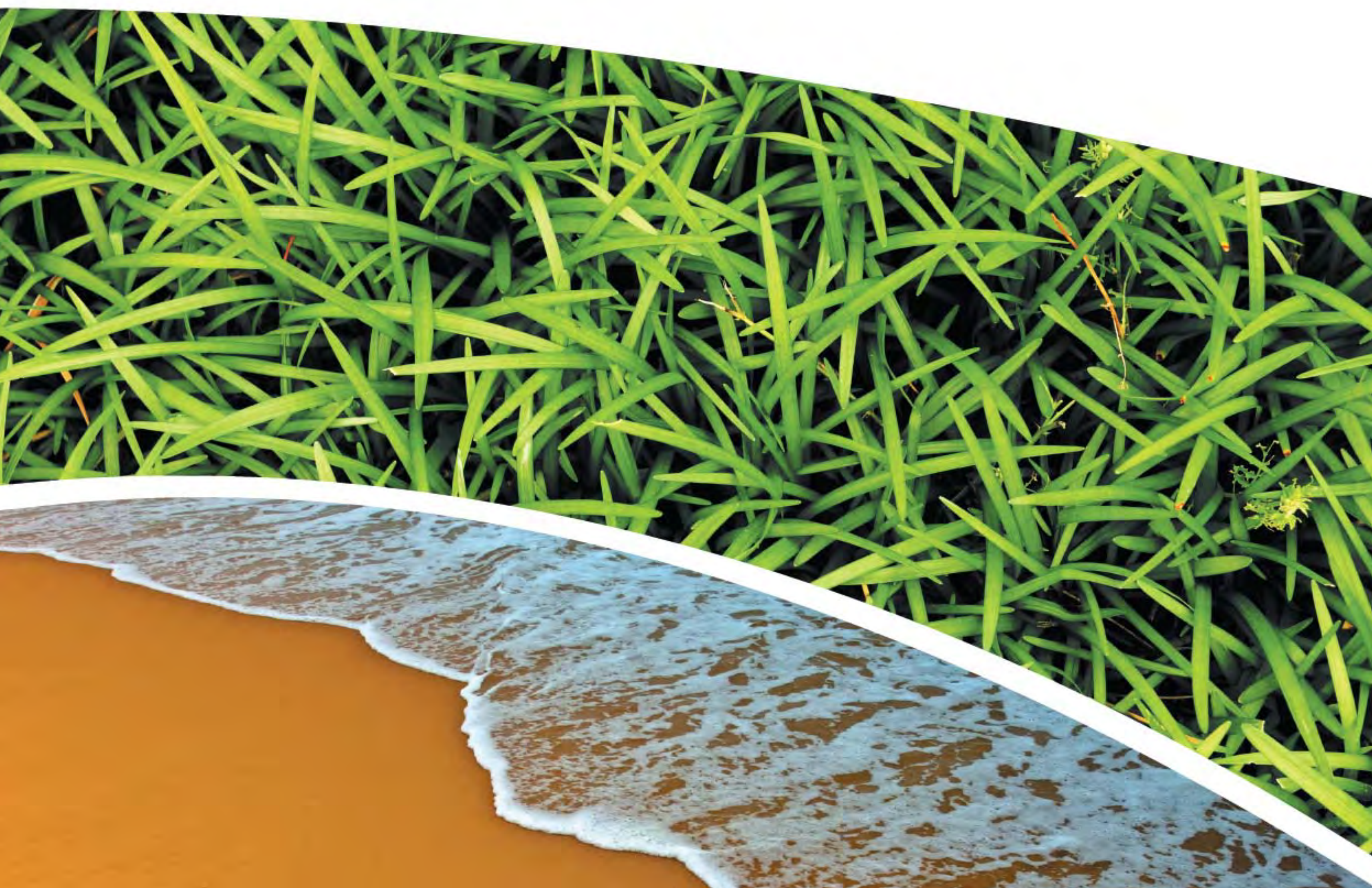
Prepared for RMS NSW

Prepared by RCA Australia

RCA ref 11717-818/2

June 2019

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL



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/1	1	Electronic (email)	Peter.BORRELLI@rms.nsw.gov.au	04.06.19
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Contents

RCA ref 11717-818/2
Client ref 14.2166.0517.0020

GROUNDWATER MODELLING OF MAJOR CUTTINGS
PACIFIC HIGHWAY UPGRADE – COFFS HARBOUR BYPASS

1 INTRODUCTION

The Coffs Harbour Bypass (CHB) includes approximately 14 kilometres of proposed Pacific Highway from Englands Road in the south to the new four-lane divided highway at Sapphire in the north (Roads and Maritime Services, 2016). A location plan is shown on **Figure 1**.

RCA Australia (RCA) was commissioned by the Roads and Maritime Services (RMS) to undertake groundwater modelling on the project for the significant cuttings and tunnels. RCA has undertaken the assessment for the cuttings and has teamed with PSM (sub consultant to RCA) to undertake the work for the tunnels.

Arup are preparing an Environmental Impact Statement (EIS) for the proposed works and in addition to the tunnels have identified that a number of cuts may need groundwater modelling to be carried out to inform the EIS.

The Concept Design comprises a vertical alignment gradeline which includes cuts at a number of locations and driven tunnels at Roberts Hill, Shephards Lane and Gatelys Road.

Groundwater investigation/assessments have been reported in a number of reports including the following:

- Groundwater monitoring report for the proposed CHB project presented in Ref [1] and subsequent logger downloading of groundwater levels.

- This report and Ref [2] providing groundwater modelling for the Concept Design cuts and driven tunnels respectively. Modelling for the driven tunnels has been carried out by PSM and the results are reported in Ref [2].

Figure 1 *Site Location Plan*

2 SCOPE OF WORKS

This report presents the following:

- Review of the topographic/geological/ geotechnical and hydrogeological setting.
- Review of the identified cuts expected to have the potential for other than minimal interference with the local aquifer system. Classification of the cuttings has been carried out based on the magnitude of their penetration into the groundwater system based on the groundwater monitoring results in Ref [1] and the potential impact of the cut on the groundwater system as follows:
    - o Significant potential impact (Type A) - groundwater above the cut level and there is a high likelihood that critical potential impacted cuts could:

1) Affect groundwater regimes and any associated groundwater-dependent ecosystem (GDE) if present within the environs of the alignment.
2) Have more than a minimal impact on nearby water supply works as defined by the NSW Aquifer Interference Policy (Ref [17]).
3) Cause engineering mitigations to be implemented into the design/construction or operation of the road system (ie, drainage blankets beneath pavement, pressure reduction drainage in cut batters, etc).

It is noted that a Type A cut will not necessarily have all three of the above potential impacts. For example cuts with a groundwater level at or marginally above the gradeline level will invoke point 3 (ie, with the need for a pavement drainage layer to protect the UZF/pavement) but would not be likely to cause significant impact on nearby water supply works.

- Potential impact (Type B) - groundwater within 5m of the base of the cutting where there is not expected to be an adverse impact to the groundwater regime or GDE and engineering mitigation measures are not expected to be required, however uncertainty in the fluctuation of the groundwater level requires monitoring for confirmation. Type B cuts may impact on design and construction but not nearby water works or GDEs if any (for example where water level rises to the grade level after significant weather events).
- Minimal potential impact (Type C) - groundwater levels greater than 5m below the cut subgrade level where no project or environmental groundwater induced impact is expected.

- Assessment of the local groundwater systems where proposed cuttings are considered to have the potential for other than minimal interference.
- Predict groundwater flow before and after the construction of Type A Cuttings.
- Assess groundwater interception by the cuts under steady state conditions.
- Assess groundwater drawdown resulting from the construction of the proposed cuttings.
- Assess the change in groundwater flow volumes to potential receptors due to the excavation of the cuttings.
- Comment on measures to mitigate any adverse impacts on existing water supply works or GDEs.

3 OTHER INFORMATION

The following references provide information on groundwater piezometer installations, groundwater depths, licenced groundwater supply works along the CHB alignment and rainfall:

- Ref [1] to Ref [10] which provide the results of the geotechnical and groundwater investigation work carried out for the project.
- Licensed groundwater supply works along/adjacent to the alignment from the Department of Primary Industries: Water (formerly NSW Office of Water (NOW) data base.

- Rainfall data from the Bureau of Meteorology. Average annual rainfall for Coffs Harbour MO (1943-2015) is 1699mm. Monthly rainfall data for Coffs Harbour Airport from September 2013 to March 2019 is plotted in **Figure 2**.

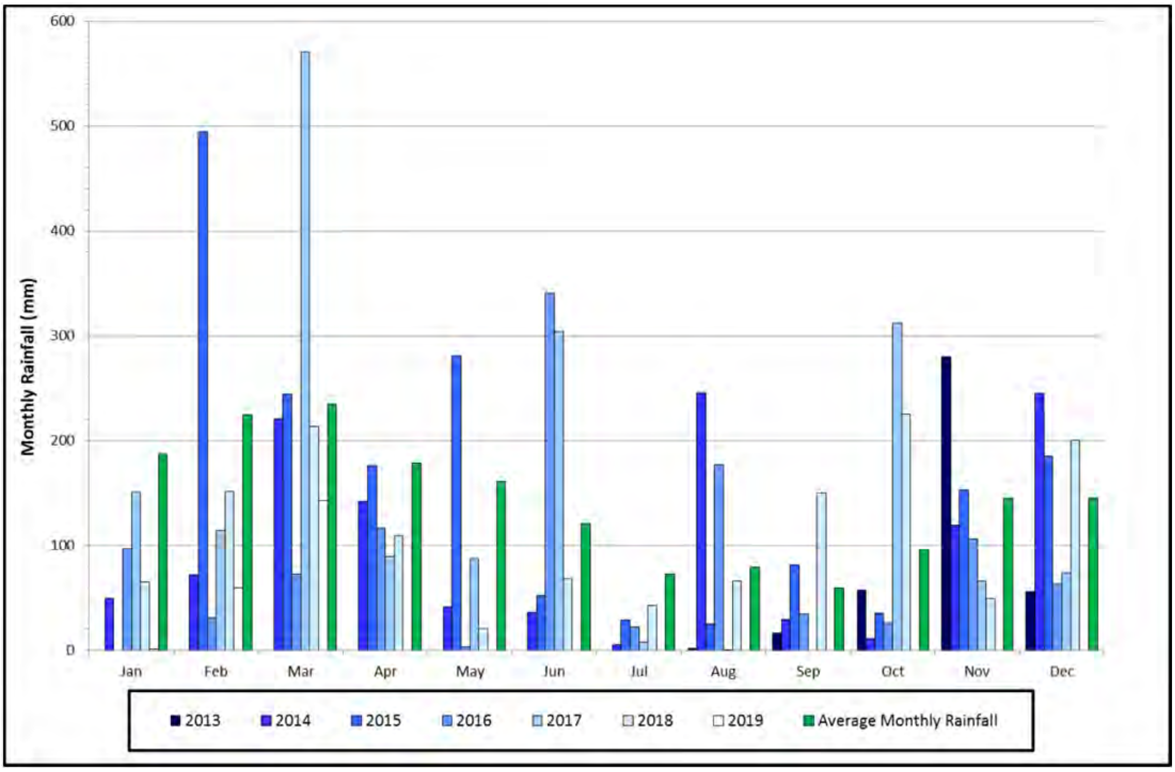


Figure 2 *Monthly rainfall data for Coffs Harbour Airport from September 2013 to March 2019 along with average monthly rainfall from 1943 to 2015 from the Coffs Harbour MO station (now closed).*

4 TOPOGRAPHIC SETTING OF CUTTINGS OF INTEREST

A general overview of the topographic setting of the cuttings of interest is shown on **Drawing 1** in **Appendix A**.

The vertical alignment is shown on **Drawings 5 to 9** in **Appendix A**. Also shown on these drawings is:

- The cut numbering system adopted for reference purposes,
- An interpreted groundwater surface profile along the alignment based on the groundwater monitor information,
- The cutting classification system which is based on the relative level of the groundwater surface and the design cut level. The cut type classification system may be summarised as follows:
    - Type A cutting – Design cut levels are below the level of the groundwater surface,
    - Type B cutting – Design cut levels are within 5m of the groundwater surface,
    - Type C cutting – The groundwater level is greater than 5m below the design cut levels

Sections through the Type A cuttings are shown on **Drawings 11 to 18** within **Appendix A**.

With reference to **Drawing 1** and **Drawings 5 to 18** the following observations are made:

- The tunnels and the cuts of interest (ie Tunnels and Type A cuts that penetrate the groundwater surface) are at discrete locations spread over approximately 7.5km of the alignment.
- Cut 4 is a side cutting into a spur off the southern side of Roberts ridge.
- Roberts Hill Tunnel transects a major east west trending ridge of the main escarpment.
- Cut 8 and 8A (located north of Roberts Hill) are side cuts into spurs off Roberts ridge.
- Cut 11 is a side cut into one of the ridges which run down from the main escarpment.
- Cut 12 is a double sided cutting into one of the ridges which run down from the main escarpment.
- Cut 14 is a double sided cutting into one of the ridges which run down from the main escarpment.
- Shephards Lane Tunnel transects one of the ridges which run down from the main escarpment.
- Cut 16 is a double sided cut through one of the ridges that runs down from the escarpment.
- Gatelys Road Tunnel transects one of the ridges which run down from the main escarpment.
- Cut 18 is a double sided cut across one of the ridges that runs down from the escarpment.

5 REGIONAL GEOLOGY

5.1.1 LOCATION

The project alignment is situated within the New England Orogen in eastern Australia (**Figure 3**). Broadly (in regard to the geological setting), the alignment runs through the Coffs Harbour Sequence. As noted in Section 5.1.2 the Coffs Harbour Sequence comprises the Moombil beds, Brooklana beds and Coramba beds.

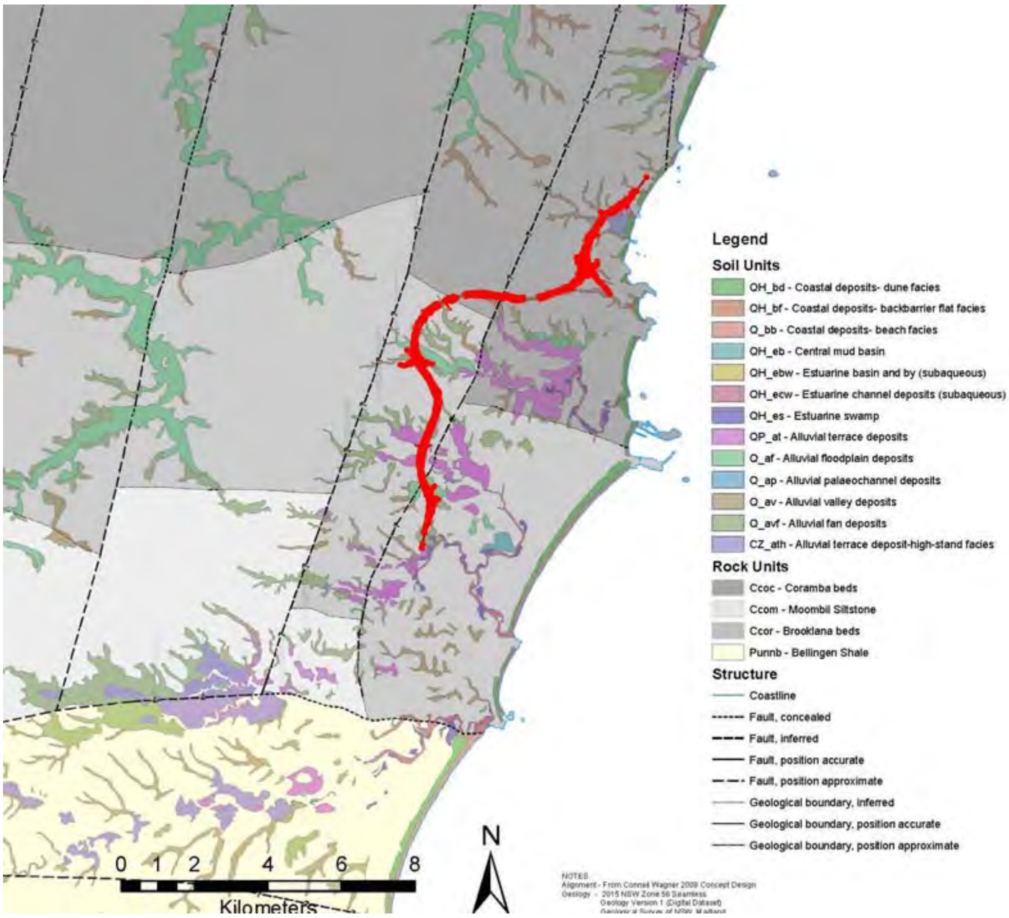


Figure 3 *Regional Geological Setting and Soil Units in the Environs of the Alignment*. (N*ote red line highlights the approximate CHB alignment in relation to the large scale geological context* (Gillian, Brownlow, Cameron, & Henley, 1992, Ref[15]) *reproduced from Ref [8].*

A schematic presentation of the geological setting along the alignment is shown on **Drawings 2 to 4** attached in **Appendix A**.

5.1.2 LITHOLOGY – ROCK

The Coffs Harbour Sequence comprises three lithostratigraphic beds with various proportions of argillite and greywacke:

- Moombil beds:
  - These are not expected to be seen along the alignment.
  - Black massive siltstone, rare lithofeldspathic wacke and granule conglomerate.
- Brooklana beds:
  - Expected from the Englands Road Interchange to just north of the North Coast Rail Line.
  - Thinly bedded siliceous mudstone and siltstone with rare lithofeldspathic wacke, locally chert, jasper, magnetite-bearing chert and metabasalt.
- Coramba beds:

- Expected from just north of the North Coast Rail Line to the Korora Interchange.
- Lithofeldspathic wacke, minor siltstone, siliceous siltstone, mudstone, metabasalt, chert and jasper, rare calcareous.

The sedimentary origin of these metamorphosed rocks is interpreted to have been deposited by turbidity currents, with minor reworking by contour currents. The sediments are derived from a volcanic arc source dominated by dacite, minor andesite and rhyolite (Korsch, 1981, Ref [14]).

5.1.3 LITHOLOGY - SOIL

Quaternary sediments are present in the Coffs Harbour region (see **Figure 3 and Drawings 2 to 4 in Appendix A)** are described as alluvial mud, silt, sand and gravel deposits; coastal sand beaches and dunes; and swamp deposits. Alluvial sediments are expected within low lying areas and at creek crossings. Residual soil and/or colluvium are expected above weathered rock at other areas of the section.

5.1.4 METAMORPHISM

Two phases of metamorphism have been identified as affecting the Coffs Harbour Sequence, termed by Graham and Korsch (1985) (Ref [15]) as ‘M_1’ and ‘M_2’.

5.1.5 REGIONAL GEOLOGICAL STRUCTURE

Due to the folding and faulting of the Coffs Harbour Sequence, the major geological structures expected in the region include faulting, shearing, bedding, foliation and jointing.

Several large scale faults shown on **Figure 3** are inferred in the Coffs Harbour region. The main orientation is parallel to the coast line striking NNE-SSW.

Jointing is expected to vary both across and within the cuts due to the folded and faulted nature of the rock mass.

6 CONDITIONS ENCOUNTERED ALONG THE ALIGNMENT

6.1 SOIL AND ROCK TYPES

Soil and rock types encountered in the geotechnical investigations along the CHB project alignment are summarised in **Table 1**.

Table 1 *Soil and Rock Types Encountered along the CHB Project Alignment*

Material	Material Origin	Material Types	
Soil	Fill Topsoil Slopewash/colluvium Alluvium Residual	Clay soils	Clay and silty clay Silty gravelly clay/gravelly silty clay
		Silt soils	Silt Gravelly sandy silt Silty sand and gravelly silty sand
		Gravel soils	Sandy gravel Silty sandy gravel/sandy silty gravel Silty gravel and clayey gravel Clayey sandy gravel Silty clayey gravel
		Cobble soils	Silty cobbles
Rock	Low grade metamorphic rocks	Low metamorphic grade argillite(1) Brecciated chloritic argillite(1) Jasper(1) Deformed felsic metavolcanic rocks(1) Felsic metavolcanic rock(1) Modified Siliceous felsic volcanic rock(1) Deformed felsic metavolcanic sediment(1) Hydrothermally altered siliceous felsic volcanic rocks(1) Interbedded/brecciated/feldspathic litharenite(1)	

(1) Rock material type description given to samples of rocks subject to petrographic examination

The rock types along the CHB project alignment are collectively known as argillite and this has been adopted in this report. Where the argillite has taken on a siliceous appearance the qualifier *siliceous* has been added to the description. Petrographic descriptions indicate that the term argillite embraces a range of rock types, metamorphism and alteration processes along the alignment.

6.2 GEOLOGICAL STRUCTURE ALONG THE ALIGNMENT

6.2.1 GENERAL

The geological macro or large scale structure that the alignment traverses may be simplified as:

- A syncline structure on its side; with
  - the syncline axis plunging steeply down to the north; and
  - a number of macro faults of known and inferred (from lineaments) location.

A schematic presentation of the geological setting along the alignment is shown on **Drawings 2 to 4** attached in **Appendix A**.

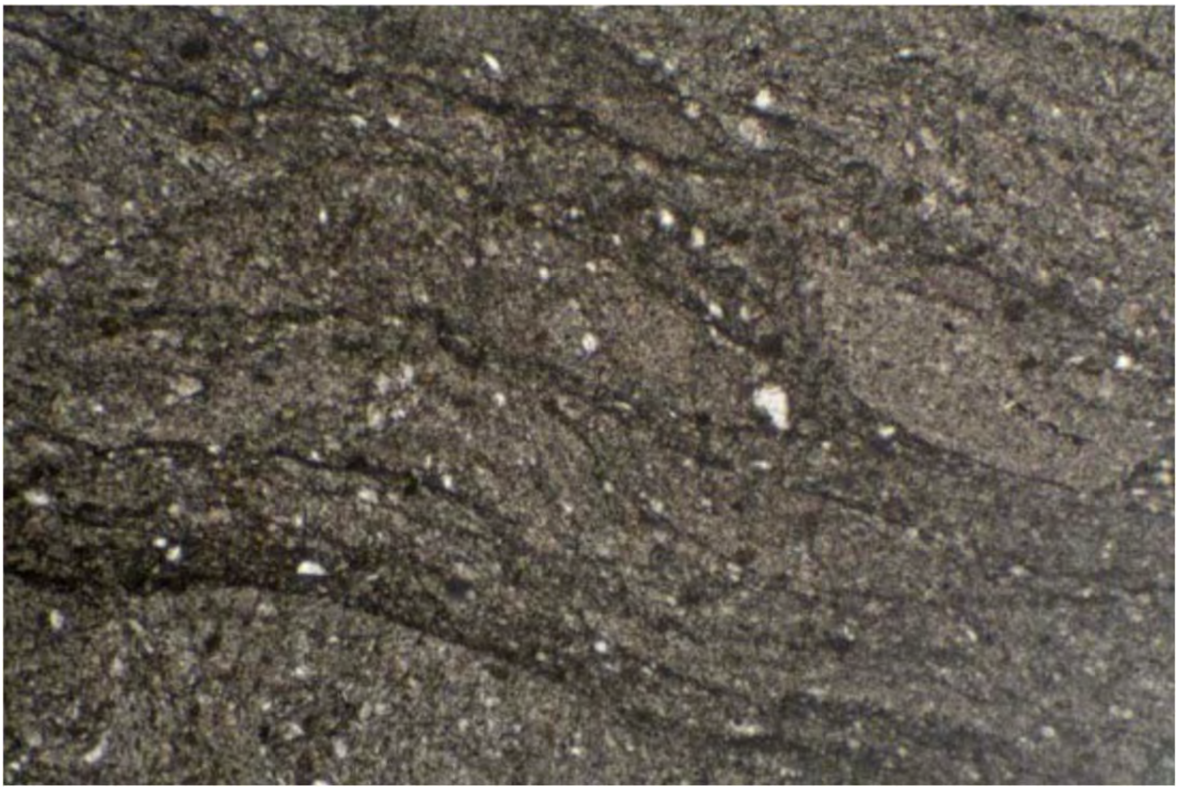
In addition to the macro (large scale) structure, the argillite contains mesoscopic (intermediate scale) and microscopic structure.

The geological structure encountered includes bedding, foliation, cleavage, faulting, shearing, jointing and veins.

6.2.2 BEDDING, FOLIATION AND CLEAVAGE

Although the argillite units are sometimes referred to as being monotonous there is bedding structure within the argillite rocks which varies from that which is visible to the naked eye in outcrop (see **Photograph 2** which shows steeply dipping bedding on a coastal wave-cut rock platform) and that which is visible on a microscopic level (See **Photograph 1**).

The beds are at least in part foliated (composed of or separable into layers) of variable thickness, often thin layers. This structure can however be at a microscopic level as shown in **Photograph 1** and is not always observable to the naked eye.



Photograph 1 *Showing thin section of rock core from BHH151 @ 87.7m (at Cut 17), a general textural view in ordinary transmitted light of this siliceous rock showing delicate bedding structures including numerous 'pockets/lenses' of very fine-grained quartz-rich rock. Note their separation by thin streaks of chlorite or carbonaceous material. Note also the scattered angular quartz crystals. Scale: side of photograph is 1.6mm (Ref petrographic description by Dr Hans Hansel)*

The explanatory notes to the Dorrigo-Coffs Harbour 1:250,000 metallogenic map (Ref [12]) report that bedding has an approximately regionally west to north-west strike to the west of the alignment swinging to predominantly northerly on the coast.

Korsch 1975 (Ref [13]) noted the strike of bedding and cleavage of the Coffs Harbour block sediments south of Red Rock is as follows:

- Bedding strike 070^o to 124^o facing to the north
- Cleavage strike 078^o to 137^o

As the bedding and cleavage are at least in part similar in orientation, separating the two can be problematic.

Korsch 1975 (Ref [13]) also notes that the bedding trends vary in between faults.

The bedding exposure observed on the coastal rock platform during mapping for this investigation (see **Photograph 2**) is consistent with Korsch 1975 with a general east west strike (tending more northerly in the north) and sub vertical dip to the north (except where distorted by folding).

Bedding strike rosettes (developed from the RAAX imaging carried out in boreholes along the CHB project alignment and outcrop mapping at the northern end of the alignment) are shown on **Drawing 2 to 4** attached in **Appendix A**.

At the southern end of the alignment, up to BHH123 in Cut 10, the bedding strike reported in the borehole imaging results are in general agreement with the reported regional strike and that observed on the coastal rock platforms. Thereafter however, the strike of the bedding reported in the borehole imaging results often has a component at variance with the regional strike. BH14 is the most northerly of the imaged boreholes to report identifying bedding structure. Boreholes north of BH14 (northern portal of Gatelys road tunnel) included BH16, BHH158 and BHH160 (Cut 18) and imaging of these did not report identifiable bedding structure. Outcrop mapping to the north and along the coastline indicated that the variations in bedding trends continues to the north and that there is frequent faulting and shearing of the strata exposed in coastal outcrops.

The variation of the strike (orientation) of the bedding noted in the RAAX imaging of boreholes may be a lead indicator to the presence of bedding disruption (i.e. faulting) along the alignment that would not otherwise be observed due to the lack of outcrop.

The number of combinations of faulting/shearing/folding that could produce the changes in bedding orientations is likely to be numerous.

The bedding and cleavage planes are rarely open except when associated with faulting or shear zones.

6.2.3 FAULTING AND SHEAR ZONES

Mapping carried out for this project indicates that faulting is more widespread than the NNE-SSW faults shown on the published maps and that macroscopic (large scale) east-west trending faults such as that exposed in T.G. Jung Quarries, Coramba Road to the west of the CHB project alignment and along the coastline are likely to be present and intersect the alignment resulting in sub-blocks or domains along the alignment.

Further to the large scale faulting mapping of bedrock exposures, the results of RAAX imaging of boreholes also identifies that faulting occurs within the environs of the alignment on a mesoscopic or intermediate scale.

The percentage of borehole length which is sheared and faulted relative to the imaged borehole length is plotted on **Figure 4**. for the boreholes which have been imaged along the alignment. The maximum aperture of the imaged shear/fault zones are also plotted in **Figure 5.**

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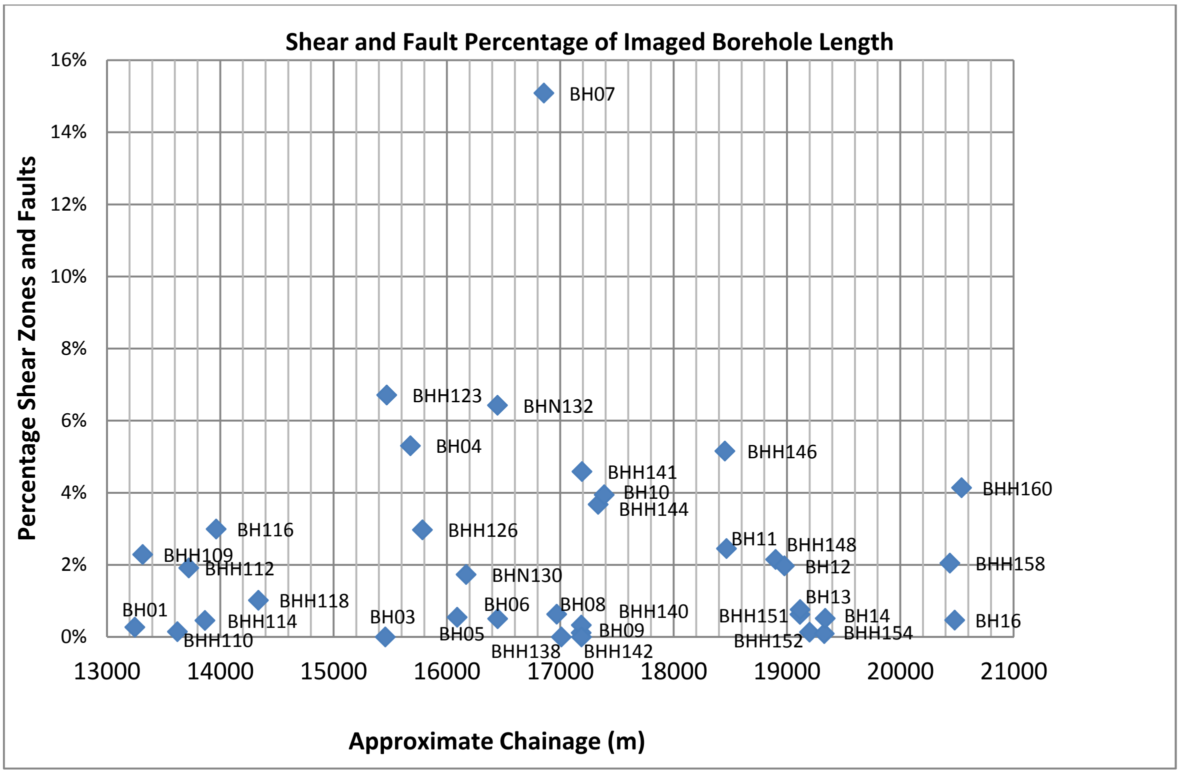
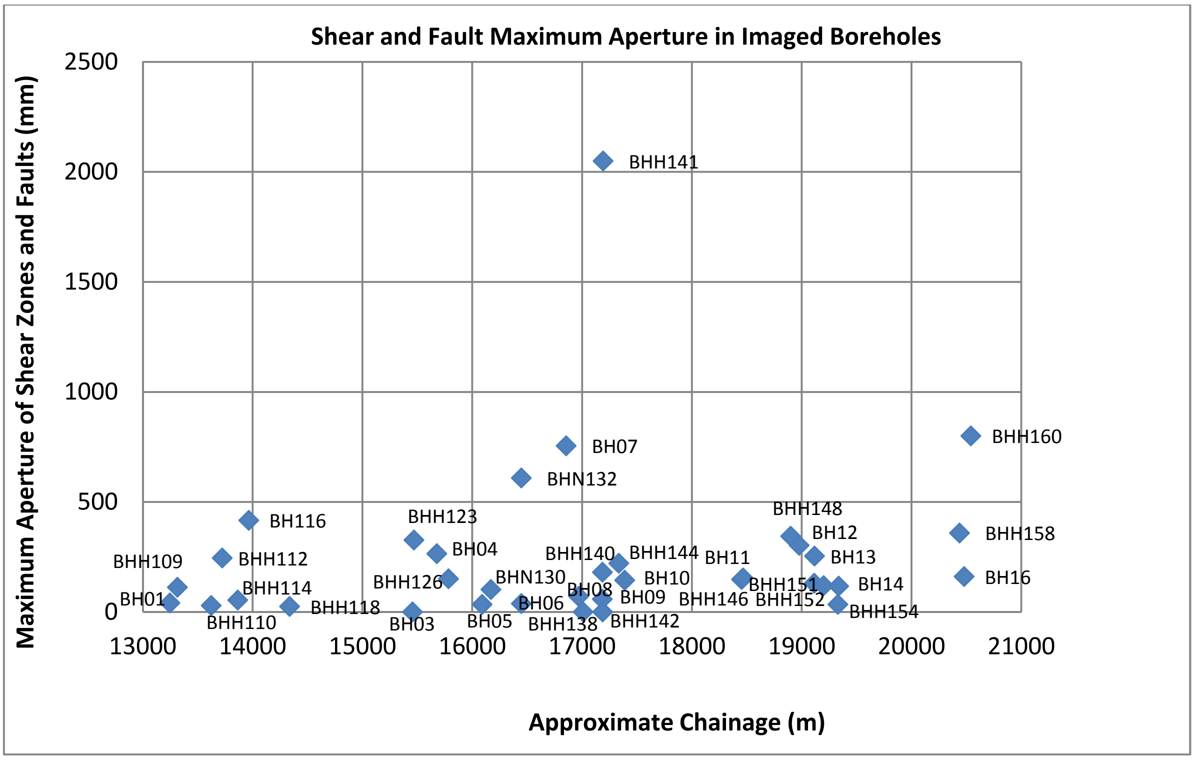


(1) 9.25m of no core and shear zones in BHH142 (between 50.7m and 60m depth) unable to be imaged and not included in above figure

(2) 3.6m of fragmented rock in BHH141 (between 56.4m and 60m depth) considered by PSM to be a fault counted as 2.05m shear zone (as provided on RAAX imaging report) in above figure

Figure 4 *Percentage of Borehole Length Composed of Fault or Shear Zones in RAAX Imaged Boreholes*



(1) 9.25m of no core and shear zones in BHH142 (between 50.7m and 60m depth) unable to be imaged and not included in above figure

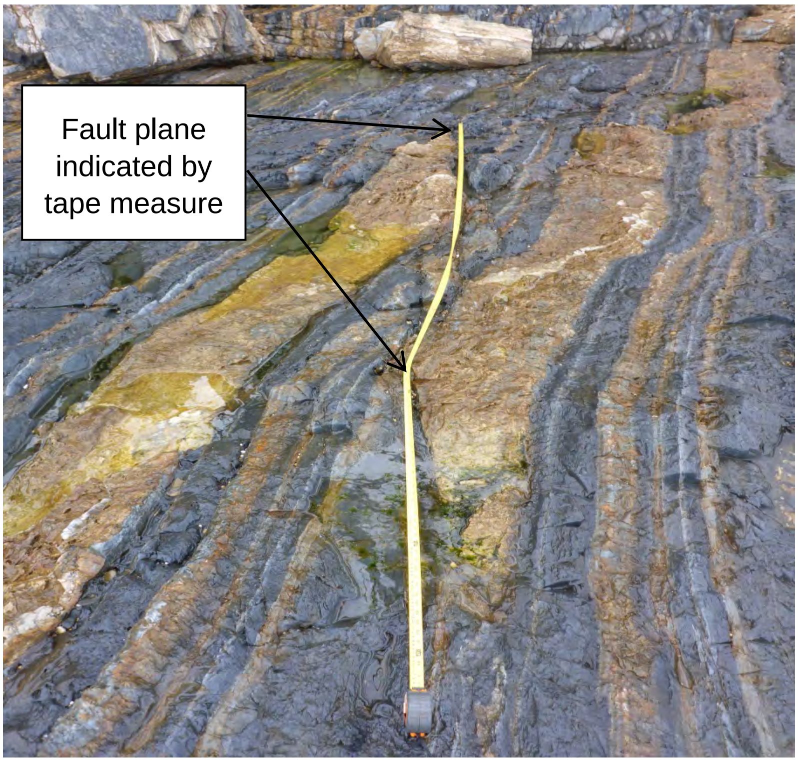
(2) 3.6m of fragmented rock in BHH141 (between 56.4m and 60m depth) considered by PSM to be a fault counted as 2.05m shear zone (as provided on RAAX imaging report) in above figure

Figure 5 *Maximum Aperture of Shear or Fault Zones in RAAX Imaged Boreholes*

The prominence of BHH141 (Shephards Hill Tunnel) on **Figure 5** is essentially due to a large fault encountered in the borehole.

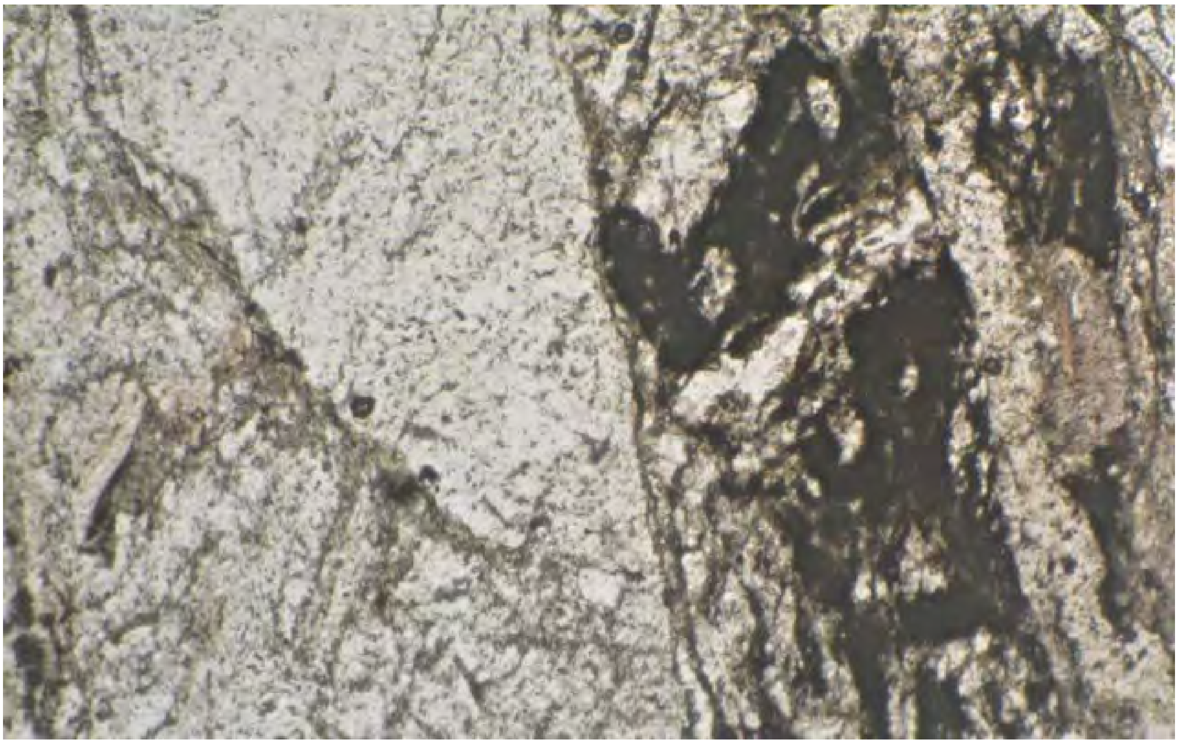
The prominence of BH07 (Cut 14) on **Figure 4** is due to numerous shear zones, with apertures up to 0.755m, being noted in the RAAX imaging of BH07 (Ref [4]). Access to Cut 14 was not available for further investigation during field work for recent geotechnical investigations.

Mesoscopic or intermediate scale faulting is also displayed in the multiple faulting (at different orientations) of the steeply dipping bedding structure exposed in the wave-cut rock platform/cliff face at Hill Beach. One such fault is shown on **Photograph 2**.



Photograph 2 *Faulting of the steeply dipping bedding on the Hill Beach wave-cut rock platform.*

Petrographic analysis also describes microscopic faulting as shown in **Photograph 3** where three rock types are present on a 1.6mm photograph of the thin section.



Photograph 3 *Thin section cut from core from BHH132 27.46m-27.63m showing where three different textural and mineralogical 'types' are juxtaposed. Scale: side of photograph is 1.6mm*

The net result of the multiple tectonic episodes reported in literature is that the original sedimentary rock structure has been disturbed on a micro to macro scale along the alignment.

The large and intermediate scale faulting is expected to be pervasive on a cutting scale.

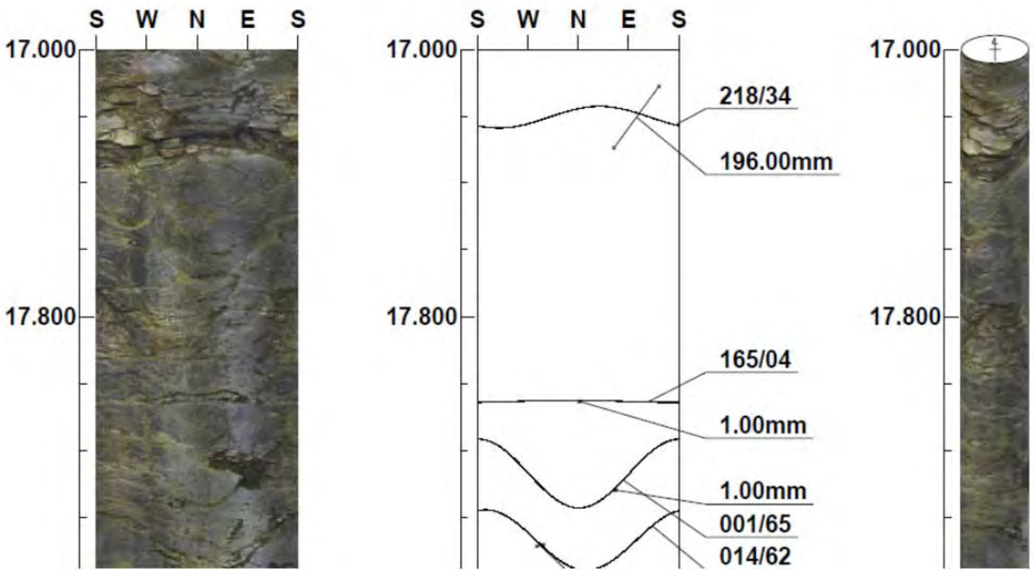
As noted previously, in addition to faulting the argillite contains numerous shear zones. Some of the major shear zones encountered along the alignment are shown on **Drawings 2 to 4** in **Appendix A**.

The major shear zones are also expected to be pervasive on a cutting scale.

In addition to the major planar shear zones, non-planar shear zones cross cut the rock mass at some locations.

A borehole image of a shear zone encountered in BHH112 at Roberts Hill is shown in **Photograph 4**.

A typical unweathered shear zone is shown on **Photograph 5** and presents as a sandy gravel in-fill with adjacent rock interfaces that are stepped, smooth surfaces on a small scale and undulating on a large scale.



Photograph 4 *RAAX imaging of 196mm wide shear zone in BHH112 (at Roberts Hill Cut)*

Photograph 5 *Steeply dipping shear zone in fresh argillite from BHH109 (Cut No.4) at depth of 44.8m*

Non planar/cross cutting shear zones were noted to be exposed at Wally Basarto's quarry to the north of Shephards Lane as shown in **Photograph 6**.

Photograph 6 *Non planar/cross cutting shear zones (Wally Basarto's quarry) highlighted in yellow.*

6.2.4 JOINTING

Korsch 1975 (Ref [13]) noted the presence of joints of many orientations in the Coffs Harbour Sequence and also noted that their seemingly random pattern at many localities made them of limited use in structural analysis.

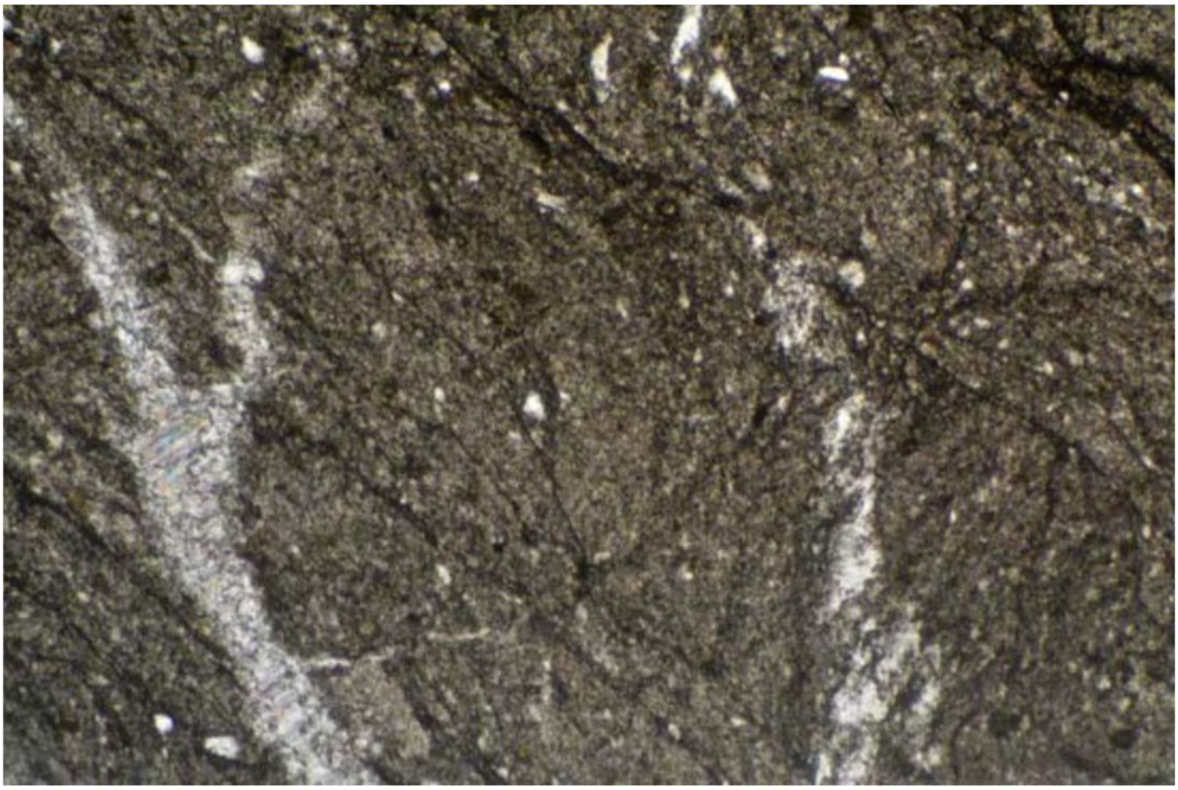
Mapping of defects linearly along cutting sequences in the North Coast Rail Line has shown that the dominant defect pattern changes with location often within the one cutting. Similarly, the plots of joint planes from imaged boreholes drilled within proposed cuttings appear to show no pattern even in areas with consistent bedding orientation. Mapping also indicates that individual joints are not necessarily pervasive on a cutting scale.

Joint infill includes quartz, carbonates, calcite, sulphides, iron oxides, chlorite and zeolites.

6.2.5 VEINS

The presence of veins within the argillite rock type containing a range of mineralogy occurs along the alignment.

Quartz and calcite minerals form the most common veining, both of which can be present within relatively small separations as shown on **Photograph 7**.



Photograph 7 *Thin section of rock from BHH151@ 87.7m. To the left is a calcite vein; to the right is a zone that has numerous small tension gashes occupied by quartz. Scale: side of photograph is 1.6mm*

Quartz veining includes veins of the type encountered in BHH127 (see **Photograph 8**) which are thought to comprise ptygmatically folded quartz. Korsch 1975 (Ref [13]) notes that these appear to be the result of buckling of the original planar quartz veins during metamorphic deformation events.

Photograph 8 *Core from BHH127 showing ptygmatically folded quartz veins*

Veins also include hydrated iron oxides, carbonates, biotite, chlorites and other minerals.

While some of the veins are composed of soft minerals they are generally closed and of limited concern in regard to stability.

6.2.6 GEOLOGICAL FEATURES IDENTIFIED AS CROSSING THE ALIGNMENT AT THE CUTS OF INTEREST

A number of geological features have been identified as crossing the alignment by the observation of pervasive defects (i.e. shears and faults) within boreholes and existing rock exposures. These major features are shown on **Drawings 2 to 4** attached in **Appendix A.** These features and others identified in the logs of boreholes and borehole imaging within the cuts of interest are summarised in **Table 2**.

Table 2 *Pervasive Defects Identified as Crossing the Alignment in the areas of Interest*

Location	Borehole(s)	Approx. Chainage	Pervasive Defects Noted
Cut 4	BHH109	CH13325	Multiple shear zones encountered in borehole core/RAAX imaging. Largest identified in RAAX imaging 112mm wide. Largest logged in rock core 230mm wide. See **Photograph 5**
Roberts Hill tunnel	BHH111	CH13675	Shear zones at 34m, 50mm and 100mm wide.
	BHH112	CH13710	Shear zone 196mm wide at a depth of approximately 17.2m. See **Photograph 4**
	BHH14	CH13850	Shear zone at 41m, 220mm wide.
Cut 8	BHN116	CH13950	Multiple shear zones encountered in borehole. Largest imaged shear zone is 416mm wide dipping at 27° to SW at 20.668m. Multiple shear zones from 20.2 to 22.27m indicating possible fault zone.
Cut 11	BH04	CH15670	Multiple shear zones the widest 265mm and 245mm striking sub parallel to the cut batter dipping ESE at 34^o & 38^o. Small number of shear zones dipping to the NE.
Cut 14	BH07	CH16835	Multiple shear zones (324mm, 627mm, and 755mm wide)
Shephards Lane tunnel	BHH141	CH17175	Approximately 2.05 to 3.5m wide fault noted in BHH141 dipping south at 35^o. Shear zones dipping west at 40^o.
Shephards Lane tunnel	BHH142	CH17175	Shear zones dipping south west at 61^o.
Cut 18	BHH160	CH20520	0.8-1.0m wide shear zone identified in RAAX imaging at 10.28m depth dipping at 14° to the south west.

The inference from the above is that the hydrogeological conditions at a cut/ tunnel level are expected to be dominated by the geological structure present and it is unlikely that the extent of the complexity of the geological structure within any one of the cuts/tunnels will be fully understood until construction.

This forms a basic uncertainty within the groundwater modelling within the environs of the cuts.

It is likely that the presence of geological structure, in particular shear zones and faults, will dominate groundwater flow direction and volume.

7 OCCURRENCE OF GROUNDWATER

The Groundwater Status report (Ref [18]) indicates that groundwater in the environs of the CHB alignment can be categorised into two (2) broad geological features including:

- Unconsolidated sediments.
- Carboniferous age fractured rocks of the Coramba Beds and Brooklana Beds.

Unconsolidated sediments of interest with respect to groundwater are largely confined to the north of the project area associated with coastal regions. Alluvium within the southern portion of the alignment associated with the valleys and waterways is largely clay bound.

The main groundwater resource which underlies the CHB alignment within the environs of the alignment is within the fractured Carboniferous rocks. The fractured Carboniferous age metamorphic rocks in the Coramba and Brooklana Beds are reported in Ref [18] to be thought to have a low porosity (with the secondary porosity consisting of tight discontinuous fractures giving a minimal increase in permeability) except where it is increased by weathering or structural deformation effects such as shear zones. Yields are indicated to be most commonly around 0.5 to 1.0L/s with occasional supplies up to 5L/s.

The pH of the water is reported as slightly acidic to near neutral. The major anion in the upper zone is bicarbonate.

With reference to the NSW Aquifer Interference Policy (Ref [17]) groundwater sources have been divided into “highly productive” and “less productive”. Highly productive groundwater is defined in this Policy as a groundwater source that is declared in the Regulations and will be based on the following criteria:

a) has total dissolved solids of less than 1,500 mg/L, and

b) contains water supply works that can yield water at a rate greater than 5 L/s.

Based on the above the groundwater resource in the fractured rock in the environs of the CHB would be generally described (with reference to the NSW Aquifer Interference Policy) as being a less productive fractured rock groundwater source.

8 EXISTING NOW LICENSED GROUNDWATER BORES

The Department of Primary Industries: Water (formerly NSW Office of Water (NOW)) database indicates that there are numerous licensed groundwater wells within proximity to the CHB alignment. The location of these is shown on Figures titled Groundwater Users and Alluvial Areas prepared for the EIS by Arup.

Groundwater interception or significant drawdown arising from excavation of cuttings for the CHB may have an impact on existing users within the zone of influence of a proposed cut.

9 REVIEW OF GROUNDWATER LEVEL MONITORING

The highest and lowest groundwater levels recorded during manual dipping of the piezometers in the cuts of interest with regard to penetration below the groundwater surface are shown on the Drawings in **Appendix A.**

It may be seen on **Figure 2** in Section 3 that March of 2017 was notably wet compared to other months by year.

Investigation fieldwork drilling was nearing completion in March 2017 and water level monitoring was being undertaken in open drill holes and drill holes that had been fitted with standpipe piezometers at that time.

Figure 6 shows a plot of the results of groundwater depth monitoring in BHH158. The groundwater depth monitoring included manual dipping of open drill holes and a standpipe piezometer at discrete times until an automatic groundwater logger was installed in BHH158.

During March 2017 groundwater depths in BHH158 were being logged manually but the results still reflected the rise in the groundwater associated with the rainfall in March 2017.

Generally the highest groundwater levels monitored to date appear to coincide with the relatively high rainfall for the month of March 2017. **Figure 6** shows a plot of rainfall and depth to groundwater verses time at BHH158 which covers the pre and post March 2017 period.

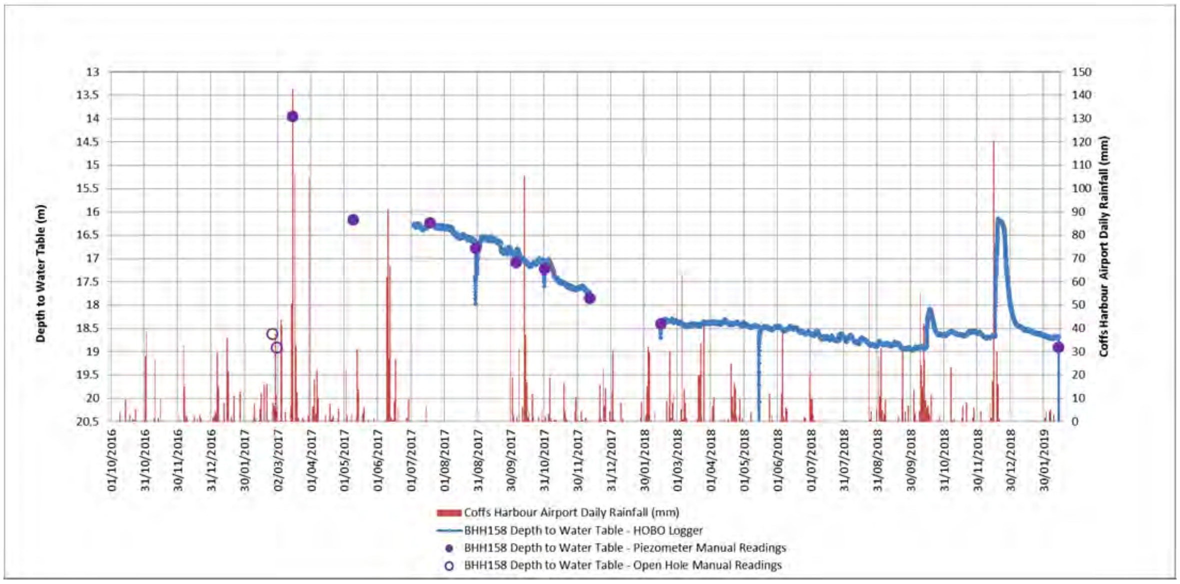


Figure 6 *Plot of rainfall and depth to groundwater verses time in BHH158*

Figure 6 shows that the groundwater level at BHH158 rose approximately 5m from 18.9m depth below ground level (dbgl) on 1/3/2017 to 13.95m dbgl on the 16/3/2017 in response to rainfall in early March 2017 then generally declined in response to subsequent drier climatic conditions. In summary the piezometer monitoring result shows the highest groundwater levels in March 2017 dropping gradually thereafter with intermittent rises in response to rainfall events. In particular a rise of approximately 2.5m in December 2018 in response to a 120mm rainfall event is noted as is the rapid fall in groundwater level thereafter.

The gap in the HOBO logger record between mid-December 2017 and mid-February 2018 occurred due to data corruption which could not be retrieved.

The monitoring results also show a rapid response in groundwater levels to individual rainfall events at some locations. Typical responses are shown on **Figure 7** for standpipe piezometers in holes BHH112 and BHH114 at Roberts Hill.

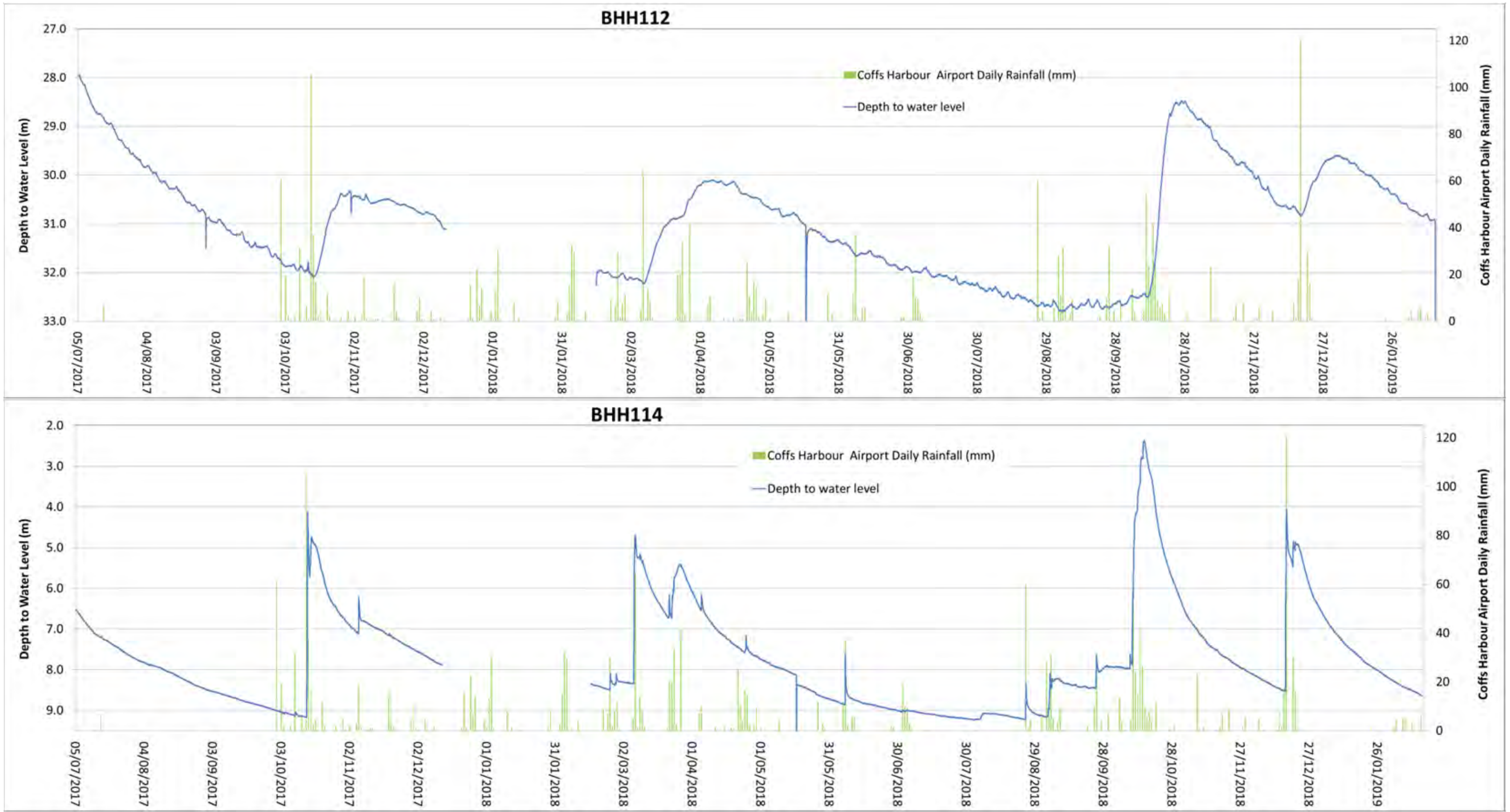


Figure 7 *Response of groundwater level in standpipe piezometers in BHH112 & BHH114*

During the drilling of BHH114 significant drilling water loss was experienced with complete drilling water loss at a depth of 27.8m. **Figure 7** shows the response of the HOBO logger (recording water levels in a sealed section of standpipe piezometer sealed over a depth from 9.57m-17.6m). Both the HOBO logger in the shallow standpipe piezometer and the deeper vibrating wire piezometer in BHH114 (located at a depth of 31m below the standpipe piezometer) show very rapid responses to rainfall events.

Drilling conditions with respect to water loss were better in BH112. **Figure 7** shows the response of the HOBO logger (recording water levels in a sealed section of standpipe piezometer sealed over a depth from 31.6m-40.05m). A vibrating wire piezometer (VWP) grouted at a depth of 60m in BHH112 below the standpipe responded similarly.

The response of the hobo logger in the standpipe piezometer and the deeper vibrating wire piezometer in BHH112 have a more muted response to the rainfall events compared to that exhibited by monitoring of groundwater levels in BHH114.

Based on the above it is likely that rainfall recharge will vary. Accompanying this, is the expectation that groundwater makes in cuts will vary (both with rainfall events and with location due to geological structure) from the predicted average groundwater make for the cutting.

10 CUTTINGS EXPECTED TO INTERSECT THE GROUNDWATER SYSTEM

Drawings 5 to 9 in Appendix A show the vertical alignment with the groundwater levels in all cuttings (ie Cut types A, B, and C).

Cuttings where the groundwater level monitoring data indicates cuts are likely to significantly intersect the local groundwater surface (Cut Type A), are presented on the **Drawings 5 to 9** in **Appendix A** with the location of a typical cross section through the cut**.** The typical cross sections through the Type A cuttings are presented on the **Drawings 11 to 18** in **Appendix A.**

An estimate of the range of penetration below the groundwater level (based on available monitoring data) summarized on **Table 3**.

Table 3 *Summary of Type A Cuttings along the alignment expected to penetrate the local groundwater surface with the estimated range of penetration below the groundwater surface*

Cut No	Reference chainage	Estimated ranges of maximum penetration of cutting excavation (positive indicates groundwater surface is above cut line) at northbound toe of cutting
Cut 4	CH13325	+7.5m to 13m
Cut 8	CH13925	+7m to 13m
Cut 8A	CH14300	+6m to 10m
Cut 11	CH15750	+4.5m to 8.5m
Cut 12	CH16075	~+2m to 6[(1)]
Cut 14	CH16850	~+2m[(2)]
Cut16	CH18450	+2m to 5m
Cut 18	CH20400	+15m to 20m

(1) Based on water level in borehole BH05 recorded during RAAX imaging in 2008 (no piezometer installation)

(2) No reliable groundwater levels available. Pre construction groundwater profile based on simulation using assumed parameters.

With reference to **Table 3** it may be seen that the following cuts have penetrations into the groundwater that may result in greater than minimal groundwater interference (as defined in Ref [17]) at some distance from the cut depending on the surrounding development and environment include the following:

- Cut 4,
- Cut 8,
- Cut 8A,
- Cut 11,
- Cut 12,
- Cut 14,
- Cut 16,
- Cut 18.

Groundwater modelling has been undertaken to predict the distance from the cuts to a groundwater level drawdown of 2m (referenced water level decline in Ref [17]) and 1m together with the effect on groundwater flows.

11 FIELD TESTING

Work carried out during the investigation field testing to provide data for assessment of the hydraulic conductivity of the strata comprised:

- packer testing in boreholes during drilling, and
- falling head permeability testing in standpipe piezometers, together with

Results of field testing are detailed in Ref [1]. In summary the results indicate that the hydraulic conductivity of the argillite varies between $1x10^{-6}$ m/sec to less than $6.3x10^{-9}$m/sec. Variations in the hydraulic conductivity are likely to be due to changes in fracturing of the rock mass, opening and pervasiveness of fractures, presence of geological structures such as shear zones/ faults etc, and weathering of the strata.

It is noted that the majority of the packer permeability testing has been carried out in the boreholes within the environs of Robert Hill, Shephards Lane and Gatelys Road tunnels.

It is also noted that groundwater level monitoring data presented in Ref [1] is periodically updated to provide an extended monitoring time period beyond the date on the draft report.

12 GROUNDATER MODELLING OF CUTS

12.1 MODELLING

12.1.1 GENERAL/CONCEPTUAL GROUNDWATER MODEL FOR CUTS

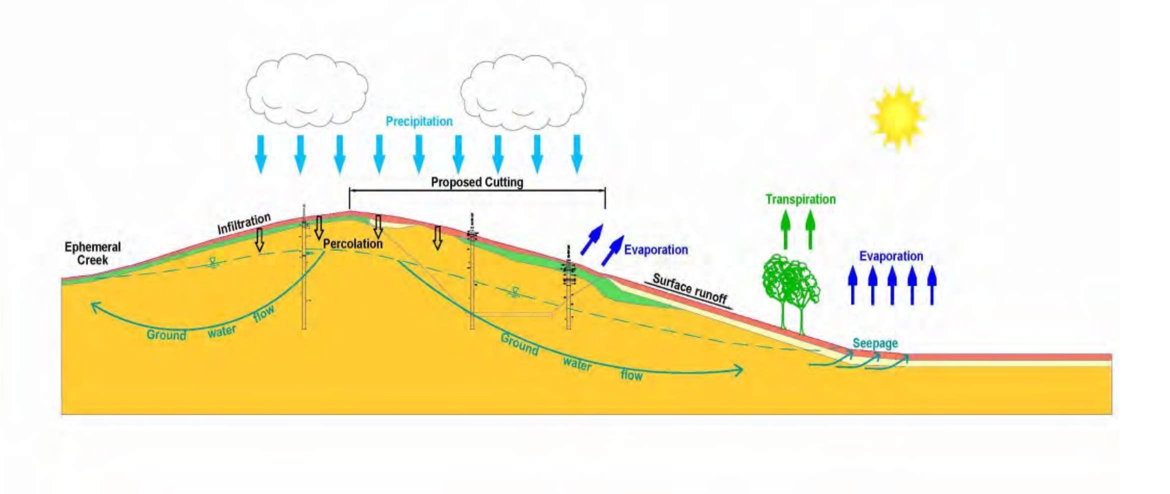
As noted previously in Section 4, the cuts of interest are at discrete locations spread over approximately 7.5km of the alignment and are located in the ridges and spurs off the escarpment west and north of Coffs Harbour.

Cuts in the ridges on the foot slopes of the escarpment are expected to intersect perched water tables and the local groundwater system that infiltrates quickly and discharges in the seeps, valleys and dams downslope of the ridges and on the flats.

The local groundwater system has developed in response to rainfall infiltration, topography and the hydrogeological setting.

Cuttings which penetrate the local groundwater table are considered unlikely to have a significant effect on the behaviour of the underlying regional groundwater system**.** It is expected that cuttings may have effects on the local groundwater system within the environs of the cutting. The objective of the modelling is to provide information to understand that local effect.

A typical general conceptual groundwater model for the cuttings is shown in **Figure 8**.



`**Figure 8** *Conceptual groundwater model*

The pre-cut groundwater surface has been assumed to generally follow the landform in a subdued shape. It is however likely that the rock structure in particularly shear zones and faulting will distort the groundwater surface shape and flow direction locally.

After excavation of the cut, the provision of a drainage layer beneath the pavement will result in the base of the cutting acting as a long-term groundwater sink causing drawdown of the local groundwater surface into the cutting.

Typical effect of cutting excavation on the groundwater level in the ridge lines/ hills is shown on **Figure 9.**

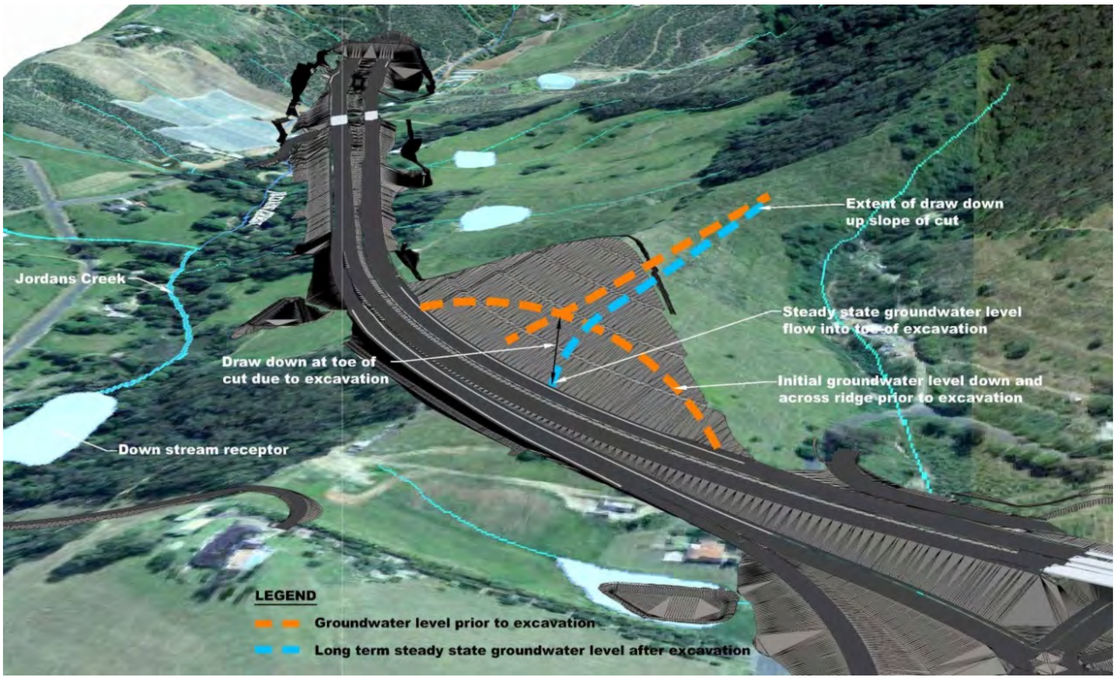


Figure 9 *Schematic representation of effect of the cuttings through the ridge on groundwater levels*

12.1.2 PREDICTIVE GROUNDWATER MODELLING OF CUTTINGS

RCA has compiled two dimensional (2D) groundwater models at each of the significant proposed cut locations to provide information on the effects of cuttings on groundwater systems and surrounding environs.

Commercially available 2D RS2 (Phase 2) finite element software has been used to predict the behaviour of groundwater systems at the proposed cuts with significant penetrations below the groundwater level.

The following cuts have been modelled:

- Cut 4.
- Cut 8.
- Cut 8A.
- Cut 11.
- Cut 12.
- Cut 14.
- Cut 16.
- Cut 18

The local groundwater resource is a fractured rock aquifer which has been modelled as an unconfined equivalent porous medium.

Each of the identified cuts has its individual topography, geological structure and own unique influence on the way recharge water (precipitation) runs off or infiltrates into the subsurface. This is expected to produce a complex groundwater flow pattern, which may only be partially represented in modelling. As such the predictions of groundwater models should be regarded as an indicative predictive tool rather than a definitive means of calculation.

As previously noted the proposed cuts penetrate the groundwater system to varying depths, refer to **Table 3** and the drawings in **Appendix A**. Accordingly, the impact on the local groundwater system may be expected to be commensurate with the depth of groundwater penetration.

2D groundwater models have been constructed to:

- Calculate an estimate of the existing component of flow across the proposed cut and to a down gradient receptor on the lowlands, for example Jordans Creek in Cut 18 shown in **Figure 9**.
- Calculate an estimate of the groundwater seepage into the cut after construction and to the down gradient receptor on the lowlands.
- Calculate an estimate of the distance from the toe of the cut to groundwater surface drawdowns of 2m and 1m from the pre-cut estimated groundwater surface.

Calibrated cross-section groundwater flow models were used to predict both groundwater flow through the proposed cut prior to excavation and to predict groundwater inflows into the constructed cut together with the accompanying groundwater drawdown and effect on a downstream receptor.

12.1.3 *MODEL GEOMETRY*

Spatial dimensions were assigned by importing a cross-section of each cutting from the design drawings in DXF format and extending the boundaries away from the cutting based on available topographic contours. The lateral dimensions were set at a sufficient distance to minimise the effect of the boundary conditions on the computational analysis of the model.

Each model was generally assigned a down gradient receptor to replicate the groundwater flow direction to a receptor down gradient of the cutting.

Existing groundwater conditions were developed by matching the steady state analysis groundwater levels to the measured heads in the piezometers.

12.1.4 *HYDRAULIC CONDUCTIVITY*

In situ testing for the assessment of hydraulic conductivity has comprised borehole packer testing and falling head permeability testing in piezometers. The results of this testing are reported in Refs [1] and [2].

The hydraulic conductivity measured ranged over three orders of magnitude, from $1x10^{-6}$m/s to less than $6.3x10^{-9}$m/s. The higher permeability values were generally associated with the more weathered strata.

For modelling purposes hydraulic conductivities were assigned for each model based on the results of hydraulic conductivity testing, with adjustments made in order to calibrate modelled existing groundwater levels to measured levels.

For the cuts where limited packer test data is available hydraulic conductivity of $1.0x10^{-6}$m/s to $2x10^{-6}$m/s was adopted for the soil unit, Rock Unit 2A and Rock Unit 2B (as defined on **Drawing 10**). The adopted hydraulic conductivity of Rock Unit 2C (as defined on **Drawing 10**) varied from $1.5x10^{-6}$m/s to $6x10^{-9}$m/s. Variation in hydraulic conductivity was also required as part of the calibration process where variable recharge rates of 5% and 15% were incorporated in the model. The values chosen as part of the calibration process are presented in **Appendix B**.

At some locations high hydraulic conductivities are indicated in some parts of the rock mass. It is interpreted that these are associated with local shearing or fracturing or similar.

12.1.5 *BOUNDARY CONDITIONS*

Typically a constant head boundary condition has been set as the down gradient receptor at a potential water body, generally based on the topography of the cutting. The constant head represents a location whereby discharge to a creek or a low lying area is likely.

12.1.6 *RECHARGE*

Ref [19] titled *Water Sharing Plan for the North Coast Fractured and Porous Rock Groundwater Sources- Background document* provides an estimate of the average annual recharge for the New England Fold Belt Coastal Groundwater Source of fractured rock of 4%.

It is noted however that variations in recharge are to be expected with differing transmissibility characteristics.

A lower bound recharge of 5% of annual rainfall has been adopted based on Ref [19]. This equates to an infiltration rate of approximately 85mm/year.

In addition to the above as noted in Section 9 of this report, rainfall event based recharge levels are expected to be variable. Modelling in Ref [2] suggests local recharge levels up to 15% are required to effect the groundwater level reponses measured in some piezometers. This could be expected to be associated with higher than average groundwater flows following rainfall events.

Based on the above, modelling has been undertaken for recharge rates of 5% and 15%. As the majority of the groundwater flow in the local groundwater system is from rainfall infiltration the resulting flows for recharge rates of 5% and 15% may be regarded as lower bound and upper bound estimates respectively.

12.1.7 CALIBRATION

12.1.7.1 CALIBRATION METHODOLOGY

Each model was manually calibrated under steady state conditions by assigning a hydraulic conductivity and the position of the down gradient constant head boundary along the section under consideration. The combination of recharge, hydraulic conductivity and boundary condition used for each model is considered plausible for the project site and surrounds although it may not be a unique solution. It is possible that other combinations of recharge, hydraulic conductivity and boundary conditions are available that can provide a similar approximation to the steady state conditions. Assigned parameter values for each cutting are shown in **Appendix B**.

12.1.7.2 CALIBRATION CRITERIA

The objective of the model calibration was to ensure the model predicted the observed groundwater levels identified in groundwater piezometers within the model domain. These groundwater levels formed the starting steady-state groundwater head condition. Variations in groundwater levels occur with seasonal and climatic conditions. Accordingly, a precise match of every groundwater level is neither required nor possible. However, to obtain a representative model calibration the following criteria was adopted:

- Simulated groundwater table levels are generally in accordance with the groundwater table measurement determined in the field.
- An acceptable percentage difference value is achieved when comparing observed heads and modelled heads.

The calibration targets for each model are shown in **Appendix C**.

12.1.8 SIMULATED CUTTING

Following calibration of the model to simulate the existing condition, the model was run with the cut geometry imposed to simulate the effects of the cuttings on the local groundwater regime.

The effect of the cut has been simulated by the following:

(1) Nulling out the cut geometry.

(2) Specifying a constant head boundary at cut subgrade level to represent the effect of a subgrade drainage blanket where the cutting base is below the existing groundwater level.

(3) Specifying a vertical infiltration with “seepage face condition” applied on the cut batters to allow for any seepage break out on the batters.

(4) Carrying out a steady state analysis.

(5) The batter slopes were assigned a reduced recharge rate to simulate the enhanced runoff expected from the batters. It is noted that this will vary depending on batter disturbance during blasting/ excavation.

12.1.9 MODELLING CLASSIFICATION

The predictive models are classified as Class 1 with attributes of Class 2 models under the Australian Groundwater Modelling Guidelines (Ref [20]) .

12.2 RESULTS OF GROUNDWATER MODELLING OF CUTTINGS

12.2.1 GENERAL

Seepage flows and head change at the potential or hypothetical downstream receptor have been calculated in the model by assigning a discharge section (flux line) at the down gradient constant head boundary before and after cut simulation.

A flux line was assigned between the groundwater table and the depth of cut to determine the flow through before the cutting.

For the cut simulation a discharge section was assigned across the constant head (drainage blanket) subgrade boundaries and across the seepage face of the batter slopes to calculate groundwater flows into the cuttings.

The groundwater flow rates along the seepage face of the lower batter slopes were compared to the average daily pan evaporation data for the Coffs Harbour Station from 1968 to 2015. Results indicate that the average steady state seepage flow emanating from the lower cut batter slopes (where occurring) is less than the average daily pan evaporation rate (2.3 to 6.2mm/day) and might be expected to be generally lost to evaporation. It is noted that it is likely that seepage will concentrate at faults, shear zones, open joints and fractures, etc, and the effect of averages may not uniformly apply. Accordingly, the seepage face volumes have been included for each cut.

It is also noted that the flows in the long-term steady state conditions are likely to be lower than those occurring in the transient case as the cut is excavated and high permeability defect zones are encountered. Accordingly, the inference above that under steady state seepage conditions the cut batter faces may appear substantially dry should not be interpreted to mean that the cut will be a dry cut for construction purposes.

The following sections discuss the individual geological and hydrogeological setting and the conceptual site model for each cut.

12.2.2 GROUNDWATER THROUGH FLOW ACROSS CUT PRIOR TO CONSTRUCTION

The groundwater through flow at the cutting prior to the cutting is shown on **Table 4**.

Table 4 *Calculated Groundwater Flow through the Proposed Cut Prior to Construction*

Cut/Chainage	Calculated groundwater flow through the proposed cut (at the reference section) prior to construction at the modelled cross section chainage (L/day per m length of cut)
Cut 4 / CH13325	16.03 (42.95)
Cut 8 / CH13925	17.85 (40.82)
Cut 8a / CH14300	3.67 (12.26)
Cut 11 / CH15750	9.80 (15.38)
Cut 12/CH16075	0.77 (1.67)
Cut 14/ CH16850	14.14 (20.28)
Cut 16/CH18450	10.71 (25.81)
Cut 18 /CH20400	9.22 (21.39)

1. The values reported are as calculated and it should be noted that the number of significant figures does not infer a level of precision.
2. The flow rate without brackets is based on the adopted recharge rate of 5% of the average annual rainfall.
3. The bracketed flow rates represent the calculated flow rate for 15% recharge.
4. No reliable groundwater data available for Cut 14. Results based off simulations using assumed parameters.

It is noted that the difference between the through flow rates at the cut (shown in **Table 4**) and the down gradient boundary flow rates (potential groundwater receptor) shown in **Table 7** arise due to the addition of infiltration into the model between the flux line at the cut location and the flux line at the down gradient boundary (potential groundwater receptor). In addition, the measured flow through is taken from the groundwater table level in the middle of the proposed cut to the base of the proposed cut and does not include any through flows beneath the cut. The flow through rate is expected to vary along the length of the cut due to variations in groundwater level with respect to the base of the cut.

12.2.3 GROUNDWATER SEEPAGE INTO CUTS AFTER CONSTRUCTION

The results of the calculated steady state flow rate and estimated seepage volumes at each cutting are shown in **Table 5**.

The total calculated flow rates into the cutting produced by each cross-section modelled is on a per metre length of cut perpendicular to the cross-section. The estimate of seepage into the cutting is based on the following assumptions and simplifications:

- Estimation of the length of cut subgrade below the groundwater table based on the assumption that the depth to groundwater remains similar to the depth measured at the piezometers along the cut profile.
- Assumption that the seepage calculated in the model at one location can be varied proportionally to the depth of groundwater above the proposed cut level along the length of the cut.
- Adoption of triangular integration of the flow over the estimated length of the cut at subgrade level below the groundwater surface level.

Table 5 *Calculated Steady State Groundwater Flow Rate (from 2D Cross-section Model) and the Extrapolated Estimated Volume of Groundwater Seepage into the Cutting over the Longitudinal Length of the Cutting*

Cut / (Chainage)	Seepage emanating on lower batter(s) at modelled cross-section chainage (mm/day over the effected batter length)	Total Seepage flow into cut along batter slopes at modelled cross-section chainage (L/day) per m	Total Seepage flow into cut along drainage blanket at modelled cross-section chainage (L/day) per m	Total seepage into cut (L/day) per m at the modelled cross section chainage	Estimated longitudinal length of cut subgrade below the groundwater surface (m)	Calculated (1) total seepage volume (kL/day)
Cut 4 / CH13325	0.0 (0.0)	0.0 (0.0)	18.90 (63.82)	18.90 (63.82)	235	2.22 (7.50)
Cut 8 / CH13925	0.5 (0.5)	5.00 (5.67)	44.77 (155.62)	49.78 (161.29)	215	5.35 (17.34)
Cut 8a / CH14300	0.0 (0.0)	0.0 (0.0)	15.89 (117.52)	15.89 (117.52)	205	1.63 (12.05)
Cut 11 / CH15750	0.0 (0.0)	0.0 (0.0)	7.11 (33.79)	7.11 (33.79)	195	0.69 (3.29)
Cut12/CH16075	0.0 (0.0)	0.0 (0.0)	1.53 (1.94)	1.53 (1.94)	76	0.06 (0.07)
Cut 14 / CH16850 (4)	0.0 (0.0)	0.0 (0.0)	13.95 (34.71)	13.95 (34.71)	72	0.50 (1.25)
Cut 16/CH18450	0.0 (0.0)	0.0 (0.0)	53.67 (141.46)	53.67 (141.46)	106	2.84 (7.50)
Cut 18 / CH20425	0.0 (0.0)	0.0 (0.0)	16.35 (62.75)	16.35 (62.75)	241	1.97 (7.56)

(1) The values reported are as calculated and it should be noted that the number of significant figures does not infer a level of precision.

(2) The flow rate without brackets is based on a recharge rate of 5% of the average annual rainfall.

(3) The bracketed flow rates represent the calculated flow rate for 15% recharge.

(4) No reliable groundwater data for Cut 14. Results based off simulation using assumed parameters.

(5) Absence of modelled seepage emanating from the lower cut batters (in most cuts),is assessed to be due to the effectiveness of the modelled drainage blanket at the base of the cut.

With reference to **Table 5** and the pan evaporation rates noted in Section 12.2.1 it is expected that:

- Any seepage emanating on the cut batter may generally be largely lost to evaporation and will only be captured if the seepage is concentrated by dominant discontinuities or the flow is higher than the evaporation rate.
- The seepage volumes collected at the base of the cuts may also be reduced by evaporation if it was exposed and spread over the cut base. It is expected however, that this seepage will be isolated from evaporation effects by the overlying pavement and will be collectable.

It is also noted that for a given hydraulic conductivity the seepage flow rate into the cuts predicted by the model is a function of the depth of the cut below the groundwater table (and resulting hydraulic gradient into the cut) and the infiltration/recharge rate applied to the model. Increasing the infiltration/recharge rate will increase the seepage volumes into the cuts predicted by the models. As such, they are likely to represent the 'on average condition' and at any point of time local climatic effects (departures from the adopted average rainfall) are likely to produce variations from the modelled condition.

In addition to the above, higher permeability zones were encountered during drilling in some cuts and during hydraulic conductivity testing as previously noted. Similarly lower permeability zones are also expected.

12.2.4 LOCAL WATER MAKES IN QUARRIES

Little information is available on the groundwater flows into excavations such a local quarries however TJ Jungs Bennetts Road Quarry (located approximately 1.7km to the west of the alignment) is a well-known quarry in the area that makes water.

Photograph 9 shows a portion of the quarry face at TJ Jungs Bennetts Road Quarry which exhibits seepage variation along the quarry face.

Photograph 9 *Groundwater seepage from shear zone in batter of TG Jungs Bennetts Road Quarry with dry batter faces on either side*

Water being pumped from T G Jungs Bennetts Road quarry on the 13 June 2017 was measured at 1L/sec (i.e., 86.4kL/day). It is understood that the pump is run continuously. It is noted that the extent of the surface water catchment included in this pumpage is unknown.

Considerations that impact on this water make are as follows:

- The quarry is relatively deep varying up to the order of 100m on the southern boundary.
- As shown on **Figure 10** and **Figure 11** Lidar indicates that Coffs Creek lies in near proximity to the crest of the quarry about 50m above the quarry floor.
- The quarry's western face has a shear zone which is assessed as providing a relatively higher permeability zone of transmission for groundwater.

Accordingly, it is considered that the inferred groundwater make in the quarry is not indicative of the groundwater makes likely from the cuts along the alignment and arises from an unusual concurrence of the surface water source above the crest of a relatively deep quarry with a zone of permeable rock providing connectivity between the two.

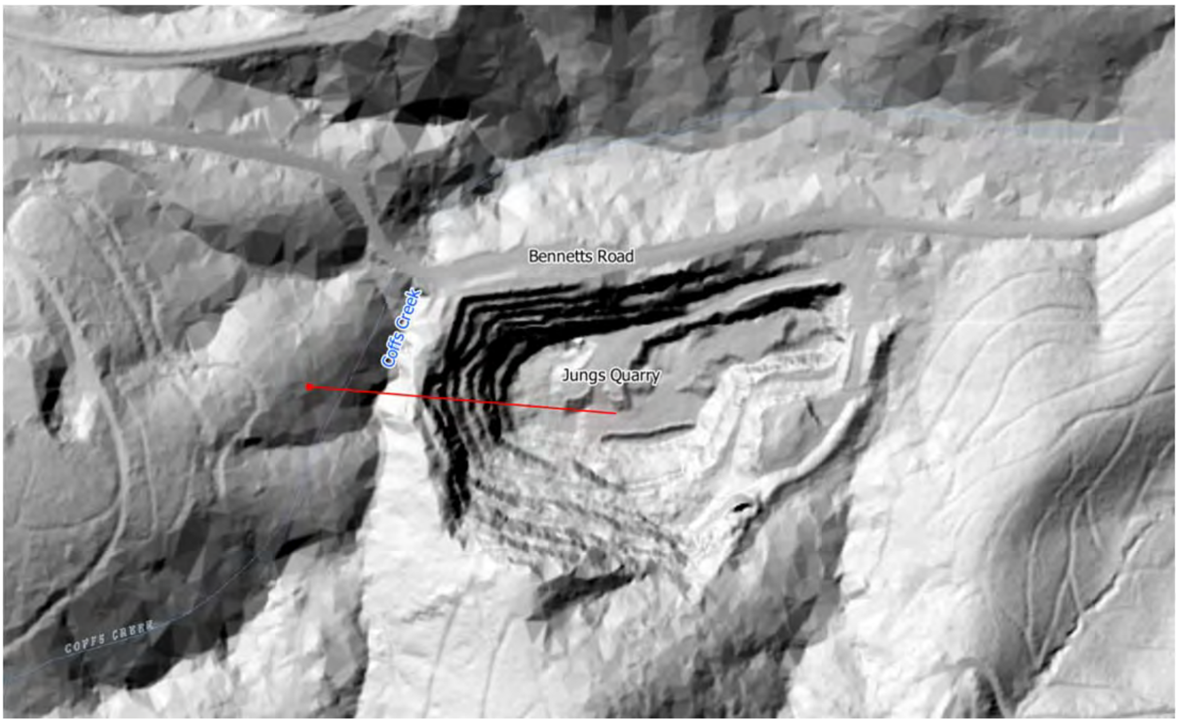


Figure 10 *Topography of Jungs Bennetts Rd Quarry and Coffs Creek (cross section line shown in red)*

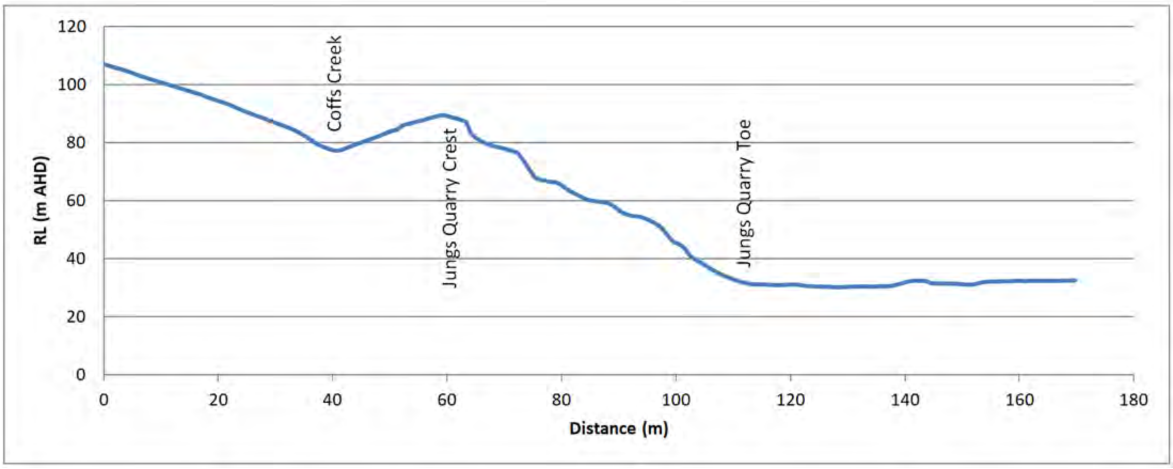


Figure 11 *Cross section of Jungs Bennetts Rd Quarry western wall*

12.2.5 GROUNDWATER DRAWDOWN

The general shape of the modelled groundwater drawdown with distance from the cuts is shown in figures in **Appendix D** .

The predicted distance to 1m and 2m groundwater surface drawdowns from the toe of the cut batters is shown on **Table 6**.

Table 6 *Distance from the toe of the cut to 1m and 2m drawdown (along the modelled section)*

Cut	Recharge Infiltration	Distance from Up-slope Cut Toe to 1m Drawdown (m)	Distance from Up-slope Cut Toe to 2m Drawdown (m)	Distance from Down-slope Cut Toe to 1m Drawdown (m)	Distance from Down-slope Cut Toe to 2m Drawdown (m)
Cut 4	5%	223.29	206.51	49.87	39.75
	15%	221.29	208.25	33.07	29.78
Cut 8	5%	68.69	60.38	31.96	12.66
	15%	98.99	75.57	36.69	27.92
Cut 8a	5%	77.98	61.38	202.91	163.24
	15%	99.53	87.07	106.16	91.49
Cut 11	5%	130.99	95.48	84.59	60.77
	15%	142.75	131.74	189.59	125.75
Cut 12	5%	63.60	53.61	56.30	48.86
	15%	65.20	52.61	93.73	44.76
Cut 14(2)	5%	56.88	5.32	9.68	N/A(3)
	15%	148.17	38.00	29.77	N/A(3)
Cut 16	5%	146.45	133.97	90.28	74.96
	15%	133.94	128.17	90.07	74.25
Cut 18	5%	166.87	161.31	130.96	66.33
	15%	190.78	168.97	124.66	111.93

1. The values reported are as calculated and it should be noted that the number of significant figures does not infer a level of precision.
2. No reliable groundwater data for Cut 14. Results based off simulations using assumed parameters.
3. Simulated drawdown falls within the footprint of the cutting base.

It is expected the drawdown effect will be minimal in the adjacent valleys.

12.2.6 GROUNDWATER FLOWS INTO THE DOWN GRADIENT BOUNDARY (MODELLED GROUNDWATER RECEPTOR)

Table 7 provides a summary of the calculated groundwater flow rates at the down gradient boundary of the cuts determined by the groundwater models.

Table 7 *Calculated Flow Rate at Down Gradient Modelled Receptor (L/day) and Approximate Effect*

Cut	Pre-cut flow rate predicted at down gradient modelled receptor Q1 (L/day) per m	Post-cut flow rate at modelled receptor Q2 (L/day) per m	% reduction in flow rate at modelled downstream receptor in the absence of mitigation measures
Cut 4 / CH13325	44.01 (141.96)	9.73 (23.71)	77.9% (83.3%)
Cut 8 / CH13925	80.18 (208.66)	15.75 (39.46)	80.4% (81.1%)
Cut 8a / CH14300	58.69 (169.08)	35.51 (152.50)	39.5% (9.8%)
Cut 11 / CH15750	25.95 (76.50)	2.81 (3.29)	89.2% (95.7%)
Cut12/CH16075	22.15 (75.95)	13.25 (48.97)	40.2% (35.5%)
Cut 14/CH16850	198.29 (532.39)	181.16 (471.80)	8.6% (11.4%)
Cut 16/CH18450	153.92 (437.44)	76.80 (233.42)	50.1% (46.6%)
Cut 18 / CH20425	58.29 (169.58)	27.56 (82.83)	52.7% (51.2%)

1. The values reported are as calculated and it should be noted that the number of significant figures does not infer a level of precision.
2. The value without brackets is based on a recharge rate of 5% of the average annual rainfall
3. The bracketed values represent the calculated flow rate for 15% recharge.
4. No reliable groundwater data for Cut 14. Results based off simulations using assumed parameters.

13 MONITORING AND MITIGATION

13.1 GENERAL

The potential impacts on the local groundwater conditions at each cut indicated by the modelling are summarised in **Table 8**.

Potential requirements for monitoring and mitigation are also identified. The implementation of the management measures described in Section 13.3 would be expected to reduce the impacts described in **Table 8**.

Table 8 *Groundwater Impact Assessment for the Modelled Cuts*

Cut Name	Groundwater Impact Assessment before Mitigation
Type A Cut with more than minimal aquifer interference	
Cut C4	Head waters of Newport Creek and foot slopes identified as potential groundwater receptors. Drawdown in the vicinity of the cut is expected to be in excess of the fluctuation in groundwater levels produced by climatic variations.
Cut C8/8A	Foots slopes and head waters of Coffs Creek, drainage channels, dams and foot slopes identified as potential groundwater receptors. Reduction of groundwater to local creeks and dams expected. Drawdown of groundwater surface expected in environs of cutting. Drawdown in the vicinity of the cut is expected to be in excess of the fluctuation in groundwater levels produced by climatic variations.
Cut C11/C12/C14(1)/C16	Foots slopes and head waters of Coffs Creek/Treefern Creek identified as potential groundwater receptor. Drawdown in the vicinity of the cut is expected to be in excess of the fluctuation in groundwater levels produced by climatic variations.
Cut C18	Jordans Creek identified as potential groundwater receptor. Some reduction of groundwater for downstream receptors expected. Drawdown in the vicinity of the cut is expected to be in excess of the fluctuation in groundwater levels produced by climatic variations.
Type B Cuttings	
	Groundwater generally below cut depth. Accordingly no reduction of groundwater to down gradient receptors.

(1) No groundwater monitoring data is available for Cut C14.

The results of the analysis summarised in **Table 8** suggest that there are potential impacts from the proposed upgrade cuttings on the local groundwater regime. The management regime identified in Section 13.3, could be employed to mitigate the impacts of the cuttings and the reduction in groundwater where groundwater dependent ecosystems are identified.

13.2 GENERAL MITIGATION AND MONITORING

Analysis indicates the following general approach to effectively manage and mitigate groundwater impacts, and potential uncertainties about the actual impacts.

Type A cuts - The implementation of engineering measures such as subgrade drainage blankets beneath the pavement are expected to be required as part of design/construction to mitigate any groundwater impacts on the proposed pavement.

There is a high likelihood that cuts could affect the local groundwater regime in the vicinity of the cut and reduction of groundwater flow to local streams. Groundwater collected in the pavement drainage system could be diverted back into any such natural creek systems.

Monitoring of existing groundwater works (GWW) could be undertaken to quantify any effects such as loss of saturated bore depth which could be restabilised by deepening of the bores or similar.

Type B cuts - It is unlikely that these cuts would have an adverse impact on groundwater regimes. During wet climatic periods the water level in Type B cuts may rise above the cut level. Accordingly, the installation of pavement subgrade drainage blankets in these cuts would be a prudent measure.

Monitoring of the groundwater level is an essential measure to mitigate uncertainty in predictions of groundwater behaviour, which have been based largely on groundwater observations over a relatively short period of time and two dimensional steady state groundwater modelling.

13.3 ENGINEERING MITIGATION MEASURES

Where monitoring indicates a reduction in the saturated depth at existing GWW occurs this could be addressed by the deepening of the bores to re-establish the pre road works saturated bore depth.

Environmental engineering mitigation measures that could be considered at Type A cuts (and at Type B cuts, if groundwater monitoring and mapping of GDE indicates that engineering mitigation is required) include the following:

- Transferring the surface and seepage water captured by the cuts via water quality control ponds to downstream natural drainage water courses.
- Transferring groundwater (where present) captured in the subgrade drainage blankets and subsurface drainage systems back into the groundwater system immediately downstream of the cut. The collected water could be returned to the groundwater system through absorption trenches or similar infiltration galleries.

From the perspective of reducing the impact on downstream receptors, the second option above, would provide some amelioration of the effect of the cuttings on the local groundwater system. It is expected that a system combining both a return of water to the surface water and groundwater system downstream of the cutting would be generally applicable as a mitigation measure. The preferred method and form of the mitigation measures would be the subject of ongoing development of the concept design and environmental assessment process.

The volumes of groundwater predicted to enter the cuttings under steady state conditions are not large. Despite this protection of pavements is required to prevent poor performance. Typical requirements are for pavement subgrade drainage layers as specified in RMS R44.

In addition to this, it is noted that there is a likelihood of seepage emanating on the batters of the cuts. While this is considered not to be an issue in regard to cut stability for cuts batted at 2H:1V or less, should the batters be steepened the effect of groundwater pressure may become significant and should be considered in cut batter stability assessments.

Yours faithfully
RCA AUSTRALIA

Robert Carr

Thomas Hosking

Robert Carr
Principal Geotechnical Engineer

Thomas Hosking
Senior Geotechnical Engineer

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[2] PSM, Groundwater Assessment Report (Draft), Pacific Highway Upgrade, Coffs Harbour Bypass Option B, prepared for RCA Australia for the Roads and Maritime Services by PSM, ref PSM2876-057R Draft, 26 April 2019.

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[5] RCA Australia, *Geotechnical Investigation Report, Section 1 (Tie-in south of Englands Road to Ch15340), Coffs Harbour Bypass*, RCA Australia, ref 11717-806/1, June 2018.

[6] RCA Australia, *Geotechnical Investigation Report, Section 2, Coffs Harbour Bypass,* RCA Australia, ref: 11717-803/1, June 2018.

[7] RCA Australia, *Geotechnical Investigation Report, Section 3 (North of Gatelys Tunnel northern portal (Ch19570) to Korora tie-in), Coffs Harbour Bypass*, RCA Australia ref: 11717-807/1, June 2018.

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[14] Korsch, RJ, *Some tectonic implications of sandstone petrofacies in the Coffs Harbour Association, New England Orogen, New South Wales*. Journal of the Geological Society of Australia, 1981.

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[17] NSW Government Policy for the Licencing and Assessment of Aquifer Interference Activities, NSW Department of Primary Industries, 2012.

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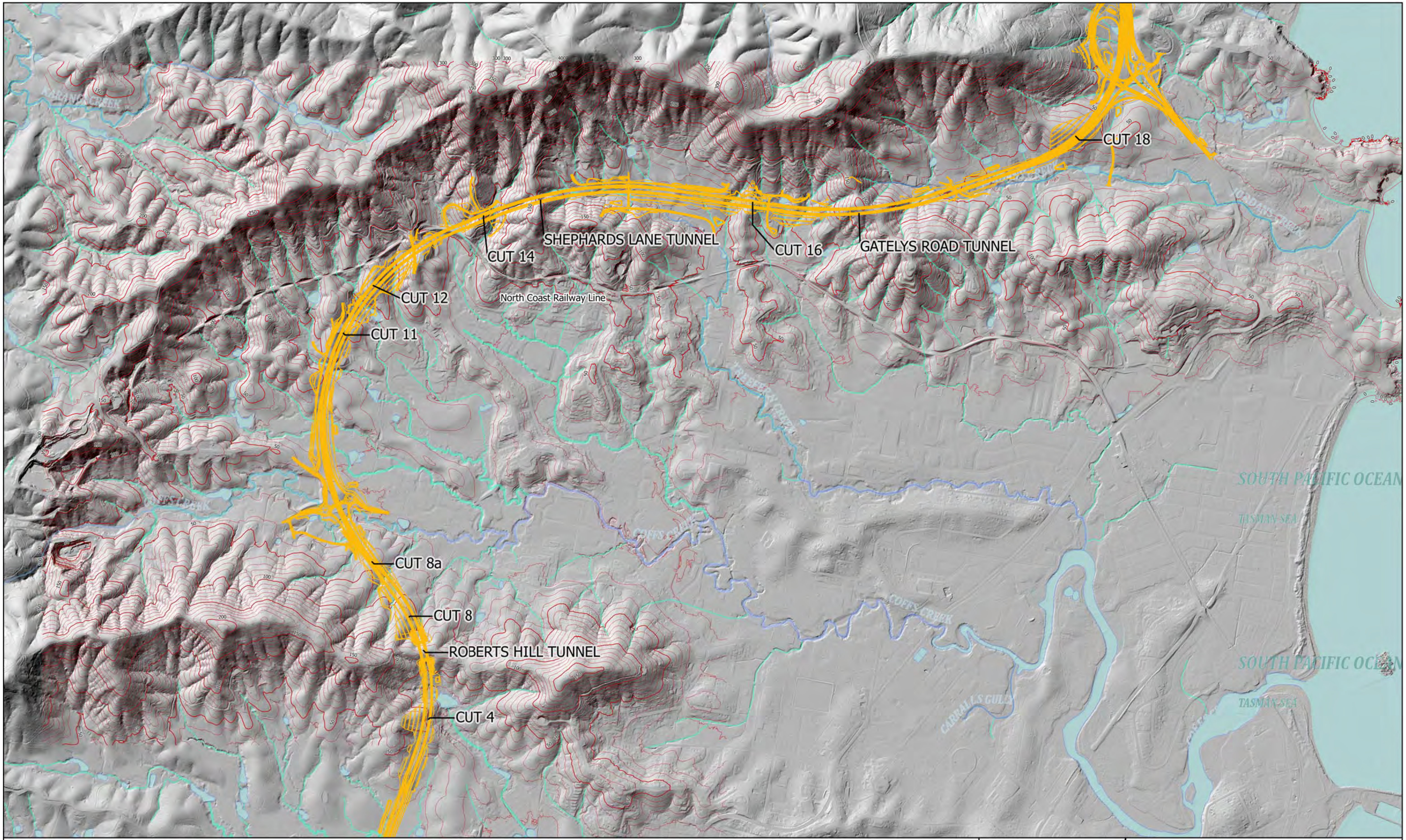
[20] Barnett B, Townley LR, Post V, Evans RE, Hunt RJ, Peeters L, Richardson S, Werner AD, Knapton A and Boronkay A., 2012. Australian groundwater modelling guidelines, Waterlines Series Report No. 82, National Water Commission, Canberra. June 203pp

Appendix A

Drawings showing:

- Topographic Setting (Drawing 1)
- Geological Setting (Drawings 2, 3 & 4)
- Long Section showing Groundwater Levels, Cutting Type Classification and Location of Modelled Cross-sections (Drawings 5 to 9)
- The adopted Cross-Sections, Groundwater Surfaces and Subsurface Profiles for Modelling purposes (Drawings 10 to 18):
  - Cut 4
  - Cut 8
  - Cut 8A
  - Cut 11
  - Cut 12
  - Cut 14
  - Cut 16
  - Cut 18



NOTES: Topographic data obtained from DFSI Spatial Services
Allignment supplied by ARUP
(Ref: 248379-000-RD-MOD-003, February 2019)

Contour Interval 10 Meters

0 500 1000 1500 2000 m

N

RCA AUSTRALIA
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TOPOGRAPHIC SETTING
PACIFIC HIGHWAY UPGRADE
COFFS HARBOUR BYPASS

CLIENT	Roads and Maritime Services			RCA Ref	11717-818/2		
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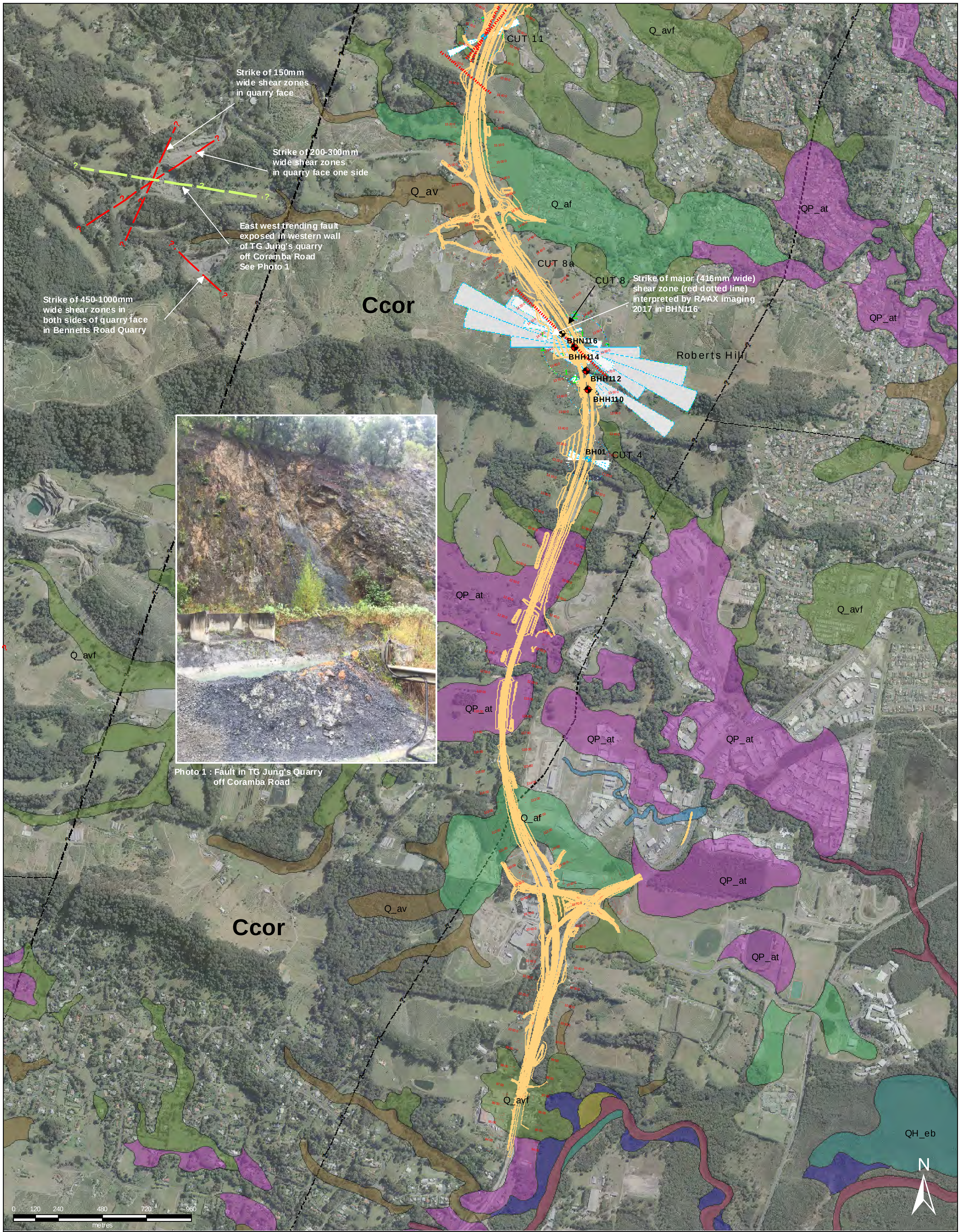
APPROVED BY	RJC	DATE	11/06/2019	OFFICE	NEWCASTLE		



LEGEND

RCA GIR Test Locations

- N bore alignment
- H bore alignment

Connell Wagner Report S10369/9-AX 2007

- Borehole location
- Rossettes of bedding strike from exposure mapping
- Rossettes of bedding strike from borehole imaging

- Fault in exposure
- Shear zone in exposure
- Faults interpreted from imaging
- Major shear zones interpreted from imaging

Geological Units NSW Code

- QH_bd - Coastal deposits- dune facies
- QH_bf - Coastal deposits - backbarrier flat facies
- Q_bb - Coastal deposits- beach facies
- QH_eb - Central mud basin
- QH_ebw - Estuarine basin and by (subaqueous)
- QH_ecw - Estuarine channel deposits (subaqueous)
- QH_es - Estuarine swamp
- QP_at - Alluvial terrace deposits
- Q_af - Alluvial floodplain deposits
- Q_ap - Alluvial palaeochannel deposits
- Q_av - Alluvial valley deposits
- Q_avf - Alluvial fan deposits
- CZ_ath - Alluvial terrace deposit-high-stand facies

- Ccoc - Coramba beds
- Ccor - Brooklana beds

Geological Boundary

- Geological boundary, position approximate

Fault

- Fault, inferred

NOTES
Alignment - Supplied by ARUP
(Ref: 248379-000-RD-MOD-0002 &
248379-000-RD-MOD-0003, Febraury 2019)

Geology - 2015 NSW Zone 56 Seamless
Geology Version 1 (Digital Dataset)
Geological Survey of NSW, Maitland

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GEOLOGICAL SETTING
PACIFIC HIGHWAY UPGRADE
COFFS HARBOUR BYPASS

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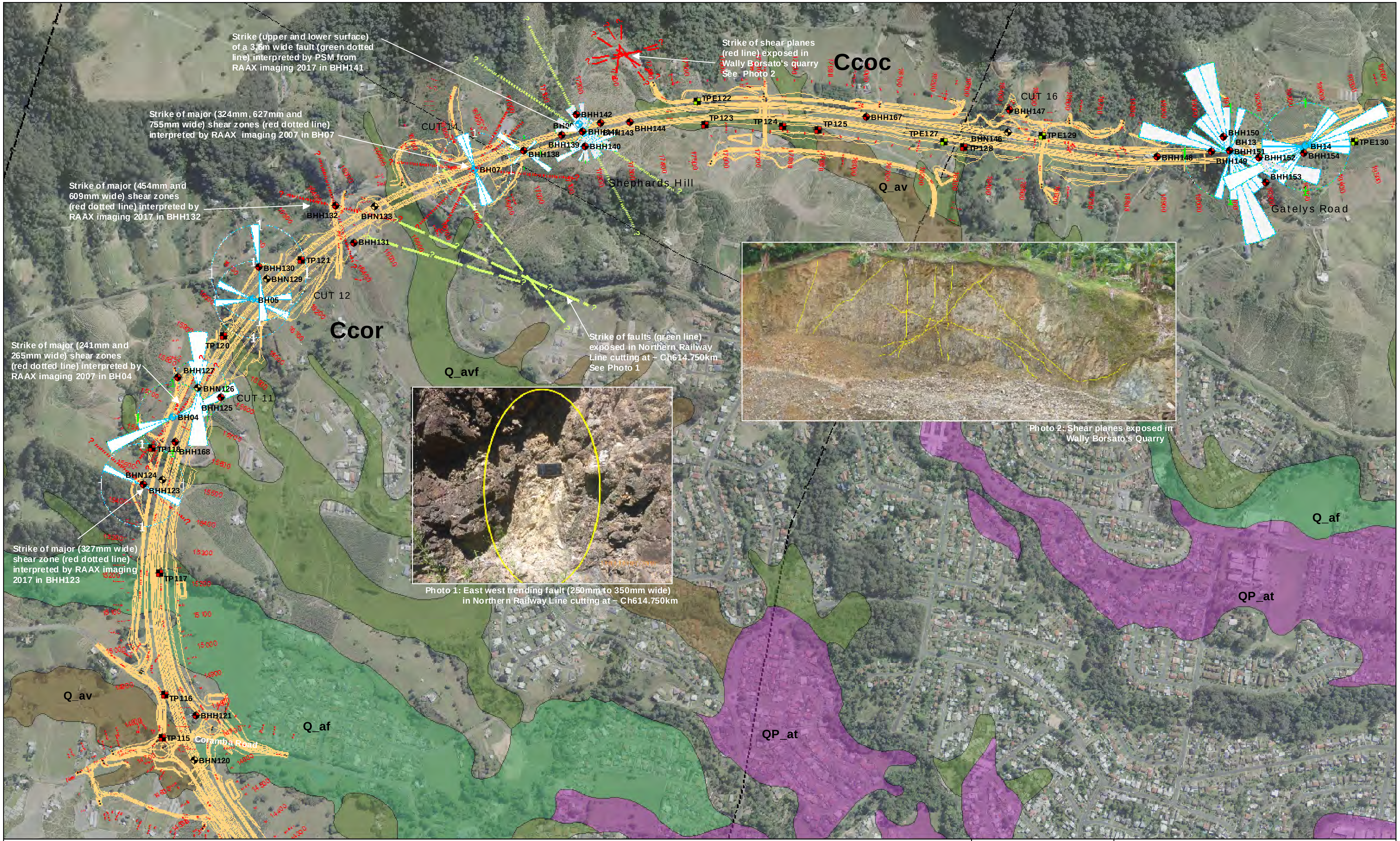

Photo 2: Shear planes exposed in Wally Borsato's Quarry

Photo 1: East west trending fault (250mm to 350mm wide) in Northern Railway Line cutting at ~ Ch614.750km

LEGEND

RCA GIR Test Locations
- N bore alignment
- H bore alignment

Connell Wagner Report S10369/9-AX 2007
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- QH_es - Estuarine swamp
- QP_at - Alluvial terrace deposits
- Q_af - Alluvial floodplain deposits
- Q_ap - Alluvial palaeochannel deposits
- Q_av - Alluvial valley deposits
- Q_avf - Alluvial fan deposits
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Geological Boundary
- Geological boundary, position approximate

Fault
- Fault, inferred

NOTES
Alignment - Supplied by ARUP
(Ref: 248379-000-RD-MOD-0002 &
248379-000-RD-MOD-0003, Febraury 2019)

Geology - 2015 NSW Zone 56 Seamless
Geology Version 1 (Digital Dataset)
Geological Survey of NSW, Maitland

0 75 150 300 450 600
metres

N

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GEOLOGICAL SETTING
PACIFIC HIGHWAY UPGRADE
COFFS HARBOUR BYPASS

CLIENT	Roads and Maritime Services			RCA Ref	11717-818/2
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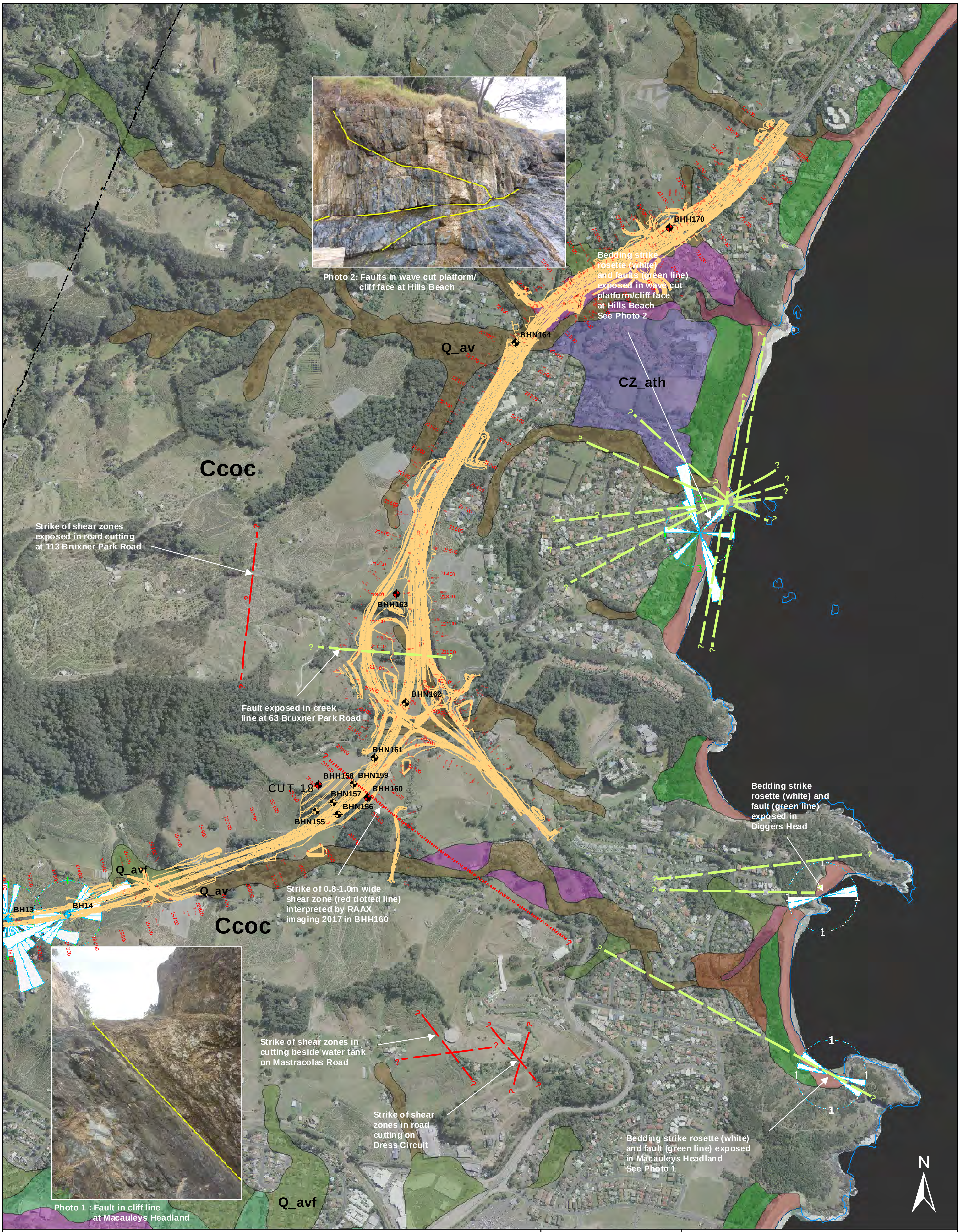

LEGEND

RCA GIR Test Locations

- N bore alignment
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Connell Wagner Report S10369/9-AX 2007

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- QH_ebw - Estuarine basin and by (subaqueous)
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- Ccoc - Coramba beds
- Ccor - Brooklana beds

Geological Boundary

- Geological boundary, position approximate

Fault

- Fault, inferred

NOTES
Alignment - Supplied by ARUP
(Ref: 248379-000-RD-MOD-0002 &
248379-000-RD-MOD-0003, Febraury 2019)

Geology - 2015 NSW Zone 56 Seamless
Geology Version 1 (Digital Dataset)
Geological Survey of NSW, Maitland

0 50 100 200 300 400
metres

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GEOLOGICAL SETTING
PACIFIC HIGHWAY UPGRADE
COFFS HARBOUR BYPASS

CLIENT	Roads and Maritime Services			RCA Ref	11717-818/2
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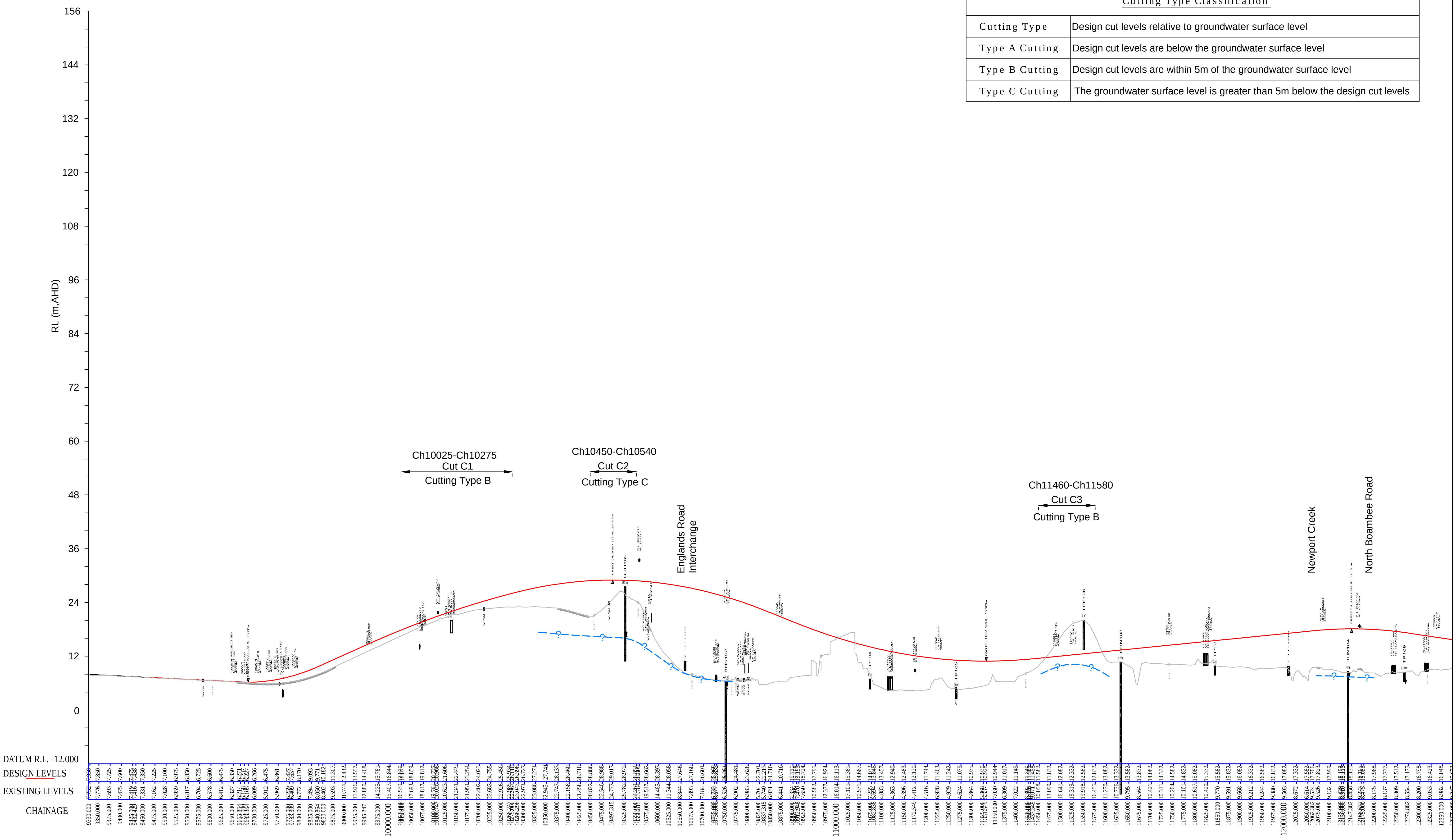
APPROVED BY	MA	DATE	11/06/2019	OFFICE	NEWCASTLE

Cutting Type Classification	
Cutting Type	Design cut levels relative to groundwater surface level
Type A Cutting	Design cut levels are below the groundwater surface level
Type B Cutting	Design cut levels are within 5m of the groundwater surface level
Type C Cutting	The groundwater surface level is greater than 5m below the design cut levels



LEGEND

Vertical alignment

Approximate groundwater surface

SCALE:
1 : 800 V
1 : 8000 H

RCA AUSTRALIA
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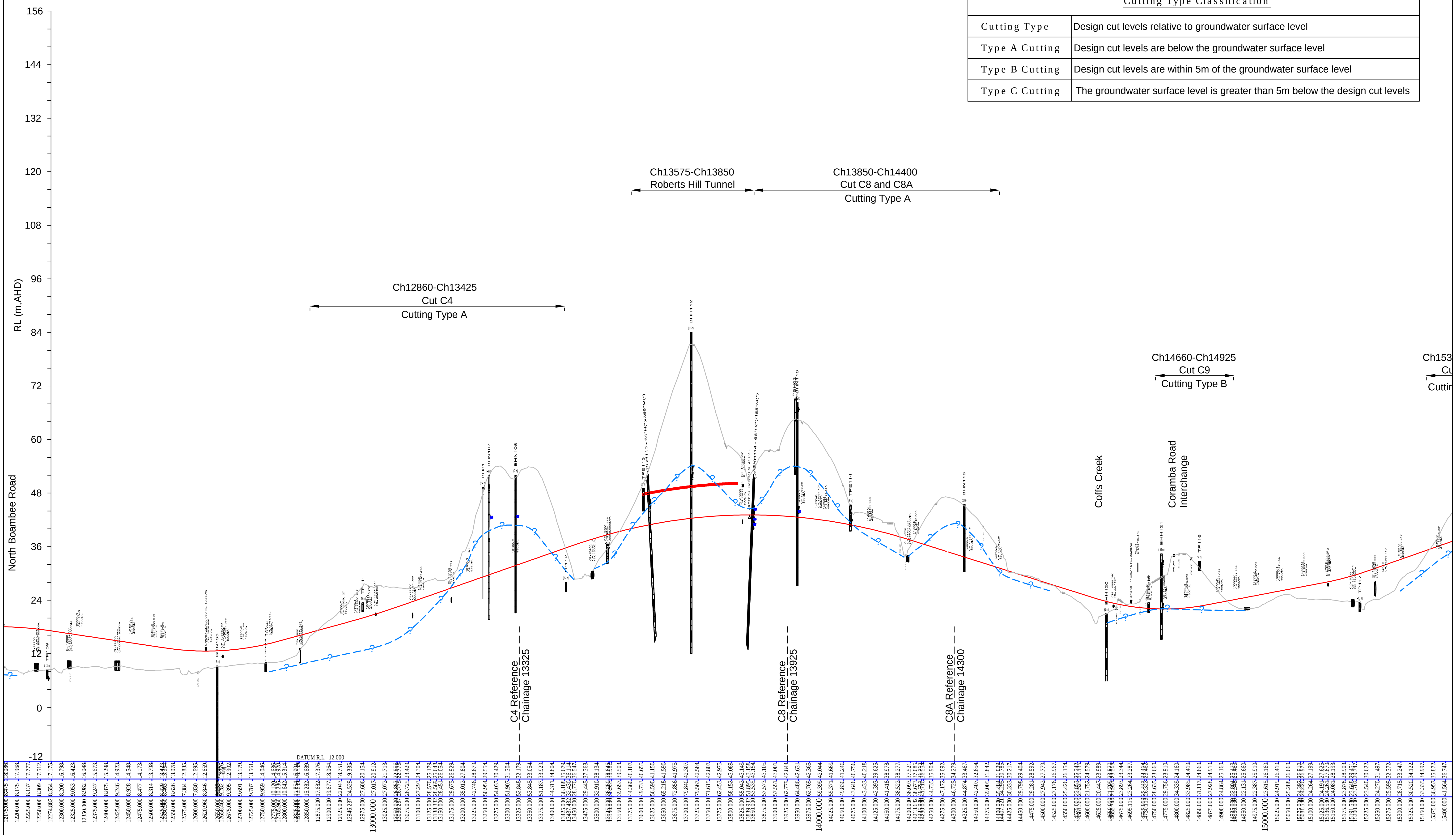
LONG SECTION SHOWING CUT NUMBERS, INTERPRETED GROUNDWATER SURFACE, CUTTING TYPE CLASSIFICATION AND LOCATION OF MODELLED CROSS SECTIONS
PACIFIC HIGHWAY UPGRADE
COFFS HARBOUR BYPASS

CLIENT	Roads and Maritime Services			RCA Ref	11717-818/2
DRAWN BY	TH	SCALE	As Shown	DRAWING No	5 REV 0
APPROVED BY	RJC	DATE	11/06/2019	OFFICE	NEWCASTLE

CDT-DWG-A3H-001/1

Cutting Type Classification	
Cutting Type	Design cut levels relative to groundwater surface level
Type A Cutting	Design cut levels are below the groundwater surface level
Type B Cutting	Design cut levels are within 5m of the groundwater surface level
Type C Cutting	The groundwater surface level is greater than 5m below the design cut levels



LEGEND

Vertical alignment

Approximate groundwater surface

SCALE:
1 : 800 V
1 : 8000 H

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

LONG SECTION SHOWING CUT NUMBERS, INTERPRETED GROUNDWATER SURFACE, CUTTING TYPE CLASSIFICATION AND LOCATION OF MODELLED CROSS SECTIONS
PACIFIC HIGHWAY UPGRADE
COFFS HARBOUR BYPASS

CLIENT	Roads and Maritime Services			RCA Ref	11717-818/2
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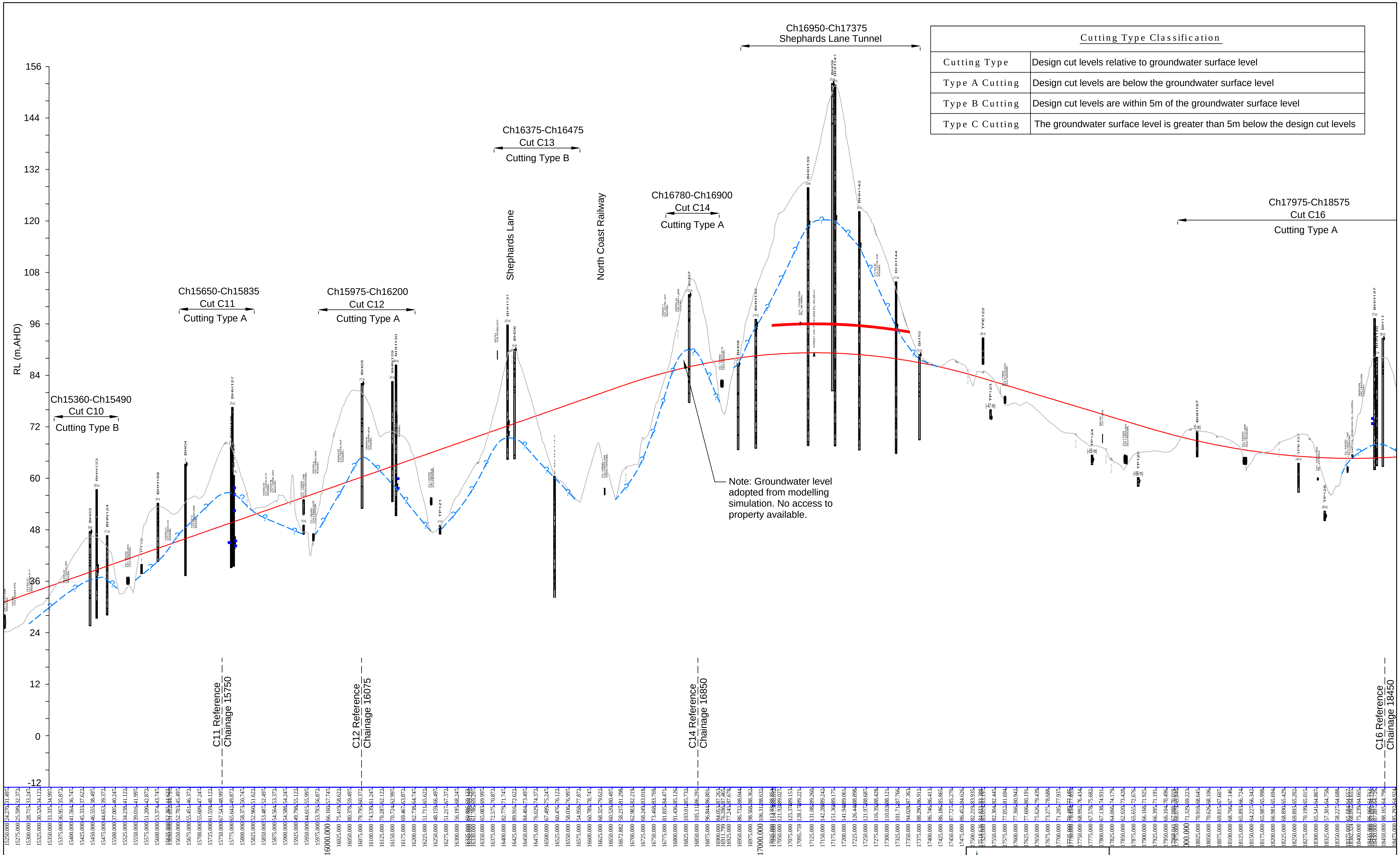
CDT-DWG-A3H-001/1



Cutting Type Classification	
Cutting Type	Design cut levels relative to groundwater surface level
Type A Cutting	Design cut levels are below the groundwater surface level
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LEGEND

Vertical alignment

Approximate groundwater surface

SCALE:
1 : 800 V
1 : 8000 H

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

LONG SECTION SHOWING CUT NUMBERS, INTERPRETED GROUNDWATER SURFACE, CUTTING TYPE CLASSIFICATION AND LOCATION OF MODELLED CROSS SECTIONS
PACIFIC HIGHWAY UPGRADE
COFFS HARBOUR BYPASS

CLIENT	Roads and Maritime Services			RCA Ref	11717-818/2
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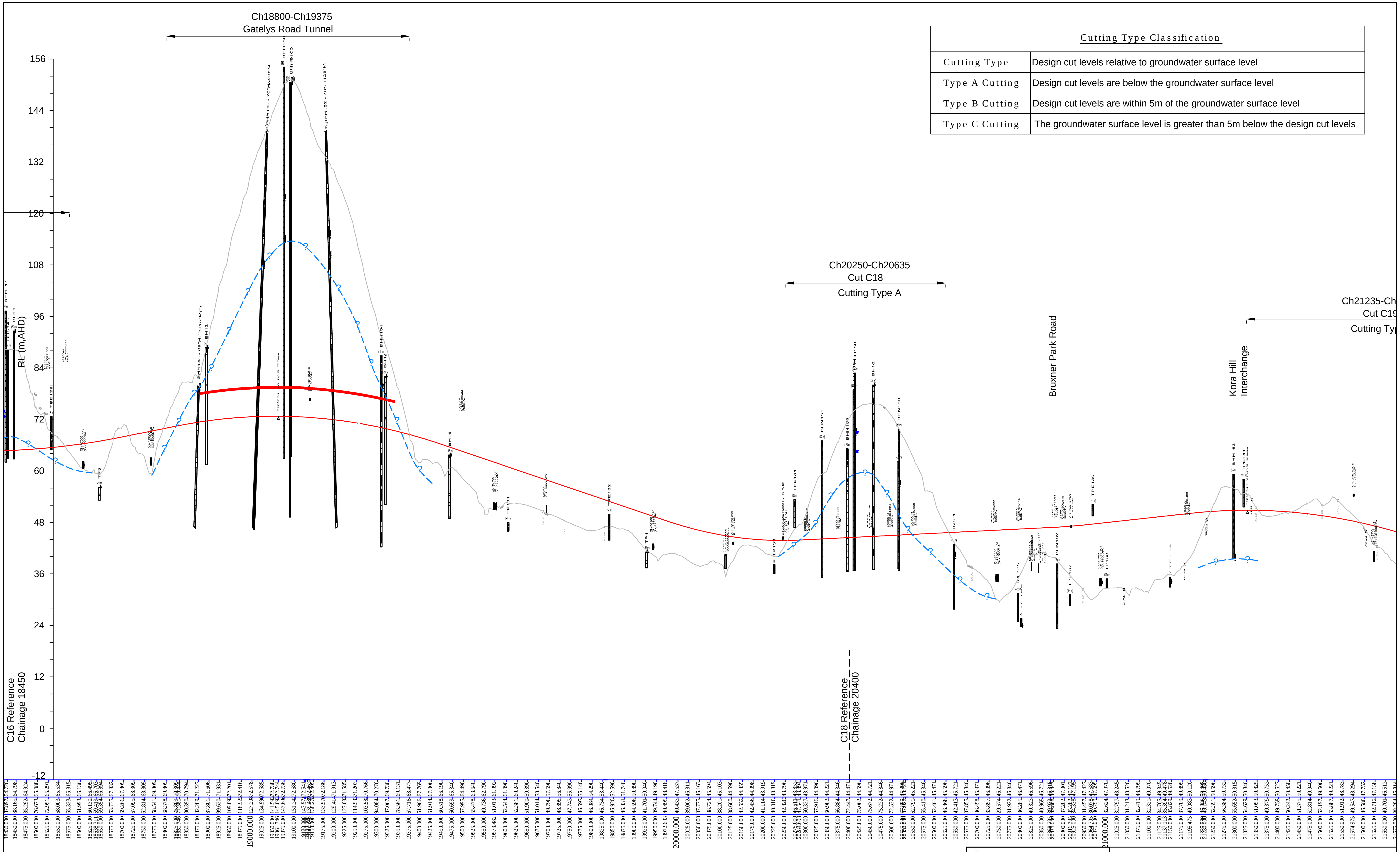
CDT-DWG-A3H-001/1



Cutting Type Classification	
Cutting Type	Design cut levels relative to groundwater surface level
Type A Cutting	Design cut levels are below the groundwater surface level
Type B Cutting	Design cut levels are within 5m of the groundwater surface level
Type C Cutting	The groundwater surface level is greater than 5m below the design cut levels

LEGEND

Vertical alignment

Approximate groundwater surface

SCALE:
1 : 800 V
1 : 8000 H

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

LONG SECTION SHOWING CUT NUMBERS, INTERPRETED GROUNDWATER SURFACE, CUTTING TYPE CLASSIFICATION AND LOCATION OF MODELLED CROSS SECTIONS
PACIFIC HIGHWAY UPGRADE
COFFS HARBOUR BYPASS

CLIENT	Roads and Maritime Services			RCA Ref	11717-818/2
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APPROVED BY	RJC	DATE	11/06/2019	OFFICE	NEWCASTLE

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Cutting Type Classification	
Cutting Type	Design cut levels relative to groundwater surface level
Type A Cutting	Design cut levels are below the groundwater surface level
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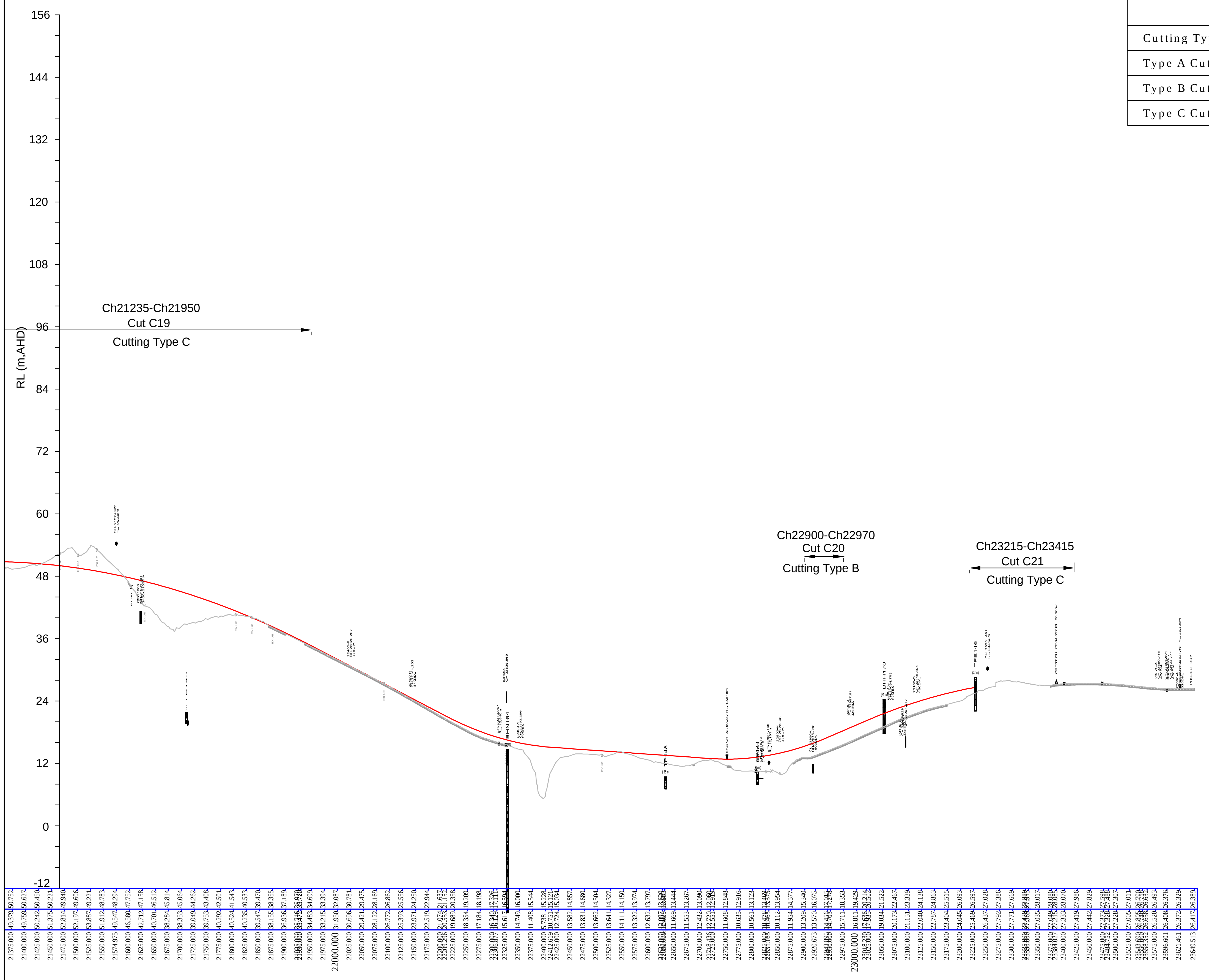

LEGEND

Vertical alignment

Approximate groundwater surface

SCALE:
1 : 800 V
1 : 8000 H

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

LONG SECTION SHOWING CUT NUMBERS, INTERPRETED GROUNDWATER SURFACE, CUTTING TYPE CLASSIFICATION AND LOCATION OF MODELLED CROSS SECTIONS
PACIFIC HIGHWAY UPGRADE
COFFS HARBOUR BYPASS

CLIENT	Roads and Maritime Services			RCA Ref	11717-818/2
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APPROVED BY	RJC	DATE	11/06/2019	OFFICE	NEWCASTLE

CDT-DWG-A3H-001/1

LEGEND

- ASPHALT
- FILL
- CI - Medium Plasticity CLAY
- CI - CH Medium Plasticity to High Plasticity CLAY
- ML - Low Plasticity SILT
- GP - Poorly Graded GRAVEL
- CORE LOSS
- ARGILLITE/SILICEOUS ARGILLITE
- Meta SANDSTONE/SANDSTONE

GEOLOGICAL UNITS

Soils	Indicative Seismic Velocitys (m/sc)
1a Fill	350-800
1b Topsoil	
1c Alluvium	
1d Colluvium/Slopewash	
1e Residual	
Rock	
2a EL to L strength, EW-HW	>800-1500
2b L-H strength, MW-SW	>1500-2500
2c M-VH strength, SW-F	>2500

Groundwater/Seepage Description

- Groundwater level in open hole
- Groundwater level in piezometer
- Seepage recorded at time of investigation

Seismic Line

- Existing surface level on Seismic line
- 1800 — Seismic velocity (m/s) and interpreted refractor boundary based on reciprocal method minus times and time depths
- 1500 — Seismic velocity (m/s) and interpreted refractor boundary based on limited data*
- 1800 — Hatched area seismic velocity (m/s) is based on limited data* and the value is the same as the adjacent minus times velocity
- Lateral seismic velocity boundary

NB * Limited data includes harmonic mean velocity, interpolated time depth or edited data

ROCK DESCRIPTION

Rock Strength

EL	Extremely low
VL	Very low
L	Low
M	Medium
H	High
VH	Very high
EH	Extremely High

Rock Weathering

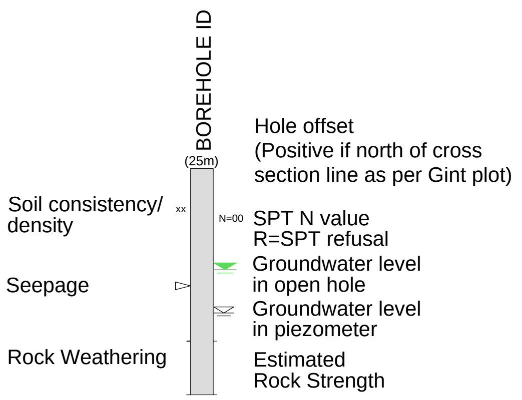
EW	Extremely weathered
HW	Highly weathered
MW	Moderately weathered
SW	Slightly weathered
F	Fresh

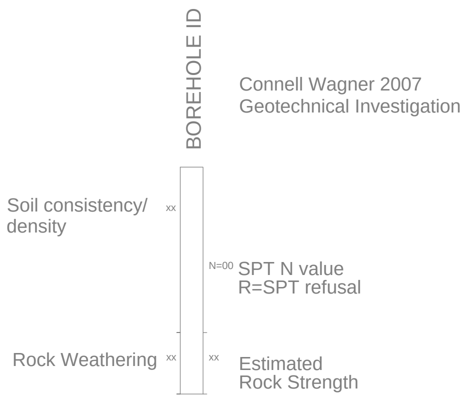
Cohesive Soil Consistency

VS	Very soft
S	Soft
F	Firm
St	Stiff
VSt	Very stiff
H	Hard

Granular Soil Relative Density

VL	Very loose
L	Loose
MD	Medium dense
D	Dense
VD	Very dense





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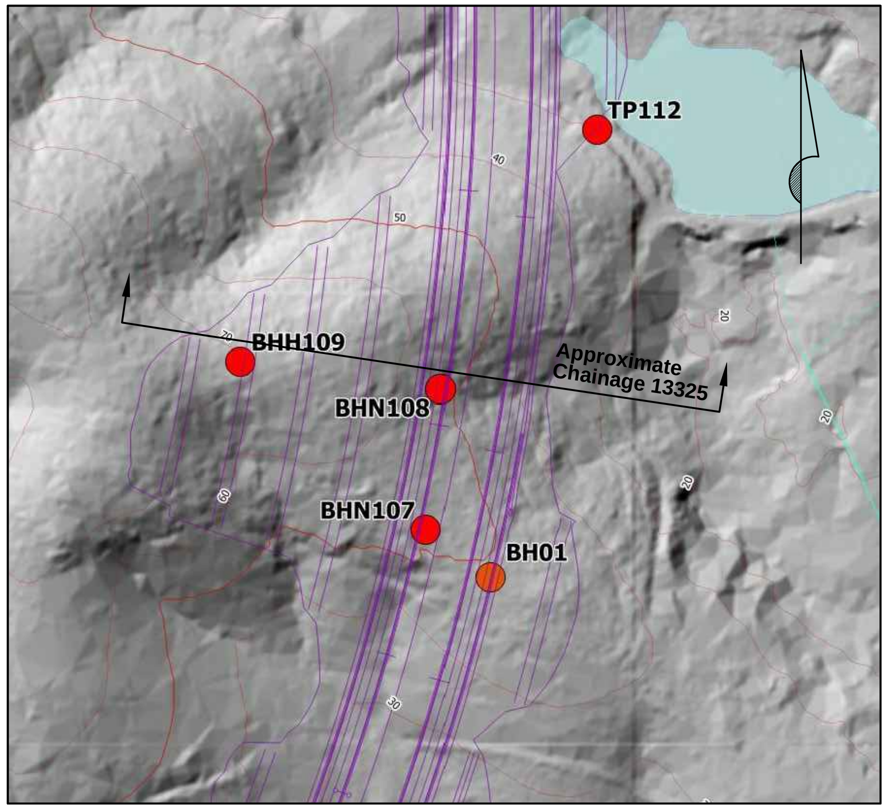
GEOTECHNICAL LEGEND
PACIFIC HIGHWAY UPGRADE
COFFS HARBOUR BYPASS

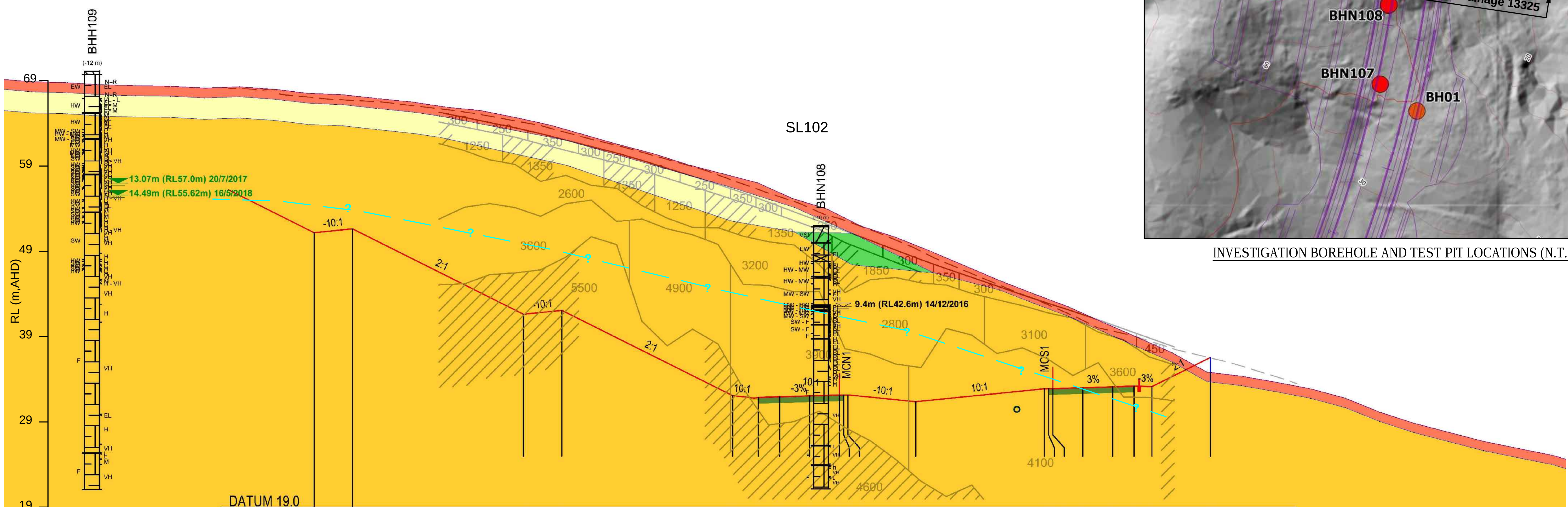
CLIENT	Roads and Maritime Services				
DRAWN BY	RJC	SCALE	N.T.S.	RCA Ref	11717-818/2
APPROVED BY	RJC	DATE	11/06/2019	DRAWING No	10 Rev 0

OFFICE NEWCASTLE

CDT-DWG-A4V-001/1



INVESTIGATION BOREHOLE AND TEST PIT LOCATIONS (N.T.S)



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IBN1	51.644	66.690	-57.000
ICN0	41.644	64.114	-37.000
IBN0	42.094	63.062	-32.500
EHN0	32.094	57.332	-12.500
CEN0	31.894	56.259	-9.500
LQN0	31.969	55.407	-7.000
L3N0	32.074	54.102	-3.500
LUNI	32.179	52.848	0.000
EONI	32.194	52.683	0.500
EVNI	32.179	52.498	1.000
DMNI	31.386	49.330	8.935
EVS0	32.892	42.936	24.000
EOS0	32.917	42.718	24.500
LUS0	32.932	42.501	25.000
L3SI	33.037	40.929	28.500
LQSI	33.142	39.657	32.000
ESSI	33.217	39.078	34.500
EHSI	33.154	38.399	36.602
IASI	36.579	36.519	43.453

CHAINAGE 13325
CUT 4

NOTE:

1. Cross section taken from Arup, Feb 2019
   Dwg. No. 248379-000-RD-MOD-1501_2019.dwg
2. The geological units and stratigraphy shown on the sections should be regarded as a generalised summary only. For further detail and for design purposes reference should be made to the relevant borehole and test pit logs.
3. Thin surficial soil layers (ie topsoil, slopewash etc) are not shown for clarity.
4. Strength and weathering nomenclature on boreholes drilled by others prior to this investigation has been retained.
5. The groundwater level/profile shown is based on groundwater levels measured in piezometers at the date shown and the groundwater level/profile could be expected to change with variations in climatic conditions etc.

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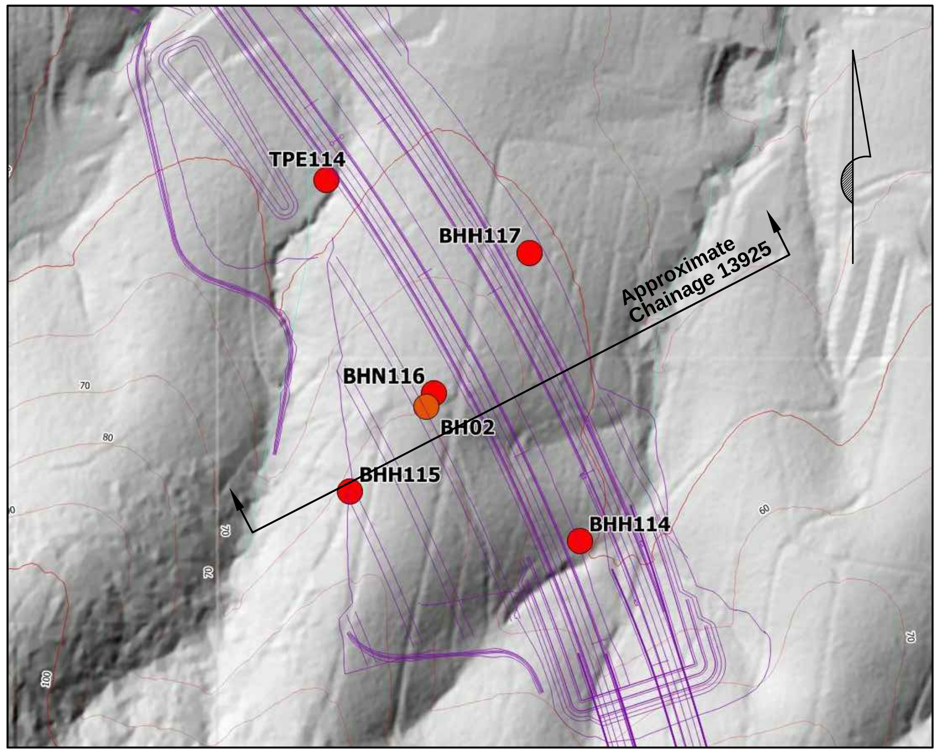
GEOTECHNICAL • ENVIRONMENTAL

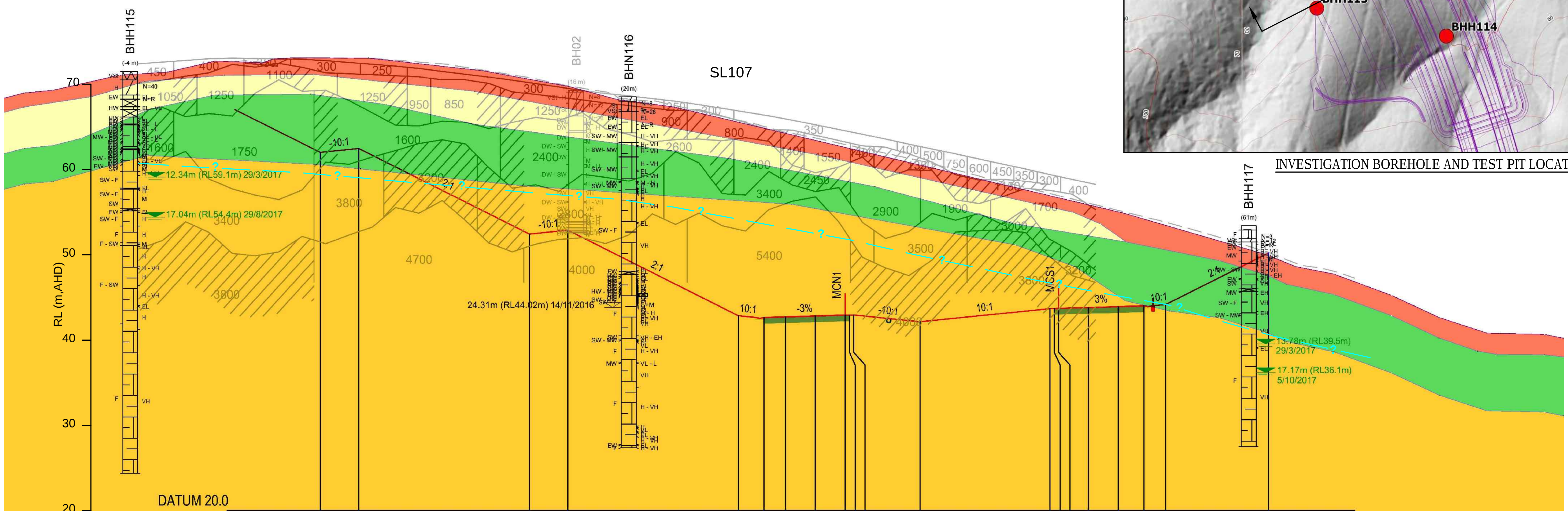
CUT 4 - CHAINAGE 13325
PACIFIC HIGHWAY UPGRADE
COFFS HARBOUR BYPASS

CLIENT	Roads and Maritime Services			RCA Ref	11717-818/2
DRAWN BY	TH	SCALE	1 : 500 (A3)	DRAWING No	11 REV 0
APPROVED BY	RJC	DATE	11/06/2019	OFFICE	NEWCASTLE

CDT-DWG-A3H-001/1



INVESTIGATION BOREHOLE AND TEST PIT LOCATIONS (N.T.S)



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IBN0	52.759	69.649	-32.500
EHN0	42.759	65.630	-12.500
ESN0	42.559	64.887	-9.500
LQN0	42.634	64.210	-7.000
L3N0	42.739	63.537	-3.500
LUNI	42.844	62.779	0.000
E0NI	42.859	62.671	0.500
EVNI	42.844	62.563	1.000
DMNI	42.065	60.875	8.785
EVS0	43.587	57.049	24.000
E0S0	43.612	56.925	24.500
LUS0	43.627	56.802	25.000
L3SI	43.732	55.937	28.500
LQSI	43.837	55.073	32.000
ESSI	43.927	54.333	35.000
EHSI	44.082	53.703	37.555
IASI	50.102	50.168	49.594

CHAINAGE 13925
CUT 8

NOTE:

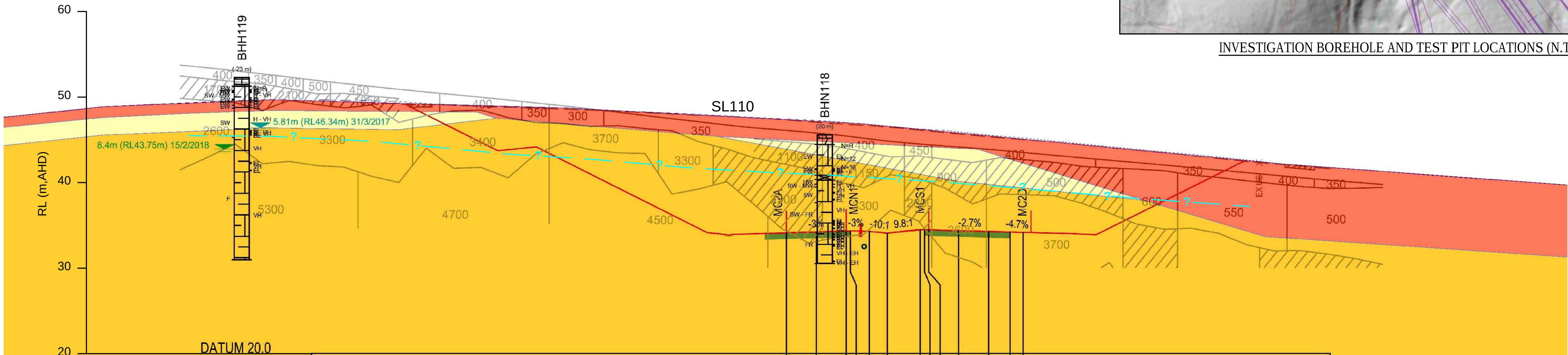
1. Cross section taken from Arup, Feb 2019 Dwg. No. 248379-000-RD-MOD-1501_2019.dwg
2. The geological units and stratigraphy shown on the sections should be regarded as a generalised summary only. For further detail and for design purposes reference should be made to the relevant borehole and test pit logs.
3. Thin surficial soil layers (ie topsoil, slopewash etc) are not shown for clarity.
4. Strength and weathering nomenclature on boreholes drilled by others prior to this investigation has been retained.
5. The groundwater level/profile shown is based on groundwater levels measured in piezometers at the date shown and the groundwater level/profile could be expected to change with variations in climatic conditions etc.

CDT-DWG-A3H-001/1

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

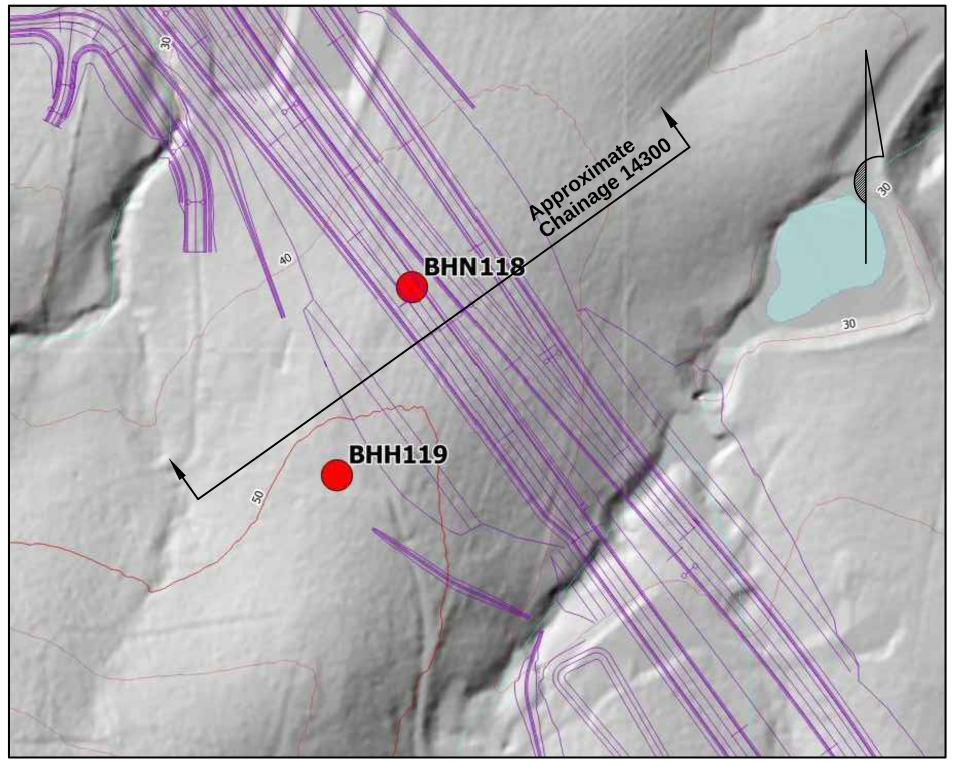
CUT 8 - CHAINAGE 13925
PACIFIC HIGHWAY UPGRADE
COFFS HARBOUR BYPASS

CLIENT	Roads and Maritime Services			RCA Ref	11717-818/2
DRAWN BY	TH	SCALE	1 : 500 (A3)	DRAWING No	12 REV 0
APPROVED BY	RJC	DATE	11/06/2019	OFFICE	NEWCASTLE



STRING LABELS	LAN0	L3N0	LUNI	EONI	EVNI	DMNI	EVS0	EDS0	LUS0	L3SI	LQSI	ESSI	EHSI
DESIGN LEVELS	34.069	34.174	34.279	34.294	34.228	34.019	34.414	34.430	34.417	34.322	34.227	34.159	34.088
EXISTING LEVELS	47.225	46.962	46.725	46.696	46.567	46.404	46.033	45.983	45.932	45.598	45.273	45.070	44.949
OFFSET	-7.000	-3.500	0.000	0.500	2.700	4.797	8.651	9.153	9.655	13.168	16.681	19.190	20.695

CHAINAGE 14300
CUT 8a



INVESTIGATION BOREHOLE AND TEST PIT LOCATIONS (N.T.S)

NOTE:

1. Cross section taken from Arup, Feb 2019
   Dwg. No. 248379-000-RD-MOD-1501_2019.dwg
2. The geological units and stratigraphy shown on the sections should be regarded as a generalised summary only. For further detail and for design purposes reference should be made to the relevant borehole and test pit logs.
3. Thin surficial soil layers (ie topsoil, slopewash etc) are not shown for clarity.
4. Strength and weathering nomenclature on boreholes drilled by others prior to this investigation has been retained.
5. The groundwater level/profile shown is based on groundwater levels measured in piezometers at the date shown and the groundwater level/profile could be expected to change with variations in climatic conditions etc.

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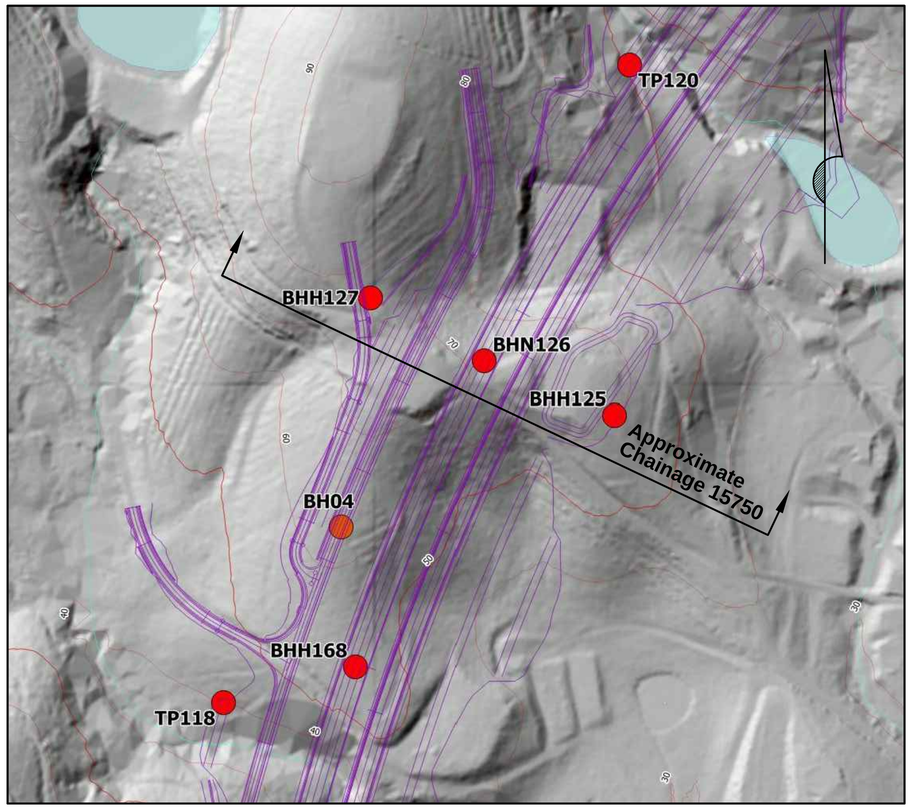
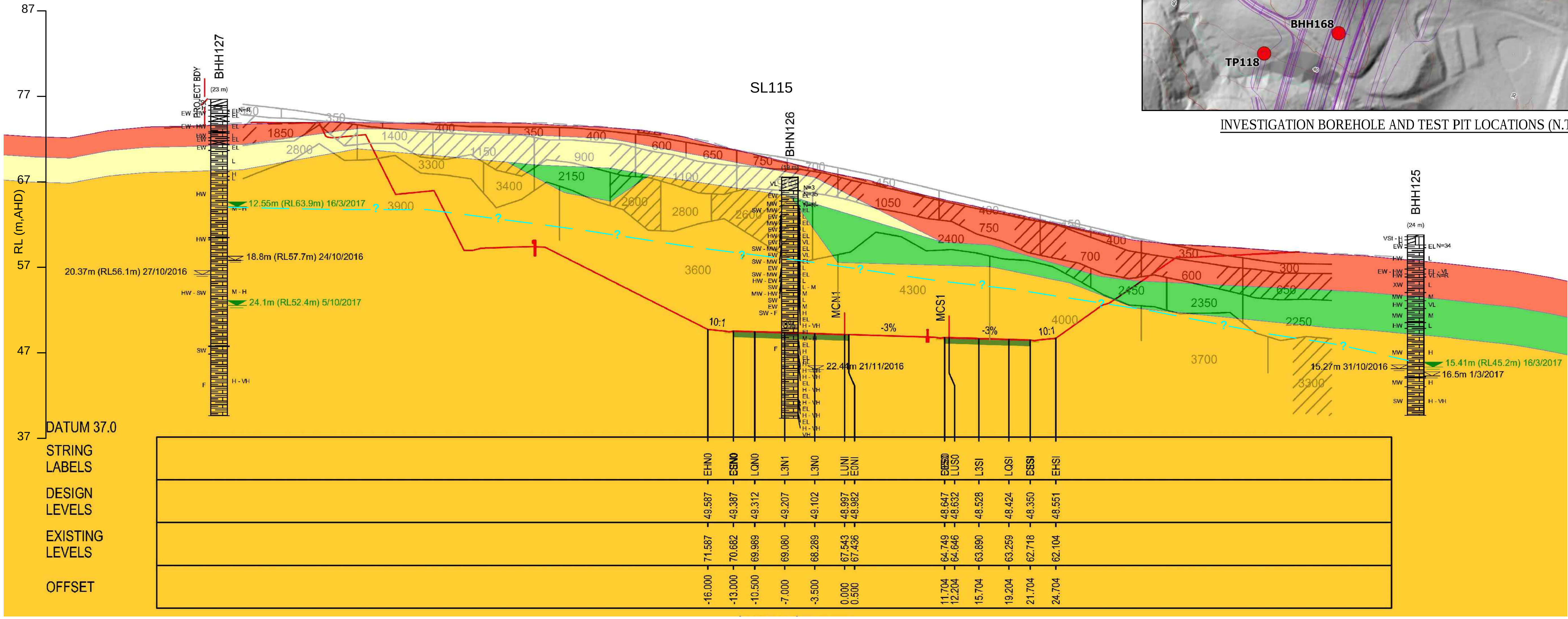
CUT 8a - CHAINAGE 14300
PACIFIC HIGHWAY UPGRADE
COFFS HARBOUR BYPASS

CLIENT	Roads and Maritime Services			RCA Ref	11717-818/2
DRAWN BY	TH	SCALE	1 : 500 (A3)	DRAWING No	13 REV 0
APPROVED BY	RJC	DATE	11/06/2019	OFFICE	NEWCASTLE

CDT-DWG-A3H-001/1



INVESTIGATION BOREHOLE AND TEST PIT LOCATIONS (N.T.S)



CHAINAGE 15750
CUT 11

NOTE:

1. Cross section taken from Arup, Feb 2019 Dwg. No. 248379-000-RD-MOD-1501_2019.dwg
2. The geological units and stratigraphy shown on the sections should be regarded as a generalised summary only. For further detail and for design purposes reference should be made to the relevant borehole and test pit logs.
3. Thin surficial soil layers (ie topsoil, slopewash etc) are not shown for clarity.
4. Strength and weathering nomenclature on boreholes drilled by others prior to this investigation has been retained.
5. The groundwater level/profile shown is based on groundwater levels measured in piezometers at the date shown and the groundwater level/profile could be expected to change with variations in climatic conditions etc.

CDT-DWG-A3H-001/1

RCA AUSTRALIA
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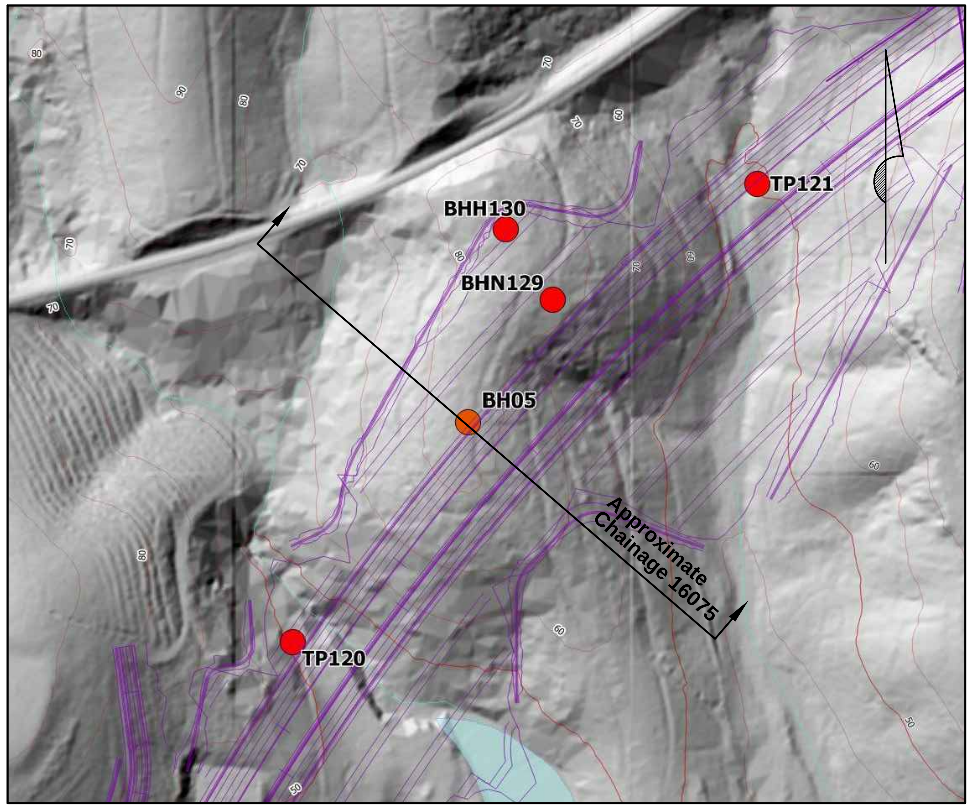
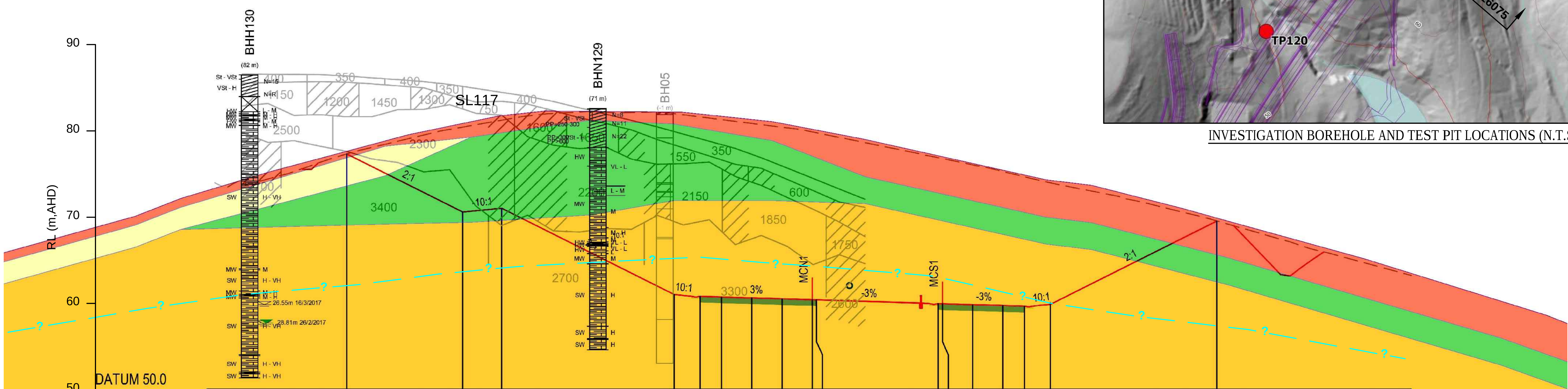
CUT 11 - CHAINAGE 15750
PACIFIC HIGHWAY UPGRADE
COFFS HARBOUR BYPASS

CLIENT	Roads and Maritime Services			RCA Ref	11717-818/2
DRAWN BY	TH	SCALE	1 : 500 (A3)	DRAWING No	14 REV 0
APPROVED BY	RJC	DATE	11/06/2019	OFFICE	NEWCASTLE



INVESTIGATION BOREHOLE AND TEST PIT LOCATIONS (N.T.S)



STRING LABELS	DESIGN LEVELS	EXISTING LEVELS	OFFSET
IAN0	77.240	77.240	-53.956
ICN0	70.512	80.350	-40.500
IBN0	70.962	81.146	-36.000
EHN0	60.962	81.913	-16.000
[illegible]	60.762	81.535	-13.000
LQN0	60.687	81.218	-10.500
L3N1	60.582	80.775	-7.000
L3N0	60.477	80.337	-3.500
LUNI	60.372	79.795	0.000
E0NI	60.357	79.693	0.500
[illegible]	59.922	76.810	14.597
LUS0	59.907	76.707	15.097
L3SI	59.804	75.983	18.597
LQSI	59.700	75.214	22.097
[illegible]	59.626	74.596	24.597
EHSI	59.827	73.897	27.598
IASI	69.466	69.495	46.865

CHAINAGE 16075
CUT 12

NOTE:

1. Cross section taken from Arup, Feb 2019
   Dwg. No. 248379-000-RD-MOD-1501_2019.dwg
2. The geological units and stratigraphy shown on the sections should be regarded as a generalised summary only. For further detail and for design purposes reference should be made to the relevant borehole and test pit logs.
3. Thin surficial soil layers (ie topsoil, slopewash etc) are not shown for clarity.
4. Strength and weathering nomenclature on boreholes drilled by others prior to this investigation has been retained.
5. The groundwater level/profile shown is based on groundwater levels measured in piezometers at the date shown and the groundwater level/profile could be expected to change with variations in climatic conditions etc.

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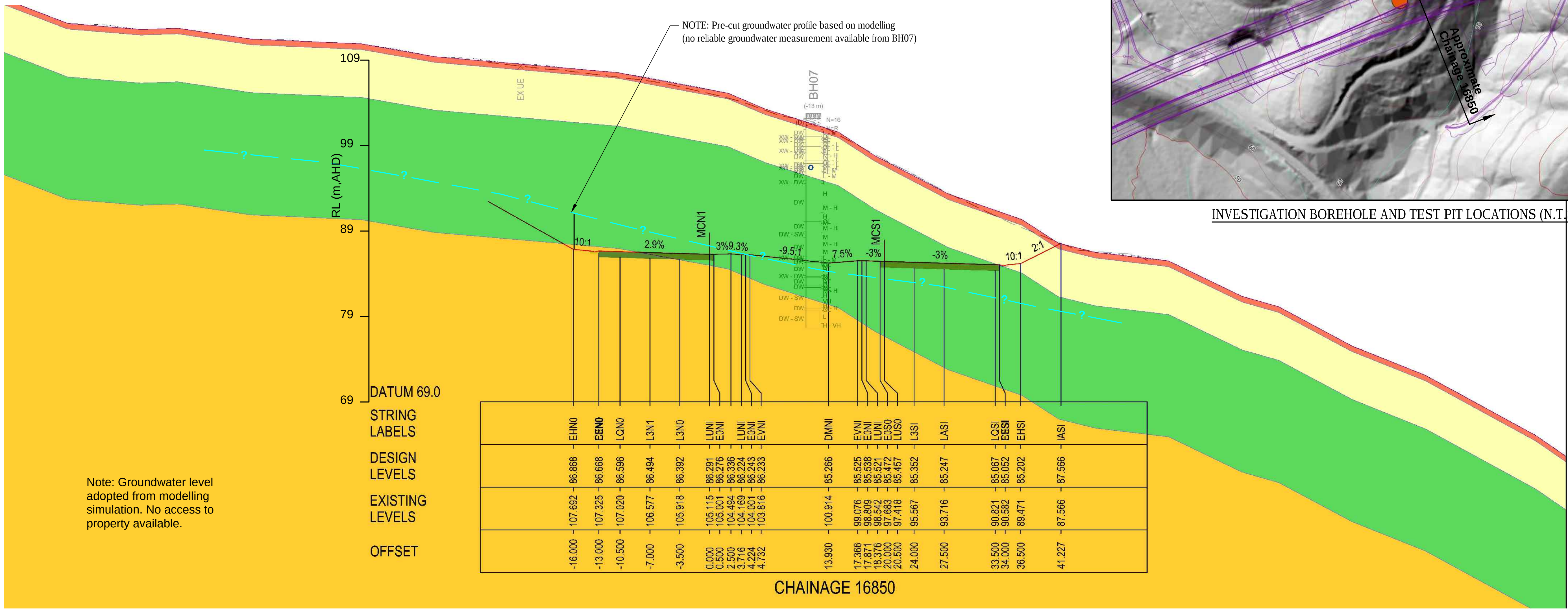
CUT 12 - CHAINAGE 16075
PACIFIC HIGHWAY UPGRADE
COFFS HARBOUR BYPASS

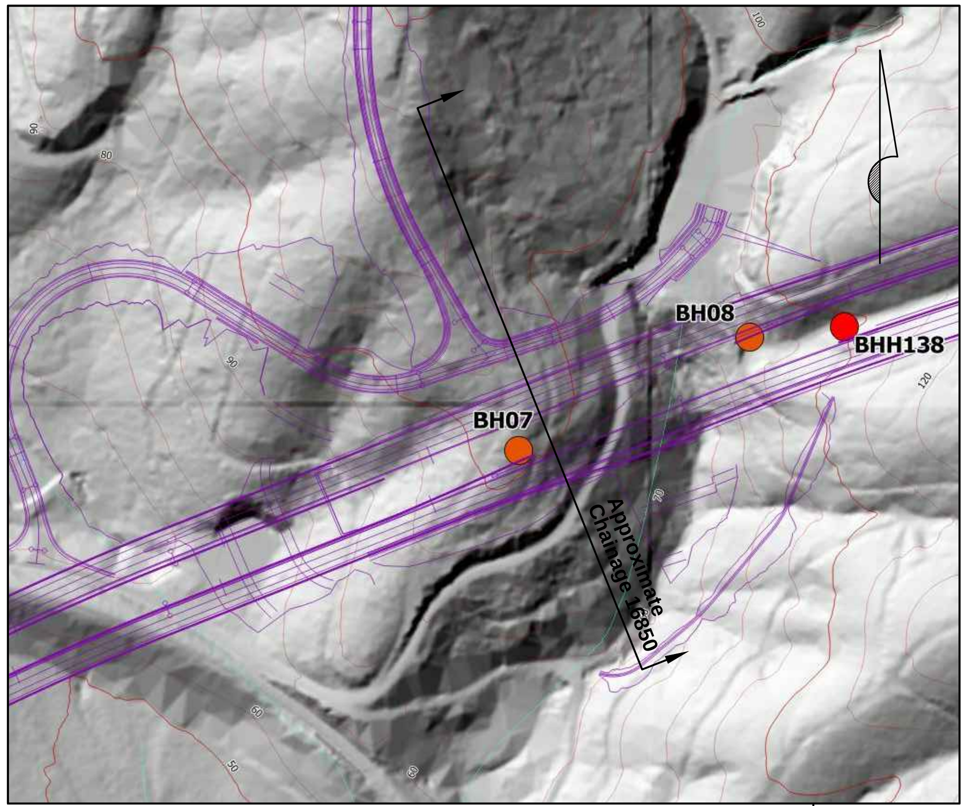
CLIENT	Roads and Maritime Services			RCA Ref	11717-818/2
DRAWN BY	TH	SCALE	1 : 500 (A3)	DRAWING No	15 REV 0
APPROVED BY	RJC	DATE	11/06/2019	OFFICE	NEWCASTLE

CDT-DWG-A3H-001/1



STRING LABELS	DESIGN LEVELS	EXISTING LEVELS	OFFSET
EHN0	86.868	107.692	-16.000
ESN0	86.668	107.325	-13.000
LQN0	86.596	107.020	-10.500
L3N1	86.494	106.577	-7.000
L3N0	86.392	105.918	-3.500
LUNI	86.291	105.115	0.000
E0NI	86.276	105.001	0.500
	86.336	104.494	2.500
LUNI	86.224	104.169	3.716
E0NI	86.243	104.001	4.224
EVNI	86.233	103.816	4.732
DMNI	85.266	100.914	13.930
EVNI	85.525	99.076	17.366
E0NI	85.538	98.809	17.871
LUNI	85.521	98.542	18.376
E0S0	85.472	97.683	20.000
LUS0	85.457	97.418	20.500
L3SI	85.352	95.567	24.000
LASI	85.247	93.716	27.500
LQSI	85.067	90.821	33.500
ESSI	85.052	90.582	34.000
EHSI	85.202	89.471	36.500
IASI	87.566	87.566	41.227

CHAINAGE 16850
CUT 14



INVESTIGATION BOREHOLE AND TEST PIT LOCATIONS (N.T.S)

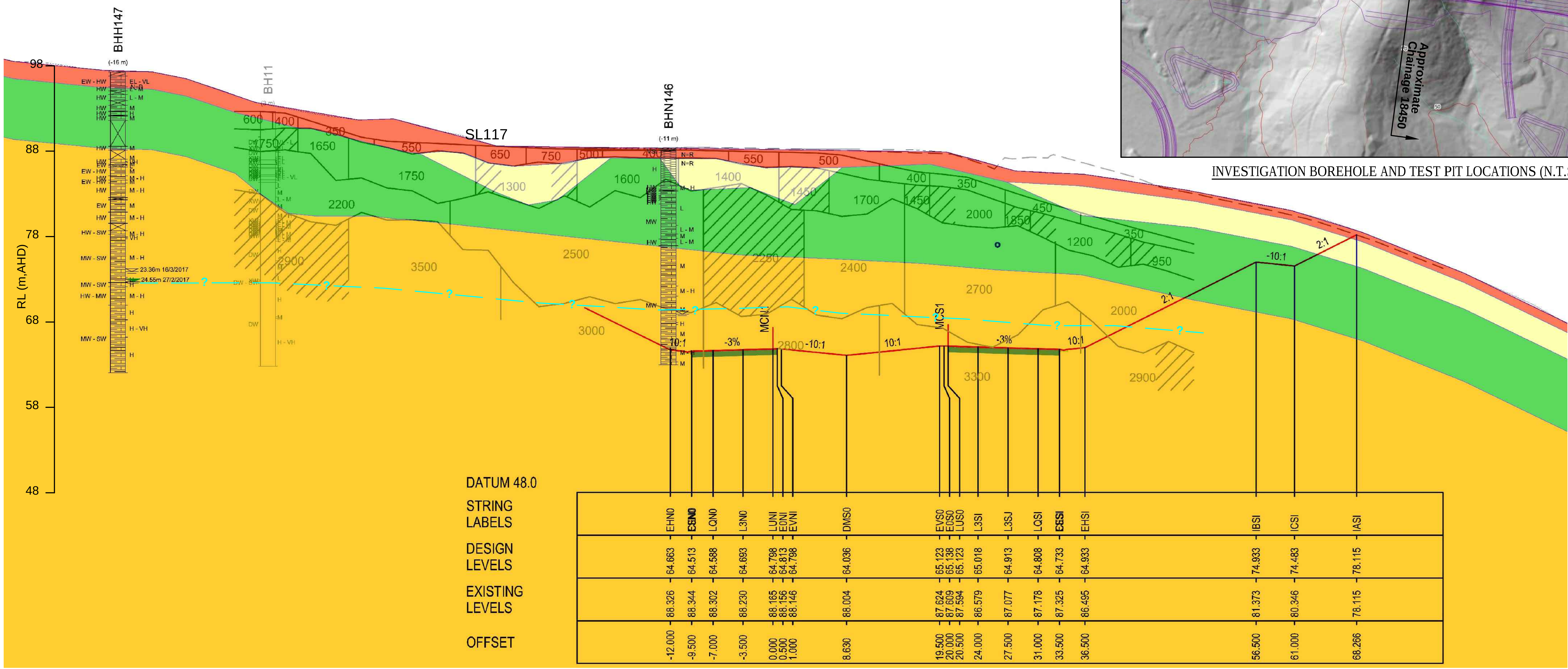
NOTE:
1. Cross section taken from Arup, Feb 2019
   Dwg. No. 248379-000-RD-MOD-1501_2019.dwg
2. The geological units and stratigraphy shown on the sections should be regarded as a generalised summary only. For further detail and for design purposes reference should be made to the relevant borehole and test pit logs.
3. Thin surficial soil layers (ie topsoil, slopewash etc) are not shown for clarity.
4. Strength and weathering nomenclature on boreholes drilled by others prior to this investigation has been retained.
5. The groundwater level/profile shown is based on groundwater levels measured in piezometers at the date shown and the groundwater level/profile could be expected to change with variations in climatic conditions etc.

CDT–DWG–A3H–001/1

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

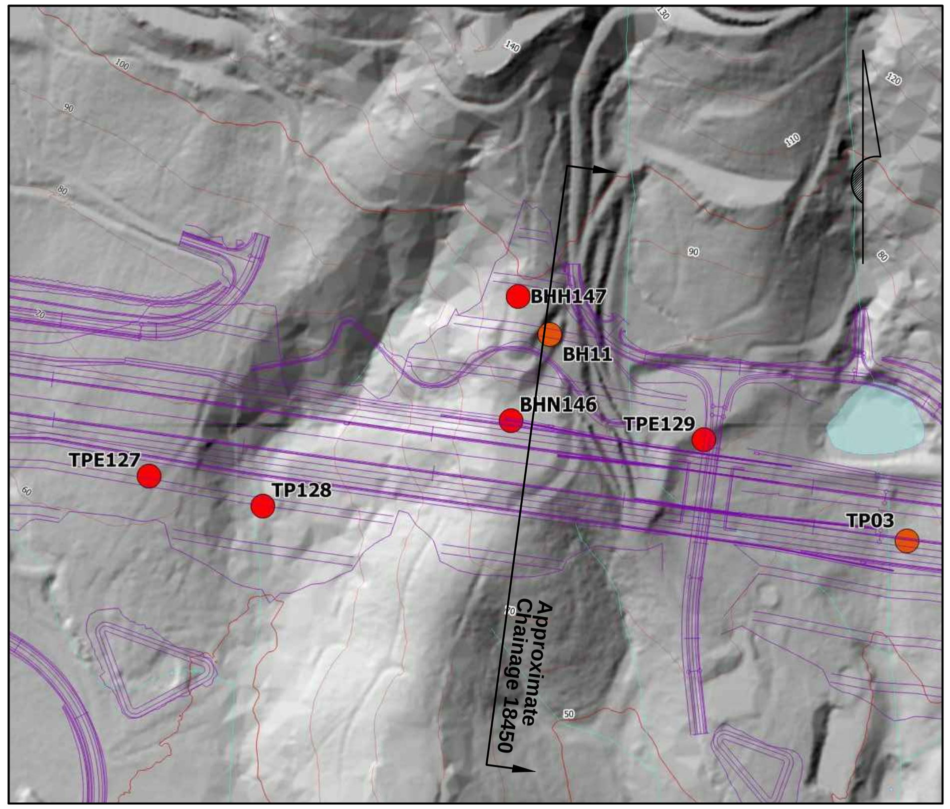
CUT 14 - CHAINAGE 16850
PACIFIC HIGHWAY UPGRADE
COFFS HARBOUR BYPASS

CLIENT	Roads and Maritime Services			RCA Ref	11717-818/2
DRAWN BY	TH	SCALE	1 : 500 (A3)	DRAWING No	16 REV 0
APPROVED BY	RJC	DATE	11/06/2019	OFFICE	NEWCASTLE



STRING LABELS	DESIGN LEVELS	EXISTING LEVELS	OFFSET
EHN0	64.663	88.326	-12.000
ESN0	64.513	88.344	-9.500
LQN0	64.588	88.302	-7.000
L3N0	64.693	88.230	-3.500
LUNI	64.798	88.165	0.000
EDNI	64.813	88.156	0.500
EVNI	64.798	88.146	1.000
DMS0	64.036	88.004	8.630
EVS0	65.123	87.624	19.500
EDS0	65.138	87.609	20.000
LUS0	65.123	87.594	20.500
L3SI	65.018	86.579	24.000
L3SJ	64.913	87.077	27.500
LQSI	64.808	87.178	31.000
ESSI	64.733	87.325	33.500
EHSI	64.933	86.495	36.500
IBSI	74.933	81.373	56.500
ICSI	74.483	80.346	61.000
IASI	78.115	78.115	68.266

CHAINAGE 18450
CUT 16



INVESTIGATION BOREHOLE AND TEST PIT LOCATIONS (N.T.S)

NOTE:

1. Cross section taken from Arup, Feb 2019
   Dwg. No. 248379-000-RD-MOD-1501_2019.dwg
2. The geological units and stratigraphy shown on the sections should be regarded as a generalised summary only. For further detail and for design purposes reference should be made to the relevant borehole and test pit logs.
3. Thin surficial soil layers (ie topsoil, slopewash etc) are not shown for clarity.
4. Strength and weathering nomenclature on boreholes drilled by others prior to this investigation has been retained.
5. The groundwater level/profile shown is based on groundwater levels measured in piezometers at the date shown and the groundwater level/profile could be expected to change with variations in climatic conditions etc.

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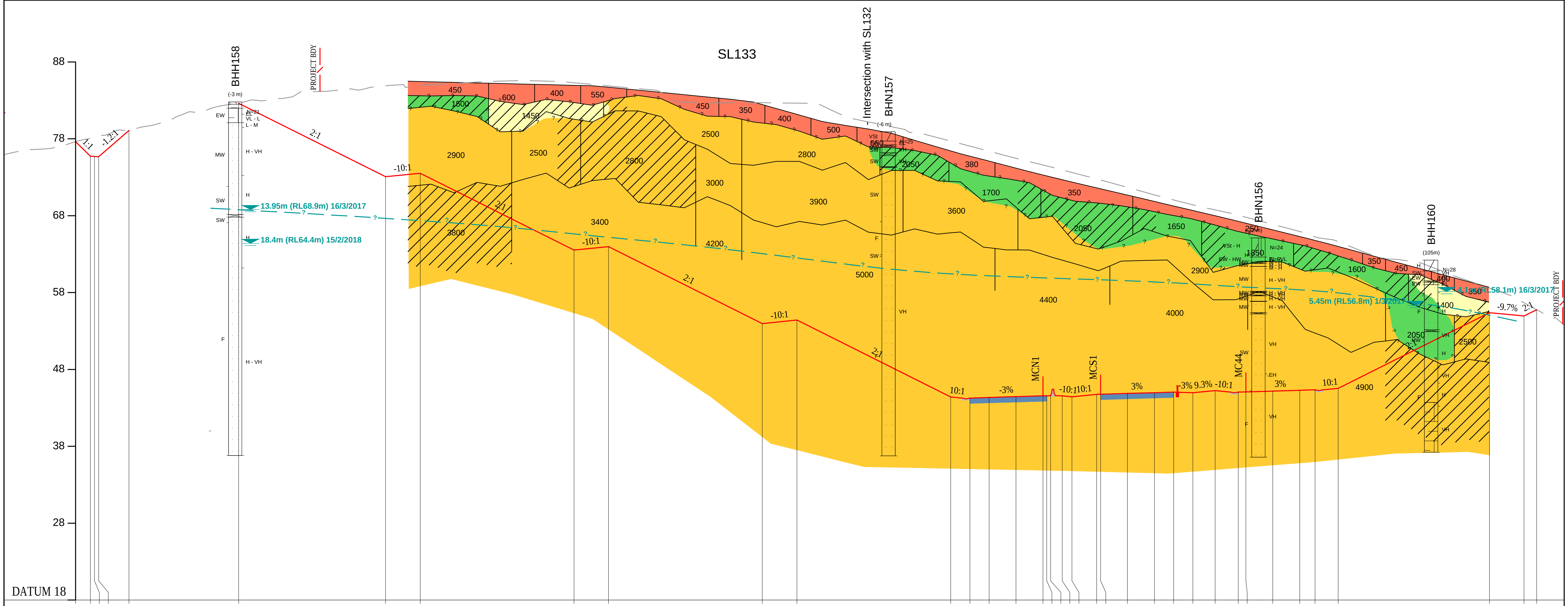
CUT 16 - CHAINAGE 18450
PACIFIC HIGHWAY UPGRADE
COFFS HARBOUR BYPASS

CLIENT	Roads and Maritime Services			RCA Ref	11717-818/2
DRAWN BY	TH	SCALE	1 : 500 (A3)	DRAWING No	17 REV 0
APPROVED BY	RJC	DATE	11/06/2019	OFFICE	NEWCASTLE

CDT-DWG-A3H-001/1



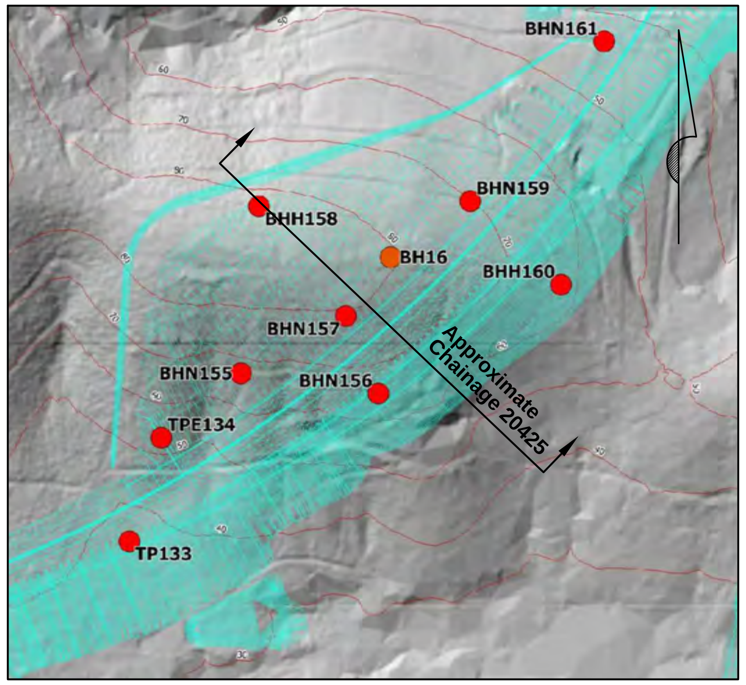
DESIGN LEVELS	77.619	75.737	75.705	75.674	79.076	82.628	73.072	73.522	63.522	63.972	53.972	54.422
EXISTING LEVELS	77.632	78.051	78.155	78.324	79.076	82.628	84.905	84.945	85.292	84.875	82.697	82.638
OFFSET	-125.826	-123.906	-123.364	-122.822	-118.885	-104.612	-85.500	-81.000	-61.000	-56.500	-36.500	-32.000

DESIGN LEVELS	44.422	44.272	44.347	44.452	44.557	44.572	44.587	44.560	44.432	44.754	44.769	44.874
EXISTING LEVELS	77.704	77.044	76.385	75.461	74.597	74.474	74.352	73.963	73.629	72.787	72.656	71.681
OFFSET	-12.000	-9.500	-7.000	-3.500	0.000	0.500	1.000	2.500	3.780	7.000	7.500	11.000

DESIGN LEVELS	44.979	45.054	44.979	45.247	45.047	45.077	45.182	45.286	45.346	45.546	55.385	54.950	55.771
EXISTING LEVELS	70.705	70.045	69.417	68.754	68.059	67.795	66.871	65.948	65.250	64.629	58.578	56.666	55.771
OFFSET	14.500	17.000	19.500	22.401	25.402	26.402	29.902	33.403	35.403	38.404	58.087	62.581	64.223

CHAINAGE 20425
CUT 18

INVESTIGATION BOREHOLE AND TEST PIT LOCATIONS (N.T.S)



NOTE:

1. Cross section taken from Arup, July 2018 Dwg. No. 248379-000-RD-MOD-1599.dwg
2. The geological units and stratigraphy shown on the sections should be regarded as a generalised summary only. For further detail and for design purposes reference should be made to the relevant borehole and test pit logs.
3. Thin surficial soil layers (ie topsoil, slopewash etc) are not shown for clarity.
4. Strength and weathering nomenclature on boreholes drilled by others prior to this investigation has been retained.
5. The groundwater level/profile shown is based on groundwater levels measured in piezometers at the date shown and the groundwater level/profile could be expected to change with variations in climatic conditions etc.

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

CUT 18 - CHAINAGE 20425
PACIFIC HIGHWAY UPGRADE
COFFS HARBOUR BYPASS

CLIENT	Roads and Maritime Services			RCA Ref	11717-818/2
DRAWN BY	RJC	SCALE	1 : 500 (A3)	DRAWING No	18 REV 0
APPROVED BY	RJC	DATE	11/06/2019	OFFICE	NEWCASTLE

CDT-DWG-A3H-001/1

Appendix B

Calibration Parameters

Cut	Hydraulic Conductivity K of Soil, Rock Unit A and Rock Unit B (m/s)	Hydraulic Conductivity K of Rock Unit C (m/s)	Infiltration rate I (m/s)	Infiltration rate I (mm/year)	Constant Head LHS (m AHD)	Constant Head RHS (m AHD)
4	**5% -** $1x10^{-6}$ **15% -** $1x10^{-6}$	**5% -** $5.3x10^{-8}$, $1.7x10^{-8}$, $6.0x10^{-9}$ **15% -** $1.75x10^{-7}$ $4.5x10^{-8}$ $2.0x10^{-8}$	**5%** - $2.7x10^{-9}$ at surface $1.35x10^{-9}$ on cut batter slopes **15%** - $8.1x10^{-9}$ at surface $4.05x10^{-9}$ on cut batter slopes	**5%** - 85 at surface 42 on cut batter slopes **15%** - 255 at surface 122 on cut batter slopes	51.94	18.58
8	**5% -** $2x10^{-6}$ **15% -** $2x10^{-6}$	**5% -** $8.0x10^{-8}$ **15% -** $2.5x10^{-7}$	**5%** - $2.7x10^{-9}$ at surface $1.35x10^{-9}$ on cut batter slopes **15%** - $8.1x10^{-9}$ at surface $4.05x10^{-9}$ on cut batter slopes	**5%** - 85 at surface 42 on cut batter slopes **15%** - 255 at surface 122 on cut batter slopes	48.1	15.51
8a	**5% -** $1x10^{-6}$ **15% -** $1x10^{-6}$	**5% -** $2.45x10^{-8}$ **15% -** $2.0x10^{-7}$	**5%** - $2.7x10^{-9}$ at surface $1.35x10^{-9}$ on cut batter slopes **15%** - $8.1x10^{-9}$ at surface $4.05x10^{-9}$ on cut batter slopes	**5%** - 85 at surface 42 on cut batter slopes **15%** - 255 at surface 122 on cut batter slopes	46.76	15.88
11	**5% -** $2x10^{-6}$ **15% -** $2x10^{-6}$	**5% -** $9x10^{-9}$ **15% -** $6.2x10^{-8}$	**5%** - $2.7x10^{-9}$ at surface $1.35x10^{-9}$ on cut batter slopes **15%** - $8.1x10^{-9}$ at surface $4.05x10^{-9}$ on cut batter slopes	**5%** - 85 at surface 42 on cut batter slopes **15%** - 255 at surface 122 on cut batter slopes	48.19	30.68

Cut	Hydraulic Conductivity K of Soil, Rock Unit A and Rock Unit B (m/s)	Hydraulic Conductivity K of Rock Unit C (m/s)	Infiltration rate I (m/s)	Infiltration rate I (mm/year)	Constant Head LHS (m AHD)	Constant Head RHS (m AHD)
12	**5% -** $1x10^{-6}$ **15% -** $1x10^{-6}$	**5% -** $2.1x10^{-8}$ **15% -** $1.0x10^{-7}$	**<u>5%</u>** - $2.7x10^{-9}$ at surface $1.35x10^{-9}$ on cut batter slopes **<u>15%</u>** - $8.1x10^{-9}$ at surface $4.05x10^{-9}$ on cut batter slopes	**<u>5%</u>** - 85 at surface 42 on cut batter slopes **<u>15%</u>** - 255 at surface 122 on cut batter slopes	78.25	36.78
14	**5% -** $1x10^{-6}$ **15% -** $2x10^{-6}$	**5% -** $1.2x10^{-7}$, 1.5e-8 **15% -** $3.6x10^{-7}$, 1.5e-8	**<u>5%</u>** - $2.7x10^{-9}$ at surface $1.35x10^{-9}$ on cut batter slopes **<u>15%</u>** - $8.1x10^{-9}$ at surface $4.05x10^{-9}$ on cut batter slopes	**<u>5%</u>** - 85 at surface 42 on cut batter slopes **<u>15%</u>** - 255 at surface 122 on cut batter slopes	289.67	23.26
16	**5% -** $1x10^{-6}$ **15% -** $2x10^{-6}$	**5% -** $4.0x10^{-7}$, $6.0x10^{-9}$, **15% -** $1.3x10^{-6}$ $1.0x10^{-8}$	**<u>5%</u>** - $2.7x10^{-9}$ at surface $1.35x10^{-9}$ on cut batter slopes **<u>15%</u>** - $8.1x10^{-9}$ at surface $4.05x10^{-9}$ on cut batter slopes	**<u>5%</u>** - 85 at surface 42 on cut batter slopes **<u>15%</u>** - 255 at surface 122 on cut batter slopes	207.90	44.71
18	**5% -** $1x10^{-6}$ **15% -** $1x10^{-6}$	**5% -** $2.0x10^{-8}$ **15% -** $8.0x10^{-8}$	**<u>5%</u>** - $2.7x10^{-9}$ at surface $1.35x10^{-9}$ on cut batter slopes **<u>15%</u>** - $8.1x10^{-9}$ at surface $4.05x10^{-9}$ on cut batter slopes	**<u>5%</u>** - 85 at surface 42 on cut batter slopes **<u>15%</u>** - 255 at surface 122 on cut batter slopes	26	58

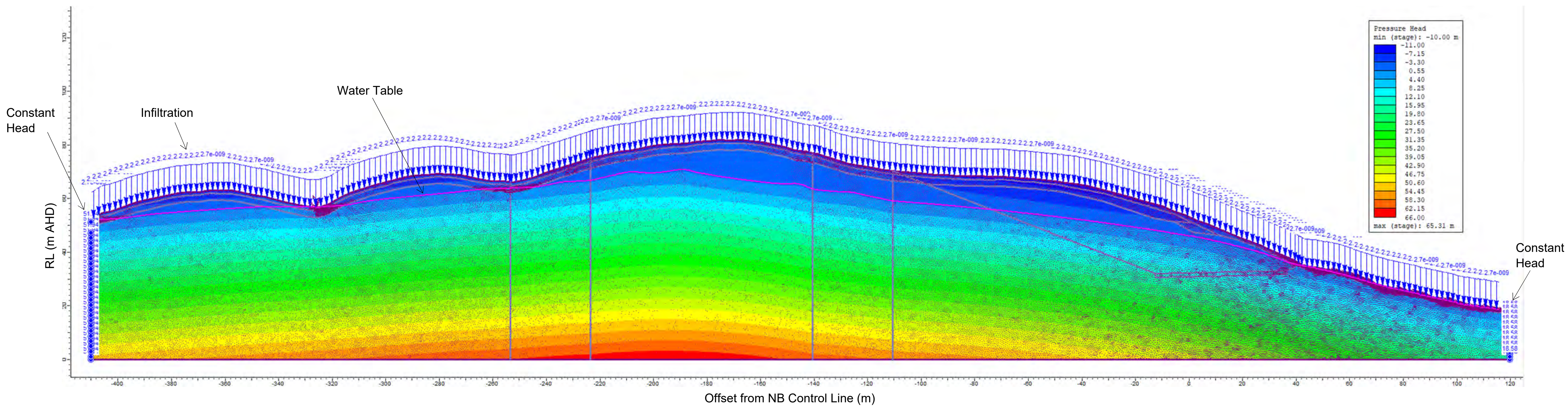
Appendix C

Calibration Targets

Cut	Borehole	Infiltration Rate	Observed Head (mAHD)	Simulated Head (mAHD)	Difference (m)	% Difference
4	BHH109	5%	57.00	57.19	0.19	0.33%
		15%	57.00	56.67	-0.33	-0.58%
8	BHH115	5%	59.10	61.52	2.42	4.09%
		15%	59.10	61.46	2.36	3.99%
8a	BHH119	5%	46.34	46.29	-0.05	-0.11%
		15%	46.34	46.57	0.23	0.50%
11	BHH127	5%	63.90	64.72	0.82	1.28%
		15%	63.90	64.33	0.43	0.67%
12	BH05	5%	63.72	64.54	0.64	1.00%
		15%	63.72	64.35	0.45	0.70%
14	No groundwater measurements available					
16	BHH147	5%	73.86	73.38	-0.48	-0.65%
		15%	73.86	72.50	-1.36	-1.84%
18	BHH158	5%	68.90	69.37	0.47	0.68%
		15%	68.90	69.42	0.52	0.75%

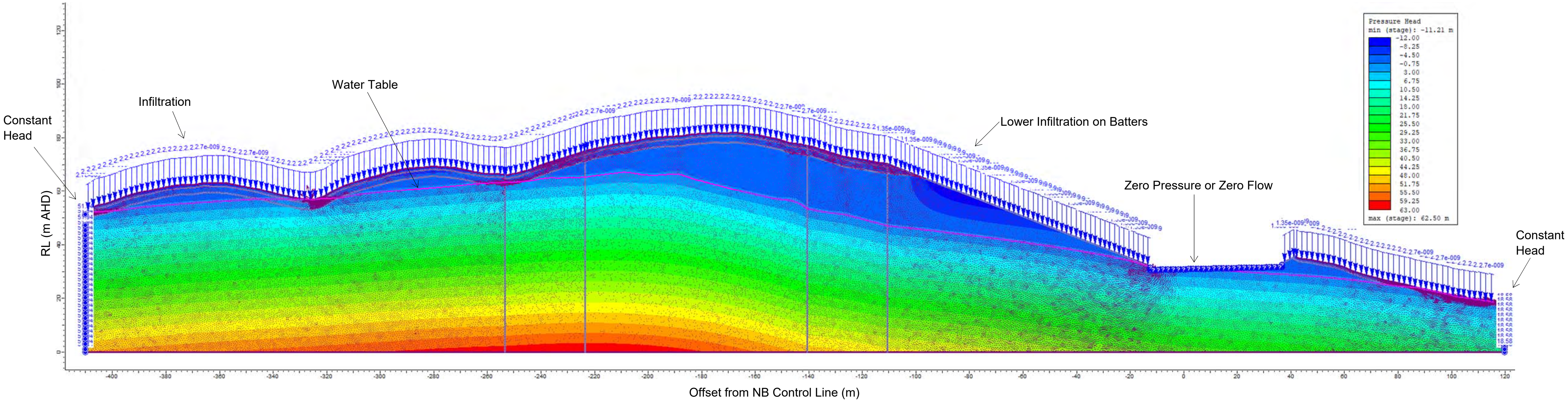
Appendix D

Cross-sectional Drawings of Existing Model and Simulated Cut Model

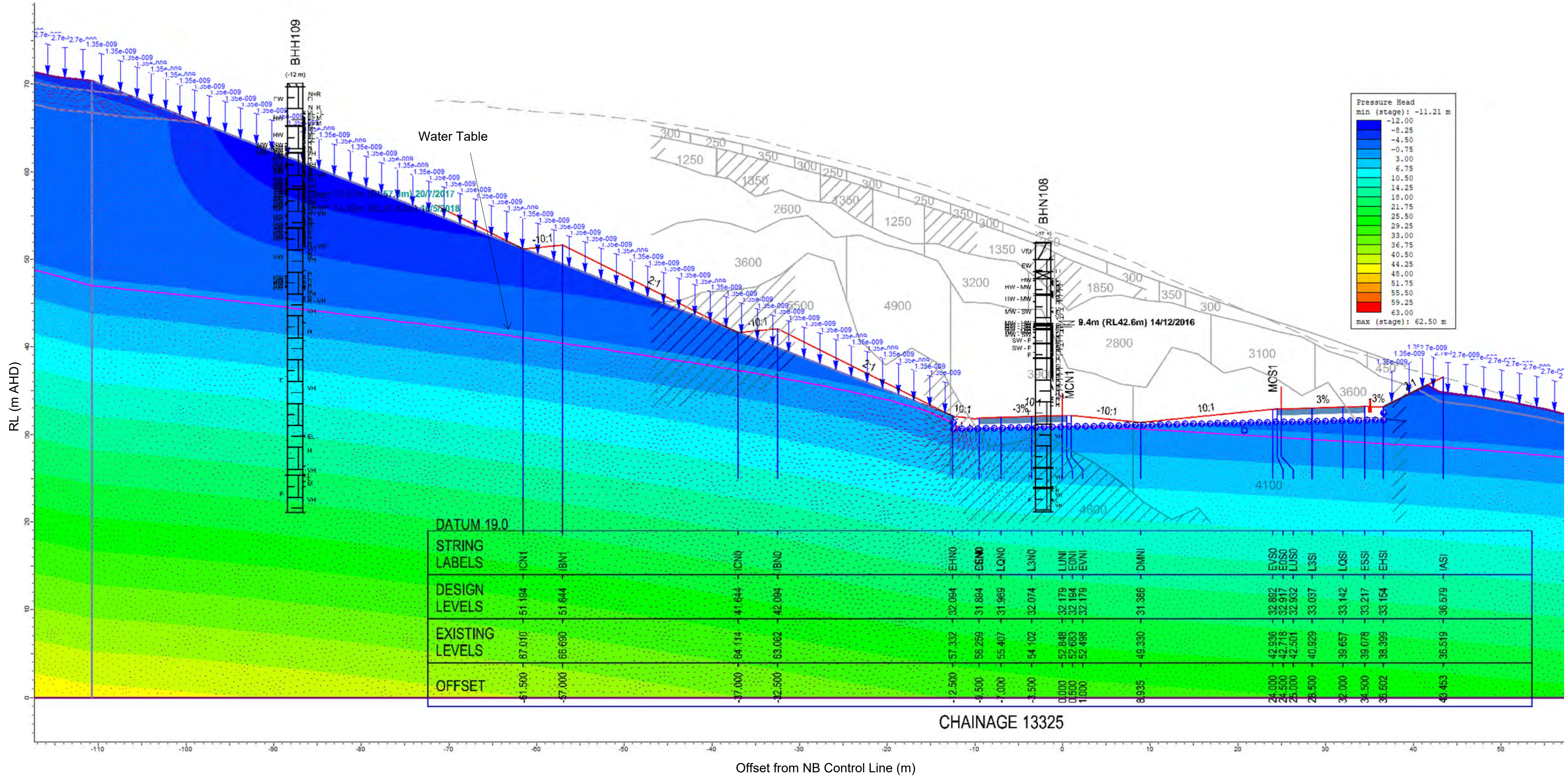


Cut 4 (CH13325) – Existing Model

Client:	Roads and Maritime Services	**RCA Australia**
Project:	Coffs Harbour Bypass	
Location:	Coffs Harbour	**RCA ref:** 11717-818

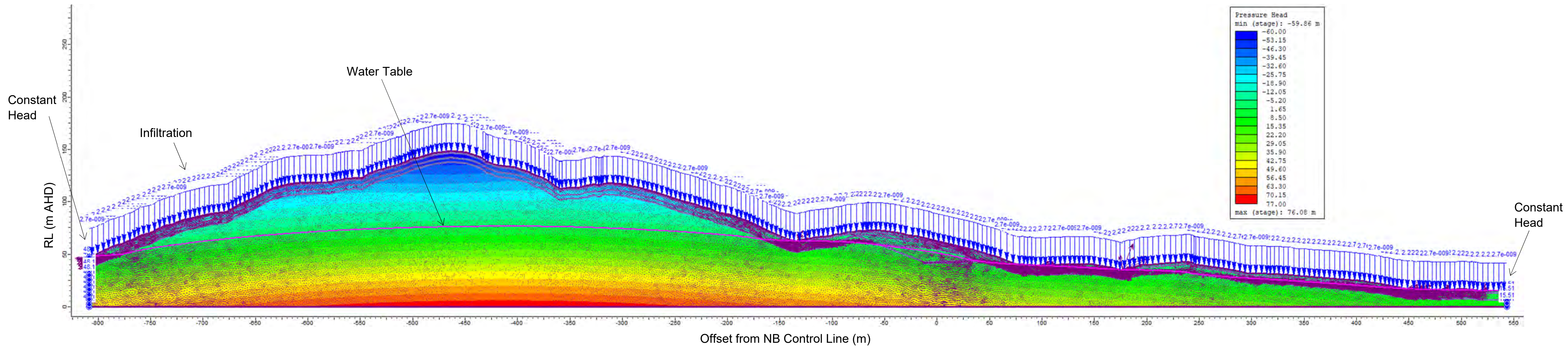


Cut 4 (CH13325) – Simulated Cut Model



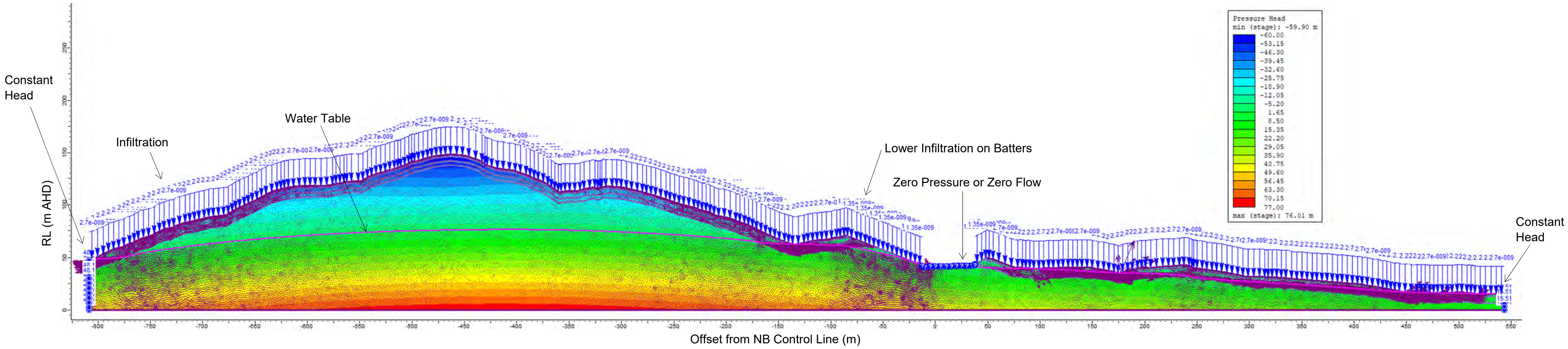
Cut 4 (CH13325) – Simulated Cut Model Showing Cutting Detail

Client:	Roads and Maritime Services	**RCA Australia**	
Project:	Coffs Harbour Bypass		
Location:	Coffs Harbour	**RCA ref:** 11717-818	



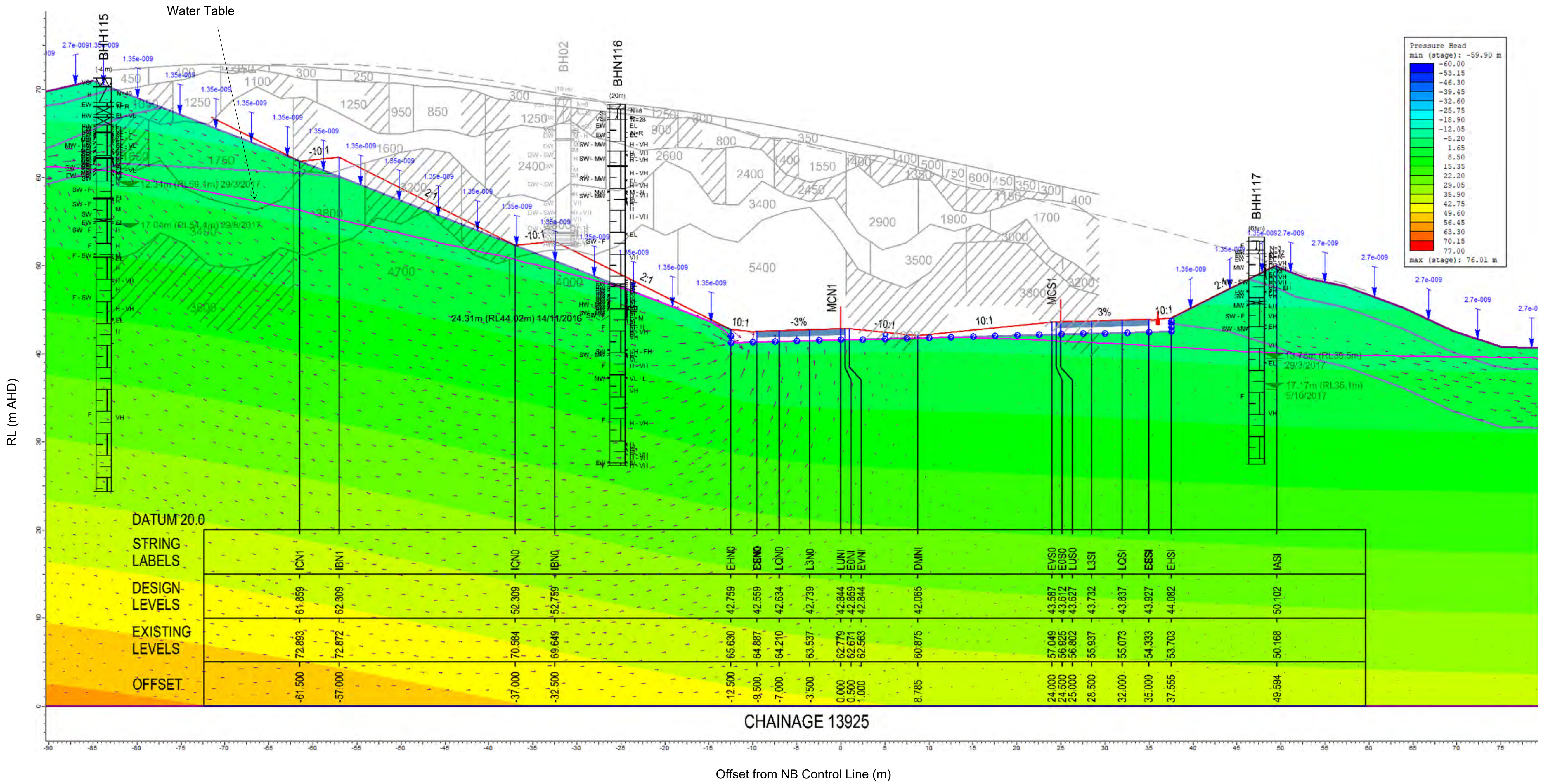
Cut 8 (CH13925) – Existing Model

Client:	Roads and Maritime Services	**RCA Australia**
Project:	Coffs Harbour Bypass	
Location:	Coffs Harbour	**RCA ref:** 11717-818



Cut 8 (CH13925) – Simulated Cut Model

Client:	Roads and Maritime Services	**RCA Australia**	
Project:	Coffs Harbour Bypass		
Location:	Coffs Harbour	**RCA ref:** 11717-818	



Cut 8 (CH13925) – Simulated Cut Model Showing Cutting Detail

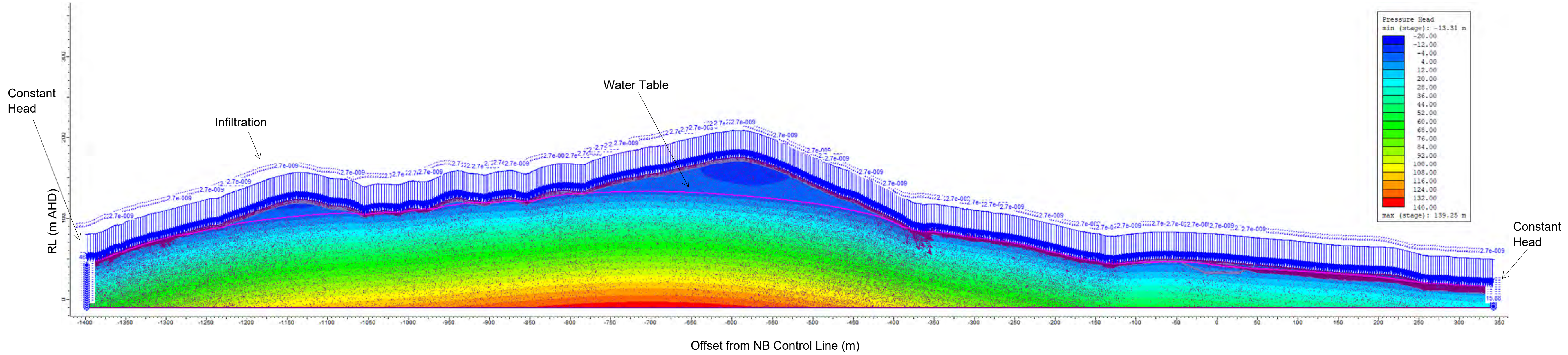
Client: Roads and Maritime Services

RCA Australia

Project: Coffs Harbour Bypass

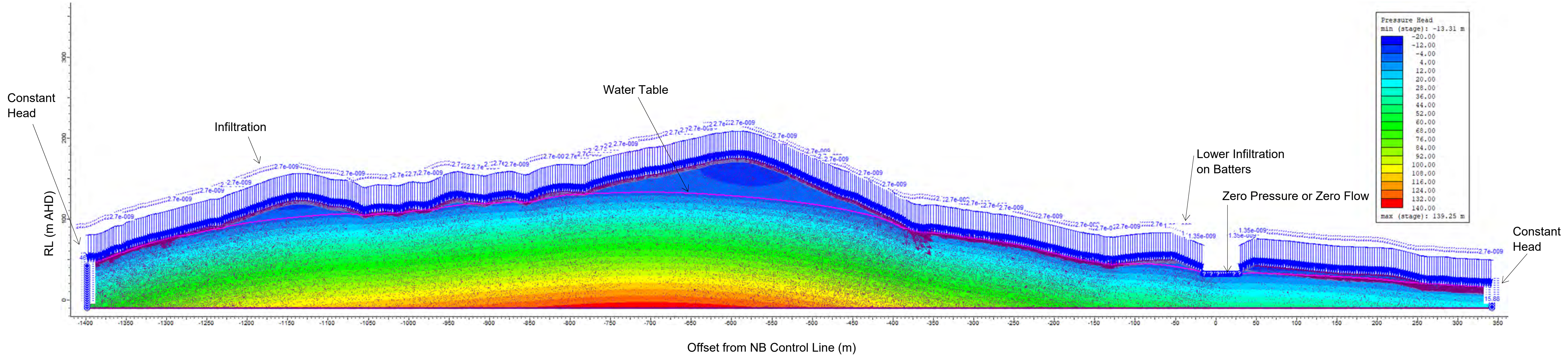
Location: Coffs Harbour

RCA ref: 11717-818



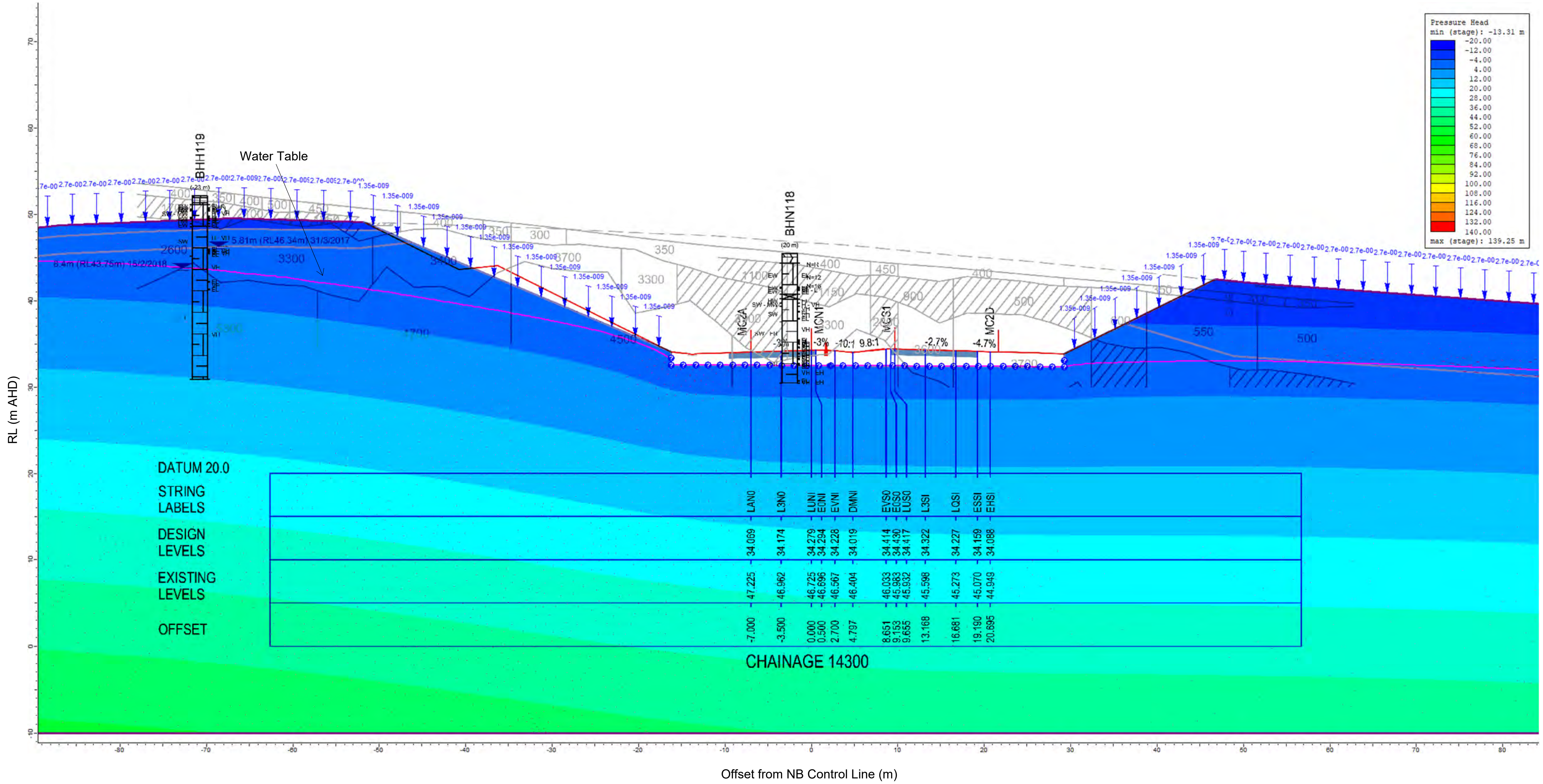
Cut 8A (CH14300) – Existing Model

Client:	Roads and Maritime Services	**RCA Australia**
Project:	Coffs Harbour Bypass	
Location:	Coffs Harbour	**RCA ref:** 11717-818



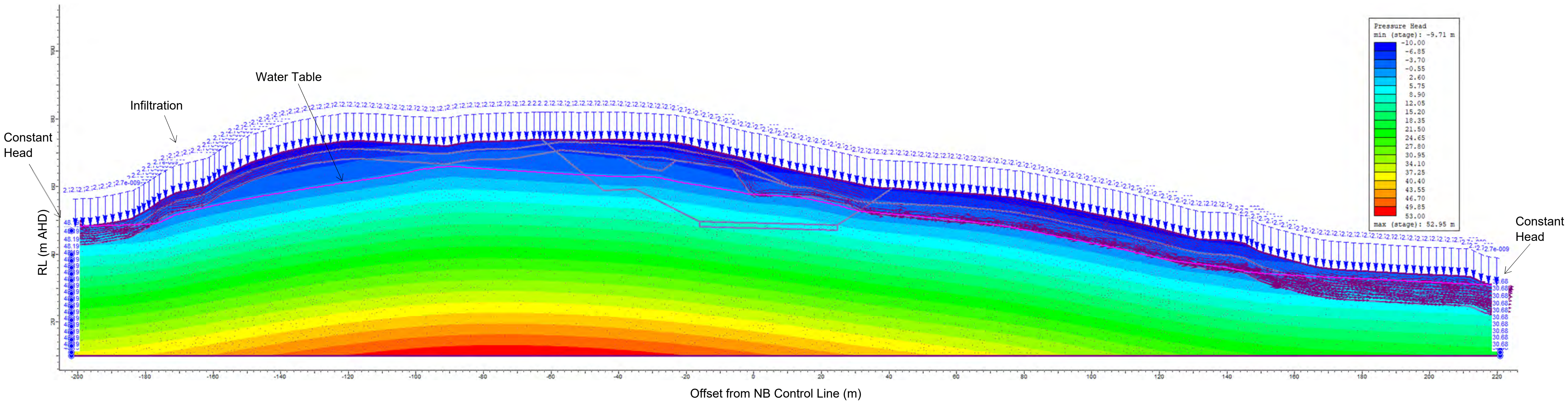
Cut 8A (CH14300) – Simulated Cut Model

Client:	Roads and Maritime Services	**RCA Australia**
Project:	Coffs Harbour Bypass	
Location:	Coffs Harbour	**RCA ref:** 11717-818



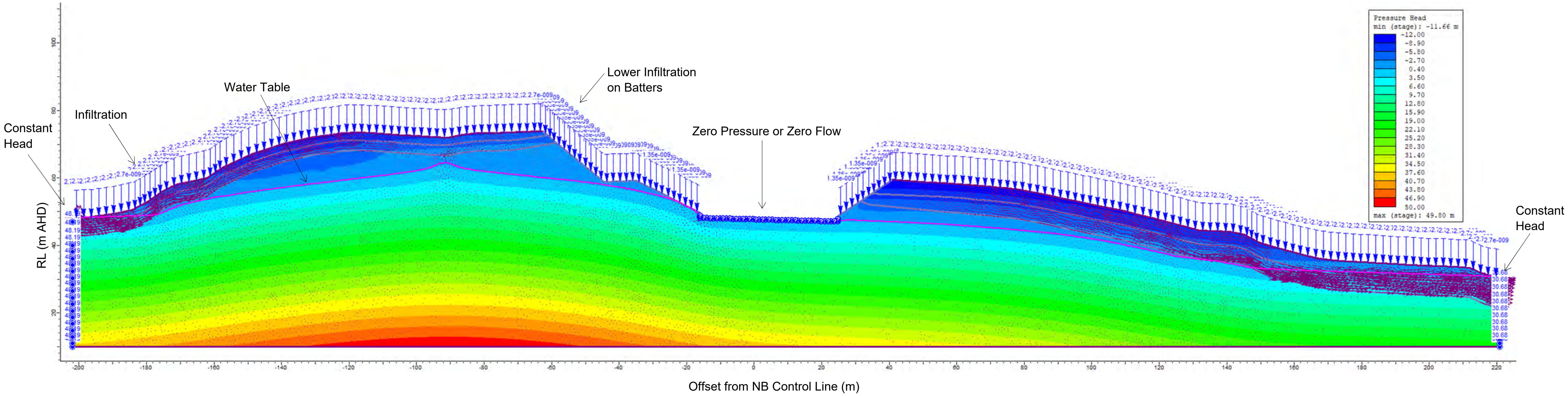
Cut 8A (CH14300) – Simulated Cut Model Showing Cutting Detail

Client:	Roads and Maritime Services	**RCA Australia**
Project:	Coffs Harbour Bypass	
Location:	Coffs Harbour	**RCA ref:** 11717-818



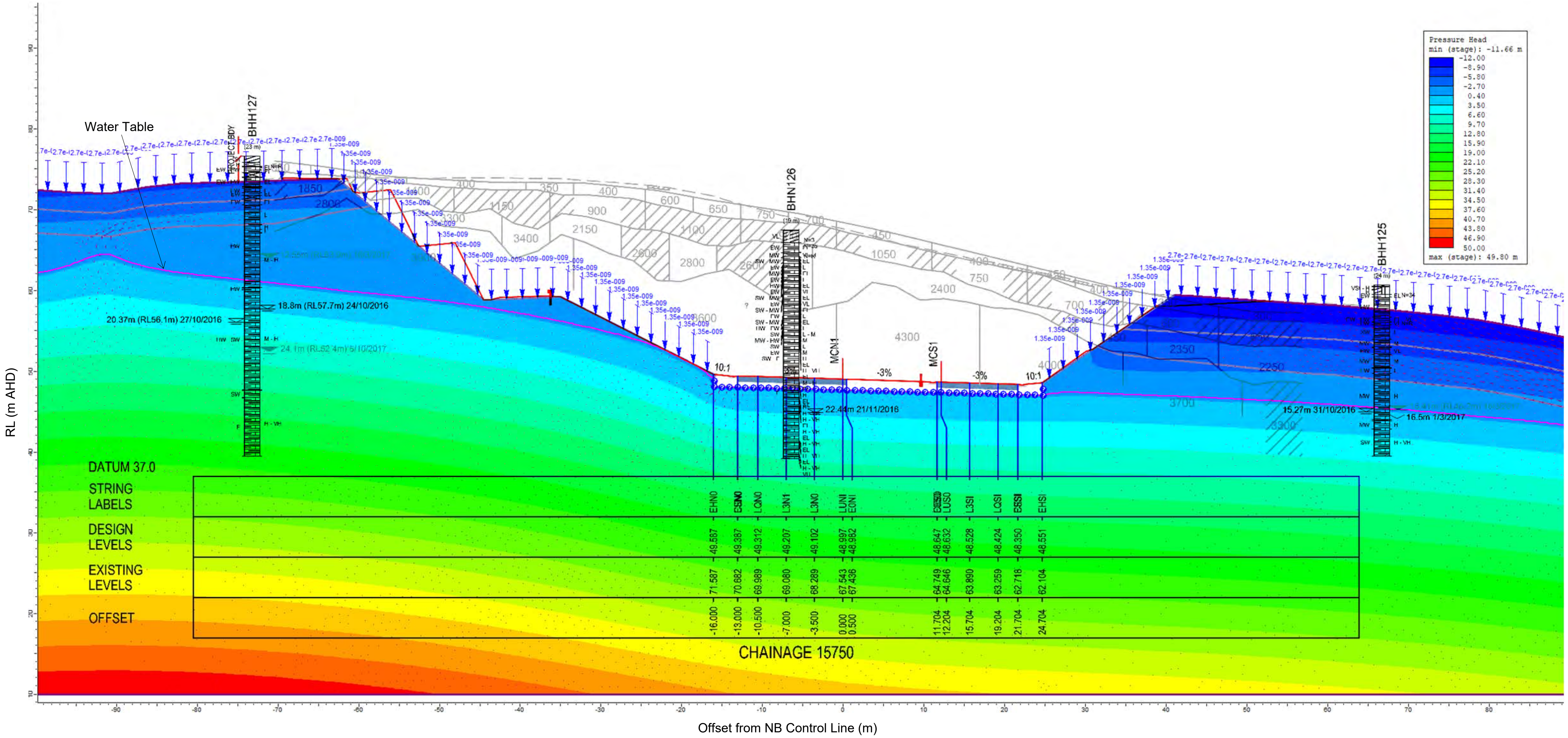
Cut 11 (CH15750) – Existing Model

Client:	Roads and Maritime Services	**RCA Australia**
Project:	Coffs Harbour Bypass	
Location:	Coffs Harbour	**RCA ref:** 11717-818



Cut 11 (CH15750) – Simulated Cut Model

Client:	Roads and Maritime Services	**RCA Australia**
Project:	Coffs Harbour Bypass	
Location:	Coffs Harbour	**RCA ref:** 11717-818

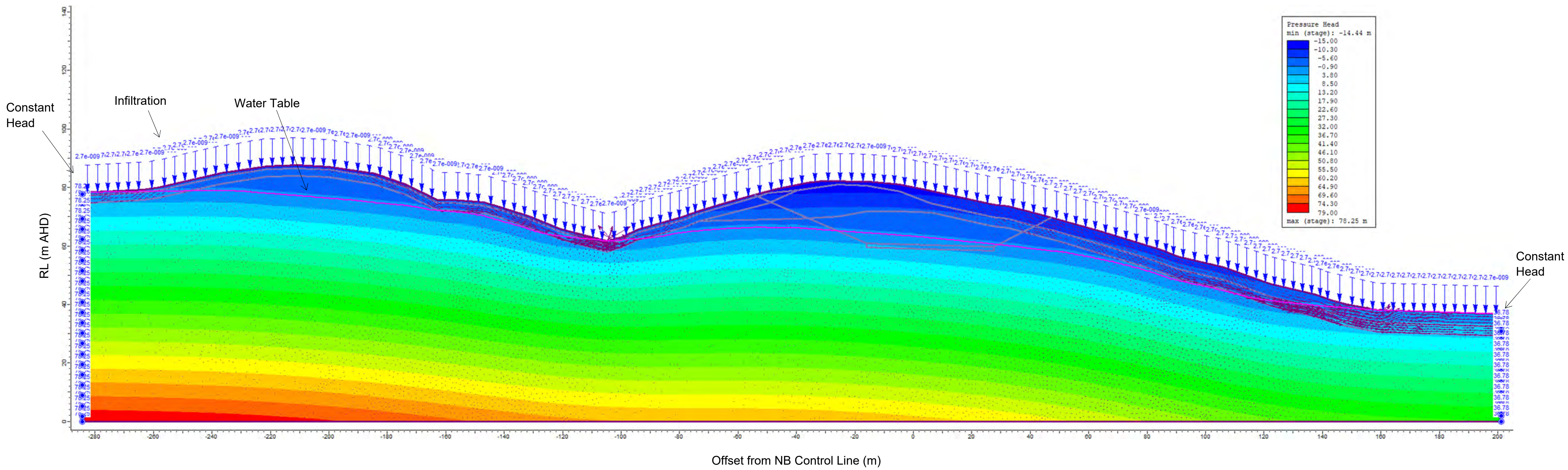


Cut 11 (CH15750) – Simulated Cut Model Showing Cutting Detail

Client: Roads and Maritime Services **RCA Australia**

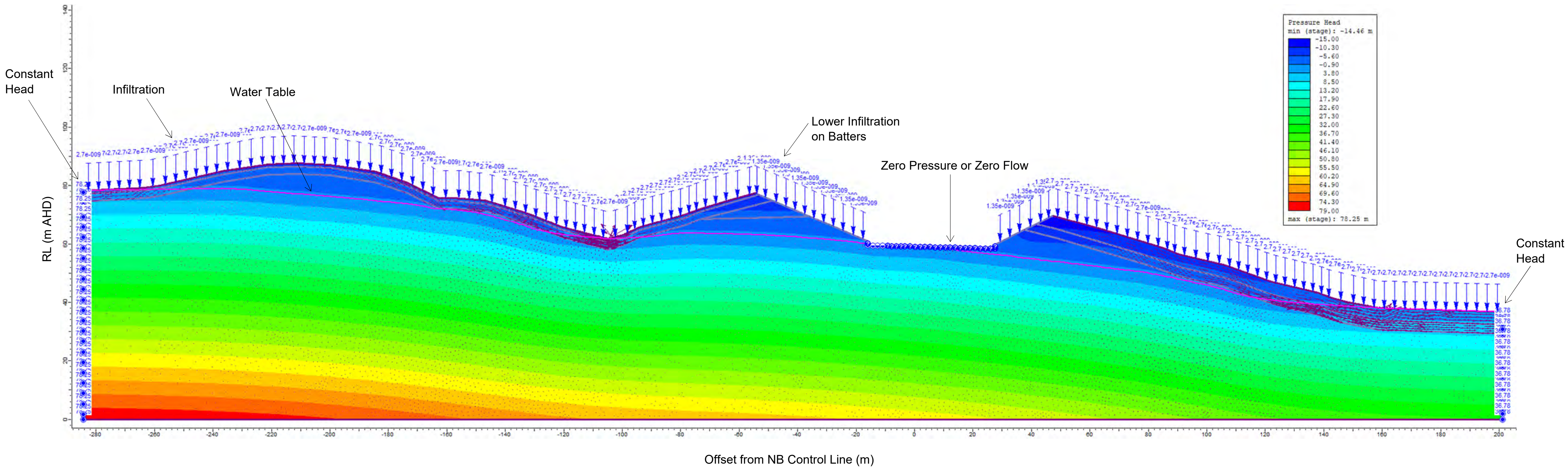
Project: Coffs Harbour Bypass

Location: Coffs Harbour **RCA ref:** 11717-818



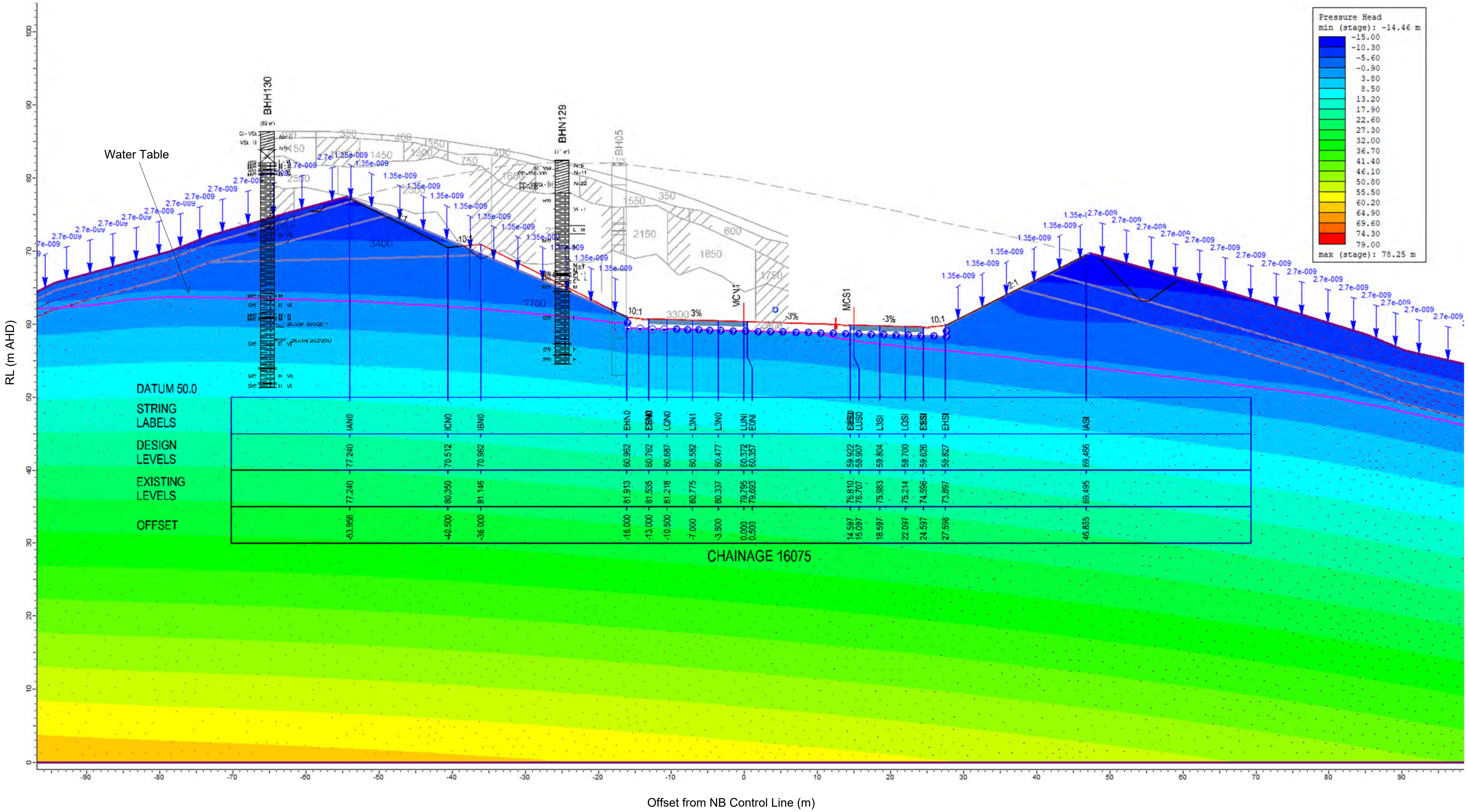
Cut 12 (CH16075) – Existing Model

Client:	Roads and Maritime Services	**RCA Australia**
Project:	Coffs Harbour Bypass	
Location:	Coffs Harbour	**RCA ref:** 11717-818



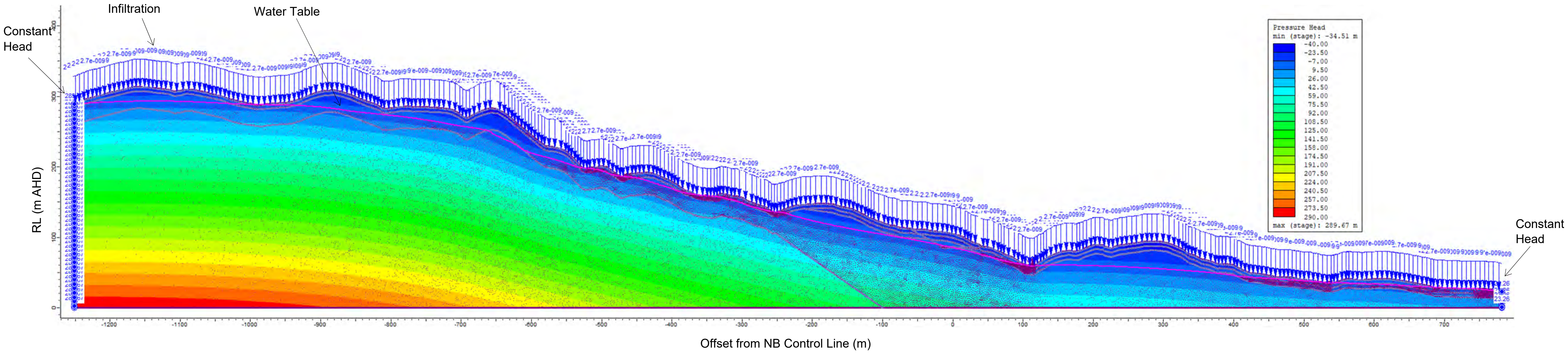
Cut 12 (CH16075) – Simulated Cut Model

Client:	Roads and Maritime Services	**RCA Australia**
Project:	Coffs Harbour Bypass	
Location:	Coffs Harbour	**RCA ref:** 11717-818



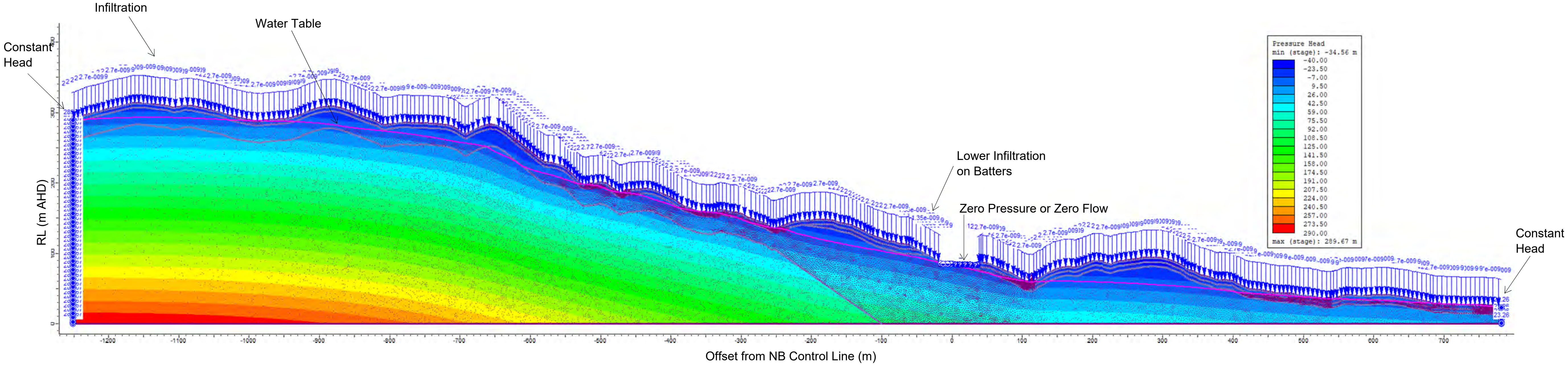
Cut 12 (CH16075) – Simulated Cut Model Showing Cutting Detail

Client:	Roads and Maritime Services	**RCA Australia**
Project:	Coffs Harbour Bypass	
Location:	Coffs Harbour	**RCA ref:** 11717-818



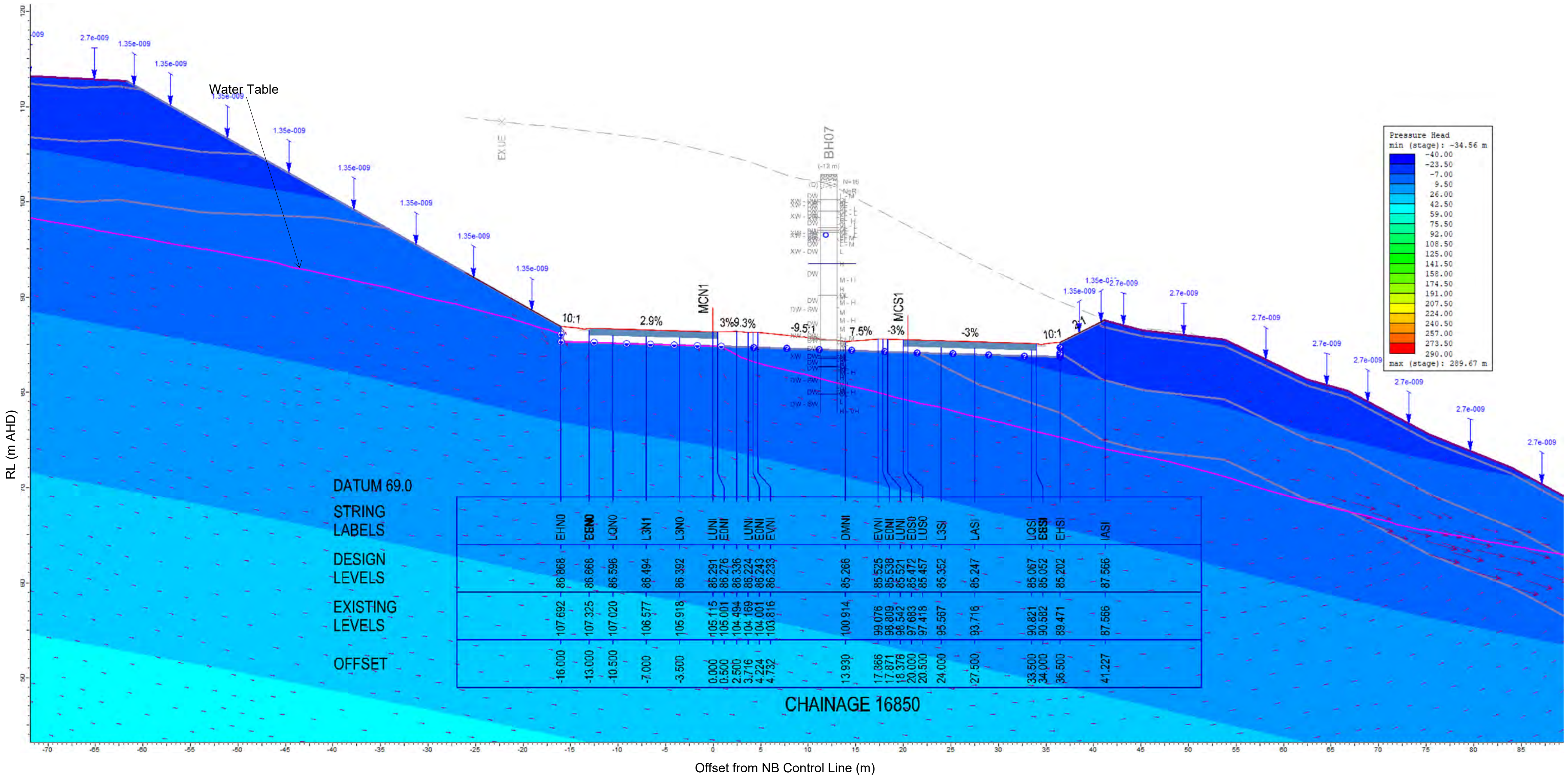
Cut 14 (CH16850) – Existing Model

Client:	Roads and Maritime Services	**RCA Australia**
Project:	Coffs Harbour Bypass	
Location:	Coffs Harbour	**RCA ref:** 11717-818



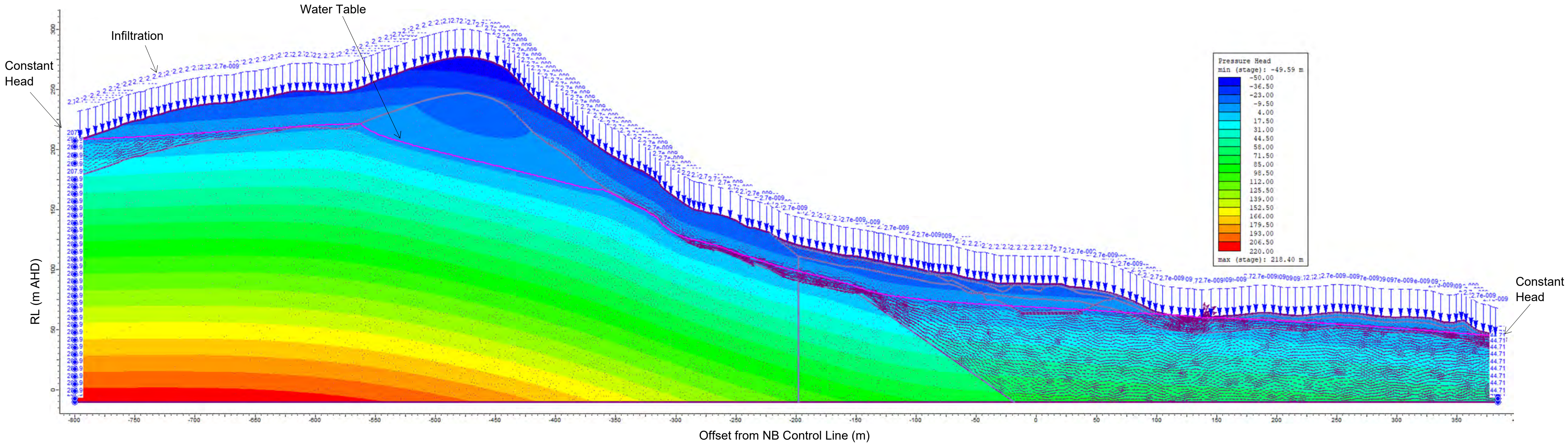
Cut 14 (CH16850) – Simulated Cut Model

Client:	Roads and Maritime Services	**RCA Australia**	
Project:	Coffs Harbour Bypass		
Location:	Coffs Harbour	**RCA ref:** 11717-818	



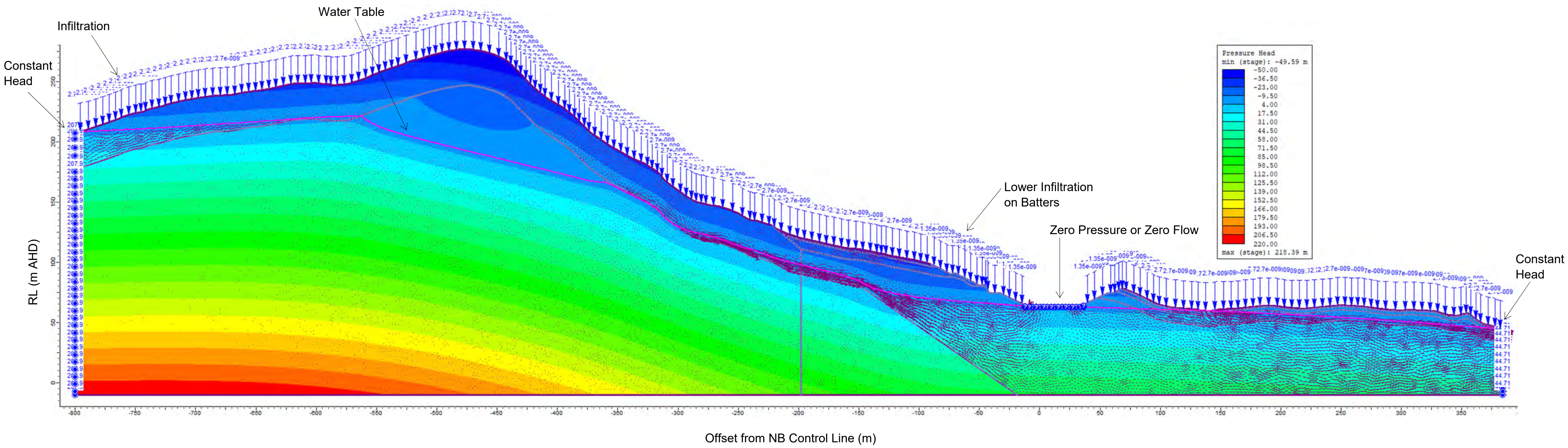
Cut 14 (CH16850) – Simulated Cut Model Showing Cutting Detail

Client:	Roads and Maritime Services	**RCA Australia**	
Project:	Coffs Harbour Bypass		
Location:	Coffs Harbour	**RCA ref:** 11717-818	



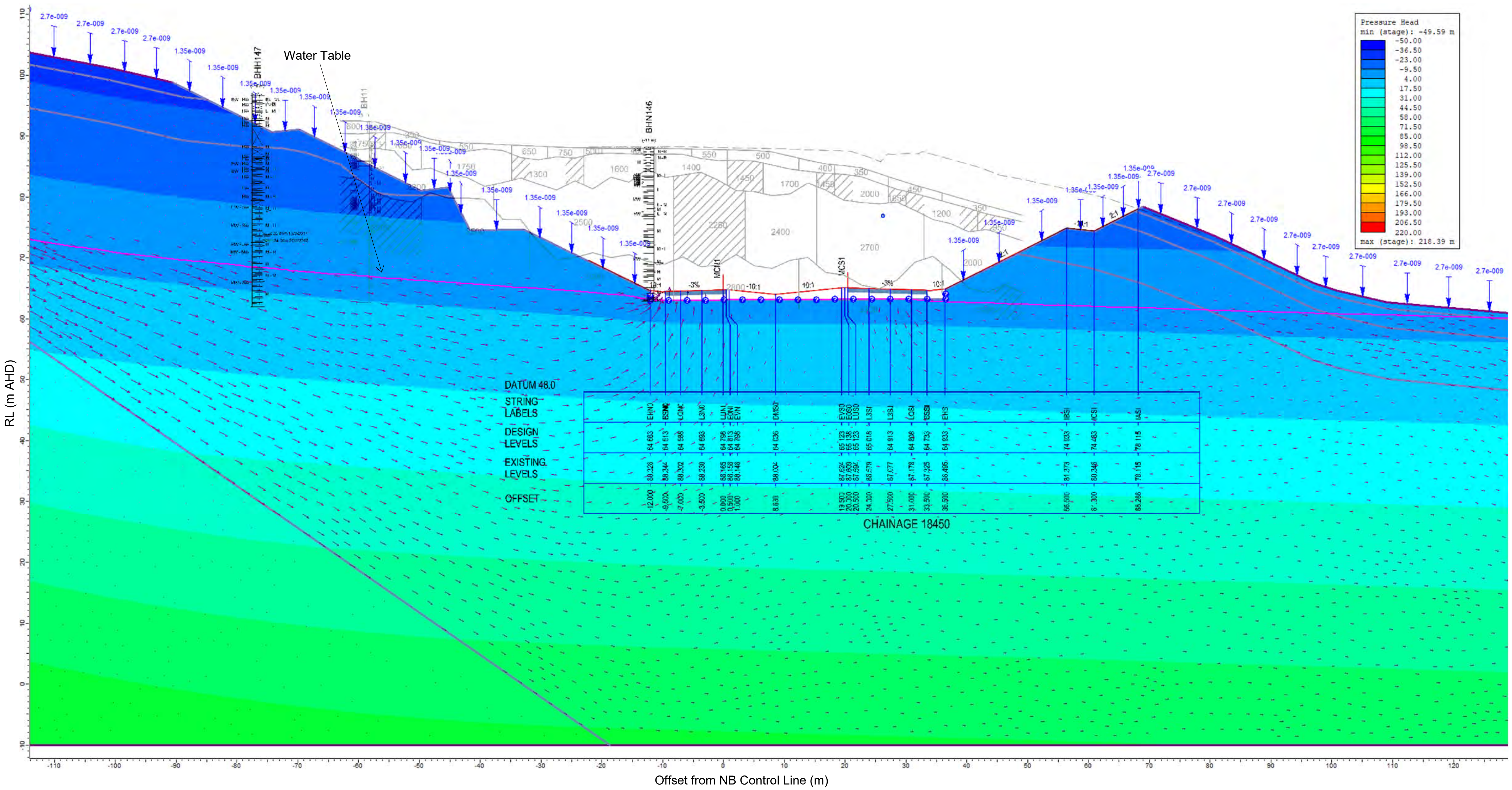
Cut 16 (CH18450) – Existing Model

Client:	Roads and Maritime Services	**RCA Australia**
Project:	Coffs Harbour Bypass	
Location:	Coffs Harbour	**RCA ref:** 11717-818



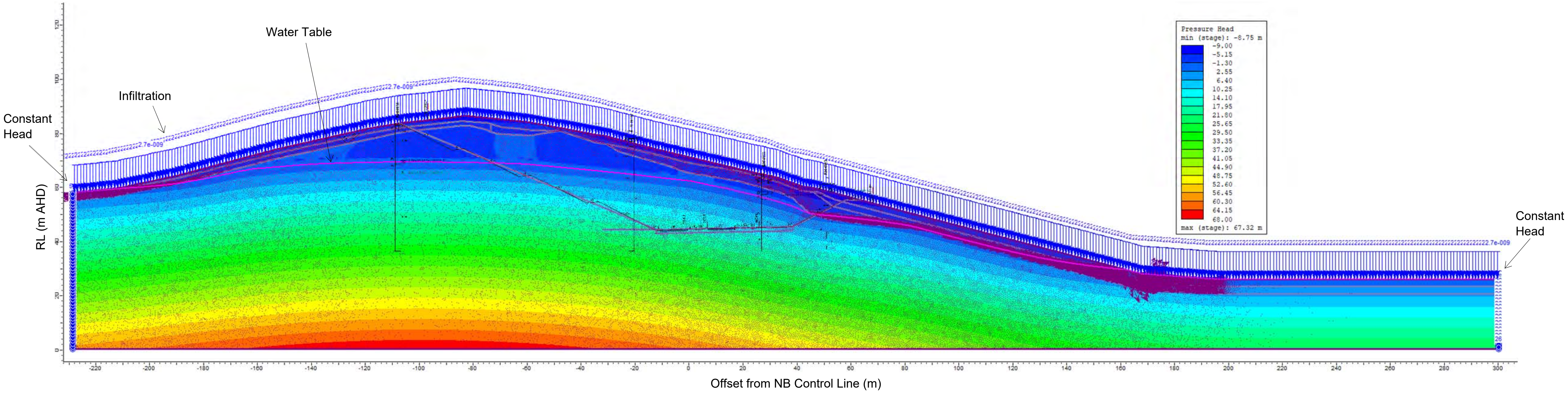
Cut 16 (CH18450) – Simulated Cut Model

Client:	Roads and Maritime Services	**RCA Australia**
Project:	Coffs Harbour Bypass	
Location:	Coffs Harbour	**RCA ref:** 11717-818



Cut 16 (CH18450) – Simulated Cut Model Showing Cutting Detail

Client:	Roads and Maritime Services	**RCA Australia**
Project:	Coffs Harbour Bypass	
Location:	Coffs Harbour	**RCA ref:** 11717-818



Cut 18 (CH20425) – Existing Model

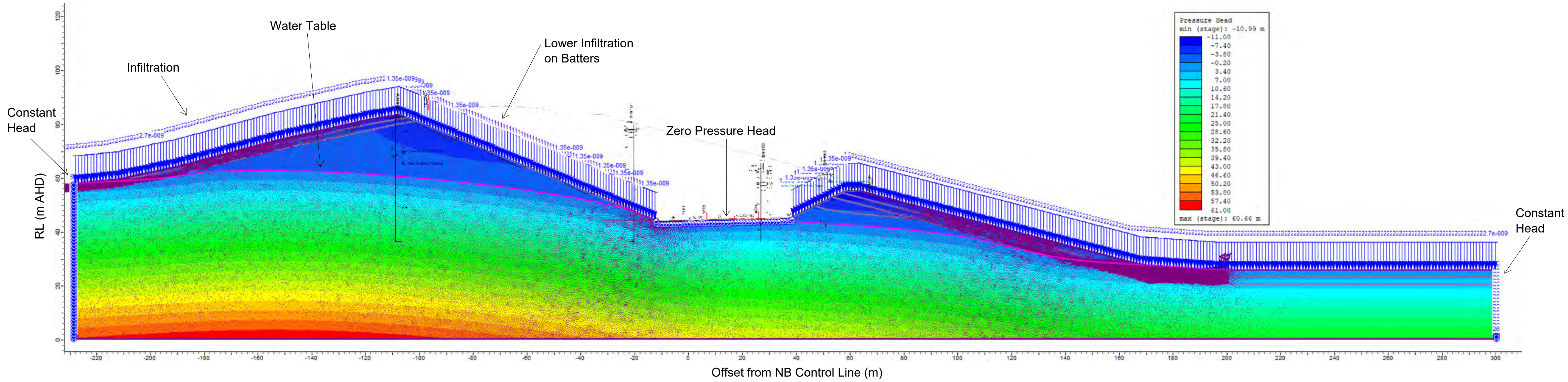
Client: Roads and Maritime Services

Project: Coffs Harbour Bypass

Location: Coffs Harbour

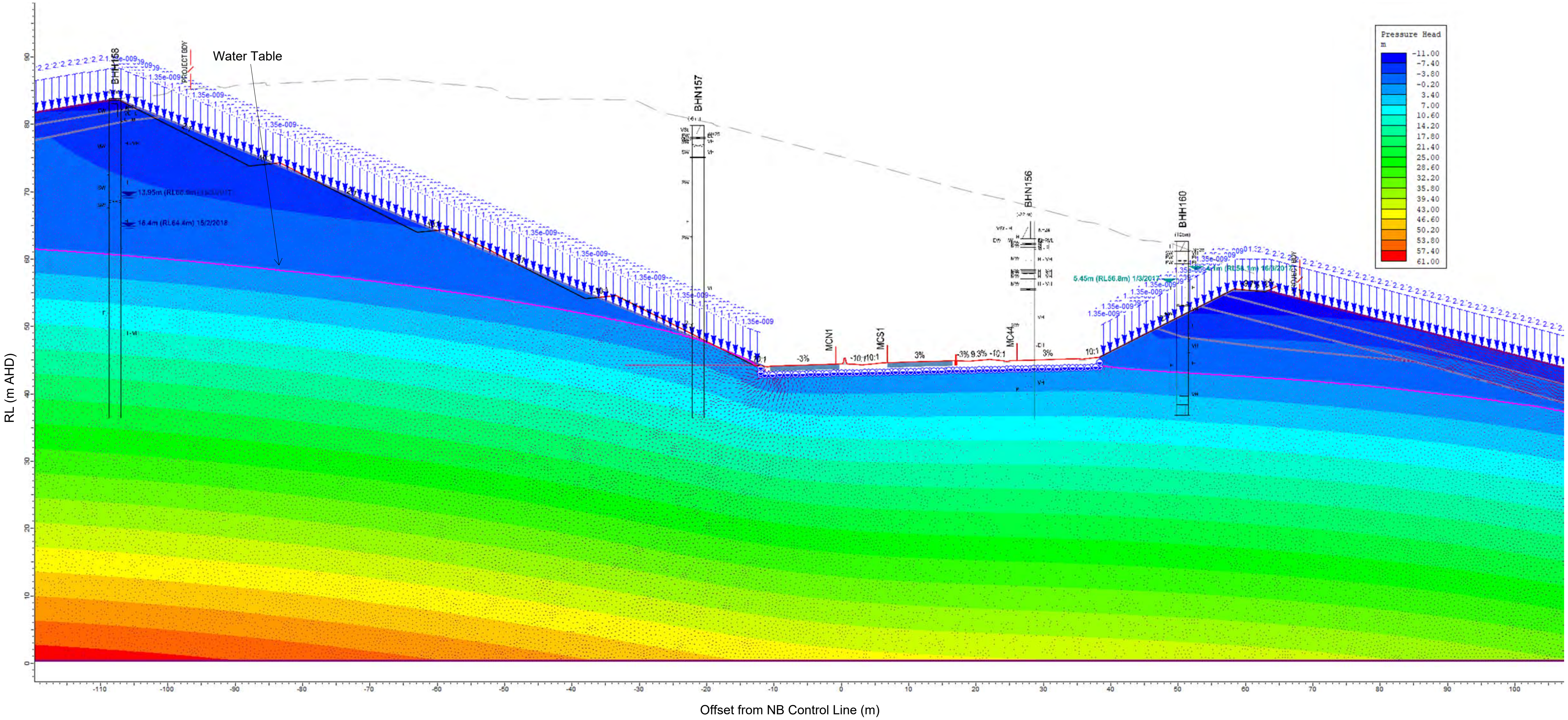
RCA Australia

RCA ref: 11717-818



Cut 18 (CH20425) – Simulated Cut Model

Client:	Roads and Maritime Services	**RCA Australia**
Project:	Coffs Harbour Bypass	
Location:	Coffs Harbour	**RCA ref:** 11717-818



Cut 18 (CH20425) – Simulated Cut Model Showing Cutting Detail

Client:	Roads and Maritime Services	**RCA Australia**
Project:	Coffs Harbour Bypass	
Location:	Coffs Harbour	**RCA ref:** 11717-818

C2 PSM Modelling Report

RCA Australia

Coffs Harbour Bypass - Groundwater Assessment Report

PSM2876-057R 10 June 2019

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1 Introduction

This Groundwater Assessment Report presents the conceptual hydrogeological models and numerical groundwater flow models for the discrete project areas of the Roberts Hill, Shephards Lane and Gatelys Road tunnels. The areas of interest for this study encompass the individual tunnels, adjoining portals and extend laterally within the local catchments, Figure 1. The conceptual hydrogeological models have been informed by available site investigation data and interpretations of the local catchments in context to rainfall recharge and groundwater discharge landforms. The numerical groundwater flow models represent the conceptual hydrogeological models and provide a predictive tool developed in accordance with the Australian Groundwater Modelling Guidelines (Barnett et.al., 2012).

Groundwater inflows to the proposed tunnel excavations and potential for associated changes to the existing groundwater environment were assessed using predictive groundwater flow models.

1.1 Background

The Coffs Harbour Bypass includes approximately 14 km of motorway upgrade of the Pacific Highway, from Englands Road in the south, to the new four-lane divided highway at Sapphire in the north (Roads and Maritime Services, 2016). This report uses the 'The project' Concept Design as the reference design for the three tunnels (see Figure 2).

Robert Carr & Associates (RCA) have been commissioned by the Roads and Maritime Services (RMS) to undertake detailed geotechnical investigations for the proposed bypass. PSM formed part of the RCA team, bringing specialisation in tunnelling to the geotechnical investigation. Arup have been commissioned by the RMS to refine the concept design and prepare the Environmental Impact Statement.

The groundwater assessment presented in this report includes:

- A review and compilation of all available data into the relevant electronic formats, summary tables and commentary as necessary – Section 2, 3 and 4
- A description of the work, including collection and testing of groundwater samples to provide data for the establishment of baseline groundwater quality– Section 3, 4 and 5
- Presentation of conceptual and numerical groundwater flow models before, during and after the tunnel construction – Section 4 and 5
- Assessment of groundwater drawdown from the drained tunnels - Section 6, 7, 8 and 9
- Assessment of current groundwater use and the possible effect on the aquifer where existing users are abstracting groundwater - Section 7, 8 and 9
- Advice regarding groundwater monitoring requirements to assess impacts of construction and operation of the Coffs Harbour Bypass – Section 10.

Copies of all factual data, laboratory test results and collected information from the geotechnical investigations are provided in Appendices A and B.

2 Site Description

Descriptions of the location, topography, land-use, geology (including bedrocks, metamorphism and structure) and soils are provided in the Tunnel Geotechnical Investigation Reports:

- Roberts Hill (PSM2876-013R, August 2018)
- Shephards Lane (PSM2876-030R, August 2018)
- Gatelys Road (PSM2876-031R, August 2018).

2.1 Climate and Rainfall

Coffs Harbour has a humid, subtropical sub-climate. Rainfall is seasonal, with the mean monthly rainfall ranging from a low of 60 mm in September to a high of 235 mm in March. Mean annual rainfall depth is 1,651 mm.

The mean monthly temperature is also seasonal, ranging from a low of 7.6 to 18.8°C in July to a high of 19.5 to 27°C in January.

Mean annual pan evaporation potential is 1,602 mm, with a monthly high of 192 mm in January and a monthly low of 69 mm in June (Bureau of Meteorology, 2017).

2.2 Catchment Hydrology

The Roberts Hill, Shephards Lane and Gatelys Road tunnel alignments traverse the mid to upper reaches of the catchment divides between Newports Creek, and Tributaries of Coffs Creek and Jordans Creek, respectively.

Catchment areas for each of the tunnel settings are summarised in Table 1. Surface water catchments in relation to each tunnel setting are shown on Figure 2.

Table 1 – Local Catchment Areas

Creek	Catchment Areas (km^2)		
	Roberts Hill	Shephards Lane	Gatelys Road
Newports	27.2		
Coffs	25.4	25.4	25.4
Jordans			4.2

Rainfall sheds off the ridges and converges on the local creek systems. In the upper reaches of the catchments, surface water flows are expected to be episodic in response to rainfall. Surface water flows in the channel and overbank areas will infiltrate and recharge the shallow aquifer.

In the downstream reaches the water table appears shallow and with surface expressions as isolated ponds in the landscape. In the lowermost reaches of the creek systems groundwater may contribute as baseflow.

3 Data Sources

3.1 Site Investigations

RCA have undertaken site investigations in each tunnel setting. A summary of these investigations is provided in Table 2. A summary of individual borehole completions at Roberts Hill, Shephards Land and Gatelys Road is provided in Table 3. The site investigation plan for Roberts Hill, Shephards Lane and Gately Road is shown on Figures 3, 4 and 5, respectively. Included on each plan are stereo plots of defect orientations.

Table 2 – Summary of Site Investigation Activities

Activity	Number of Activities and Description		
	Roberts Hill	Shephards Lane	Gatelys Road
Seismic Refraction Surveys	2	4	3
Boreholes – Angled	4	2	3
Boreholes – Vertical	1	5	4
Borehole Imaging	3	5	4
Test Pits	1	-	-
Groundwater Testing:			
Packer Tests	In 2 Boreholes	In 7 Boreholes	In 6 Boreholes
VWP and Standpipe Piezometer	5	2	4
Dual Level VWP	-	3	3
Standpipe Piezometer	-	2	-
Laboratory Testing of Samples	Yes	Yes	Yes

Aspects of the site investigations with specific relevance to the groundwater assessment are discussed below. Detailed geological logs and discussion on other aspects of the site investigations can be found in the Tunnel Geotechnical Investigation Reports.

Table 3 – Summary of Borehole Completions

Borehole	East (m E)	North (m N)	Collar (m AHD)	Depth (m)	Dip (°) (1)	Azimuth (°) (2)	Packer Testing	Imaging	Completion
ROBERTS HILL TUNNEL									
BHH110	508249.99	6648447.58	51.54	40.25	-64	356	No	Yes	VWP and Standpipe / HOBO Data Logger
BHH111	508319.25	6648532.99	83.73	56.30	-70	114	No	No	VWP and Standpipe / HOBO Data Logger
BHH112	508238.04	6648546.74	84.03	71.85	-90	-	Yes	Yes	VWP and Standpipe / HOBO Data Logger
BHH113	508150.05	6648548.44	83.92	56.00	-70	240	Yes	No	VWP and Standpipe / HOBO Data Logger
BHH114	508177.86	6648677.61	51.61	41.80	-66	185	No	Yes	VWP and Standpipe / HOBO Data Logger
TPE113	508213.05	6648432.64	49.06	5	-90	-	No	No	Backfilled and compacted
SHEPHARDS LANE TUNNEL									
BHH138	508716.81	6651133.94	97.03	30.00	-90	-	Yes	Yes	Standpipe / HOBO Data Logger
BHH139	508829.42	6651180.56	127.74	60.10	-90	-	Yes	No	2 x VWP
BHH140	508898.71	6651148.73	160.84	82.65	-68	045	Yes	Yes	VWP and Standpipe
BHH141	508891.18	6651191.80	151.66	84.10	-90	-	Yes	Yes	2 x VWP
BHH142	508872.26	6651241.96	157.91	80.70	-67	253	Yes	Yes	VWP and Standpipe
BHH143	508943.58	6651215.81	122.13	55.52	-90	-	Yes	No	2 x VWP
BHH144	509030.05	6651219.44	105.79	40.00	-90	-	Yes	Yes	Standpipe / HOBO Data Logger

Borehole	East (m E)	North (m N)	Collar (m AHD)	Depth (m)	Dip (°) (1)	Azimuth (°) (2)	Packer Testing	Imaging	Completion
GATELYS ROAD TUNNEL									
BHH148	510582.25	6651115.77	79.54	35.03	-69	315	No	Yes	VWP and Standpipe / HOBO Data Logger
BHH149	510742.16	6651131.76	138.70	98.00	-70	291	Yes	No	2 x VWP
BHH150	510778.01	6651175.98	154.09	91.27	-90	-	Yes	No	VWP and Standpipe
BHH151	510795.83	6651133.55	150.52	101.27	-90	-	Yes	Yes	2 x VWP
BHH152	510882.72	6651114.76	139.07	98.20	-68	131	Yes	Yes	2 x VWP
BHH153	510901.62	6651040.73	136.19	74.40	-90	-	Yes	No	VWP and Standpipe
BHH154	511013.98	6651126.49	86.82	44.55	-90	-	Yes	Yes	VWP and Standpipe / HOBO Data Logger

Notes:

1 Inclination from horizontal. Measured using Raax imaging. Where no imaging available, taken by the set-out angle.

2 Relative to magnetic north. Measured using Raax imaging. Where no imaging available, taken by survey.

3.1.1 Packer testing

Packer testing was carried out using a single inflatable packer arrangement. Testing parameters including test interval and water pressure were selected by site logging personnel for each test.

Water loss was recorded at variable time intervals with each water pressure stage maintained until repeatable measurements were achieved. This typically took between 10 to 15 minutes per stage. Summary sheets including calculated average water loss per minute for each water pressure stage, test classifications and calculated Lugeon values are presented for Roberts Hill, Shephards Lane and Gatelys Road in Appendices A1, A2 and A3 respectively.

A comparison across the three tunnel sites is provided in Appendix A4. Hydraulic conductivity was estimated using Moye's (1967) Method.

3.1.2 VWP and standpipe piezometers

Grouted-in VWPs were installed to monitor hydraulic heads at depth within the bedrock.

Standpipe piezometers were constructed to monitor changes in groundwater levels and to recover groundwater samples for chemical testing. Each standpipe was developed following construction, purging:

- A minimum groundwater volume three-times the calculated volume of the sealed screen interval, or
- Until repeatability of field testing of pH, Electrical Conductivity (EC), and turbidity achieved less than 10 per cent variation.

Construction data for the VWP and standpipe piezometers are provided in Table 4.

Table 4 – VWP and Standpipe Construction Data

Tunnel Site	Monitoring Bore	Standpipe Setting Water Table Zone					VWP Depth Setting (m)	VWP Grout Interval (m)	Construct Date
		Depth (m)	Slotted Interval (m)	Gravel Pack (m)	Bentonite (m)	HOBO Depth (m)			
Roberts Hill	BHH110	18	12 – 18	9.9 – 20	7.8 – 9.9	15.9	31	20 – 40.2	15/02/2017
	BHH111	34.9	28.9 – 34.9	26.5 – 34.9	24.5 – 26.5	34.25	54	34.9 – 56.3	15/02/2017
	BHH112	40.0	34.0 – 40.0	31.6 – 40.0	28.9 – 31.6	39	60	40.0 – 71.8	14/02/2017
	BHH113	36.6	30.6 – 36.6	26.9 – 40.4	24.6 – 26.9	34.2	54.17	40.4 – 56	14/02/2017
	BHH114	17.6	11.6 – 17.6	9.6 – 17.9	7.9 – 9.57	15.9	31	17.9 – 41.6	05/02/2017
Shephards Lane	BHH138	12.5	6.5 – 12.5	5 – 13.8	2.3 - 5	12.17	N/A	13.8 - 30	24/11/2016
	BHH139	N/A				N/A	VWP1 – 48 VWP2 - 28	0 – 54	01/12/2016
	BHH140	62.8	56.8 – 62.8	54.7 – 64.6	0.4 – 54.7	N/A	82.5	64.6 – 82.65	20/12/2016
	BHH141	N/A				N/A	VWP1 – 72.4 VWP2 – 52.35	0 – 84.1	02/12/2016
	BHH142	55.9	49.9 - 55.9	47.7 - 56.6	45.2 – 47.7	N/A	75.8	56.6 - 80	11/12/2016
	BHH143	N/A				N/A	VWP1 – 42 VWP2 – 22	0 – 55.52	08/12/2016
	BHH144	22.7	16.7 – 22.7	14.9 – 24.5	12.6 – 14.9	18	N/A	24.5 – 40	17/11/2016

Tunnel Site	Monitoring Bore	Standpipe Setting Water Table Zone					VWP Depth Setting (m)	VWP Grout Interval (m)	Construct Date
		Depth (m)	Slotted Interval (m)	Gravel Pack (m)	Bentonite (m)	HOBO Depth (m)			
Gatelys Road	BHH148	12.0	6.0 – 12.0	4.8 – 14.9	3.6 – 4.8 14.9 – 17.5	11.5	18.67	17.5 – 35.03	7/4/17
	BHH149	N/A				N/A	VWP1 – 75.2 VWP2 – 56.4	0 – 98.0	7/4/17
	BHH150	64.0	58.0 – 64.0	56.7 – 64.7	54.8 – 64.7	N/A	89.8	64.7 – 89.9	12/4/17
	BHH151	N/A				N/A	VWP1 – 88.5 VWP2 – 70.0	0 – 101.27	4/4/17
	BHH152	N/A				N/A	VWP1 – 76.5 VWP2 – 57.9	0 – 98.2	5/4/17
	BHH153	54.1	48.1 – 54.1	46.6 – 55.2	45.6 – 46.6	N/A	73.4	55.2 – 74.4	7/4/17
	BHH154	15.6	9.6 – 15.6	7.9 – 17.5	5.4 – 7.9	15	28.0	43.5 – 17.5	13/4/17

3.2 Groundwater Monitoring

3.2.1 Groundwater levels

Groundwater levels have been recorded manually from the standpipe piezometers since November 2016 and using HOBO data loggers since July 2017. Groundwater level data are shown plotted against rainfall in Appendix B.

Vibrating Wire Piezometers (VWPs) measure the hydraulic head at depth within the bedrock units. Manual measurements have been taken at all VWP's since May 2017 and using automated data loggers since July 2017 (Appendix B) at:

- Roberts Hill (BHH110, BHH112 and BHH114)
- Shephards Lane (BHH139, BHH141, and BHH143)
- Gatelys Road (BHH148, BHH149, BHH151, BHH152 and BHH154).

Comparisons of the VWP and logger data indicate the groundwater system is vertically connected, supporting the interpretation of comparatively high recharge and high vertical hydraulic conductivity.

A downwards vertical hydraulic gradient exists at all three sites, indicative of recharge zones. Upwards hydraulic gradients are observed downgradient of the ridge at Gatelys Road (BHH148, BHH154) indicating groundwater discharge zones.

At Roberts Hill, an agricultural dam is observed in proximity to the tunnel alignment, Figure 6. The landowner indicated that the dam (508354.5 m E, 6648252.9 m N) is fed by both groundwater baseflow and overland flow downgradient of where the water table intersects the topography. Inspection of the aerial imagery reveals several ponds similarly located along the 30 m RL topographic contour which is interpreted to be the approximate elevation of the water table downgradient of the ridge.

At Shephards Lane, aerial imagery reveals a pond located at the 140 m RL topographic contour, Figure 7. It is not clear whether this elevation corresponds to the water table up-gradient of the proposed tunnel or a perched aquifer system.

At Gatelys Road, aerial imagery reveals several ponds and agricultural dams located between 45 and 70 m RL, Figure 8, which is interpreted as the approximate elevation of the water table down-gradient of the ridge.

3.2.2 Groundwater quality

Several groundwater samples were collected from standpipe piezometers on 20 and 27 April 2017 for chemistry analysis. Qualitative results are summarised in Table 5 and presented in Appendix B4.

Groundwater samples were also collected in May 2018 and analysed for major ions and metals. Samples relevant to the tunnel sites are summarised in Tables B4-1, B4-2 and B4-3 of Appendix B4.

It is recommended that all boreholes sampled in May 2018 be resampled and analysed for the analytes presented in Table 5. In addition, the samples should be analysed for major ions and selected metals to ensure repeatability of results.

To improve the spatial coverage of the baseline sampling program it is recommended that the additional standpipes presented in Table 4 be utilised in future sampling campaigns. This should include BHH142 at Shephards Lane and BHH150 at Gatelys Road. Other available standpipes will be excavated and do not have suitable longevity for use in the baseline monitoring network.

It is noted that the metal concentrations reported for May 2018 are particularly high and sampling and filtering procedures should be reviewed to ensure reliability and repeatability of the results. For metals analyses, it is essential that the samples are filtered and clear to avoid analyses of suspended particulates and sediment.

Table 5 – April 2017 Groundwater Chemistry Analysis

Analysis	Units	Roberts Hill	Shephards Lane		Gatelys Road
		BHH114	BHH140	BHH144	BHH147
pH	pH unit	6.37	7.17	7.87	6.37
Total Dissolved Solids	mg/L	199	NA	210	NA
EC (@25 °C)	µS/cm	259	370	279	410
Turbidity	NTU	2	9	<1	<1
Total Alkalinity (as $CaCO_3$)	mg/L	45	121	74	69
Sulphate (as SO_4)	mg/L	35	27	26	121
Dissolved Oxygen	mg/L	NA	9.6	NA	2.5

3.3 Lineament Analysis

Geological structures have been evaluated from a lineament analysis using hill-shade models generated from the 1 m LiDAR data. The lineaments were recognised from their geomorphological expression which included:

- Well defined topographical linear features
- Defined breaks in slope
- Linear ridges and/or valleys
- Drainage control.

Lineaments associated with the main structural trends were observed to intersect the respective tunnel alignments and adjacent cuttings. The main structural trends which may influence groundwater flows to the tunnels are summarised in Table 6 and are shown for Roberts Hill, Shephards Lane and Gately Road on Figures 3, 4 and 5, respectively.

Table 6 – Main Structural Trends

Roberts Hill	Shephards Lane	Gatelys Road
NNE-SSW Trending Lineaments		
ENE-WSW Trending Lineaments		
NA	NWN-SES Trending Lineaments	

4 Conceptual Hydrogeological Model

4.1 Hydrogeological Units

The bedrock profile at each tunnel site comprises argillite and siliceous argillite. Hydrogeological parameters are correlated with weathering and defect characteristics of the bedrocks. These features are generally captured by the Rock Mass Units (RMUs) described in PSM2876-015R. Hydrogeological units (HRMUs) are shown in long-section along the CHB alignment for Roberts Hill, Shephards Lane and Gately Road on Figures 9, 10 and 11, respectively.

The individual HRMUs are described below:

Soil – The soil profile comprises:

- Colluvium material:
  - Logged on the southern flank of Roberts Hill up to 1.5 m bgl
  - Logged on the eastern flank of Shephards Lane up to 4.5 m bgl
  - Logged on the eastern flank of Gatelys Road up to 3 m bgl.
- Residual soil:
  - Across the Roberts Hill site up to 5.2 m bgl
  - Across the Shephards Lane site up to 3 m bgl, with localised depths up to 8.5 m bgl on the eastern flank
  - Residual soil across the Gatelys Road site up to 2 m bgl, with localised depths up to 7.2 m bgl on the eastern flank.

Typically, there is little information on the soil profiles and as such these two sub-units have been lumped together. The geological maps indicate that alluvial deposits occur beneath the lower slopes at each tunnel site. Alluvium has not been differentiated from the other soil units for the RMU analysis.

RMU-A – Extremely Weathered to Highly Weathered Rock.

RMU-B1 – Closely spaced argillite (cleavage dominated). This unit is split into two hydrogeological units based on packer testing data and observed groundwater behaviour:

- Moderately Weathered (MW) rock
- Slightly Weathered (SW) rock.

RMU-B2 – Widely spaced argillite (cleavage dominated). This unit is typically unweathered.

RMU-C1 – Closely spaced laminated to thinly bedded argillite and siliceous argillite. This unit is split into two hydrogeological units based on packer testing data and observed groundwater behaviour:

- Moderately Weathered (MW) rock
- Slightly Weathered (SW) rock.

RMU-C2 – Widely spaced laminated to thinly bedded argillite and siliceous argillite. This unit is typically unweathered.

RMU-D1 – Closely spaced thickly bedded argillite and siliceous argillite. This unit is split into two hydrogeological units based on packer testing data and observed groundwater behaviour:

- Moderately Weathered (MW) rock
- Slightly Weathered (SW) rock.

RMU-D2 – Widely spaced thickly bedded argillite and siliceous argillite. This unit is typically unweathered.

RMU-E – Major Fault Zone. Note that:

- At Roberts Hill, a major geological structure has been noted in BH111. Core orientation of the fault is not available. The fault is, however, potentially associated with the NNE-SSW orientated lineaments which are observed to intersect the alignment

- At Shephards Lane, major geological structures have been noted in BHH141 and BHH142:
  - A fault was included as part of a 17 m test interval in BHH141. Lugeon values do not show a high hydraulic conductivity, but this could be due in part, to averaging of the 3.6 m fault zone across the test interval
  - A fault in BHH142 was potentially associated with ENE-WSW orientated lineaments which intersect the alignment at CH17050.
- At Gatelys Road, no major structures were observed in the boreholes
- There is no packer testing discretely associated with the fault intersections, hence there are no specific data to attribute the local hydraulic characteristics and potential influences on groundwater flows
- The major fault zones are not included in the groundwater flow models. The faults are located off-section making it difficult to provide reasonable representation of potential influences using a cross-sectional model.

4.2 Flow System

Each tunnel alignment traverses beneath a ridge line that forms a local-scale catchment divide. Rainfall on the ridge line will partition between infiltration (recharge) and runoff. Schematics of the interpreted flow system in each tunnel setting are shown on Figures 9, 10 and 11.

The timing of the responses to rainfall vary significantly between boreholes; the observed responses at Shephards Lane are comparatively small in amplitude. The range of responses is typical of heterogeneity in aquifer hydraulic conductivity and storage, hydraulic anisotropy and likely presence of variable preferred flow paths.

4.2.1 Recharge

Direct rainfall will infiltrate along the ridge lines, which are recharge zones for the broader catchments, and will provide a source for groundwater flow to the tunnels. Large water table fluctuations observed beneath the top of the ridge (for example in BHH140 at Shephards Lane) are probably indicative of a fractured rock environment with comparatively high vertical hydraulic conductivity, low specific yield and short flow paths. Vertical hydraulic gradients between the standpipe piezometers and VWPs indicate that rainfall on the ridge line will infiltrate the upper soil units and migrate vertically downwards and laterally towards the water table within the upper weathered and fractured profile. It is interpreted that recharge is enhanced by the high-angle cleavage fabric in the fractured rocks (shown on Figures 9, 10 and 11), with this fabric promoting vertical infiltration and subsequence lateral watershed in the shallow water table groundwater flow system.

The depth to water table and limited native vegetation indicate that there is limited potential for evapotranspiration along the ridge. This is supported by the fresh groundwater (TDS of 199 and 210 mg/L) sampled in BHH114 and BHH144.

Rates of recharge have been interpreted (Table 7) by analysing the measured groundwater response to rainfall (Appendix B) observed in:

- Roberts Hill - BHH111, BHH112 and BHH113
- Shephards Lane – BHH140
- Gatelys Road - BHH149, BHH150, BHH151, BHH152 and BHH153.

The hydrograph recharge analysis considers:

- Observed rise in groundwater levels
- Corresponding rainfall depth for the period of the hydrograph rise
- Assumed specific yields typical of the material at the screened interval.

Table 7 – Recharge Analysis

Borehole	From	To	Rainfall (mm)	Water Table Rise (m)	Recharge (Per Cent) Specific Yield (dimensionless) 0.01	0.02
Roberts Hill						
BHH111	15/02/2017	16/03/2017	404.2	5.85	14	29
BHH112	16/03/2017	30/03/2017	302.3	4.63	15	31
BHH113	14/02/2017	16/03/2017	409.4	4.56	11	22
Shephards Lane						
BHH140	03/03/2017	16/03/2017	290.3	4.68	16	32
Gatelys Road						
BHH149	20/02/2017	05/04/2017	375.7	3.01	8	16
BHH150	20/02/2017	04/04/2017	375.7	-0.24	NA	NA
BHH151	20/02/2017	04/04/2017	375.7	5.46	15	29
	16/03/2017	04/04/2017	420.5	7.95	19	38
BHH152	20/02/2017	04/04/2017	375.7	0.93	2	5
	16/03/2017	04/04/2017	420.5	4.11	10	20
BHH153	20/02/2017	04/04/2017	375.7	1.53	4	8
	16/03/2017	04/04/2017	420.5	5.19	12	25

The analysis indicates that recharge (infiltration) rates may range from less than 10 per cent to 30 per cent of average annual rainfall. BHH111 and BHH113 show a rapid response to rainfall; more so than BHH112 which lags these two hydrographs. BHH111 and BHH113 are located higher in the profile in the MW/SW RMUs so receive recharge first, with subsequent drainage to the underlying fresh bedrocks. The wide variation in estimated recharge rates is indicative of the aquifer heterogeneity, common in fractured rocks, and the non-linearity in the relationship between rainfall and recharge. More frequent logger data would assist in refining the analysis.

4.2.2 Flow paths

Groundwater sourced from recharge zones on ridge lines is interpreted to be shed on shallow flow lines influenced by the local topography and catchments divides. Estimated flow lines derived from the topography and groundwater level data are shown for Roberts Hill, Shephards Lane and Gately Road on Figures 6, 7 and 8, respectively. There may be a minor flow component along the ridge lines, enhanced by preferential flow paths along cleavage planes. The figures show the ridge lines bound the flow paths, with watersheds to either side. Further, the watersheds off the ridge lines occurs in the form of numerous small catchments and creeks.

As rainfall infiltrates it will move downwards under gravity but also laterally at the interface between units of contrasting hydraulic conductivity. The significance of the lateral flow through the unsaturated zone is directly related to the relative transmissivity of the different units, with flow moving preferentially through the higher transmissivity profiles as perched aquifers above the baseline groundwater level, Figures 9, 10 and 11.

4.2.3 Discharge

Groundwater discharge on the flow lines occurs in several ways, including:

- From local breaks in slope where the water table, as natural springs, intersects the land surface
- Interception by evaporation and transpiration, from shallow water table zones, springs and ponded water
- Groundwater flow to the alluvial aquifer lower in the landscape
- Baseflow contribution to local watercourses.

There is likely to be transient variability in the proportion of discharge to individual discharge mechanisms. For example, discharge at breaks of slope may be episodic only after high rainfall events, whilst contributions to baseflow commonly change seasonally and episodically.

Ponds identified in the local catchments are generally interpreted to be surface expressions of the water table. These ponds act as local discharge zones where groundwater is lost to evaporation, transpiration and through-flow.

4.2.4 Springs and Dams

Several natural groundwater springs are located near the breaks in slope between the outcropping bedrock and the alluvial materials which provide a source of water to local landowners.

These conceptual perched aquifer and spring flow mechanisms are shown schematically on Figures 9, 10 and 11.

Several agricultural dams are in the vicinity of the tunnels. The dams are likely to be excavated beneath the water table to provide a more secure and perennial source of groundwater to the local landowners as shown on Figures 12 to 15. The depths of the dams are unknown.

4.3 Hydraulic Conductivity

Packer testing data from across all three tunnel sites has been used to interpret hydraulic conductivity ranges for the RMUs as shown in Table 8.

Hydraulic testing data was not available for the colluvium and residual soil. Given the bedrock comprises fine grained material it is assumed that the soil will be relatively fine grained. It is also assumed hydraulic conductivity values would not be less than those measured in the Moderately Weathered rock.

Table 8 – Interpreted Hydraulic Conductivity

HRMU	Hydraulic Conductivity (m/day)		
	Minimum	Geometric Mean	Maximum
Soil	0.03	-	0.4
RMU-A	0.03	-	0.4
RMU-B1 (MW)	NA	NA	NA
RMU-B1 (SW)	1.1×10^{-5}	9.4×10^{-4}	2.2×10^{-2}
RMU-B2	2.0×10^{-4}	5.9×10^{-4}	5.4×10^{-3}
RMU-C1 (MW)	0.13	0.14	0.15
RMU-C1 (SW)	1.8×10^{-3}	7.3×10^{-3}	5.6×10^{-2}
RMU-C2	3.6×10^{-5}	2.2×10^{-3}	4.9×10^{-2}
RMU-D1 (MW)	3.3×10^{-2}	0.14	0.43
RMU-D1 (SW)	9.7×10^{-4}	3.4×10^{-2}	0.48
RMU-D2	3.9×10^{-4}	6.1×10^{-3}	0.18

4.4 Hydraulics Anisotropy

Sub-vertical cleavage planes were identified as the dominant defect set in the fresh bedrock. This implies vertical hydraulic conductivity may be comparatively enhanced and greater than horizontal hydraulic conductivity (perpendicular to the ridge). The horizontal versus vertical hydraulic conductivity anisotropy ratio is unknown but may be in the range of 1:2 to 1:10. Anisotropy for the three tunnel sites is assumed to correspond to the assigned structural domains:

- A single structural domain has been assigned at Roberts Hill
- Two structural domains have been assigned at Shephards Lane:
  - West Domain – dipping predominantly 50-90°/N to E
  - East Domain – dipping predominantly 50-90°/S to W.
- Two structural domains have been assigned at Gatelys Road:
  - West Domain – dipping moderately to steeply to the S and W
  - East Domain – dipping steeply to the SE.

The colluvium and residual soil are assumed to be isotropic. There are no data available to further inform the anisotropy ratio of the soil.

4.5 Storage

Typical ranges for the specific yield of soil and weathered and fractured rock are presented in Table 9. Adopted values were determined through calibration of the numerical model against observed water level changes, recharge rates and a range of hydraulic conductivity values.

Table 9 – Specific Yield

HRMU	Specific Yield (Dimensionless)	
	Minimum	Maximum
Soil	0.03	0.05
RMU-A	0.03	0.05
RMU-B1	0.02	0.03
RMU-B2	0.01	0.02
RMU-C1	0.02	0.03
RMU-C2	0.01	0.02
RMU-D1	0.02	0.03
RMU-D2	0.01	0.02

5 Cross-Section Groundwater Flow Model Development

Individual cross-sectional numerical groundwater flow models were developed for the Roberts Hill, Shephards Lane and Gately Road tunnel alignments. Each model was used to explore the concepts and parameters developed for the conceptual hydrogeological model. The objective of the groundwater flow models was to predict groundwater inflows to the tunnels and assess the amplitudes and lateral extents of drawdown propagation away from the tunnels.

Important contexts regarding the cross-section groundwater flow models include:

- The observed groundwater behaviours are more complex and varied than currently captured with the cross-sectional models

- The conceptual hydrogeological model acknowledges the presence of a heterogeneous horizontally and vertically anisotropic fractured rock aquifer. The cross-sectional models apply horizontally homogeneous hydraulic properties to discrete rock mass units and similarly for storage characteristics
- More frequent monitoring data will help to assess the transient attributes of the flow paths intersected by the monitoring facilities.

5.1 Model Design

The two-dimensional (2D) cross-sectional groundwater flow models were developed in FEFLOW 7.0, the finite element sub-surface flow simulation system. The model setups for the Roberts Hill, Shephards Lane and Gatelys Road tunnel alignments are shown on Figure 16, 17 and 18, respectively. The models were vertically orientated and aligned broadly with the interpreted groundwater flow lines.

Groundwater flow was simulated using Richard's Equation for variably saturated porous media flow. A simplified linear form was used to represent the constitutive relationships between pore pressure, saturation and hydraulic conductivity.

Recharge was applied at the simulated ground surface as a percentage of average annual rainfall. Lower recharge rates were applied downgradient, where the water table is shallow, to represent lower net recharge due to evaporation and other discharge mechanisms. Fixed head boundary conditions were applied at the horizontal extents of the model domain.

5.2 Calibration

5.2.1 Steady-state

The objective of the steady-state calibration was to provide an estimate of rainfall recharge and hydraulic conductivity by matching the simulated hydraulic heads to observed hydraulic heads along the long-section. Hydraulic conductivity values were constrained by the interpreted range shown in Table 8.

Hydraulic conductivities were applied within the measured range for the individual RMUs. Hydraulic conductivity of the residual soil and colluvium was assumed to be 0.3 and 0.2 m/day, respectively. The recharge rate was then estimated, as a percentage of average annual rainfall, by matching the simulated and observed heads in steady-state. Based on the geometric mean hydraulic conductivities a recharge rate of 15 per cent (248 mm/annum) was required to achieve a satisfactory calibration. The position of the calibrated Roberts Hill, Shephards Lane and Gatelys Road steady-state water table is shown on Figures 19, 20 and 21, respectively.

The steady-state calibration results, shown in Table 10, demonstrate that over the potential range of recharge rates, the corresponding hydraulic conductivity values are within the level of confidence that can be attributed to the estimates of geometric mean hydraulic conductivity.

Anisotropy of the rock units was evaluated for vertical versus lateral hydraulic conductivity ratios between 0.1 and 10. Decreasing the ratio of vertical to horizontal hydraulic conductivity had an adverse effect on the simulated cross-section flow patterns. This result is consistent with the observation that vertical hydraulic conductivity should be greater than horizontal hydraulic conductivity based on the sub-vertical orientation of the dominant defect sets in the drill core.

Increasing the vertical to horizontal anisotropy ratio results in notable reductions in the recharge rate required to achieve a steady-state calibration. This has a compounding effect on the uncertainty highlighted in Table 10. Therefore, a nominal horizontal versus vertical hydraulic conductivity anisotropy ratio of 1:2 was assumed for the transient simulations.

Table 10 – Calibration Rainfall Recharge Rates and Hydraulic Conductivities

Rainfall Recharge (Per Cent)	20	15	10	5	2
Aspect	Hydraulic Conductivity (m/day)				
Ratio ($K/K_{GeoMean}$)	1.33	1.00	0.67	0.33	0.14
Soil	3×10^{-1}	3×10^{-1}	1×10^{-1}	7×10^{-2}	3×10^{-2}
RMU-A	3×10^{-1}	2×10^{-1}	1×10^{-1}	7×10^{-2}	$3 - 4 \times 10^{-2}$
RMU-B1 (MW)	2×10^{-1}	1×10^{-1}	9×10^{-2}	5×10^{-2}	2×10^{-2}
RMU-B1 (SW)	1×10^{-3}	7×10^{-4}	6×10^{-4}	3×10^{-4}	1×10^{-4}
RMU-B2	8×10^{-4}	6×10^{-4}	4×10^{-4}	2×10^{-4}	8×10^{-5}
RMU-C1 (MW)	2×10^{-1}	1×10^{-1}	9×10^{-2}	5×10^{-2}	2×10^{-2}
RMU-C1 (SW)	1×10^{-2}	$7 - 9 \times 10^{-3}$	5×10^{-3}	2×10^{-3}	1×10^{-3}
RMU-C2	3×10^{-3}	2×10^{-3}	2×10^{-3}	8×10^{-4}	3×10^{-4}
RMU-D1 (MW)	2×10^{-1}	1×10^{-1}	9×10^{-2}	5×10^{-2}	2×10^{-2}
RMU-D1 (SW)	5×10^{-2}	3×10^{-2}	2×10^{-2}	1×10^{-2}	5×10^{-3}
RMU-D2	8×10^{-3}	6×10^{-3}	4×10^{-3}	2×10^{-3}	8×10^{-4}

5.2.2 Transient

Calibrated steady-state heads were used as initial conditions for the transient simulations. Transient simulations were conducted for the hydraulic conductivity and recharge combinations shown in Table 10. The recharge rate was applied to the recorded daily rainfall depths for the period from October 2016 to March 2019 to create a time-varying stress on the model.

The objective of the transient calibration was to match the observed fluctuations in groundwater levels. Selected groundwater hydrographs were used as the calibration targets based on position in the landscape and recorded data for three large rainfall events that occurred in March 2017. The selected calibration boreholes included:

- Roberts Hill - BHH110 and BHH112
- Shephards Lane - BHH138, BHH140 and BHH144
- Gatelys Road - BHH148, BHH149, BHH150, BHH153 and BHH154.

Figure 22 through Figure 24, inclusive, show the simulated transient response for the 15 per cent recharge scenario in comparison to the observed data for Roberts Hill, Shephards Lane and Gately Road. The results indicate that a recharge rate of at least 10 to 15 per cent was required to replicate the observed rise in the selected hydrographs. The rapid rises and falls observed in the hydrographs are likely to be associated with high hydraulic conductivity fractures. Using mean hydraulic conductivities and a 15 percent recharge rate produced a reasonable representation of the amplitude of the groundwater changes but did not necessarily accurately capture the timing of the rises and falls of the hydrographs.

5.3 Selected Parameters

Analysis of the steady-state and transient calibration results has been used to narrow the range of reasonable model parameters as shown in Table 11.

Table 11 – Selected Hydraulic Parameters

RMU	Hydraulic Conductivity (m/day)	Hydraulic Conductivity Anisotropy Ratio (H:V)	Specific Yield (dimensionless)
Soil	0.3	1	0.05
RMU-A	0.2	1	0.05
RMU-B1 (MW)	0.1	1:2	0.02
RMU-B1 (SW)	7×10^{-4}	1:2	0.02
RMU-B2	6×10^{-4}	1:2	0.01
RMU-C1 (MW)	0.14	1:2	0.02
RMU-C1 (SW)	$7 - 9 \times 10^{-3}$	1:2	0.02
RMU-C2	2×10^{-3}	1:2	0.01
RMU-D1 (MW)	0.14	1:2	0.02
RMU-D1 (SW)	3×10^{-2}	1:2	0.02
RMU-D2	6×10^{-3}	1:2	0.01
Recharge 15 per cent			

6 Predictive Modelling

6.1 Approach and Classification

The calibrated cross-sectional groundwater flow models were used to predict groundwater inflows to the tunnels and assess the amplitudes and lateral extents of drawdown propagation away from the tunnel. Construction and operation of the individual drained tunnels will induce groundwater flow from the direction perpendicular to the tunnel alignment. To simulate lateral flow towards each tunnel a simplified 3D geometry was adopted whereby the 2D model was extruded uniformly along the ridge line. This approach allowed for the simulation of lateral flow paths induced by the tunnel but did not accurately account for the 3D geometry of the ridge line.

The predictive models are classified as Class 1 with attributes of Class 2 models under the Australian Groundwater Modelling Guidelines (Barnett et. al, 2012). Model classifications are determined for three categories:

- Data – Class 1; the available data is of limited spatial coverage with an absence of data near the sensitive receptors (agricultural dams and springs)
- Calibration – Class 2; reasonable steady-state and transient calibrations to the available data were achieved
- Prediction – Class 2; the timeframes and magnitudes of stresses for the predictive scenarios are such that the Class 2 indicators are satisfied.

6.2 Scenarios

The predictive scenarios considered a drained tunnel for the Roberts Hill, Shephards Lane and Gately Road sites. The models were run until steady-state and inflows and drawdown were assessed over this period.

A calibration-constrained sensitivity analysis was run on each of the scenarios above assuming a low groundwater recharge rate of 2 per cent of mean annual rainfall.

In each model, the tunnel and portal cuttings were simulated using seepage face boundary conditions. In the tunnel, seepage face boundary conditions were applied from the design surface to the roof of the tunnel.

The Design Surface Levels (shown in Figure 19, 20 and 21) were assumed to be the base of the tunnels. The tunnel height was assumed to be 8.8 m.

7 Roberts Hill Tunnel Predictive Change Assessments

The following is a semi-quantitative appraisal of the potentially altered groundwater environment associated with constructing and operating the Roberts Hill Tunnel. Note that potential for changes to groundwater quality were not assessed.

7.1 Groundwater Inflows and Drawdown

Steady-state groundwater inflow rates are summarised in Table 12. Transient groundwater inflow rates for the drained tunnel are shown in Figure 25a. Steady-state is reached within about 3 years.

Drawdown contours after 18 months are shown in Figure 25b and Figure 25c, in cross-section and plan view, respectively.

Drawdown contours at steady-state are shown in Figure 25d and Figure 25e, in cross-section and plan view, respectively.

Also shown on Figure 25c and 25e are the drawdowns for the low recharge sensitivity analysis.

Table 12 – Roberts Hill Steady-State Drained Tunnel Inflows

Section	Base Case		
	Groundwater Inflow (kL/day)	Length (m)	Groundwater Inflow (L/sec/100 m)
Combined	10.6	456	0.027
Tunnel	7.8	170	0.053
Southern Portal	0	35	0
Northern Portal	2.8	41	0.08

7.2 Water Balance

The steady-state passive groundwater take for the Roberts Hill Tunnel and portals was estimated at approximately 4 ML/year. Total annual recharge would be approximately 4 GL, assuming a net recharge rate of 5 per cent of average annual rainfall of 1,651 mm for the Newports Creek (27.2 km^2) and Coffs Creek (25.4 km^2) catchments. The assumption of 5 per cent recharge is applied at the catchment scale and represents an assumed net recharge across all recharge zones, including the locally high recharge zones located along the ridge lines and the low recharge zones located downgradient of the tunnel.

The potential reduction in annual recharge (assuming inflows to the tunnel were not subsequently redirected to the aquifer) at the catchment scale would be less than 0.1 per cent.

7.3 Groundwater Use

The constraints mapping identified three water supply bores near Roberts Hill.

The drawdown assessment indicates that the Roberts Hill Tunnel is unlikely to have a significant impact on the identified water supply bores.

There is a banana plantation on the northern side of the ridge within the drawdown cone of the tunnel. It is not known to what extent the banana plantation is dependent on groundwater.

7.4 Potential Groundwater Dependent Ecosystems

The location of potential GDEs identified by Arup were provided as an ESRI shapefile.

The Roberts Hill Tunnel is not expected to impact on these potential GDEs based on the predicted drawdown.

7.5 Surface Water

There are three agricultural dams on the downgradient side of the tunnel. All are at about 28 m RL and appear to be surface expressions of the water table.

The Northern feature is outside the expected zone of influence for the tunnel.

The eastern feature is marginally outside the predicted drawdown extent. There is, however, potential for change due to drawdown, particularly under circumstances where structural features allow larger groundwater inflows to the tunnel.

The southern agricultural dam is likely to be impacted by drawdown. The drawdown may impose reductions of overland flow from breaks in slope where the water table intersects the ground surface. The potential drawdown has been shown schematically on Figure 12. Figure 6 shows that there is potential for the Roberts Hill Tunnel to capture up to 50 per cent of the groundwater catchment contributing to the agricultural dam.

No surface water features have been identified within the catchment on the upstream side.

8 Shephards Lane Tunnel Predictive Change Assessments

The following is a semi-quantitative appraisal of the potentially altered groundwater environment associated with constructing and operating the Shephards Lane Tunnel. Note that potential for changes to groundwater quality were not assessed.

8.1 Groundwater Inflows and Drawdown

Steady-state groundwater inflow rates are summarised in Table 13. Transient groundwater inflow rates for the drained tunnel are shown in Figure 26a. Steady-state is reached within about 4 years.

Drawdown contours after 18 months are shown in Figure 26b and Figure 26c, in cross-section and plan view, respectively.

Drawdown contours at steady-state are shown in Figure 26d and Figure 26e, in cross-section and plan view, respectively.

Also shown on Figure 26c and 26e are the drawdowns for the low recharge sensitivity analysis.

Table 13 – Shephards Lane Steady-State Drained Tunnel Inflows

Section	Base Case		
	Groundwater Inflow (kL/day)	Length (m)	Groundwater Inflow (L/s/100 m)
Combined	22.1	417	0.06
Tunnel	18.9	305	0.07
Western Portal	0.6	56	0.01
Eastern Portal	2.6	56	0.05

8.2 Water Balance

The steady-state passive groundwater take for the Shephards Lane Tunnel and portals was estimated at approximately 8 ML/year. Total annual recharge would be approximately 2 GL, assuming a net recharge rate of 5 per cent of average annual rainfall of 1,651 mm for the Coffs Creek catchment (25.4 km^2). The assumption of 5 per cent recharge is applied at the catchment scale and represents an assumed net recharge across all recharge zones, including the locally high recharge zones located along the ridge lines and the low recharge zones located downgradient of the tunnel.

The potential reduction in annual recharge (assuming inflows to the tunnel were not subsequently redirected to the aquifer) at the catchment scale would be less than 0.5 per cent.

8.3 Groundwater Use

The constraints mapping identified several water supply bores near Shephards Lane.

The drawdown assessment indicates that the Shephards Lane Tunnel may have a minor impact on at least two of the identified groundwater users, Figure 26e.

The ridge hosts a banana plantation within the drawdown cone of the tunnel. It is not known to what extent the plantation is dependent on groundwater.

8.4 Potential Groundwater Dependent Ecosystems

The location of potential GDEs identified by Arup were provided as an ESRI shapefile.

The Shephards Lane Tunnel is expected to impact on at least one potential GDE based on the predicted drawdown. The presence of this GDE needs to be validated.

8.5 Surface Water

A small pond has been identified north of Shephards Lane at approximately 140 m RL. This pond is within the extent of the predicted drawdown, so it has potential to be impacted. The position of the pond in the landscape, however, indicates it is potentially part of a perched system which would not be impacted by drawdown, Figure 13. Furthermore, flow would continue to pass through this location after capture by the tunnel and limited change to the pond's capture zone is expected.

A site reconnaissance of potential additional ponds and shallow groundwater discharge zones is required.

The drawdown may impose reductions of overland flow from breaks in slope where the water table intersects the ground surface.

9 Gatelys Road Tunnel Predictive Change Assessments

The following is a semi-quantitative appraisal of the potentially altered groundwater environment associated with constructing and operating the Gatelys Road Tunnel. Note that potential for changes to groundwater quality were not assessed.

9.1 Groundwater Inflows and Drawdown

Steady-state groundwater inflow rates are summarised in Table 14. Transient groundwater inflow rates for the drained tunnel are shown in Figure 27a. Steady-state is reached within a period of about 3 to 4 years.

Drawdown contours after 18 months are shown in Figure 27b and Figure 27c, in cross-section and plan-view, respectively.

Drawdown contours at steady-state are shown in Figure 27d and Figure 27e, in cross-section and plan view, respectively.

Also shown on Figure 27c and 27e are the drawdowns for the low recharge sensitivity analysis.

Table 14 – Gatelys Road Steady-State Drained Tunnel Inflows

Section	Base Case		
	Groundwater Inflow (kL/day)	Length (m)	Groundwater Inflow (L/s/100 m)
Combined	55	482	0.13
Tunnel	55.3	420	0.15
Eastern Portal	0	36	0
Western Portal	0.7	36	0.02

9.2 Water Balance

The steady-state passive groundwater take for the Gatelys Road Tunnel and portals was estimated at approximately 20 ML/year. Total annual recharge would be approximately 2.4 GL, assuming a net recharge rate of 5 per cent of average annual rainfall of 1,651 mm for the Coffs Creek (25.4 km^2) and Jordans Creek (4.2 km^2) catchments. The assumption of 5 per cent recharge is applied at the catchment scale and represents an assumed net recharge across all recharge zones, including the locally high recharge zones located along the ridge lines and the low recharge zones located downgradient of the tunnel.

The potential reduction in annual recharge (assuming inflows to the tunnel were not subsequently redirected to the aquifer) at the catchment scale would be less than 1 per cent.

9.3 Groundwater use

The constraints mapping identified numerous water supply bores near Gatelys Road. The drawdown assessment indicates that the Gatelys Road Tunnel may have an impact on at least 8 of the identified groundwater users as shown in Figure 27e. The potential impacts have also been shown schematically for some of these bores on Figures 14 and 15.

There is a banana plantation and blueberry farm on the eastern side of the ridge within the drawdown cone of the tunnel. It is not known to what extent the banana plantation and blueberry farm are dependent on groundwater.

9.4 Potential groundwater dependent ecosystems

The location of potential GDEs identified by Arup were provided as an ESRI shapefile.

The nearest mapped GDE is approximately 1.5 km from the Gatelys Road Tunnel. Predicted drawdown from the Gatelys Road Tunnel does not impact on the potential GDE.

9.5 Surface water

There are at least three local ponds that are likely to be impacted based on the predicted drawdown. The potential drawdown impacts are shown schematically on Figures 14 and 15. Figure 8 shows that there is potential for the Gatelys Road Tunnel to capture a proportion of the groundwater catchment contributing to each pond. Clockwise from the pond to the east the proportions of catchment capture are estimated to be 20, 60 and 70 per cent.

The drawdown may also impose reductions of overland flow from breaks in slope where the water table intersects the ground surface.

There is potential for greater impacts than shown by the modelling under circumstances where structural features allow larger groundwater inflows to the tunnel.

10 Recommendations

The following recommendations are made to improve the conceptual hydrogeological model and hence increase confidence in the model outcomes:

1. A site visit by a hydrogeologist to improve context to the modelling assumptions regarding anisotropy, orientation of discrete flow paths, soil properties and storage. The site visit should include:
   a. Observations of the residual soils, colluvium and alluvium
   b. Infiltration rate testing using a double-ring infiltrometer. This will inform the actual rainfall recharge potentials and provide a basis for the assumed hydraulic conductivity values
   c. Characterisation of groundwater discharge zones and estimation of discharge rates
   d. Characterisation of sensitive receptors including agricultural dams and ponds.
2. Ongoing groundwater monitoring and measurement. The monitoring records are important to characterise system responses to rainfall and dry periods, enabling an improved understanding of the flow system dynamics. In this regard:
   a. Loggers should be installed at Shephards Lane (BHH140) and Gatelys Road (BHH150 and BHH153) to allow interpretation of the influence of topography and structure and, lateral flow components in the weathering profile.
3. Dependent on potential risk, additional monitoring wells may be beneficial to define baseline conditions near identified sensitive environmental receptors including agricultural dams and springs.
4. There is limited packer testing data available for Roberts Hill. Hydraulic testing (packer testing or slug testing) should be undertaken near the structures that may be intersected by the tunnel. Larger-scale pumping tests are not envisioned.
5. Samples should be collected from the slotted standpipe piezometers at each of the tunnel sites presented in Tables B4-1, B4-2 and B4-3 and analysed for major ions as well as the parameters outlined in Table 5 to ensure repeatability of results and provide an appropriate representation of baseline groundwater quality prior to construction. To improve the spatial coverage of the baseline monitoring program consideration should be given to sampling at BHH142 (Shephards Lane) and BHH150 (Gatelys Road).
6. Sampling and filtering procedures should be reviewed to ensure reliability and repeatability of the results. It is important that the analyses represent groundwater, not suspended particulates or sediment. For metals analyses, it is essential that the samples are filtered and clear.
7. Depending on perceived groundwater inflow and drawdown risk, higher order analyses could be undertaken:
   a. A fully 3D model would enable an improved representation of most groundwater flow paths and transient calibration
   b. A fully 3D model will allow the effects of preferred structural flow paths and barriers to flow to be accounted for directly in the model
   c. A fully 3D model will allow for the effect of anisotropy on drawdown patterns to be more closely represented.
   d. If the identified environmental constraints are of value, then cumulative drawdown effects of the tunnel and neighbouring cuttings should be assessed which would require a 3D model and extension of the model domain.

For and on behalf of
PELLS SULLIVAN MEYNINK

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SENIOR HYDROGEOLOGIST

IAN BRUNNER
PRINCIPAL HYDROGEOLOGIST

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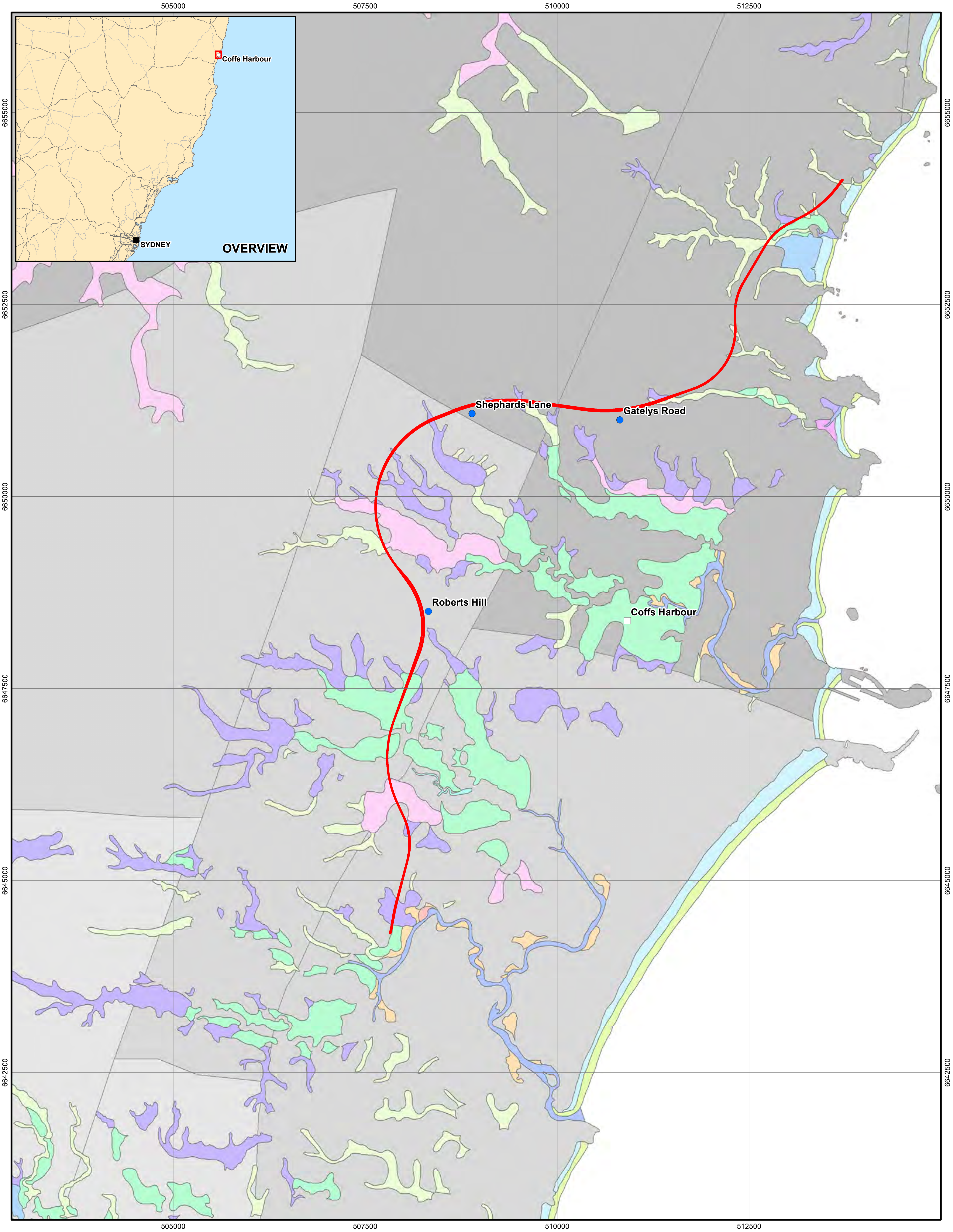

Soil Units

- QH_bd, Coastal deposits- dune facies
- QH_bf, Coastal deposits- backbarrier flat facies
- Q_bb, Coastal deposits- beach facies
- QH_ebw, Estuarine basin and bay
- QH_ecw, Estuarine channel deposits
- QH_es, Estuarine swamp
- QP_at, Alluvial terrace deposits
- Q_af, Alluvial floodplain deposits
- Q_ap, Alluvial palaeochannel deposits
- Q_av, Alluvial valley deposits
- Q_avf, Alluvial fan deposits
- CZ_ath, Alluvial terrace deposits

Rock Units

- Ccoc, Coramba Beds
- Ccom, Moombil Beds
- Ccor, Brooklana Beds
- Tunnel Location
- CHB Alignment

N

Scale 1:45,000

0 0.25 0.5 1 1.5 2 2.5 Km

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

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LOCATION PLAN

Created By:	Revision:
D.L.	A
Date: 23 Apr 2019	Paper Size: A3

PSM2876-057R | Figure 1

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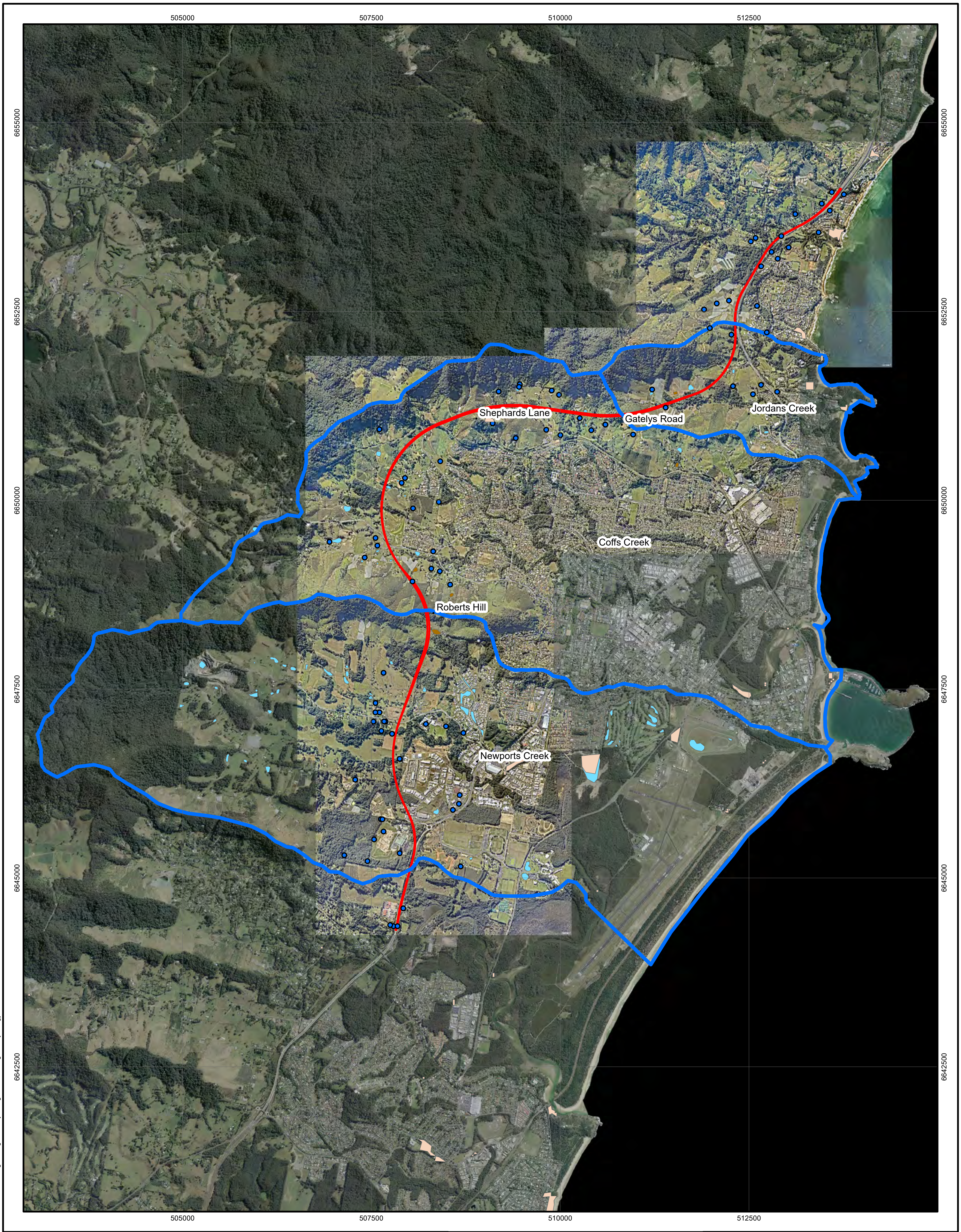

- Groundwater Bores (NOW)
- Catchment Boundaries
- Dam
- Pond
- GDE
- CHB Alignment

N

Scale 1:45,000

0 0.25 0.5 1 1.5 2 2.5 Km

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

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PROJECT ALIGNMENT AND REGIONAL HYDROLOGY

PSM2876-057R | Figure 2

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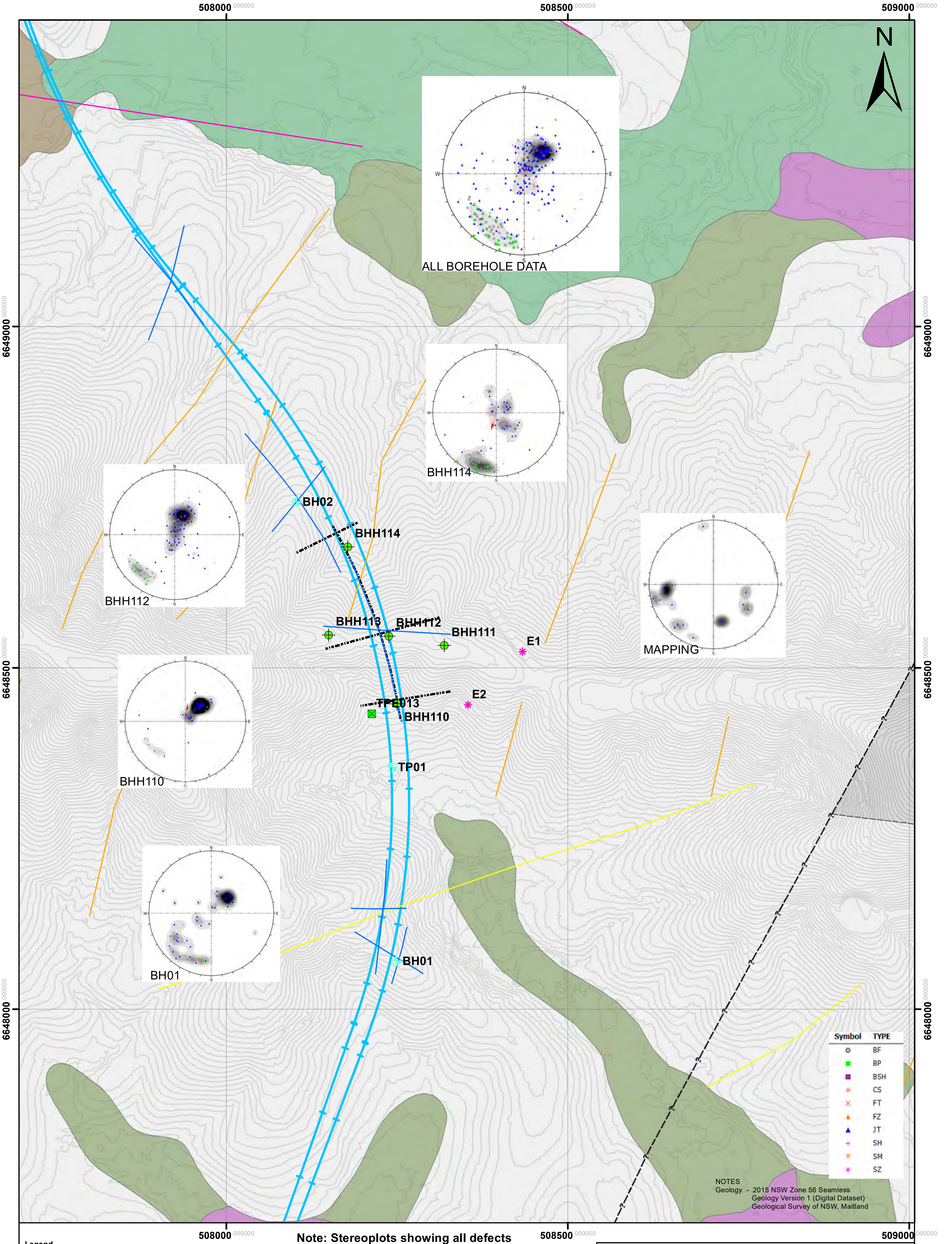

Note: Stereoplots showing all defects

Legend

- Borehole (RCA 2017)
- Test Pit (RCA 2017)
- Mapping (RCA 2017)
- Seismic lines (RCA 2017)
- Geotechnical Section
- ENE WSW Lineaments
- NNE SSW Lineaments
- ESE WNW Lineaments
- 2019 New Alignment
- Fault, inferred
- Geological boundary, position approximate
- QP_at - Alluvial terrace deposits
- Q_af - Alluvial floodplain deposits
- Q_av - Alluvial valley deposits
- Q_avf - Alluvial fan deposits
- Ccoc - Coramba beds
- Ccor - Brooklana beds
- 2m Contours

Scale

0 50 100 200 300

Meters

Pells Sullivan Meynink

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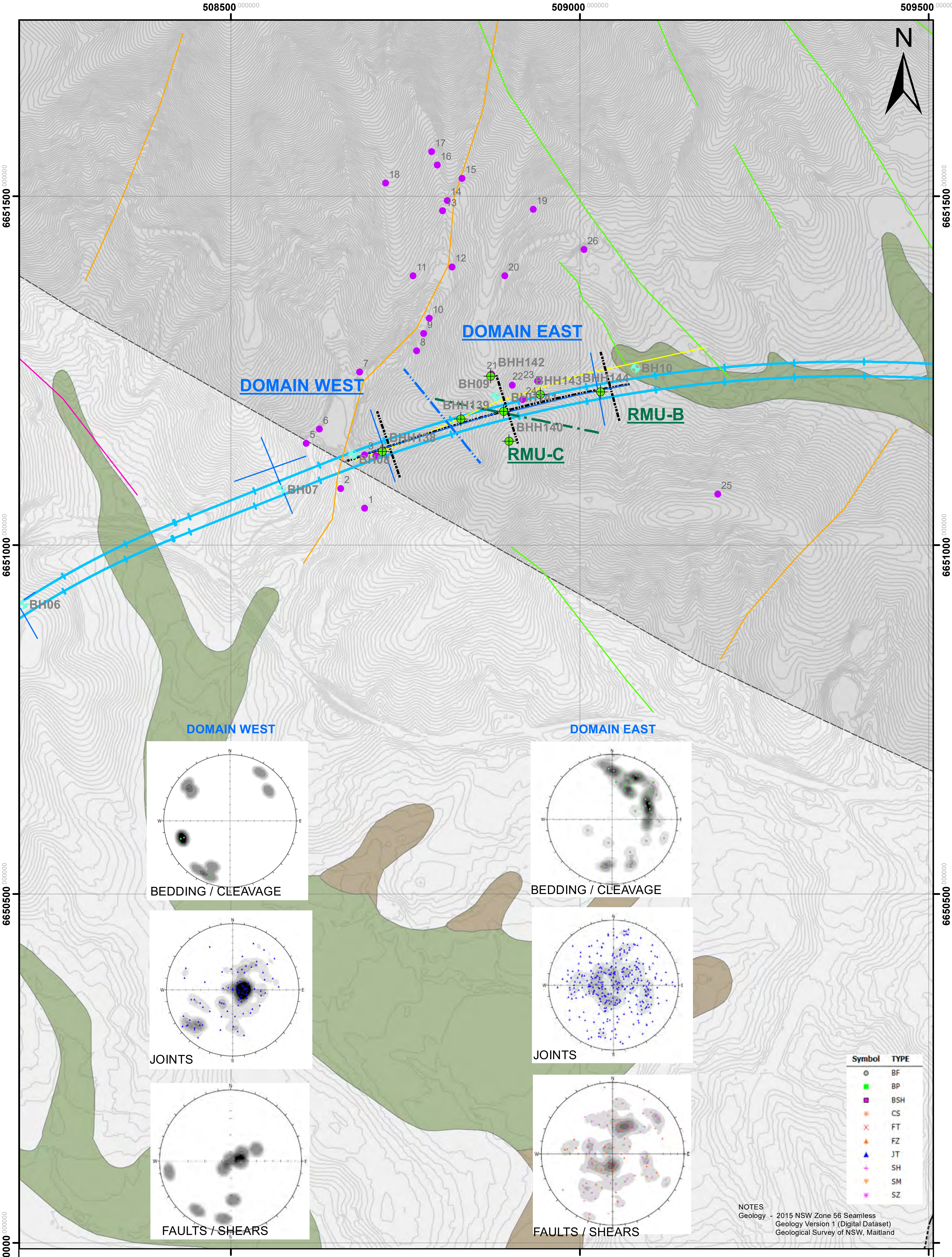
SITE INVESTIGATION PLAN
ROBERTS HILL TUNNEL

PSM2876-057R	Figure 3



Legend

- Borehole (RCA 2017)
- Mapping (RCA 2017, D10)
- Seismic lines (RCA 2017)
- Structural Domain Boundary
- Rock Mass Unit Boundary
- Geotechnical Section
- ENE WSW Lineaments
- NNE SSW Lineaments
- NW SE Lineaments
- ESE WNW Lineaments
- 2019 New Alignment
- Fault, concealed
- Fault, inferred
- Geological boundary, position approximate
- Q_av - Alluvial valley deposits
- Q_avf - Alluvial fan deposits
- Ccoc - Coramba beds
- Ccor - Brooklana beds
- 2m Contours

Scale

0 50 100 200 300

Meters

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Coffs Harbour Bypass Project
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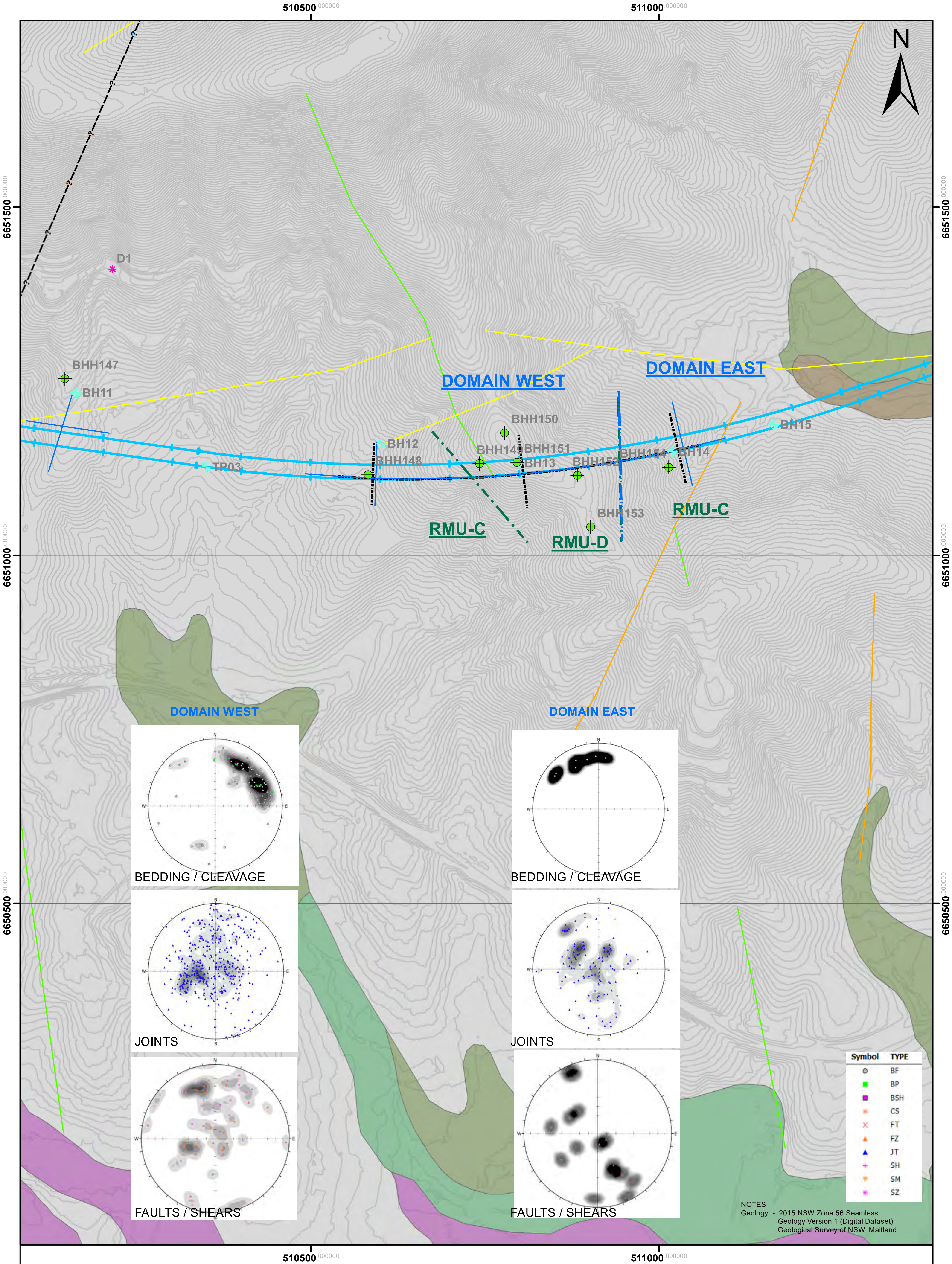
SITE INVESTIGATION PLAN
SHEPHARDS LANE TUNNEL

PSM2876-057R	Figure 4



Legend

- Borehole (RCA 2017)
- Mapping (RCA 2017)
- Seismic lines (RCA 2017)
- Structural Domain Boundary
- Rock Mass Unit Boundary
- Geotechnical Section
- ENE WSW Lineaments
- NNE SSW Lineaments
- NW SE Lineaments
- 2019 New Alignment
- Fault, inferred
- QP_at - Alluvial terrace deposits
- Q_af - Alluvial floodplain deposits
- Q_av - Alluvial valley deposits
- Q_avf - Alluvial fan deposits
- Ccoc - Coramba beds
- 2m Contours

Scale

0 50 100 200 300

Meters

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Coffs Harbour Bypass Project
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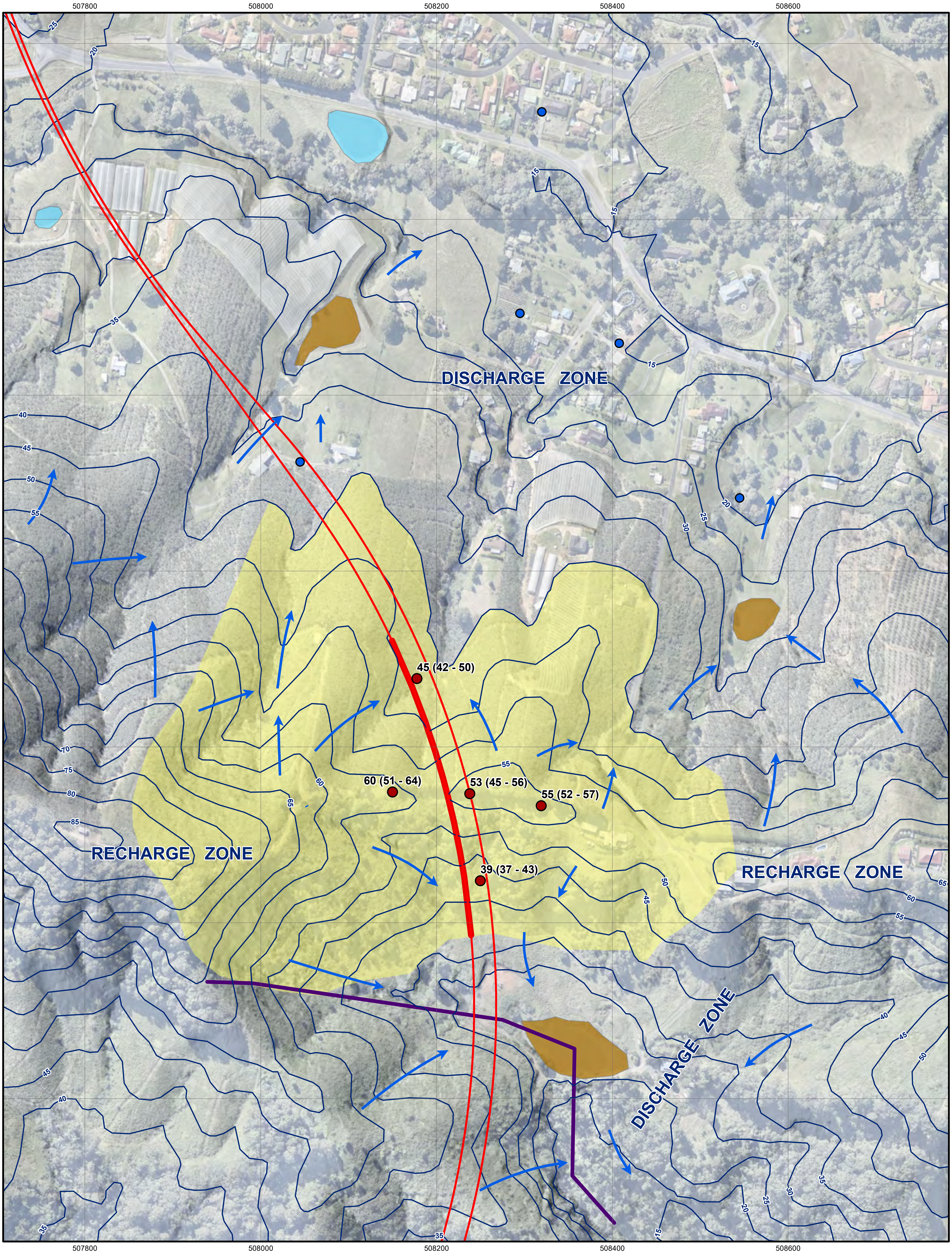
SITE INVESTIGATION PLAN
GATELYS ROAD TUNNEL

PSM2876-057R	Figure 5



Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 6 Roberts Hill Groundwater levels and Flow Paths.mxd

- Groundwater Bores NOW
- Groundwater Level (m RL) (Av (Min - Max))
- Baseline Groundwater Flow Direction
- Groundwater Level Contours
- Dam
- Pond
- Conceptual Capture Zone
- CHB Alignment
- Conceptual Model Cross-Section
- Conceptual Model Long-Section

N

Scale 1:4,000

0 25 50 100 150 200 m

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

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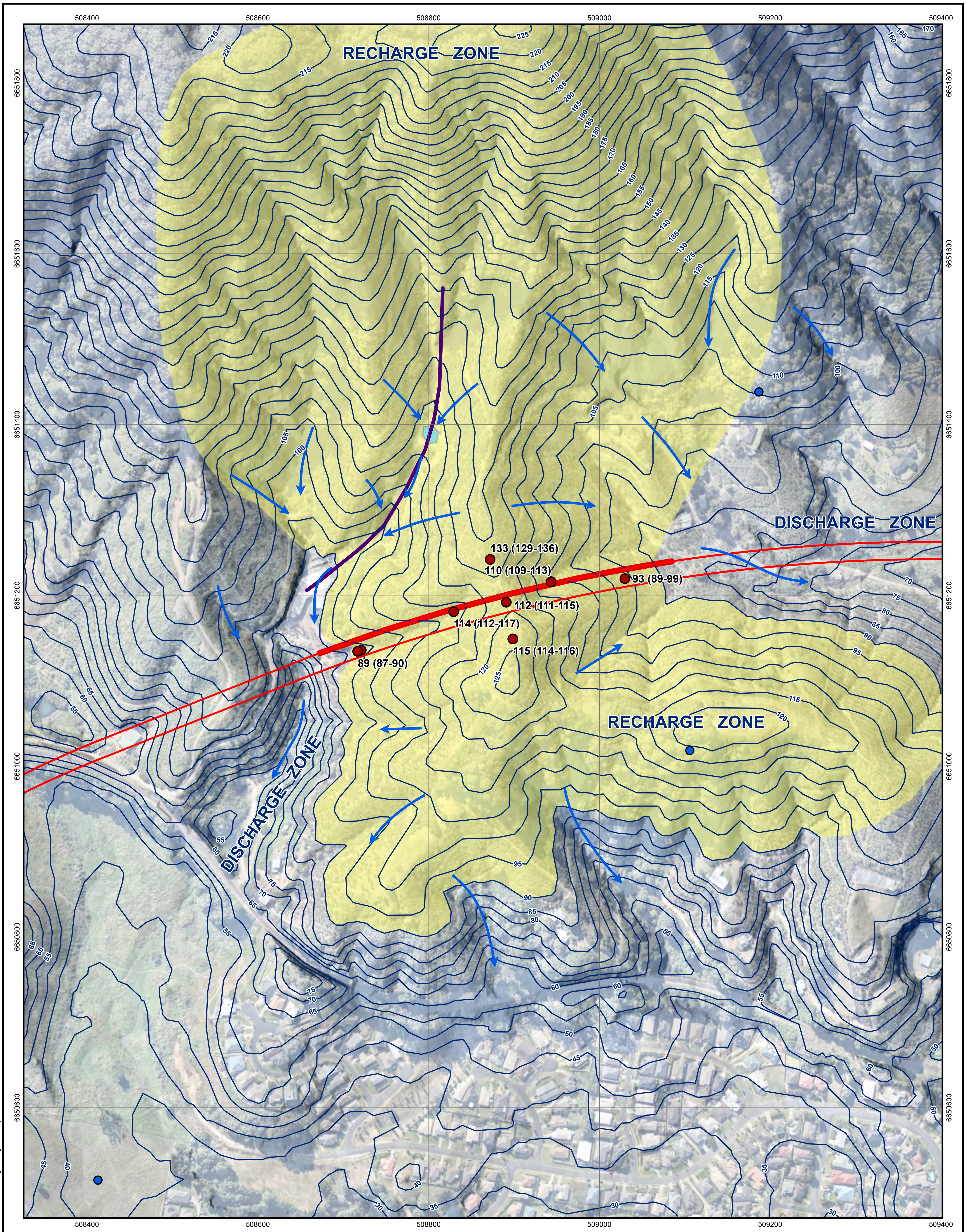
ROBERTS HILL
GROUNDWATER LEVELS AND
FLOW PATHS

PSM2876-057R | Figure 6



- Groundwater Bores NOW
- Groundwater Level (m RL) (Av (Min - Max))
- Baseline Groundwater Flow Direction
- Groundwater Level Contours
- Dam
- Pond
- Conceptual Capture Zone
- CHB Alignment
- Conceptual Model Cross-Section
- Conceptual Model Long-Section

N

Scale 1:4,000

0 25 50 100 150 200 m

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

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SHEPHARDS LANE
GROUNDWATER LEVELS AND
FLOW PATHS

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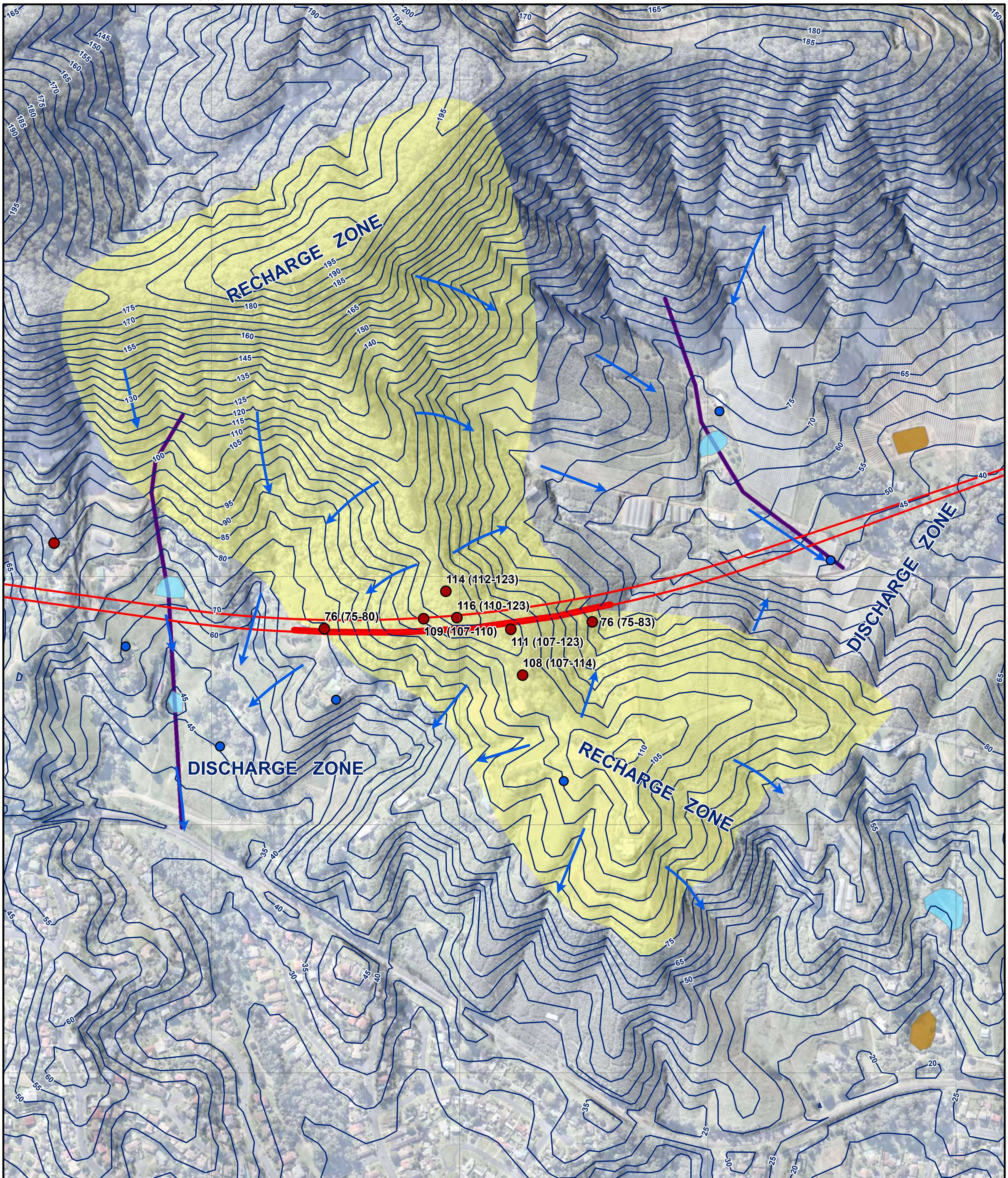

- Groundwater Bores NOW
- Groundwater Level (m RL) (Av (Min - Max))
- Baseline Groundwater Flow Direction
- Groundwater Level Contours
- Dam
- Pond
- Conceptual Capture Zone
- CHB Alignment
- Conceptual Model Cross-Section
- Conceptual Model Long-Section

N

Scale 1:6,000

0 50 100 200 300 m

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

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GATELYS ROAD
GROUNDWATER LEVELS AND
FLOW PATHS

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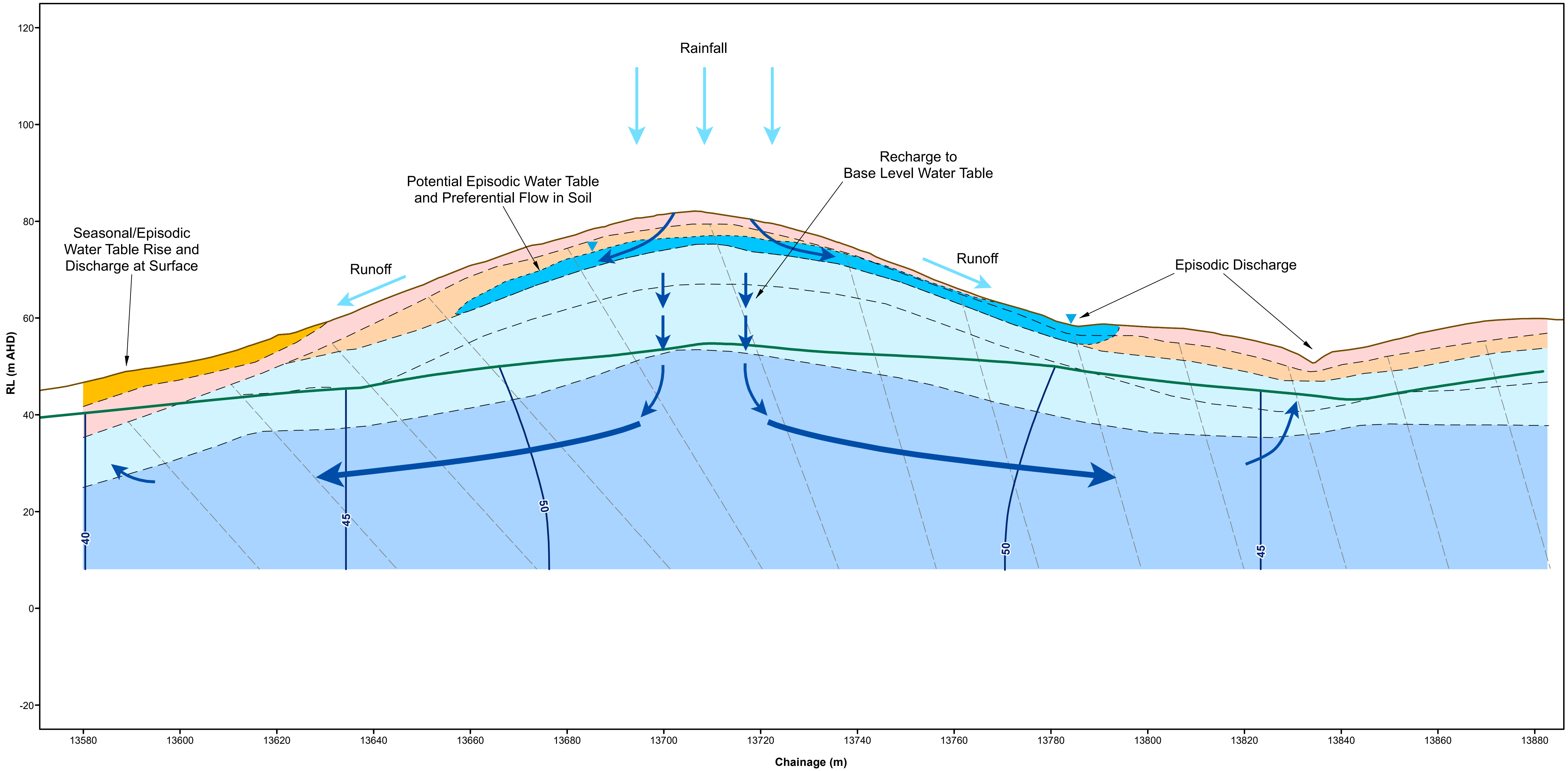
Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 8 Gatelys Road Groundwater levels and Flow Paths.mxd



Water Table (Pre Development)
Groundwater Contours
Ground Surface
Geological Boundaries
Cleavage Fabric

Geology

Colluvium
Residual Soil
RMU A - EW/HW Rock
RMU B1 - Closely spaced Argillite (Clevage Dominated)
RMU B2 - Widely spaced Argillite (Clevage Dominated)
RMU C1 - Closely spaced Argillite and Siliceous Argillite
RMU C2 - Widely spaced Argillite and Siliceous Argillite
RMU D1 - Closely spaced Argillite with thick beds of Siliceous Argillite
RMU D2 - Widely spaced Argillite with thick beds of Siliceous Argillite

Indicative Flow Vector

0 5 10 20 30 40 50 m

Horizontal and Vertical Scale

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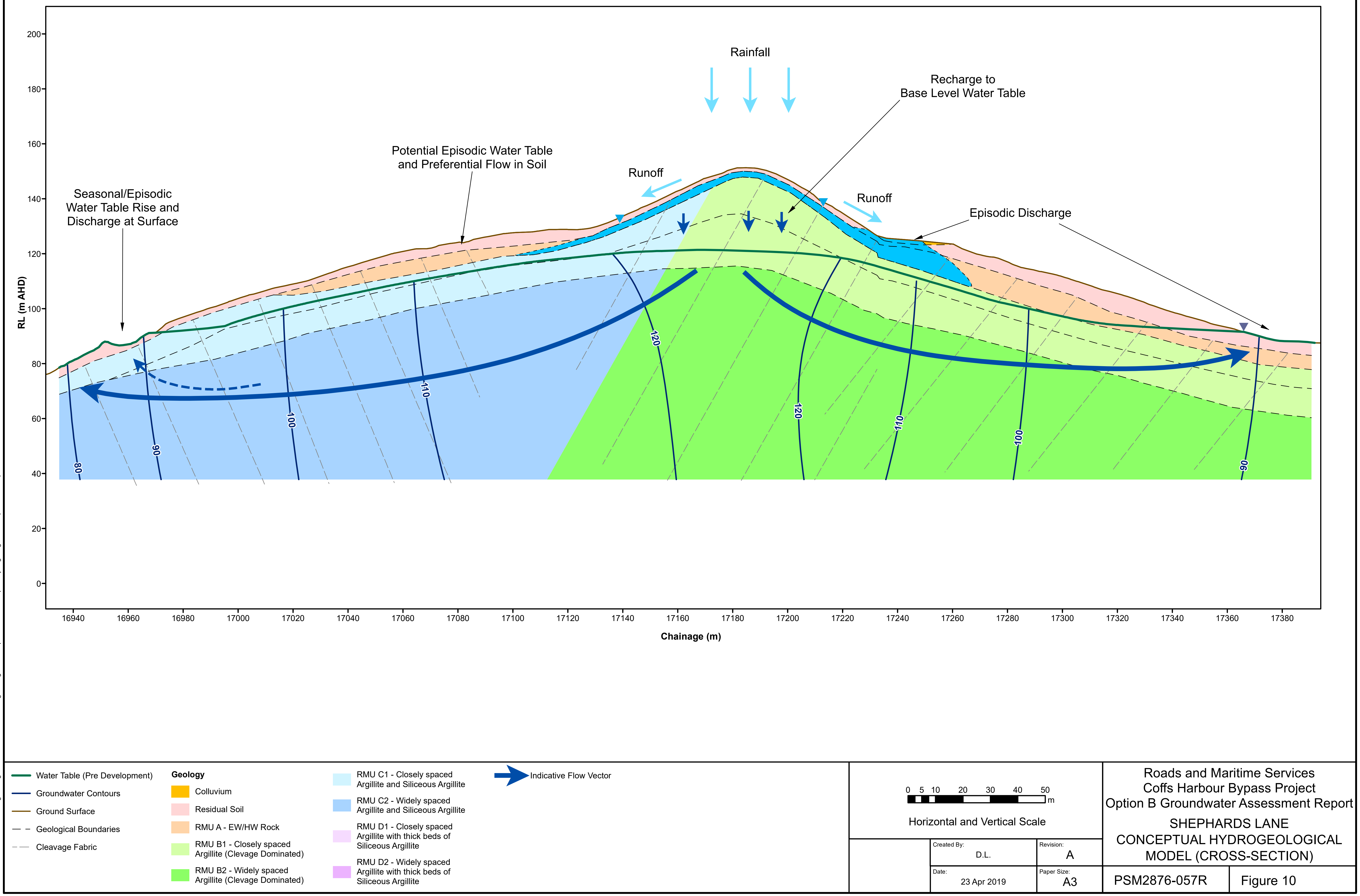
ROBERTS HILL
CONCEPTUAL HYDROGEOLOGICAL
MODEL (CROSS-SECTION)

PSM2876-057R | Figure 9

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 9 Roberts Hill Conceptual Hydrogeological Model (Cross-section).mxd



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SHEPHARDS LANE CONCEPTUAL HYDROGEOLOGICAL MODEL (CROSS-SECTION)

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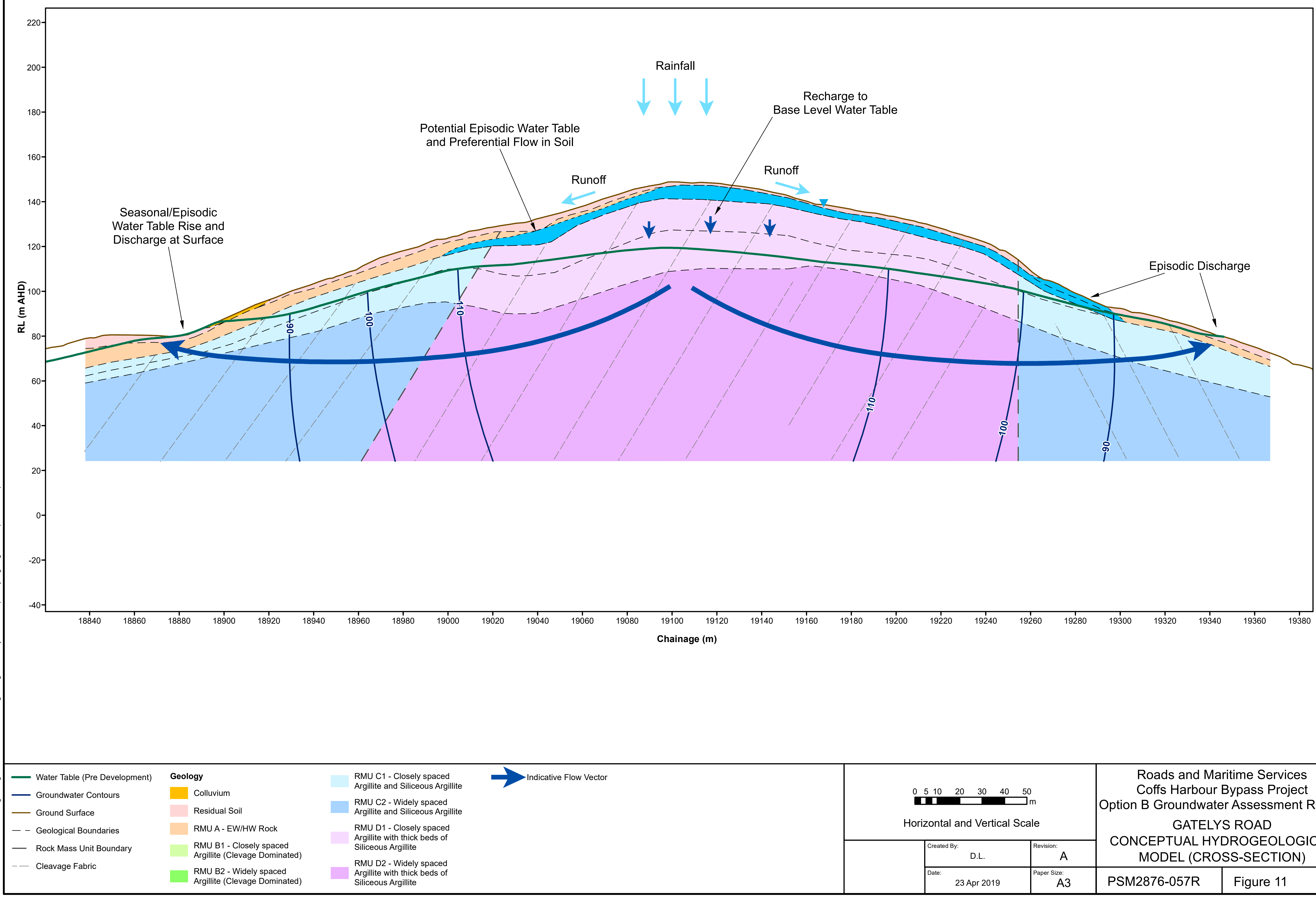
PSM2876-057R	Figure 10

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 10 Shephards Lane Conceptual Hydrogeological Model (Cross-section).mxd



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GATELYS ROAD
CONCEPTUAL HYDROGEOLOGICAL MODEL (CROSS-SECTION)

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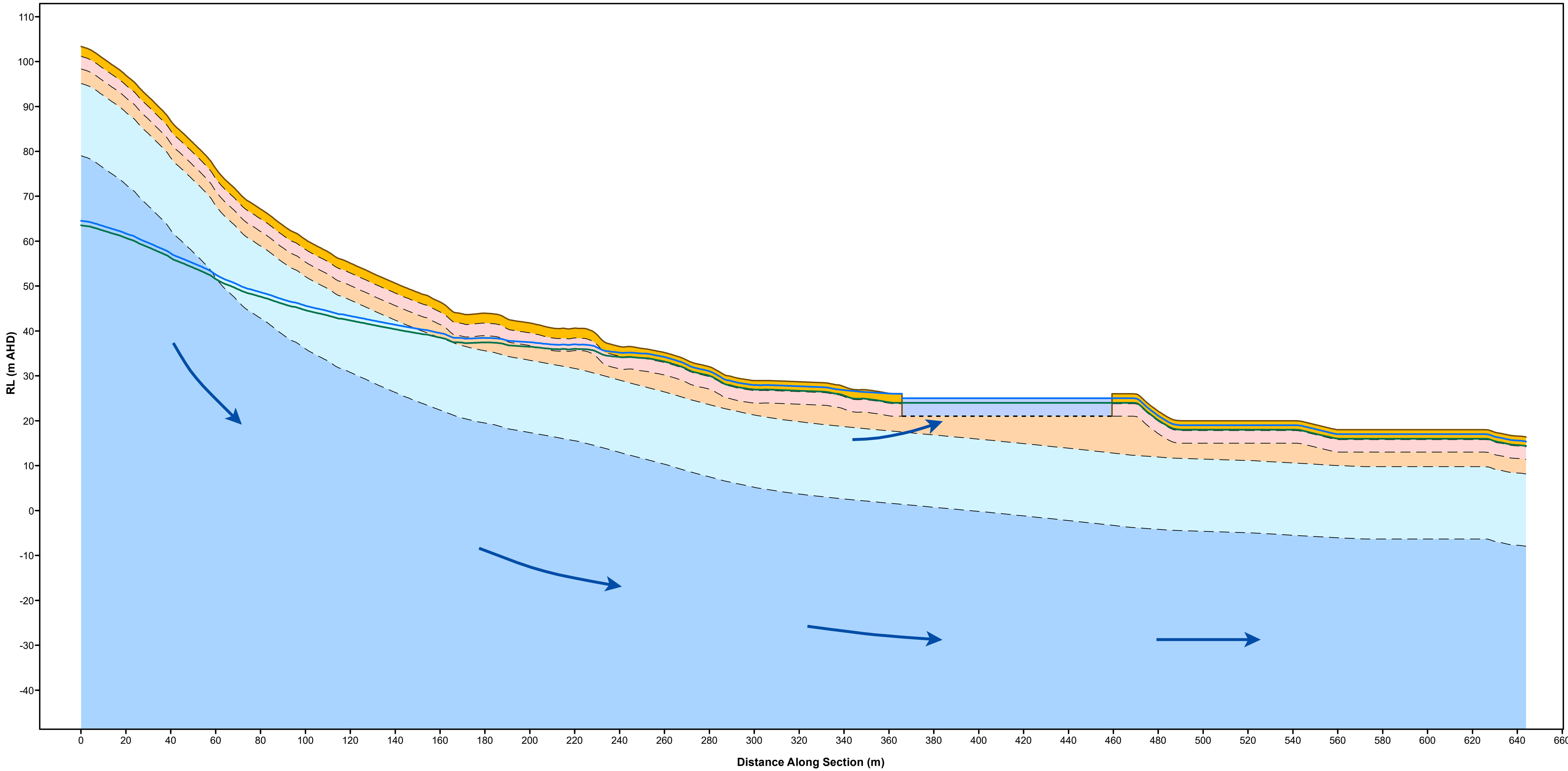
PSM2876-057R | Figure 11

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 11 Gatelys Road Conceptual Hydrogeological Model (Cross-section).mxd



- Baseline Water Table
- Post Development Water Table
- Depth Unknown
- Indicative Flow Vector
- Dam
- Geological Boundaries

Geology

- Colluvium
- Residual Soil
- RMU A - EW/HW Rock
- RMU C1 - Closely spaced Argillite and Siliceous Argillite
- RMU C2 - Widely spaced Argillite and Siliceous Argillite

NOTE: Indicative only. Actual depth to watertable unknown.

0 10 20 40 60 80 100 m

Horizontal Scale
2 x Vertical Exaggeration

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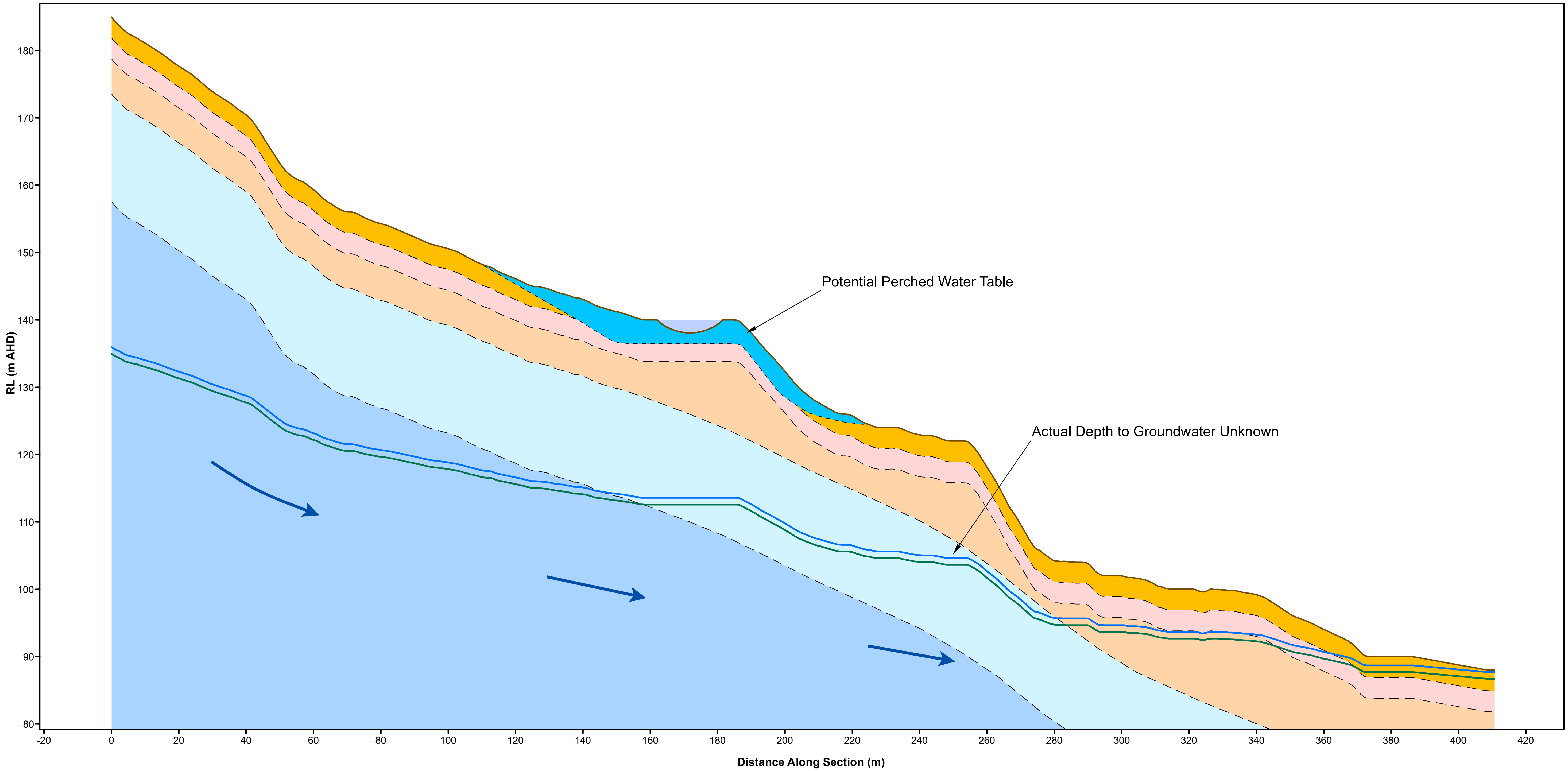
ROBERTS HILL
CONCEPTUAL HYDROGEOLOGICAL
MODEL (LONG SECTION)

PSM2876-057R
Figure 12



Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 13 Shephards Lane Conceptual Hydrogeological Model (Long Section).mxd

Baseline Water Table
Post Development Water Table
Potential Perched Water Table
Indicative Flow Vector
Pond
Geological Boundaries

Geology

Colluvium
Residual Soil
RMU A - EW/HW Rock
RMU C1 - Closely spaced Argillite and Siliceous Argillite
RMU C2 - Widely spaced Argillite and Siliceous Argillite

NOTE: Indicative only. Actual depth to watertable unknown.

0 5 10 20 30 40 50 m
Horizontal Scale
2 x Vertical Exaggeration

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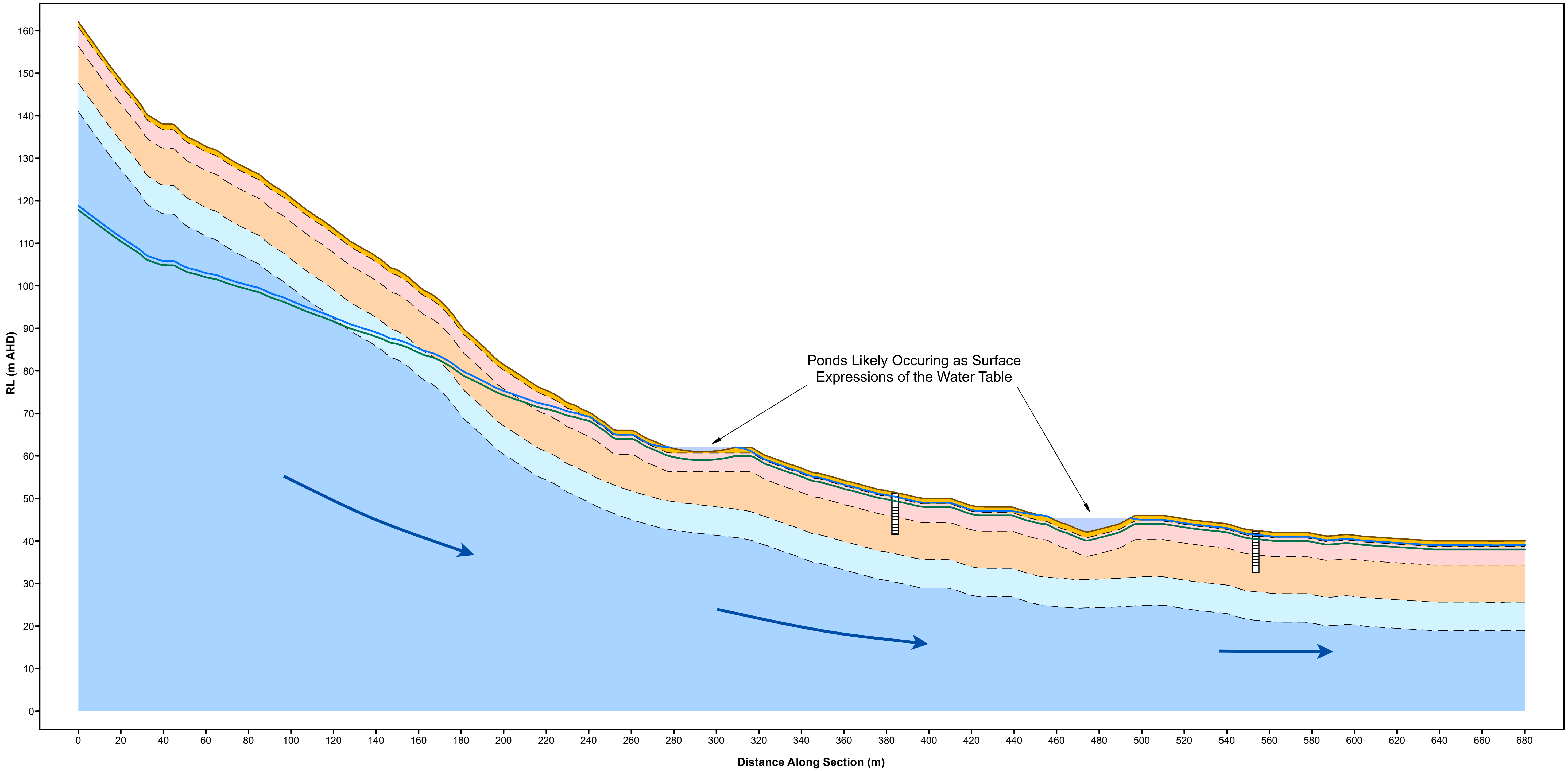
Coffs Harbour Bypass Project
Option B Groundwater Assessment Report

SHEPHARDS LANE
CONCEPTUAL HYDROGEOLOGICAL
MODEL (LONG SECTION)

PSM2876-057R

Figure 13



Baseline Water Table
Post Development Water Table
Well (Depth Unknown)
Indicative Flow Vector
Pond
Geological Boundaries

Geology

Colluvium
Residual Soil
RMU A - EW/HW Rock
RMU C1 - Closely spaced Argillite and Siliceous Argillite
RMU C2 - Widely spaced Argillite and Siliceous Argillite

NOTE: Indicative only. Actual depth to watertable unknown.

0 10 20 40 60 80 100 m
Horizontal Scale
2 x Vertical Exaggeration

Created By: D.L.
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Date: 23 Apr 2019
Paper Size: A3

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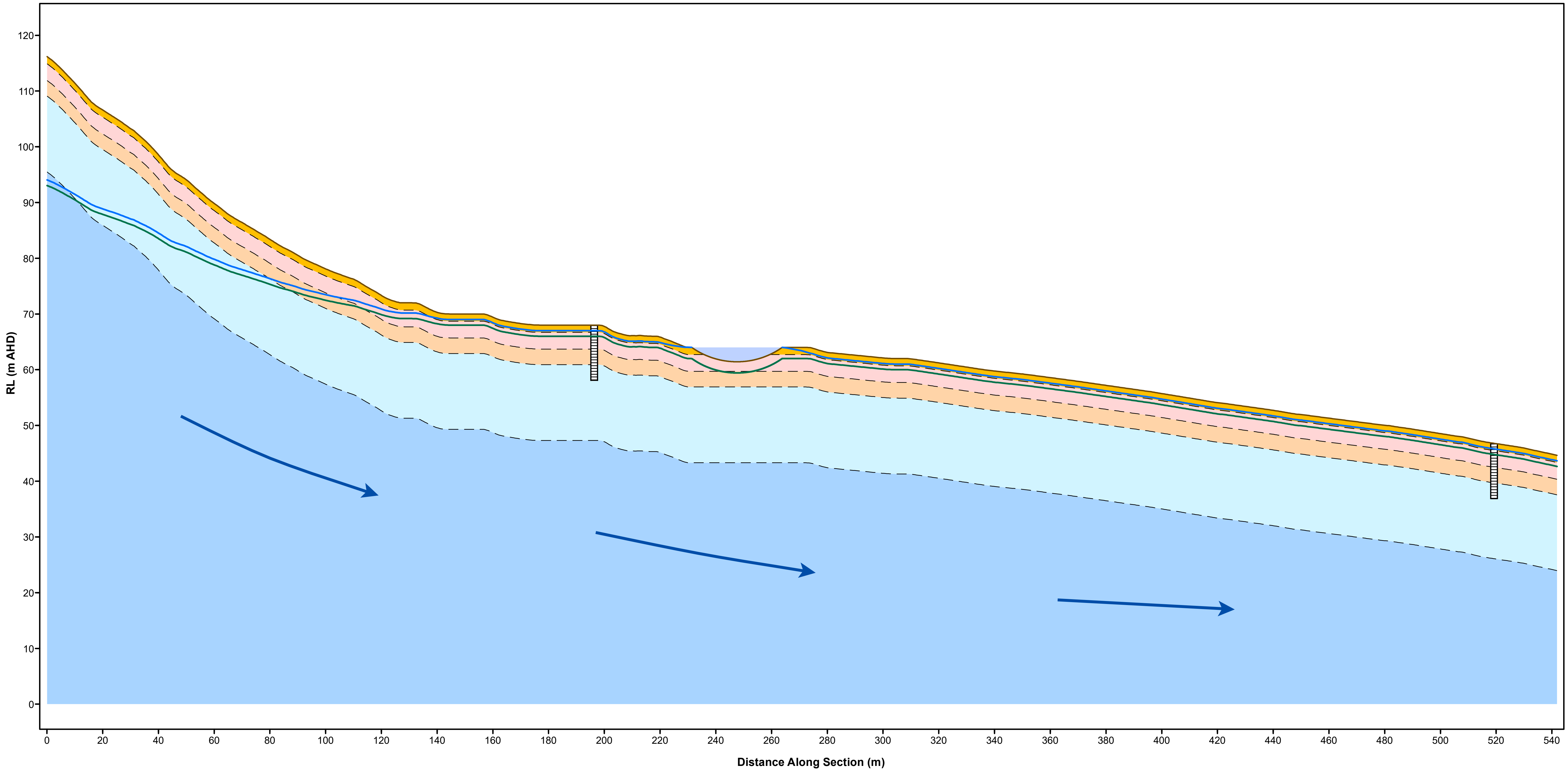
Option B Groundwater Assessment Report

GATELYS ROAD
CONCEPTUAL HYDROGEOLOGICAL
MODEL (LONG SECTION WEST)

PSM2876-057R
Figure 14

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 14 Gatelys Road Conceptual Hydrogeological Model (Long Section West).mxd



Baseline Water Table

Post Development Water Table

Well (Depth Unknown)

Indicative Flow Vector

Pond

Geological Boundaries

Geology

Colluvium

Residual Soil

RMU A - EW/HW Rock

RMU C1 - Closely spaced Argillite and Siliceous Argillite

RMU C2 - Widely spaced Argillite and Siliceous Argillite

NOTE: Indicative only. Actual depth to watertable unknown.

0 10 20 40 60 80 m

Horizontal Scale

2 x Vertical Exaggeration

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Roads and Maritime Services
Coffs Harbour Bypass Project
Option B Groundwater Assessment Report

GATELYS ROAD
CONCEPTUAL HYDROGEOLOGICAL MODEL (LONG SECTION EAST)

PSM2876-057R Figure 15

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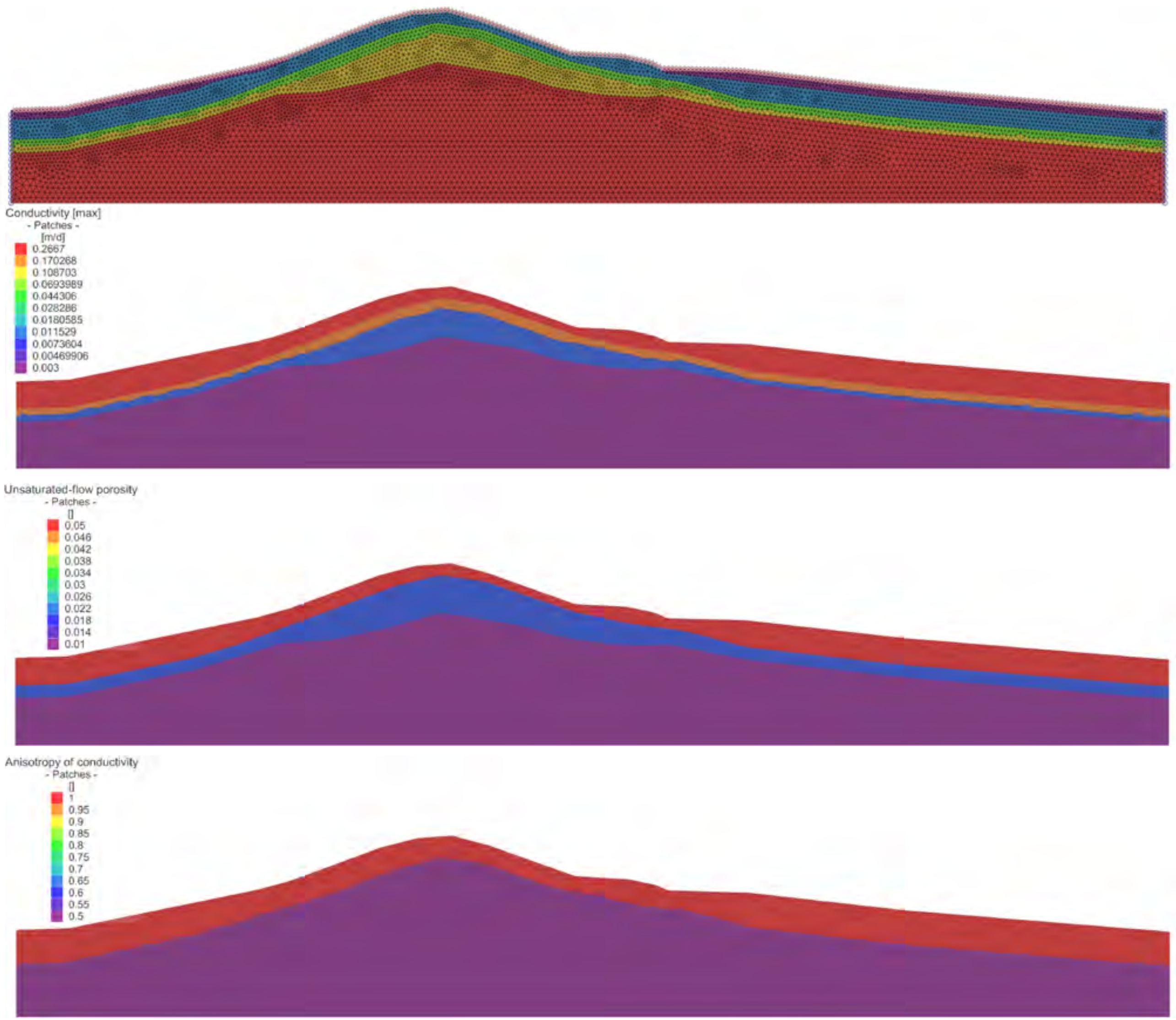

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ROBERTS HILL MODEL SETUP

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PSM2876-057R Figure 16

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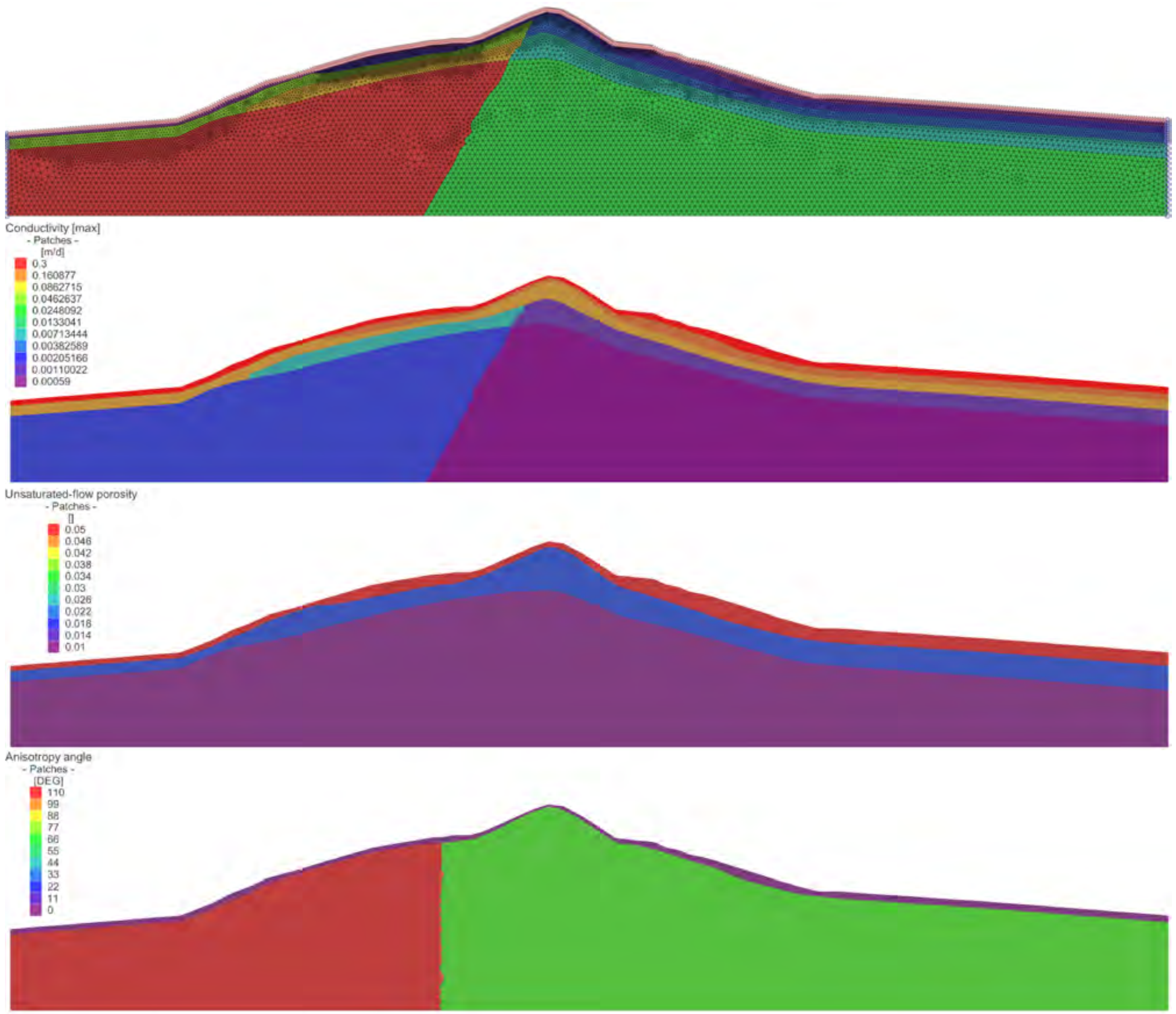

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SHEPHARDS LANE MODEL SETUP

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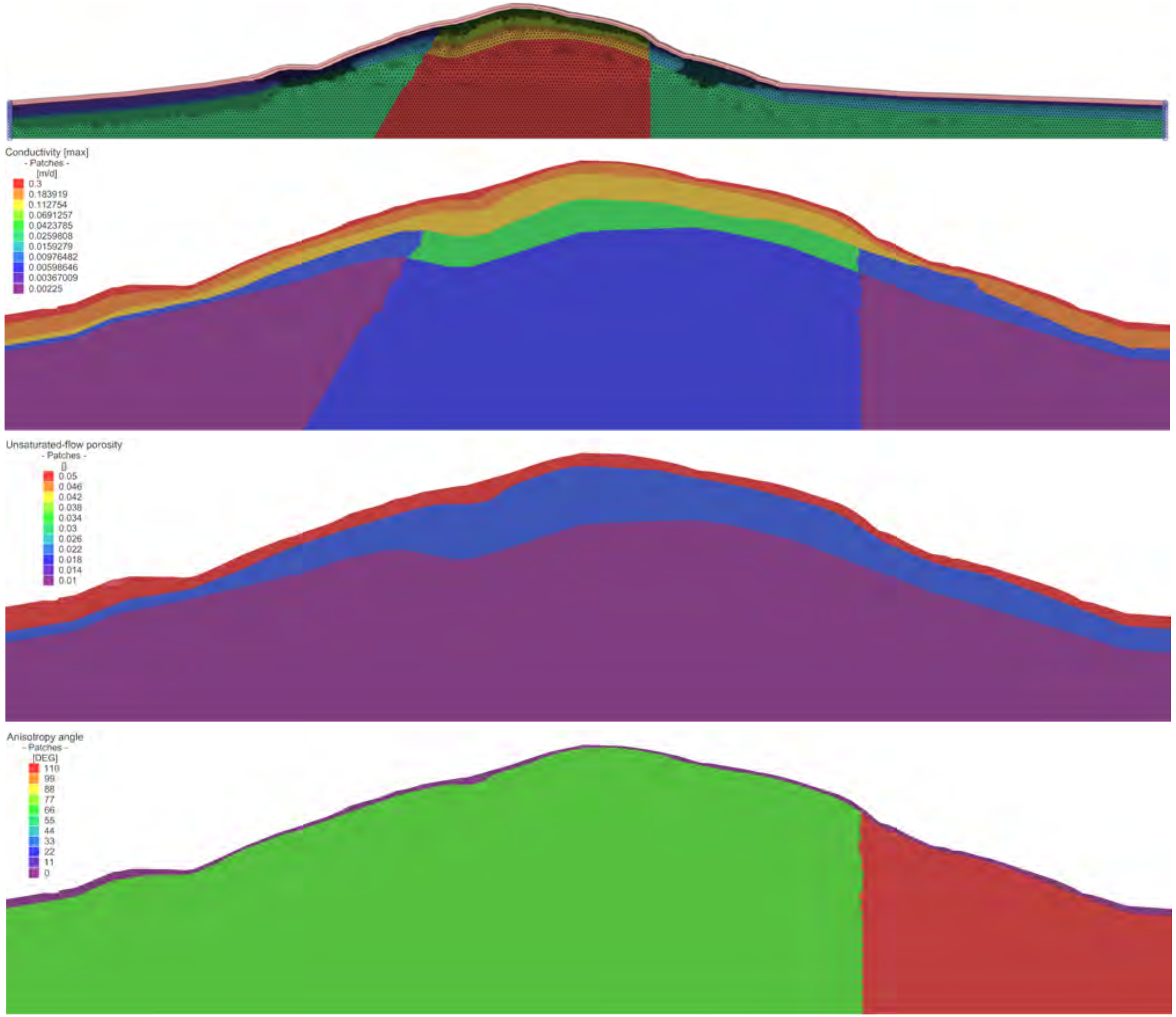
PSM2876-057R

Figure 17

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 17 Shephards Lane model setup.mxd



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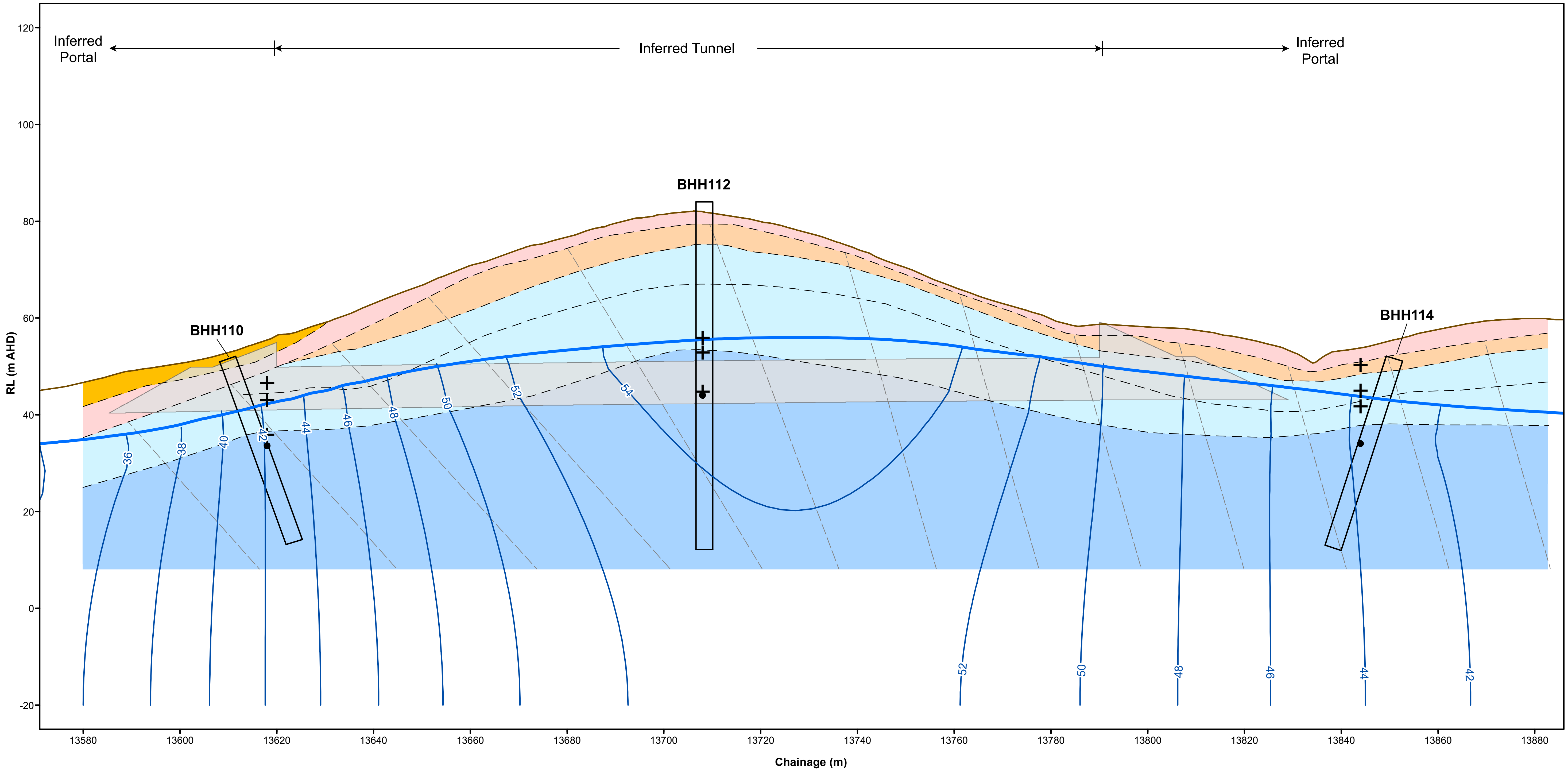
Coffs Harbour Bypass Project
Option B Groundwater Assessment Report

GATELYS ROAD MODEL SETUP

Created By: D.L.	Revision: A	
Date: 23 Apr 2019	Paper Size: A4	PSM2876-057R / Figure 18

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 18 Gatelys Road model setup.mxd



Water Table (Pre Development)
Head Contours (m AHD)
Calibration Target Range
sensor
Roberts Hill Tunnel
Ground Surface
Geological Boundaries
Cleavage Fabric

Geology

Colluvium
Residual Soil
RMU A - EW/HW Rock
RMU B1 - Closely spaced Argillite (Clevage Dominated)
RMU B2 - Widely spaced Argillite (Clevage Dominated)
RMU C1 - Closely spaced Argillite and Siliceous Argillite
RMU C2 - Widely spaced Argillite and Siliceous Argillite
RMU D1 - Closely spaced Argillite with thick beds of Siliceous Argillite
RMU D2 - Widely spaced Argillite with thick beds of Siliceous Argillite

0 5 10 20 30 40 50 m

Horizontal and Vertical Scale

Created By: D.L.
Revision: A
Date: 23 Apr 2019
Paper Size: A3

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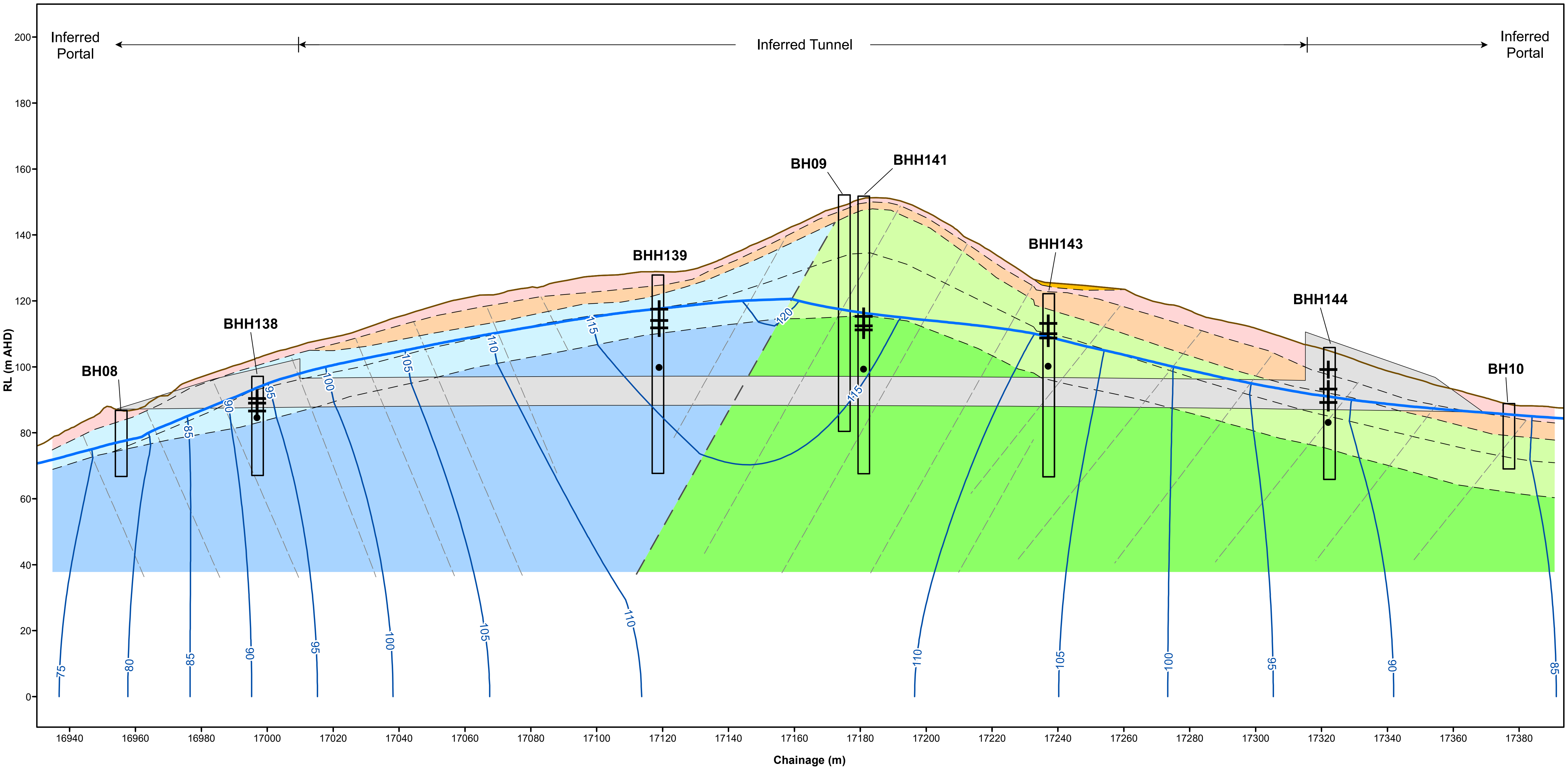
Coffs Harbour Bypass Project
Option B Groundwater Assessment Report

ROBERTS HILL
STEADY-STATE MODEL CALIBRATION

PSM2876-057R
Figure 19

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 19 Roberts Hill Steady-State Model Calibration.mxd



Water Table (Pre Development)
Head Contours (m AHD)
Calibration Target Range
Sensor
Shephards Lane Tunnel
Ground Surface
Geological Boundaries
Rock Mass Unit Boundary
Cleavage Fabric

Geology
Colluvium
Residual Soil
RMU A - EW/HW Rock
RMU B1 - Closely spaced Argillite (Clevage Dominated)
RMU B2 - Widely spaced Argillite (Clevage Dominated)
RMU C1 - Closely spaced Argillite and Siliceous Argillite
RMU C2 - Widely spaced Argillite and Siliceous Argillite
RMU D1 - Closely spaced Argillite with thick beds of Siliceous Argillite
RMU D2 - Widely spaced Argillite with thick beds of Siliceous Argillite

0 5 10 20 30 40 50 m
Horizontal and Vertical Scale

Roads and Maritime Services
Coffs Harbour Bypass Project
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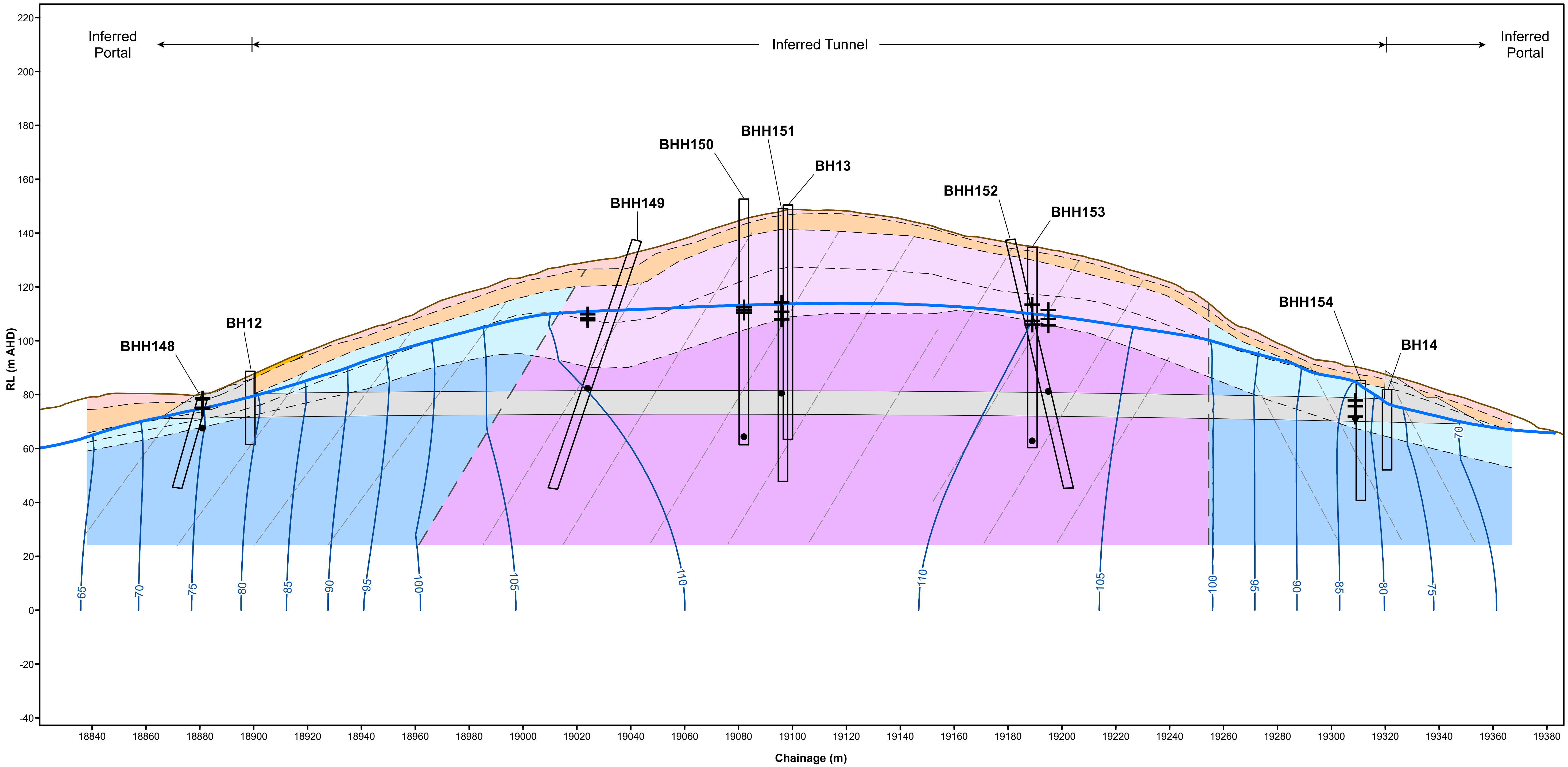
SHEPHARDS LANE
STEADY-STATE MODEL CALIBRATION

Created By: D.L.	Revision: A
Date: 23 Apr 2019	Paper Size: A3

PSM2876-057R
Figure 20

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 20 Shephards Lane Steady-State Model Calibration.mxd



Water Table (Pre Development)
Head Contours (m AHD)
Calibration Target Range
Sensor
Gatelys Road Tunnel
Ground Surface
Geological Boundaries
Rock Mass Unit Boundary
Cleavage Fabric

Geology

- Colluvium
- Residual Soil
- RMU A - EW/HW Rock
- RMU B1 - Closely spaced Argillite (Clevage Dominated)
- RMU B2 - Widely spaced Argillite (Clevage Dominated)
- RMU C1 - Closely spaced Argillite and Siliceous Argillite
- RMU C2 - Widely spaced Argillite and Siliceous Argillite
- RMU D1 - Closely spaced Argillite with thick beds of Siliceous Argillite
- RMU D2 - Widely spaced Argillite with thick beds of Siliceous Argillite

0 5 10 20 30 40 50 m

Horizontal and Vertical Scale

Created By: D.L.
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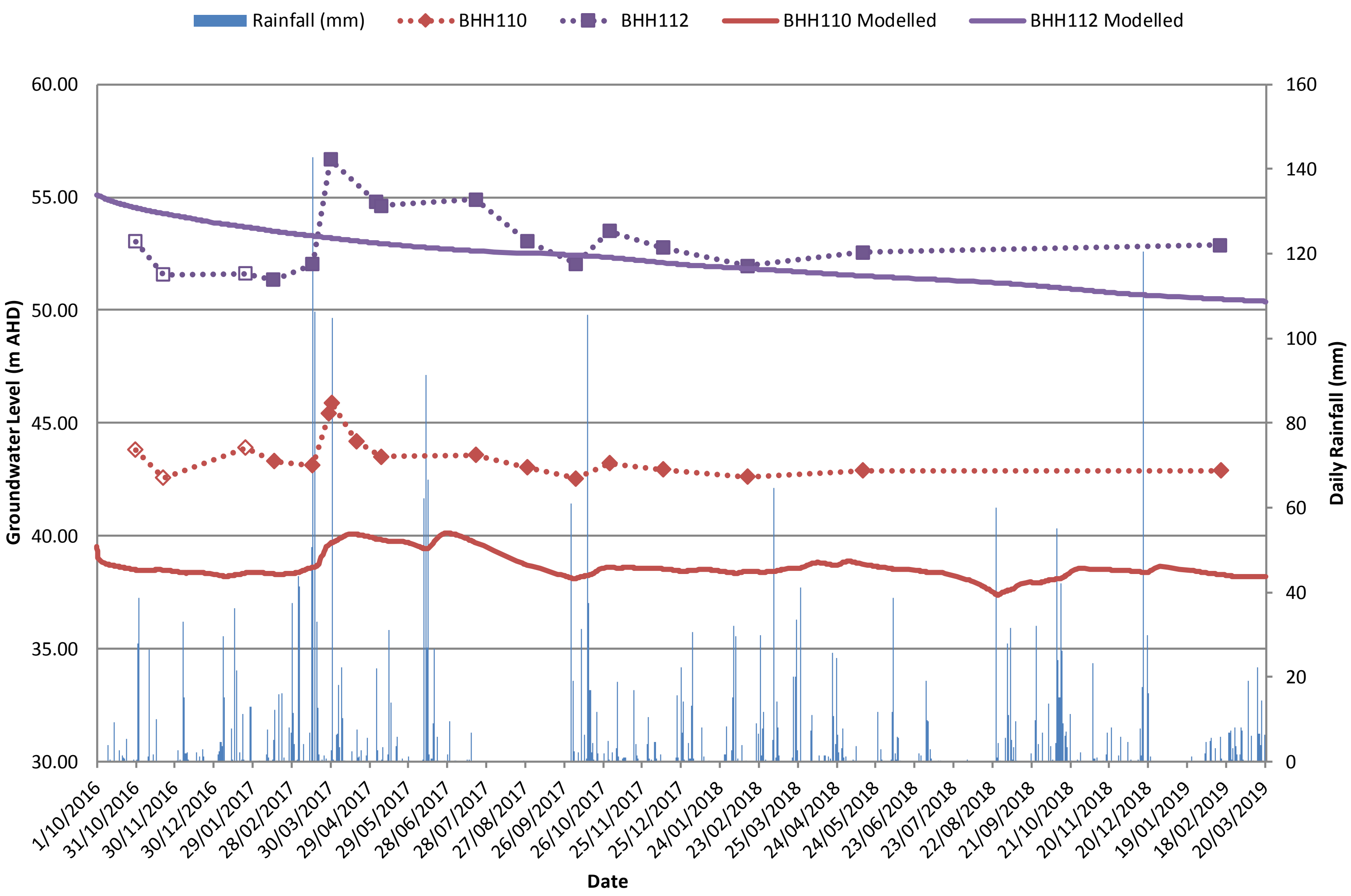
GATELYS ROAD
STEADY-STATE MODEL CALIBRATION

PSM2876-057R

Figure 21

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 21 Gatelys Road Steady-State Model Calibration.mxd



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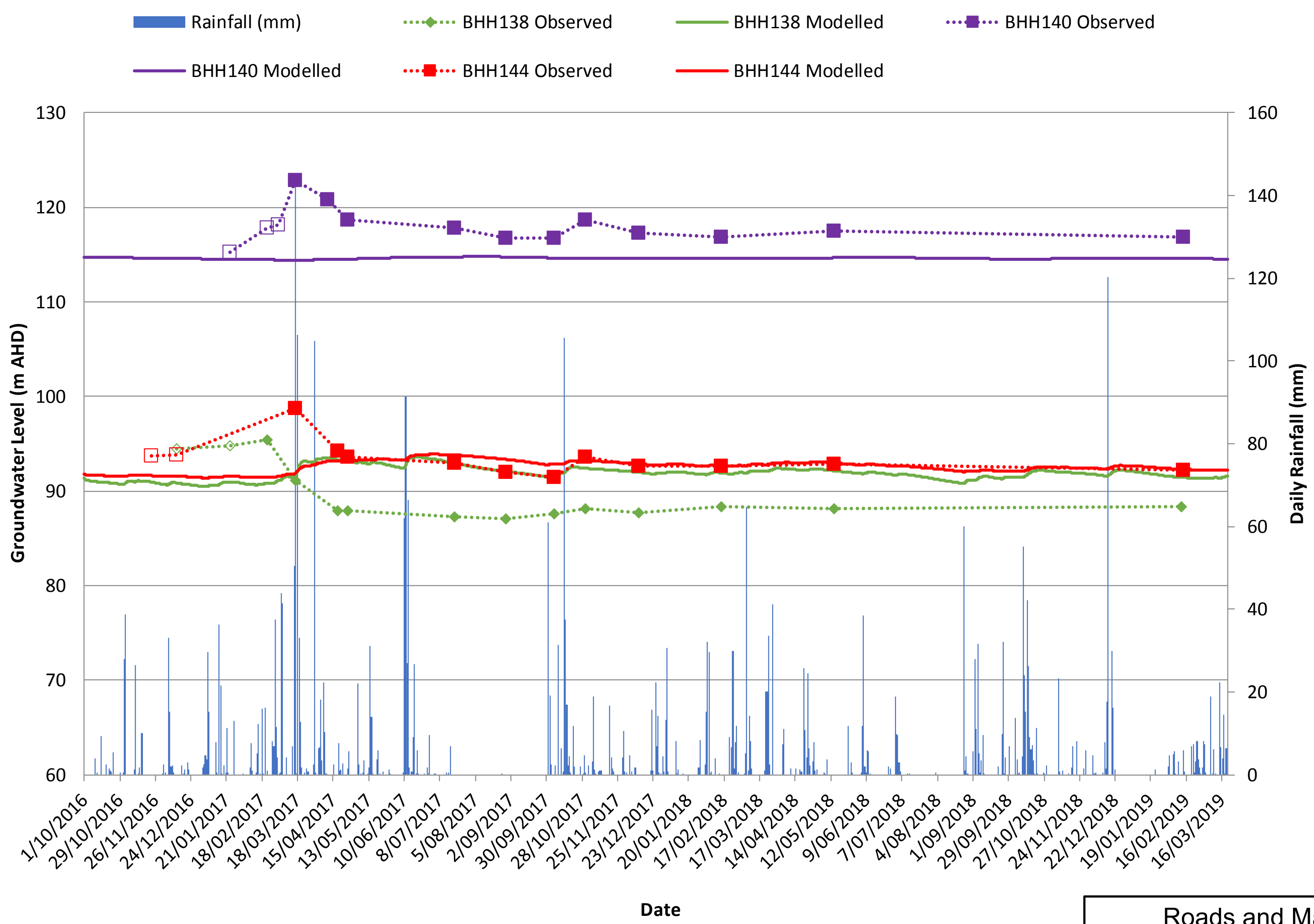
Option B Groundwater Assessment Report

ROBERTS HILL
TRANSIENT CALIBRATION

PSM2876-057R

Figure 22



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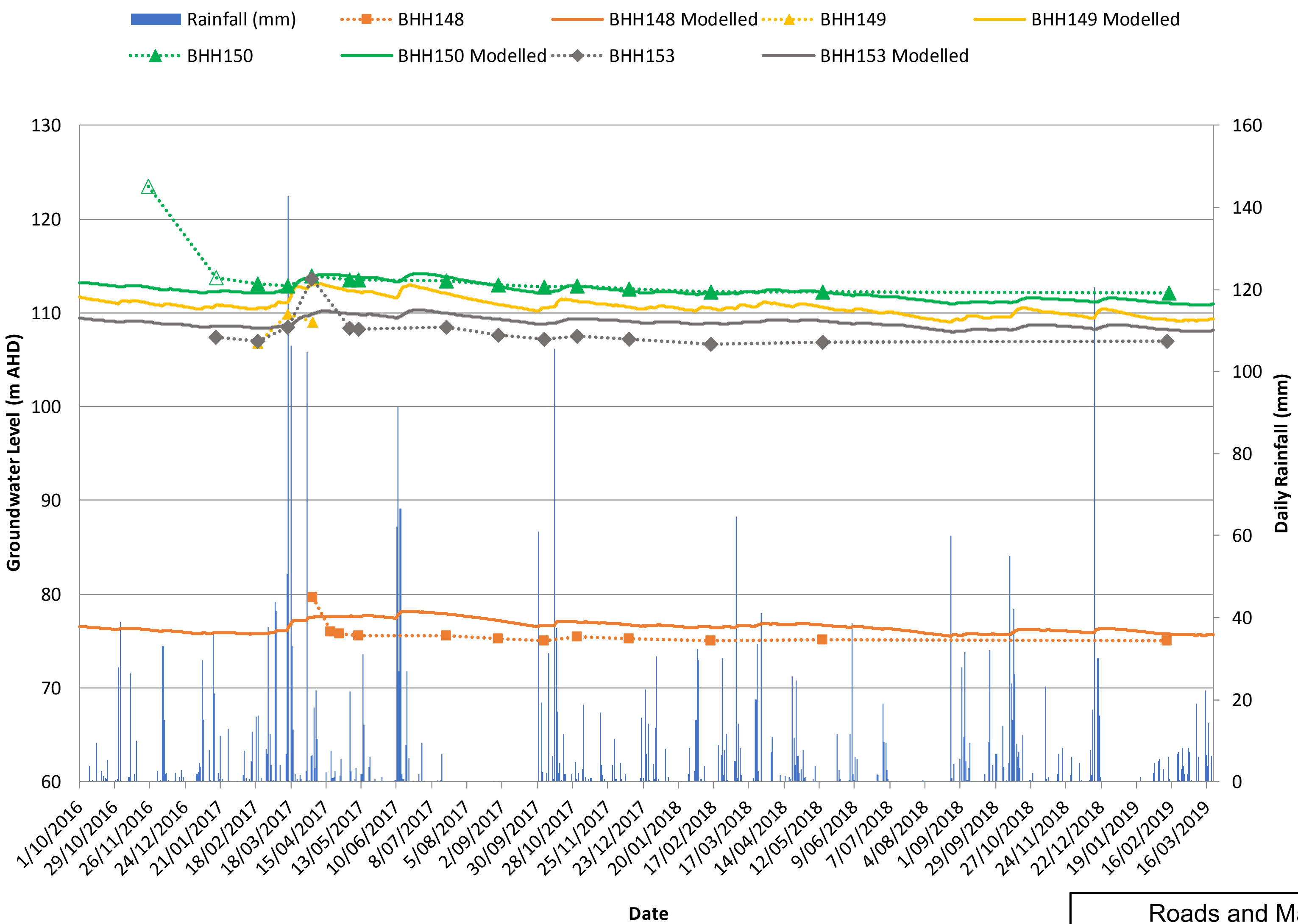
SHEPHARDS LANE
TRANSIENT CALIBRATION

Created By: D.L.	Revision: A		
Date: 23 Apr 2019	Paper Size: A4	PSM2876-057R	Figure 23

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 23 Shephards Lane Transient Calibration.mxd



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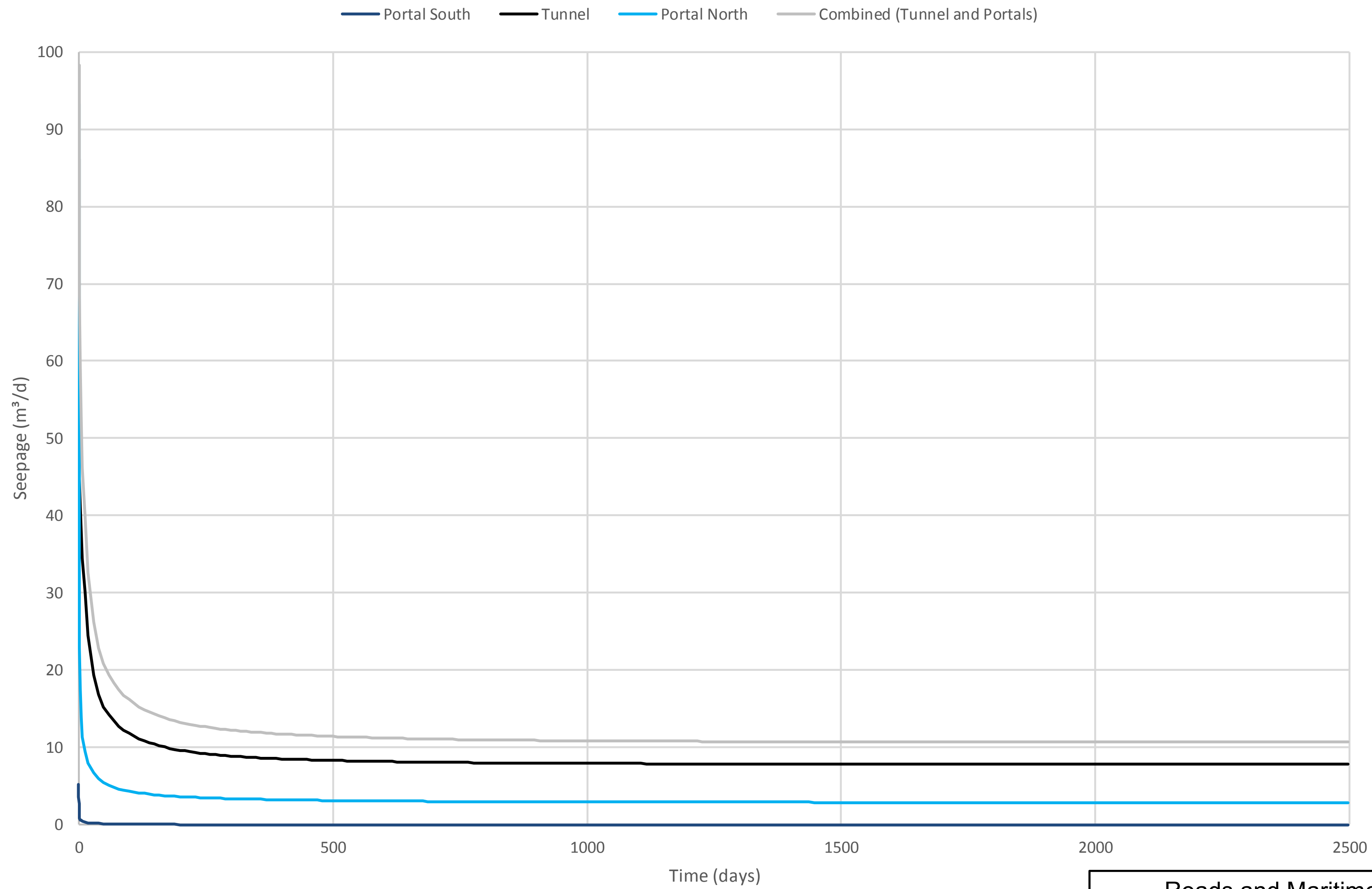
Option B Groundwater Assessment Report

GATELYS ROAD
TRANSIENT CALIBRATION

Created By: D.L.	Revision: A		
Date: 23 Apr 2019	Paper Size: A4	PSM2876-057R	Figure 24

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 24 Gatelys Road Transient Calibration.mxd



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Option B Groundwater Assessment Report

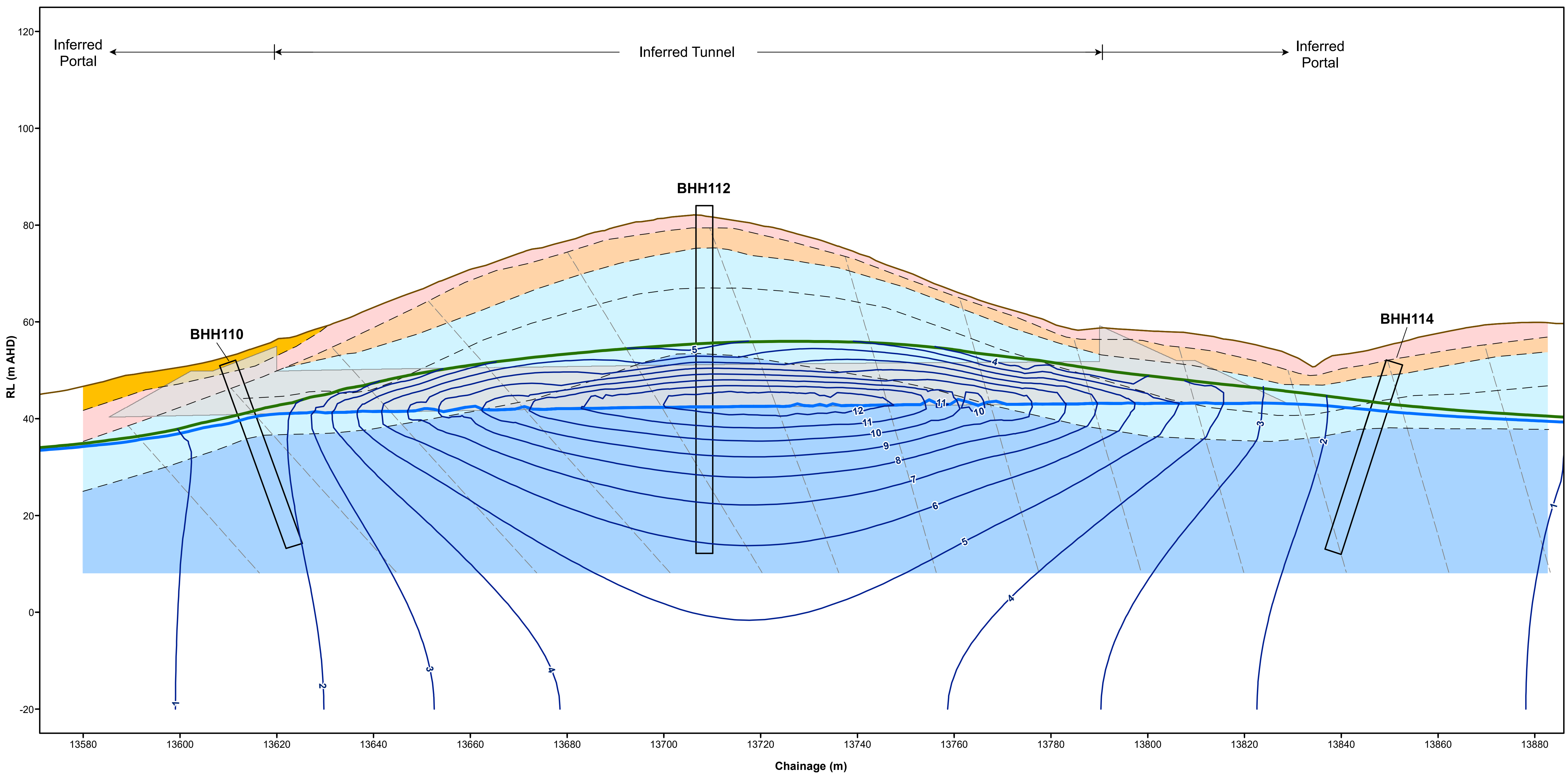
ROBERTS HILL TUNNEL PREDICTIONS
MODELLED PORTAL AND TUNNEL
INFLOW RATES

Created By: D.L.	Revision: A
Date: 23 Apr 2019	Paper Size: A4

PSM2876-057R

Figure 25a

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 25a Roberts Hill tunnel predictions Modelled portal and tunnel inflow rates.mxd



Water Table (Pre Development)
Water Table (18 Months)
Drawdown Contours (m)
Roberts Hill Tunnel
Ground Surface
Geological Boundaries
Cleavage Fabric

Geology

- Colluvium
- Residual Soil
- RMU A - EW/HW Rock
- RMU B1 - Closely spaced Argillite (Clevage Dominated)
- RMU B2 - Widely spaced Argillite (Clevage Dominated)
- RMU C1 - Closely spaced Argillite and Siliceous Argillite
- RMU C2 - Widely spaced Argillite and Siliceous Argillite
- RMU D1 - Closely spaced Argillite with thick beds of Siliceous Argillite
- RMU D2 - Widely spaced Argillite with thick beds of Siliceous Argillite

0 5 10 20 30 40 50 m

Horizontal and Vertical Scale

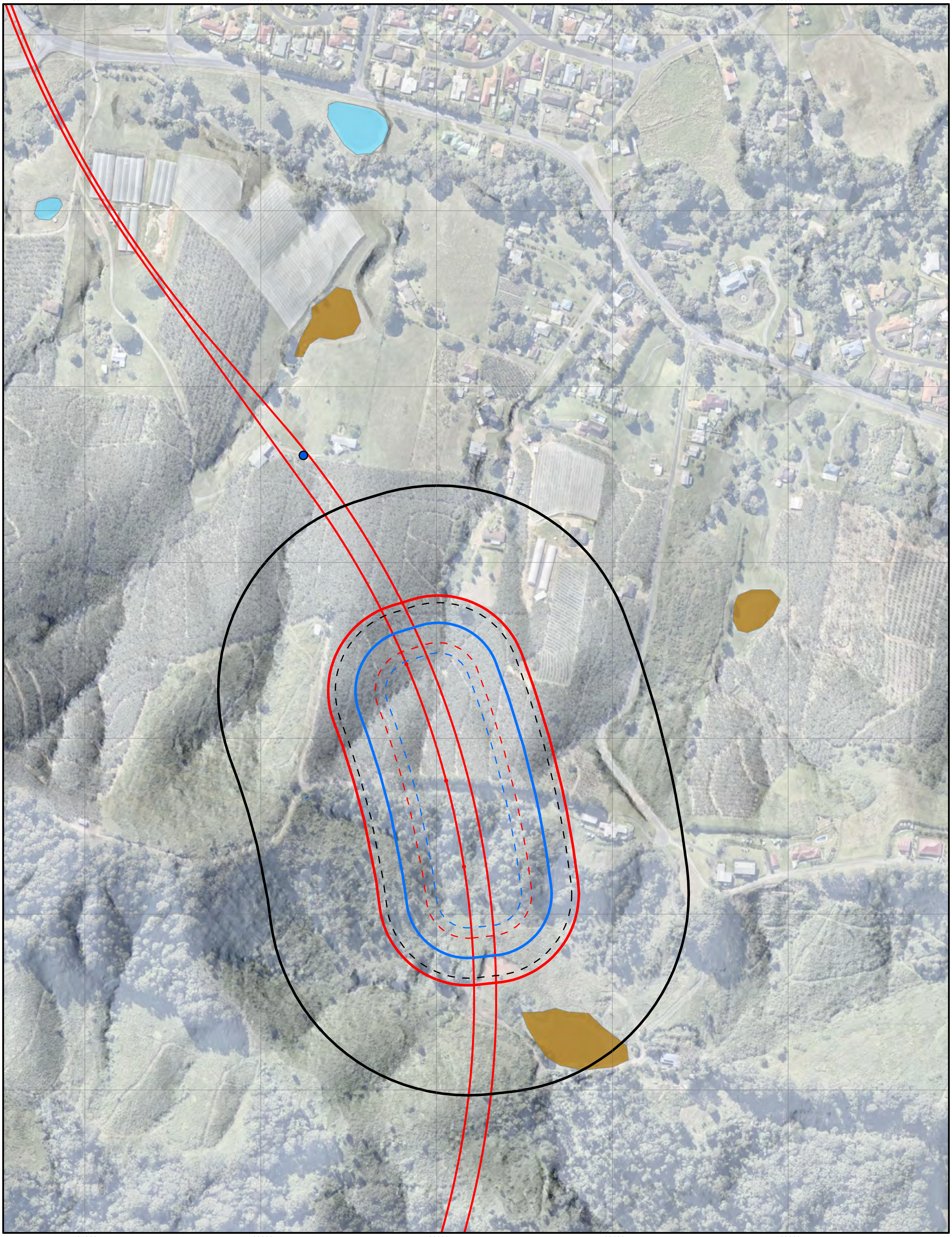
Created By: D.L.	Revision: A	
Date: 23 Apr 2019	Paper Size: A3	

Roads and Maritime Services
Coffs Harbour Bypass Project
Option B Groundwater Assessment Report

ROBERTS HILL TUNNEL PREDICTIONS - 18 MONTH DRAWDOWN (CROSS-SECTION)

PSM2876-057R | Figure 25b

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 25b Roberts Hill tunnel predictions - 18 Month Drawdown (Cross-section).mxd



Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 25c Roberts Hill tunnel predictions - 18 month drawdown and environmental constraints (Plan-view).mxd

Base Case (15% Recharge) Drawdown (m)
- 0.1m Drawdown Contour
- 1m Drawdown Contour
- 2m Drawdown Contour

Sensitivity Analysis (2% Recharge) Drawdown (m)
- 0.1m Drawdown Contour
- 1m Drawdown Contour
- 2m Drawdown Contour

- Groundwater Bores
- GDE
- Dam
- Pond
- CHB Alignment

N

Scale 1:4,000

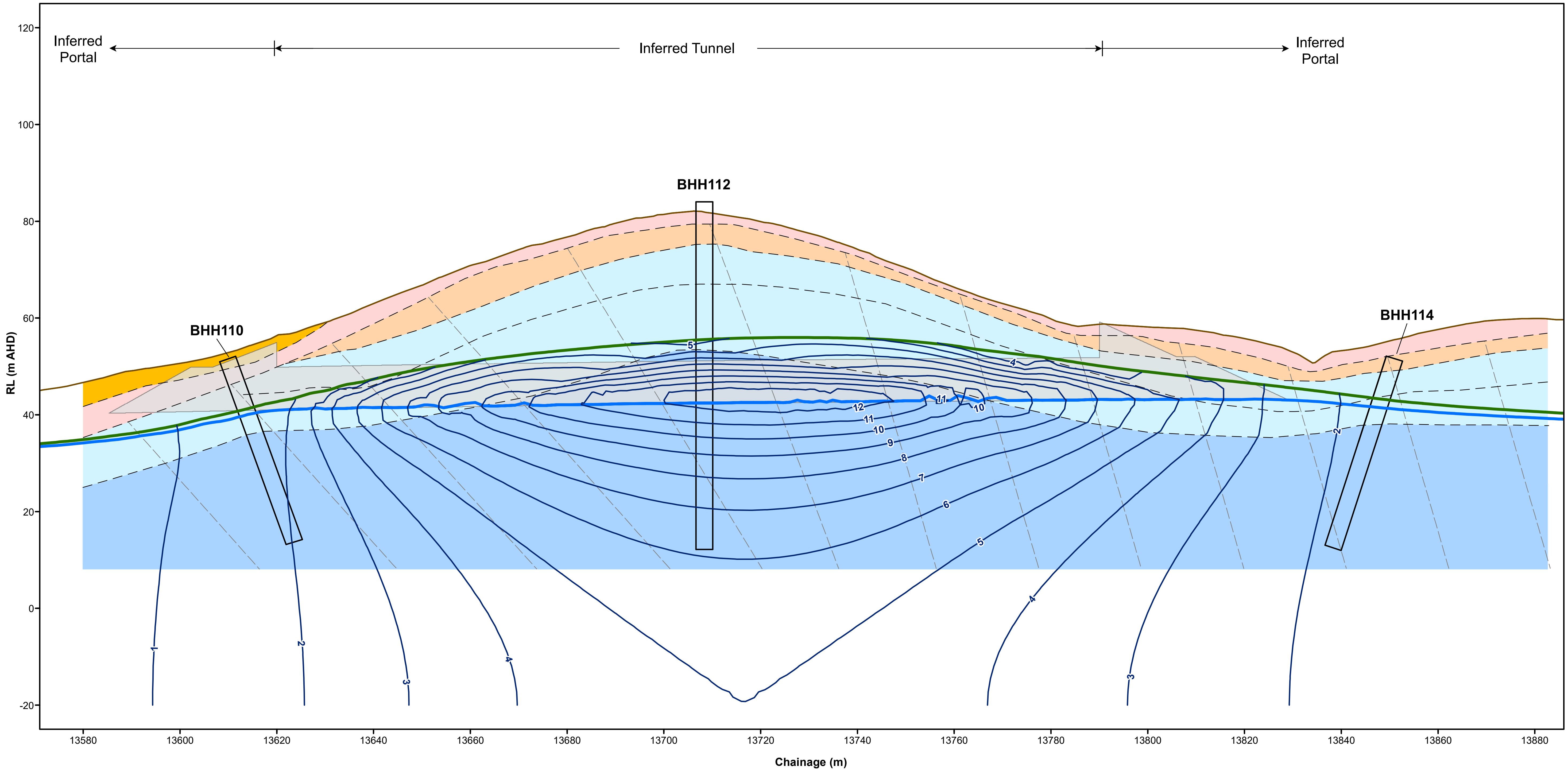
0 25 50 100 150 200 m

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

Roads and Maritime Services
Coffs Harbour Bypass Project
Option B Groundwater Assessment Report

ROBERTS HILL TUNNEL PREDICTIONS - 18 MONTH DRAWDOWN AND ENVIRONMENTAL CONSTRAINTS (PLAN-VIEW)

Created By: D.L.	Revision: A	PSM2876-057R	Figure 25c
Date: 23 Apr 2019	Paper Size: A3		



Water Table (Pre Development)
Water Table (Post Development)
Drawdown Contours (m)
Roberts Hill Tunnel
Ground Surface
Geological Boundaries
Cleavage Fabric

Geology
Colluvium
Residual Soil
RMU A - EW/HW Rock
RMU B1 - Closely spaced Argillite (Clevage Dominated)
RMU B2 - Widely spaced Argillite (Clevage Dominated)
RMU C1 - Closely spaced Argillite and Siliceous Argillite
RMU C2 - Widely spaced Argillite and Siliceous Argillite
RMU D1 - Closely spaced Argillite with thick beds of Siliceous Argillite
RMU D2 - Widely spaced Argillite with thick beds of Siliceous Argillite

0 5 10 20 30 40 50 m
Horizontal and Vertical Scale

Created By: D.L.	Revision: A
Date: 23 Apr 2019	Paper Size: A3

Roads and Maritime Services
Coffs Harbour Bypass Project
Option B Groundwater Assessment Report

ROBERTS HILL TUNNEL
PREDICTIONS - STEADY STATE
DRAWDOWN (CROSS-SECTION)

PSM2876-057R Figure 25d

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 25d Roberts Hill tunnel predictions - steady state drawdown (Cross-section).mxd

Base Case (15% Recharge) Drawdown (m)

- 0.1m Drawdown Contour
- 1m Drawdown Contour
- 2m Drawdown Contour

Sensitivity Analysis (2% Recharge) Drawdown (m)

- 0.1m Drawdown Contour
- 1m Drawdown Contour
- 2m Drawdown Contour

- Groundwater Bores
- GDE
- Dam
- Pond
- CHB Alignment

N

Scale 1:4,000

0 25 50 100 150 200 m

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

Created By: D.L.

Revision: A

Date: 23 Apr 2019

Paper Size: A3

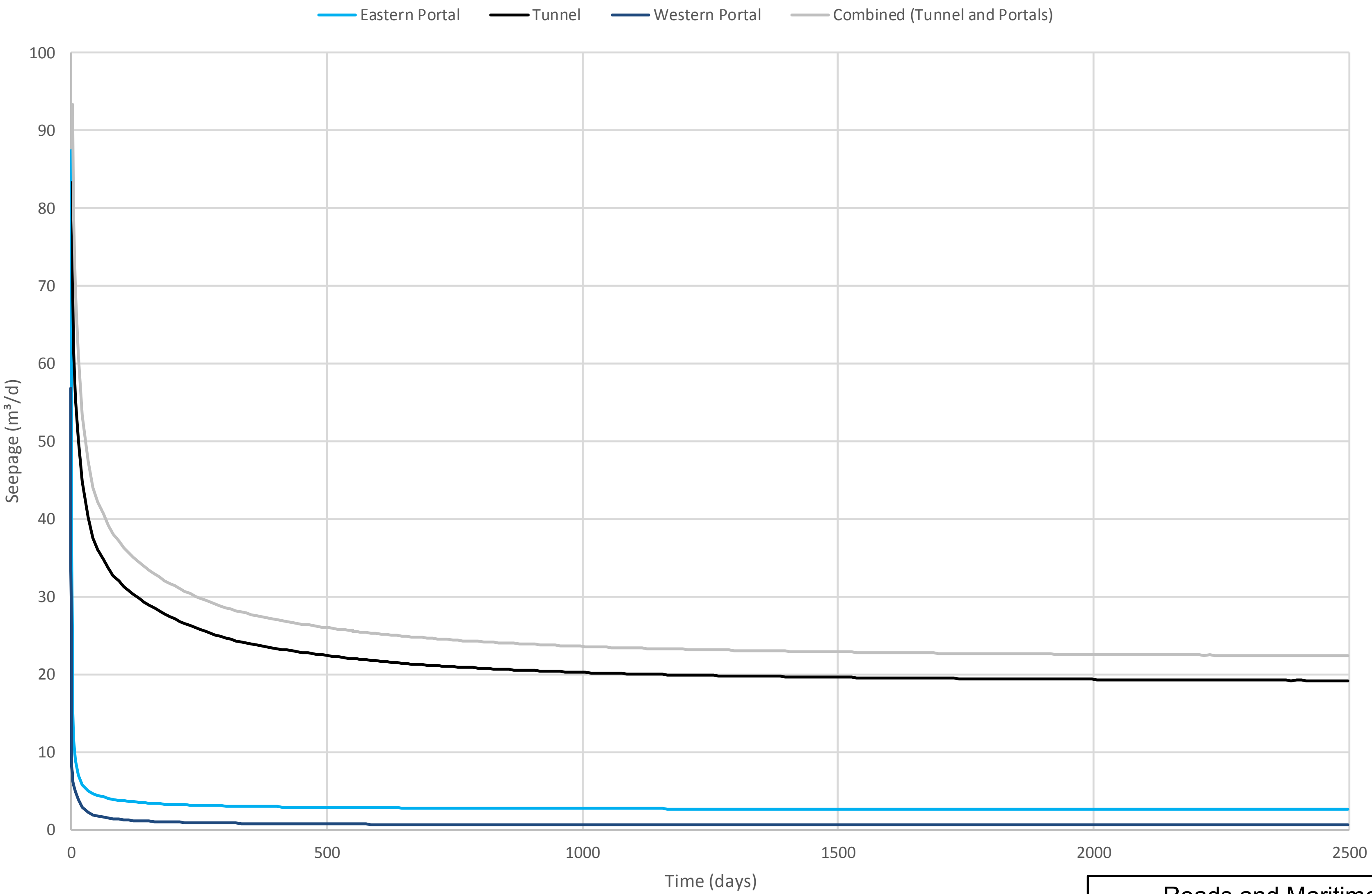
Roads and Maritime Services
Coffs Harbour Bypass Project
Option B Groundwater Assessment Report

ROBERTS HILL TUNNEL PREDICTIONS - STEADY STATE DRAWDOWN AND ENVIRONMENTAL CONSTRAINTS (PLAN-VIEW)

PSM2876-057R

Figure 25e

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 25e Roberts Hill tunnel predictions - Steady state drawdown and environmental constraints (Plan-view).mxd



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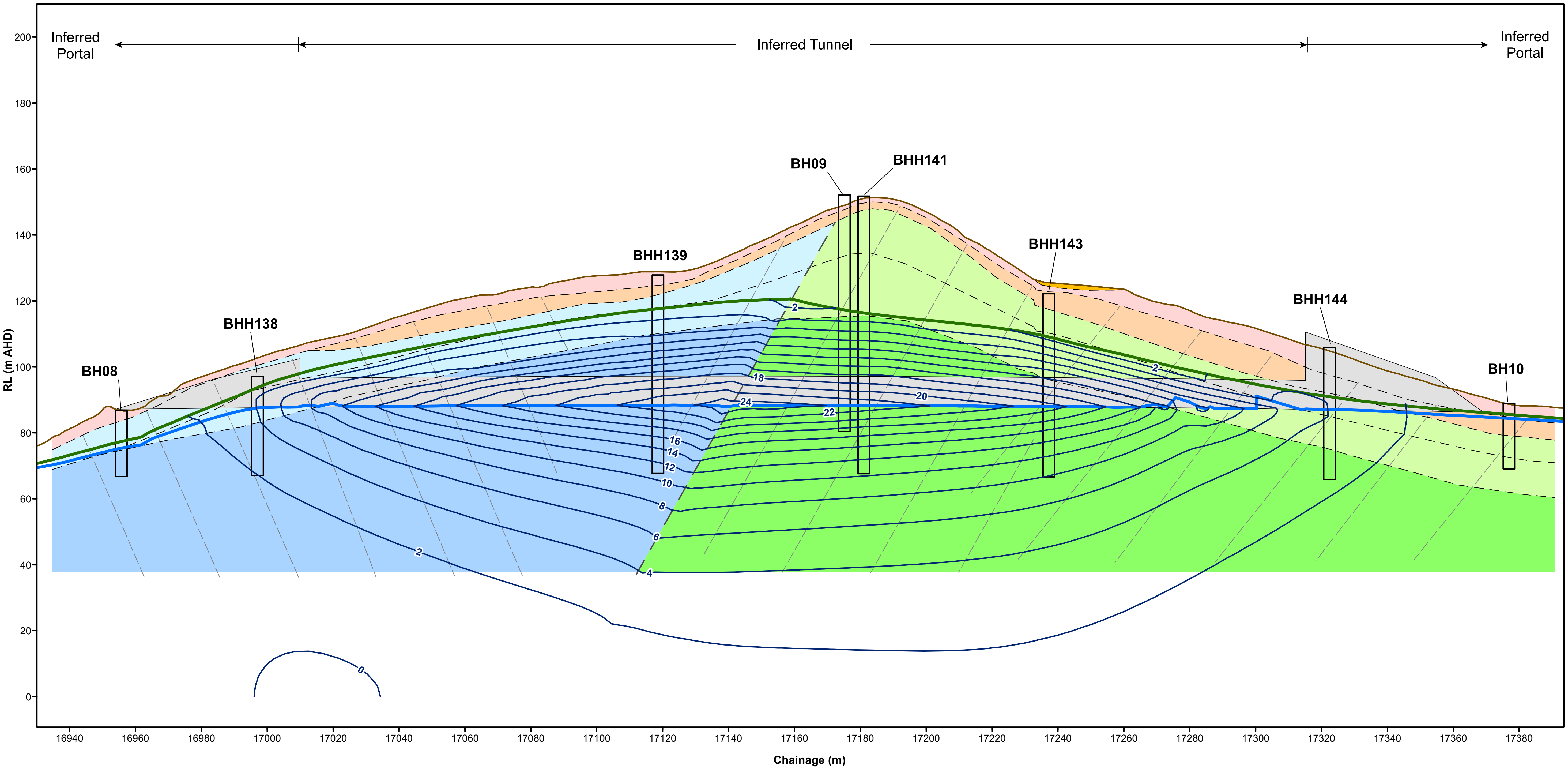
Coffs Harbour Bypass Project
Option B Groundwater Assessment Report

SHEPHARDS LANE TUNNEL PREDICTIONS - MODELLED PORTAL AND TUNNEL INFLOW RATES

Created By: D.L.	Revision: A	
Date: 23 Apr 2019	Paper Size: A4	PSM2876-057R	Figure 26a

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 26a Shephards Lane tunnel predictions Modelled portal and tunnel inflow rates.mxd



Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 26b Shephards Lane tunnel predictions - 18 Month Drawdown (Cross-section).mxd

- Water Table (Pre Development)
- Water Table (18 Months)
- Drawdown Contours (m)
- Shephards Lane Tunnel
- Ground Surface
- Geological Boundaries
- Rock Mass Unit Boundary
- Cleavage Fabric

Geology

- Colluvium
- Residual Soil
- RMU A - EW/HW Rock
- RMU B1 - Closely spaced Argillite (Clevage Dominated)
- RMU B2 - Widely spaced Argillite (Clevage Dominated)
- RMU C1 - Closely spaced Argillite and Siliceous Argillite
- RMU C2 - Widely spaced Argillite and Siliceous Argillite
- RMU D1 - Closely spaced Argillite with thick beds of Siliceous Argillite
- RMU D2 - Widely spaced Argillite with thick beds of Siliceous Argillite

0 5 10 20 30 40 50 m

Horizontal and Vertical Scale

Roads and Maritime Services
Coffs Harbour Bypass Project
Option B Groundwater Assessment Report

SHEPHARDS LANE TUNNEL PREDICTIONS - 18 MONTH DRAWDOWN (CROSS-SECTION)

Created By: D.L.	Revision: A
Date: 23 Apr 2019	Paper Size: A3

PSM2876-057R Figure 26b

Base Case (15% Recharge) Drawdown (m)
- 0.1m Drawdown Contour
- 1m Drawdown Contour
- 2m Drawdown Contour

Sensitivity Analysis (2% Recharge) Drawdown (m)
- 0.1m Drawdown Contour
- 1m Drawdown Contour
- 2m Drawdown Contour

- Groundwater Bores
- GDE
- Dam
- Pond
- 10m Contours
- CHB Alignment

N

Scale 1:4,750

0 25 50 100 150 200 250 m

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

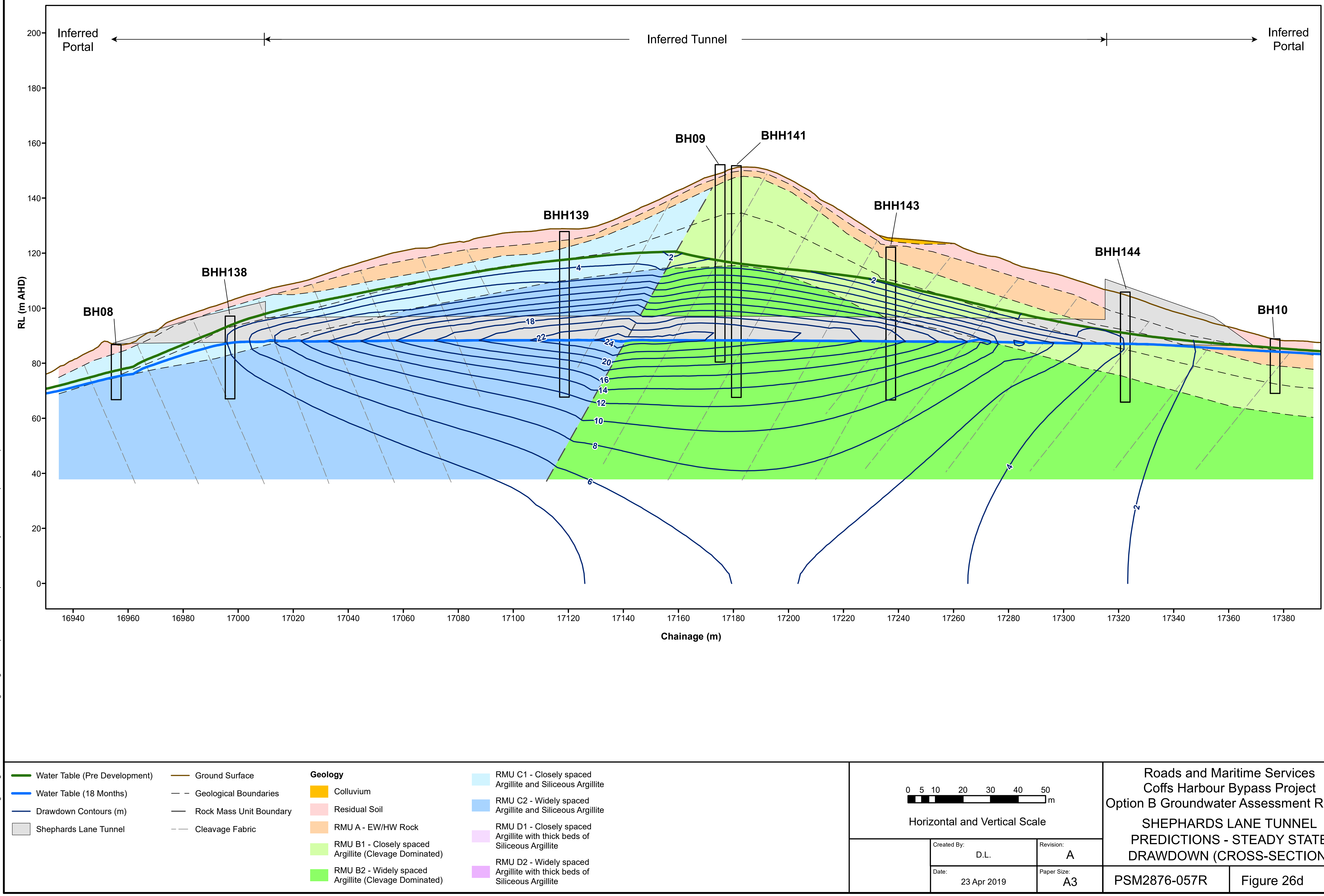
Roads and Maritime Services
Coffs Harbour Bypass Project
Option B Groundwater Assessment Report

SHEPHARDS LANE TUNNEL PREDICTIONS - 18 MONTH DRAWDOWN AND ENVIRONMENTAL CONSTRAINTS (PLAN VIEW)

Created By:	Revision:
D.L.	A
Date: 23 Apr 2019	Paper Size: A3

PSM2876-057R | Figure 26c

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 26c Shephards Lane tunnel predictions - 18 month drawdown and environmental constraints (Plan-view).mxd



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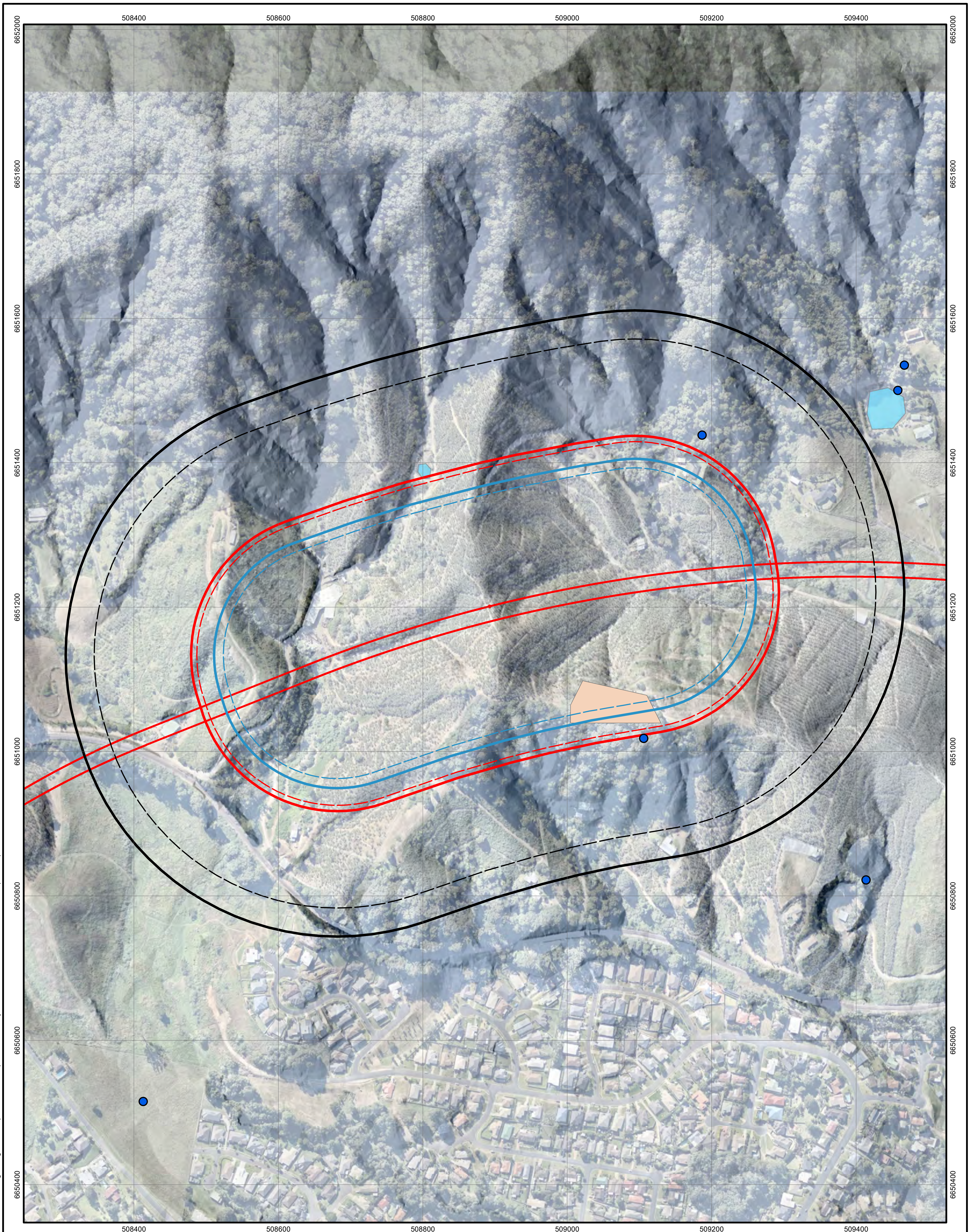
Coffs Harbour Bypass Project
Option B Groundwater Assessment Report

SHEPHARDS LANE TUNNEL PREDICTIONS - STEADY STATE DRAWDOWN (CROSS-SECTION)

Created By: D.L.	Revision: A		
Date: 23 Apr 2019	Paper Size: A3	PSM2876-057R	Figure 26d

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 26d Shephards Lane tunnel predictions - steady state drawdown (Cross-section).mxd



Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 26e Shephards Lane tunnel predictions - Steady state drawdown and environmental constraints (Plan-view).mxd

Base Case (15% Recharge) Drawdown (m)
- 0.1m Drawdown Contour
- 1m Drawdown Contour
- 2m Drawdown Contour

Sensitivity Analysis (2% Recharge) Drawdown (m)
- 0.1m Drawdown Contour
- 1m Drawdown Contour
- 2m Drawdown Contour

- Groundwater Bores
- GDE
- Dam
- Pond
- 10m Contours
- CHB Alignment

N

Scale 1:4,750

0 25 50 100 150 200 m

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

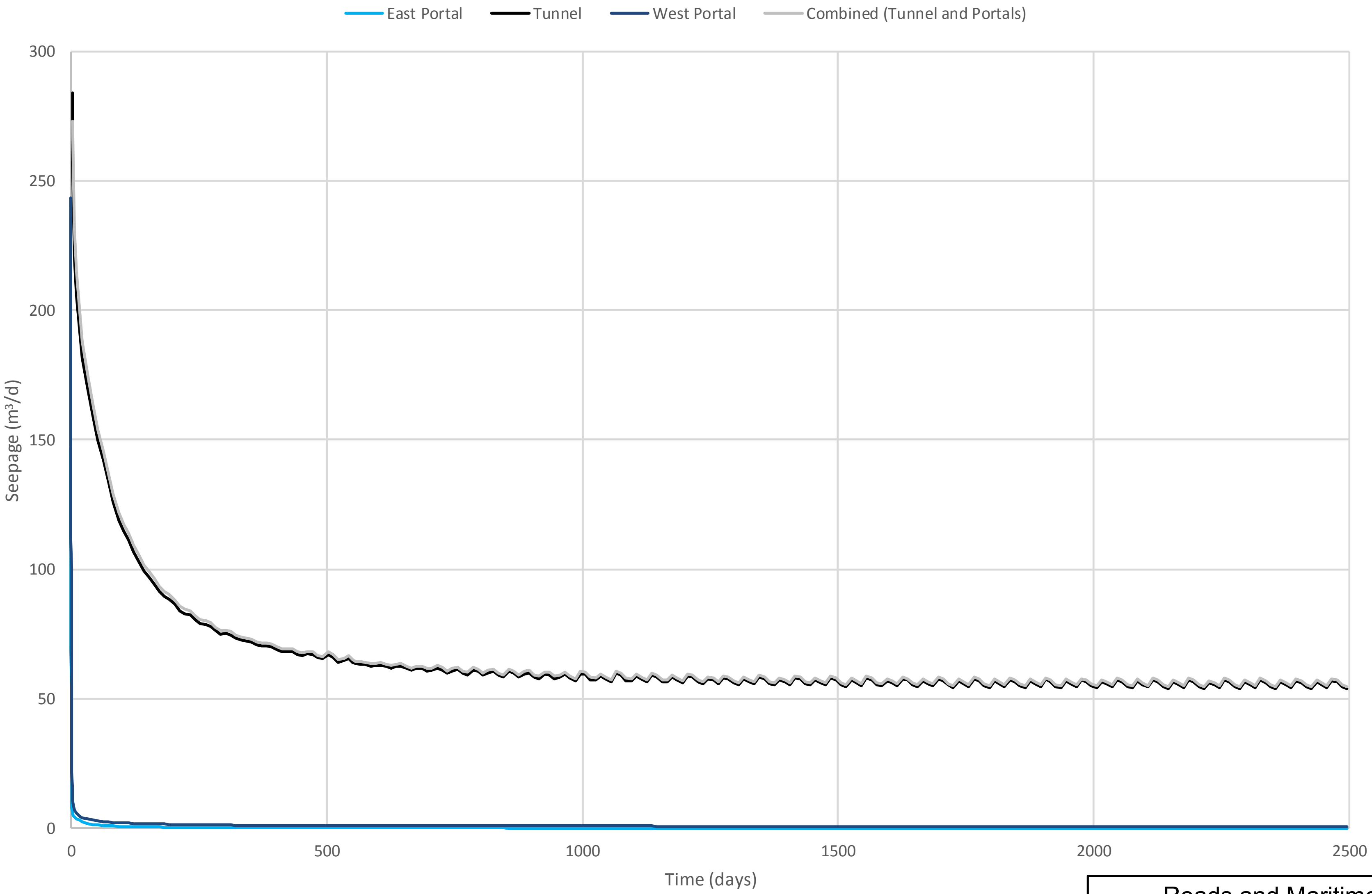
Roads and Maritime Services
Coffs Harbour Bypass Project
Option B Groundwater Assessment Report

SHEPHARDS LANE TUNNEL PREDICTIONS - STEADY STATE DRAWDOWN AND ENVIRONMENTAL CONSTRAINTS (PLAN-VIEW)

Created By:	Revision:
D.L.	A
Date: 23 Apr 2019	Paper Size: A3

PSM2876-057R

Figure 26e

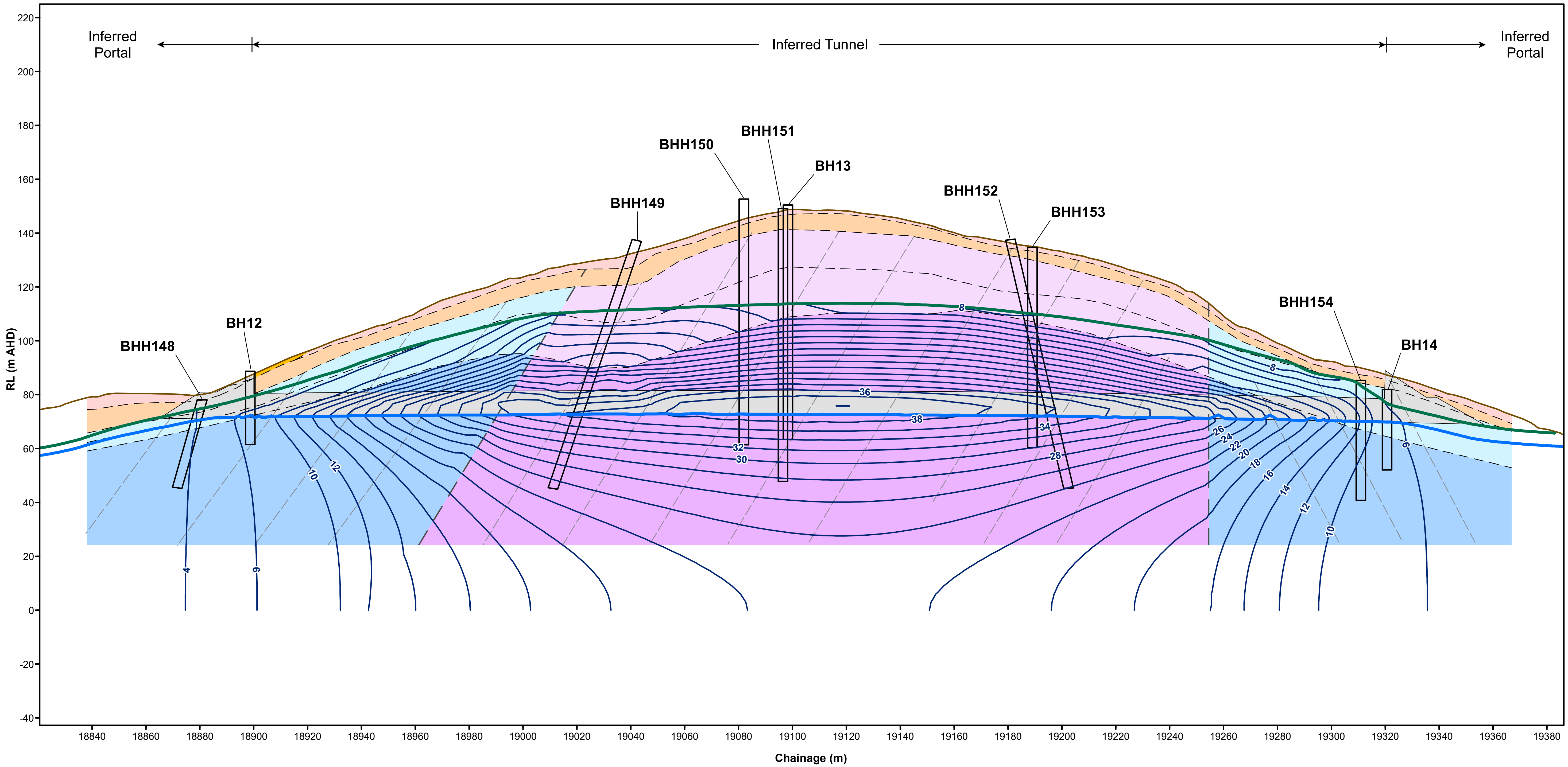


Roads and Maritime Services
Coffs Harbour Bypass Project
Option B Groundwater Assessment Report

GATELYS ROAD TUNNEL PREDICTIONS - MODELLED PORTAL AND TUNNEL INFLOW RATES

Created By: D.L.	Revision: A
Date: 23 Apr 2019	Paper Size: A4

PSM2876-057R | Figure 27a



Water Table (Pre Development)
Water Table (18 Months)
Drawdown Contours (m)
Gatelys Road Tunnel
Ground Surface
Geological Boundaries
Rock Mass Unit Boundary
Cleavage Fabric

Geology

Colluvium
Residual Soil
RMU A - EW/HW Rock
RMU B1 - Closely spaced Argillite (Clevage Dominated)
RMU B2 - Widely spaced Argillite (Clevage Dominated)
RMU C1 - Closely spaced Argillite and Siliceous Argillite
RMU C2 - Widely spaced Argillite and Siliceous Argillite
RMU D1 - Closely spaced Argillite with thick beds of Siliceous Argillite
RMU D2 - Widely spaced Argillite with thick beds of Siliceous Argillite

0 5 10 20 30 40 50 m
Horizontal and Vertical Scale

Created By: D.L.
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Date: 23 Apr 2019
Paper Size: A3

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GATELYS ROAD TUNNEL PREDICTIONS - 18 MONTH DRAWDOWN (CROSS-SECTION)

PSM2876-057R
Figure 27b

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 27b Gatelys Road tunnel predictions - 18 Month Drawdown (Cross-section).mxd

Base Case (15% Recharge) Drawdown (m)

- 0.1m Drawdown Contour
- 1m Drawdown Contour
- 2m Drawdown Contour

Sensitivity Analysis (2% Recharge) Drawdown (m)

- 0.1m Drawdown Contour
- 1m Drawdown Contour
- 2m Drawdown Contour

- Groundwater Bores
- GDE
- Dam
- Pond
- CHB Alignment

N

Scale 1:8,000

0 50 100 200 300 400 m

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

Created By:	Revision:
D.L.	A
Date: 23 Apr 2019	Paper Size: A3

Roads and Maritime Services
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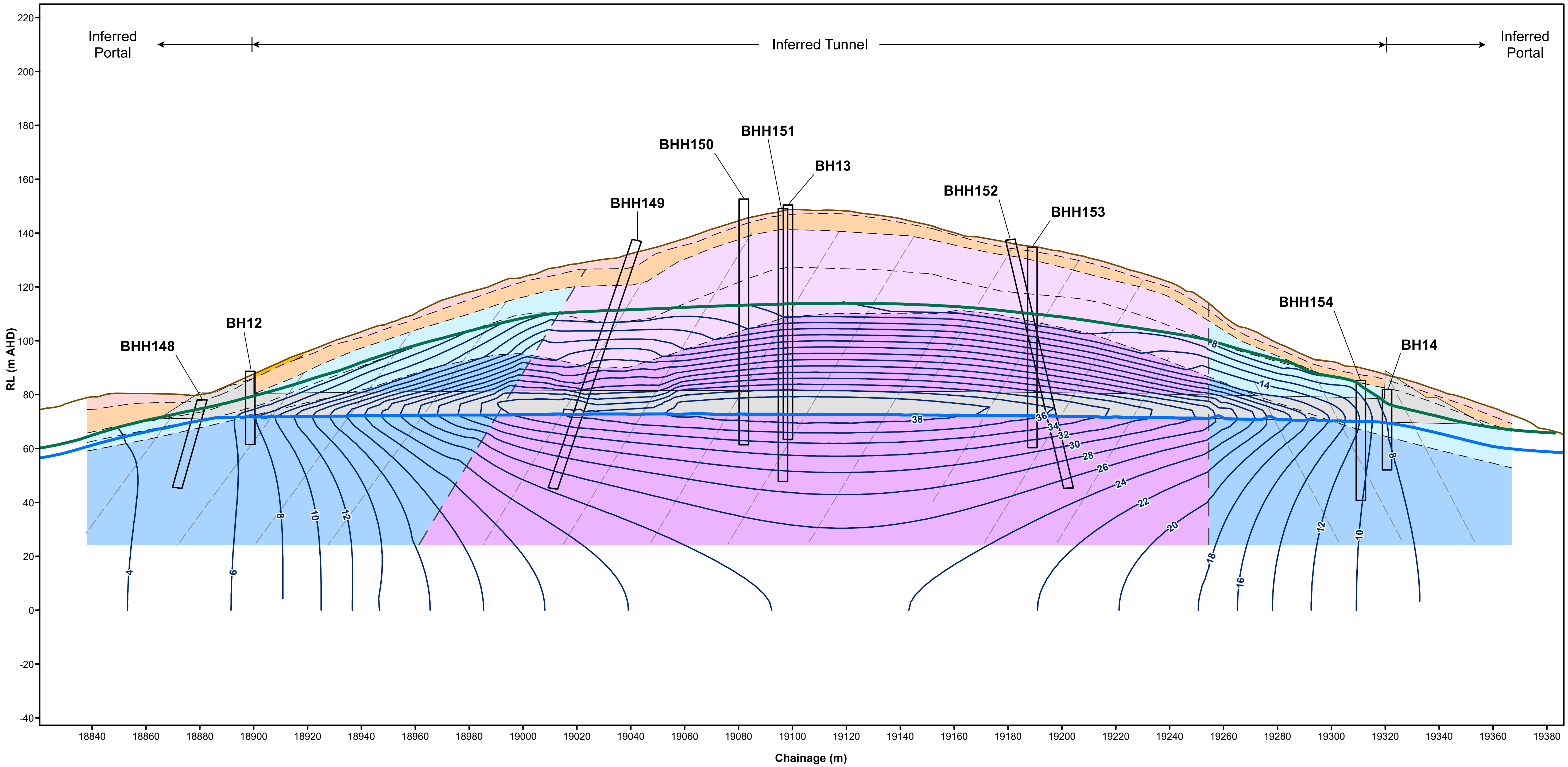
GATELYS ROAD TUNNEL PREDICTIONS
18 MONTH DRAWDOWN AND ENVIRONMENTAL CONSTRAINTS (PLAN-VIEW)

PSM2876-057R | Figure 27c

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 27c Gatelys Road tunnel predictions - 18 month drawdown and environmental constraints (Plan-view).mxd



Water Table (Pre Development)
Water Table (Post Development)
Drawdown Contours (m)
Gatelys Road Tunnel
Ground Surface
Geological Boundaries
Rock Mass Unit Boundary
Cleavage Fabric

Geology

Colluvium
Residual Soil
RMU A - EW/HW Rock
RMU B1 - Closely spaced Argillite (Clevage Dominated)
RMU B2 - Widely spaced Argillite (Clevage Dominated)
RMU C1 - Closely spaced Argillite and Siliceous Argillite
RMU C2 - Widely spaced Argillite and Siliceous Argillite
RMU D1 - Closely spaced Argillite with thick beds of Siliceous Argillite
RMU D2 - Widely spaced Argillite with thick beds of Siliceous Argillite

0 5 10 20 30 40 50 m
Horizontal and Vertical Scale

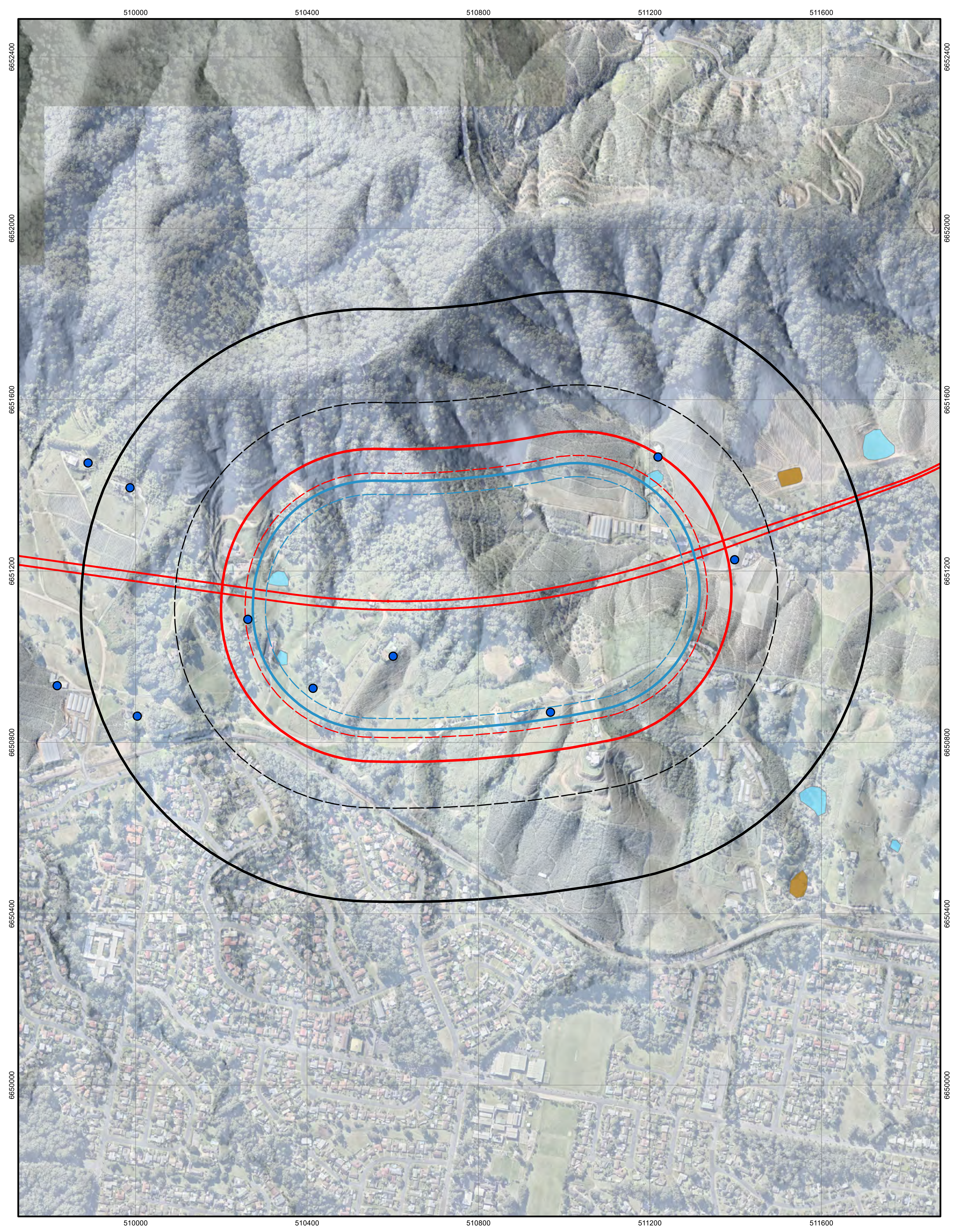
Created By: D.L.
Revision: A
Date: 23 Apr 2019
Paper Size: A3

Roads and Maritime Services
Coffs Harbour Bypass Project
Option B Groundwater Assessment Report

GATELYS ROAD TUNNEL PREDICTIONS - STEADY STATE DRAWDOWN (CROSS-SECTION)

PSM2876-057R
Figure 27d

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 27d Gatelys Road tunnel predictions - steady state drawdown (Cross-section).mxd



Base Case (15% Recharge) Drawdown (m)

- 0.1m Drawdown Contour
- 1m Drawdown Contour
- 2m Drawdown Contour

Sensitivity Analysis (2% Recharge) Drawdown (m)

- 0.1m Drawdown Contour
- 1m Drawdown Contour
- 2m Drawdown Contour

- Groundwater Bores
- GDE
- Dam
- Pond
- CHB Alignment

N

Scale 1:8,000

0 50 100 200 300 400 m

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

Created By:	Revision:
D.L.	A
Date: 23 Apr 2019	Paper Size: A3

Roads and Maritime Services
Coffs Harbour Bypass Project
Option B Groundwater Assessment Report

GATELYS ROAD TUNNEL PREDICTIONS - STEADY STATE DRAWDOWN AND ENVIRONMENTAL CONSTRAINTS (PLAN-VIEW)

PSM2876-057R | Figure 27e

Z:\Jobs 2800 to 2900\PSM2876\Eng\GIS\Figures\PSM2876-057R Final Figures\Figure 27e Gatelys Road tunnel predictions - Steady state drawdown and environmental constraints (Plan-view).mxd

Appendix A
Packer Testing

- A1 – Roberts Hill Packer Testing Data
- A2 – Shephards Lane Packer Testing Data
- A3 – Gatelys Road Packer Testing Data
- A4 – Hydraulic Conductivity Summary Tables

A1 – Roberts Hill Packer Testing Data

A1 – Packer Test Summary Table – Roberts Hill

BHID	Test From (m)	Test To (m)	Lugeon Value 1 [(1)]	Lugeon Value 2 [(2)]	Lugeon Classification	Weathering	RMU	Hydraulic Conductivity (m/d) [(3)]
BHH112	6.85	11.85	10.1	11.4	Mod/High	MW	C1	0.13
BHH112	12.85	17.85	11.7	13.4	Mod/High	MW	C1	0.15
BHH112	18.85	23.85	0.0	0.3	Low	SW	C1	3.90×10^{-3}
BHH112	24.85	29.85	0.0	0.0	No Flow	F	C2	
BHH112	30.85	35.85	0.0	0.0	No Flow	F	C2	
BHH112	36.85	41.85	7.2	4.4	Low/Mod to Mod/High	F	C2	4.90×10^{-2}
BHH112	42.85	47.85	0.7	0.4	Low	F	C2	4.80×10^{-3}
BHH112	48.85	53.85	0.6	0.4	Low	F	C2	4.00×10^{-3}
BHH112	54.85	59.85	0.9	0.8	Low	F	C2	8.90×10^{-3}
BHH112	60.85	65.85	3.4	2.3	Low/Mod	F	C2	2.50×10^{-2}
BHH112	66.85	71.85	0.4	0.3	Low	F	C2	3.10×10^{-3}
BHH113	50.10	56.00	1.8	2.2	Low/Mod	F	C2	2.60×10^{-2}

Notes:

[1] Calculated from Fell et al., 2005

[2] Calculated from Houlsby, 1976

[3] Calculated from Moye, 1967

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Test undertaken above water table

Job no. PSM2876

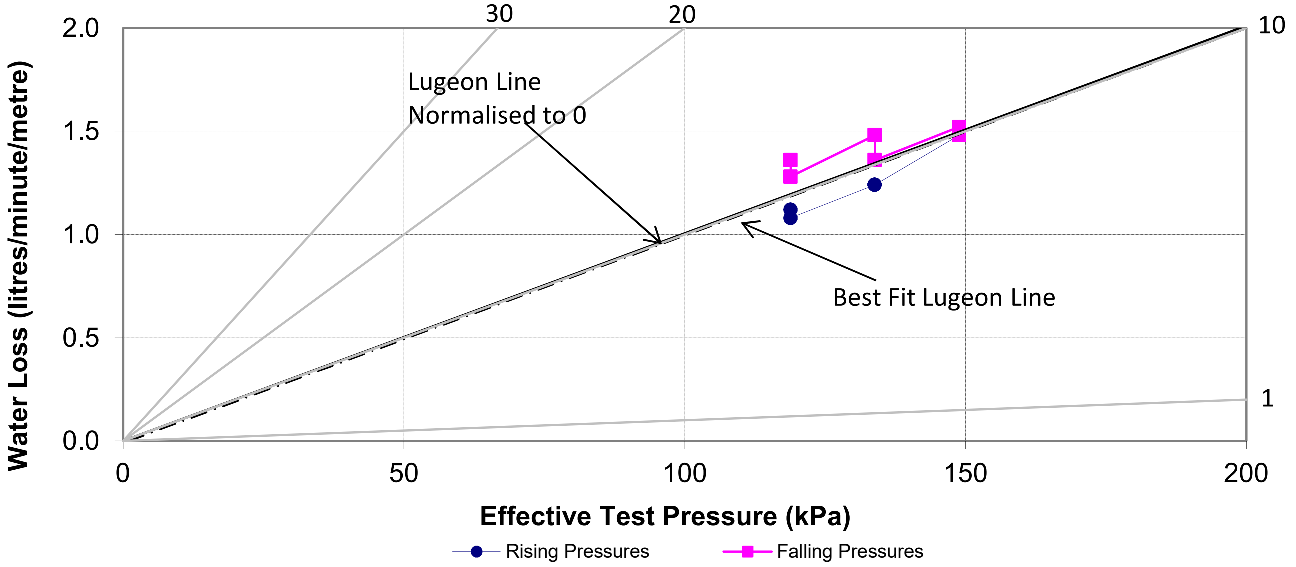
BHH112 - 6.85m - 11.85m

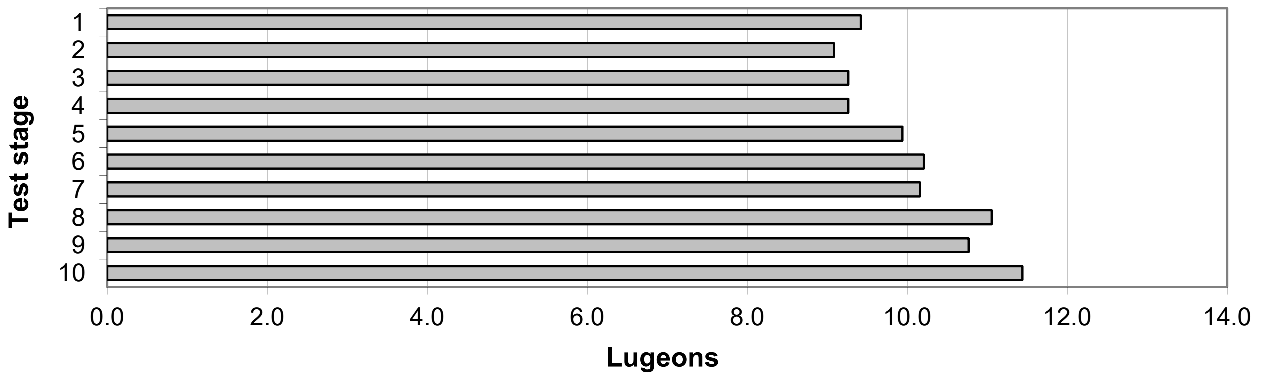
hole no:	BHH112	date:	12-Oct-15
project:	Coffs Harbour Bypass		
location:	Roberts Hill	by:	M Kobler

section tested: from 6.85 m to 11.85 m length 5.00 m

relative level at surface:	84.0 m RL	centre of test section (downhole):	9.4 m
gauge height:	1.3 m	test depth converted to vertical:	9.4 m
inclination of borehole:	90 °	relative level at centre of test section:	74.7 m RL
inferred groundwater level:	51.5 m RL	correction to gauge pressure (1)(2):	103.9 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	15	119	28.0	1.1	49%	9.4
5	15	119	27.0	1.1	49%	9.1
5	30	134	31.0	1.2	55%	9.3
5	30	134	31.0	1.2	55%	9.3
5	45	149	37.0	1.5	61%	9.9
5	45	149	38.0	1.5	61%	10.2
5	30	134	34.0	1.4	55%	10.2
5	30	134	37.0	1.5	55%	11.1
5	15	119	32.0	1.3	49%	10.8
5	15	119	34.0	1.4	49%	11.4





Interpreted flow mechanism: Wash out

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	No Core / Joints

Lugeon (uL) Assessment

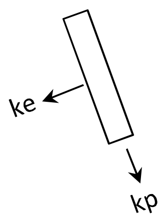
uL (Fell et al., 2005) - upper chart	10.1
uL (Houlsby, 1976) - lower chart	11.4

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	2.1.E-01	m/day
ke/kp = 100	1.8.E-01	m/day
ke/kp = 10	1.5.E-01	m/day
ke/kp = 1	1.2.E-01	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 1.3.E-01 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Test undertaken above water table

Job no. PSM2876

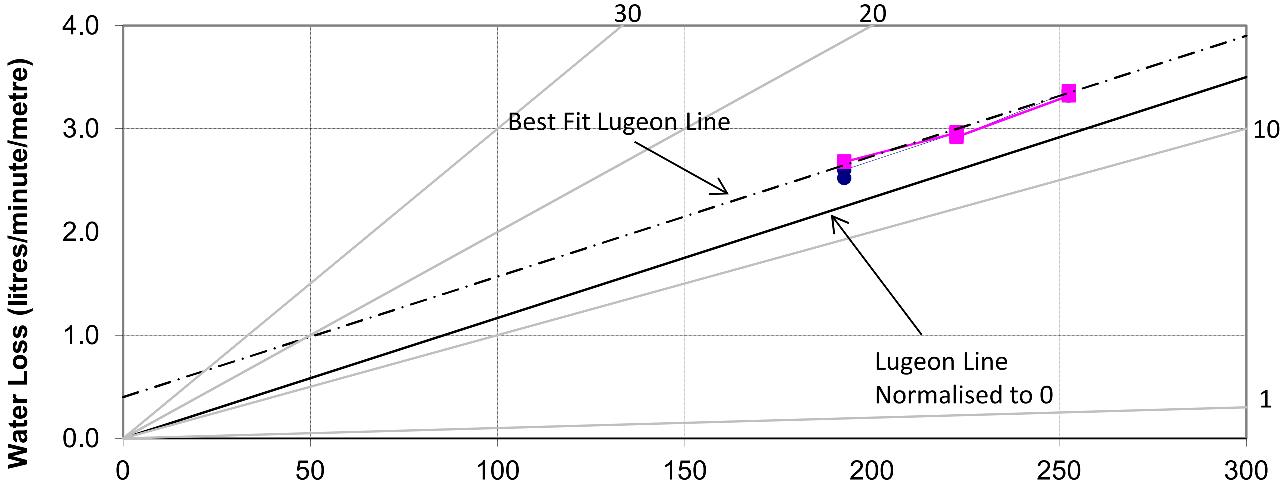
BHH112 - 12.85m - 17.85m

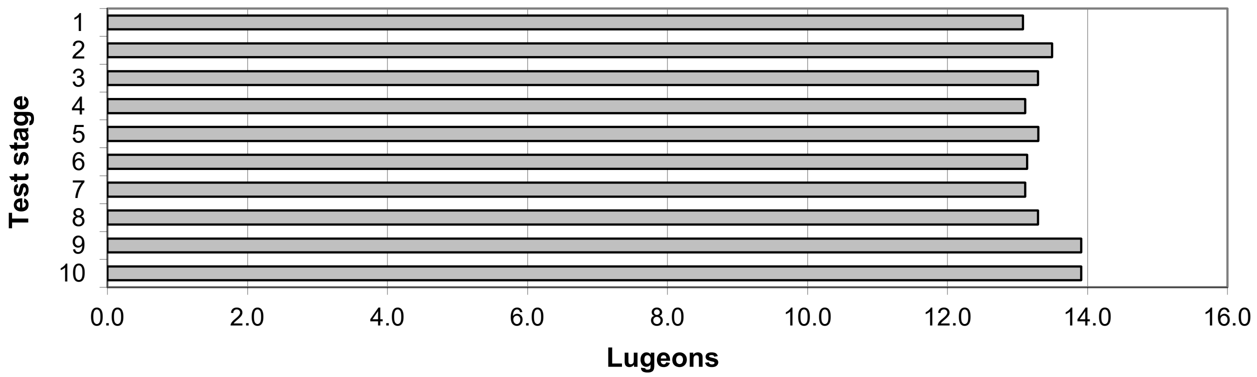
hole no:	BHH112	date:	12-Oct-15
project:	Coffs Harbour Bypass		
location:	Roberts Hill	by:	M Kobler

section tested: from 12.85 m to 17.85 m length 5.00 m

relative level at surface:	84.0 m RL	centre of test section (downhole):	15.4 m
gauge height:	1.3 m	test depth converted to vertical:	15.4 m
inclination of borehole:	90 °	relative level at centre of test section:	68.7 m RL
inferred groundwater level:	51.5 m RL	correction to gauge pressure [(1)(2)]:	162.7 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure [(3)]	lugeons litres / min / m / 1000kPa
5	30	193	63.0	2.5	48%	13.1
5	30	193	65.0	2.6	48%	13.5
5	60	223	74.0	3.0	56%	13.3
5	60	223	73.0	2.9	56%	13.1
5	90	253	84.0	3.4	63%	13.3
5	90	253	83.0	3.3	63%	13.1
5	60	223	73.0	2.9	56%	13.1
5	60	223	74.0	3.0	56%	13.3
5	30	193	67.0	2.7	48%	13.9
5	30	193	67.0	2.7	48%	13.9





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	No Core / Joints

Lugeon (uL) Assessment

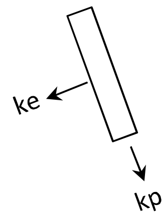
uL (Fell et al., 2005) - upper chart	11.7
uL (Houlsby, 1976) - lower chart	13.4

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	2.4.E-01	m/day
ke/kp = 100	2.1.E-01	m/day
ke/kp = 10	1.7.E-01	m/day
ke/kp = 1	1.4.E-01	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 1.5.E-01 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Test undertaken above water table

Job no. PSM2876

BHH112 - 18.85m - 23.85m

hole no: BHH112 | date: 14-Oct-16

project: Coffs Harbour Bypass

location: Roberts Hill | by: M Kobler

section tested: from 18.85 m to 23.85 m length 5.00 m

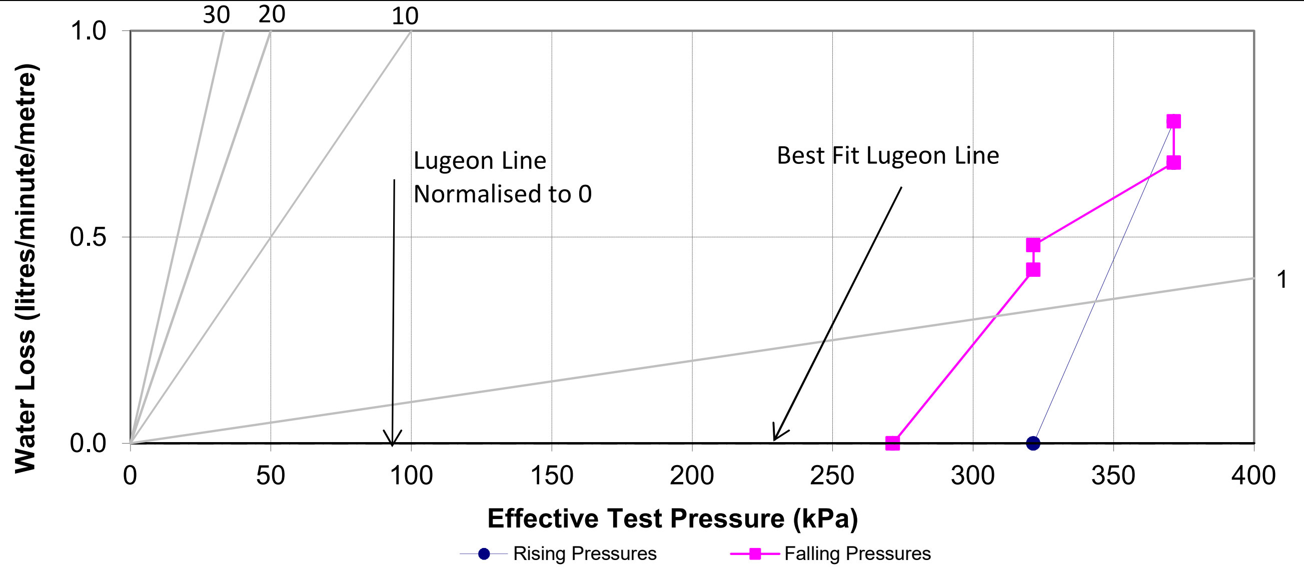
relative level at surface: 84.0 m RL | centre of test section (downhole): 21.4 m

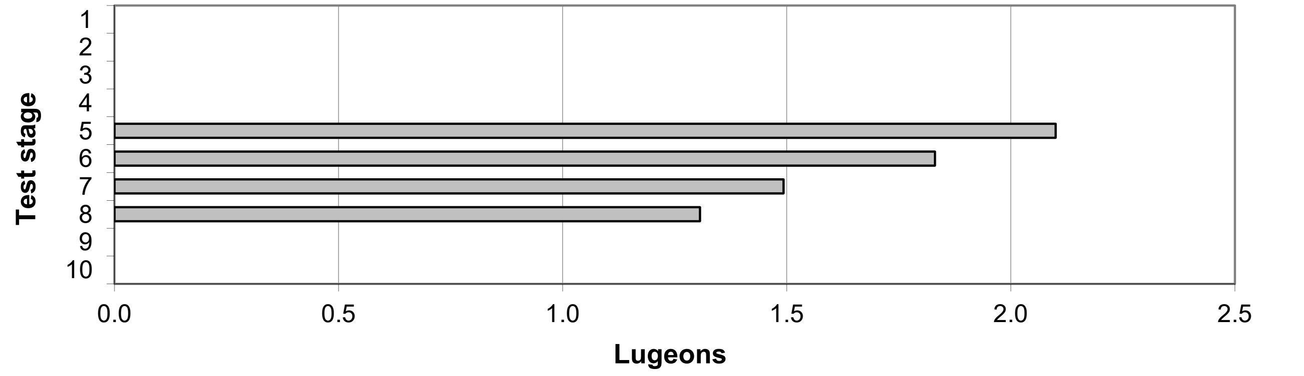
gauge height: 1.3 m | test depth converted to vertical: 21.4 m

inclination of borehole: 90 ° | relative level at centre of test section: 62.7 m RL

inferred groundwater level: 51.5 m RL | correction to gauge pressure (1)(2): 221.5 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	50	271	0.0	0.0	49%	0.0
5	50	271	0.0	0.0	49%	0.0
5	100	321	0.0	0.0	58%	0.0
5	100	321	0.0	0.0	58%	0.0
5	150	371	19.5	0.8	67%	2.1
5	150	371	17.0	0.7	67%	1.8
5	100	321	12.0	0.5	58%	1.5
5	100	321	10.5	0.4	58%	1.3
5	50	271	0.0	0.0	49%	0.0
5	50	271	0.0	0.0	49%	0.0





Interpreted flow mechanism: Dilation

Image of packer test section:

LEGEND:

Packer zone boundary

Core size: HQ3

Lithology: Argillite

Primary Defect Set: Joints

Lugeon (uL) Assessment

uL (Fell et al., 2005) - upper chart 0.0

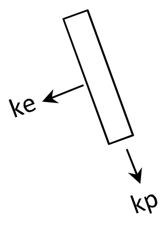
uL (Houlsby, 1976) - lower chart 0.3

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	6.4.E-03	m/day
ke/kp = 100	5.5.E-03	m/day
ke/kp = 10	4.6.E-03	m/day
ke/kp = 1	3.7.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 3.9.E-03 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Test undertaken above water table

Job no. PSM2876

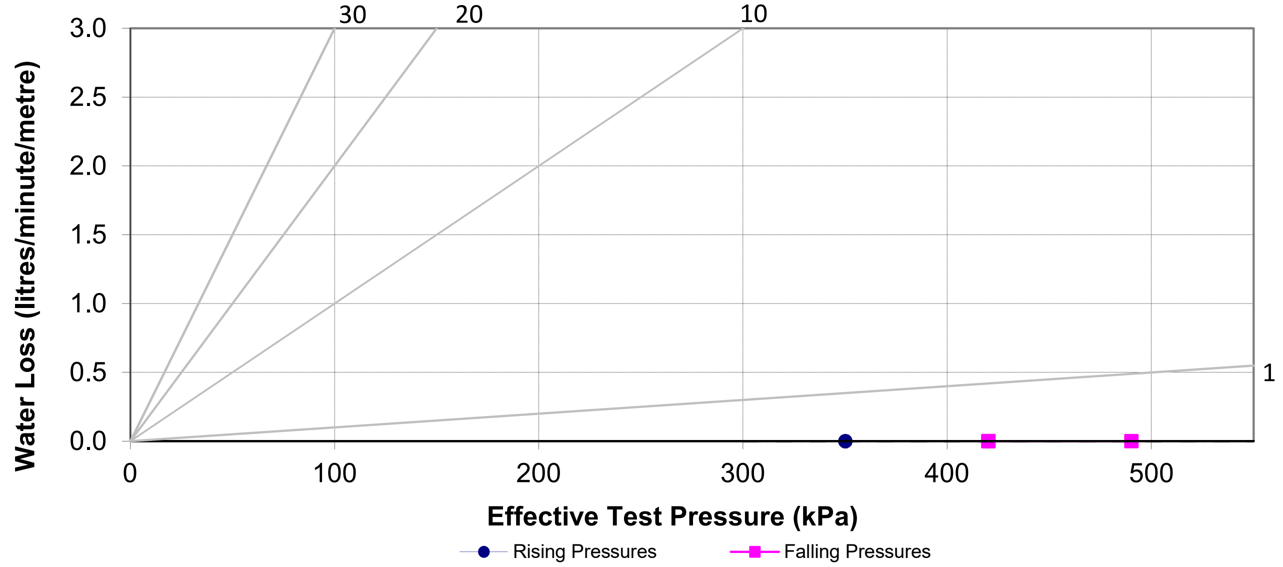
BHH112 - 24.85m - 29.85m

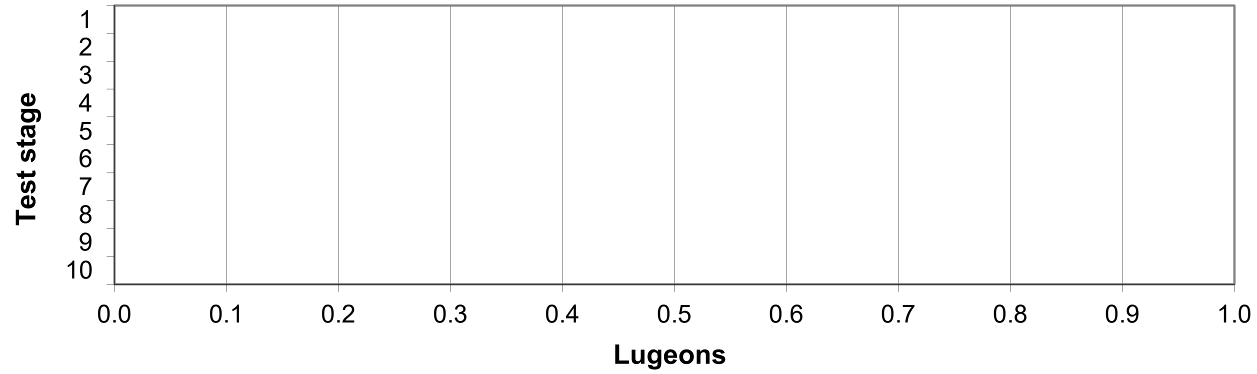
hole no:	BHH112	date:	14-Oct-16
project:	Coffs Harbour Bypass		
location:	Roberts Hill	by:	M Kobler

section tested: from 24.85 m to 29.85 m length 5.00 m

relative level at surface:	84.0 m RL	centre of test section (downhole):	27.4 m
gauge height:	1.3 m	test depth converted to vertical:	27.4 m
inclination of borehole:	90 °	relative level at centre of test section:	56.7 m RL
inferred groundwater level:	51.5 m RL	correction to gauge pressure (1)(2):	280.3 kPa

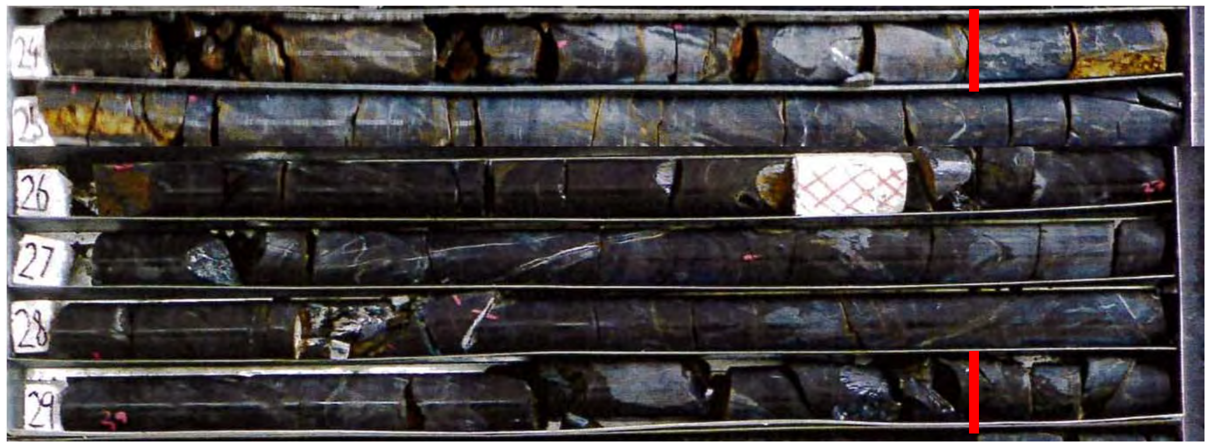
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	70	350	0.0	0.0	49%	0.0
5	70	350	0.0	0.0	49%	0.0
5	140	420	0.0	0.0	59%	0.0
5	140	420	0.0	0.0	59%	0.0
5	210	490	0.0	0.0	69%	0.0
5	210	490	0.0	0.0	69%	0.0
5	140	420	0.0	0.0	59%	0.0
5	140	420	0.0	0.0	59%	0.0
5	70	350	0.0	0.0	49%	0.0
5	70	350	0.0	0.0	49%	0.0





Interpreted flow mechanism: No Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

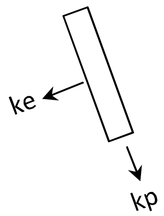
uL (Fell et al., 2005) - upper chart	0.0
uL (Houlsby, 1976) - lower chart	0.0

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	0.0.E+00	m/day
ke/kp = 100	0.0.E+00	m/day
ke/kp = 10	0.0.E+00	m/day
ke/kp = 1	0.0.E+00	m/day
ke value using Moye Method (1967) - equivalent to ke/kp = 1:	0.0.E+00	m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

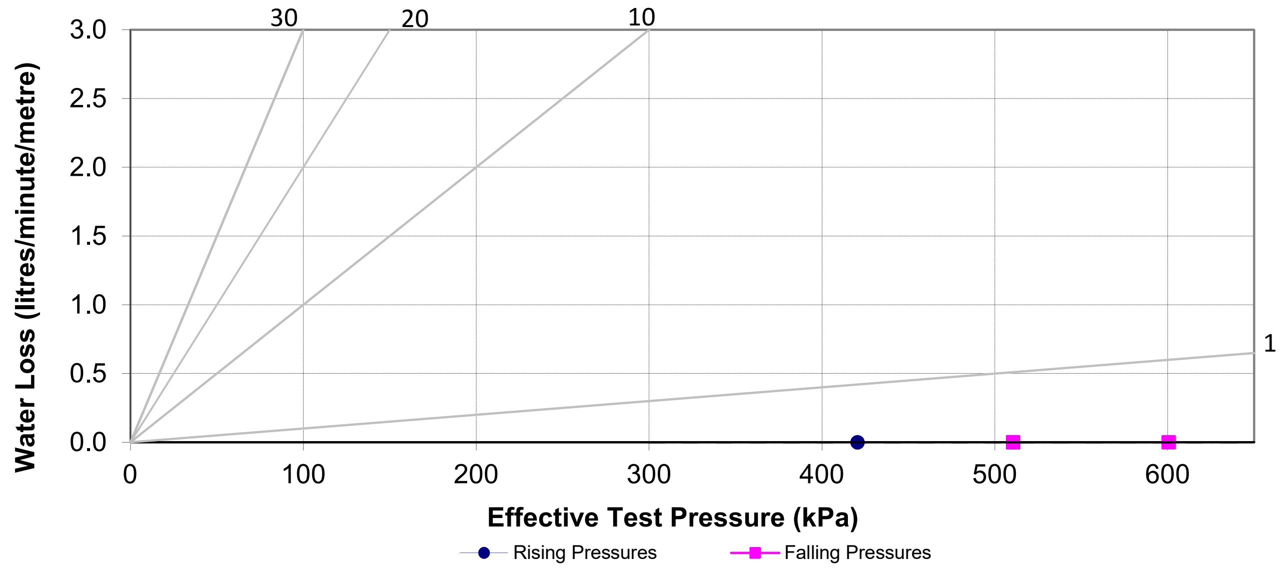
BHH112 - 30.85m - 35.85m

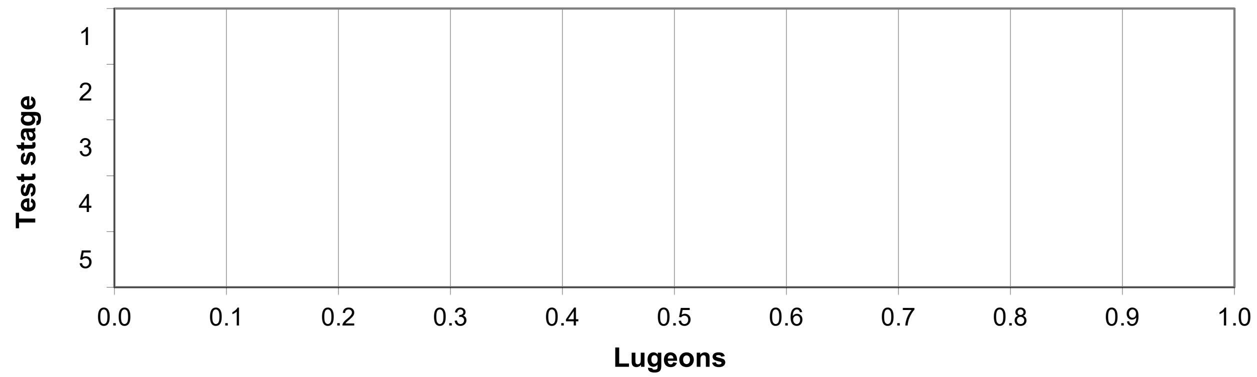
hole no:	BHH112	date:	14-Oct-16
project:	Coffs Harbour Bypass		
location:	Roberts Hill	by:	M Kobler

section tested: from 30.85 m to 35.85 m length 5.00 m

relative level at surface:	84.0 m RL	centre of test section (downhole):	33.4 m
gauge height:	1.3 m	test depth converted to vertical:	33.4 m
inclination of borehole:	90 °	relative level at centre of test section:	50.7 m RL
inferred groundwater level:	51.5 m RL	correction to gauge pressure (1)(2):	330.8 kPa

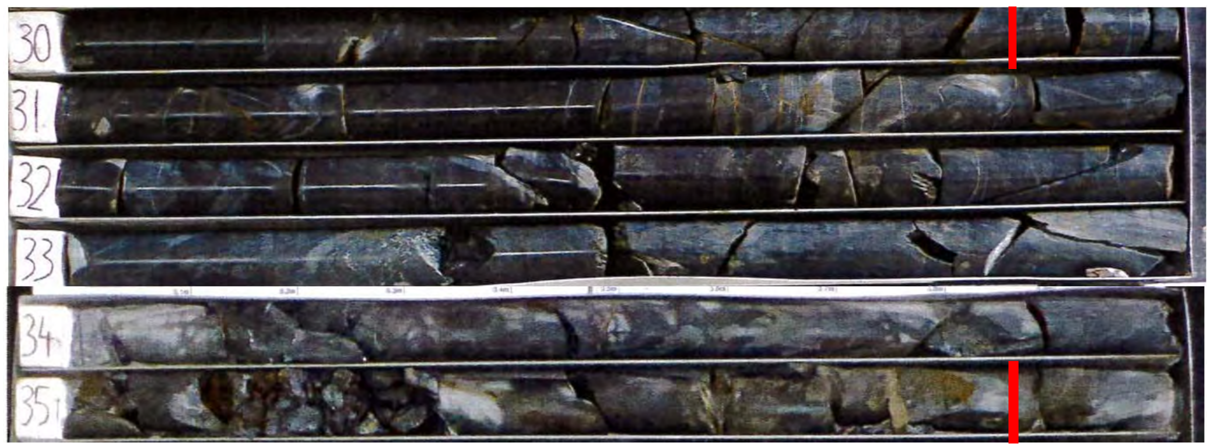
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	90	421	0.0	0.0	49%	0.0
5	90	421	0.0	0.0	49%	0.0
5	180	511	0.0	0.0	59%	0.0
5	180	511	0.0	0.0	59%	0.0
5	270	601	0.0	0.0	69%	0.0





Interpreted flow mechanism: No Flow

Image of packer test section:



LEGEND:
Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

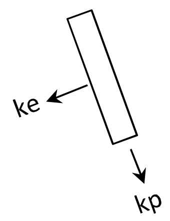
uL (Fell et al., 2005) - upper chart	0.0
uL (Houlsby, 1976) - lower chart	0.0

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	0.0.E+00	m/day
ke/kp = 100	0.0.E+00	m/day
ke/kp = 10	0.0.E+00	m/day
ke/kp = 1	0.0.E+00	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 0.0.E+00 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

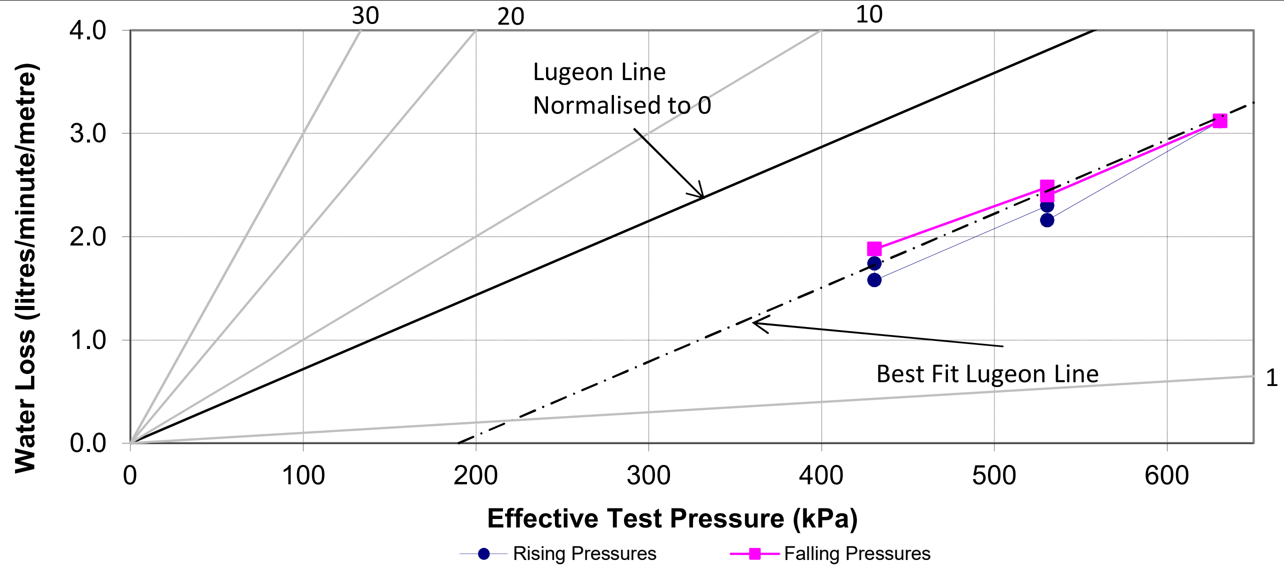
BHH112 - 36.85m - 41.85m

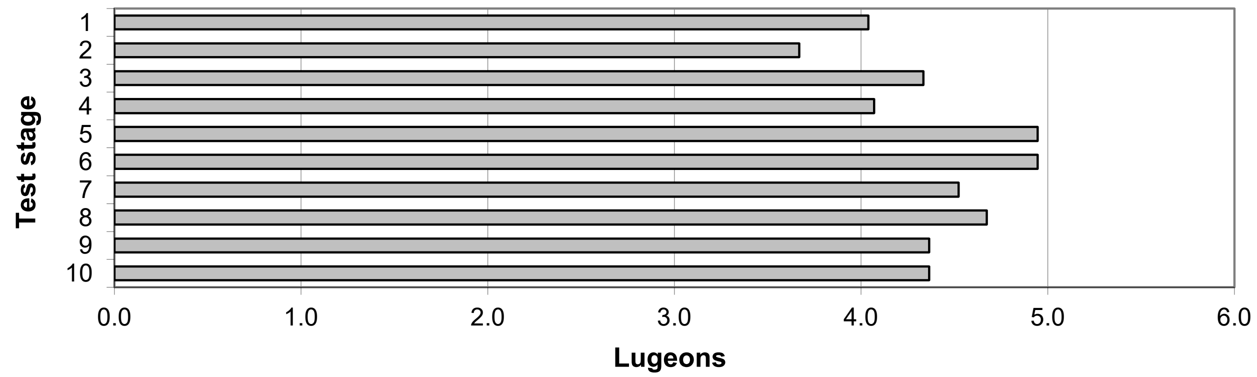
hole no:	BHH112	date:	14-Oct-16
project:	Coffs Harbour Bypass		
location:	Roberts Hill	by:	M Kobler

section tested: from 36.85 m to 41.85 m length 5.00 m

relative level at surface:	84.0 m RL	centre of test section (downhole):	39.4 m
gauge height:	1.3 m	test depth converted to vertical:	39.4 m
inclination of borehole:	90 °	relative level at centre of test section:	44.7 m RL
inferred groundwater level:	51.5 m RL	correction to gauge pressure (1)(2):	330.8 kPa

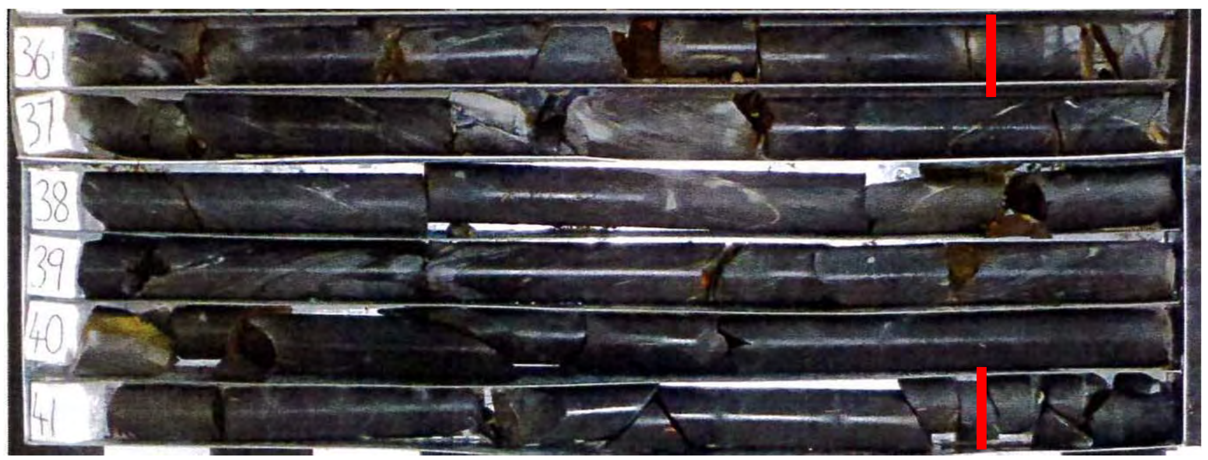
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	100	431	43.5	1.7	42%	4.0
5	100	431	39.5	1.6	42%	3.7
5	200	531	57.5	2.3	52%	4.3
5	200	531	54.0	2.2	52%	4.1
5	300	631	78.0	3.1	62%	4.9
5	300	631	78.0	3.1	62%	4.9
5	200	531	60.0	2.4	52%	4.5
5	200	531	62.0	2.5	52%	4.7
5	100	431	47.0	1.9	42%	4.4
5	100	431	47.0	1.9	42%	4.4





Interpreted flow mechanism: Laminar Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

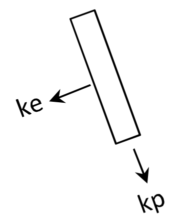
uL (Fell et al., 2005) - upper chart	7.2
uL (Houlsby, 1976) - lower chart	4.4

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	8.0.E-02	m/day
ke/kp = 100	6.9.E-02	m/day
ke/kp = 10	5.7.E-02	m/day
ke/kp = 1	4.6.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 4.9.E-02 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

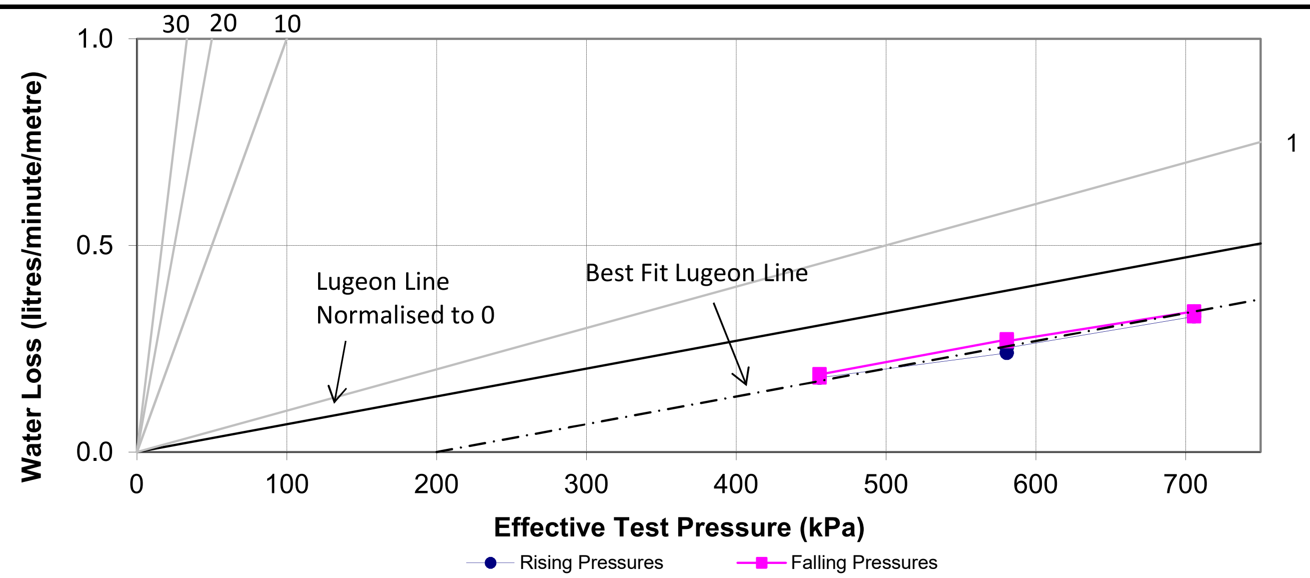
BHH112 - 42.85m - 47.85m

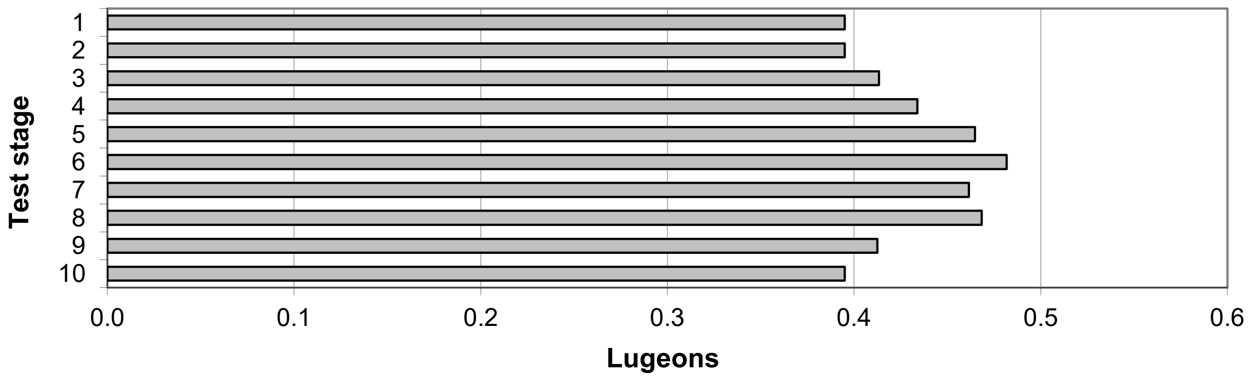
hole no:	BHH112	date:	14-Oct-16
project:	Coffs Harbour Bypass		
location:	Roberts Hill	by:	M Kobler

section tested: from 42.85 m to 47.85 m length 5.00 m

relative level at surface:	84.0 m RL	centre of test section (downhole):	45.4 m
gauge height:	1.3 m	test depth converted to vertical:	45.4 m
inclination of borehole:	90 °	relative level at centre of test section:	38.7 m RL
inferred groundwater level:	51.5 m RL	correction to gauge pressure [(1)(2)]:	330.8 kPa

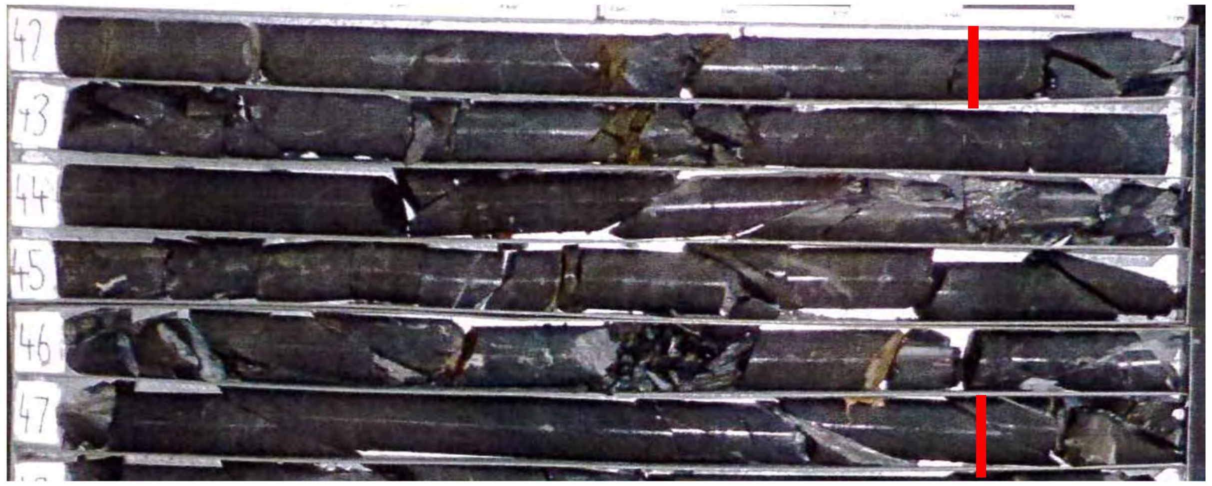
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure [(3)]	lugeons litres / min / m / 1000kPa
5	125	456	4.5	0.2	39%	0.4
5	125	456	4.5	0.2	39%	0.4
5	250	581	6.0	0.2	49%	0.4
5	250	581	6.3	0.3	49%	0.4
5	375	706	8.2	0.3	60%	0.5
5	375	706	8.5	0.3	60%	0.5
5	250	581	6.7	0.3	49%	0.5
5	250	581	6.8	0.3	49%	0.5
5	125	456	4.7	0.2	39%	0.4
5	125	456	4.5	0.2	39%	0.4





Interpreted flow mechanism: Laminar Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

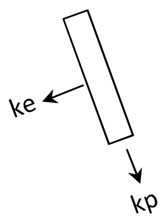
uL (Fell et al., 2005) - upper chart	0.7
uL (Houlsby, 1976) - lower chart	0.4

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	7.9.E-03	m/day
ke/kp = 100	6.7.E-03	m/day
ke/kp = 10	5.6.E-03	m/day
ke/kp = 1	4.5.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 4.8.E-03 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink Job no. PSM2876

Borehole Water Pressure Test Summary

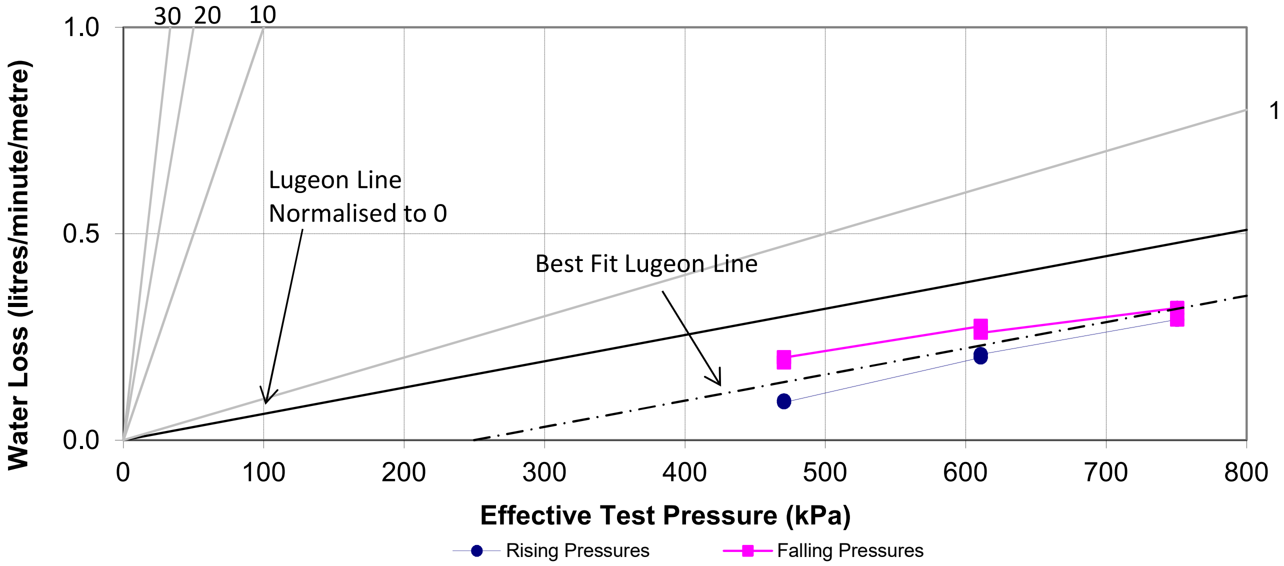
BHH112 - 48.85m - 53.85m

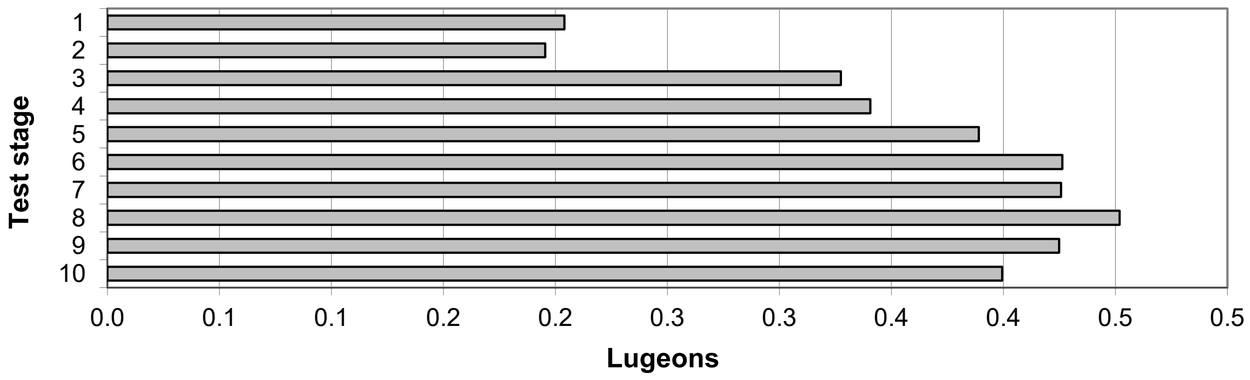
hole no:	BHH112	date:	14-Oct-16
project:	Coffs Harbour Bypass		
location:	Roberts Hill	by:	M Kobler

section tested: from 48.85 m to 53.85 m length 5.00 m

relative level at surface:	84.0 m RL	centre of test section (downhole):	51.4 m
gauge height:	1.3 m	test depth converted to vertical:	51.4 m
inclination of borehole:	90 °	relative level at centre of test section:	32.7 m RL
inferred groundwater level:	51.5 m RL	correction to gauge pressure (1)(2):	330.8 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	140	471	2.4	0.1	35%	0.2
5	140	471	2.3	0.1	35%	0.2
5	280	611	5.0	0.2	46%	0.3
5	280	611	5.2	0.2	46%	0.3
5	420	751	7.3	0.3	56%	0.4
5	420	751	8.0	0.3	56%	0.4
5	280	611	6.5	0.3	46%	0.4
5	280	611	6.9	0.3	46%	0.5
5	140	471	5.0	0.2	35%	0.4
5	140	471	4.7	0.2	35%	0.4





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

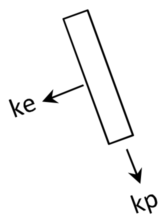
uL (Fell et al., 2005) - upper chart	0.6
uL (Houlsby, 1976) - lower chart	0.4

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	6.5.E-03	m/day
ke/kp = 100	5.6.E-03	m/day
ke/kp = 10	4.7.E-03	m/day
ke/kp = 1	3.7.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 4.0.E-03 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

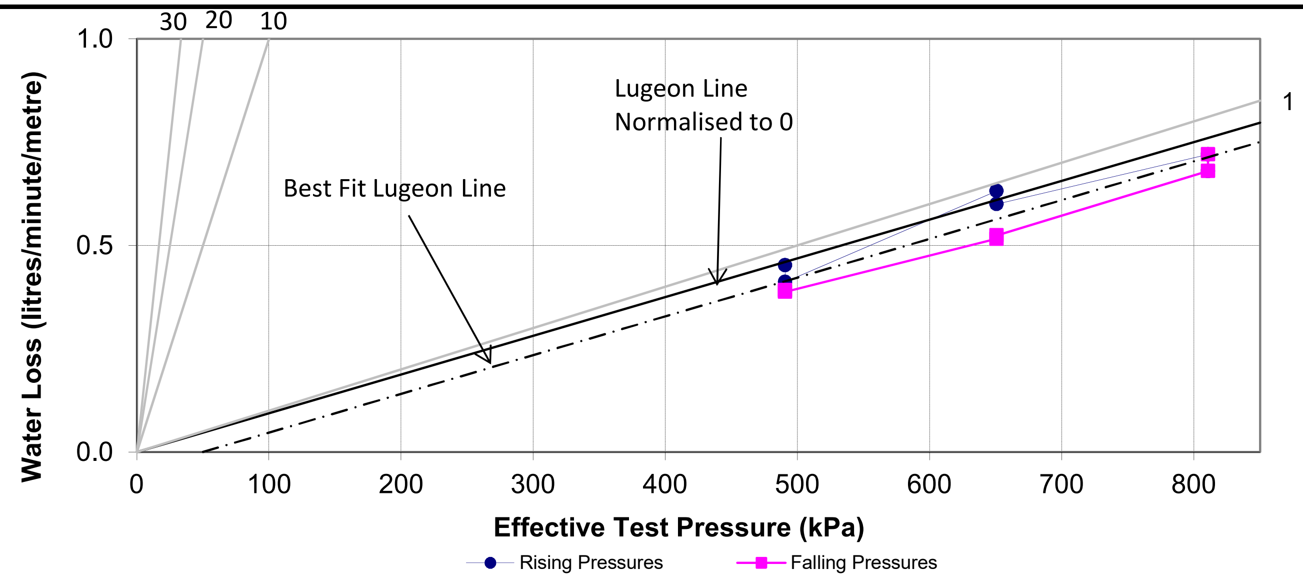
BHH112 - 54.85m - 59.85m

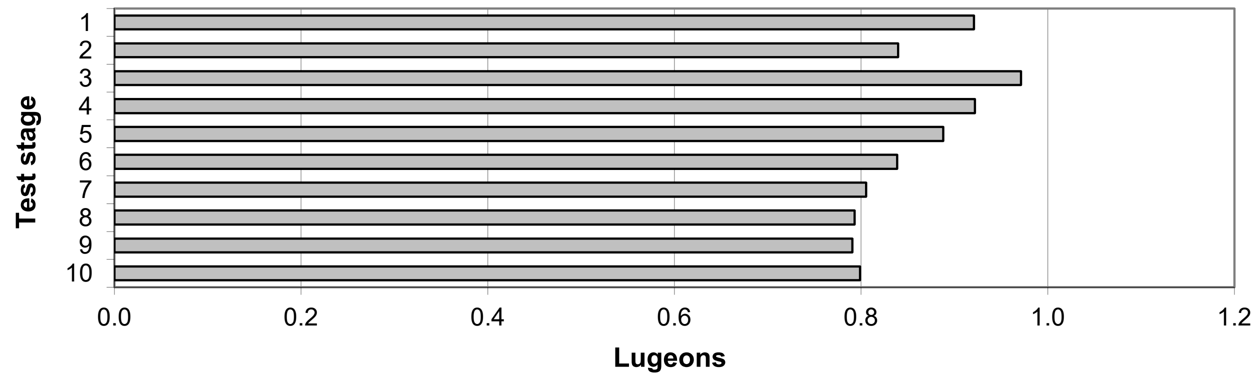
hole no:	BHH112	date:	14-Oct-16
project:	Coffs Harbour Bypass		
location:	Roberts Hill	by:	M Kobler

section tested: from 54.85 m to 59.85 m length 5.00 m

relative level at surface:	84.0 m RL	centre of test section (downhole):	57.4 m
gauge height:	1.3 m	test depth converted to vertical:	57.4 m
inclination of borehole:	90 °	relative level at centre of test section:	26.7 m RL
inferred groundwater level:	51.5 m RL	correction to gauge pressure (1)(2):	330.8 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	160	491	11.3	0.5	33%	0.9
5	160	491	10.3	0.4	33%	0.8
5	320	651	15.8	0.6	44%	1.0
5	320	651	15.0	0.6	44%	0.9
5	480	811	18.0	0.7	54%	0.9
5	480	811	17.0	0.7	54%	0.8
5	320	651	13.1	0.5	44%	0.8
5	320	651	12.9	0.5	44%	0.8
5	160	491	9.7	0.4	33%	0.8
5	160	491	9.8	0.4	33%	0.8





Interpreted flow mechanism: Void filling

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

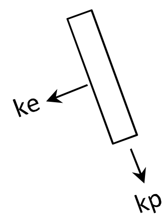
uL (Fell et al., 2005) - upper chart	0.9
uL (Houlsby, 1976) - lower chart	0.8

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	1.5.E-02	m/day
ke/kp = 100	1.2.E-02	m/day
ke/kp = 10	1.0.E-02	m/day
ke/kp = 1	8.3.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 8.9.E-03 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

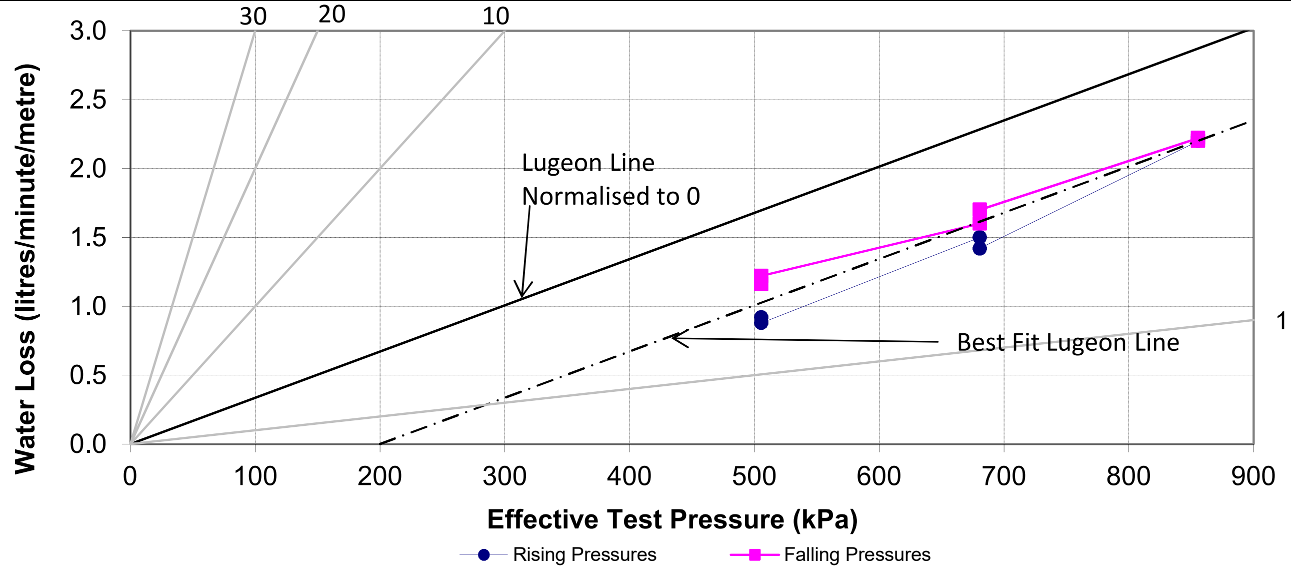
BHH112 - 60.85m - 65.85m

hole no:	BHH112	date:	14-Oct-16
project:	Coffs Harbour Bypass		
location:	Roberts Hill	by:	M Kobler

section tested: from 60.85 m to 65.85 m length 5.00 m

relative level at surface:	84.0 m RL	centre of test section (downhole):	63.4 m
gauge height:	1.3 m	test depth converted to vertical:	63.4 m
inclination of borehole:	90 °	relative level at centre of test section:	20.7 m RL
inferred groundwater level:	51.5 m RL	correction to gauge pressure [(1)(2)]:	330.8 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure [(3)]	lugeons litres / min / m / 1000kPa
5	175	506	23.0	0.9	31%	1.8
5	175	506	22.0	0.9	31%	1.7
5	350	681	37.5	1.5	41%	2.2
5	350	681	35.5	1.4	41%	2.1
5	525	856	55.0	2.2	52%	2.6
5	525	856	55.5	2.2	52%	2.6
5	350	681	42.5	1.7	41%	2.5
5	350	681	40.0	1.6	41%	2.4
5	175	506	30.5	1.2	31%	2.4
5	175	506	29.0	1.2	31%	2.3



Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

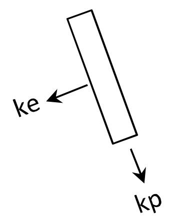
uL (Fell et al., 2005) - upper chart	3.4
uL (Houlsby, 1976) - lower chart	2.3

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	4.1.E-02	m/day
ke/kp = 100	3.5.E-02	m/day
ke/kp = 10	2.9.E-02	m/day
ke/kp = 1	2.4.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.5.E-02 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

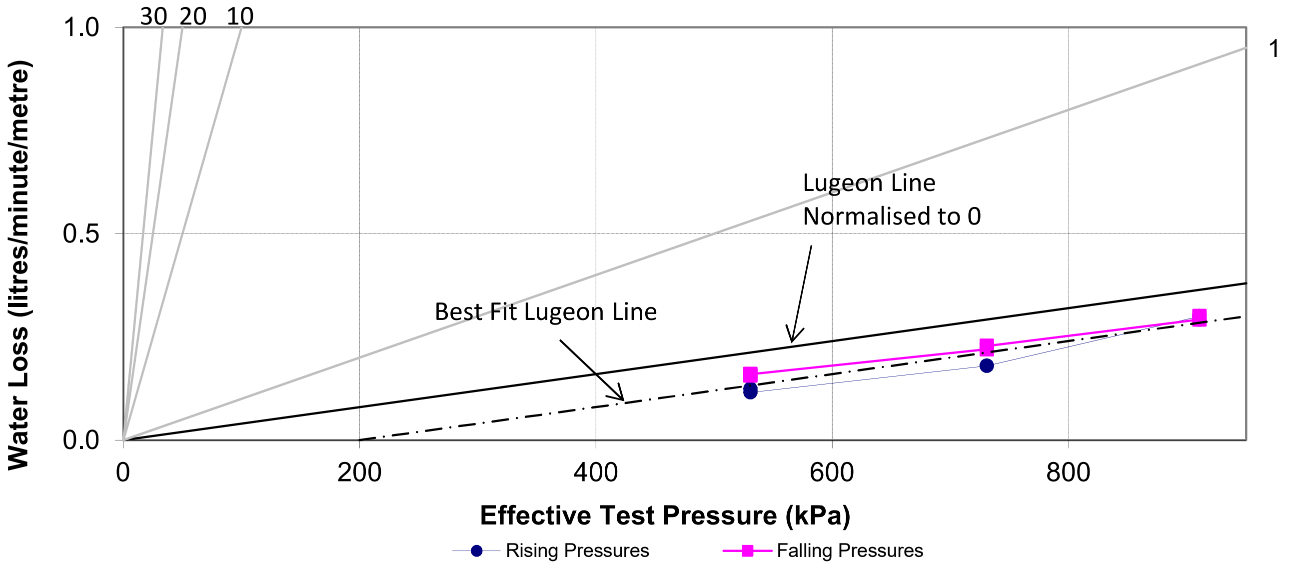
BHH112 - 66.85m - 71.85m

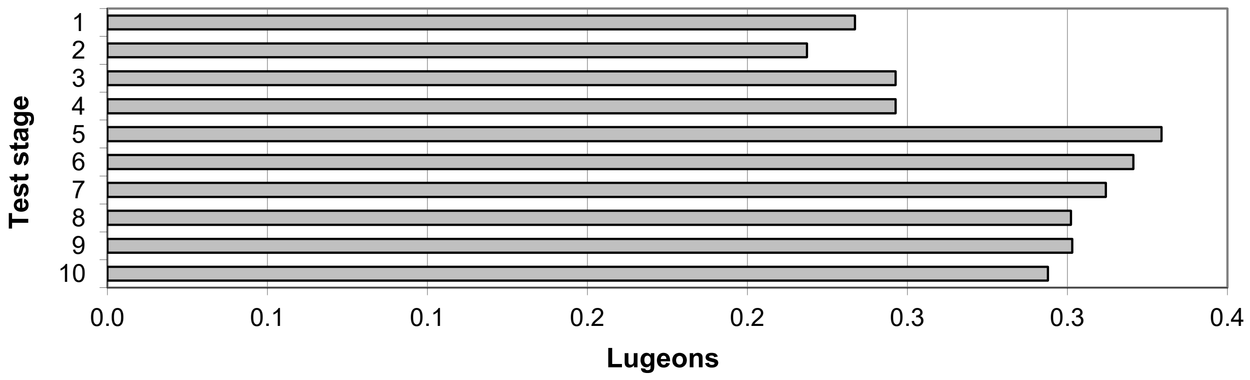
hole no:	BHH112	date:	14-Oct-16
project:	Coffs Harbour Bypass		
location:	Roberts Hill	by:	M Kobler

section tested: from 66.85 m to 71.85 m length 5.00 m

relative level at surface:	84.0 m RL	centre of test section (downhole):	69.4 m
gauge height:	1.3 m	test depth converted to vertical:	69.4 m
inclination of borehole:	90 °	relative level at centre of test section:	14.7 m RL
inferred groundwater level:	51.5 m RL	correction to gauge pressure (1)(2):	330.8 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	200	531	3.1	0.1	29%	0.2
5	200	531	2.9	0.1	29%	0.2
5	400	731	4.5	0.2	41%	0.2
5	400	731	4.5	0.2	41%	0.2
5	580	911	7.5	0.3	51%	0.3
5	580	911	7.3	0.3	51%	0.3
5	400	731	5.7	0.2	41%	0.3
5	400	731	5.5	0.2	41%	0.3
5	200	531	4.0	0.2	29%	0.3
5	200	531	3.9	0.2	29%	0.3





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size: HQ3

Lithology: Argillite

Primary Defect Set: Joints

Lugeon (uL) Assessment

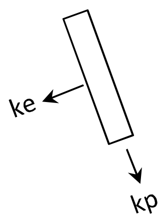
uL (Fell et al., 2005) - upper chart	0.4
uL (Houlsby, 1976) - lower chart	0.3

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	5.1.E-03	m/day
ke/kp = 100	4.4.E-03	m/day
ke/kp = 10	3.6.E-03	m/day
ke/kp = 1	2.9.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 3.1.E-03 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

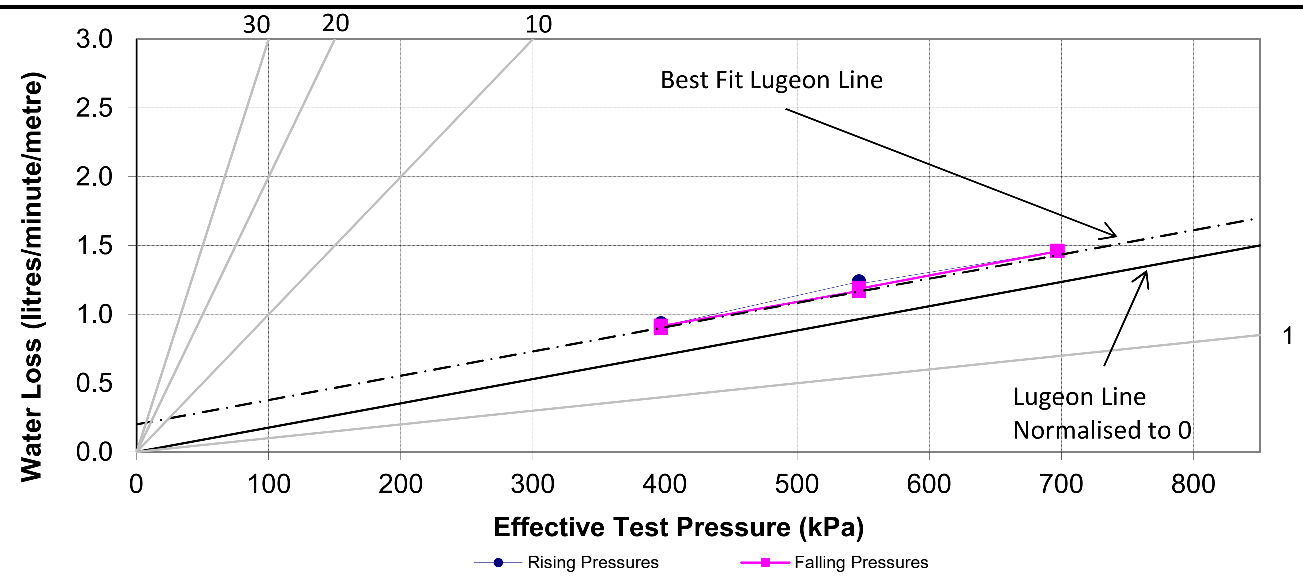
BHH113 - 50.1m - 56m

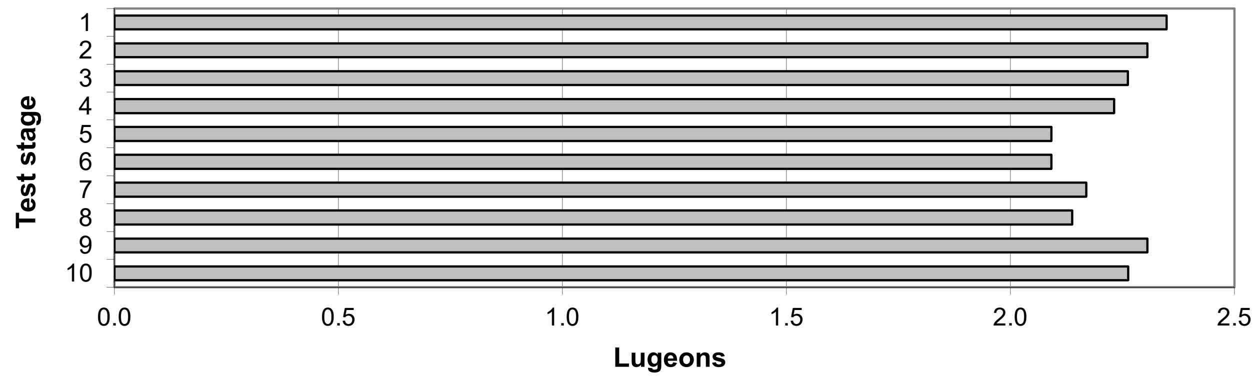
hole no:	BHH113	date:	24-Oct-16
project:	Coffs Harbour Bypass		
location:	Roberts Hill	by:	M Kobler

section tested: from 50.10 m to 56.00 m length 5.90 m

relative level at surface:	85.5 m RL	centre of test section (downhole):	53.1 m
gauge height:	1.0 m	test depth converted to vertical:	49.9 m
inclination of borehole:	70 °	relative level at centre of test section:	35.6 m RL
inferred groundwater level:	61.3 m RL	correction to gauge pressure [(1)(2)]:	247.0 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure [(3)]	lugeons litres / min / m / 1000kPa
5	150	397	27.5	0.9	31%	2.3
5	150	397	27.0	0.9	31%	2.3
5	300	547	36.5	1.2	42%	2.3
5	300	547	36.0	1.2	42%	2.2
5	450	697	43.0	1.5	54%	2.1
5	450	697	43.0	1.5	54%	2.1
5	300	547	35.0	1.2	42%	2.2
5	300	547	34.5	1.2	42%	2.1
5	150	397	27.0	0.9	31%	2.3
5	150	397	26.5	0.9	31%	2.3





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

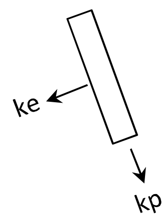
uL (Fell et al., 2005) - upper chart	1.8
uL (Houlsby, 1976) - lower chart	2.2

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	4.1.E-02	m/day
ke/kp = 100	3.5.E-02	m/day
ke/kp = 10	3.0.E-02	m/day
ke/kp = 1	2.4.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.6.E-02 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

A2 – Shephards Lane Packer Testing Data

A2 – Packer Test Summary Table – Shephards Lane

BHID	Test From (m)	Test To (m)	Lugeon Value 1 (1)	Lugeon Value 2 (2)	Lugeon Classification	Weathering	RMU	Hydraulic Conductivity (m/d) (3)
BHH138	5	19.5	2.8	4.1	Low/Mod	SW	C1	5.60×10^{-2}
BHH138	19	30	0	0	No flow	F	C2	3.60×10^{-5}
BHH139	10.8	28.6	0.12	0.13	Low	SW	C1	1.80×10^{-3}
BHH139	26.6	39.1	0.05	0.01	Low	F	C2	1.20×10^{-4}
BHH139	38.6	60.1	0.05	0.02	Low	F	C2	2.00×10^{-4}
BHH140	46.25	60.65	6.7	3.5	Low/Mod to Mod/High	F	C2	4.80×10^{-2}
BHH140	55.25	60.65	4.1	3.2	Low/Mod	F	C2	3.70×10^{-2}
BHH140	61.25	69.65	0.35	0.18	Low	F	C2	2.20×10^{-3}
BHH140	70.25	82.65	11.3	2.4	Low/Mod to Mod/High	F	C2	3.20×10^{-2}
BHH140	76.25	82.65	4.4	3.2	Low/Mod	F	C2	3.70×10^{-2}
BHH141	17.32	32.3	0.18	0.06	Low	SW	B1	7.90×10^{-4}
BHH141	32.3	48.1	0.06	0.02	Low	F	B2	3.30×10^{-4}
BHH141	48.8	66.1	0.04	0.02	Low	F	B2	2.20×10^{-4}
BHH141	66.8	84.1	0.03	0.01	Low	F	B2	2.00×10^{-4}
BHH142	31.2	48.65	0.5	0.3	Low	SW	B1	4.10×10^{-3}
BHH142	64.85	80.7	0.3	0.1	Low	F	B2	2.00×10^{-3}
BHH143	23	37.62	0.01	0	Low Flow	SW	B1	1.10×10^{-5}
BHH143	39	55.52	0.03	0.02	Low	F	B2	2.60×10^{-4}

BHID	Test From (m)	Test To (m)	Lugeon Value 1 [(1)]	Lugeon Value 2 [(2)]	Lugeon Classification	Weathering	RMU	Hydraulic Conductivity (m/d) [(3)]
BHH144	22.2	33.2	2.7	1.7	Low/Mod	SW	B1	2.20×10^{-2}
BHH144	32	40	0.7	0.4	Low	F	B2	5.40×10^{-3}

Notes:

[1] Calculated from Fell et al., 2005

[2] Calculated from Houlsby, 1976

[3] Calculated from Moye, 1967

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

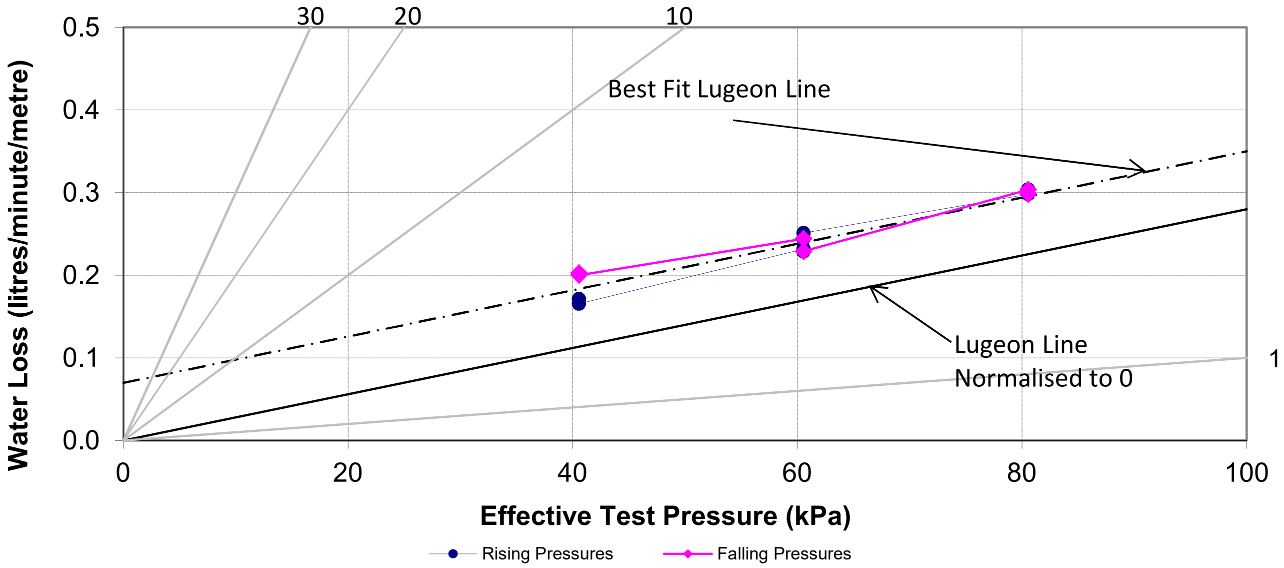
BHH138 - 5m - 19.5m

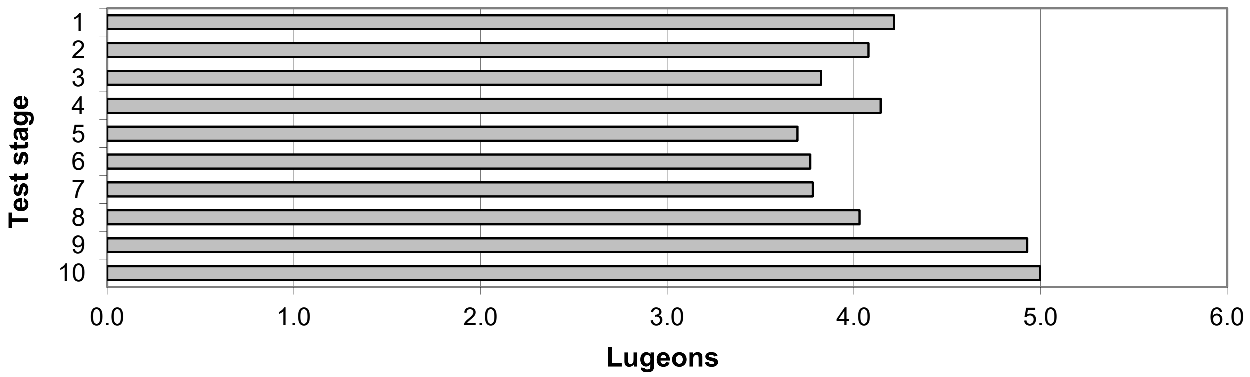
hole no:	BHH138	date:	23-Nov-16
project:	Coffs Harbour Bypass		
location:	Shephards Lane	by:	R Cater

section tested: from 5.00 m to 19.50 m length 14.50 m

relative level at surface:	97.0 m RL	centre of test section (downhole):	12.3 m
gauge height:	0.4 m	test depth converted to vertical:	12.3 m
inclination of borehole:	90 °	relative level at centre of test section:	84.8 m RL
inferred groundwater level:	95.3 m RL	correction to gauge pressure (1)(2):	20.6 kPa

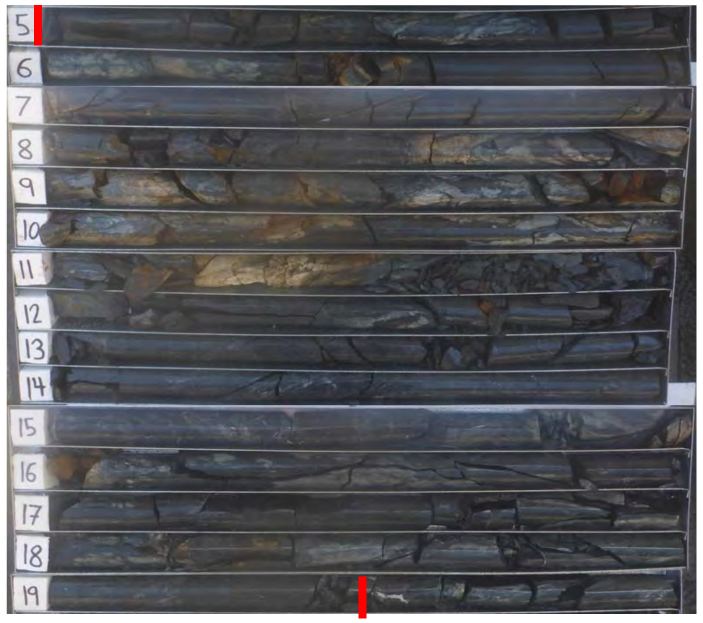
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	20	41	12.4	0.2	13%	4.2
5	20	41	12.0	0.2	13%	4.1
5	40	61	16.8	0.2	19%	3.8
5	40	61	18.2	0.3	19%	4.1
5	60	81	21.6	0.3	25%	3.7
5	60	81	22.0	0.3	25%	3.8
5	40	61	16.6	0.2	19%	3.8
5	40	61	17.7	0.2	19%	4.0
5	20	41	14.5	0.2	13%	4.9
5	20	41	14.7	0.2	13%	5.0





Interpreted flow mechanism: Laminar Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size: HQ3

Lithology: Argillite

Primary Defect Set: Joints

Lugeon (uL) Assessment

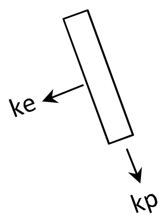
uL (Fell et al., 2005) - upper chart	2.8
uL (Houlsby, 1976) - lower chart	4.1

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	8.5.E-02	m/day
ke/kp = 100	7.5.E-02	m/day
ke/kp = 10	6.4.E-02	m/day
ke/kp = 1	5.3.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 5.6.E-02 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink Job no. PSM2876

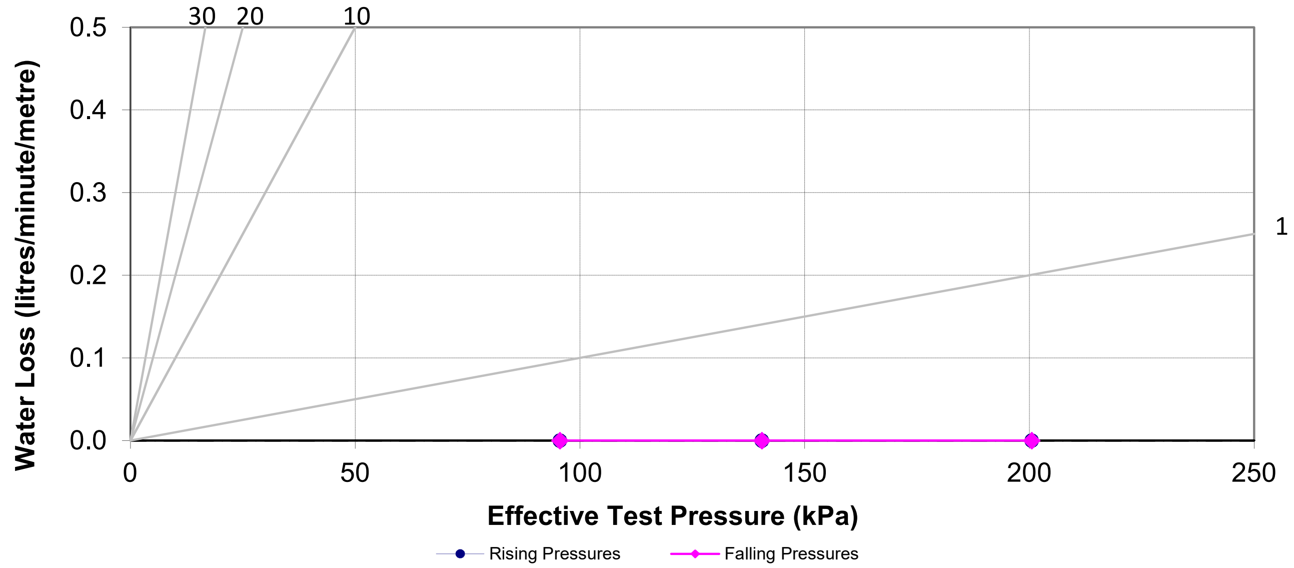
Borehole Water Pressure Test Summary **BHH138 - 19m - 30m**

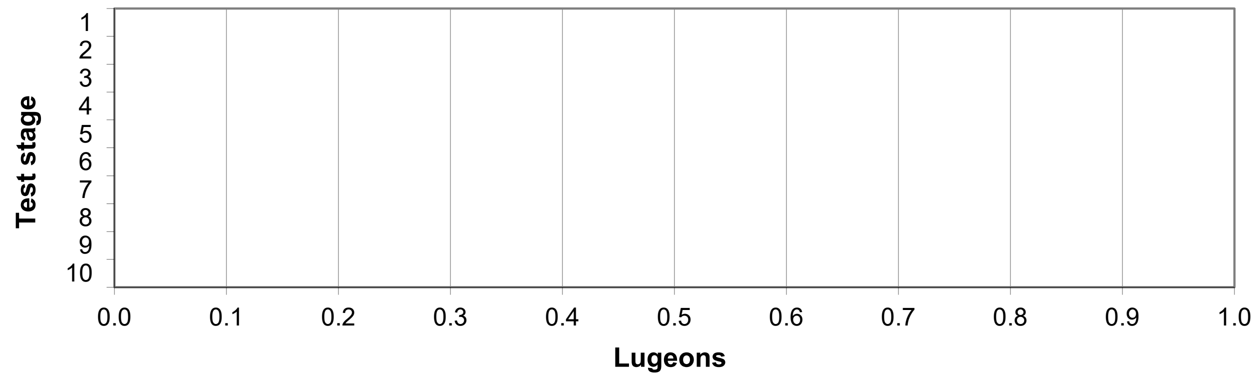
hole no:	BHH138	date:	24-Nov-16
project:	Coffs Harbour Bypass		
location:	Shephards Lane	by:	R Cater

section tested: from 19.00 m to 30.00 m length 11.00 m

relative level at surface:	97.0 m RL	centre of test section (downhole):	24.5 m
gauge height:	0.4 m	test depth converted to vertical:	24.5 m
inclination of borehole:	90 °	relative level at centre of test section:	72.5 m RL
inferred groundwater level:	95.3 m RL	correction to gauge pressure (1)(2):	20.6 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	75	96	0.0	0.0	15%	0.0
5	75	96	0.0	0.0	15%	0.0
5	120	141	0.0	0.0	22%	0.0
5	120	141	0.0	0.0	22%	0.0
5	180	201	0.0	0.0	31%	0.0
5	180	201	0.0	0.0	31%	0.0
5	120	141	0.0	0.0	22%	0.0
5	120	141	0.0	0.0	22%	0.0
5	75	96	0.0	0.0	15%	0.0
5	75	96	0.0	0.0	15%	0.0





Interpreted flow mechanism: No Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

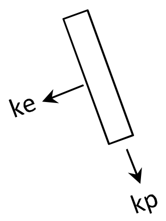
uL (Fell et al., 2005) - upper chart	0.0
uL (Houlsby, 1976) - lower chart	0.0

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	0.0.E+00	m/day
ke/kp = 100	0.0.E+00	m/day
ke/kp = 10	0.0.E+00	m/day
ke/kp = 1	0.0.E+00	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 0.0.E+00 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

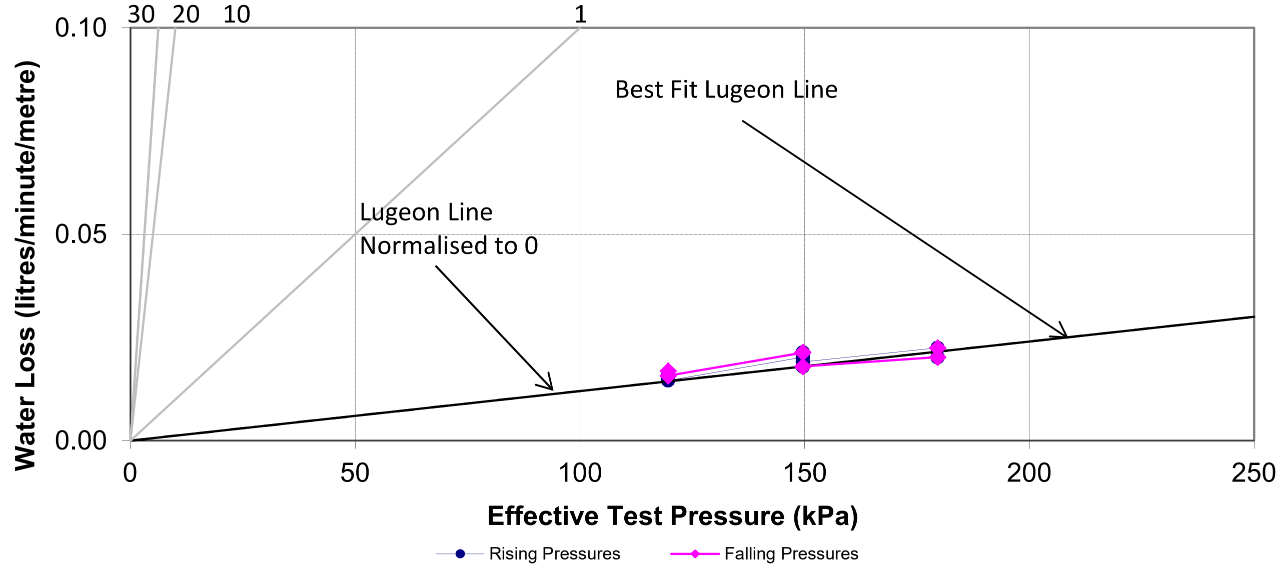
BHH139 - 10.8m - 28.6m

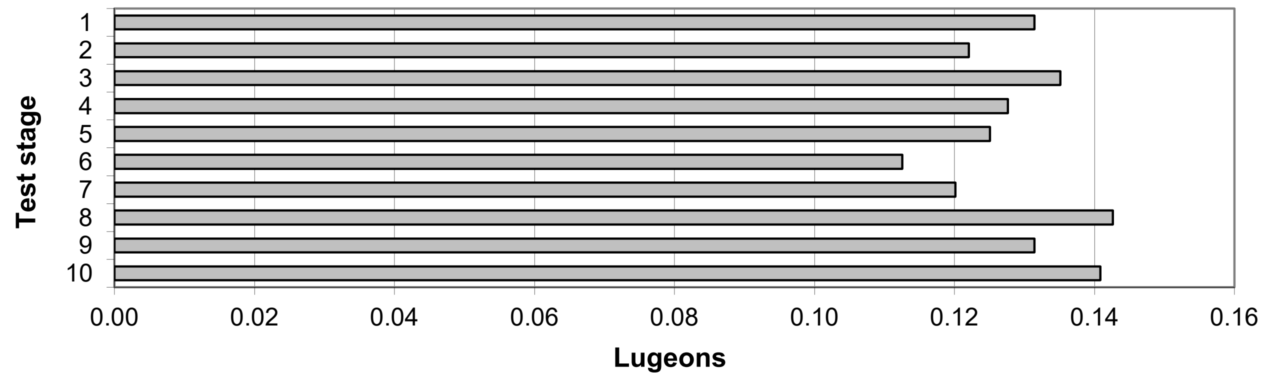
hole no:	BHH139	date:	29-Nov-16
project:	Coffs Harbour Bypass		
location:	Shephards Lane	by:	R Cater

section tested: from 10.80 m to 28.60 m length 17.80 m

relative level at surface:	127.7 m RL	centre of test section (downhole):	19.7 m
gauge height:	0.4 m	test depth converted to vertical:	19.7 m
inclination of borehole:	90 °	relative level at centre of test section:	108.0 m RL
inferred groundwater level:	119.0 m RL	correction to gauge pressure (1)(2):	89.7 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	30	120	1.4	0.0	23%	0.13
5	30	120	1.3	0.0	23%	0.12
5	60	150	1.8	0.0	29%	0.14
5	60	150	1.7	0.0	29%	0.13
5	90	180	2.0	0.0	35%	0.13
5	90	180	1.8	0.0	35%	0.11
5	60	150	1.6	0.0	29%	0.12
5	60	150	1.9	0.0	29%	0.14
5	30	120	1.4	0.0	23%	0.13
5	30	120	1.5	0.0	23%	0.14





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size: HQ3
Lithology: Argillite
Primary Defect Set: Joints

Lugeon (uL) Assessment

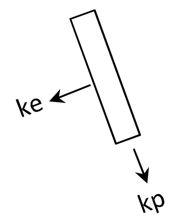
uL (Fell et al., 2005) - upper chart	0.12
uL (Houlsby, 1976) - lower chart	0.13

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	2.7.E-03	m/day
ke/kp = 100	2.4.E-03	m/day
ke/kp = 10	2.0.E-03	m/day
ke/kp = 1	1.7.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 1.8.E-03 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

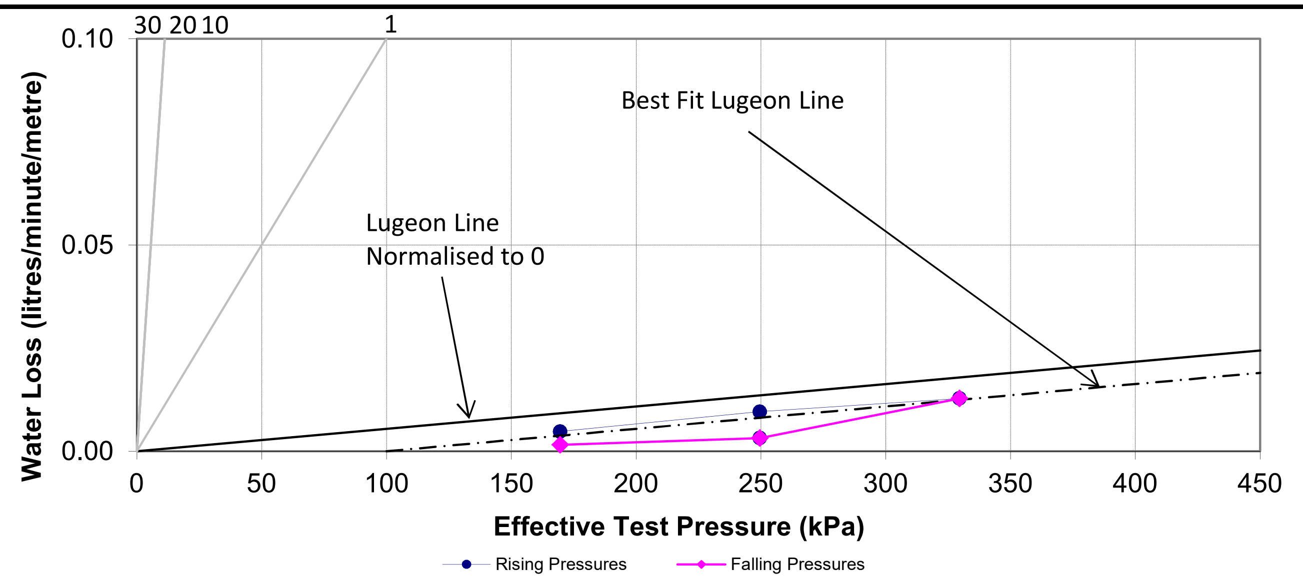
BHH139 - 26.6m - 39.1m

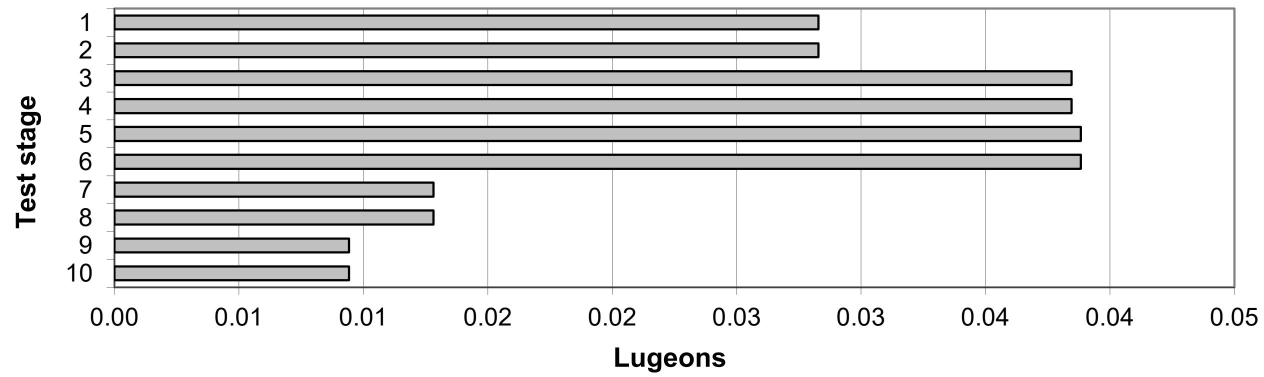
hole no:	BHH139	date:	29-Nov-16
project:	Coffs Harbour Bypass		
location:	Shephards Lane	by:	R Cater

section tested: from 26.60 m to 39.10 m length 12.50 m

relative level at surface:	127.7 m RL	centre of test section (downhole):	32.9 m
gauge height:	0.4 m	test depth converted to vertical:	32.9 m
inclination of borehole:	90 °	relative level at centre of test section:	94.9 m RL
inferred groundwater level:	119.0 m RL	correction to gauge pressure (1)(2):	89.7 kPa

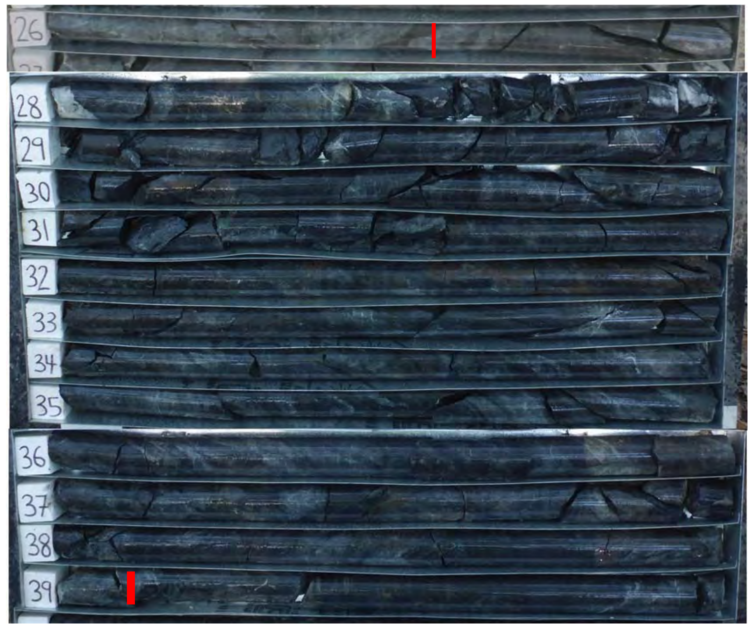
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	80	170	0.3	0.0	20%	0.03
5	80	170	0.3	0.0	20%	0.03
5	160	250	0.6	0.0	29%	0.04
5	160	250	0.6	0.0	29%	0.04
5	240	330	0.8	0.0	39%	0.04
5	240	330	0.8	0.0	39%	0.04
5	160	250	0.2	0.0	29%	0.01
5	160	250	0.2	0.0	29%	0.01
5	80	170	0.1	0.0	20%	0.01
5	80	170	0.1	0.0	20%	0.01





Interpreted flow mechanism: Void Filling

Image of packer test section:



LEGEND:

Packer zone

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

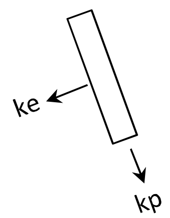
uL (Fell et al., 2005) - upper chart	0.05
uL (Houlsby, 1976) - lower chart	0.01

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	1.9.E-04	m/day
ke/kp = 100	1.7.E-04	m/day
ke/kp = 10	1.4.E-04	m/day
ke/kp = 1	1.2.E-04	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 1.2.E-04 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

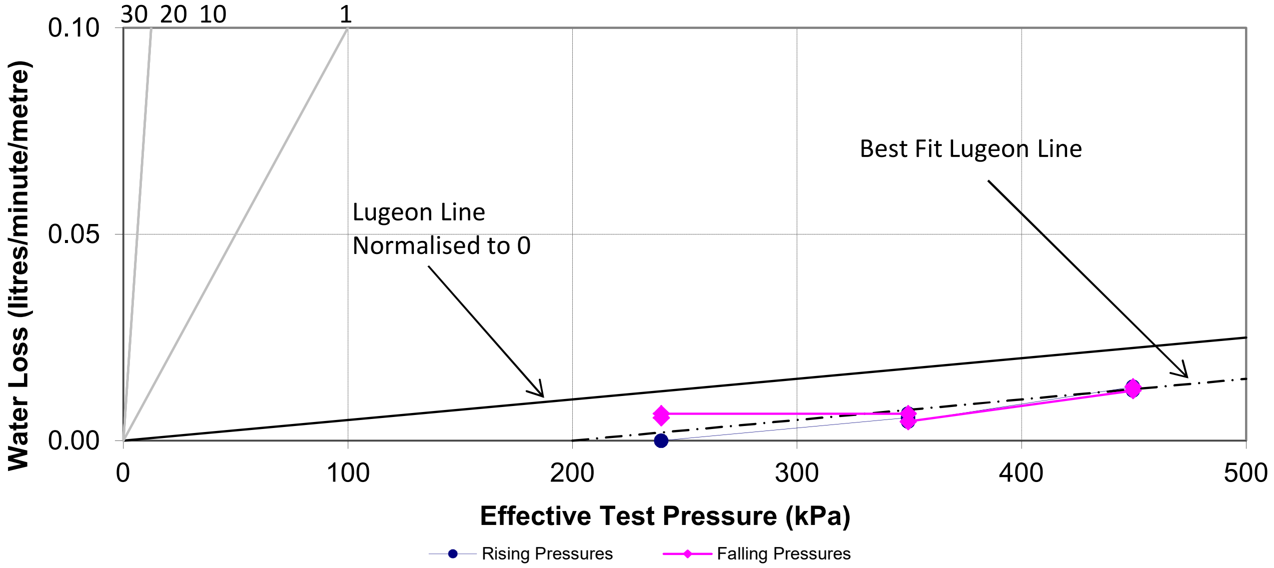
BHH139 - 38.6m - 60.1m

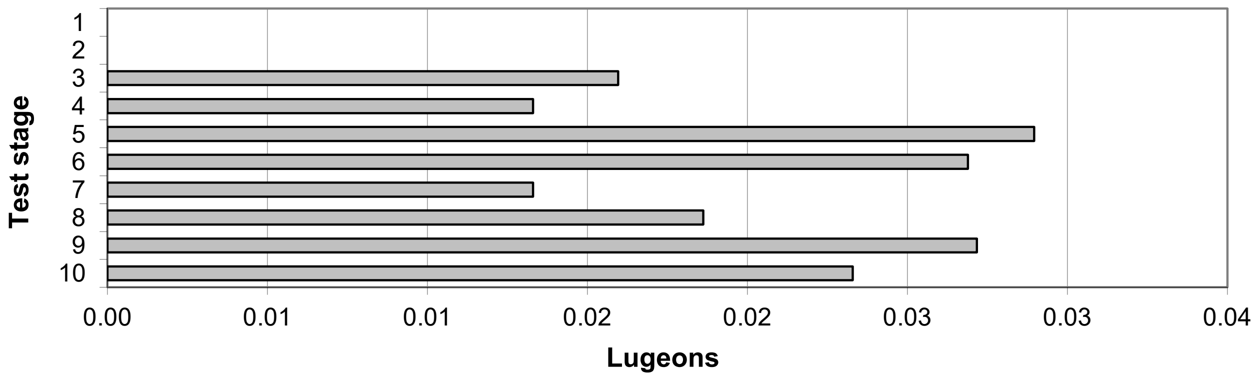
hole no:	BHH139	date:	1-Dec-16
project:	Coffs Harbour Bypass		
location:	Shephards Lane	by:	R Cater

section tested: from 38.60 m to 60.10 m length 21.50 m

relative level at surface:	127.7 m RL	centre of test section (downhole):	49.4 m
gauge height:	0.4 m	test depth converted to vertical:	49.4 m
inclination of borehole:	90 °	relative level at centre of test section:	78.4 m RL
inferred groundwater level:	119.0 m RL	correction to gauge pressure (1)(2):	89.7 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	240	0.0	0.0	19%	0.00
5	150	240	0.0	0.0	19%	0.00
5	260	350	0.6	0.0	27%	0.02
5	260	350	0.5	0.0	27%	0.01
5	360	450	1.4	0.0	35%	0.03
5	360	450	1.3	0.0	35%	0.03
5	260	350	0.5	0.0	27%	0.01
5	260	350	0.7	0.0	27%	0.02
5	150	240	0.7	0.0	19%	0.03
5	150	240	0.6	0.0	19%	0.02





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

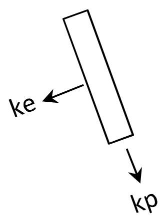
uL (Fell et al., 2005) - upper chart	0.05
uL (Houlsby, 1976) - lower chart	0.02

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	3.6.E-04	m/day
ke/kp = 100	3.2.E-04	m/day
ke/kp = 10	2.7.E-04	m/day
ke/kp = 1	2.3.E-04	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.0.E-04 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

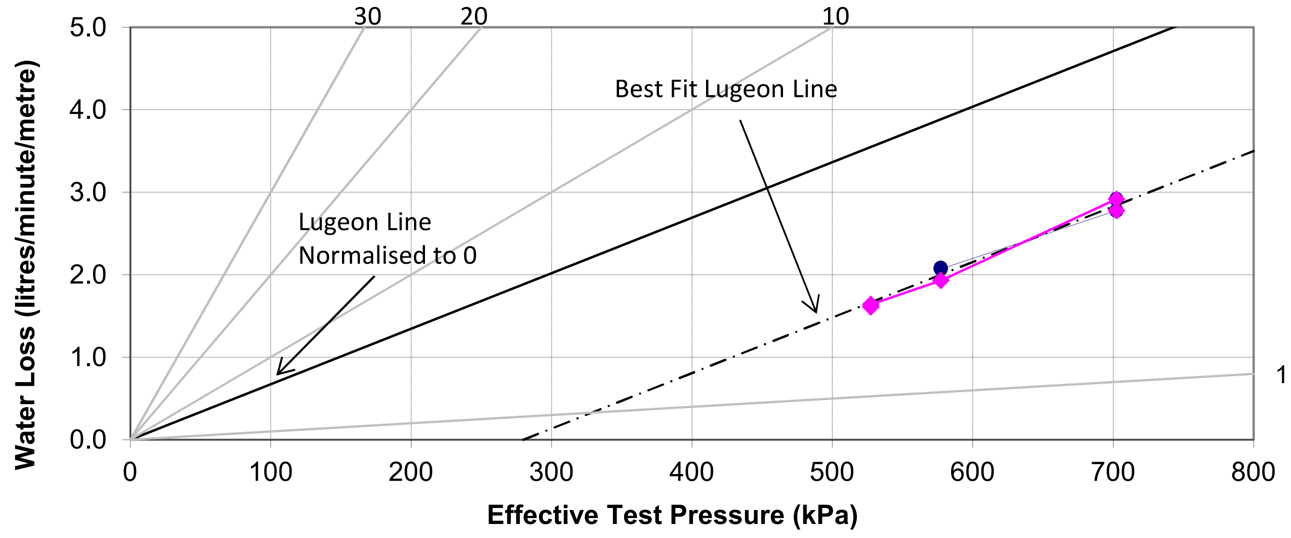
BHH140 - 46.25m - 60.65m

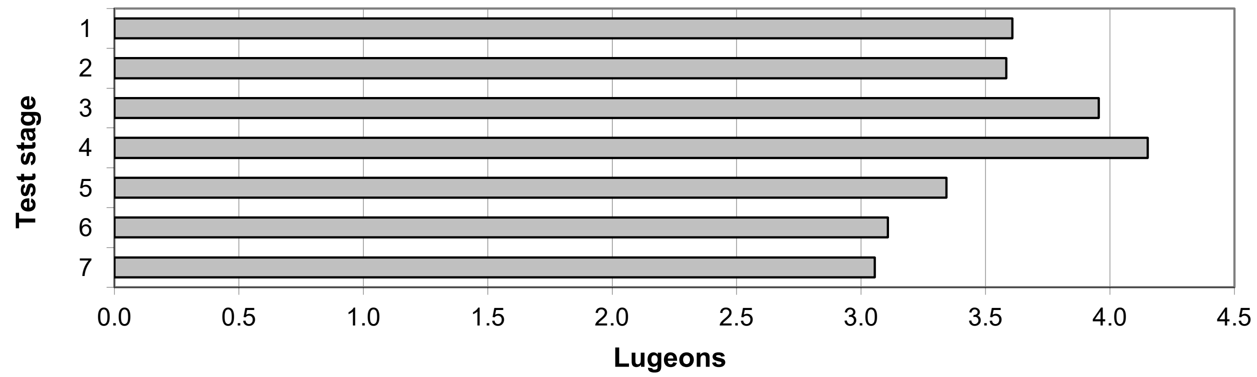
hole no:	BHH140	date:	16-Dec-16
project:	Coffs Harbour Bypass		
location:	Shephards Lane	by:	M Kobler

section tested: from 46.3 m to 60.7 m length 14.4 m

relative level at surface:	160.8 m RL	centre of test section (downhole):	53.5 m
gauge height:	0.5 m	test depth converted to vertical:	50.2 m
inclination of borehole:	70 °	relative level at centre of test section:	110.6 m RL
inferred groundwater level:	115.2 m RL	correction to gauge pressure (1)(2):	452.5 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	125	577	150.0	2.1	44%	3.6
5	125	577	149.0	2.1	44%	3.6
1	250	702	40.0	2.8	54%	4.0
1	250	702	42.0	2.9	54%	4.2
5	125	577	139.0	1.9	44%	3.3
5	75	527	118.0	1.6	40%	3.1
5	75	527	116.0	1.6	40%	3.1





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

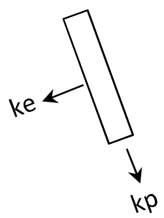
uL (Fell et al., 2005) - upper chart	6.7
uL (Houlsby, 1976) - lower chart	3.5

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	7.3.E-02	m/day
ke/kp = 100	6.4.E-02	m/day
ke/kp = 10	5.5.E-02	m/day
ke/kp = 1	4.5.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 4.8.E-02 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

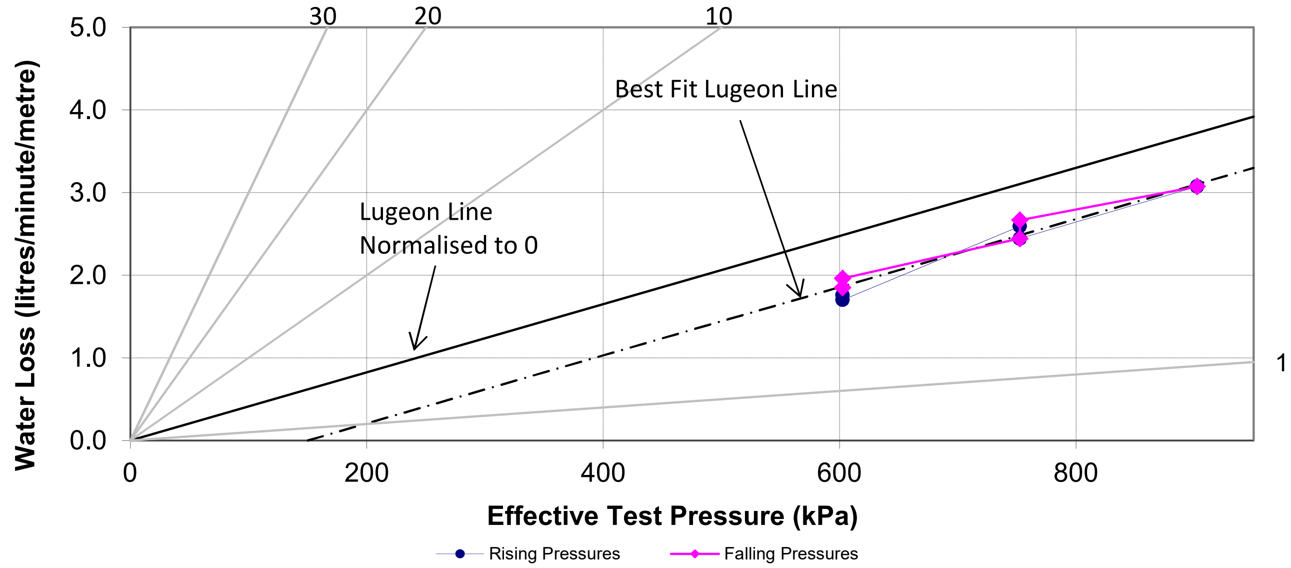
BHH140 - 55.25m - 60.65m

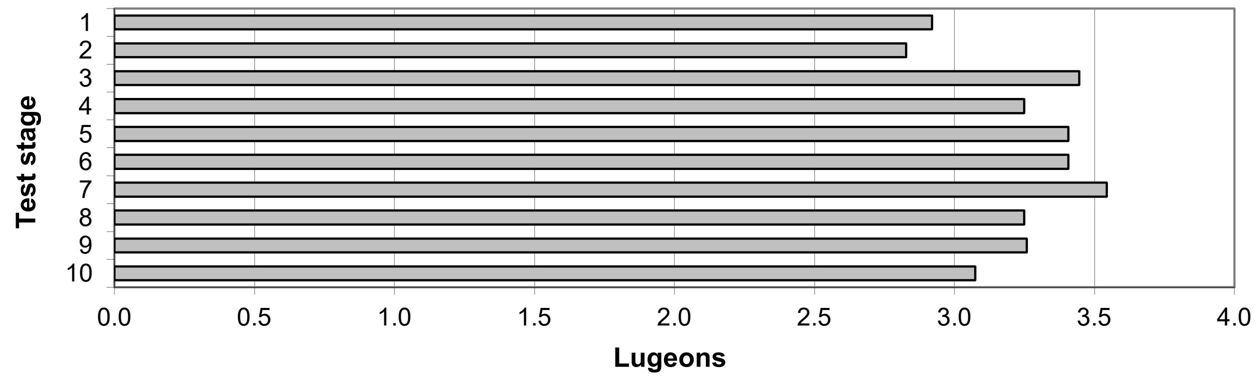
hole no:	BHH140	date:	16-Dec-16
project:	Coffs Harbour Bypass		
location:	Shephards Lane	by:	M Kobler

section tested: from 55.3 m to 60.7 m length 5.4 m

relative level at surface:	160.8 m RL	centre of test section (downhole):	58.0 m
gauge height:	0.5 m	test depth converted to vertical:	54.5 m
inclination of borehole:	70 °	relative level at centre of test section:	106.4 m RL
inferred groundwater level:	115.2 m RL	correction to gauge pressure (1)(2):	452.5 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	602	47.5	1.8	43%	2.9
5	150	602	46.0	1.7	43%	2.8
5	300	752	70.0	2.6	53%	3.4
5	300	752	66.0	2.4	53%	3.2
5	450	902	83.0	3.1	64%	3.4
5	450	902	83.0	3.1	64%	3.4
5	300	752	72.0	2.7	53%	3.5
5	300	752	66.0	2.4	53%	3.2
5	150	602	53.0	2.0	43%	3.3
5	150	602	50.0	1.9	43%	3.1





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

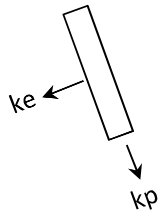
uL (Fell et al., 2005) - upper chart	4.1
uL (Houlsby, 1976) - lower chart	3.2

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	5.9.E-02	m/day
ke/kp = 100	5.1.E-02	m/day
ke/kp = 10	4.3.E-02	m/day
ke/kp = 1	3.4.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 3.7.E-02 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink — Job no. PSM2876

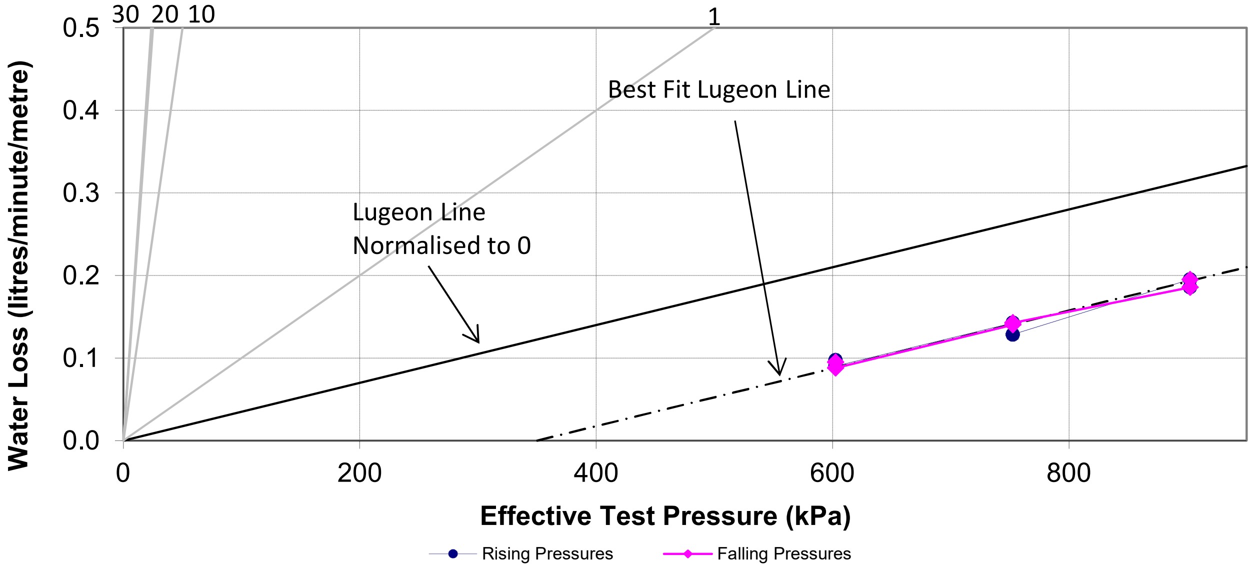
Borehole Water Pressure Test Summary — **BHH140 - 61.25m - 69.65m**

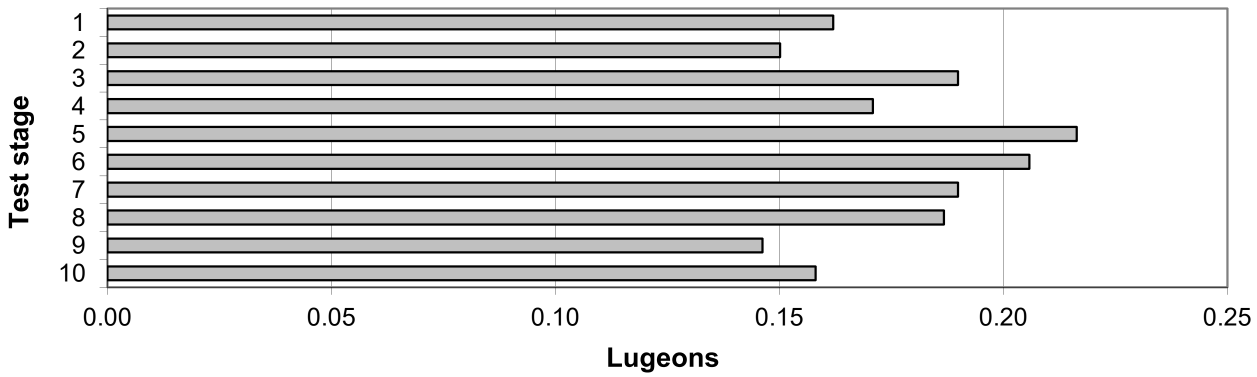
hole no:	BHH140	date:	19-Dec-16
project:	Coffs Harbour Bypass		
location:	Shephards Lane	by:	M Kobler

section tested: from 61.3 m to 69.7 m length 8.4 m

relative level at surface:	160.8 m RL	centre of test section (downhole):	65.5 m
gauge height:	0.5 m	test depth converted to vertical:	61.5 m
inclination of borehole:	70 °	relative level at centre of test section:	99.3 m RL
inferred groundwater level:	115.2 m RL	correction to gauge pressure (1)(2):	452.5 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	602	4.1	0.1	38%	0.16
5	150	602	3.8	0.1	38%	0.15
5	300	752	6.0	0.1	47%	0.19
5	300	752	5.4	0.1	47%	0.17
5	450	902	8.2	0.2	56%	0.22
5	450	902	7.8	0.2	56%	0.21
5	300	752	6.0	0.1	47%	0.19
5	300	752	5.9	0.1	47%	0.19
5	150	602	3.7	0.1	38%	0.15
5	150	602	4.0	0.1	38%	0.16





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size: HQ3

Lithology: Argillite

Primary Defect Set: Joints

Lugeon (uL) Assessment

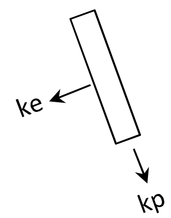
uL (Fell et al., 2005) - upper chart	0.35
uL (Houlsby, 1976) - lower chart	0.18

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	3.4.E-03	m/day
ke/kp = 100	3.0.E-03	m/day
ke/kp = 10	2.5.E-03	m/day
ke/kp = 1	2.1.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.2.E-03 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

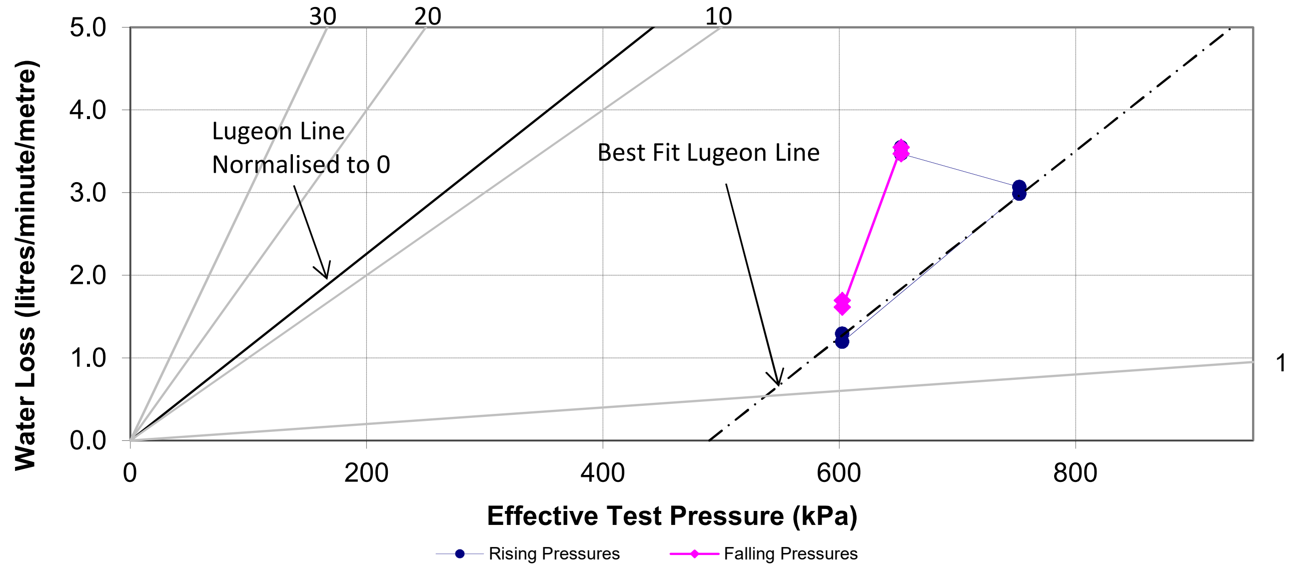
BHH140 - 70.25m - 82.65m

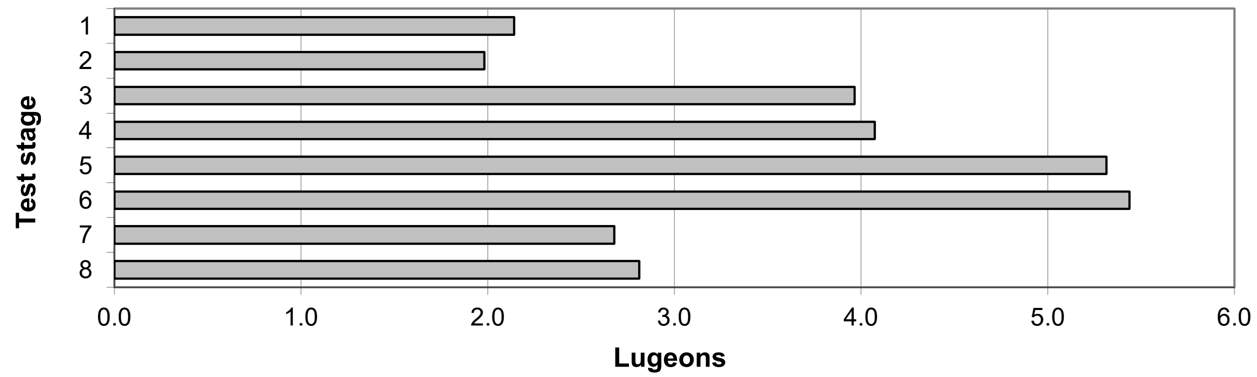
hole no:	BHH140	date:	20-Dec-16
project:	Coffs Harbour Bypass		
location:	Shephards Lane	by:	M Kobler

section tested: from 70.3 m to 82.7 m length 12.4 m

relative level at surface:	160.8 m RL	centre of test section (downhole):	76.5 m
gauge height:	0.5 m	test depth converted to vertical:	71.8 m
inclination of borehole:	70 °	relative level at centre of test section:	89.0 m RL
inferred groundwater level:	115.2 m RL	correction to gauge pressure (1)(2):	452.5 kPa

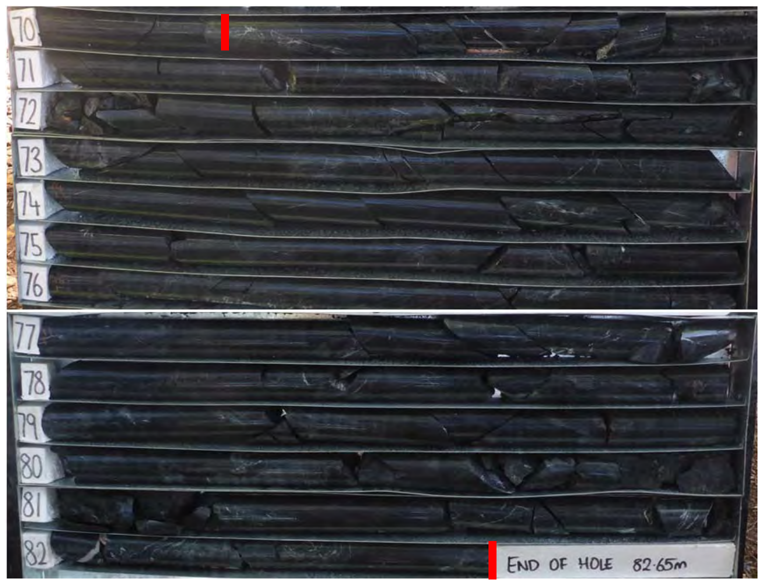
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	602	80.0	1.3	32%	2.1
5	150	602	74.0	1.2	32%	2.0
1	300	752	37.0	3.0	40%	4.0
1	300	752	38.0	3.1	40%	4.1
1	200	652	43.0	3.5	35%	5.3
1	200	652	44.0	3.5	35%	5.4
1	150	602	20.0	1.6	32%	2.7
1	150	602	21.0	1.7	32%	2.8





Interpreted flow mechanism: Dilation

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

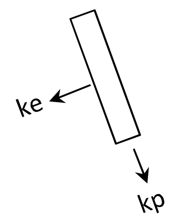
uL (Fell et al., 2005) - upper chart	11.3
uL (Houlsby, 1976) - lower chart	2.4

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	4.9.E-02	m/day
ke/kp = 100	4.2.E-02	m/day
ke/kp = 10	3.6.E-02	m/day
ke/kp = 1	3.0.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 3.2.E-02 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink Job no. PSM2876

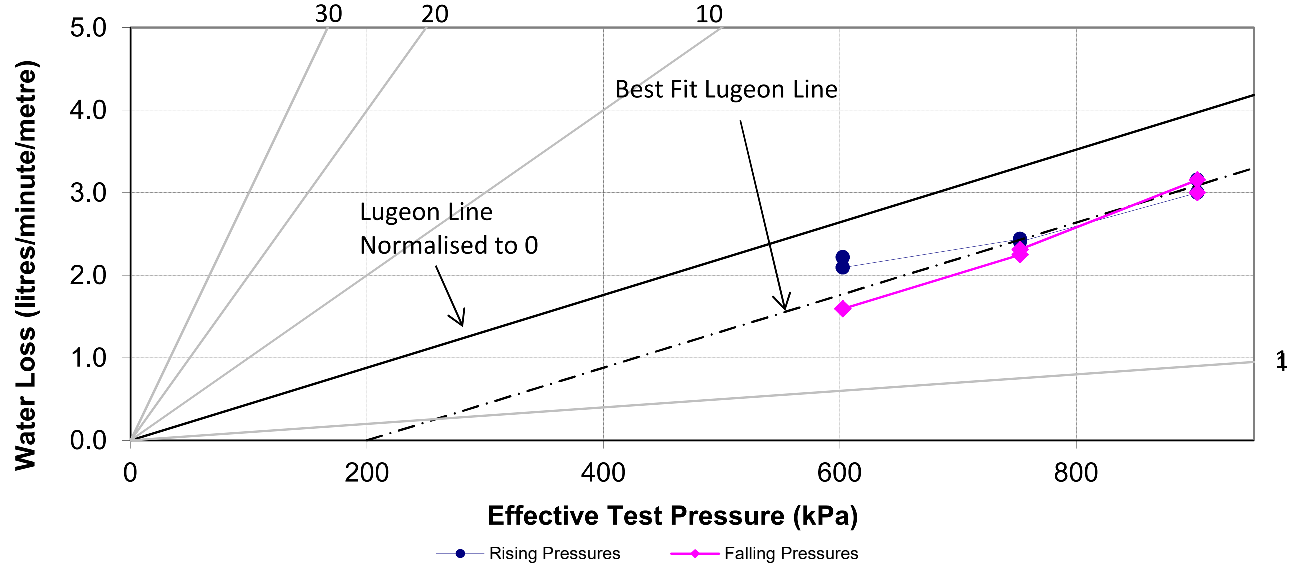
Borehole Water Pressure Test Summary **BHH140 - 76.25m - 82.65m**

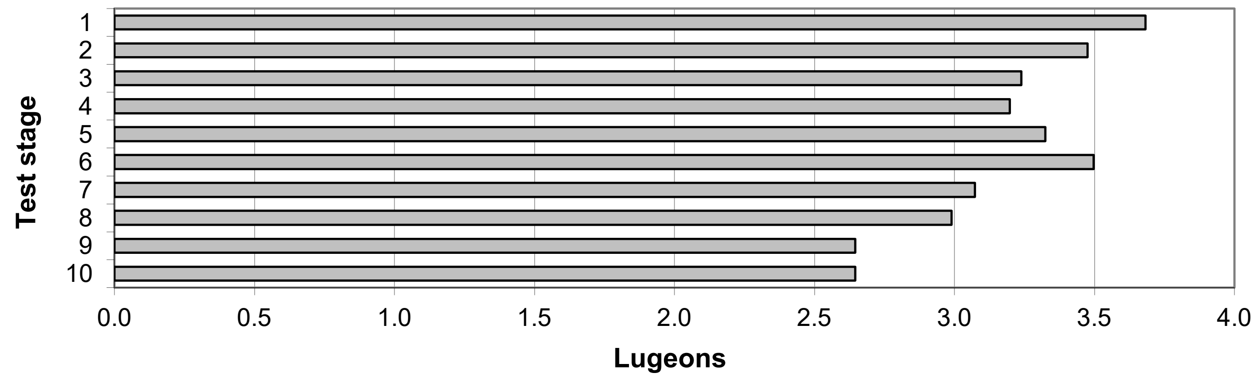
hole no:	BHH140	date:	20-Dec-16
project:	Coffs Harbour Bypass		
location:	Shephards Lane	by:	M Kobler

section tested: from 76.3 m to 82.7 m length 6.4 m

relative level at surface:	160.8 m RL	centre of test section (downhole):	79.5 m
gauge height:	0.5 m	test depth converted to vertical:	74.7 m
inclination of borehole:	70 °	relative level at centre of test section:	86.2 m RL
inferred groundwater level:	115.2 m RL	correction to gauge pressure (1)(2):	452.5 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	602	71.0	2.2	31%	3.7
5	150	602	67.0	2.1	31%	3.5
5	300	752	78.0	2.4	39%	3.2
5	300	752	77.0	2.4	39%	3.2
5	450	902	96.0	3.0	46%	3.3
5	450	902	101.0	3.2	46%	3.5
5	300	752	74.0	2.3	39%	3.1
5	300	752	72.0	2.3	39%	3.0
5	150	602	51.0	1.6	31%	2.6
5	150	602	51.0	1.6	31%	2.6





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

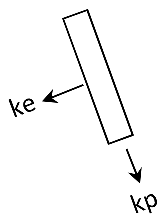
uL (Fell et al., 2005) - upper chart	4.4
uL (Houlsby, 1976) - lower chart	3.2

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	6.0.E-02	m/day
ke/kp = 100	5.1.E-02	m/day
ke/kp = 10	4.3.E-02	m/day
ke/kp = 1	3.5.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 3.7.E-02 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Test undertaken above water table

Job no. PSM2876

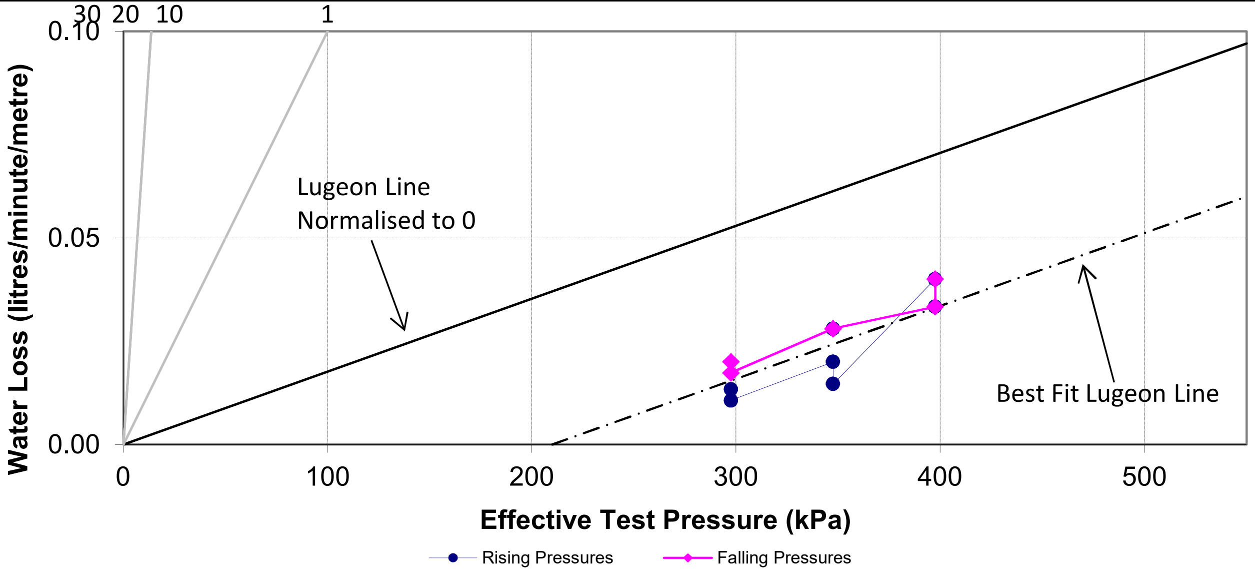
BHH141 - 17.32m - 32.3m

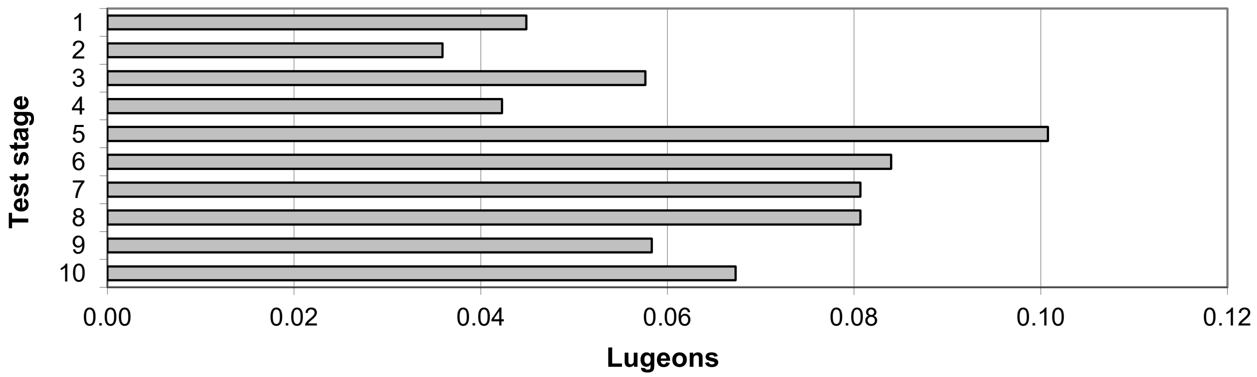
hole no:	BHH141	date:	28-Nov-16
project:	Coffs Harbour Bypass		
location:	Shephards Lane	by:	M Kobler

section tested: from 17.32 m to 32.30 m length 14.98 m

relative level at surface:	151.7 m RL	centre of test section (downhole):	24.8 m
gauge height:	0.5 m	test depth converted to vertical:	24.8 m
inclination of borehole:	90 °	relative level at centre of test section:	126.9 m RL
inferred groundwater level:	120.2 m RL	correction to gauge pressure (1)(2):	247.5 kPa

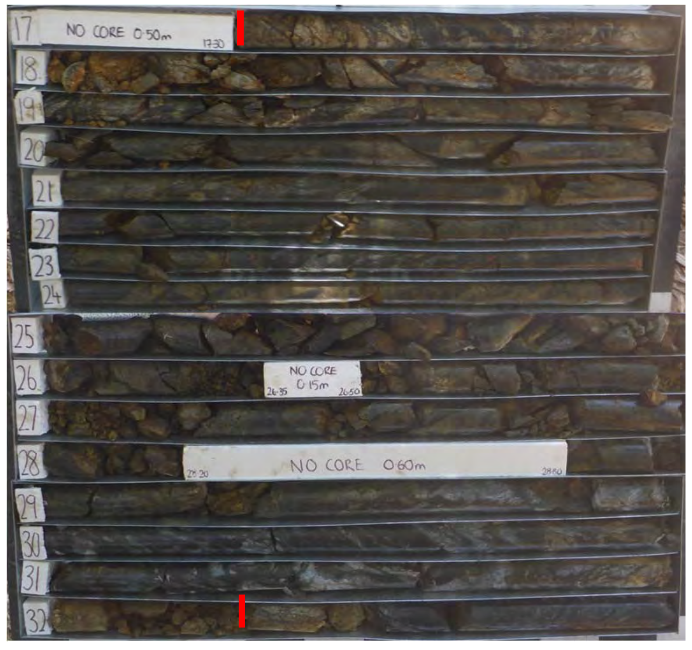
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	50	298	1.0	0.0	46%	0.04
5	50	298	0.8	0.0	46%	0.04
5	100	348	1.5	0.0	54%	0.06
5	100	348	1.1	0.0	54%	0.04
5	150	398	3.0	0.0	62%	0.10
5	150	398	2.5	0.0	62%	0.08
5	100	348	2.1	0.0	54%	0.08
5	100	348	2.1	0.0	54%	0.08
5	50	298	1.3	0.0	46%	0.06
5	50	298	1.5	0.0	46%	0.07





Interpreted flow mechanism: Dilation

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	No Core / Joints

Lugeon (uL) Assessment

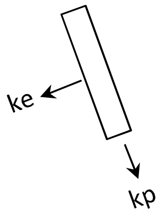
uL (Fell et al., 2005) - upper chart	0.18
uL (Houlsby, 1976) - lower chart	0.06

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	1.2.E-03	m/day
ke/kp = 100	1.1.E-03	m/day
ke/kp = 10	9.0.E-04	m/day
ke/kp = 1	7.5.E-04	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 7.9.E-04 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

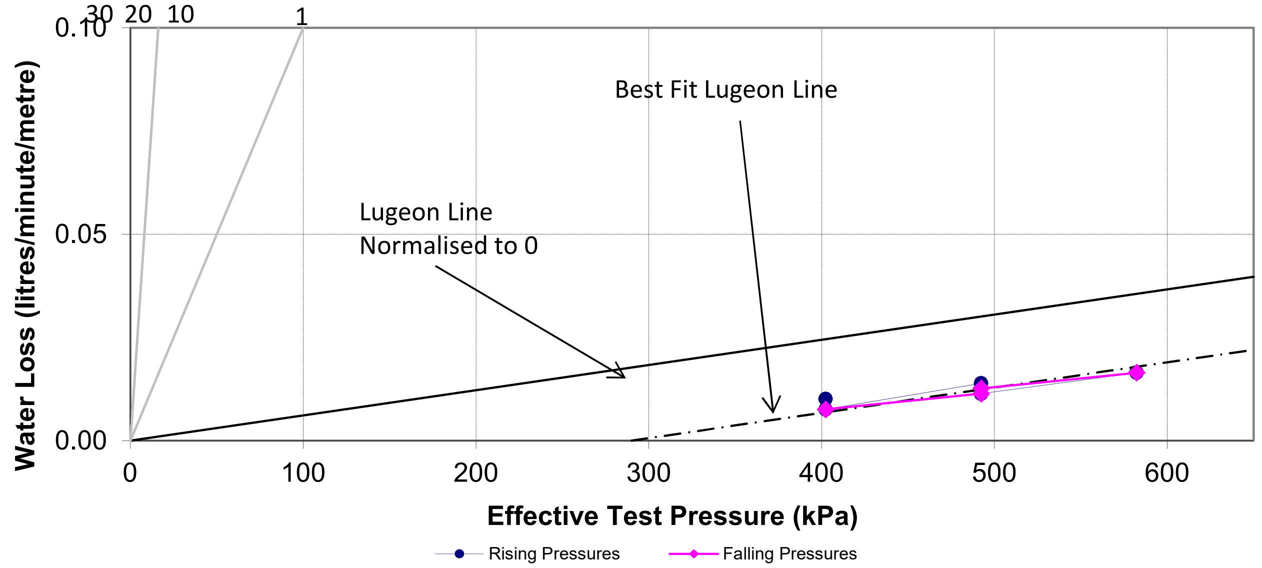
BHH141 - 32.3m - 48.1m

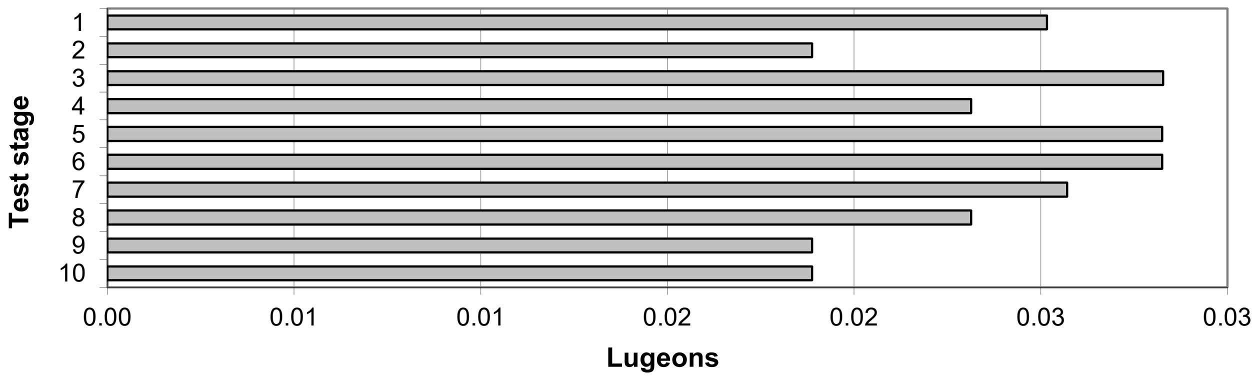
hole no:	BHH141	date:	29-Nov-16
project:	Coffs Harbour Bypass		
location:	Shapherds Lane	by:	M Kobler

section tested: from 32.30 m to 48.10 m length 15.80 m

relative level at surface:	151.7 m RL	centre of test section (downhole):	40.2 m
gauge height:	0.5 m	test depth converted to vertical:	40.2 m
inclination of borehole:	90 °	relative level at centre of test section:	111.5 m RL
inferred groundwater level:	120.2 m RL	correction to gauge pressure [(1)(2)]:	312.4 kPa

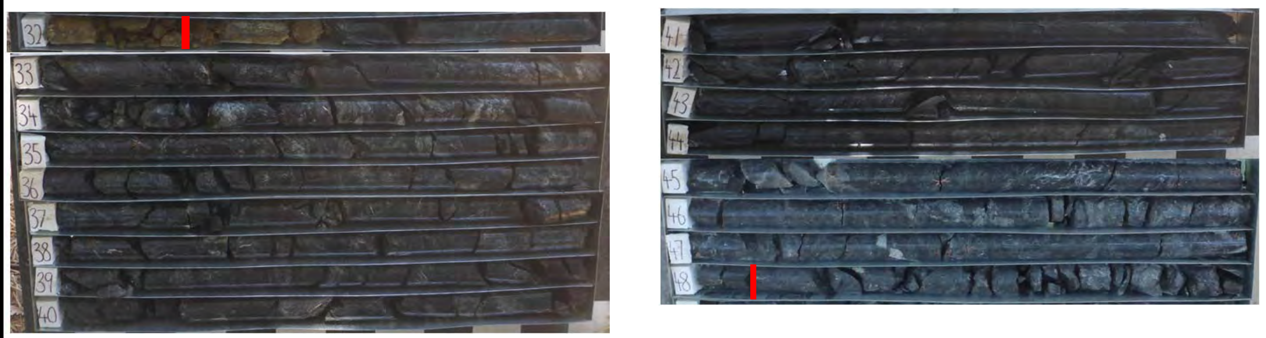
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure [(3)]	lugeons litres / min / m / 1000kPa
5	90	402	0.8	0.0	39%	0.03
5	90	402	0.6	0.0	39%	0.02
5	180	492	1.1	0.0	47%	0.03
5	180	492	0.9	0.0	47%	0.02
5	270	582	1.3	0.0	56%	0.03
5	270	582	1.3	0.0	56%	0.03
5	180	492	1.0	0.0	47%	0.03
5	180	492	0.9	0.0	47%	0.02
5	90	402	0.6	0.0	39%	0.02
5	90	402	0.6	0.0	39%	0.02





Interpreted flow mechanism: Laminar Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

uL (Fell et al., 2005) - upper chart	0.06
uL (Houlsby, 1976) - lower chart	0.02

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:

ke
kp

ke/kp = 1000	5.0.E-04	m/day
ke/kp = 100	4.3.E-04	m/day
ke/kp = 10	3.7.E-04	m/day
ke/kp = 1	3.1.E-04	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 3.3.E-04 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

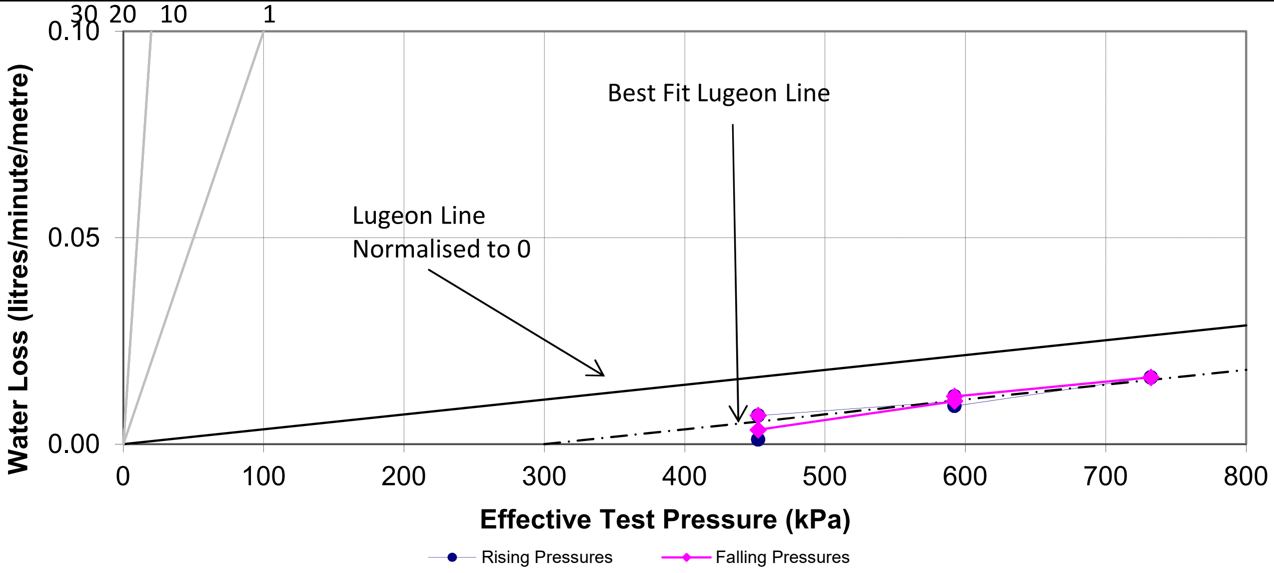
BHH141 - 48.8m - 66.1m

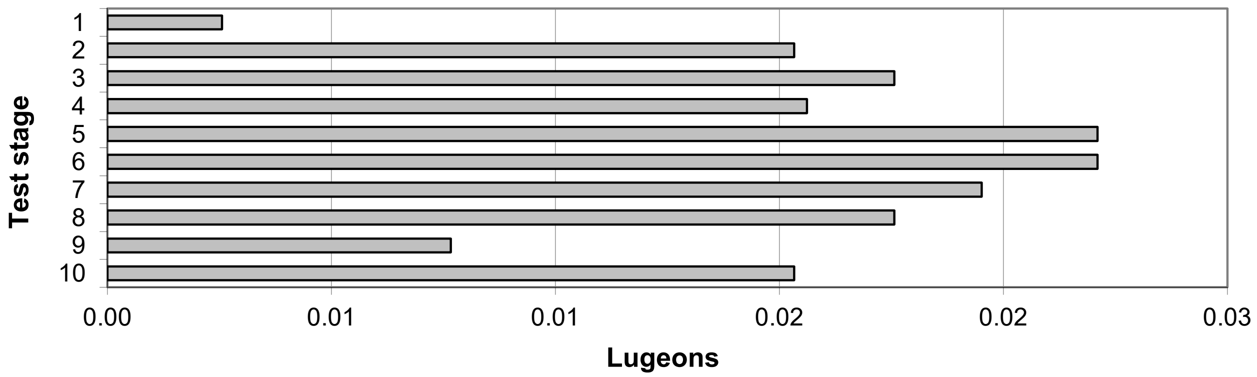
hole no:	BHH141	date:	30-Nov-16
project:	Coffs Harbour Bypass		
location:	Shephards Lane	by:	M Kobler

section tested: from 48.80 m to 66.10 m length 17.30 m

relative level at surface:	151.7 m RL	centre of test section (downhole):	57.5 m
gauge height:	0.5 m	test depth converted to vertical:	57.5 m
inclination of borehole:	90 °	relative level at centre of test section:	94.2 m RL
inferred groundwater level:	120.2 m RL	correction to gauge pressure (1)(2):	312.4 kPa

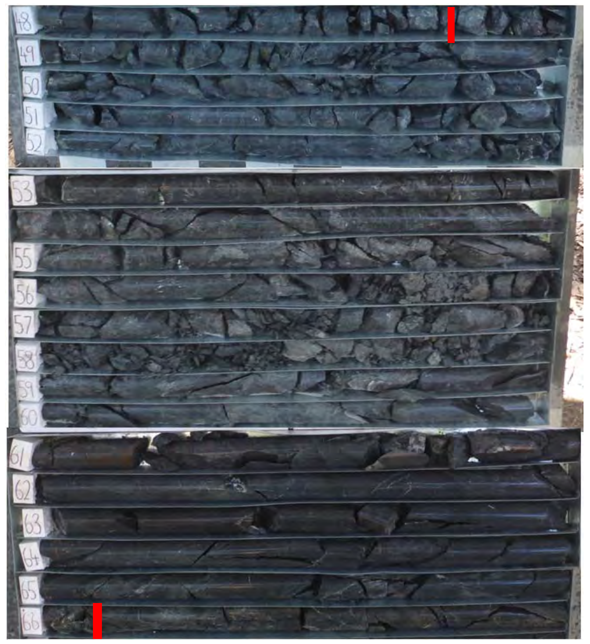
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	140	452	0.1	0.0	30%	0.00
5	140	452	0.6	0.0	30%	0.02
5	280	592	0.9	0.0	40%	0.02
5	280	592	0.8	0.0	40%	0.02
5	420	732	1.4	0.0	49%	0.02
5	420	732	1.4	0.0	49%	0.02
5	280	592	1.0	0.0	40%	0.02
5	280	592	0.9	0.0	40%	0.02
5	140	452	0.3	0.0	30%	0.01
5	140	452	0.6	0.0	30%	0.02





Interpreted flow mechanism: Laminar Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints and shears

Lugeon (uL) Assessment

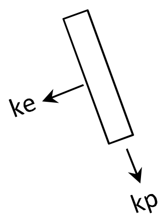
uL (Fell et al., 2005) - upper chart	0.04
uL (Houlsby, 1976) - lower chart	0.02

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	3.3.E-04	m/day
ke/kp = 100	2.9.E-04	m/day
ke/kp = 10	2.5.E-04	m/day
ke/kp = 1	2.1.E-04	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.2.E-04 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

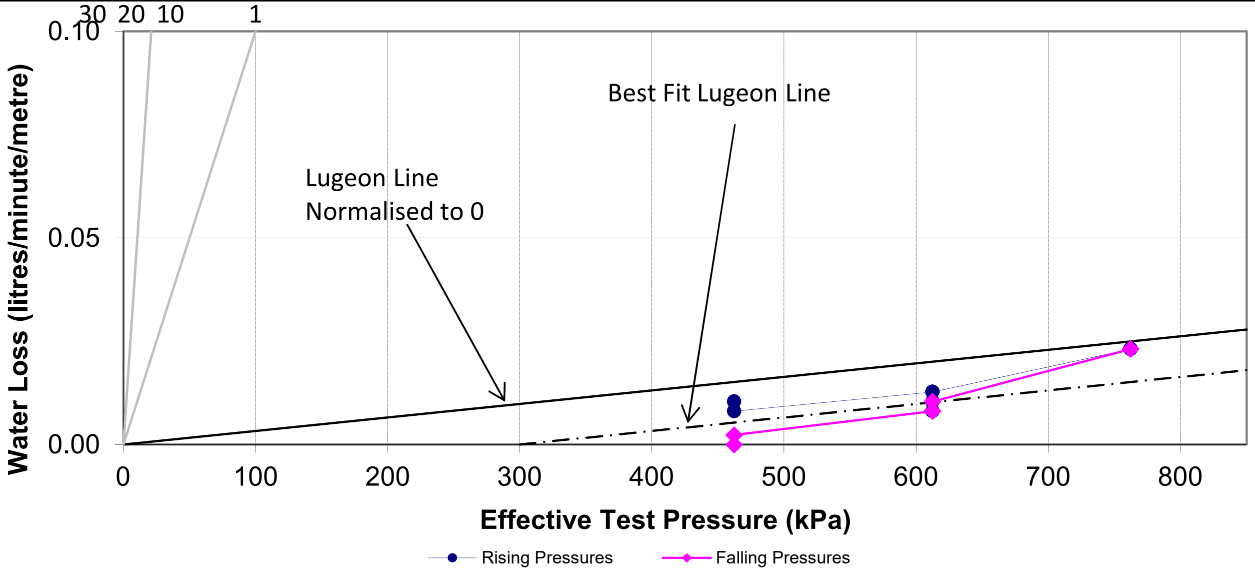
BHH141 - 66.8m - 84.1m

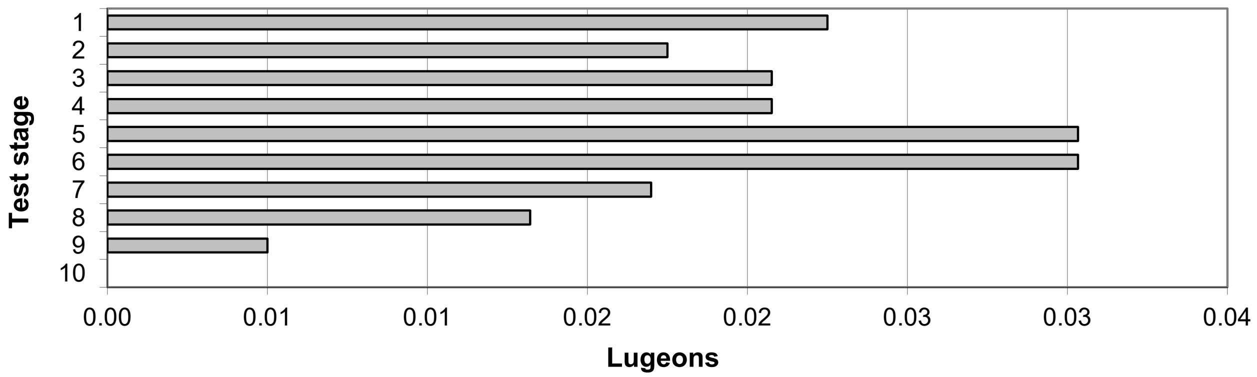
hole no:	BHH141	date:	1-Dec-16
project:	Coffs Harbour Bypass		
location:	Shephards Lane	by:	M Kobler

section tested: from 66.8 m to 84.1 m length 17.30 m

relative level at surface:	151.7 m RL	centre of test section (downhole):	75.5 m
gauge height:	0.5 m	test depth converted to vertical:	75.5 m
inclination of borehole:	90 °	relative level at centre of test section:	76.2 m RL
inferred groundwater level:	120.2 m RL	correction to gauge pressure (1)(2):	312.4 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	462	0.9	0.0	24%	0.02
5	150	462	0.7	0.0	24%	0.02
5	300	612	1.1	0.0	31%	0.02
5	300	612	1.1	0.0	31%	0.02
5	450	762	2.0	0.0	39%	0.03
5	450	762	2.0	0.0	39%	0.03
5	300	612	0.9	0.0	31%	0.02
5	300	612	0.7	0.0	31%	0.01
5	150	462	0.2	0.0	24%	0.01
5	150	462	0.0	0.0	24%	0.00





Interpreted flow mechanism: Dilation

Image of packer test section:

LEGEND:

Packer zone boundary

Core size: HQ3

Lithology: Argillite

Primary Defect Set: Joints

Lugeon (uL) Assessment

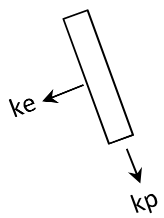
uL (Fell et al., 2005) - upper chart	0.03
uL (Houlsby, 1976) - lower chart	0.01

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	3.1.E-04	m/day
ke/kp = 100	2.7.E-04	m/day
ke/kp = 10	2.3.E-04	m/day
ke/kp = 1	1.9.E-04	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.0.E-04 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink Job no. PSM2876

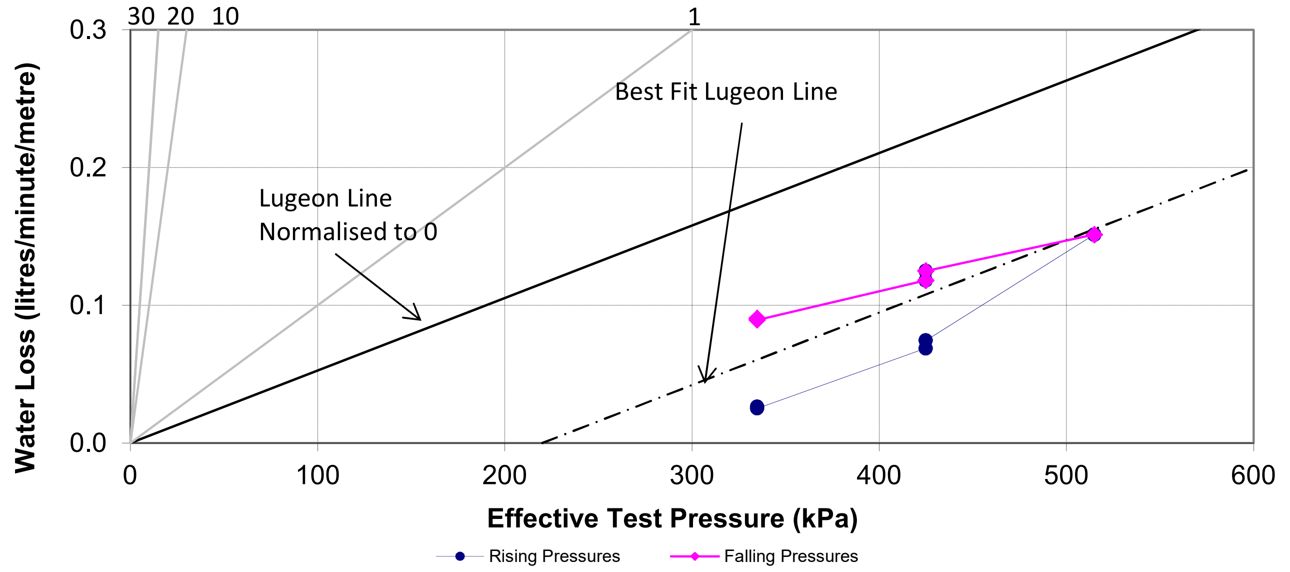
Borehole Water Pressure Test Summary **BHH142 - 31.2m - 48.65m**

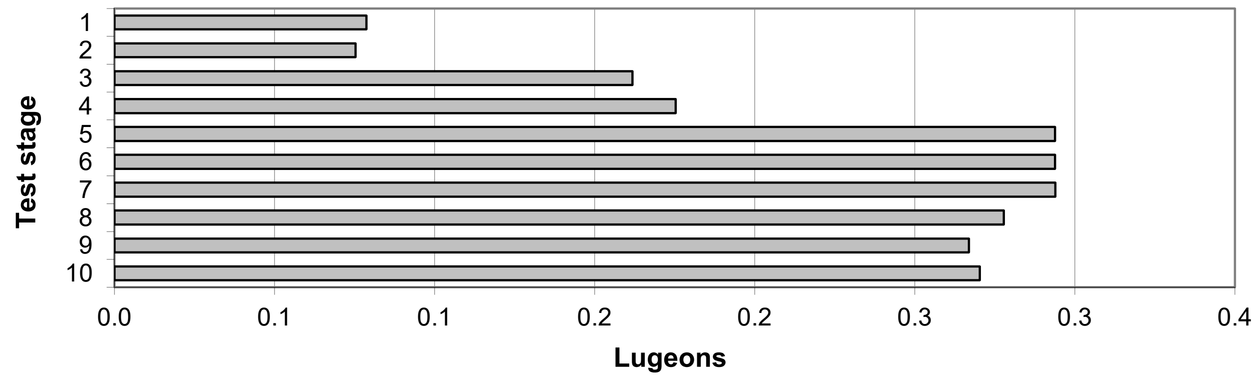
hole no:	BHH142	date:	8-Dec-16
project:	Coffs Harbour Bypass		
location:	Shephards Lane	by:	M Kobler

section tested: from 31.20 m to 48.65 m length 17.45 m

relative level at surface:	157.9 m RL	centre of test section (downhole):	39.9 m
gauge height:	0.0 m	test depth converted to vertical:	37.5 m
inclination of borehole:	70 °	relative level at centre of test section:	120.4 m RL
inferred groundwater level:	132.9 m RL	correction to gauge pressure (1)(2):	245.0 kPa

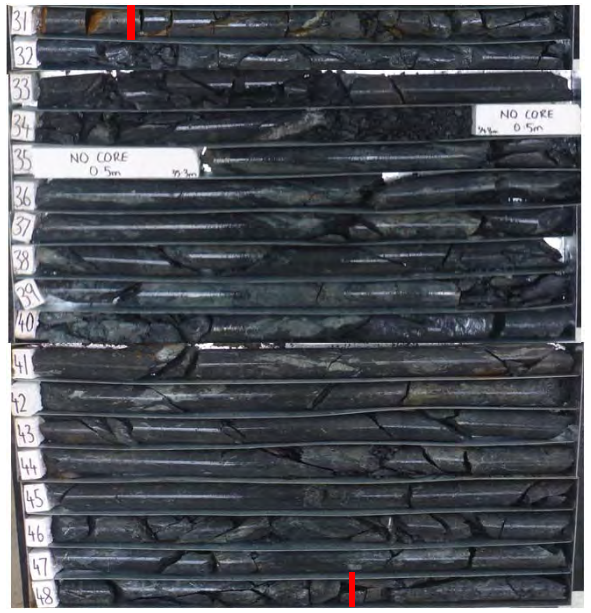
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	90	335	2.3	0.0	34%	0.1
5	90	335	2.2	0.0	34%	0.1
5	180	425	6.0	0.1	44%	0.2
5	180	425	6.5	0.1	44%	0.2
5	270	515	13.2	0.2	53%	0.3
5	270	515	13.2	0.2	53%	0.3
5	180	425	10.9	0.1	44%	0.3
5	180	425	10.3	0.1	44%	0.3
5	90	335	7.8	0.1	34%	0.3
5	90	335	7.9	0.1	34%	0.3





Interpreted flow mechanism: Wash Out

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints / No Core

Lugeon (uL) Assessment

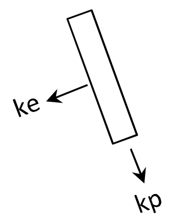
uL (Fell et al., 2005) - upper chart	0.5
uL (Houlsby, 1976) - lower chart	0.3

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	6.2.E-03	m/day
ke/kp = 100	5.4.E-03	m/day
ke/kp = 10	4.7.E-03	m/day
ke/kp = 1	3.9.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 4.1.E-03 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink Job no. PSM2876

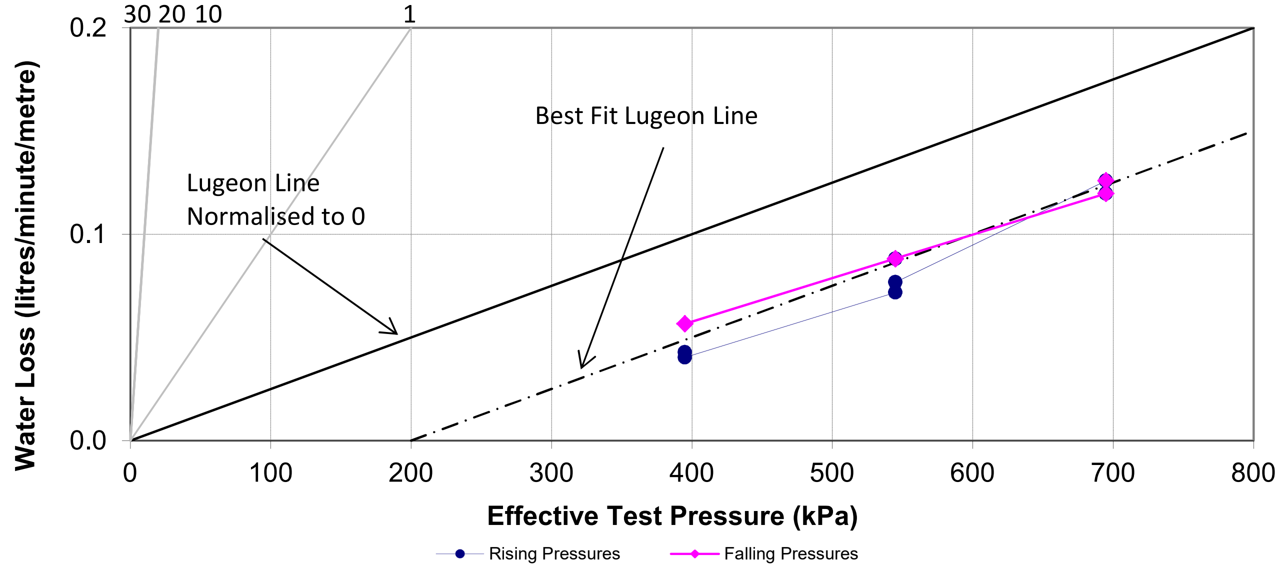
Borehole Water Pressure Test Summary **BHH142 - 64.85m - 80.73m**

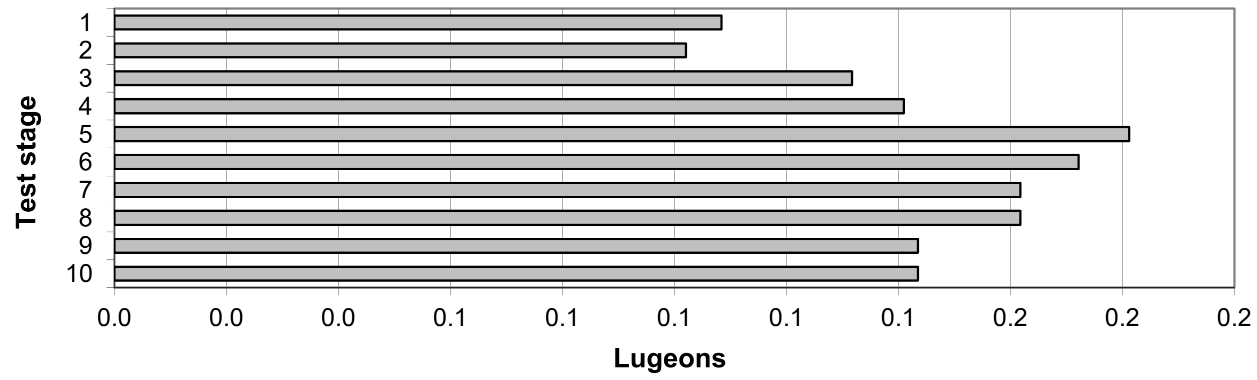
hole no: BHH142 date: 12-Dec-16
project: Coffs Harbour Bypass
location: Shephards Lane by: M Kobler

section tested: from 64.85 m to 80.73 m length 15.88 m

relative level at surface: 157.9 m RL | centre of test section (downhole): 72.8 m
gauge height: 0.0 m | test depth converted to vertical: 68.4 m
inclination of borehole: 70 ° | relative level at centre of test section: 89.5 m RL
inferred groundwater level: 132.9 m RL | correction to gauge pressure (1)(2): 245.0 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	395	3.4	0.0	22%	0.1
5	150	395	3.2	0.0	22%	0.1
5	300	545	5.7	0.1	31%	0.1
5	300	545	6.1	0.1	31%	0.1
5	450	695	10.0	0.1	39%	0.2
5	450	695	9.5	0.1	39%	0.2
5	300	545	7.0	0.1	31%	0.2
5	300	545	7.0	0.1	31%	0.2
5	150	395	4.5	0.1	22%	0.1
5	150	395	4.5	0.1	22%	0.1





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size: HQ3
Lithology: Argillite
Primary Defect Set: Joints / No Core

Lugeon (uL) Assessment

uL (Fell et al., 2005) - upper chart 0.3

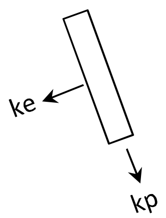
uL (Houlsby, 1976) - lower chart 0.1

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	3.0.E-03	m/day
ke/kp = 100	2.6.E-03	m/day
ke/kp = 10	2.3.E-03	m/day
ke/kp = 1	1.9.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.0.E-03 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

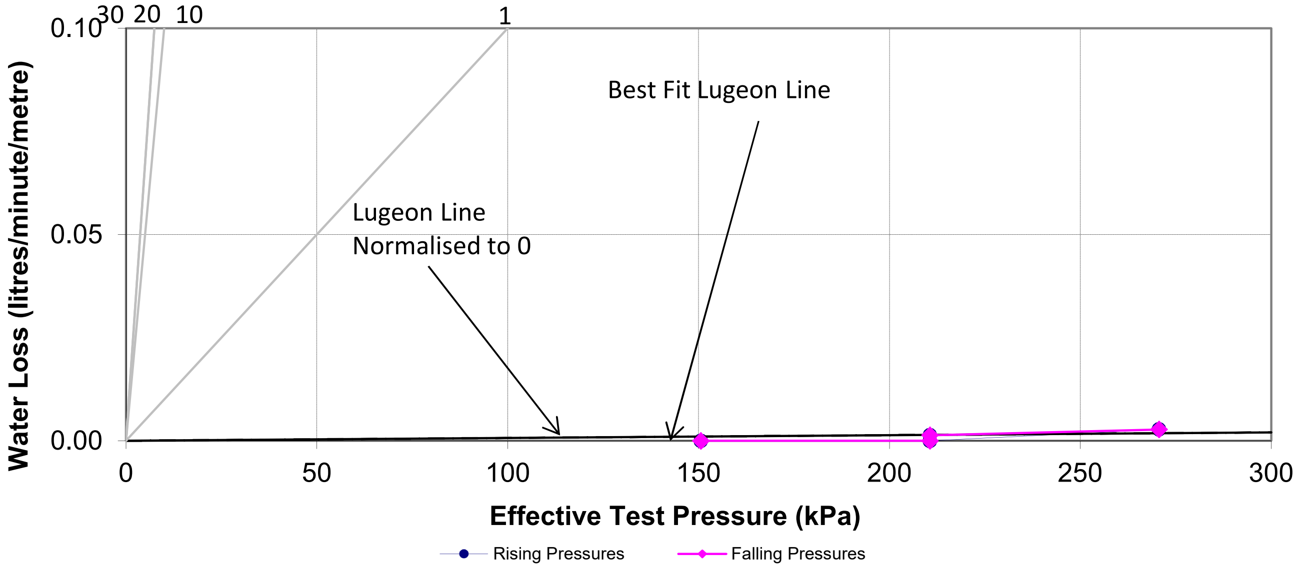
BHH143 - 23m - 37.62m

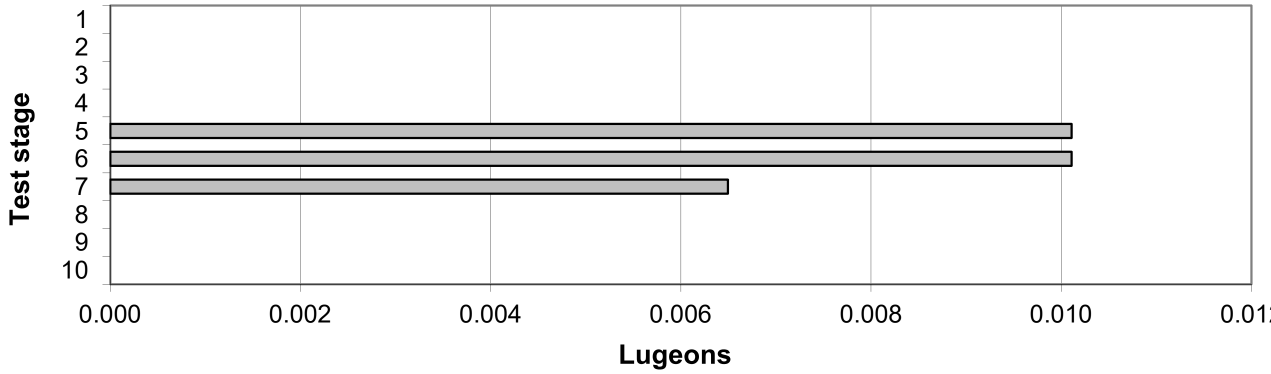
hole no:	BHH143	date:	7-Dec-16
project:	Coffs Harbour Bypass		
location:	Shephards Lane	by:	T Hosking

section tested: from 23.00 m to 37.62 m length 14.62 m

relative level at surface:	122.1 m RL	centre of test section (downhole):	30.3 m
gauge height:	1.0 m	test depth converted to vertical:	30.3 m
inclination of borehole:	90 °	relative level at centre of test section:	91.8 m RL
inferred groundwater level:	113.9 m RL	correction to gauge pressure [(1)(2)]:	90.7 kPa

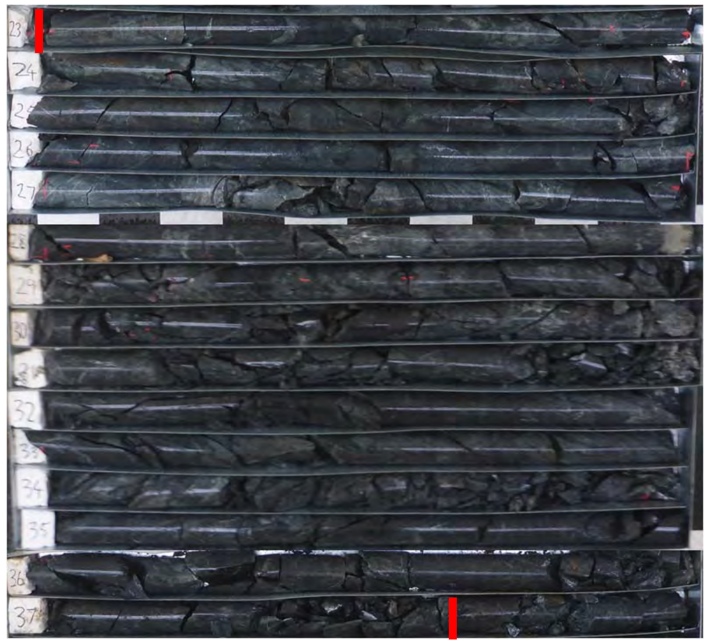
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure [(3)]	lugeons litres / min / m / 1000kPa
5	60	151	0.0	0.0	19%	0.00
5	60	151	0.0	0.0	19%	0.00
5	120	211	0.0	0.0	27%	0.00
5	120	211	0.0	0.0	27%	0.00
5	180	271	0.2	0.0	34%	0.01
5	180	271	0.2	0.0	34%	0.01
5	120	211	0.1	0.0	27%	0.01
5	120	211	0.0	0.0	27%	0.00
5	60	151	0.0	0.0	19%	0.00
5	60	151	0.0	0.0	19%	0.00





Interpreted flow mechanism: Dilation

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

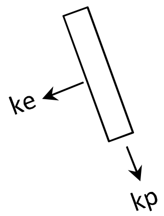
uL (Fell et al., 2005) - upper chart	0.01
uL (Houlsby, 1976) - lower chart	0.00

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	1.7.E-05	m/day
ke/kp = 100	1.5.E-05	m/day
ke/kp = 10	1.3.E-05	m/day
ke/kp = 1	1.0.E-05	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 1.1.E-05 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

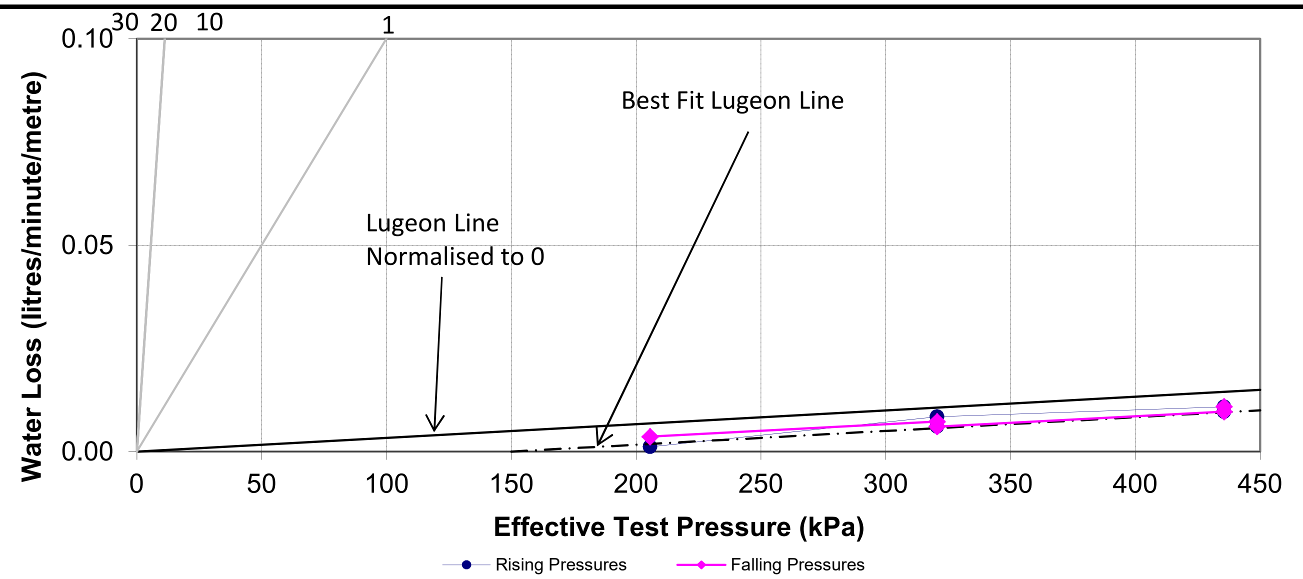
BHH143 - 39m - 55.52m

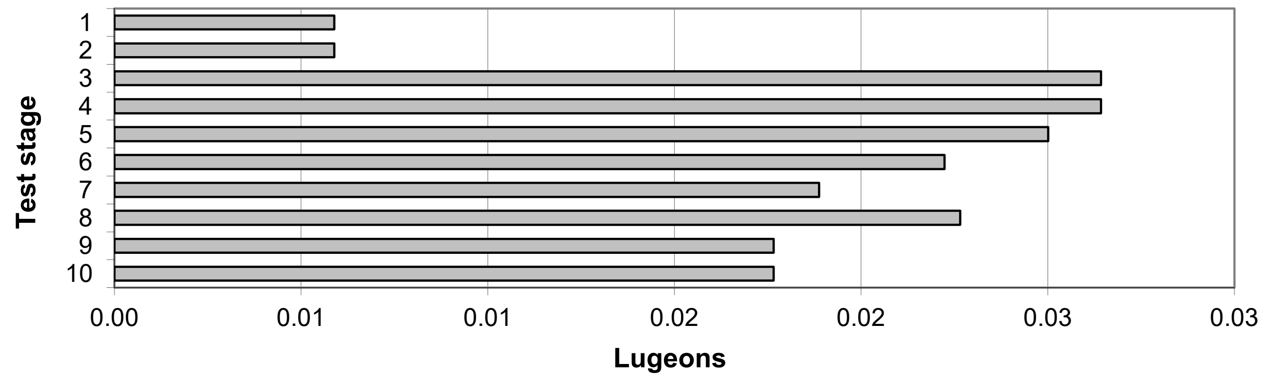
hole no: BHH143 | date: 8-Dec-16
project: Coffs Harbour Bypass
location: Shephards Lane | by: T Hosking

section tested: from 39.00 m to 55.52 m length 16.52 m

relative level at surface: 122.1 m RL | centre of test section (downhole): 47.3 m
gauge height: 1.0 m | test depth converted to vertical: 47.3 m
inclination of borehole: 90 ° | relative level at centre of test section: 74.9 m RL
inferred groundwater level: 113.9 m RL | correction to gauge pressure [(1)(2)]: 90.7 kPa

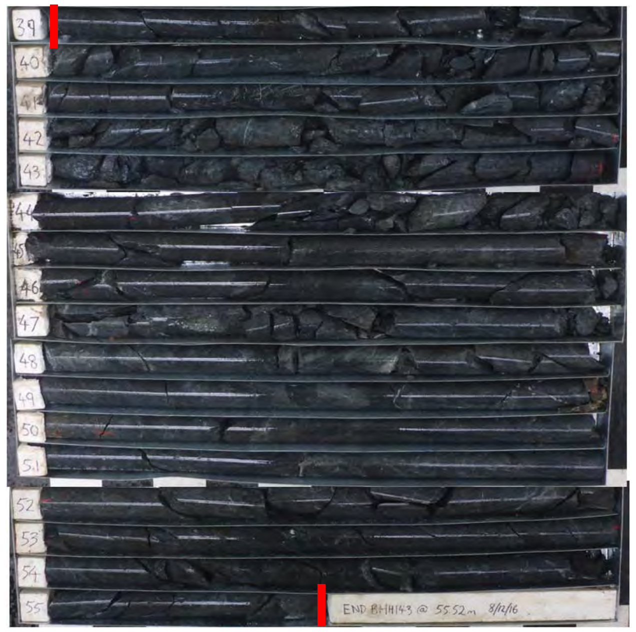
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure [(3)]	lugeons litres / min / m / 1000kPa
5	115	206	0.1	0.0	17%	0.01
5	115	206	0.1	0.0	17%	0.01
5	230	321	0.7	0.0	26%	0.03
5	230	321	0.7	0.0	26%	0.03
5	345	436	0.9	0.0	35%	0.03
5	345	436	0.8	0.0	35%	0.02
5	230	321	0.5	0.0	26%	0.02
5	230	321	0.6	0.0	26%	0.02
5	115	206	0.3	0.0	17%	0.02
5	115	206	0.3	0.0	17%	0.02





Interpreted flow mechanism: Laminr Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size: HQ3
Lithology: Argillite
Primary Defect Set: Joints

Lugeon (uL) Assessment

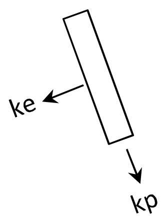
uL (Fell et al., 2005) - upper chart	0.03
uL (Houlsby, 1976) - lower chart	0.02

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	3.9.E-04	m/day
ke/kp = 100	3.5.E-04	m/day
ke/kp = 10	3.0.E-04	m/day
ke/kp = 1	2.5.E-04	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.6.E-04 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink Job no. PSM2876

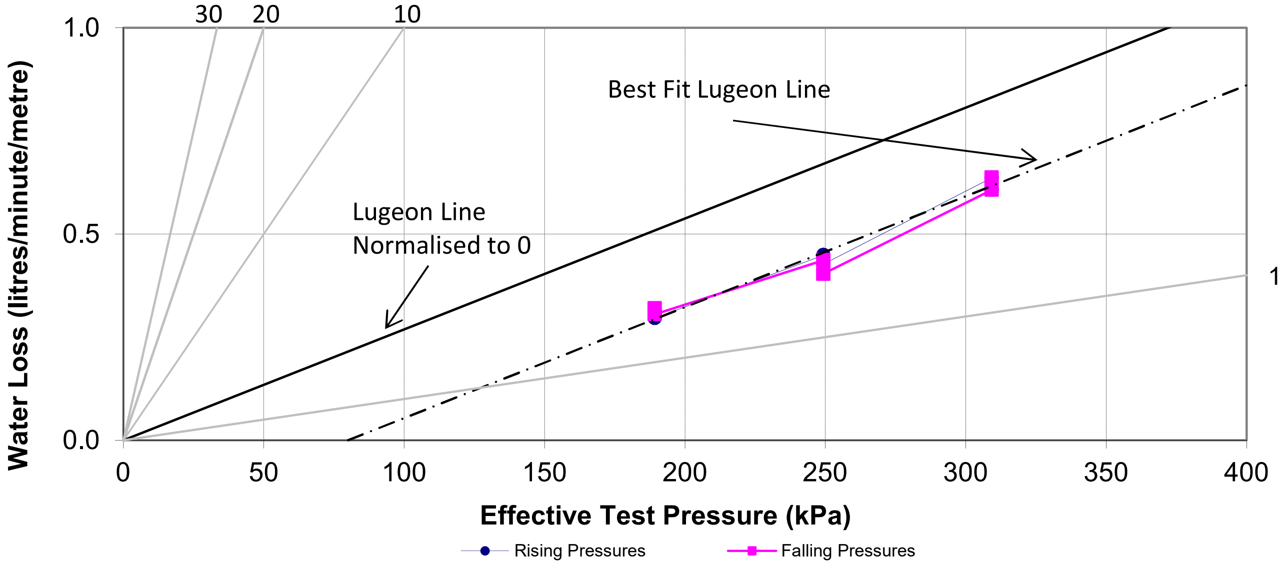
Borehole Water Pressure Test Summary **BHH144 - 22.2m - 33.2m**

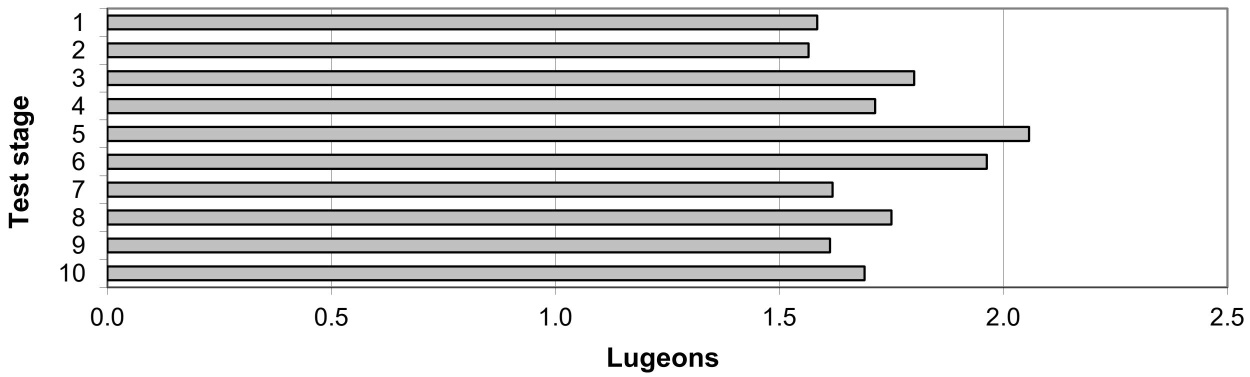
hole no:	BHH144	date:	16-Nov-16
project:	Coffs Harbour Bypass		
location:	Shephards Lane	by:	R Cater

section tested: from 22.20 m to 33.20 m length 11.00 m

relative level at surface:	105.8 m RL	centre of test section (downhole):	27.7 m
gauge height:	1.0 m	test depth converted to vertical:	27.7 m
inclination of borehole:	90 °	relative level at centre of test section:	78.1 m RL
inferred groundwater level:	93.6 m RL	correction to gauge pressure (1)(2):	129.4 kPa

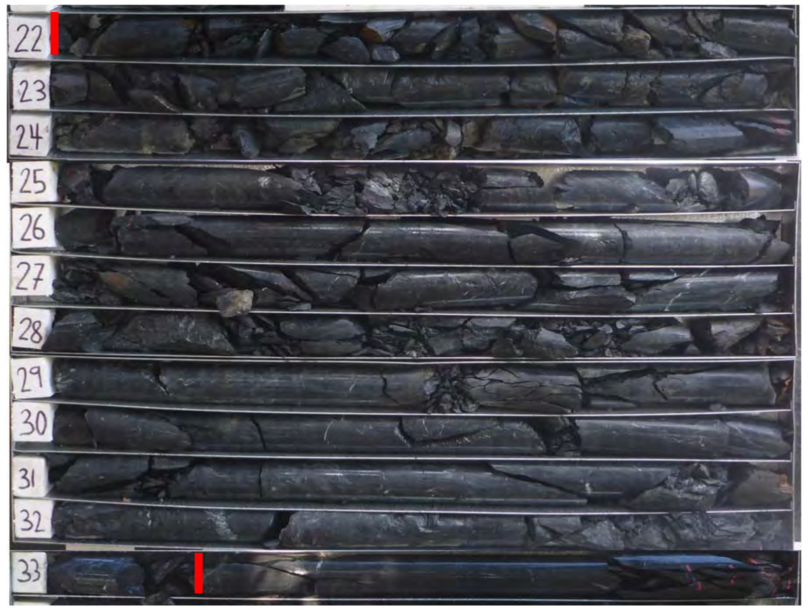
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	60	189	16.5	0.3	26%	1.6
5	60	189	16.3	0.3	26%	1.6
5	120	249	24.7	0.4	35%	1.8
5	120	249	23.5	0.4	35%	1.7
5	180	309	35.0	0.6	43%	2.1
5	180	309	33.4	0.6	43%	2.0
5	120	249	22.2	0.4	35%	1.6
5	120	249	24.0	0.4	35%	1.7
5	60	189	16.8	0.3	26%	1.6
5	60	189	17.6	0.3	26%	1.7





Interpreted flow mechanism: Laminar Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

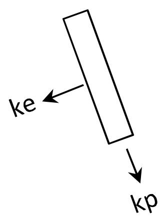
uL (Fell et al., 2005) - upper chart	2.7
uL (Houlsby, 1976) - lower chart	1.7

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	3.5.E-02	m/day
ke/kp = 100	3.0.E-02	m/day
ke/kp = 10	2.6.E-02	m/day
ke/kp = 1	2.1.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.2.E-02 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

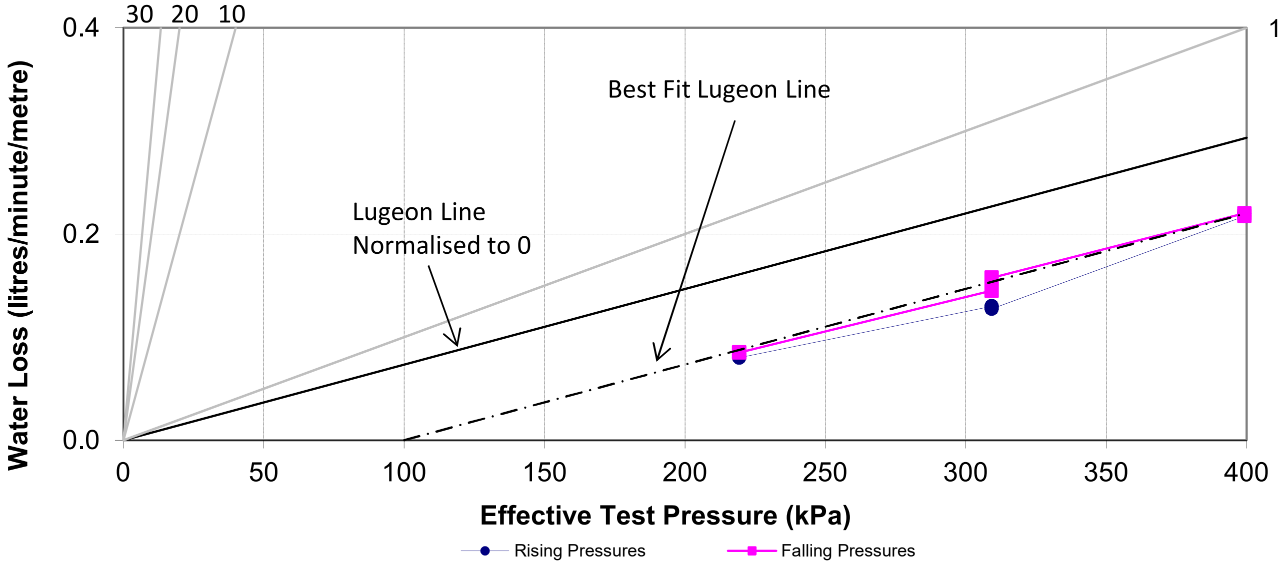
BHH144 - 32m - 40m

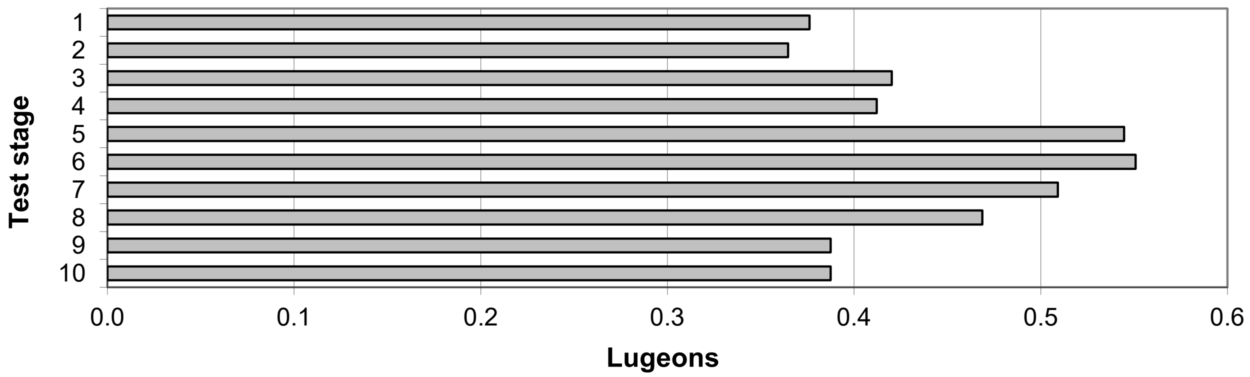
hole no:	BHH144	date:	16-Nov-16
project:	Coffs Harbour Bypass		
location:	Shephards Lane	by:	R Cater

section tested: from 32.00 m to 40.00 m length 8.00 m

relative level at surface:	105.8 m RL	centre of test section (downhole):	36.0 m
gauge height:	1.0 m	test depth converted to vertical:	36.0 m
inclination of borehole:	90 °	relative level at centre of test section:	69.8 m RL
inferred groundwater level:	93.6 m RL	correction to gauge pressure [(1)(2)]:	129.4 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure [(3)]	lugeons litres / min / m / 1000kPa
5	90	219	3.3	0.1	23%	0.4
5	90	219	3.2	0.1	23%	0.4
5	180	309	5.2	0.1	33%	0.4
5	180	309	5.1	0.1	33%	0.4
5	270	399	8.7	0.2	43%	0.5
5	270	399	8.8	0.2	43%	0.6
5	180	309	6.3	0.2	33%	0.5
5	180	309	5.8	0.1	33%	0.5
5	90	219	3.4	0.1	23%	0.4
5	90	219	3.4	0.1	23%	0.4





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints / No Core

Lugeon (uL) Assessment

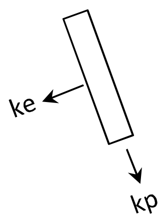
uL (Fell et al., 2005) - upper chart	0.7
uL (Houlsby, 1976) - lower chart	0.4

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	8.5.E-03	m/day
ke/kp = 100	7.4.E-03	m/day
ke/kp = 10	6.2.E-03	m/day
ke/kp = 1	5.1.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 5.4.E-03 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

A3 – Gatelys Road Packer Testing Data

A3 – Packer Test Summary Table – Gatelys Road

BHID	Test From (m)	Test To (m)	Lugeon Value 1 (1)	Lugeon Value 2 (2)	Lugeon Classification	Weathering	RMU	Hydraulic Conductivity (m/d) (4)
BHH149	22.45	33.85	13.9	12.7	Mod/High	MW	D1	0.16
BHH149	28.45	33.85	0.2	0.1	Low	SW	D1	1.10×10^{-3}
BHH149	34.45	45.85	8.9	7.0	Mod/High	SW	D1	9.10×10^{-2}
BHH149	40.45	45.85	20.4	15.3	Mod/High	SW	D1	0.17
BHH149	45.85	57.85	0.3	0.2	Low	F	D2	2.80×10^{-3}
BHH149	58.45	69.85	0	0	Low	F	D2	2.40×10^{-2}
BHH149 (3)	63.85	75.85	10.9	5.9	Mod/High	F	D2	7.70×10^{-2}
BHH149 (3)	69.85	75.85	3.1	3.2	Mod/High	F	D2	3.70×10^{-2}
BHH149 (3)	75.85	87.85	2.0	5.2	Low/Mod	F	D2	6.80×10^{-2}
BHH149 (3)	81.85	87.85	4.0	11.1	Low/Mod to Mod/High	F	D2	0.13
BHH149	88	98	3.3	6.8	Low/Mod to Mod/High	F	D2	8.60×10^{-2}
BHH149	93	98	14.6	16.1	Mod/High	F	D2	0.18
BHH150 (3)	31	42.85	9.0	11.5	Mod/High	SW	D1	0.15
BHH150 (3)	37	42.85	N/A	21.8	Mod/High	SW	D1	0.25
BHH150	42.5	54.03	0.5	0.2	Low	F	D2	2.90×10^{-3}
BHH150	55.13	66.23	1.2	0.6	Low	F	D2	8.10×10^{-3}
BHH150	67.6	78.6	0.12	0.04	Low	F	D2	5.10×10^{-4}
BHH150	80.27	91.27	0.11	0.05	Low	F	D2	7.00×10^{-4}

BHID	Test From (m)	Test To (m)	Lugeon Value 1 (1)	Lugeon Value 2 (2)	Lugeon Classification	Weathering	RMU	Hydraulic Conductivity (m/d) (4)
BHH151	7.85	12.85	5.5	18.4	Mod/High	MW	D1	0.21
BHH151	13.85	18.85	30.0	38.1	Mod/High	MW	D1	0.43
BHH151	18.1	24.85	12.3	10.2	Mod/High	MW	D1	0.12
BHH151 (3)	25.65	30.65	N/A	43.1	Mod/High	SW	D1	0.48
BHH151	33.96	36.96	22.9	28.1	Mod/High	SW	D1	0.28
BHH151	37.4	42.8	20.7	2.5	Low/Mod to Mod/High	SW	D1	2.90×10^{-2}
BHH151	43.4	48.8	3.6	3.1	Low/Mod	F	D2	3.50×10^{-2}
BHH151	49.5	54.7	0.24	0.12	Low	F	D2	1.30×10^{-3}
BHH151	55.4	60.65	0.21	0.09	Low	F	D2	9.90×10^{-4}
BHH151	61.4	66.65	0.4	0.2	Low	F	D2	2.20×10^{-3}
BHH151	67.4	72.65	0.2	0.2	Low	F	D2	2.00×10^{-3}
BHH151	73.4	78.65	0.05	0.03	Low	F	D2	3.90×10^{-4}
BHH151	79.4	84.65	0.5	0.2	Low	F	D2	2.70×10^{-3}
BHH151	85.4	90.65	8.8	5.3	Mod/High	F	D2	5.90×10^{-2}
BHH151	91.4	96.4	0.03	0.04	Low	F	D2	4.10×10^{-4}
BHH151	96.4	101.69	0.06	0.06	Low	F	D2	6.20×10^{-4}
BHH152	20	31	0.03	0.00	Low	SW	D1	9.70×10^{-4}
BHH152	32	49	1.1	0.5	Low	F	D2	7.20×10^{-3}
BHH152 (3)	51	62	N/A	N/A	N/A	F	D2	0.15

BHID	Test From (m)	Test To (m)	Lugeon Value 1 (1)	Lugeon Value 2 (2)	Lugeon Classification	Weathering	RMU	Hydraulic Conductivity (m/d) (4)
BHH152	57	62	2.1	1.1	Low/Mod	F	D2	1.20×10^{-2}
BHH152	62.26	74.76	0.2	0.2	Low	F	D2	2.70×10^{-3}
BHH152	74.96	85.96	0.6	0.1	Low	F	D2	1.40×10^{-3}
BHH152	87.2	98.2	0.12	0.11	Low	F	D2	1.40×10^{-3}
BHH153	16	27	5.1	2.5	Low/Mod	MW	D1	3.30×10^{-2}
BHH153	27.07	38.07	0.23	0.07	Low	SW	D1	9.70×10^{-4}
BHH153	39	50	0.6	0.3	Low	F	D2	3.40×10^{-3}
BHH153	51	62	2.6	1.9	Low/Mod	F	D2	2.40×10^{-2}
BHH153	63	74	0.12	0.06	Low	F	D2	7.40×10^{-4}
BHH154	17	27.9	0.22	0.13	Low	F	C2	3.60×10^{-5}
BHH154	29	44.55	0.03	0.0	Low	F	C2	1.60×10^{-3}

Notes:

1 Calculated from Fell et al., 2005

2 Calculated from Houlsby, 1976

3 Target pressure not achieved during testing, may not be a valid test

4 Calculated from Moye, 1967

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Test undertaken above water table

Job no. PSM2876

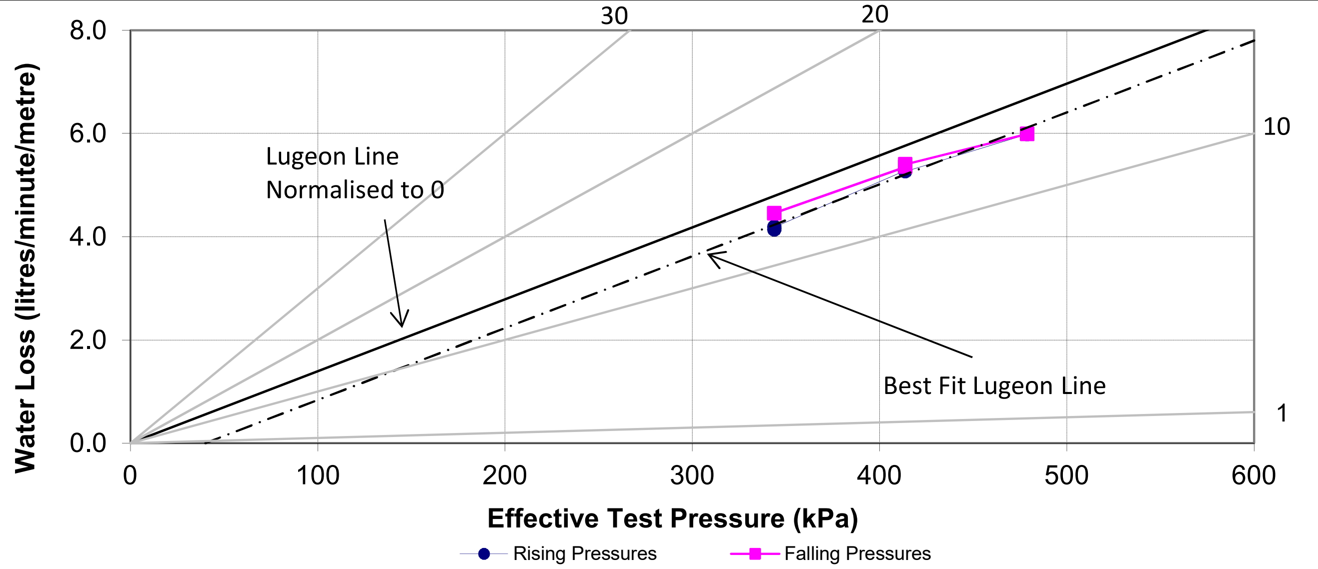
BHH149 - 22.45m - 33.85m

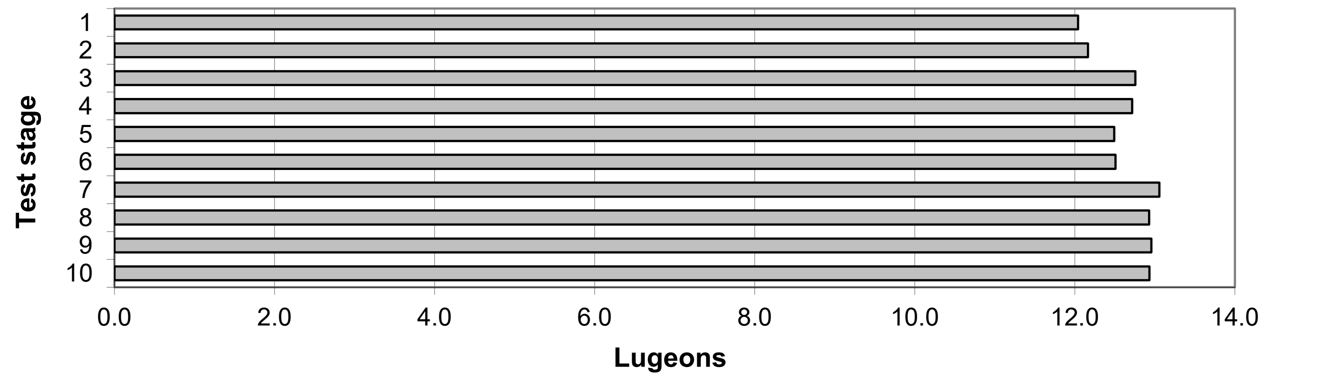
hole no:	BHH149	date:	11-Jan-17
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

section tested: from 22.45 m to 33.85 m length 11.40 m

relative level at surface:	138.7 m RL	centre of test section (downhole):	28.2 m
gauge height:	1.5 m	test depth converted to vertical:	26.5 m
inclination of borehole:	70 °	relative level at centre of test section:	112.2 m RL
inferred groundwater level:	106.7 m RL	correction to gauge pressure (1)(2):	273.9 kPa

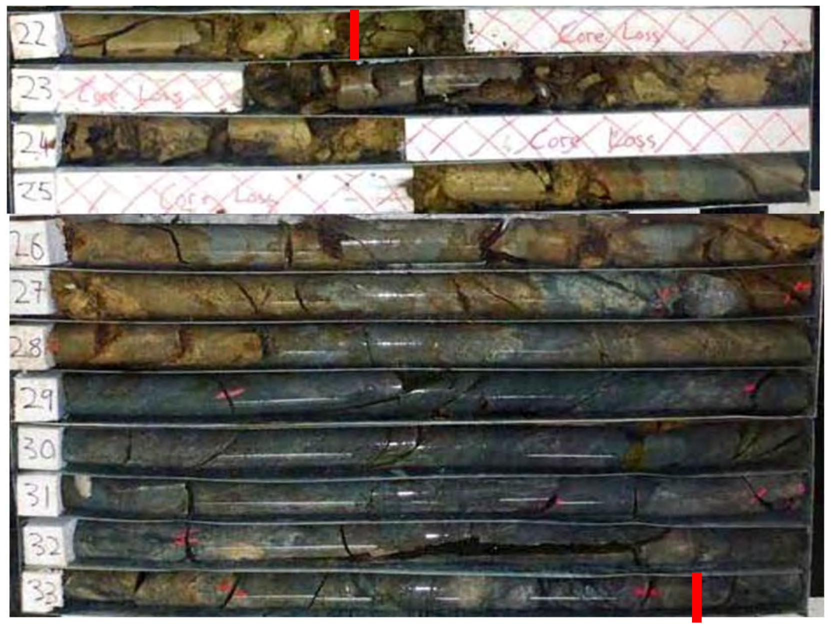
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	70	344	236.0	4.1	50%	12.0
5	70	344	238.5	4.2	50%	12.2
5	140	414	301.0	5.3	60%	12.8
5	140	414	300.0	5.3	60%	12.7
5	205	479	341.0	6.0	70%	12.5
5	205	479	341.5	6.0	70%	12.5
5	140	414	308.0	5.4	60%	13.1
5	140	414	305.0	5.4	60%	12.9
5	70	344	254.0	4.5	50%	13.0
5	70	344	253.5	4.4	50%	12.9





Interpreted flow mechanism: Laminar Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size: HQ3

Lithology: Argillite

Primary Defect Set: Joints / Core Loss

Lugeon (uL) Assessment

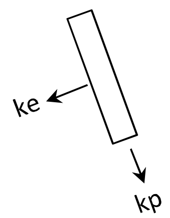
uL (Fell et al., 2005) - upper chart	13.9
uL (Houlsby, 1976) - lower chart	12.7

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	2.5.E-01	m/day
ke/kp = 100	2.2.E-01	m/day
ke/kp = 10	1.9.E-01	m/day
ke/kp = 1	1.6.E-01	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 1.6.E-01 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Test undertaken above water table

Job no. PSM2876

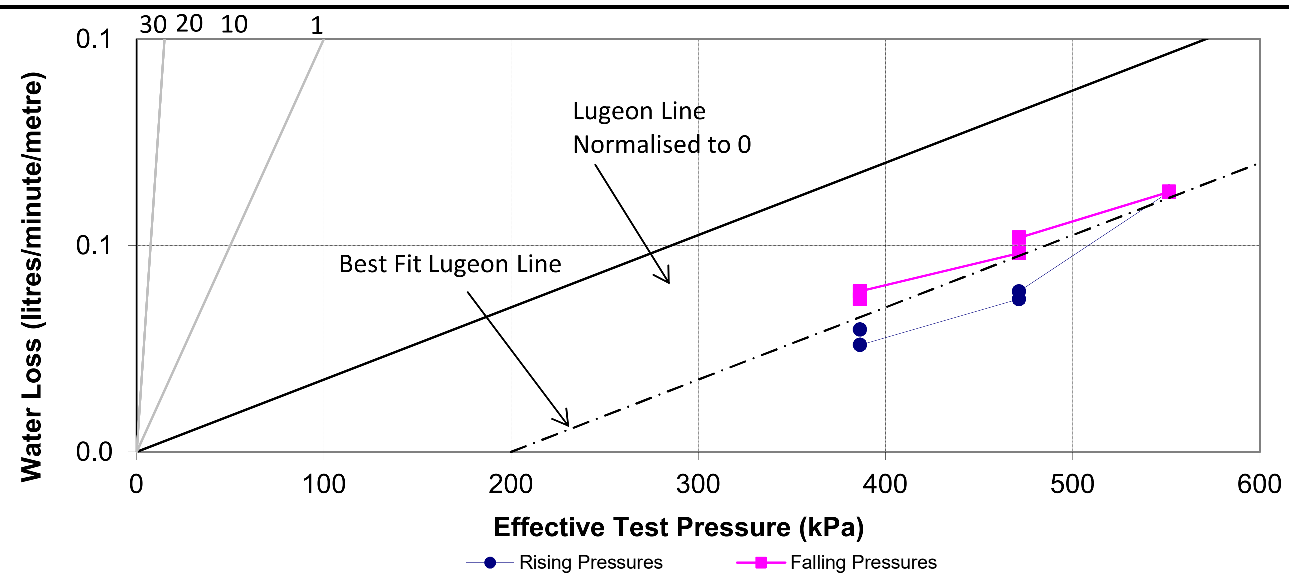
BHH149 - 28.45m - 33.85m

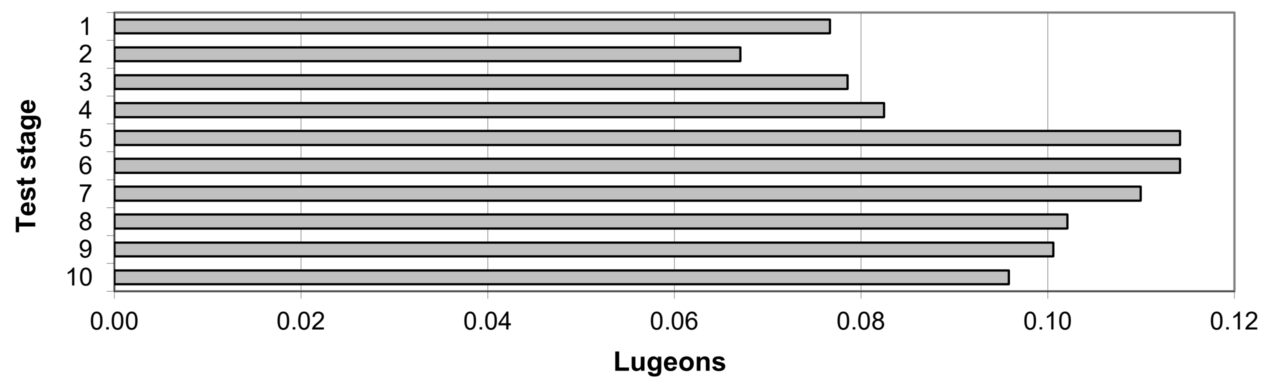
hole no:	BHH149	date:	12-Jan-17
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

section tested: from 28.45 m to 33.85 m length 5.40 m

relative level at surface:	138.7 m RL	centre of test section (downhole):	31.2 m
gauge height:	1.5 m	test depth converted to vertical:	29.3 m
inclination of borehole:	70 °	relative level at centre of test section:	109.4 m RL
inferred groundwater level:	106.7 m RL	correction to gauge pressure (1)(2):	301.6 kPa

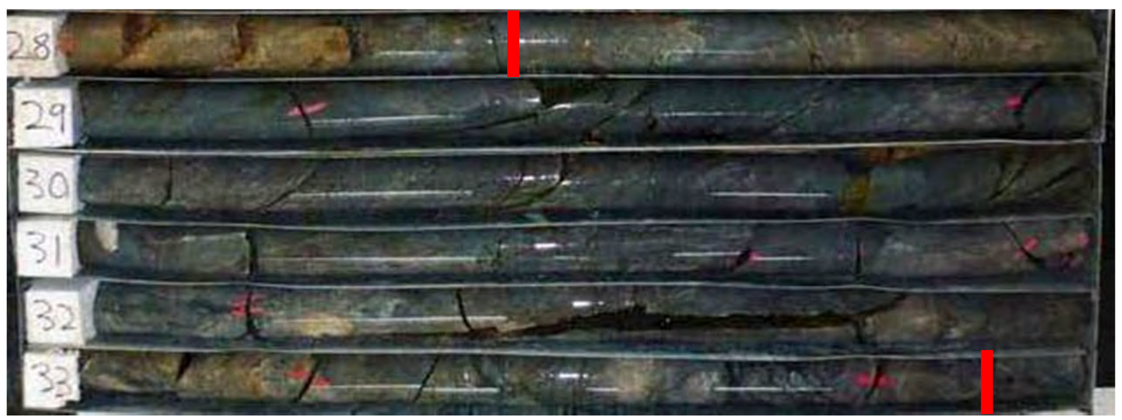
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	85	387	0.8	0.0	51%	0.08
5	85	387	0.7	0.0	51%	0.07
5	170	472	1.0	0.0	62%	0.08
5	170	472	1.1	0.0	62%	0.08
5	250	552	1.7	0.1	72%	0.11
5	250	552	1.7	0.1	72%	0.11
5	170	472	1.4	0.1	62%	0.11
5	170	472	1.3	0.0	62%	0.10
5	85	387	1.1	0.0	51%	0.10
5	85	387	1.0	0.0	51%	0.10





Interpreted flow mechanism: Laminar Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

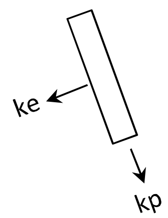
uL (Fell et al., 2005) - upper chart	0.2
uL (Houlsby, 1976) - lower chart	0.1

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	1.7.E-03	m/day
ke/kp = 100	1.5.E-03	m/day
ke/kp = 10	1.2.E-03	m/day
ke/kp = 1	1.0.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 1.1.E-03 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

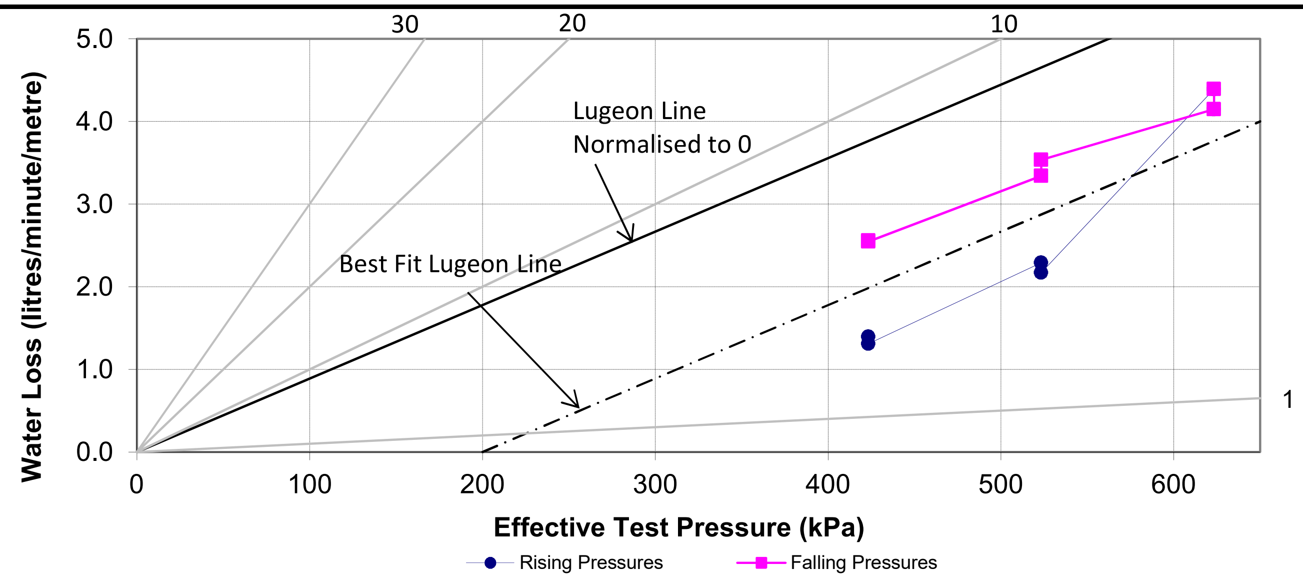
BHH149 - 34.45m - 45.88m

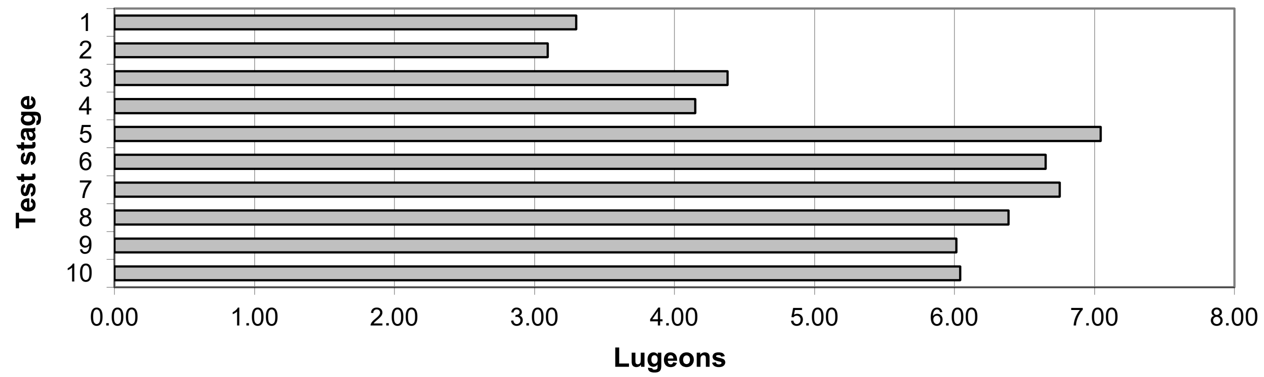
hole no:	BHH149	date:	13-Jan-17
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

section tested: from 34.45 m to 45.88 m length 11.43 m

relative level at surface:	138.7 m RL	centre of test section (downhole):	40.2 m
gauge height:	1.0 m	test depth converted to vertical:	37.7 m
inclination of borehole:	70 °	relative level at centre of test section:	101.0 m RL
inferred groundwater level:	106.7 m RL	correction to gauge pressure (1)(2):	323.4 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	100	423	79.8	1.4	43%	3.30
5	100	423	74.9	1.3	43%	3.10
5	200	523	131.0	2.3	53%	4.38
5	200	523	124.1	2.2	53%	4.15
5	300	623	251.0	4.4	64%	7.05
5	300	623	237.0	4.1	64%	6.65
5	200	523	202.0	3.5	53%	6.75
5	200	523	191.0	3.3	53%	6.39
5	100	423	145.5	2.5	43%	6.01
5	100	423	146.2	2.6	43%	6.04





Interpreted flow mechanism: Wash Out

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

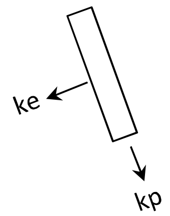
uL (Fell et al., 2005) - upper chart	8.9
uL (Houlsby, 1976) - lower chart	7.0

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	1.4.E-01	m/day
ke/kp = 100	1.2.E-01	m/day
ke/kp = 10	1.0.E-01	m/day
ke/kp = 1	8.7.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 9.1.E-02 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

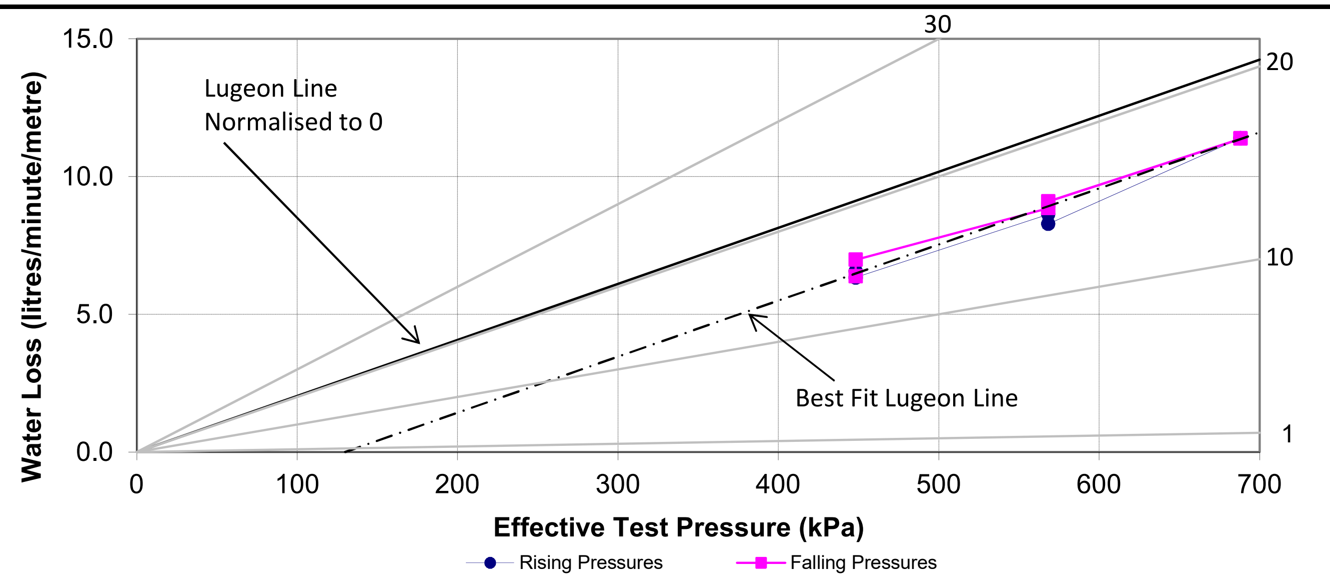
BHH149 - 40.45m - 45.85m

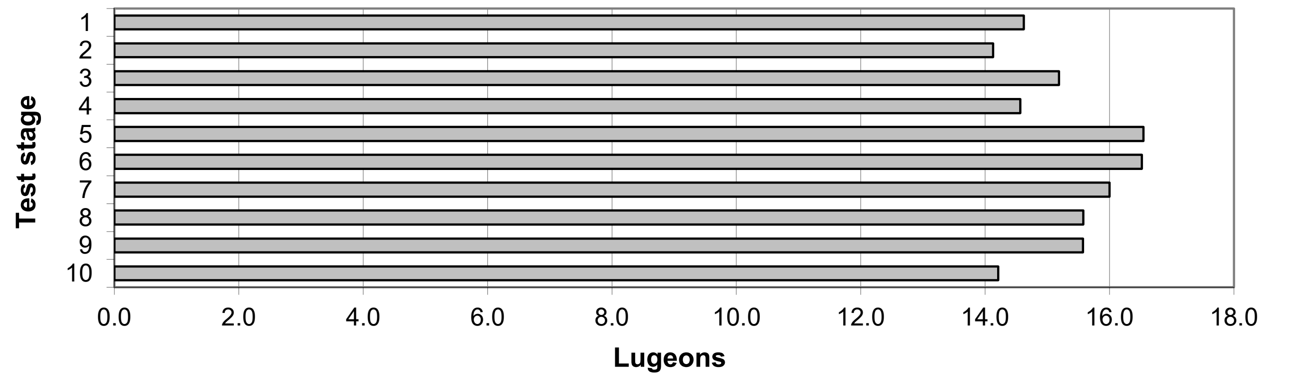
hole no:	BHH149	date:	12-Jan-17
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

section tested: from 40.45 m to 45.85 m length 5.40 m

relative level at surface:	138.7 m RL	centre of test section (downhole):	43.2 m
gauge height:	1.5 m	test depth converted to vertical:	40.5 m
inclination of borehole:	70 °	relative level at centre of test section:	98.2 m RL
inferred groundwater level:	106.7 m RL	correction to gauge pressure (1)(2):	328.3 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	120	448	177.0	6.6	43%	14.6
5	120	448	171.0	6.3	43%	14.1
5	240	568	233.0	8.6	54%	15.2
5	240	568	223.5	8.3	54%	14.6
5	360	688	307.5	11.4	65%	16.5
5	360	688	307.0	11.4	65%	16.5
5	240	568	245.5	9.1	54%	16.0
5	240	568	239.0	8.9	54%	15.6
5	120	448	188.5	7.0	43%	15.6
5	120	448	172.0	6.4	43%	14.2





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

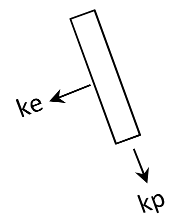
uL (Fell et al., 2005) - upper chart	20.4
uL (Houlsby, 1976) - lower chart	15.3

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	2.8.E-01	m/day
ke/kp = 100	2.4.E-01	m/day
ke/kp = 10	2.0.E-01	m/day
ke/kp = 1	1.6.E-01	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 1.7.E-01 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

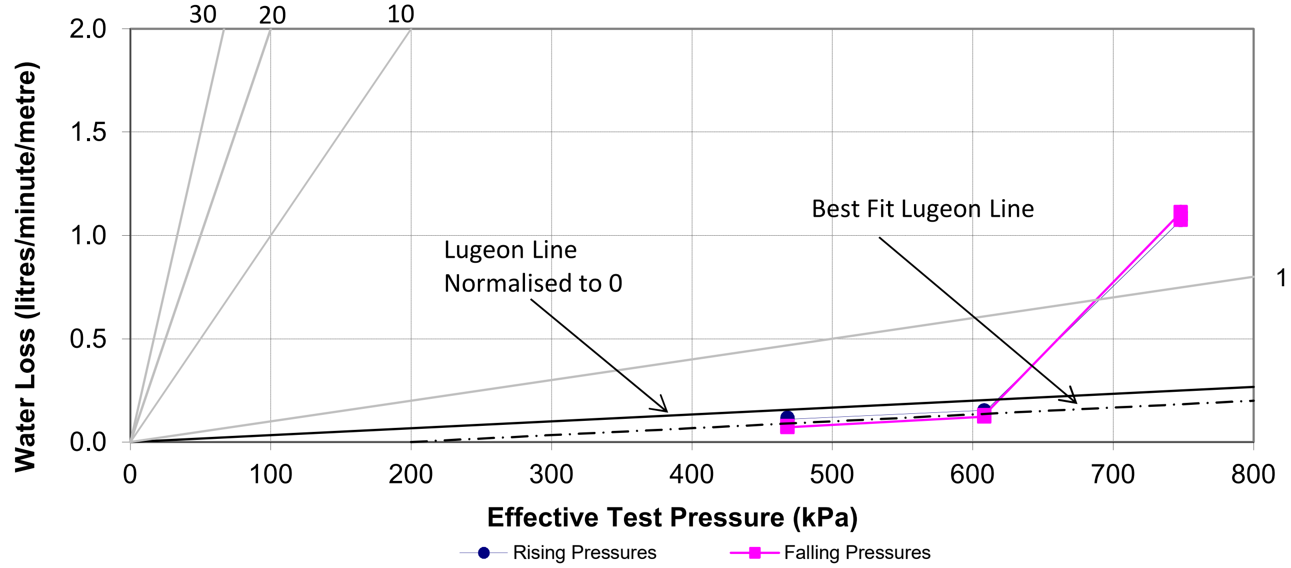
BHH149 - 45.85m - 57.85m

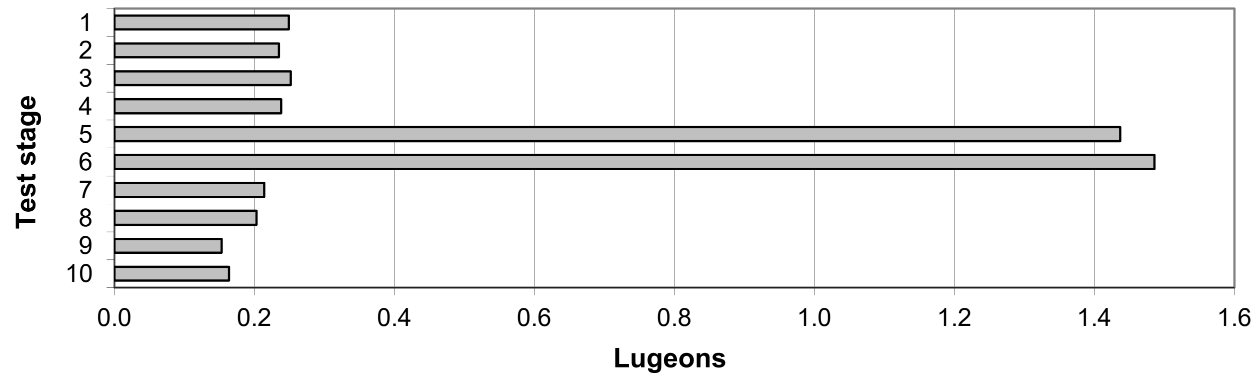
hole no:	BHH149	date:	17-Jan-17
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

section tested: from 45.85 m to 57.85 m length 12.00 m

relative level at surface:	138.7 m RL	centre of test section (downhole):	51.9 m
gauge height:	1.5 m	test depth converted to vertical:	48.7 m
inclination of borehole:	70 °	relative level at centre of test section:	90.0 m RL
inferred groundwater level:	106.7 m RL	correction to gauge pressure [(1)(2)]:	328.3 kPa

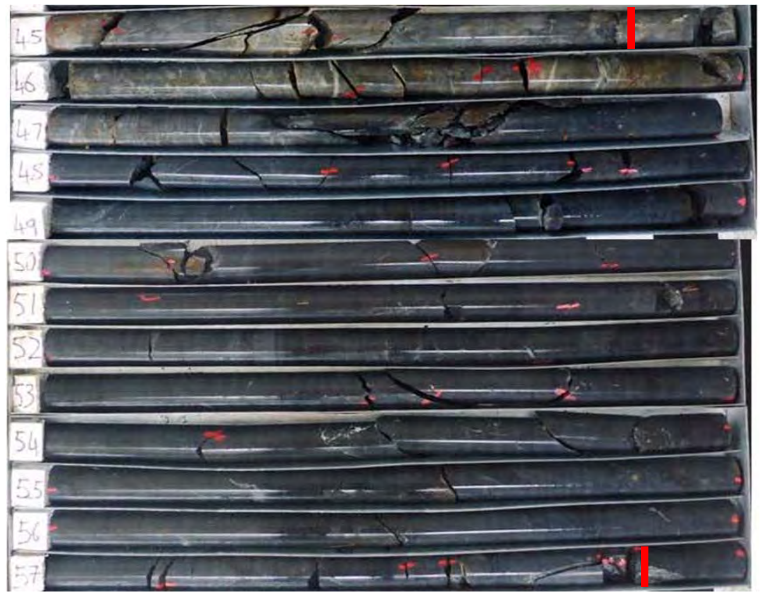
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure [(3)]	lugeons litres / min / m / 1000kPa
5	140	468	7.0	0.1	37%	0.2
5	140	468	6.6	0.1	37%	0.2
5	280	608	9.2	0.2	48%	0.3
5	280	608	8.7	0.1	48%	0.2
5	420	748	64.5	1.1	59%	1.4
5	420	748	66.7	1.1	59%	1.5
5	280	608	7.8	0.1	48%	0.2
5	280	608	7.4	0.1	48%	0.2
5	140	468	4.3	0.1	37%	0.2
5	140	468	4.6	0.1	37%	0.2





Interpreted flow mechanism: Dilation

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

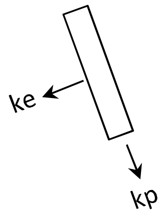
uL (Fell et al., 2005) - upper chart	0.3
uL (Houlsby, 1976) - lower chart	0.2

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	4.3.E-03	m/day
ke/kp = 100	3.8.E-03	m/day
ke/kp = 10	3.2.E-03	m/day
ke/kp = 1	2.6.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.8.E-03 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

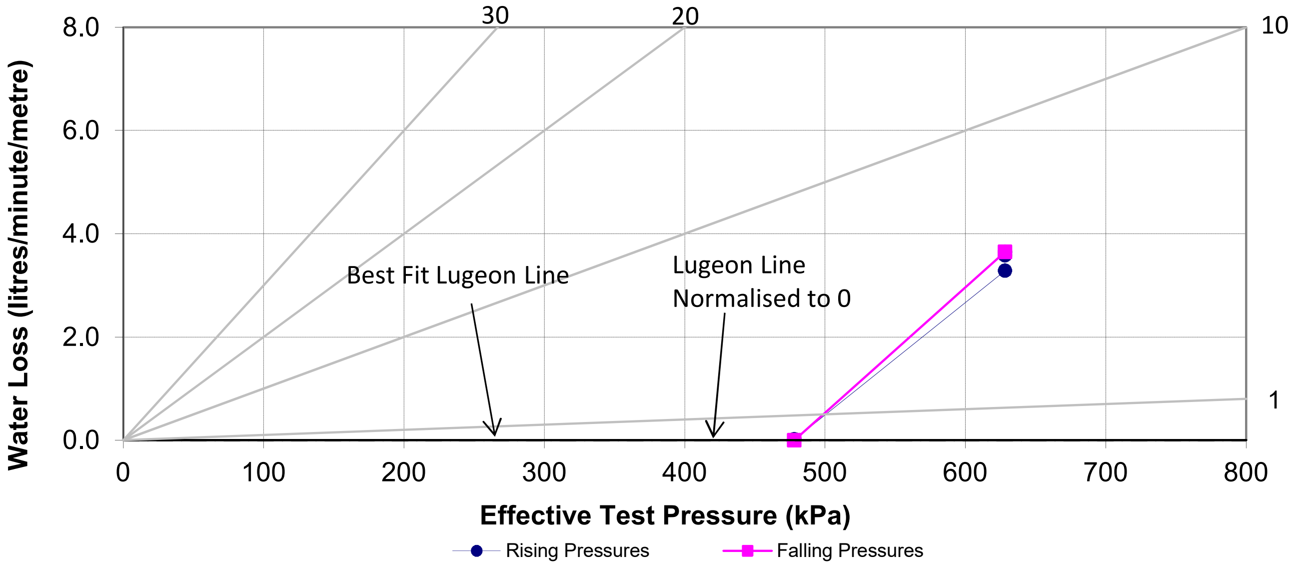
BHH149 - 58.45m - 69.85m

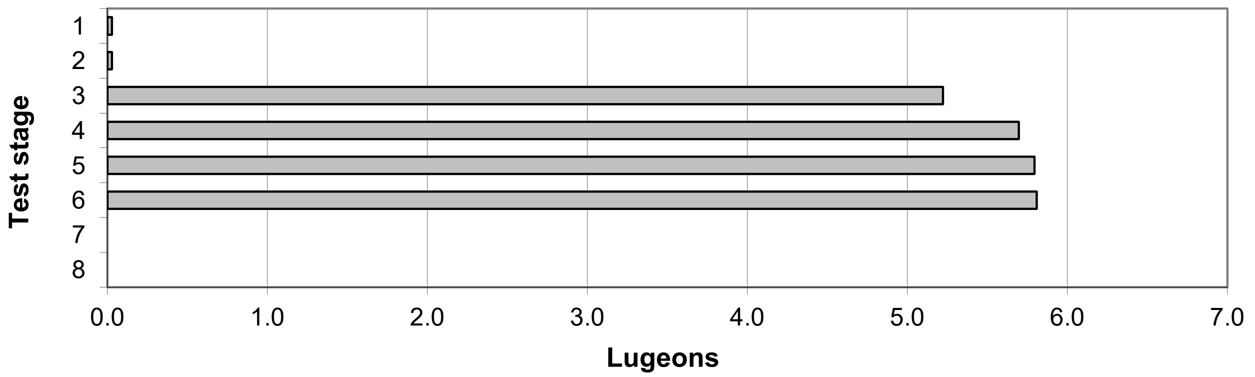
hole no:	BHH149	date:	18-Jan-17
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

section tested: from 58.45 m to 69.85 m length 11.40 m

relative level at surface:	138.7 m RL	centre of test section (downhole):	64.2 m
gauge height:	1.5 m	test depth converted to vertical:	60.3 m
inclination of borehole:	70 °	relative level at centre of test section:	78.4 m RL
inferred groundwater level:	106.7 m RL	correction to gauge pressure (1)(2):	328.3 kPa

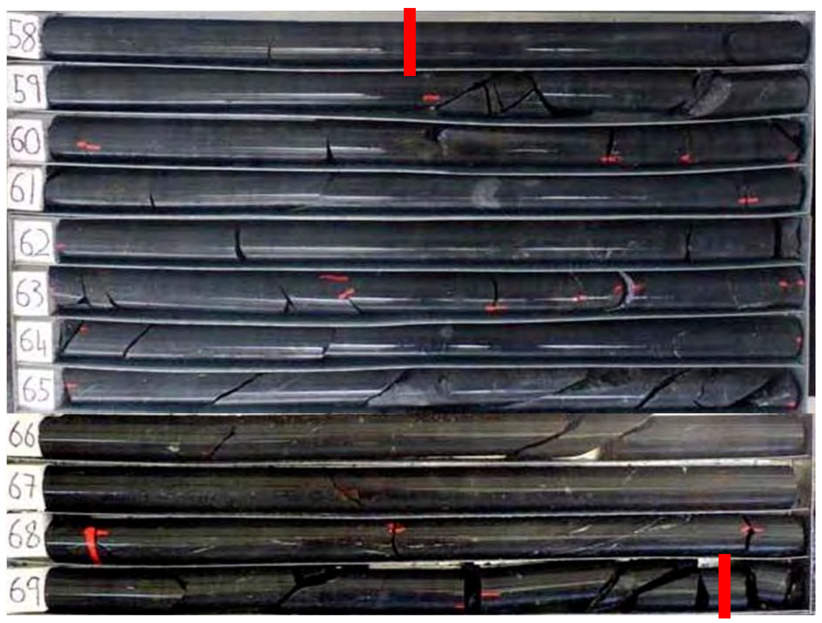
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	478	0.8	0.0	31%	0.0
5	150	478	0.8	0.0	31%	0.0
5	300	628	187.0	3.3	40%	5.2
5	300	628	204.0	3.6	40%	5.7
5	300	628	207.5	3.6	40%	5.8
5	300	628	208.0	3.6	40%	5.8
5	150	478	0.0	0.0	31%	0.0
5	150	478	0.0	0.0	31%	0.0





Interpreted flow mechanism: Dilation

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

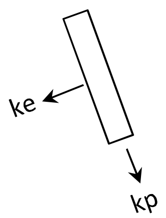
uL (Fell et al., 2005) - upper chart	0.0
uL (Houlsby, 1976) - lower chart	0.0

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	3.7.E-02	m/day
ke/kp = 100	3.2.E-02	m/day
ke/kp = 10	2.7.E-02	m/day
ke/kp = 1	2.2.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.4.E-02 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink | Job no. PSM2876

Borehole Water Pressure Test Summary | **BHH149 - 63.85m - 75.85m**

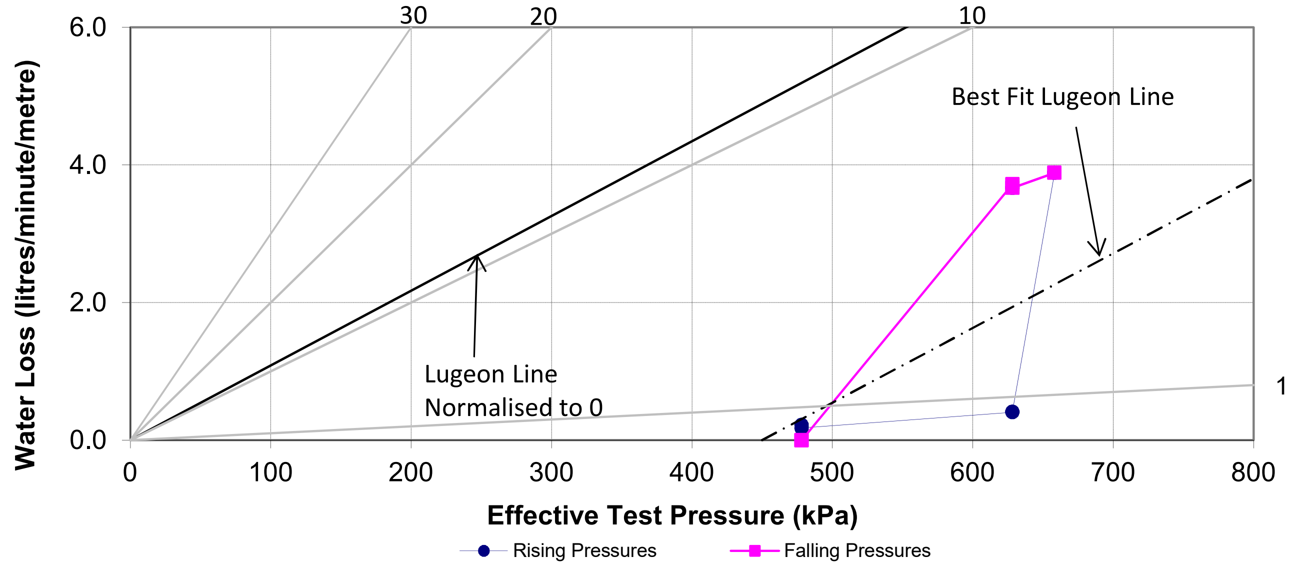
hole no:	BHH149	date:	19-Jan-17
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

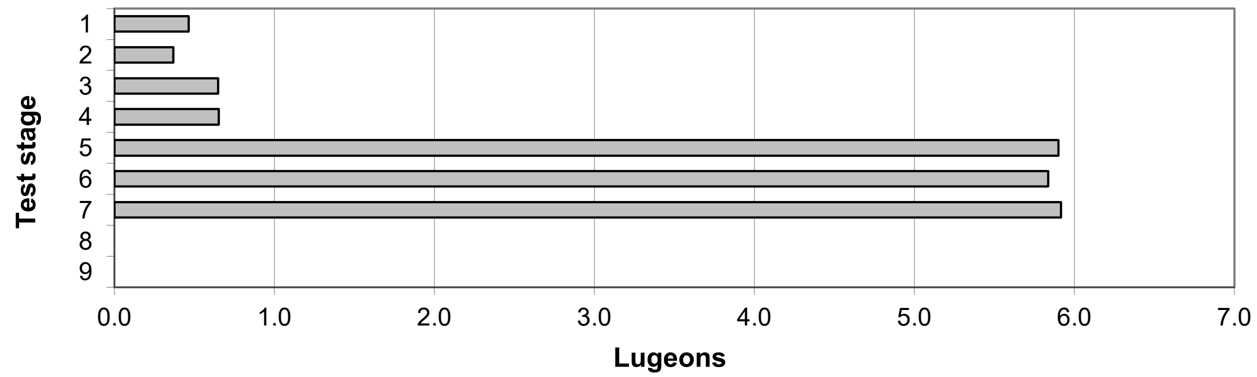
section tested: from 63.85 m to 75.85 m length 12.00 m

relative level at surface:	138.7 m RL	centre of test section (downhole):	69.9 m
gauge height:	1.5 m	test depth converted to vertical:	65.6 m
inclination of borehole:	70 °	relative level at centre of test section:	73.1 m RL
inferred groundwater level:	106.7 m RL	correction to gauge pressure (1)(2):	328.3 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	478	13.3	0.2	28%	0.5
5	150	478	10.6	0.2	28%	0.4
5	300	628	24.4	0.4	37%	0.6
5	300	628	24.6	0.4	37%	0.7
5	330	658	233.0	3.9	39%	5.9
5	300	628	220.0	3.7	37%	5.8
5	300	628	223.0	3.7	37%	5.9
5	150	478	0.0	0.0	28%	0.0
5	150	478	0.0	0.0	28%	0.0

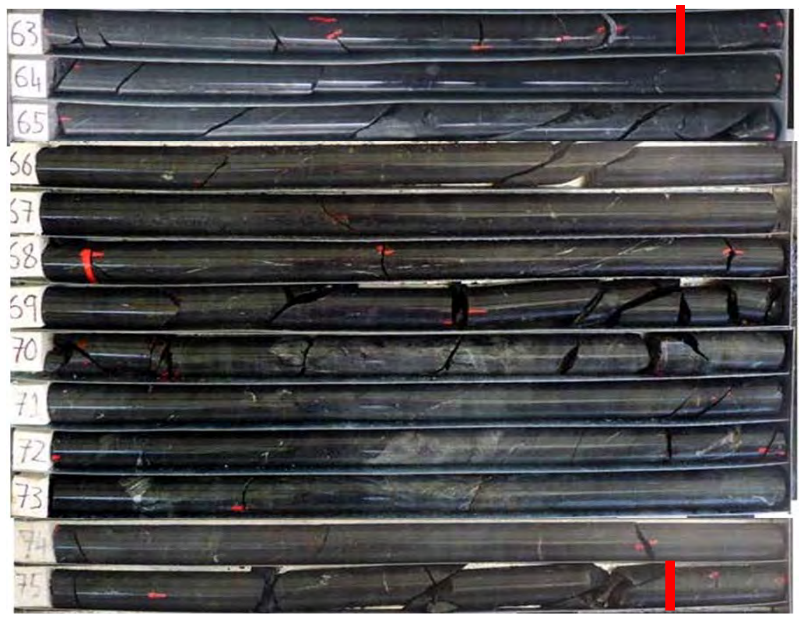
330kPa maximum pressure achievable during test





Interpreted flow mechanism: Wash Out

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

uL (Fell et al., 2005) - upper chart	10.9
uL (Houlsby, 1976) - lower chart	5.9

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:

ke
kp

ke/kp = 1000	1.2.E-01	m/day
ke/kp = 100	1.0.E-01	m/day
ke/kp = 10	8.9.E-02	m/day
ke/kp = 1	7.3.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 7.7.E-02 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink | **Job no.** PSM2876

Borehole Water Pressure Test Summary | **BHH149 - 69.85m - 75.85m**

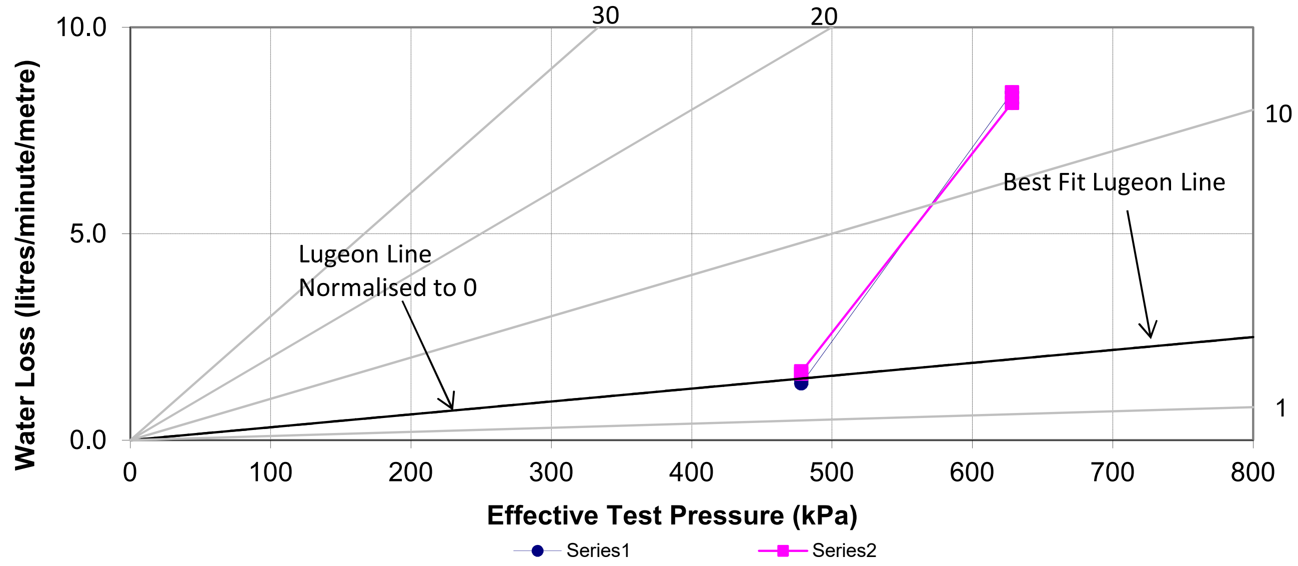
hole no:	BHH149	date:	19-Jan-17
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

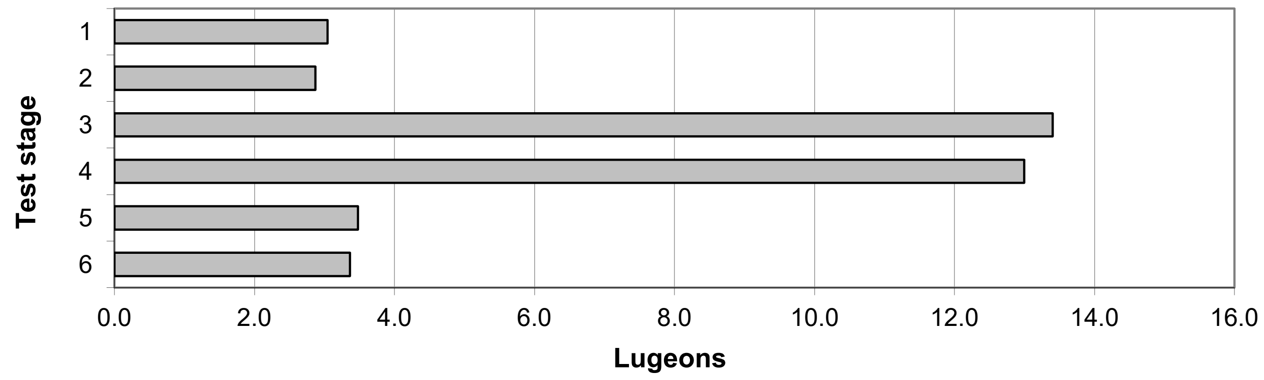
section tested: from 69.85 m to 75.85 m length 6.00 m

relative level at surface:	138.7 m RL	centre of test section (downhole):	72.9 m
gauge height:	1.5 m	test depth converted to vertical:	68.5 m
inclination of borehole:	70 °	relative level at centre of test section:	70.2 m RL
inferred groundwater level:	106.7 m RL	correction to gauge pressure (1)(2):	328.3 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	478	43.7	1.5	27%	3.0
5	150	478	41.2	1.4	27%	2.9
5	300	628	252.6	8.4	35%	13.4
5	300	628	245.0	8.2	35%	13.0
5	150	478	49.9	1.7	27%	3.5
5	150	478	48.3	1.6	27%	3.4

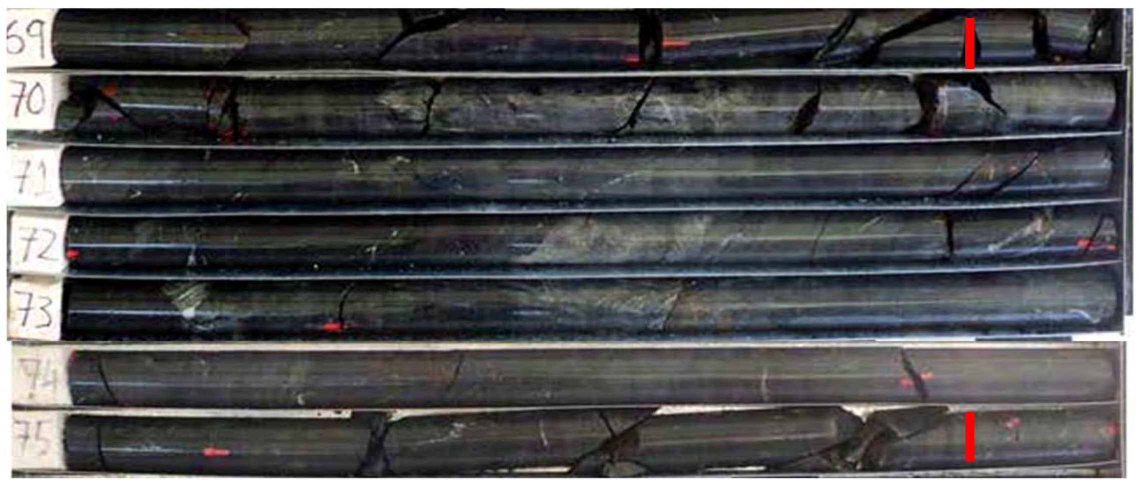
300kPa maximum pressure achievable during test





Interpreted flow mechanism: Dilation

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

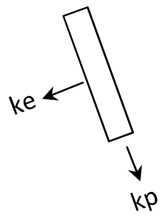
uL (Fell et al., 2005) - upper chart	3.1
uL (Houlsby, 1976) - lower chart	3.2

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	5.9.E-02	m/day
ke/kp = 100	5.1.E-02	m/day
ke/kp = 10	4.3.E-02	m/day
ke/kp = 1	3.5.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 3.7.E-02 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink — Job no. PSM2876

Borehole Water Pressure Test Summary — **BHH149 - 75.85m - 87.85m**

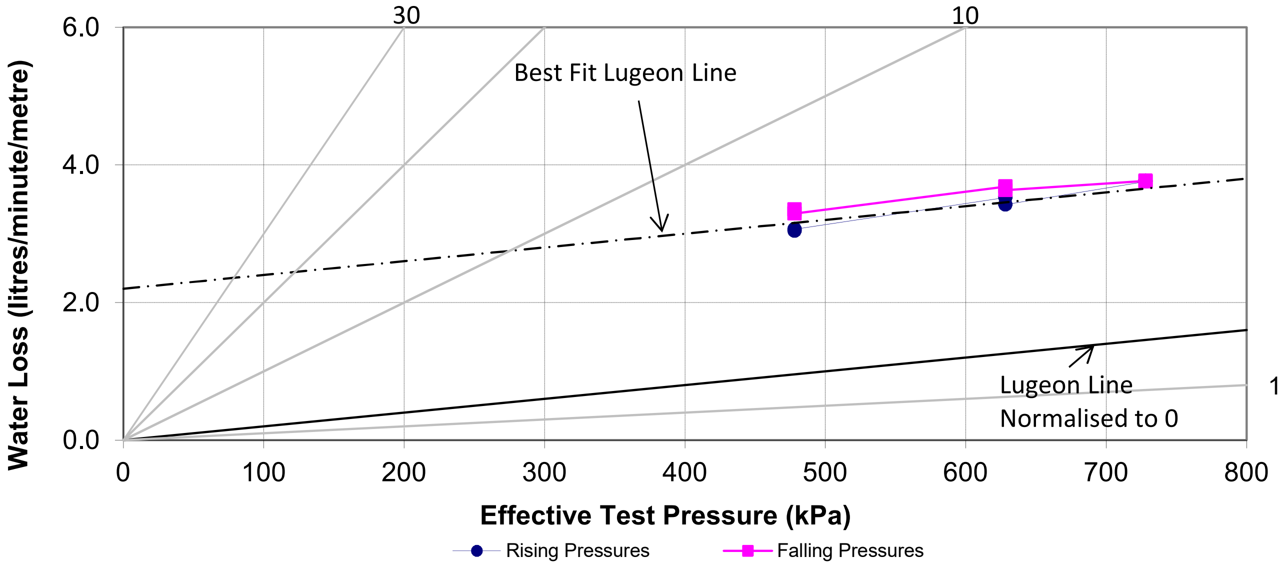
hole no: BHH149 | date: 20-Jan-17
project: Coffs Harbour Bypass
location: Gatelys Road | by: D Cooper

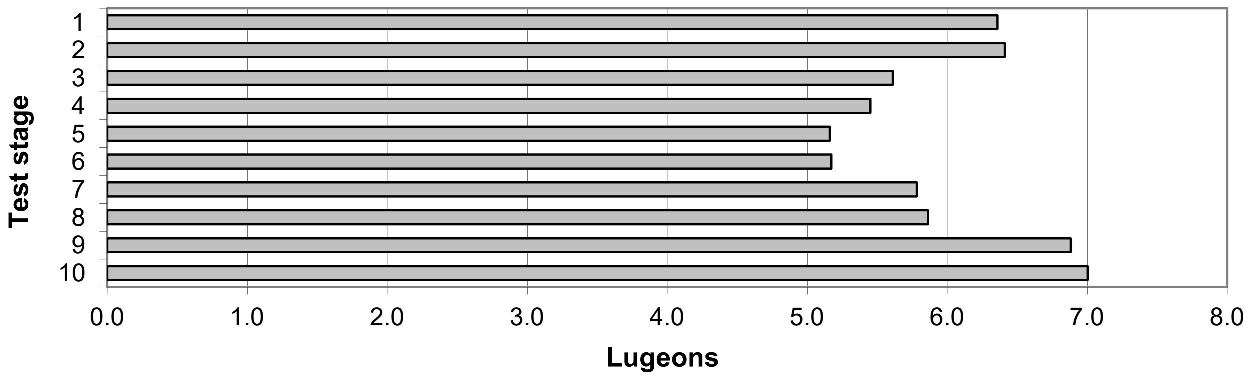
section tested: from 75.85 m to 87.85 m length 12.00 m

relative level at surface: 138.7 m RL | centre of test section (downhole): 81.9 m
gauge height: 1.5 m | test depth converted to vertical: 76.9 m
inclination of borehole: 70 ° | relative level at centre of test section: 61.8 m RL
inferred groundwater level: 106.7 m RL | correction to gauge pressure (1)(2): 328.3 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	478	182.5	3.0	24%	6.4
5	150	478	184.0	3.1	24%	6.4
5	300	628	211.5	3.5	31%	5.6
5	300	628	205.5	3.4	31%	5.5
5	400	728	225.5	3.8	36%	5.2
5	400	728	226.0	3.8	36%	5.2
5	300	628	218.0	3.6	31%	5.8
5	300	628	221.0	3.7	31%	5.9
5	150	478	197.5	3.3	24%	6.9
5	150	478	201.0	3.4	24%	7.0

400kPa maximum pressure achievable during test





Interpreted flow mechanism: Turbulent Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size: HQ3
Lithology: Argillite
Primary Defect Set: Joints

Lugeon (uL) Assessment

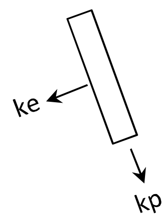
uL (Fell et al., 2005) - upper chart	2.0
uL (Houlsby, 1976) - lower chart	5.2

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	1.0.E-01	m/day
ke/kp = 100	9.1.E-02	m/day
ke/kp = 10	7.7.E-02	m/day
ke/kp = 1	6.4.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 6.8.E-02 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

BHH149 - 81.85m - 87.85m

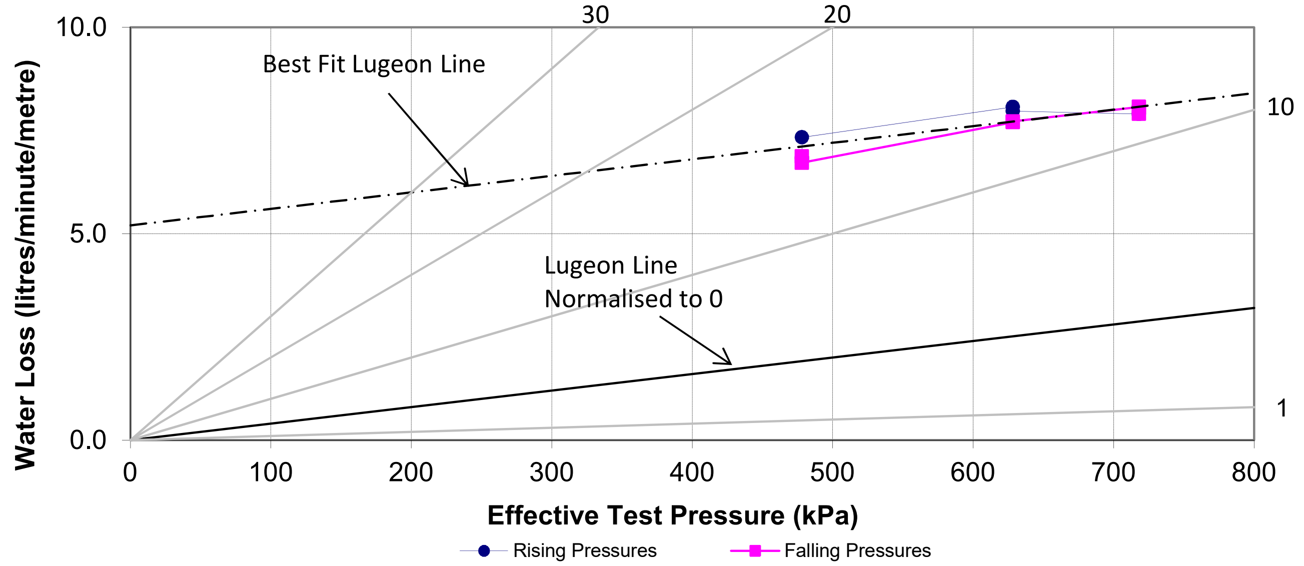
hole no:	BHH149	date:	20-Jan-17
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

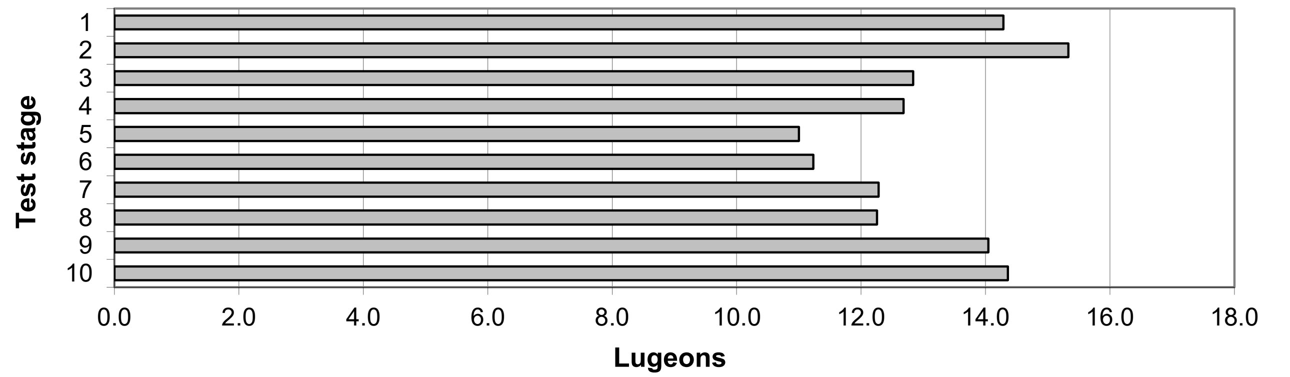
section tested: from 81.85 m to 87.85 m length 6.00 m

relative level at surface:	138.7 m RL	centre of test section (downhole):	84.9 m
gauge height:	1.5 m	test depth converted to vertical:	79.7 m
inclination of borehole:	70 °	relative level at centre of test section:	59.0 m RL
inferred groundwater level:	106.7 m RL	correction to gauge pressure (1)(2):	328.3 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	478	205.0	6.8	23%	14.3
5	150	478	220.0	7.3	23%	15.3
5	300	628	242.0	8.1	30%	12.8
5	300	628	239.0	8.0	30%	12.7
5	390	718	237.0	7.9	35%	11.0
5	390	718	242.0	8.1	35%	11.2
5	300	628	231.5	7.7	30%	12.3
5	300	628	231.0	7.7	30%	12.3
5	150	478	201.5	6.7	23%	14.0
5	150	478	206.0	6.9	23%	14.4

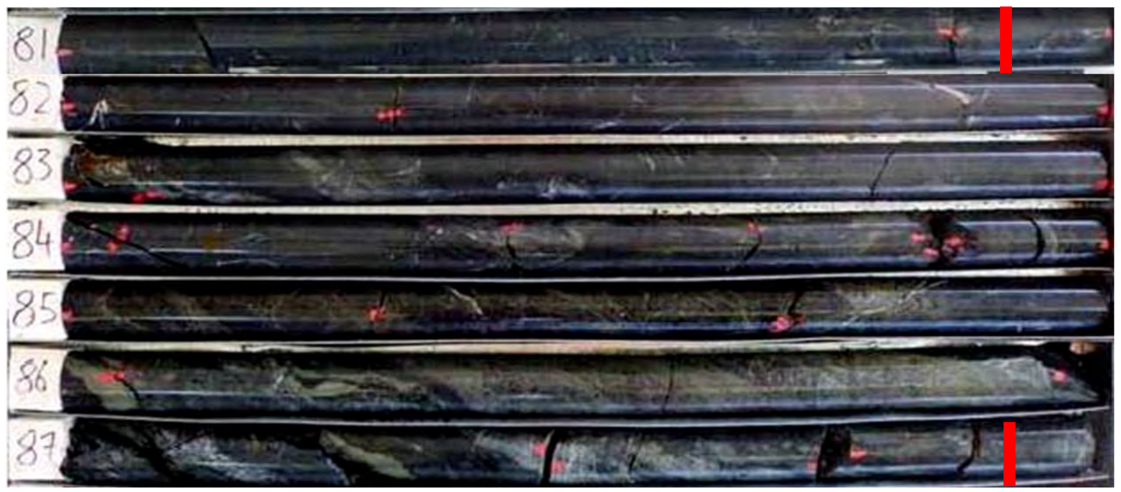
390kPa maximum pressure achievable during test





Interpreted flow mechanism: Turbulent Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

uL (Fell et al., 2005) - upper chart	4.0
uL (Houlsby, 1976) - lower chart	11.1

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:

ke/kp = 1000	2.1.E-01	m/day
ke/kp = 100	1.8.E-01	m/day
ke/kp = 10	1.5.E-01	m/day
ke/kp = 1	1.2.E-01	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 1.3.E-01 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

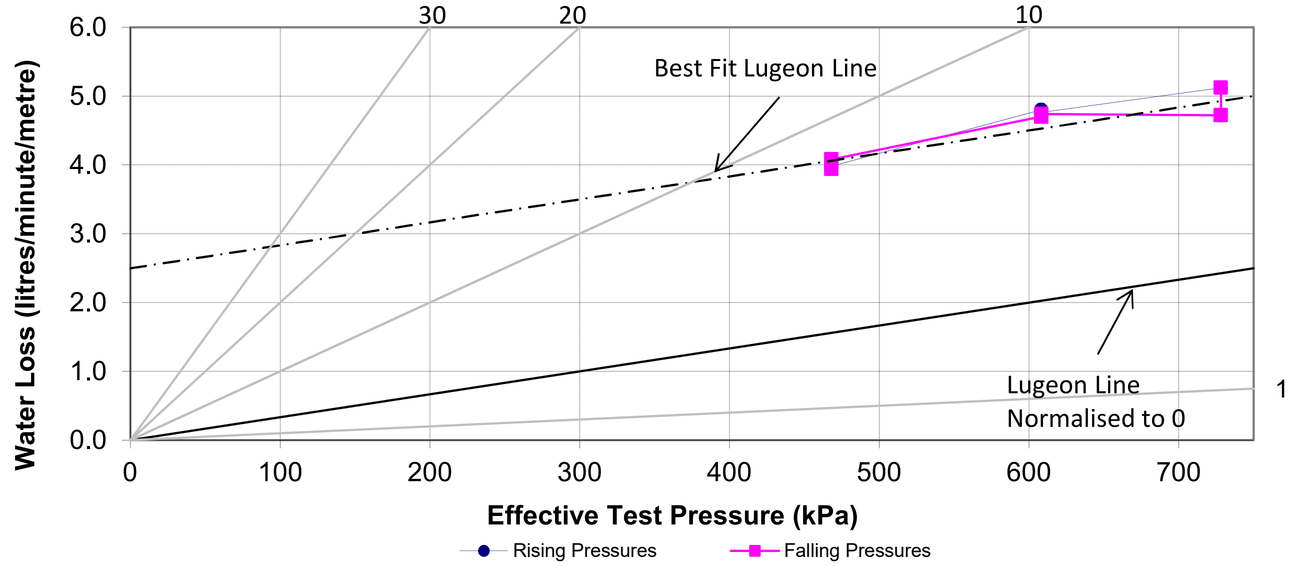
BHH149 - 88m - 98m

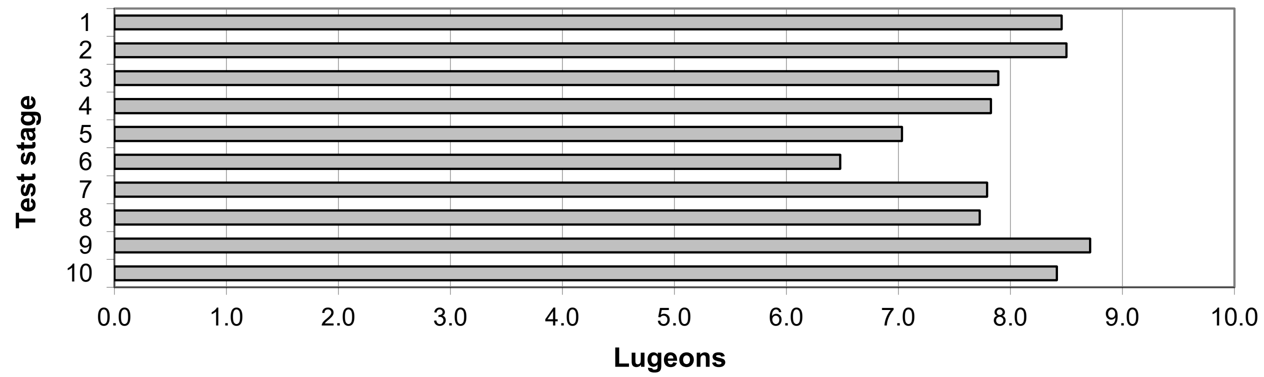
hole no: BHH149 | date: 23-Jan-17
project: Coffs Harbour Bypass
location: Gatelys Road | by: R Cater

section tested: from 88.00 m to 98.00 m length 10.00 m

relative level at surface: 138.7 m RL | centre of test section (downhole): 93.0 m
gauge height: 1.5 m | test depth converted to vertical: 87.4 m
inclination of borehole: 70 ° | relative level at centre of test section: 51.3 m RL
inferred groundwater level: 106.7 m RL | correction to gauge pressure (1)(2): 328.3 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	140	468	198.0	4.0	21%	8.5
5	140	468	199.0	4.0	21%	8.5
5	280	608	240.0	4.8	27%	7.9
5	280	608	238.0	4.8	27%	7.8
2.5	400	728	128.0	5.1	32%	7.0
2.5	400	728	118.0	4.7	32%	6.5
5	280	608	237.0	4.7	27%	7.8
5	280	608	235.0	4.7	27%	7.7
5	140	468	204.0	4.1	21%	8.7
5	140	468	197.0	3.9	21%	8.4





Interpreted flow mechanism: Turbulent Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size: HQ3
Lithology: Argillite
Primary Defect Set: Joints

Lugeon (uL) Assessment

uL (Fell et al., 2005) - upper chart 3.3

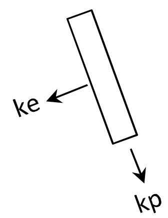
uL (Houlsby, 1976) - lower chart 6.8

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	1.3.E-01	m/day
ke/kp = 100	1.2.E-01	m/day
ke/kp = 10	9.8.E-02	m/day
ke/kp = 1	8.1.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 8.6.E-02 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink Job no. PSM2876

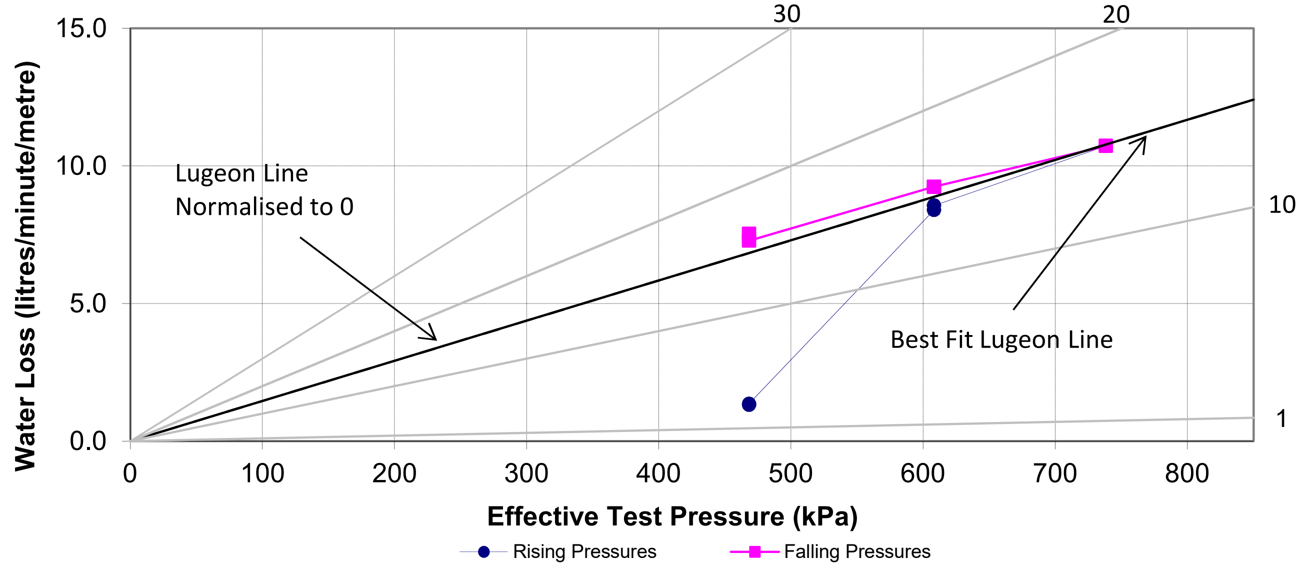
Borehole Water Pressure Test Summary **BHH149 - 93m - 98m**

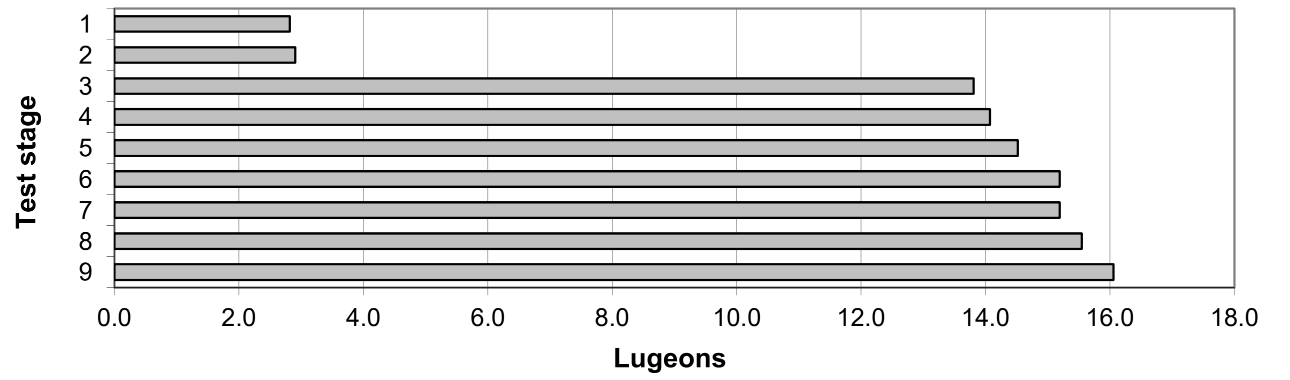
hole no:	BHH149	date:	19-Jan-17
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	R Cater

section tested: from 93.00 m to 98.00 m length 5.00 m

relative level at surface:	138.7 m RL	centre of test section (downhole):	95.5 m
gauge height:	1.5 m	test depth converted to vertical:	89.7 m
inclination of borehole:	70 °	relative level at centre of test section:	49.0 m RL
inferred groundwater level:	106.7 m RL	correction to gauge pressure (1)(2):	328.3 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	140	468	33.0	1.3	20%	2.8
5	140	468	34.0	1.4	20%	2.9
5	280	608	210.0	8.4	26%	13.8
5	280	608	214.0	8.6	26%	14.1
5	410	738	268.0	10.7	32%	14.5
5	280	608	231.0	9.2	26%	15.2
5	280	608	231.0	9.2	26%	15.2
5	140	468	182.0	7.3	20%	15.5
5	140	468	188.0	7.5	20%	16.1





Interpreted flow mechanism: Wash Out

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

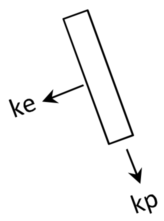
uL (Fell et al., 2005) - upper chart	14.6
uL (Houlsby, 1976) - lower chart	16.1

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	2.9.E-01	m/day
ke/kp = 100	2.5.E-01	m/day
ke/kp = 10	2.1.E-01	m/day
ke/kp = 1	1.7.E-01	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 1.8.E-01 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Test undertaken above water table

Job no. PSM2876

BHH150 - 31m - 42.85m

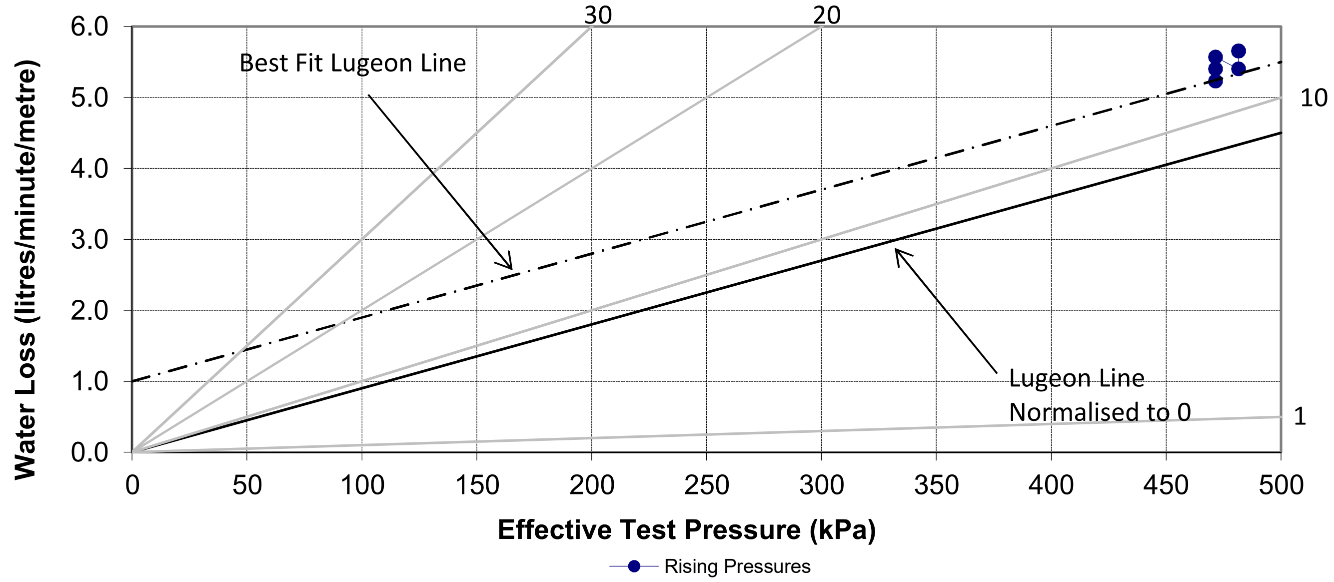
hole no:	BHH150	date:	21-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	M Kobler

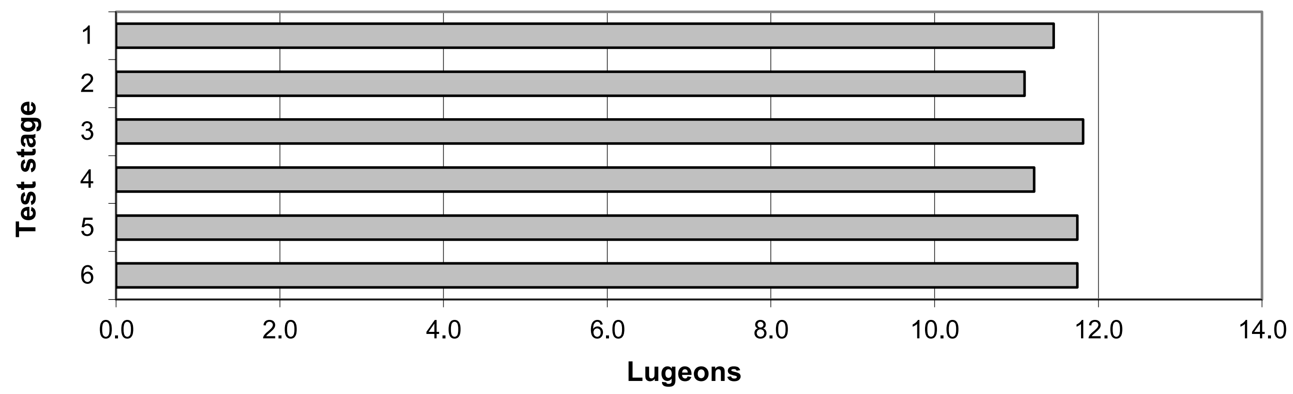
section tested: from 31.0 m to 42.9 m length 11.9 m

relative level at surface:	154.1 m RL	centre of test section (downhole):	36.9 m
gauge height:	2.0 m	test depth converted to vertical:	36.9 m
inclination of borehole:	90 °	relative level at centre of test section:	117.2 m RL
inferred groundwater level:	112.5 m RL	correction to gauge pressure (1)(2):	381.5 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
1	90	471	64.0	5.4	49%	11.5
1	90	471	62.0	5.2	49%	11.1
1	90	471	66.0	5.6	49%	11.8
1	100	481	64.0	5.4	50%	11.2
1	100	481	67.0	5.7	50%	11.7
1	100	481	67.0	5.7	50%	11.7

100kPa maximum pressure achievable during test





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

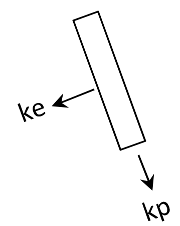
uL (Fell et al., 2005) - upper chart	9.0
uL (Houlsby, 1976) - lower chart	11.5

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	2.3.E-01	m/day
ke/kp = 100	2.0.E-01	m/day
ke/kp = 10	1.7.E-01	m/day
ke/kp = 1	1.4.E-01	m/day
ke value using Moye Method (1967) - equivalent to ke/kp = 1:	1.5.E-01	m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Test undertaken above water table

Job no. PSM2876

BHH150 - 37m - 42.85m

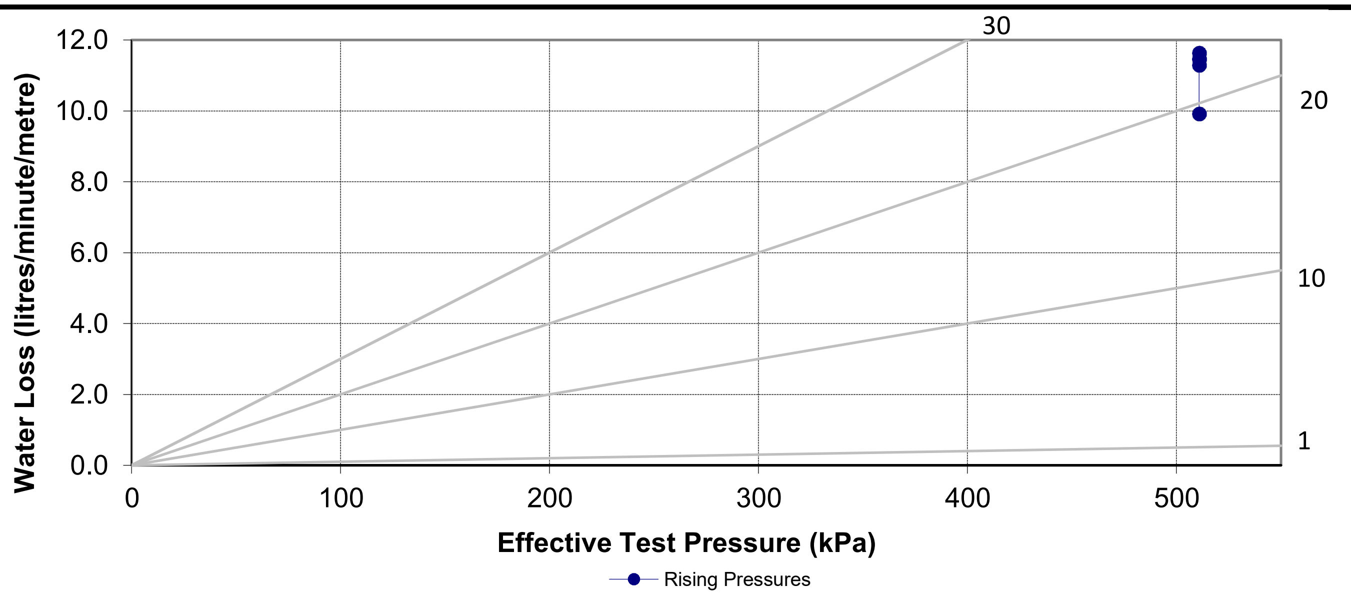
hole no:	BHH150	date:	21-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	M Kobler

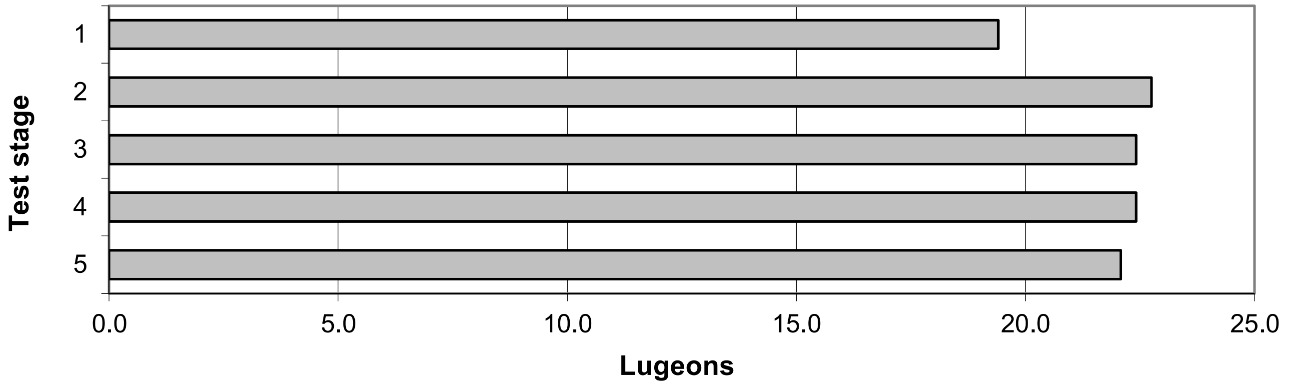
section tested: from 37.0 m to 42.9 m length 5.9 m

relative level at surface:	154.1 m RL	centre of test section (downhole):	39.9 m
gauge height:	2.0 m	test depth converted to vertical:	39.9 m
inclination of borehole:	90 °	relative level at centre of test section:	114.2 m RL
inferred groundwater level:	112.5 m RL	correction to gauge pressure (1)(2):	410.9 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
1	100	511	58.0	9.9	49%	19.4
1	100	511	68.0	11.6	49%	22.8
1	100	511	67.0	11.5	49%	22.4
1	100	511	67.0	11.5	49%	22.4
1	100	511	66.0	11.3	49%	22.1

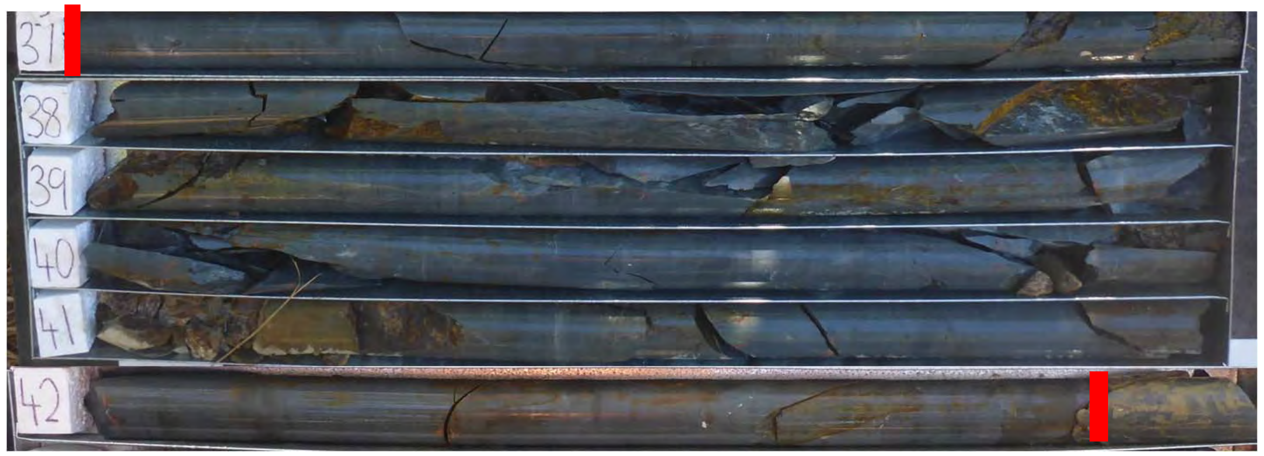
100kPa maximum pressure achievable during test





Interpreted flow mechanism: Laminar Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

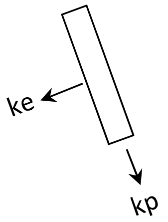
uL (Fell et al., 2005) - upper chart	n/a
uL (Houlsby, 1976) - lower chart	21.8

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	4.0.E-01	m/day
ke/kp = 100	3.5.E-01	m/day
ke/kp = 10	2.9.E-01	m/day
ke/kp = 1	2.4.E-01	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.5.E-01 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

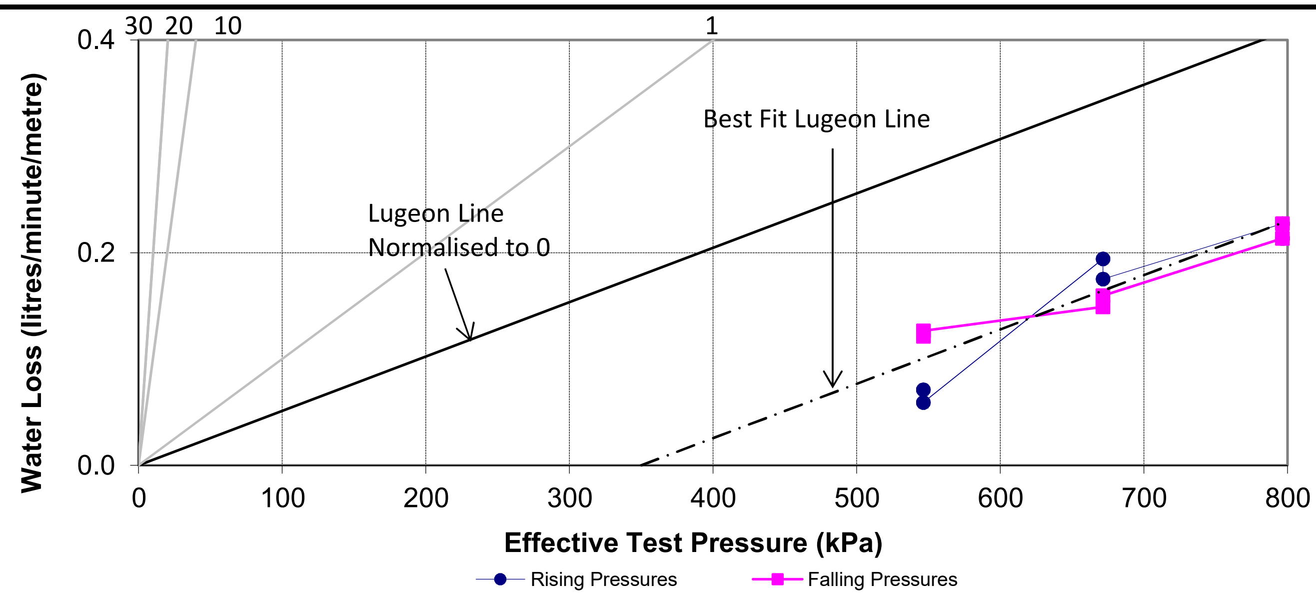
BHH150 - 42.5m - 54.03m

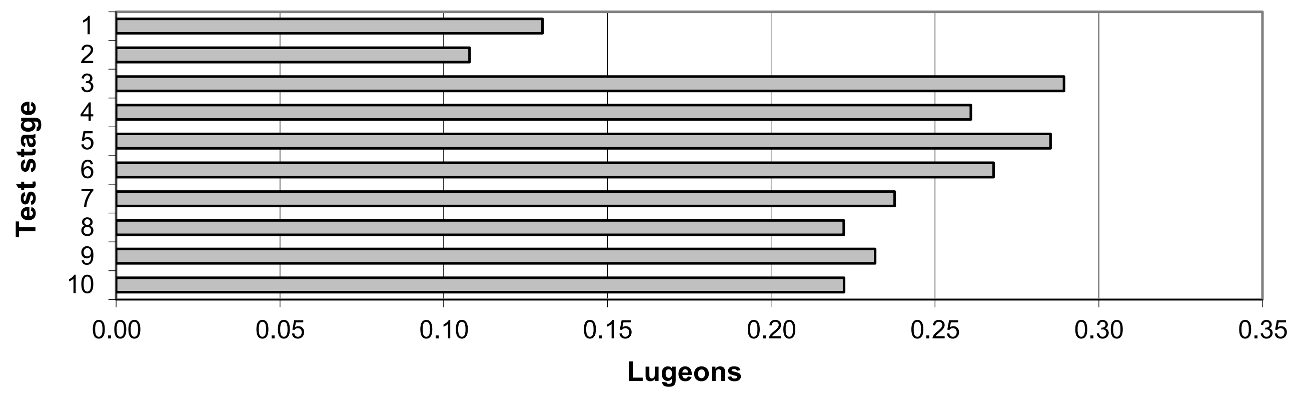
hole no:	BHH150	date:	21-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	M Kobler

section tested: from 42.5 m to 54.0 m length 11.5 m

relative level at surface:	154.1 m RL	centre of test section (downhole):	48.3 m
gauge height:	1.4 m	test depth converted to vertical:	48.3 m
inclination of borehole:	90 °	relative level at centre of test section:	105.8 m RL
inferred groundwater level:	112.5 m RL	correction to gauge pressure (1)(2):	421.4 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	125	546	4.1	0.1	44%	0.13
5	125	546	3.4	0.1	44%	0.11
5	250	671	11.2	0.2	54%	0.29
5	250	671	10.1	0.2	54%	0.26
5	375	796	13.1	0.2	63%	0.29
5	375	796	12.3	0.2	63%	0.27
5	250	671	9.2	0.2	54%	0.24
5	250	671	8.6	0.1	54%	0.22
5	125	546	7.3	0.1	44%	0.23
5	125	546	7.0	0.1	44%	0.22





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

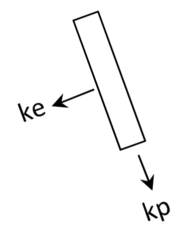
uL (Fell et al., 2005) - upper chart	0.5
uL (Houlsby, 1976) - lower chart	0.2

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	4.5.E-03	m/day
ke/kp = 100	3.9.E-03	m/day
ke/kp = 10	3.4.E-03	m/day
ke/kp = 1	2.8.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.9.E-03 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

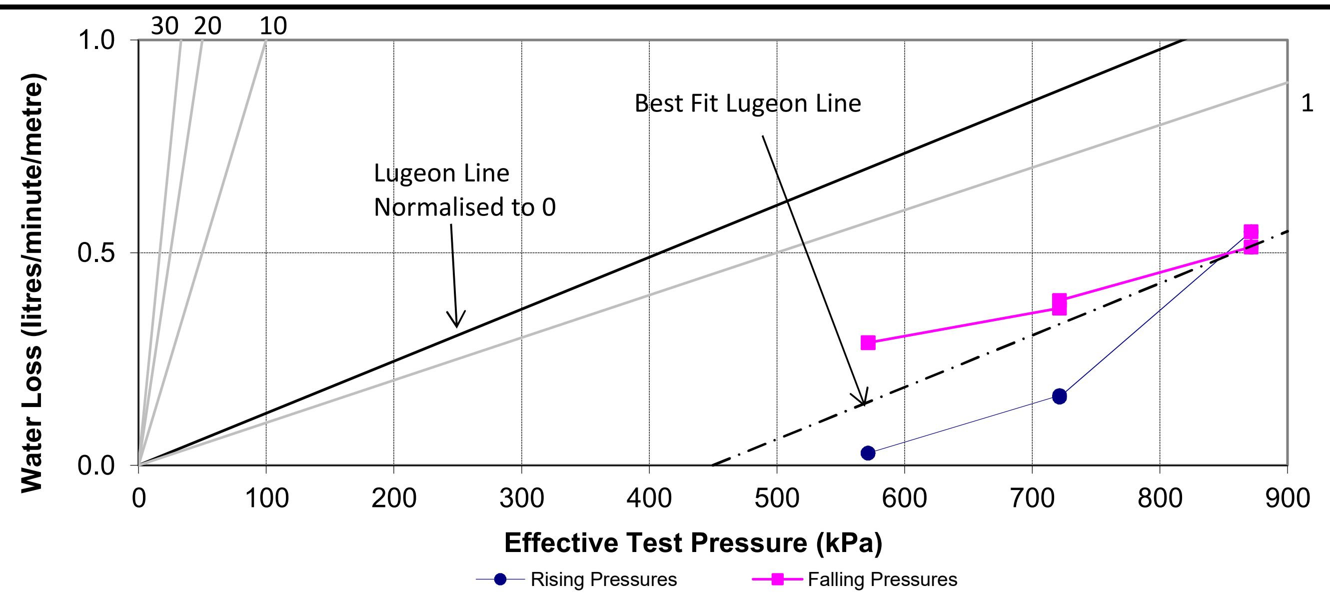
BHH150 - 55.13m - 66.23m

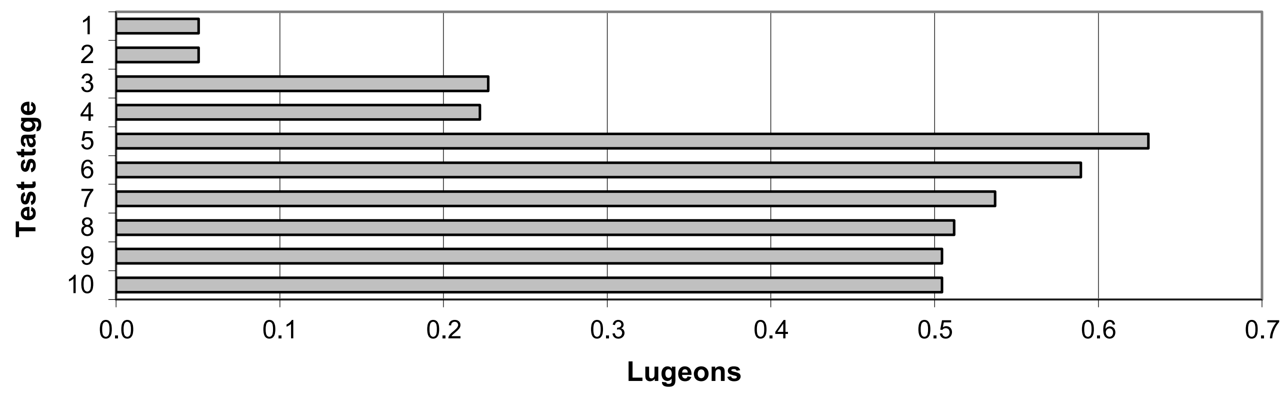
hole no:	BHH150	date:	22-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	M Kobler

section tested: from 55.1 m to 66.2 m length 11.1 m

relative level at surface:	154.1 m RL	centre of test section (downhole):	60.7 m
gauge height:	1.4 m	test depth converted to vertical:	60.7 m
inclination of borehole:	90 °	relative level at centre of test section:	93.4 m RL
inferred groundwater level:	112.5 m RL	correction to gauge pressure (1)(2):	421.4 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	571	1.6	0.0	36%	0.1
5	150	571	1.6	0.0	36%	0.1
5	300	721	9.1	0.2	46%	0.2
5	300	721	8.9	0.2	46%	0.2
5	450	871	30.5	0.5	55%	0.6
5	450	871	28.5	0.5	55%	0.6
5	300	721	21.5	0.4	46%	0.5
5	300	721	20.5	0.4	46%	0.5
5	150	571	16.0	0.3	36%	0.5
5	150	571	16.0	0.3	36%	0.5





Interpreted flow mechanism: Wash Out

Image of packer test section:

LEGEND:

Packer zone boundary

Core size: HQ3

Lithology: Argillite

Primary Defect Set: Joints

Lugeon (uL) Assessment

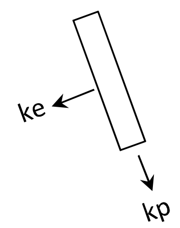
uL (Fell et al., 2005) - upper chart	1.2
uL (Houlsby, 1976) - lower chart	0.6

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	1.3.E-02	m/day
ke/kp = 100	1.1.E-02	m/day
ke/kp = 10	9.3.E-03	m/day
ke/kp = 1	7.7.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 8.1.E-03 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

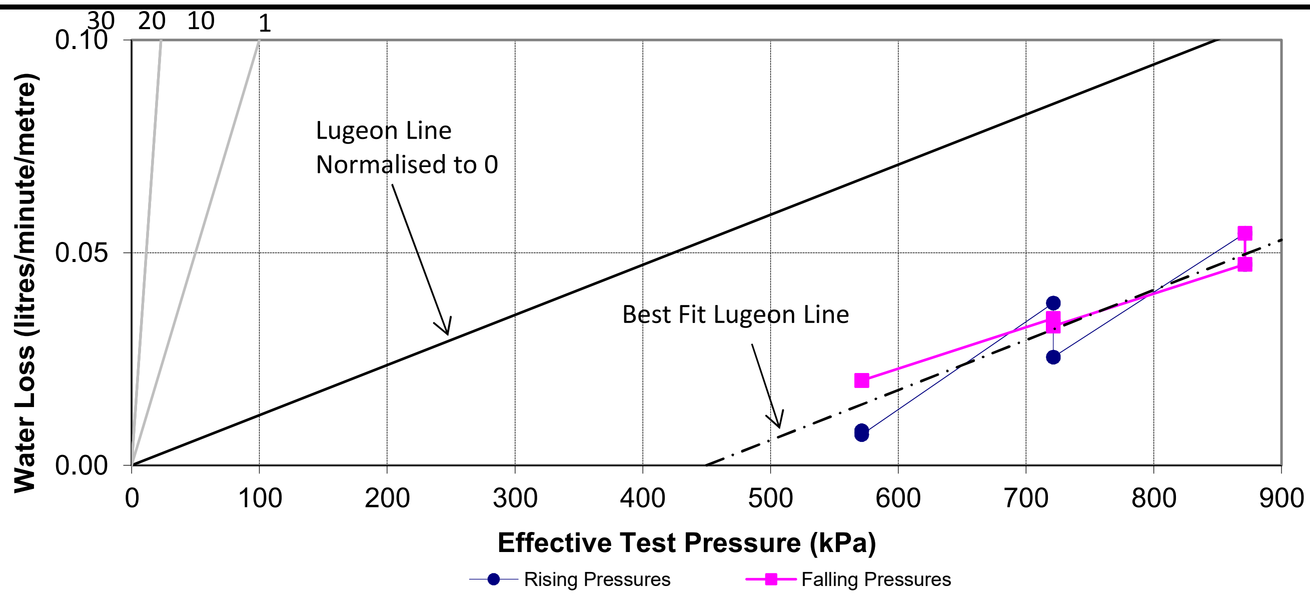
BHH150 - 67.6m - 78.6m

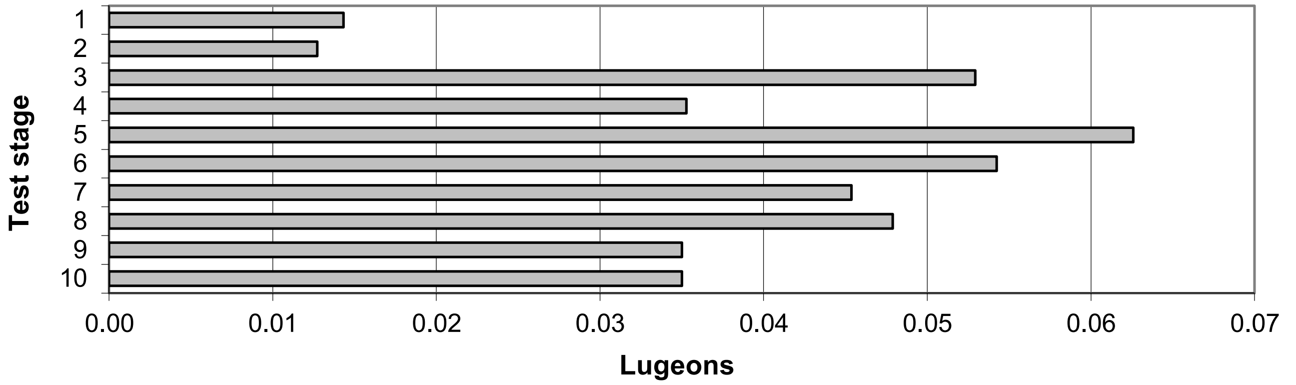
hole no:	BHH150	date:	23-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

section tested: from 67.6 m to 78.6 m length 11.0 m

relative level at surface:	154.1 m RL	centre of test section (downhole):	73.1 m
gauge height:	1.4 m	test depth converted to vertical:	73.1 m
inclination of borehole:	90 °	relative level at centre of test section:	81.0 m RL
inferred groundwater level:	112.5 m RL	correction to gauge pressure (1)(2):	421.4 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	571	0.5	0.0	30%	0.01
5	150	571	0.4	0.0	30%	0.01
5	300	721	2.1	0.0	38%	0.05
5	300	721	1.4	0.0	38%	0.04
5	450	871	3.0	0.1	46%	0.06
5	450	871	2.6	0.0	46%	0.05
5	300	721	1.8	0.0	38%	0.05
5	300	721	1.9	0.0	38%	0.05
5	150	571	1.1	0.0	30%	0.04
5	150	571	1.1	0.0	30%	0.04





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size: HQ3

Lithology: Argillite

Primary Defect Set: Joints

Lugeon (uL) Assessment

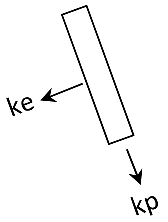
uL (Fell et al., 2005) - upper chart	0.12
uL (Houlsby, 1976) - lower chart	0.04

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	7.9.E-04	m/day
ke/kp = 100	6.9.E-04	m/day
ke/kp = 10	5.8.E-04	m/day
ke/kp = 1	4.8.E-04	m/day
ke value using Moye Method (1967) - equivalent to ke/kp = 1:	5.1.E-04	m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

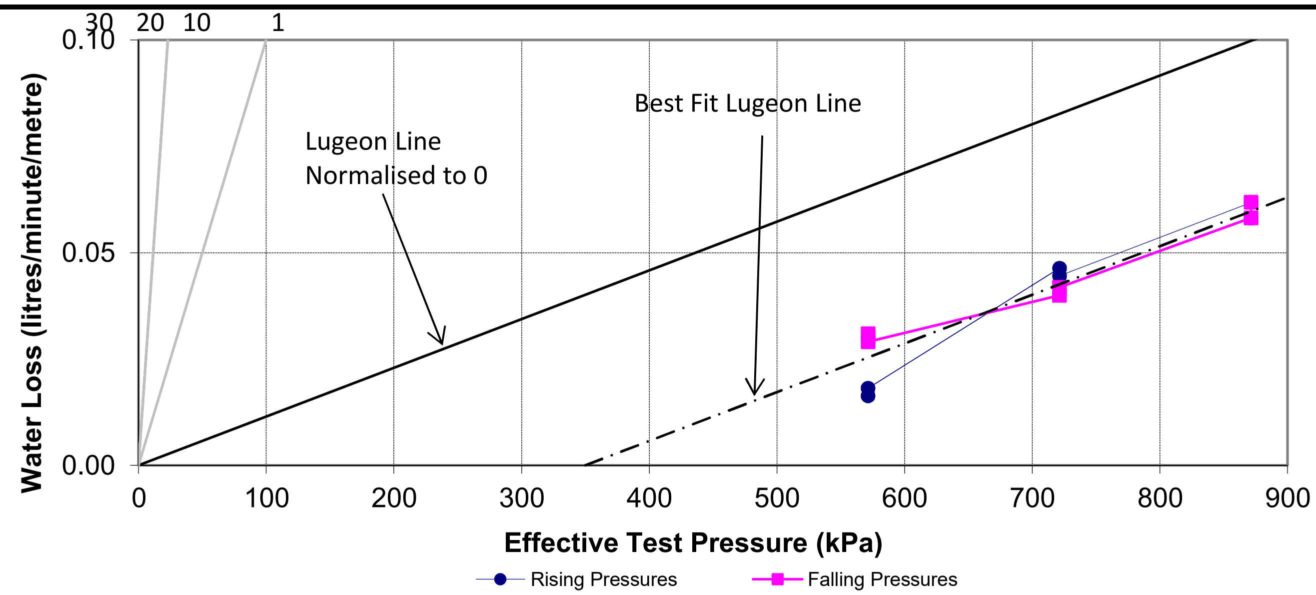
BHH150 - 80.27m - 91.27m

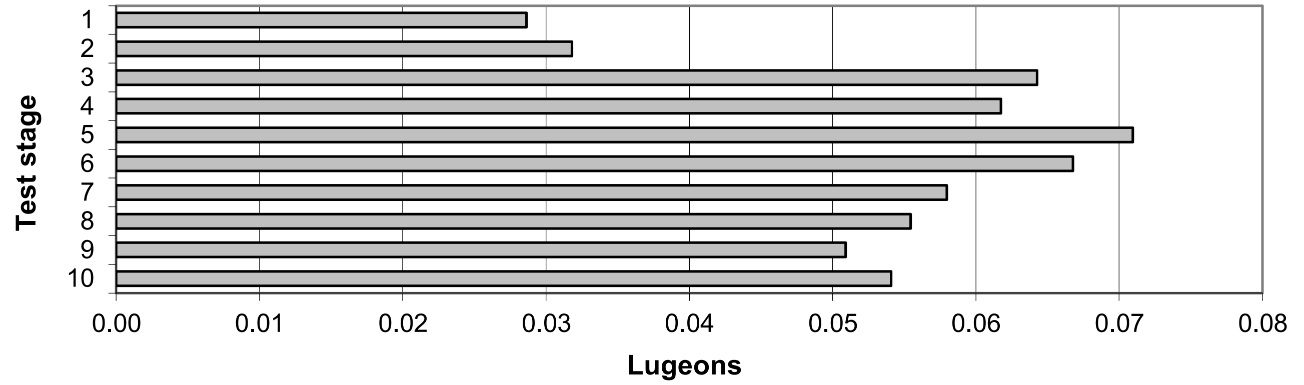
hole no:	BHH150	date:	23-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

section tested: from 80.3 m to 91.3 m length 11.0 m

relative level at surface:	154.1 m RL	centre of test section (downhole):	85.8 m
gauge height:	1.4 m	test depth converted to vertical:	85.8 m
inclination of borehole:	90 °	relative level at centre of test section:	68.3 m RL
inferred groundwater level:	112.5 m RL	correction to gauge pressure (1)(2):	421.4 kPa

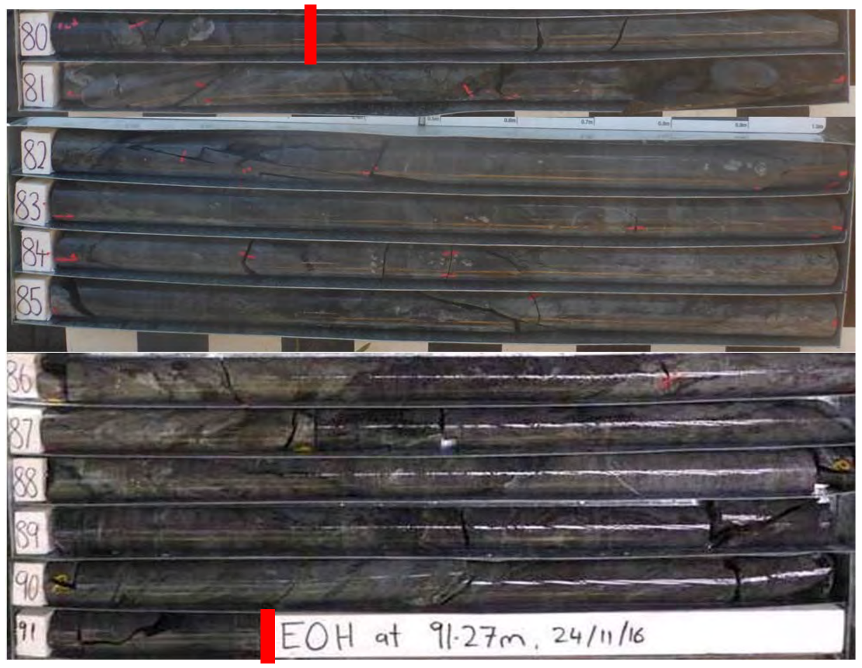
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	571	0.9	0.0	26%	0.03
5	150	571	1.0	0.0	26%	0.03
5	300	721	2.6	0.0	32%	0.06
5	300	721	2.4	0.0	32%	0.06
5	450	871	3.4	0.1	39%	0.07
5	450	871	3.2	0.1	39%	0.07
5	300	721	2.3	0.0	32%	0.06
5	300	721	2.2	0.0	32%	0.06
5	150	571	1.6	0.0	26%	0.05
5	150	571	1.7	0.0	26%	0.05





Interpreted flow mechanism: Laminar Flow

Image of packer test section:



Lugeon (uL) Assessment

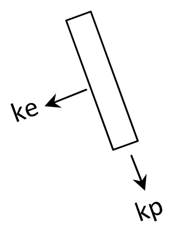
uL (Fell et al., 2005) - upper chart	0.11
uL (Houlsby, 1976) - lower chart	0.05

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	1.1.E-03	m/day
ke/kp = 100	9.4.E-04	m/day
ke/kp = 10	8.0.E-04	m/day
ke/kp = 1	6.6.E-04	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 7.0.E-04 m/day

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Test undertaken above water table

Job no. PSM2876

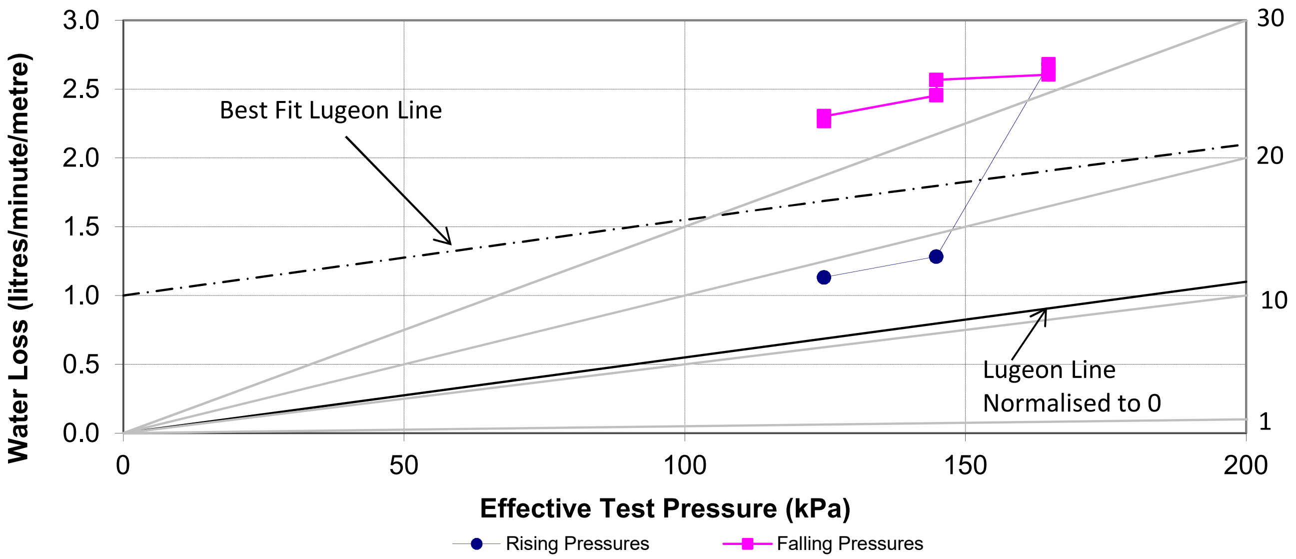
BHH151 - 7.55m - 12.85m

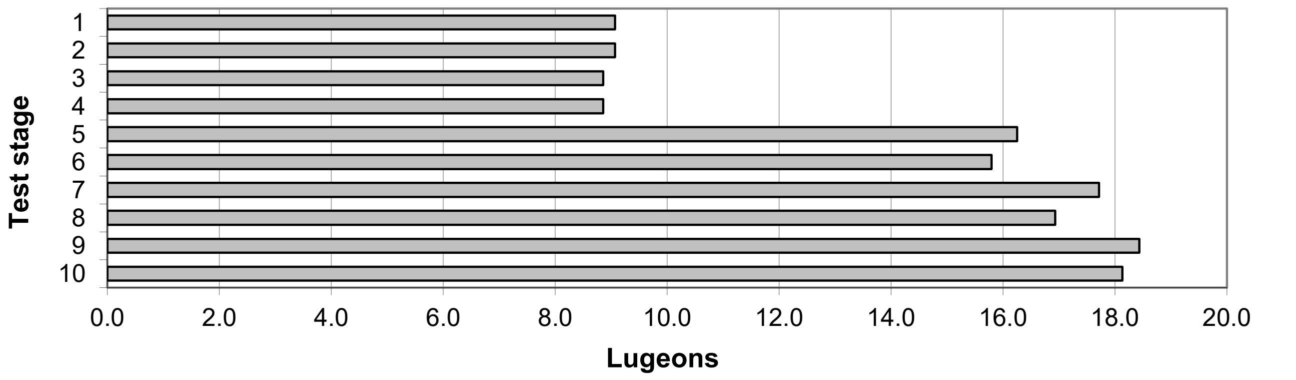
hole no:	BHH151	date:	3-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	M Kobler

section tested: from 7.55 m to 12.85 m length 5.30 m

relative level at surface:	150.5 m RL	centre of test section (downhole):	10.2 m
gauge height:	0.5 m	test depth converted to vertical:	10.2 m
inclination of borehole:	90 °	relative level at centre of test section:	140.3 m RL
inferred groundwater level:	113.0 m RL	correction to gauge pressure (1)(2):	104.9 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	20	125	30.0	1.1	47%	9.1
5	20	125	30.0	1.1	47%	9.1
5	40	145	34.0	1.3	55%	8.9
5	40	145	34.0	1.3	55%	8.9
5	60	165	71.0	2.7	62%	16.3
5	60	165	69.0	2.6	62%	15.8
5	40	145	68.0	2.6	55%	17.7
5	40	145	65.0	2.5	55%	16.9
5	20	125	61.0	2.3	47%	18.4
5	20	125	60.0	2.3	47%	18.1





Interpreted flow mechanism: Wash Out

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	No core / Crush Zones

Lugeon (uL) Assessment

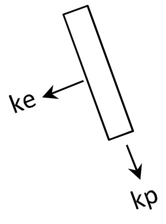
uL (Fell et al., 2005) - upper chart	5.5
uL (Houlsby, 1976) - lower chart	18.4

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	3.4.E-01	m/day
ke/kp = 100	2.9.E-01	m/day
ke/kp = 10	2.4.E-01	m/day
ke/kp = 1	1.9.E-01	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.1.E-01 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Test undertaken above water table

Job no. PSM2876

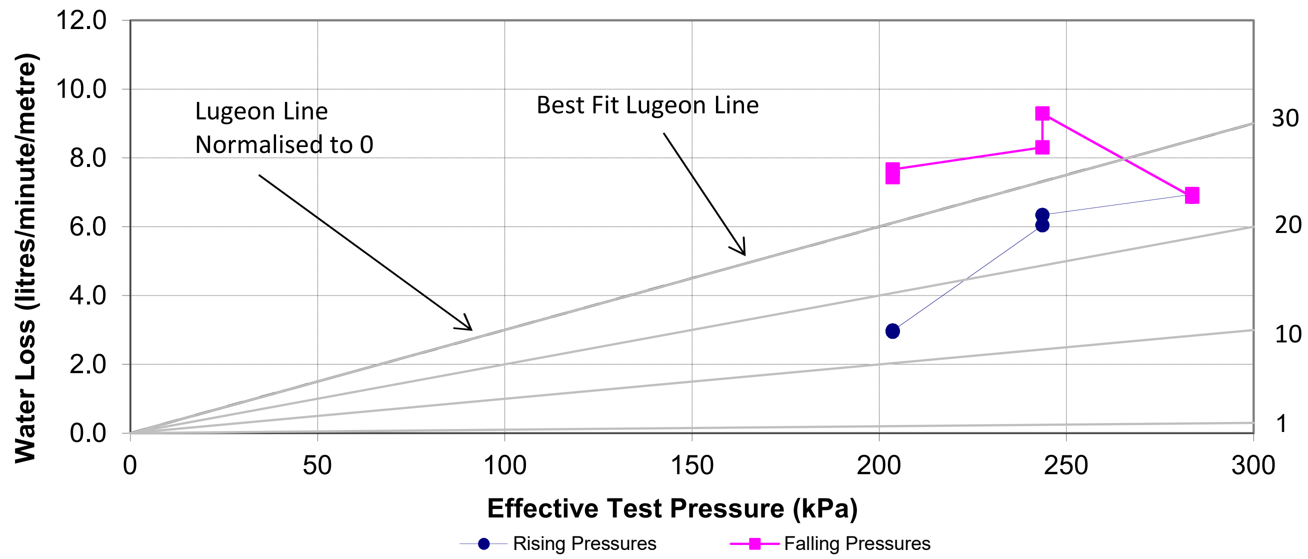
BHH151 - 13.55m - 18.85m

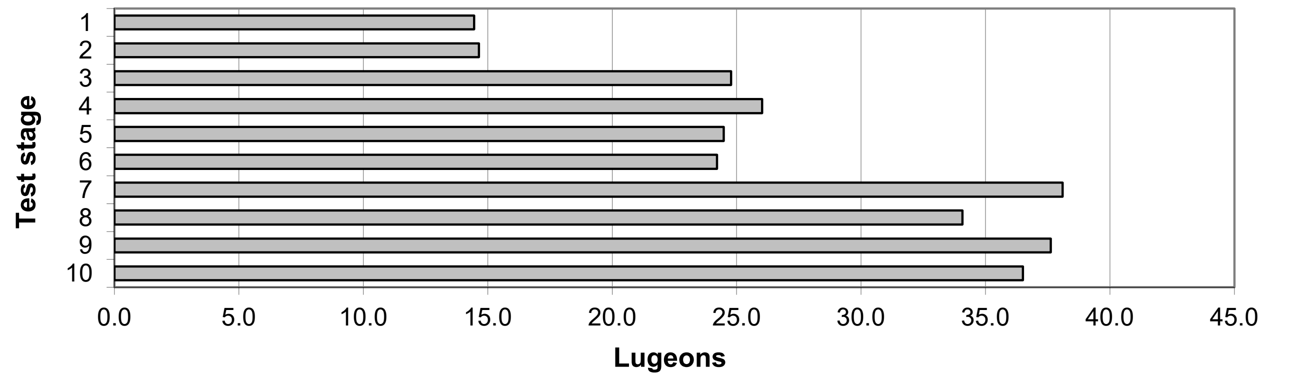
hole no:	BHH151	date:	4-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	M Kobler

section tested: from 13.55 m to 18.85 m length 5.30 m

relative level at surface:	150.5 m RL	centre of test section (downhole):	16.2 m
gauge height:	0.5 m	test depth converted to vertical:	16.2 m
inclination of borehole:	90 °	relative level at centre of test section:	134.3 m RL
inferred groundwater level:	113.0 m RL	correction to gauge pressure (1)(2):	163.7 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	40	204	78.0	2.9	48%	14.5
5	40	204	79.0	3.0	48%	14.6
5	80	244	160.0	6.0	58%	24.8
5	80	244	168.0	6.3	58%	26.0
5	120	284	184.0	6.9	67%	24.5
5	120	284	182.0	6.9	67%	24.2
5	80	244	246.0	9.3	58%	38.1
5	80	244	220.0	8.3	58%	34.1
5	40	204	203.0	7.7	48%	37.6
5	40	204	197.0	7.4	48%	36.5





Interpreted flow mechanism: Wash Out

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Shear Zone / Joints

Lugeon (uL) Assessment

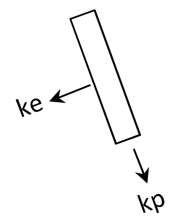
uL (Fell et al., 2005) - upper chart	30.0
uL (Houlsby, 1976) - lower chart	38.1

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	7.0.E-01	m/day
ke/kp = 100	6.0.E-01	m/day
ke/kp = 10	5.0.E-01	m/day
ke/kp = 1	4.0.E-01	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 4.3.E-01 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Test undertaken above water table

Job no. PSM2876

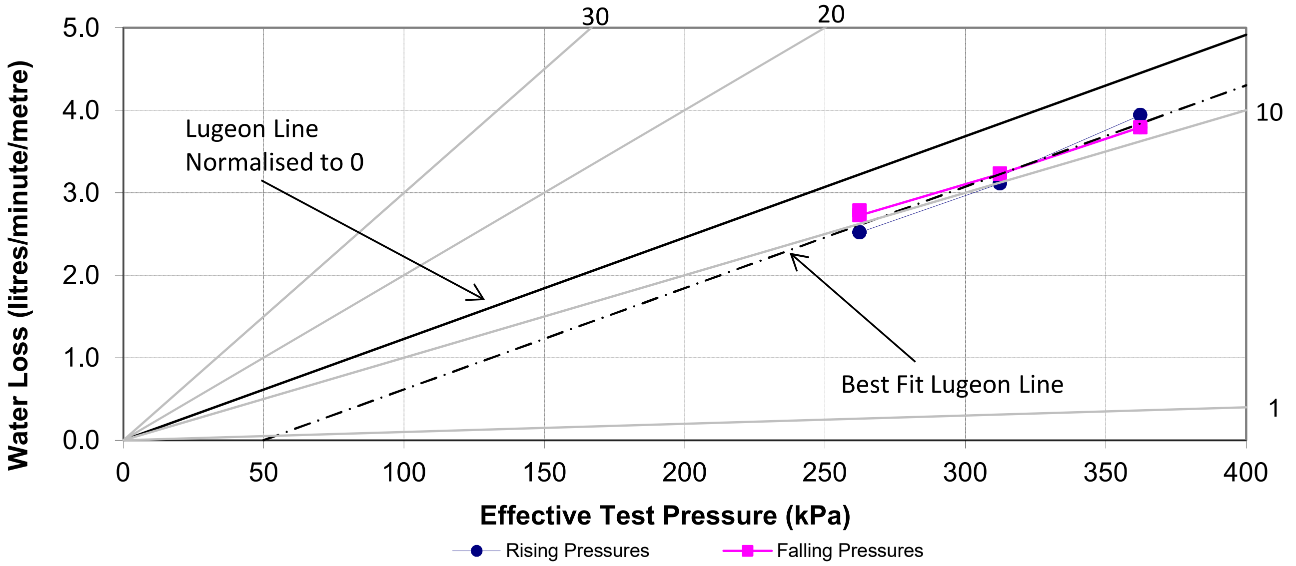
BHH151 - 18.1m - 24.85m

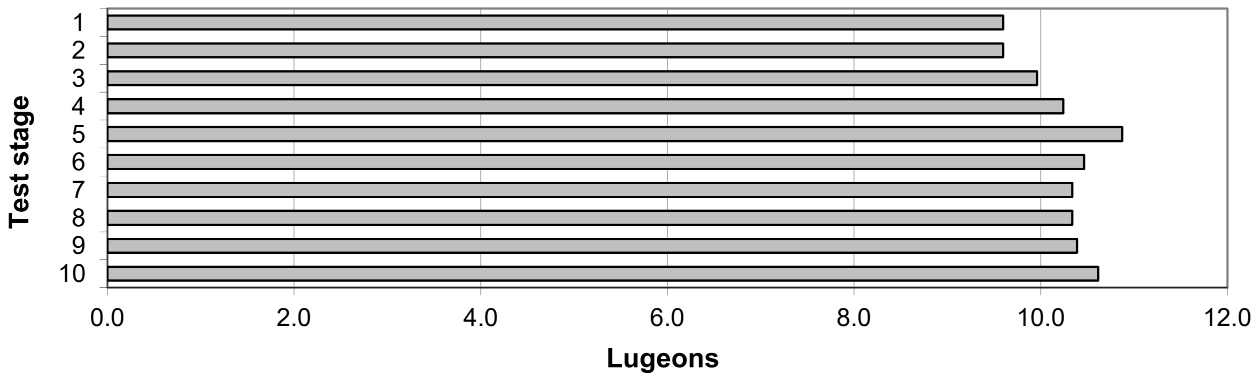
hole no:	BHH151	date:	4-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	M Kobler

section tested: from 18.10 m to 24.85 m length 6.75 m

relative level at surface:	150.5 m RL	centre of test section (downhole):	21.5 m
gauge height:	0.2 m	test depth converted to vertical:	21.5 m
inclination of borehole:	90 °	relative level at centre of test section:	129.0 m RL
inferred groundwater level:	113.0 m RL	correction to gauge pressure (1)(2):	212.4 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	50	262	85.0	2.5	47%	9.6
5	50	262	85.0	2.5	47%	9.6
5	100	312	105.0	3.1	56%	10.0
5	100	312	108.0	3.2	56%	10.2
5	150	362	133.0	3.9	65%	10.9
5	150	362	128.0	3.8	65%	10.5
5	100	312	109.0	3.2	56%	10.3
5	100	312	109.0	3.2	56%	10.3
5	50	262	92.0	2.7	47%	10.4
5	50	262	94.0	2.8	47%	10.6





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	No Core / Joints

Lugeon (uL) Assessment

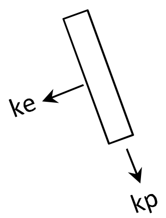
uL (Fell et al., 2005) - upper chart	12.3
uL (Houlsby, 1976) - lower chart	10.2

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	1.9.E-01	m/day
ke/kp = 100	1.7.E-01	m/day
ke/kp = 10	1.4.E-01	m/day
ke/kp = 1	1.1.E-01	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 1.2.E-01 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Test undertaken above water table

Job no. PSM2876

BHH151 - 25.35m - 30.65m

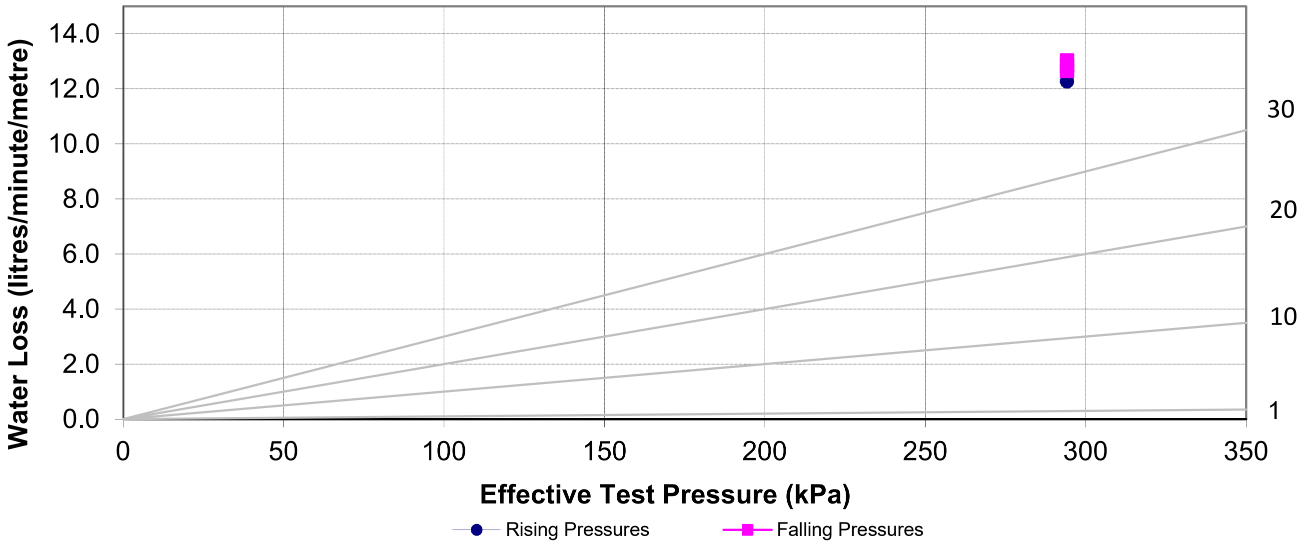
hole no:	BHH151	date:	8-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	M Kobler

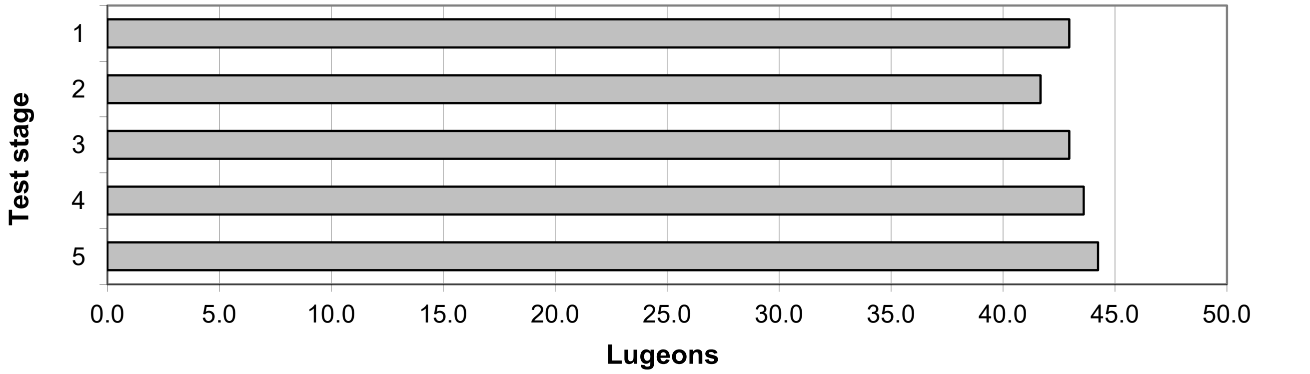
section tested: from 25.35 m to 30.65 m length 5.30 m

relative level at surface:	150.5 m RL	centre of test section (downhole):	28.0 m
gauge height:	0.5 m	test depth converted to vertical:	28.0 m
inclination of borehole:	90 °	relative level at centre of test section:	122.5 m RL
inferred groundwater level:	113.0 m RL	correction to gauge pressure (1)(2):	279.3 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
1	15	294	67.0	12.6	40%	43.0
1	15	294	65.0	12.3	40%	41.7
1	15	294	67.0	12.6	40%	43.0
1	15	294	68.0	12.8	40%	43.6
1	15	294	69.0	13.0	40%	44.2

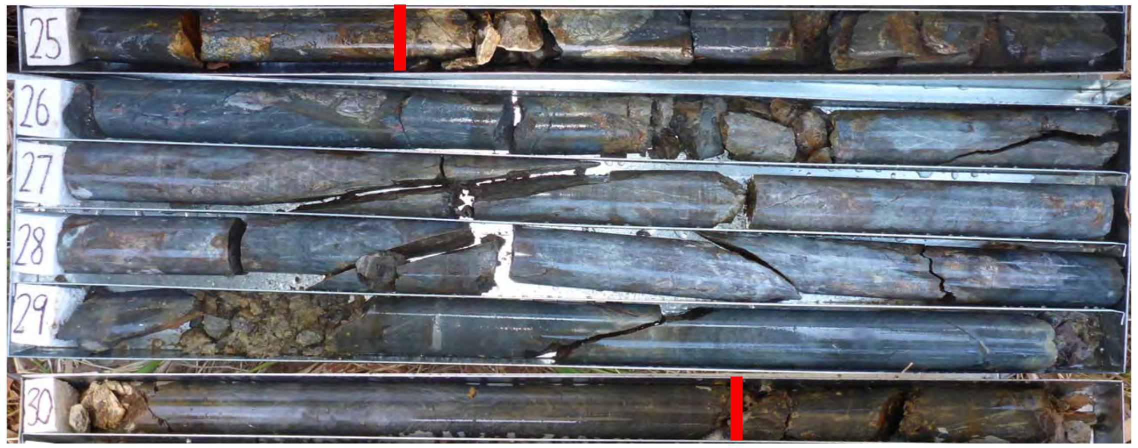
15kPa maximum pressure achievable during test





Interpreted flow mechanism: Laminar Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

uL (Fell et al., 2005) - upper chart	n/a
uL (Houlsby, 1976) - lower chart	43.1

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:

ke kp

ke/kp = 1000	7.9.E-01	m/day
ke/kp = 100	6.8.E-01	m/day
ke/kp = 10	5.7.E-01	m/day
ke/kp = 1	4.6.E-01	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 4.8.E-01 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Test undertaken above water table

Job no. PSM2876

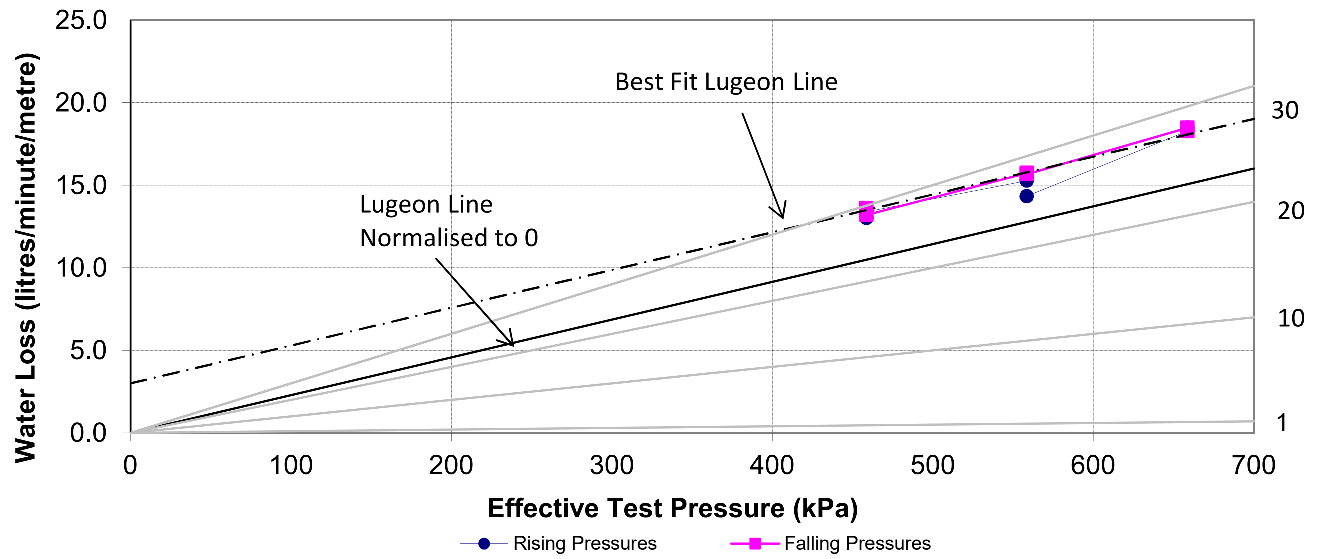
BHH151 - 33.96m - 36.96m

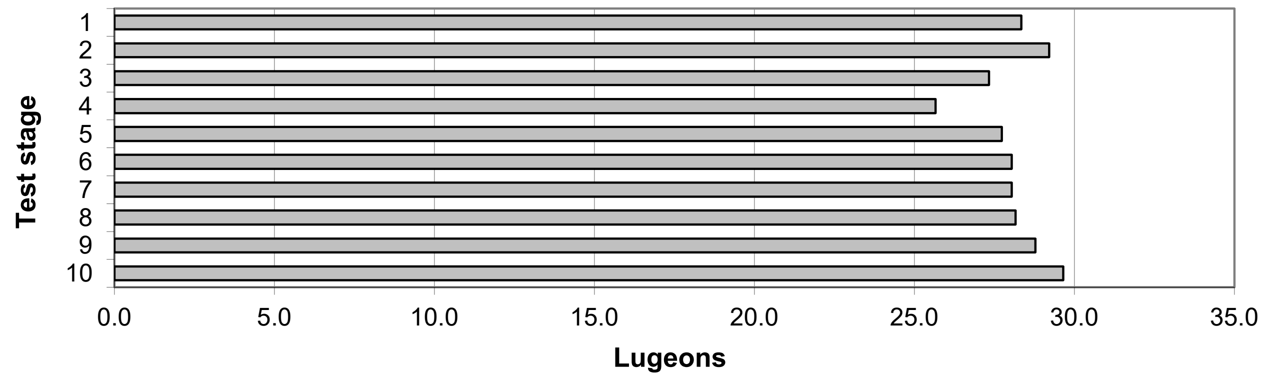
hole no:	BHH151	date:	8-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	M Kobler

section tested: from 33.96 m to 36.96 m length 3.00 m

relative level at surface:	150.5 m RL	centre of test section (downhole):	35.5 m
gauge height:	1.1 m	test depth converted to vertical:	35.5 m
inclination of borehole:	90 °	relative level at centre of test section:	115.1 m RL
inferred groundwater level:	113.0 m RL	correction to gauge pressure (1)(2):	358.7 kPa

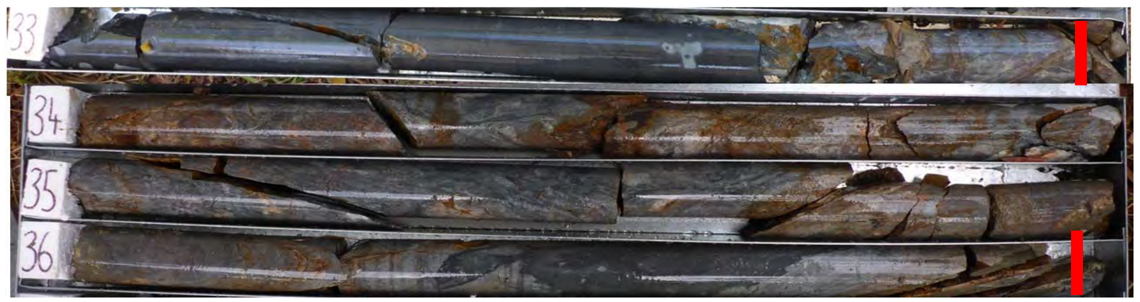
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	100	459	195.0	13.0	50%	28.3
5	100	459	201.0	13.4	50%	29.2
5	200	559	229.0	15.3	61%	27.3
5	200	559	215.0	14.3	61%	25.7
5	300	659	274.0	18.3	71%	27.7
5	300	659	277.0	18.5	71%	28.0
5	200	559	235.0	15.7	61%	28.0
5	200	559	236.0	15.7	61%	28.2
5	100	459	198.0	13.2	50%	28.8
5	100	459	204.0	13.6	50%	29.7





Interpreted flow mechanism: Laminar Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

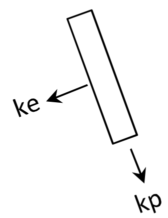
uL (Fell et al., 2005) - upper chart	22.9
uL (Houlsby, 1976) - lower chart	28.1

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	4.8.E-01	m/day
ke/kp = 100	4.1.E-01	m/day
ke/kp = 10	3.3.E-01	m/day
ke/kp = 1	2.6.E-01	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.8.E-01 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

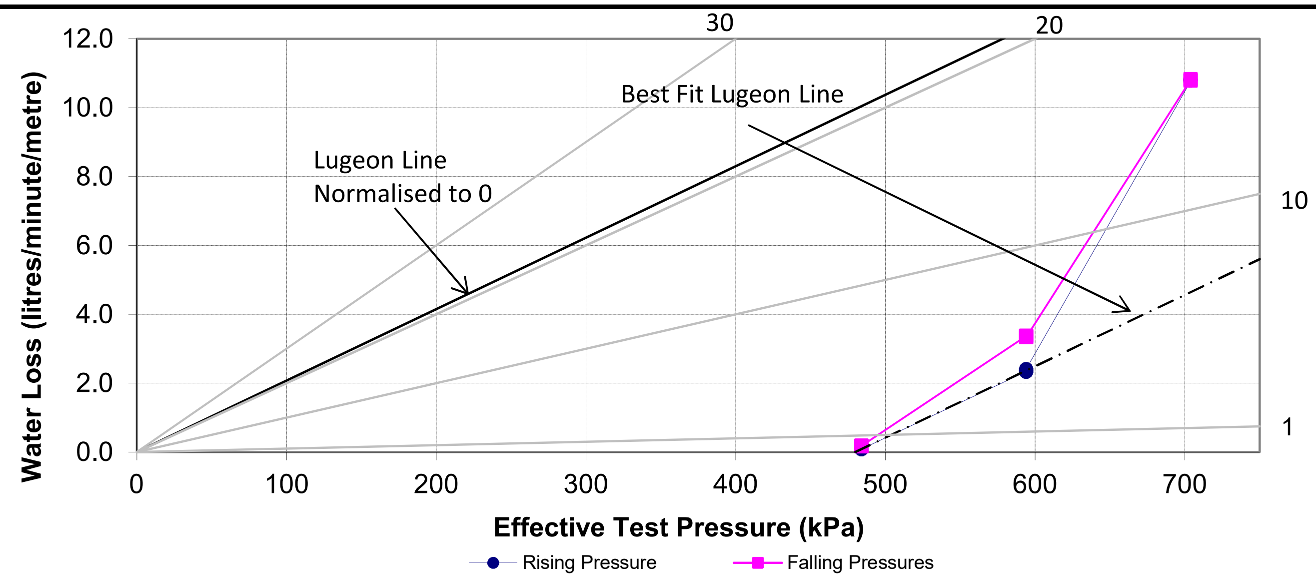
BHH151 - 37.4m - 42.8m

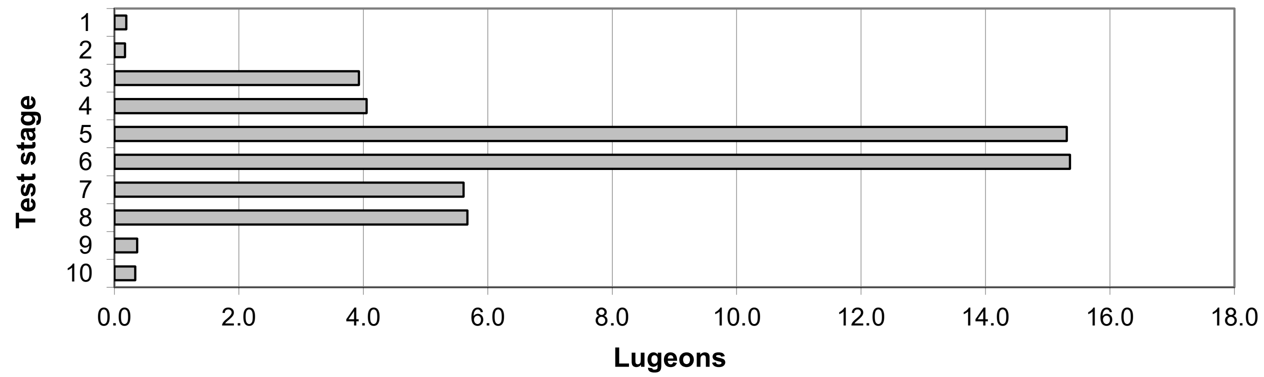
hole no:	BHH151	date:	9-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	M Kobler

section tested: from 37.40 m to 42.80 m length 5.40 m

relative level at surface:	150.5 m RL	centre of test section (downhole):	40.1 m
gauge height:	0.7 m	test depth converted to vertical:	40.1 m
inclination of borehole:	90 °	relative level at centre of test section:	110.4 m RL
inferred groundwater level:	113.0 m RL	correction to gauge pressure (1)(2):	374.2 kPa

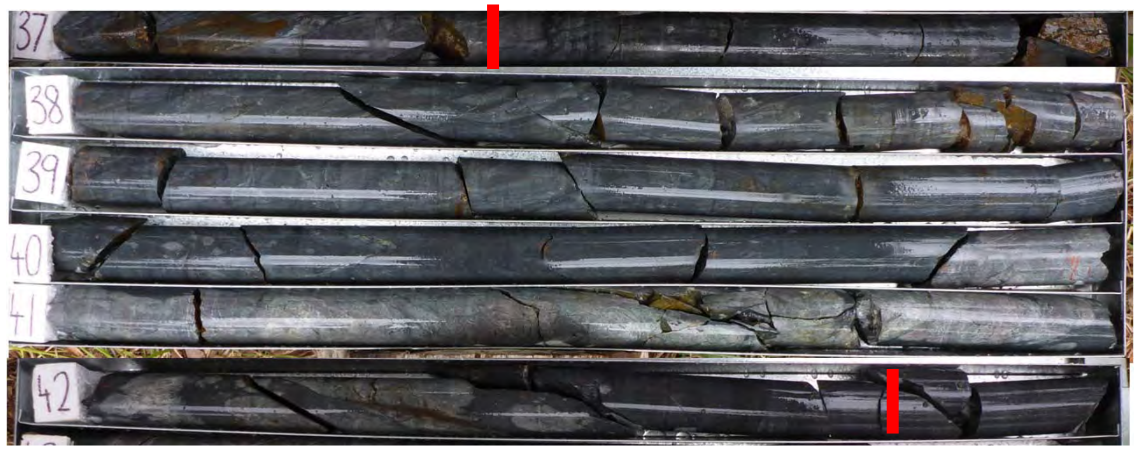
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	110	484	2.5	0.1	46%	0.2
5	110	484	2.2	0.1	46%	0.2
5	220	594	63.0	2.3	57%	3.9
5	220	594	65.0	2.4	57%	4.1
5	330	704	291.0	10.8	68%	15.3
5	330	704	292.0	10.8	68%	15.4
5	220	594	90.0	3.3	57%	5.6
5	220	594	91.0	3.4	57%	5.7
5	110	484	4.8	0.2	46%	0.4
5	110	484	4.4	0.2	46%	0.3





Interpreted flow mechanism: Dilation

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

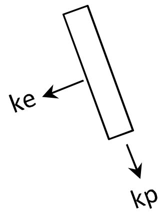
uL (Fell et al., 2005) - upper chart	20.7
uL (Houlsby, 1976) - lower chart	2.5

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	4.7.E-02	m/day
ke/kp = 100	4.0.E-02	m/day
ke/kp = 10	3.4.E-02	m/day
ke/kp = 1	2.7.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.9.E-02 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

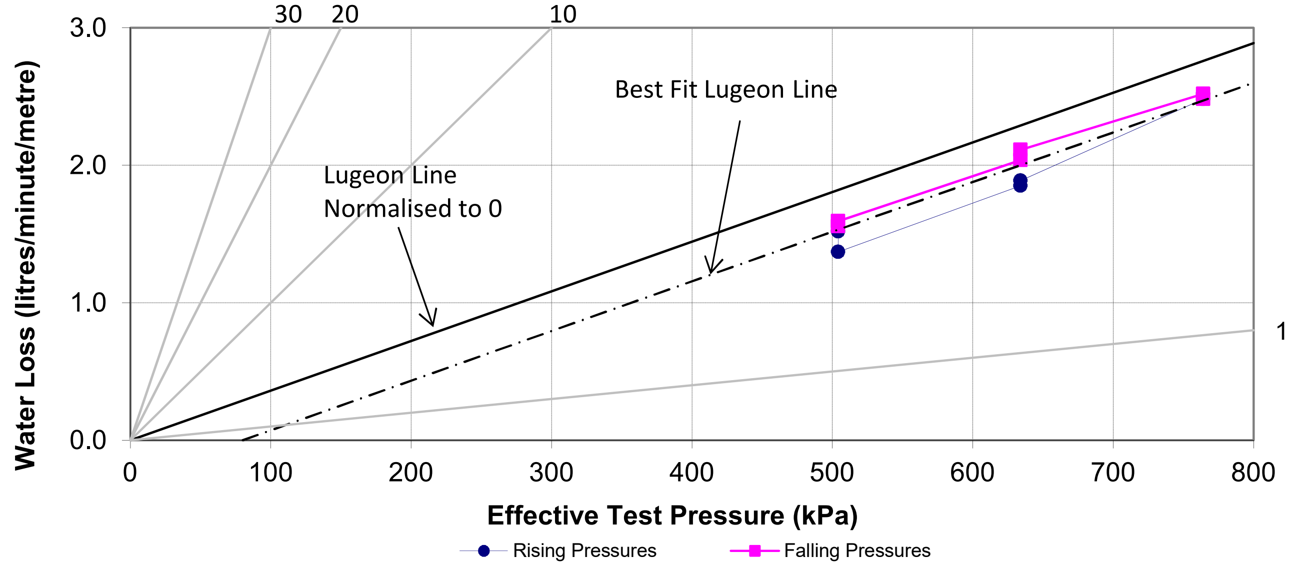
BHH151 - 43.4m - 48.8m

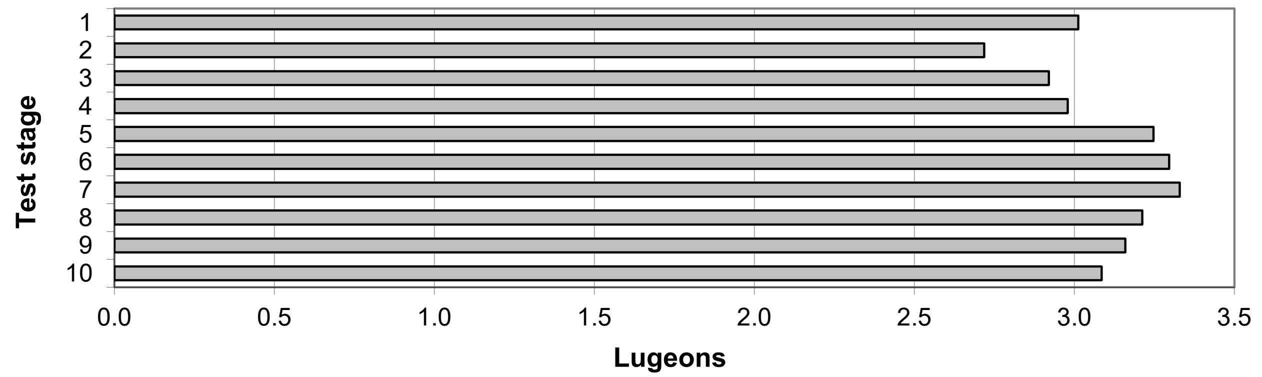
hole no:	BHH151	date:	9-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	M Kobler

section tested: from 43.40 m to 48.80 m length 5.40 m

relative level at surface:	150.5 m RL	centre of test section (downhole):	46.1 m
gauge height:	0.7 m	test depth converted to vertical:	46.1 m
inclination of borehole:	90 °	relative level at centre of test section:	104.4 m RL
inferred groundwater level:	113.0 m RL	correction to gauge pressure (1)(2):	374.2 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	130	504	41.0	1.5	42%	3.0
5	130	504	37.0	1.4	42%	2.7
5	260	634	50.0	1.9	53%	2.9
5	260	634	51.0	1.9	53%	3.0
5	390	764	67.0	2.5	64%	3.2
5	390	764	68.0	2.5	64%	3.3
5	260	634	57.0	2.1	53%	3.3
5	260	634	55.0	2.0	53%	3.2
5	130	504	43.0	1.6	42%	3.2
5	130	504	42.0	1.6	42%	3.1





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

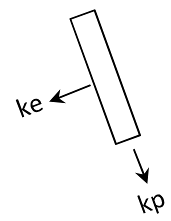
uL (Fell et al., 2005) - upper chart	3.6
uL (Houlsby, 1976) - lower chart	3.1

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	5.7.E-02	m/day
ke/kp = 100	4.9.E-02	m/day
ke/kp = 10	4.1.E-02	m/day
ke/kp = 1	3.3.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 3.5.E-02 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

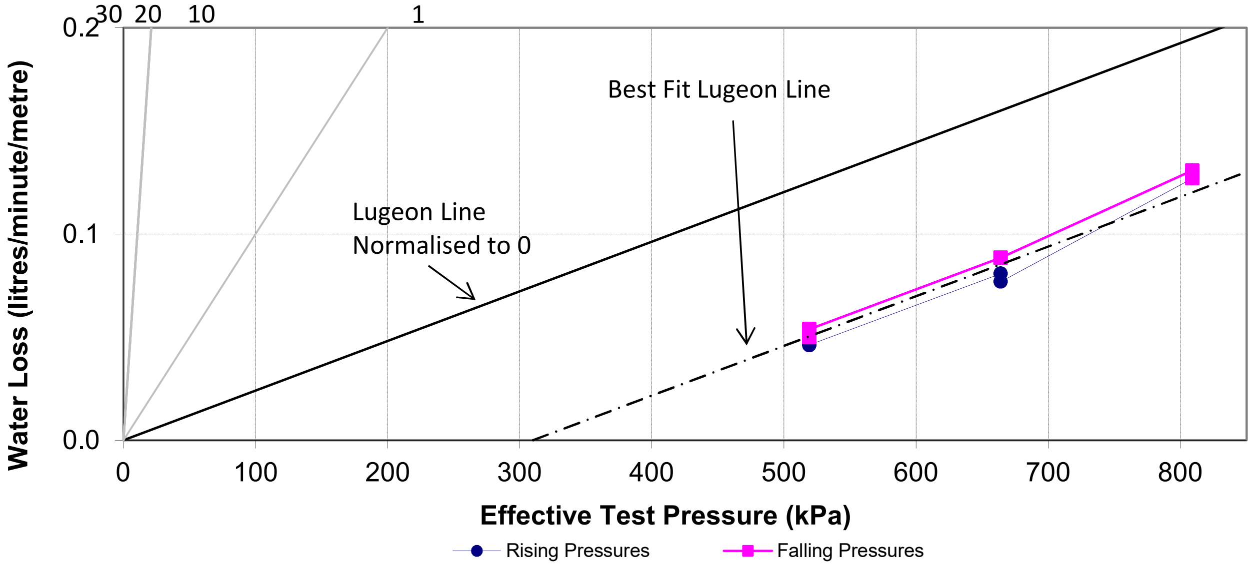
BHH151 - 49.5m - 54.7m

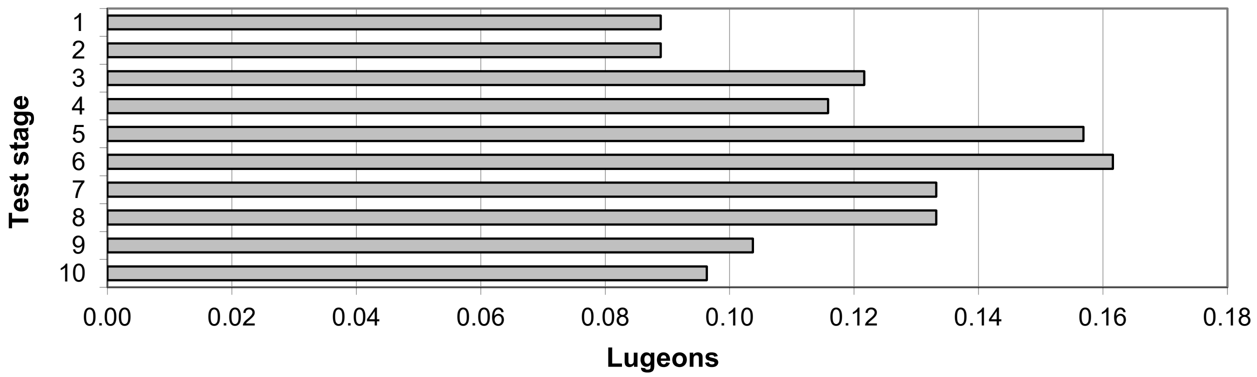
hole no: BHH151 | date: 9-Nov-16
project: Coffs Harbour Bypass
location: Gatelys Road | by: M Kobler

section tested: from 49.50 m to 54.70 m length 5.20 m

relative level at surface: 150.5 m RL | centre of test section (downhole): 52.1 m
gauge height: 0.7 m | test depth converted to vertical: 52.1 m
inclination of borehole: 90 ° | relative level at centre of test section: 98.4 m RL
inferred groundwater level: 113.0 m RL | correction to gauge pressure [(1)(2)]: 374.2 kPa

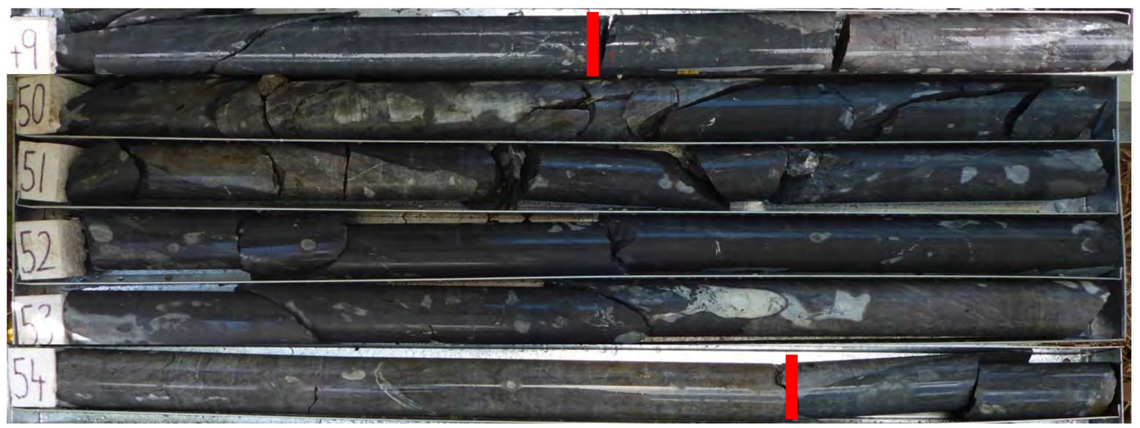
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure [(3)]	lugeons litres / min / m / 1000kPa
5	145	519	1.2	0.0	38%	0.09
5	145	519	1.2	0.0	38%	0.09
5	290	664	2.1	0.1	49%	0.12
5	290	664	2.0	0.1	49%	0.12
5	435	809	3.3	0.1	60%	0.16
5	435	809	3.4	0.1	60%	0.16
5	290	664	2.3	0.1	49%	0.13
5	290	664	2.3	0.1	49%	0.13
5	145	519	1.4	0.1	38%	0.10
5	145	519	1.3	0.1	38%	0.10





Interpreted flow mechanism: Laminar Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size: HQ3
Lithology: Argillite
Primary Defect Set: Joints

Lugeon (uL) Assessment

uL (Fell et al., 2005) - upper chart 0.24

uL (Houlsby, 1976) - lower chart 0.12

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:

ke
kp

ke/kp = 1000	2.2.E-03	m/day
ke/kp = 100	1.9.E-03	m/day
ke/kp = 10	1.6.E-03	m/day
ke/kp = 1	1.3.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 1.3.E-03 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

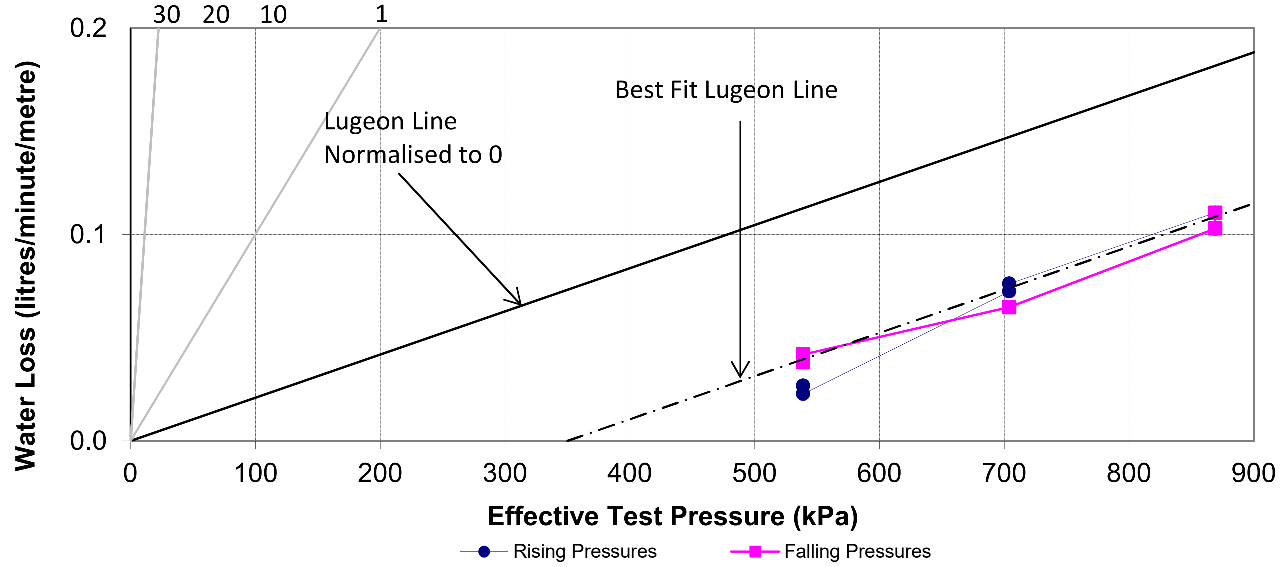
BHH151 - 55.4m - 60.65m

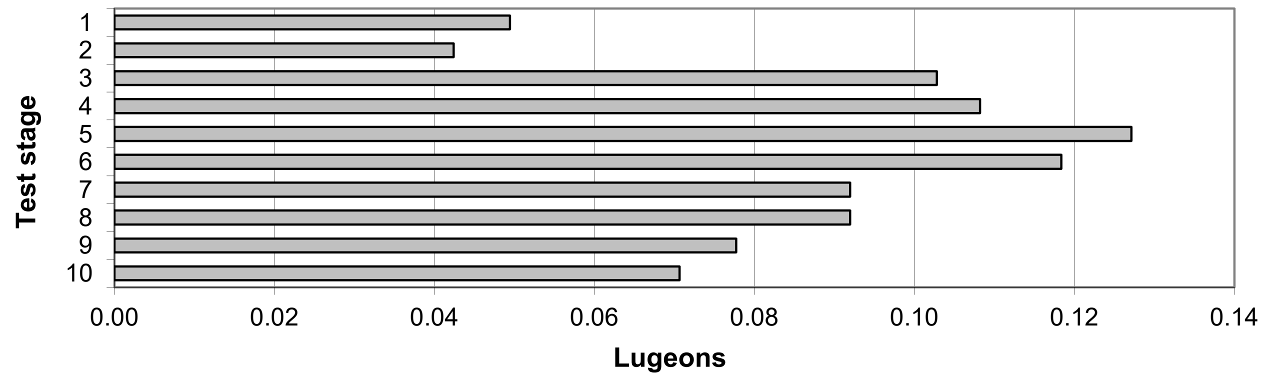
hole no:	BHH151	date:	10-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	M Kobler

section tested: from 55.40 m to 60.65 m length 5.25 m

relative level at surface:	150.5 m RL	centre of test section (downhole):	58.0 m
gauge height:	0.7 m	test depth converted to vertical:	58.0 m
inclination of borehole:	90 °	relative level at centre of test section:	92.5 m RL
inferred groundwater level:	113.0 m RL	correction to gauge pressure (1)(2):	374.2 kPa

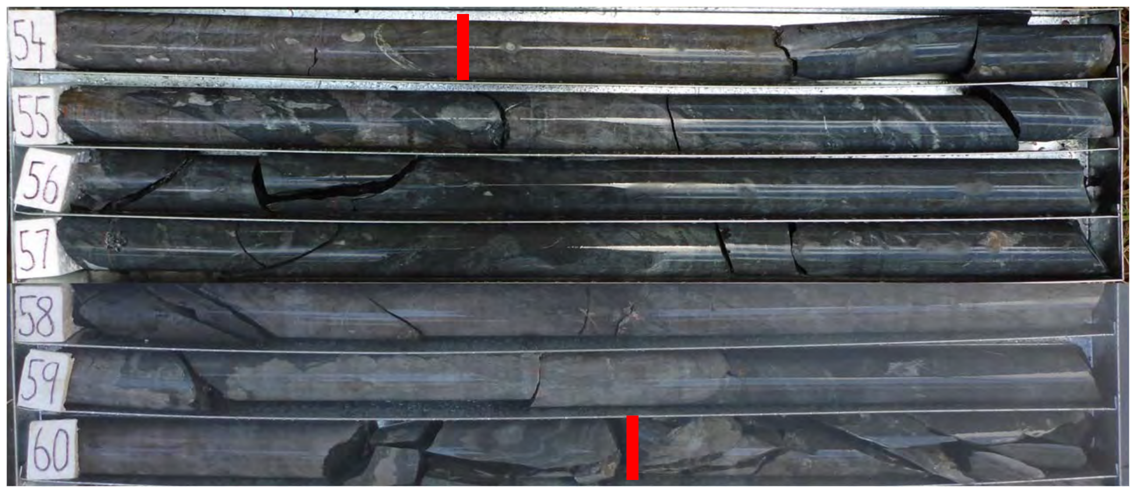
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	165	539	0.7	0.0	36%	0.05
5	165	539	0.6	0.0	36%	0.04
5	330	704	1.9	0.1	47%	0.10
5	330	704	2.0	0.1	47%	0.11
5	495	869	2.9	0.1	58%	0.13
5	495	869	2.7	0.1	58%	0.12
5	330	704	1.7	0.1	47%	0.09
5	330	704	1.7	0.1	47%	0.09
5	165	539	1.1	0.0	36%	0.08
5	165	539	1.0	0.0	36%	0.07





Interpreted flow mechanism: Laminar Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

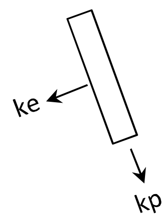
uL (Fell et al., 2005) - upper chart	0.21
uL (Houlsby, 1976) - lower chart	0.09

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	1.6.E-03	m/day
ke/kp = 100	1.4.E-03	m/day
ke/kp = 10	1.2.E-03	m/day
ke/kp = 1	9.3.E-04	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 9.9.E-04 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

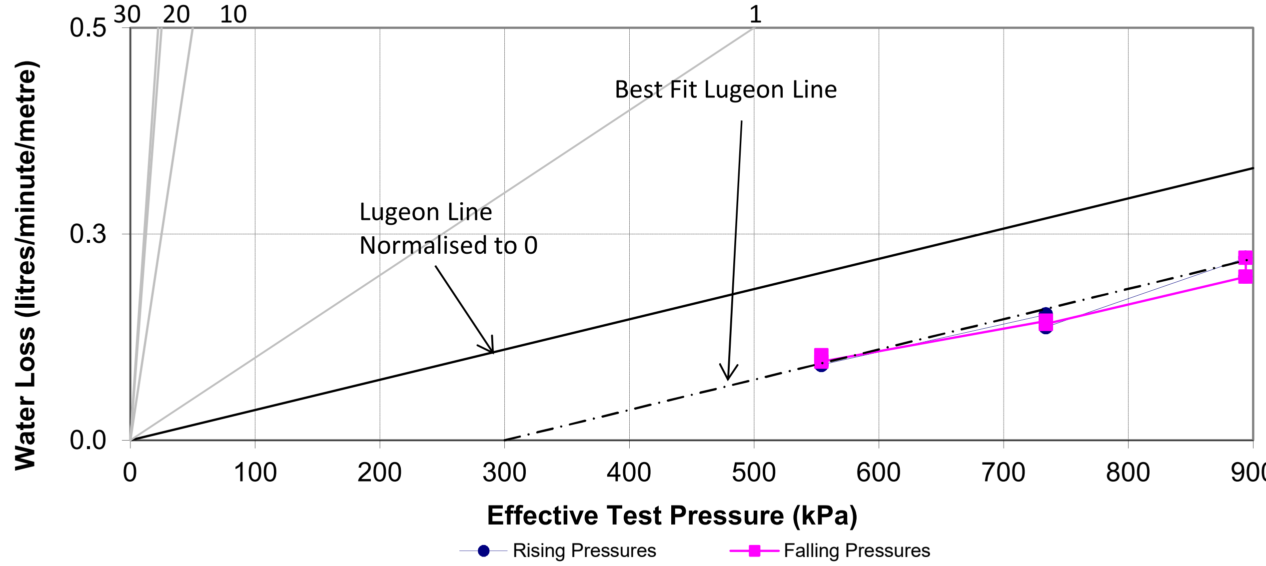
BHH151 - 61.4m - 66.65m

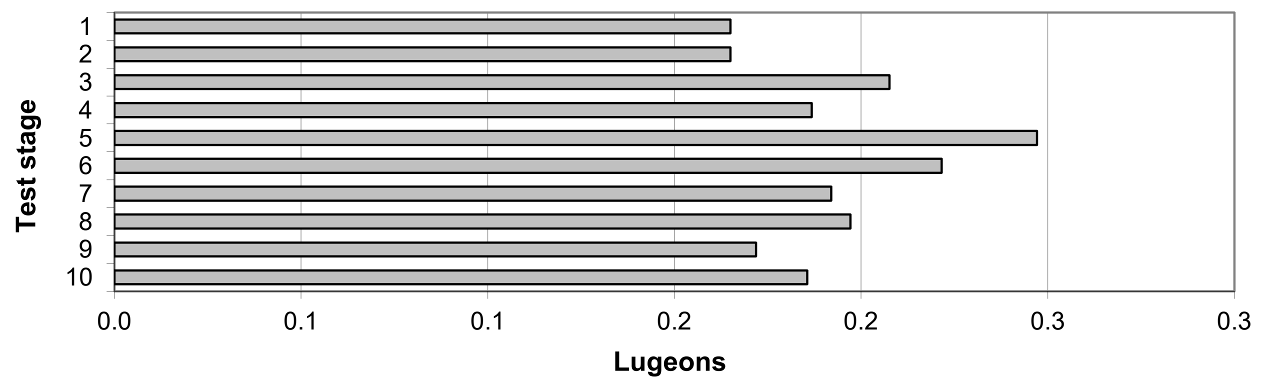
hole no:	BHH151	date:	10-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	M Kobler

section tested: from 61.40 m to 66.65 m length 5.25 m

relative level at surface:	150.5 m RL	centre of test section (downhole):	64.0 m
gauge height:	0.7 m	test depth converted to vertical:	64.0 m
inclination of borehole:	90 °	relative level at centre of test section:	86.5 m RL
inferred groundwater level:	113.0 m RL	correction to gauge pressure (1)(2):	374.2 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	180	554	2.4	0.1	33%	0.2
5	180	554	2.4	0.1	33%	0.2
5	360	734	4.0	0.2	44%	0.2
5	360	734	3.6	0.1	44%	0.2
5	520	894	5.8	0.2	54%	0.2
5	520	894	5.2	0.2	54%	0.2
5	360	734	3.7	0.1	44%	0.2
5	360	734	3.8	0.1	44%	0.2
5	180	554	2.5	0.1	33%	0.2
5	180	554	2.7	0.1	33%	0.2





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

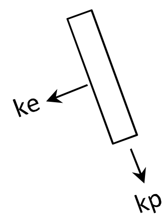
uL (Fell et al., 2005) - upper chart	0.4
uL (Houlsby, 1976) - lower chart	0.2

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	3.5.E-03	m/day
ke/kp = 100	3.0.E-03	m/day
ke/kp = 10	2.5.E-03	m/day
ke/kp = 1	2.0.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.2.E-03 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

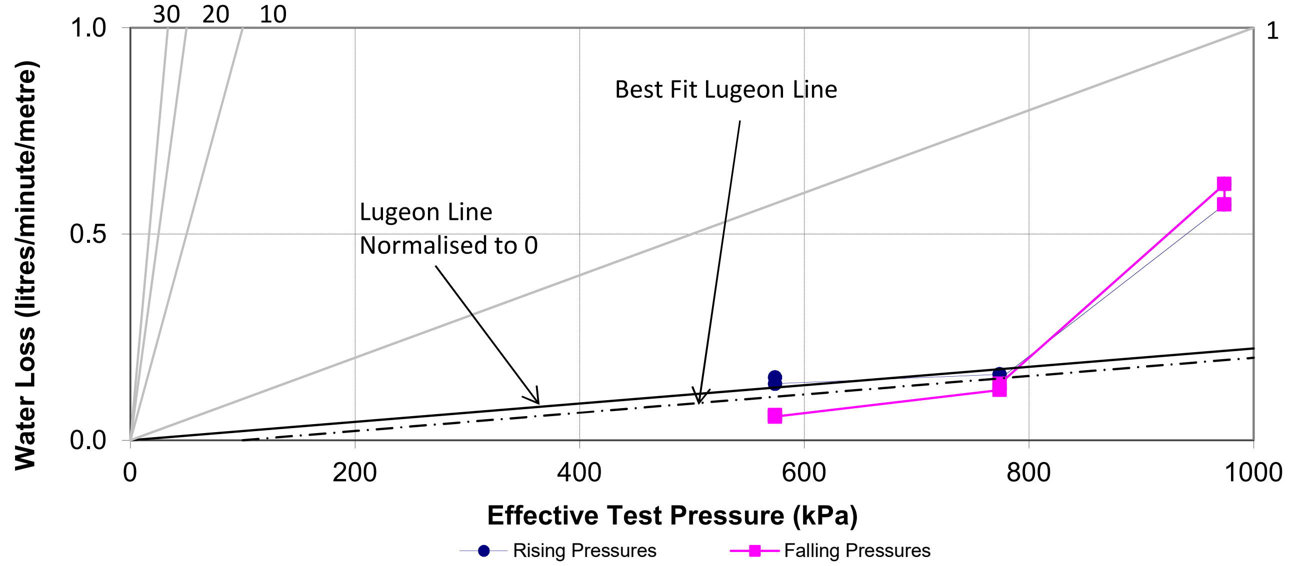
BHH151 - 67.4m - 72.65m

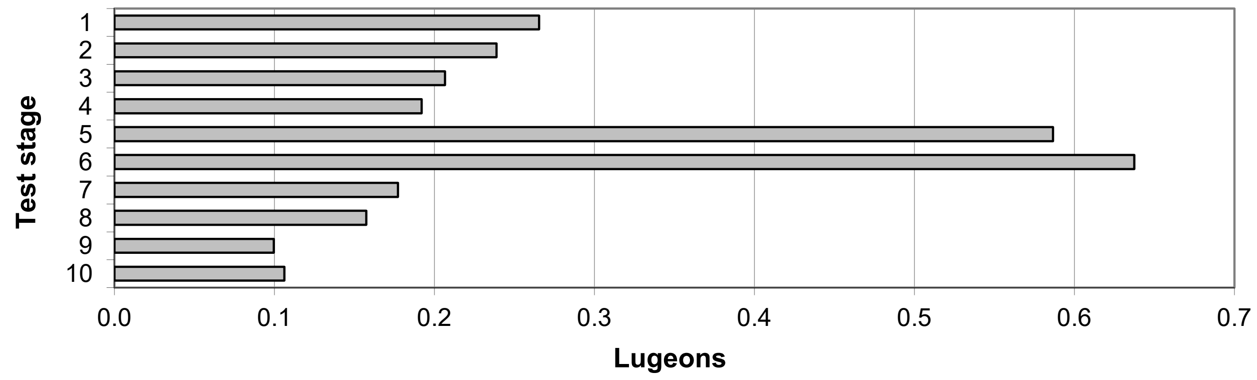
hole no:	BHH151	date:	11-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	M Kobler

section tested: from 67.40 m to 72.65 m length 5.25 m

relative level at surface:	150.5 m RL	centre of test section (downhole):	70.0 m
gauge height:	0.7 m	test depth converted to vertical:	70.0 m
inclination of borehole:	90 °	relative level at centre of test section:	80.5 m RL
inferred groundwater level:	113.0 m RL	correction to gauge pressure (1)(2):	374.2 kPa

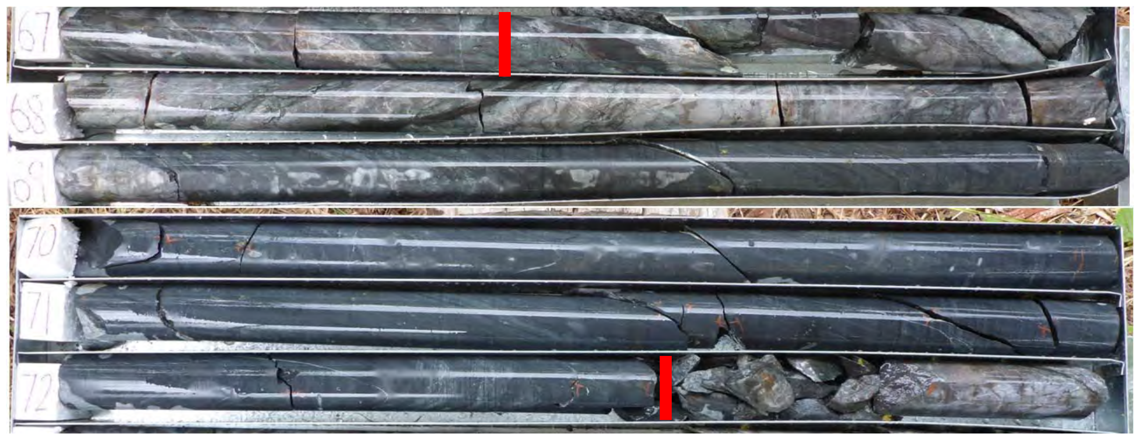
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	200	574	4.0	0.2	32%	0.3
5	200	574	3.6	0.1	32%	0.2
5	400	774	4.2	0.2	43%	0.2
5	400	774	3.9	0.1	43%	0.2
5	600	974	15.0	0.6	54%	0.6
5	600	974	16.3	0.6	54%	0.6
5	400	774	3.6	0.1	43%	0.2
5	400	774	3.2	0.1	43%	0.2
5	200	574	1.5	0.1	32%	0.1
5	200	574	1.6	0.1	32%	0.1





Interpreted flow mechanism: Dilation

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

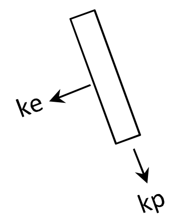
uL (Fell et al., 2005) - upper chart	0.2
uL (Houlsby, 1976) - lower chart	0.2

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	3.3.E-03	m/day
ke/kp = 100	2.8.E-03	m/day
ke/kp = 10	2.4.E-03	m/day
ke/kp = 1	1.9.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.0.E-03 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink Job no. PSM2876

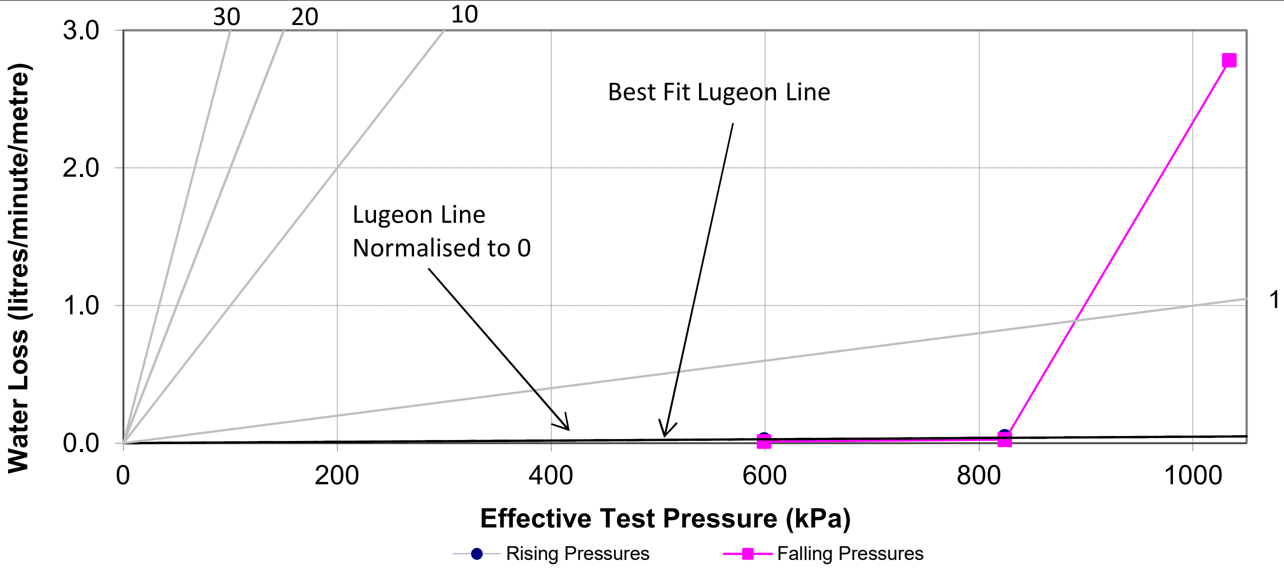
Borehole Water Pressure Test Summary **BHH151 - 73.4m - 78.65m**

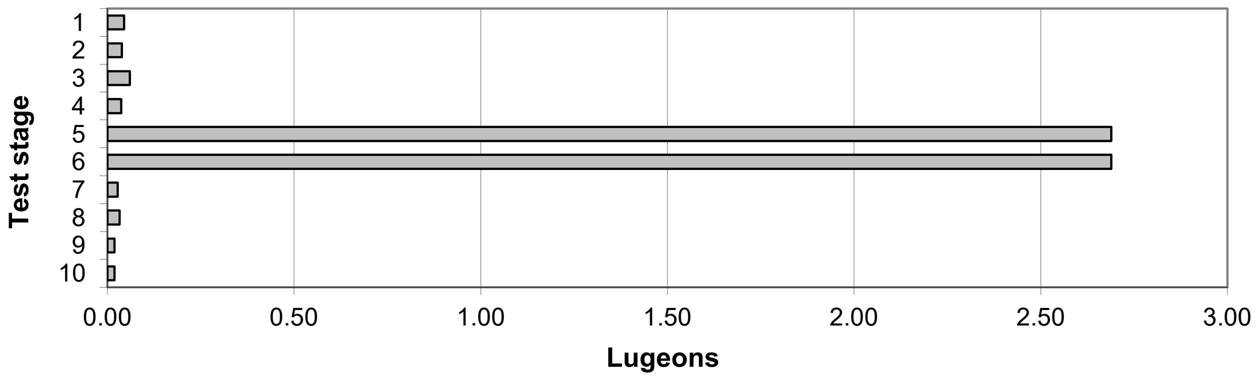
hole no:	BHH151	date:	11-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	M Kobler

section tested: from 73.40 m to 78.65 m length 5.25 m

relative level at surface:	150.5 m RL	centre of test section (downhole):	76.0 m
gauge height:	0.7 m	test depth converted to vertical:	76.0 m
inclination of borehole:	90 °	relative level at centre of test section:	74.5 m RL
inferred groundwater level:	113.0 m RL	correction to gauge pressure (1)(2):	374.2 kPa

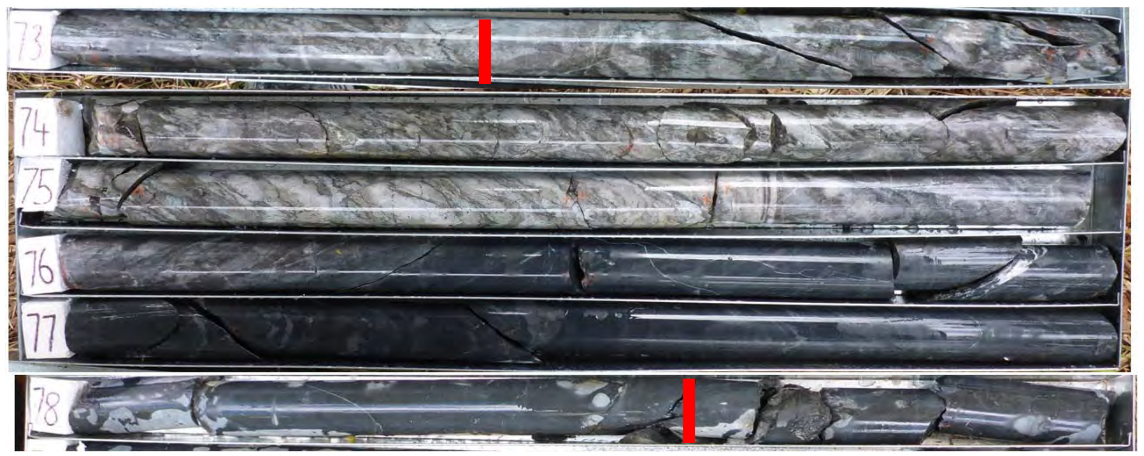
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	225	599	0.7	0.0	30%	0.04
5	225	599	0.6	0.0	30%	0.04
5	450	824	1.3	0.0	42%	0.06
5	450	824	0.8	0.0	42%	0.04
5	660	1034	73.0	2.8	52%	2.69
5	660	1034	73.0	2.8	52%	2.69
5	450	824	0.6	0.0	42%	0.03
5	450	824	0.7	0.0	42%	0.03
5	225	599	0.3	0.0	30%	0.02
5	225	599	0.3	0.0	30%	0.02





Interpreted flow mechanism: Dilation

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

uL (Fell et al., 2005) - upper chart	0.05
uL (Houlsby, 1976) - lower chart	0.03

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:

ke/kp = 1000	6.4.E-04	m/day
ke/kp = 100	5.5.E-04	m/day
ke/kp = 10	4.6.E-04	m/day
ke/kp = 1	3.7.E-04	m/day

ke kp

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 3.9.E-04 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

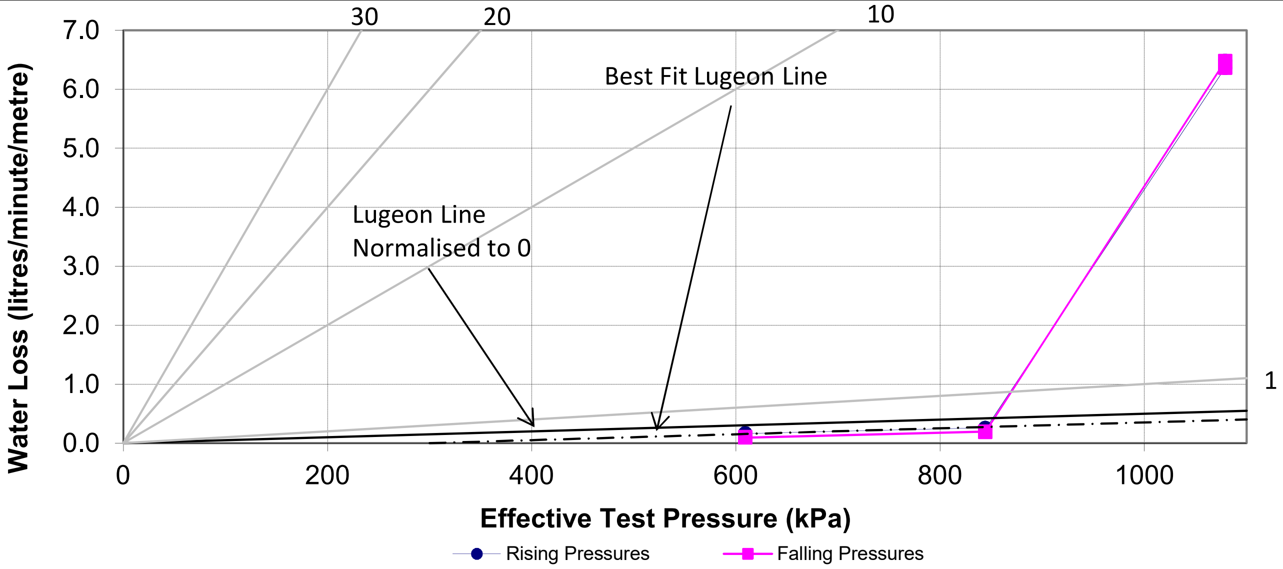
BHH151 - 79.4m - 84.65m

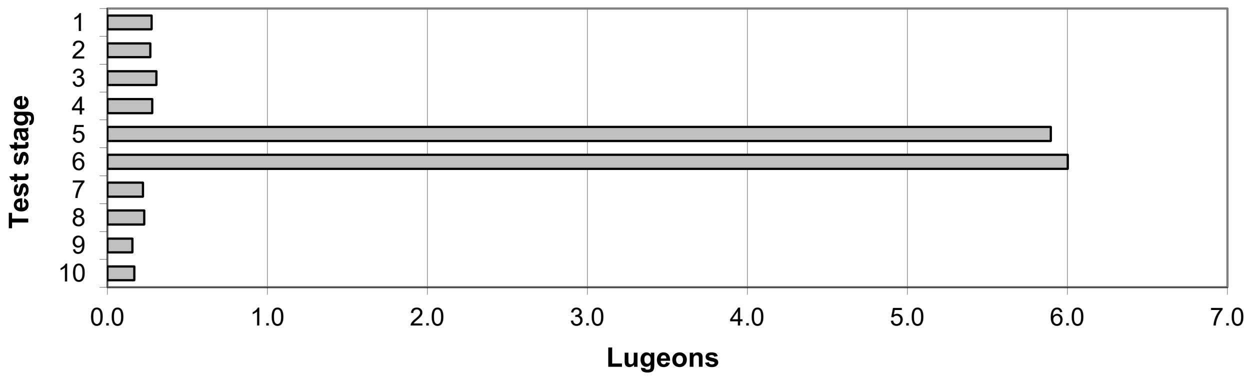
hole no:	BHH151	date:	14-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	M Kobler

section tested: from 79.40 m to 84.65 m length 5.25 m

relative level at surface:	150.5 m RL	centre of test section (downhole):	82.0 m
gauge height:	0.7 m	test depth converted to vertical:	82.0 m
inclination of borehole:	90 °	relative level at centre of test section:	68.5 m RL
inferred groundwater level:	113.0 m RL	correction to gauge pressure (1)(2):	374.2 kPa

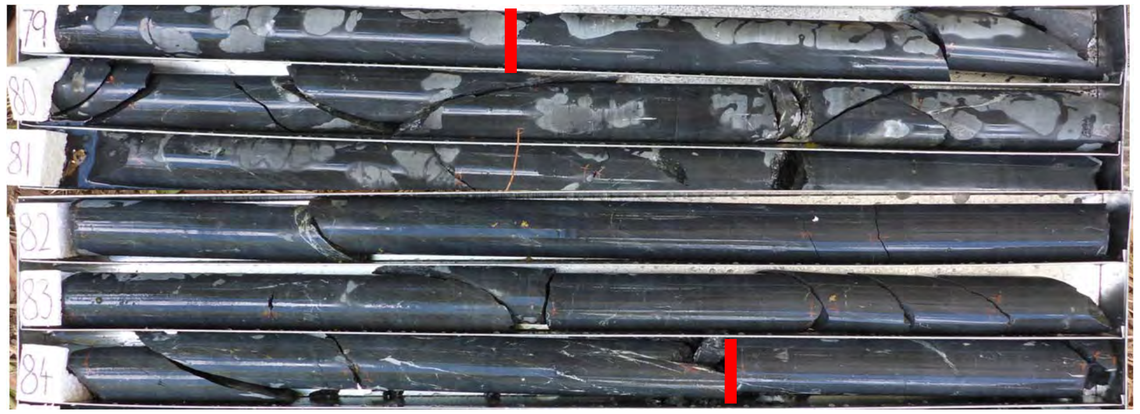
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	235	609	4.4	0.2	29%	0.3
5	235	609	4.3	0.2	29%	0.3
5	470	844	6.8	0.3	40%	0.3
5	470	844	6.2	0.2	40%	0.3
5	705	1079	167.0	6.4	51%	5.9
5	705	1079	170.0	6.5	51%	6.0
5	470	844	4.9	0.2	40%	0.2
5	470	844	5.1	0.2	40%	0.2
5	235	609	2.5	0.1	29%	0.2
5	235	609	2.7	0.1	29%	0.2





Interpreted flow mechanism: Dilation

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

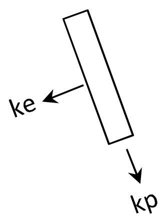
uL (Fell et al., 2005) - upper chart	0.5
uL (Houlsby, 1976) - lower chart	0.2

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	4.4.E-03	m/day
ke/kp = 100	3.7.E-03	m/day
ke/kp = 10	3.1.E-03	m/day
ke/kp = 1	2.5.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.7.E-03 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

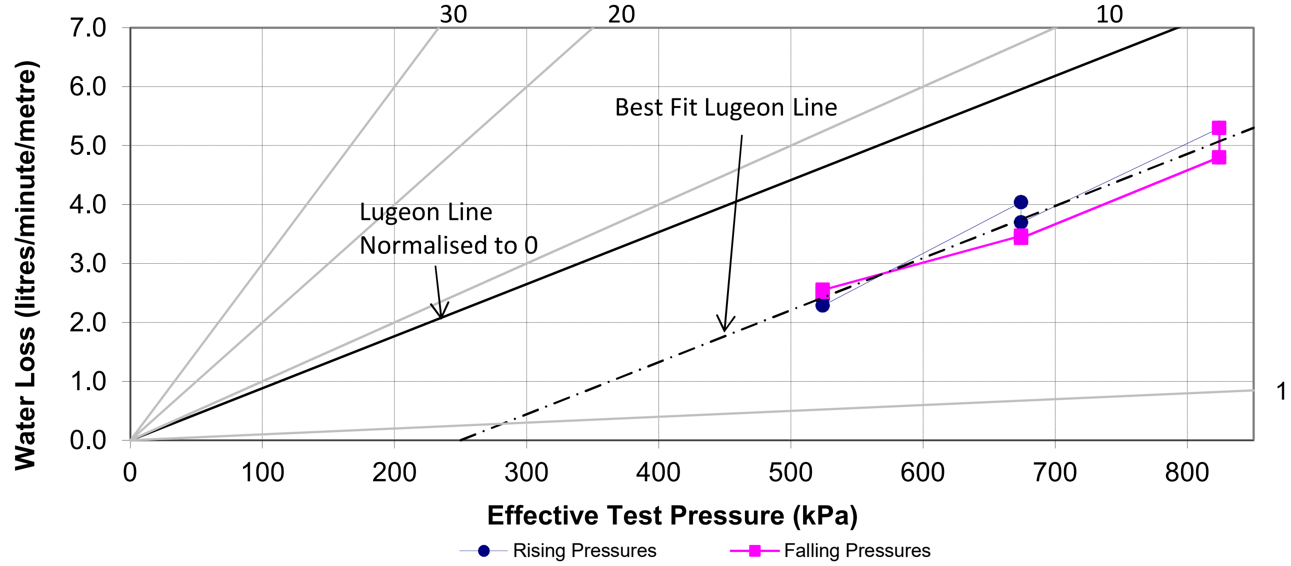
BHH151 - 85.4m - 90.65m

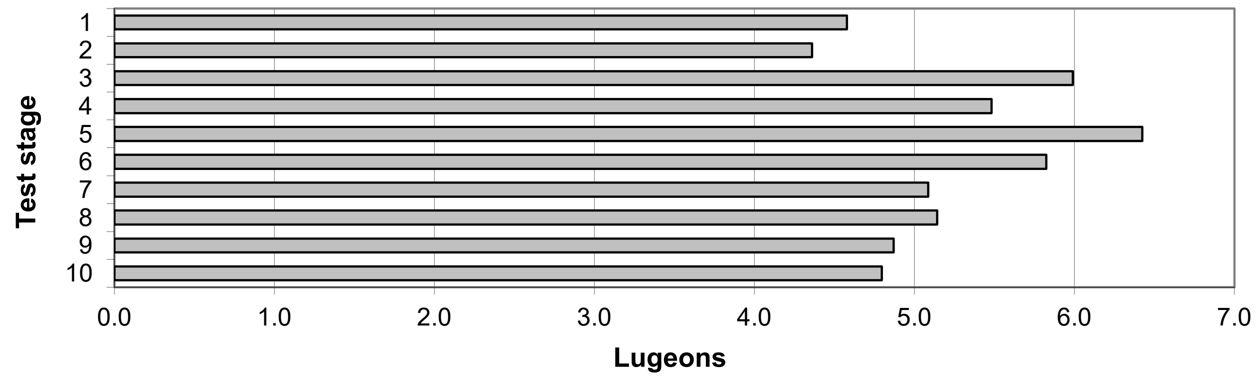
hole no:	BHH151	date:	14-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	M Kobler

section tested: from 85.40 m to 90.65 m length 5.25 m

relative level at surface:	150.5 m RL	centre of test section (downhole):	88.0 m
gauge height:	0.7 m	test depth converted to vertical:	88.0 m
inclination of borehole:	90 °	relative level at centre of test section:	62.5 m RL
inferred groundwater level:	113.0 m RL	correction to gauge pressure [(1)(2)]:	374.2 kPa

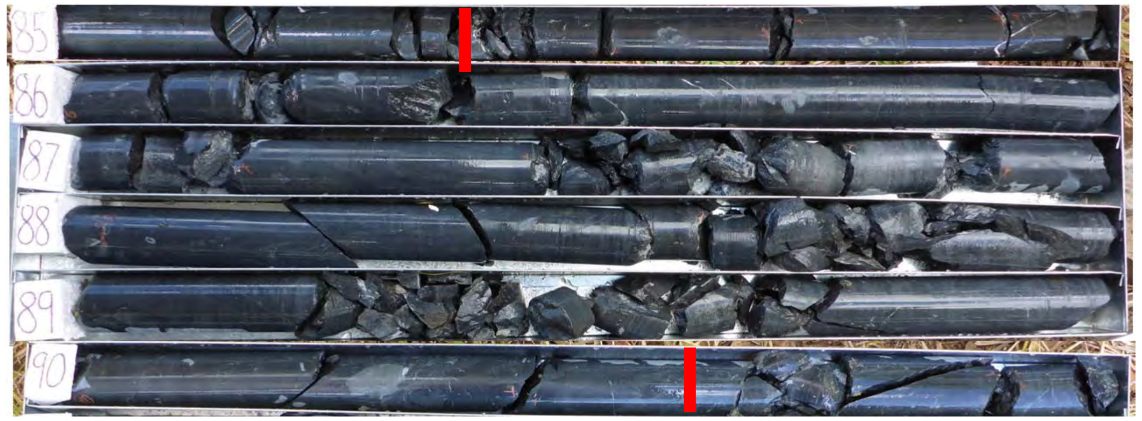
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure [(3)]	lugeons litres / min / m / 1000kPa
5	150	524	63.0	2.4	23%	4.6
5	150	524	60.0	2.3	23%	4.4
5	300	674	106.0	4.0	29%	6.0
5	300	674	97.0	3.7	29%	5.5
5	450	824	139.0	5.3	36%	6.4
5	450	824	126.0	4.8	36%	5.8
5	300	674	90.0	3.4	29%	5.1
5	300	674	91.0	3.5	29%	5.1
5	150	524	67.0	2.6	23%	4.9
5	150	524	66.0	2.5	23%	4.8





Interpreted flow mechanism: Lamianr Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

uL (Fell et al., 2005) - upper chart	8.8
uL (Houlsby, 1976) - lower chart	5.3

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:

ke kp

ke/kp = 1000	9.6.E-02	m/day
ke/kp = 100	8.3.E-02	m/day
ke/kp = 10	6.9.E-02	m/day
ke/kp = 1	5.5.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 5.9.E-02 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

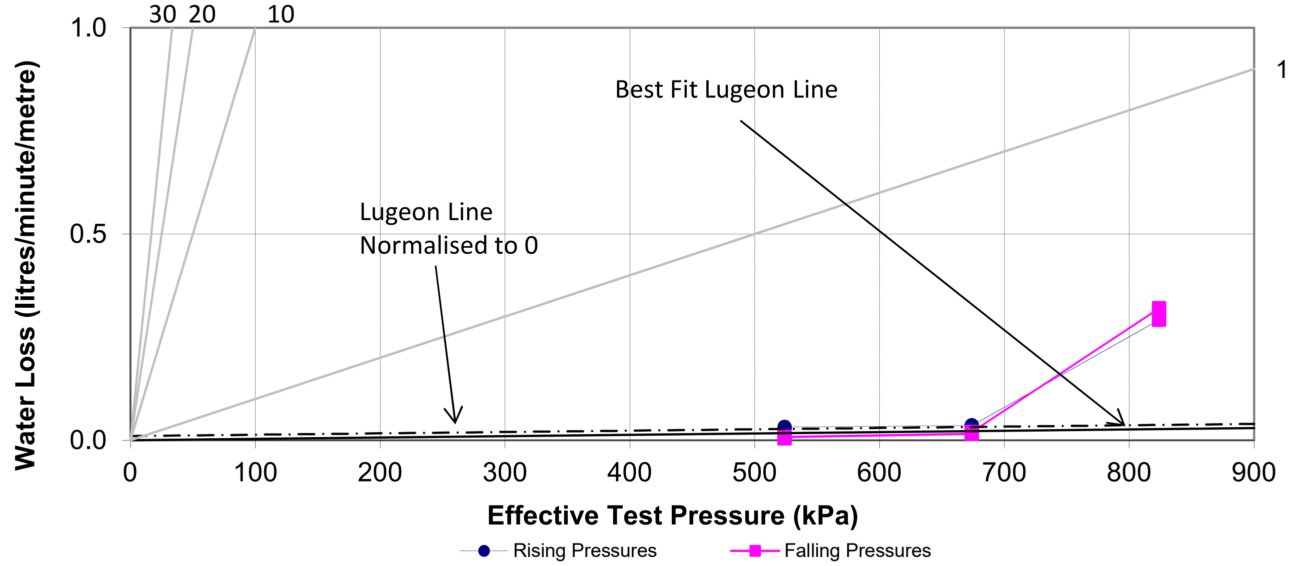
BHH151 - 91.4m - 96.4m

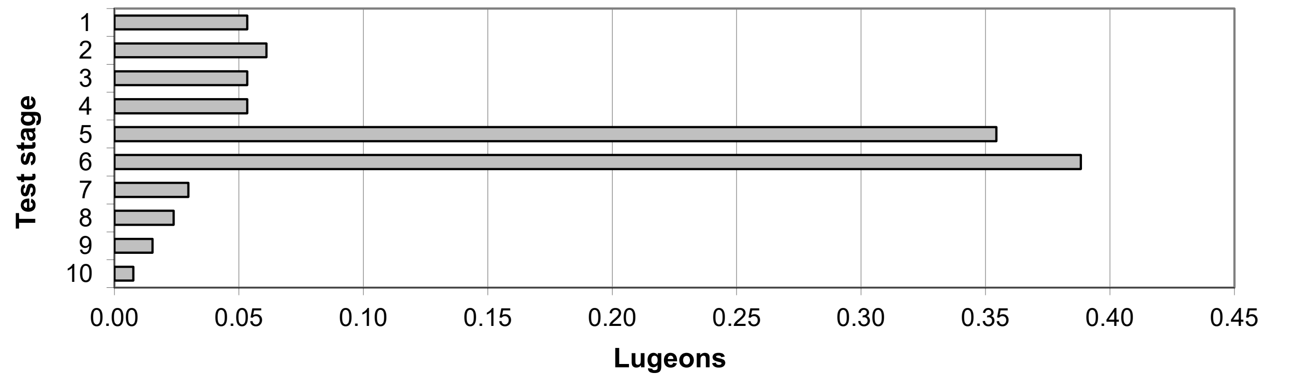
hole no:	BHH151	date:	15-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	M Kobler

section tested: from 91.40 m to 96.40 m length 5.00 m

relative level at surface:	150.5 m RL	centre of test section (downhole):	93.9 m
gauge height:	0.7 m	test depth converted to vertical:	93.9 m
inclination of borehole:	90 °	relative level at centre of test section:	56.6 m RL
inferred groundwater level:	113.0 m RL	correction to gauge pressure (1)(2):	374.2 kPa

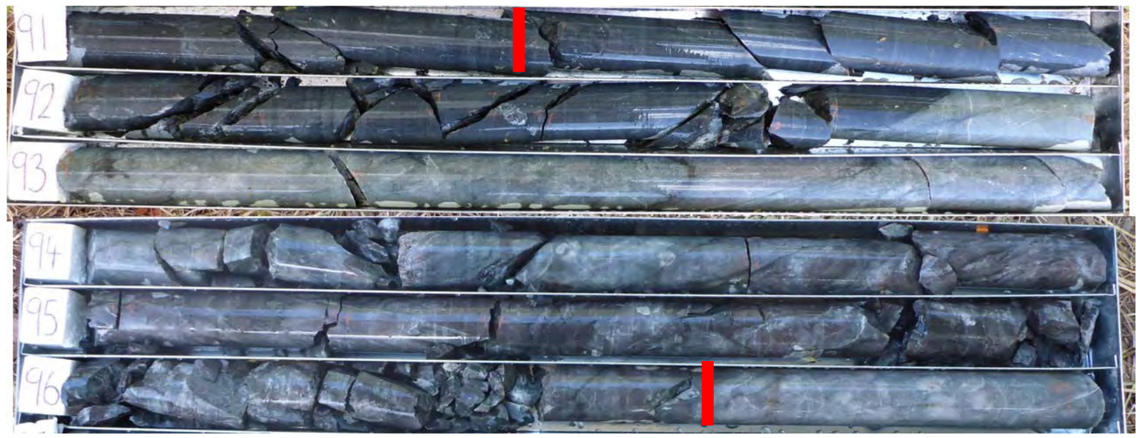
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	524	0.7	0.0	21%	0.05
5	150	524	0.8	0.0	21%	0.06
5	300	674	0.9	0.0	28%	0.05
5	300	674	0.9	0.0	28%	0.05
5	450	824	7.3	0.3	34%	0.35
5	450	824	8.0	0.3	34%	0.39
5	300	674	0.5	0.0	28%	0.03
5	300	674	0.4	0.0	28%	0.02
5	150	524	0.2	0.0	21%	0.02
5	150	524	0.1	0.0	21%	0.01





Interpreted flow mechanism: Dilation

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

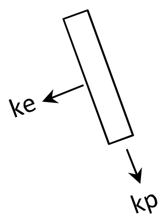
uL (Fell et al., 2005) - upper chart	0.03
uL (Houlsby, 1976) - lower chart	0.04

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	6.8.E-04	m/day
ke/kp = 100	5.8.E-04	m/day
ke/kp = 10	4.8.E-04	m/day
ke/kp = 1	3.9.E-04	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 4.1.E-04 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink | **Job no.** PSM2876

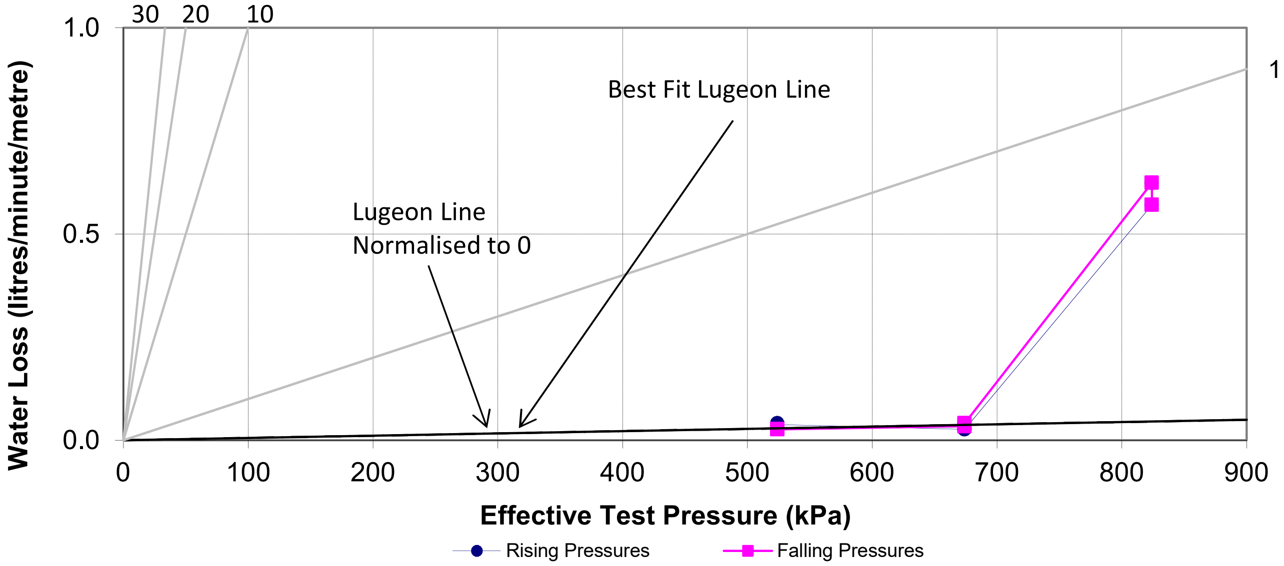
Borehole Water Pressure Test Summary | **BHH151 - 96.4m - 101.69m**

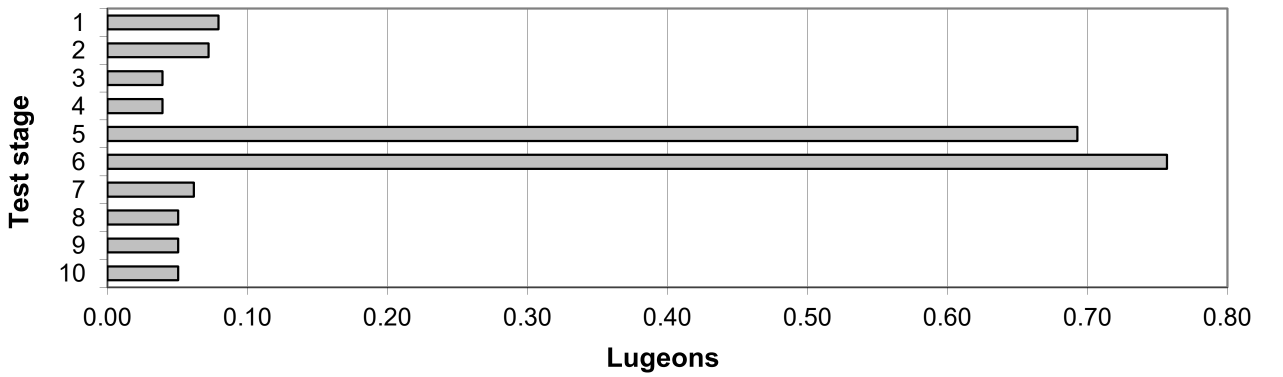
hole no: BHH151 | date: 15-Nov-16
project: Coffs Harbour Bypass
location: Gatelys Road | by: M Kobler

section tested: from 96.40 m to 101.69 m length 5.29 m

relative level at surface: 150.5 m RL | centre of test section (downhole): 99.0 m
gauge height: 0.7 m | test depth converted to vertical: 99.0 m
inclination of borehole: 90 ° | relative level at centre of test section: 51.5 m RL
inferred groundwater level: 113.0 m RL | correction to gauge pressure (1)(2): 374.2 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	524	1.1	0.0	20%	0.08
5	150	524	1.0	0.0	20%	0.07
5	300	674	0.7	0.0	26%	0.04
5	300	674	0.7	0.0	26%	0.04
5	450	824	15.1	0.6	32%	0.69
5	450	824	16.5	0.6	32%	0.76
5	300	674	1.1	0.0	26%	0.06
5	300	674	0.9	0.0	26%	0.05
5	150	524	0.7	0.0	20%	0.05
5	150	524	0.7	0.0	20%	0.05





Interpreted flow mechanism: Dilation

Image of packer test section:

LEGEND:
Packer zone boundary

Core size: HQ3
Lithology: Argillite
Primary Defect Set: Joints

Lugeon (uL) Assessment

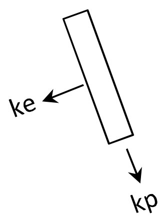
uL (Fell et al., 2005) - upper chart 0.06
uL (Houlsby, 1976) - lower chart 0.06

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	1.0.E-03	m/day
ke/kp = 100	8.7.E-04	m/day
ke/kp = 10	7.3.E-04	m/day
ke/kp = 1	5.8.E-04	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 6.2.E-04 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Test undertaken above water table

Job no. PSM2876

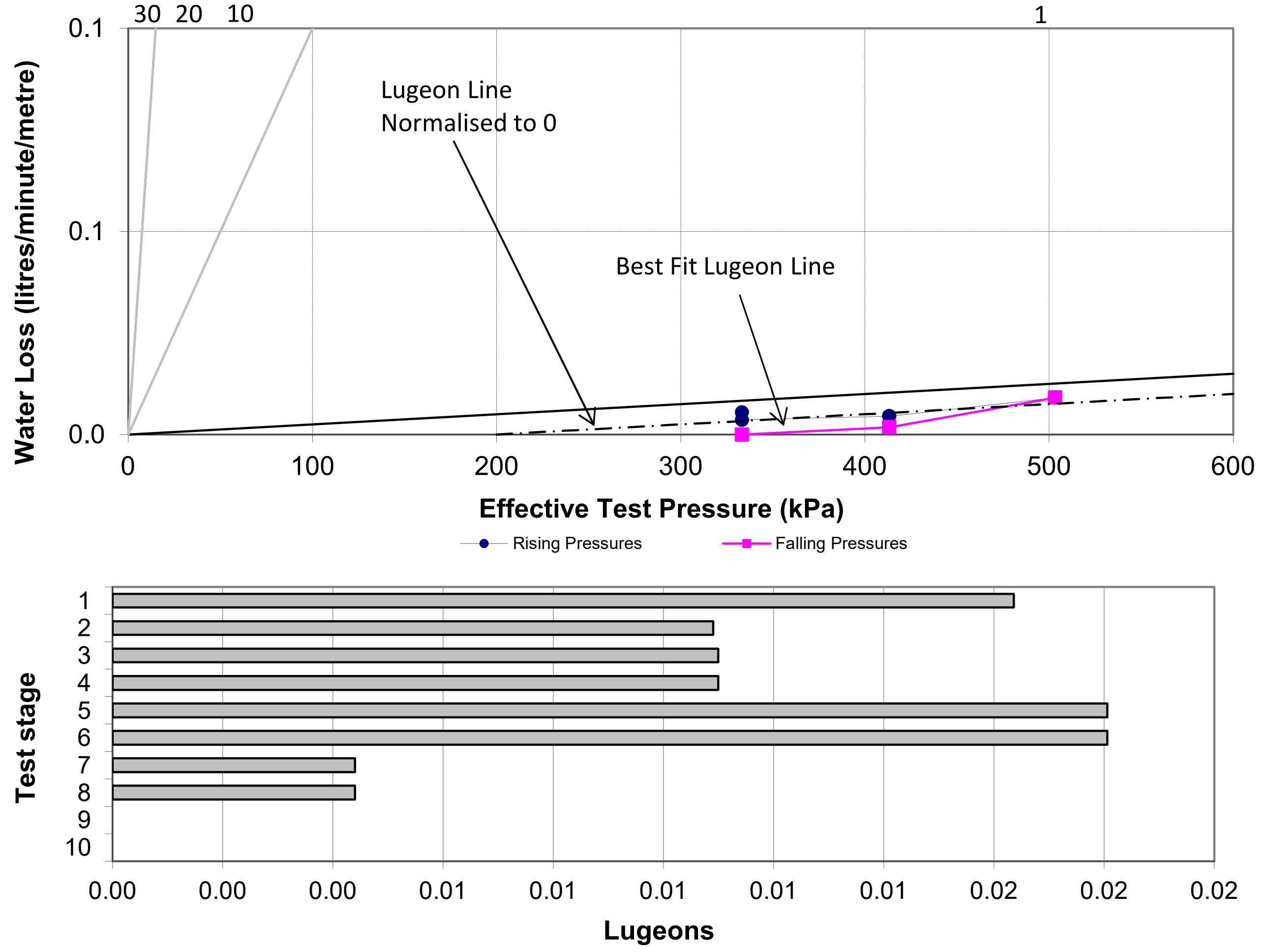
BHH152 - 20m - 31m

hole no:	BHH152	date:	7-Dec-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

section tested: from 20.00 m to 31.00 m length 11.00 m

relative level at surface:	139.1 m RL	centre of test section (downhole):	25.5 m
gauge height:	1.9 m	test depth converted to vertical:	24.0 m
inclination of borehole:	70 °	relative level at centre of test section:	115.1 m RL
inferred groundwater level:	108.5 m RL	correction to gauge pressure (1)(2):	253.4 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	80	333	0.3	0.0	54%	0.02
5	80	333	0.2	0.0	54%	0.01
5	160	413	0.3	0.0	66%	0.01
5	160	413	0.3	0.0	66%	0.01
5	250	503	0.5	0.0	81%	0.02
5	250	503	0.5	0.0	81%	0.02
5	160	413	0.1	0.0	66%	0.00
5	160	413	0.1	0.0	66%	0.00
5	80	333	0.0	0.0	54%	0.00
5	80	333	0.0	0.0	54%	0.00



Interpreted flow mechanism: Void Filling

Image of packer test section:

LEGEND:

Packer zone boundary

Core size: HQ3

Lithology: Argillite

Primary Defect Set: Joints

Lugeon (uL) Assessment

uL (Fell et al., 2005) - upper chart 0.03

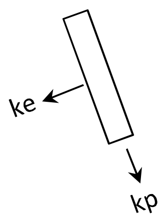
uL (Houlsby, 1976) - lower chart 0.00

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	0.0.E+00	m/day
ke/kp = 100	0.0.E+00	m/day
ke/kp = 10	0.0.E+00	m/day
ke/kp = 1	0.0.E+00	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 0.0.E+00 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

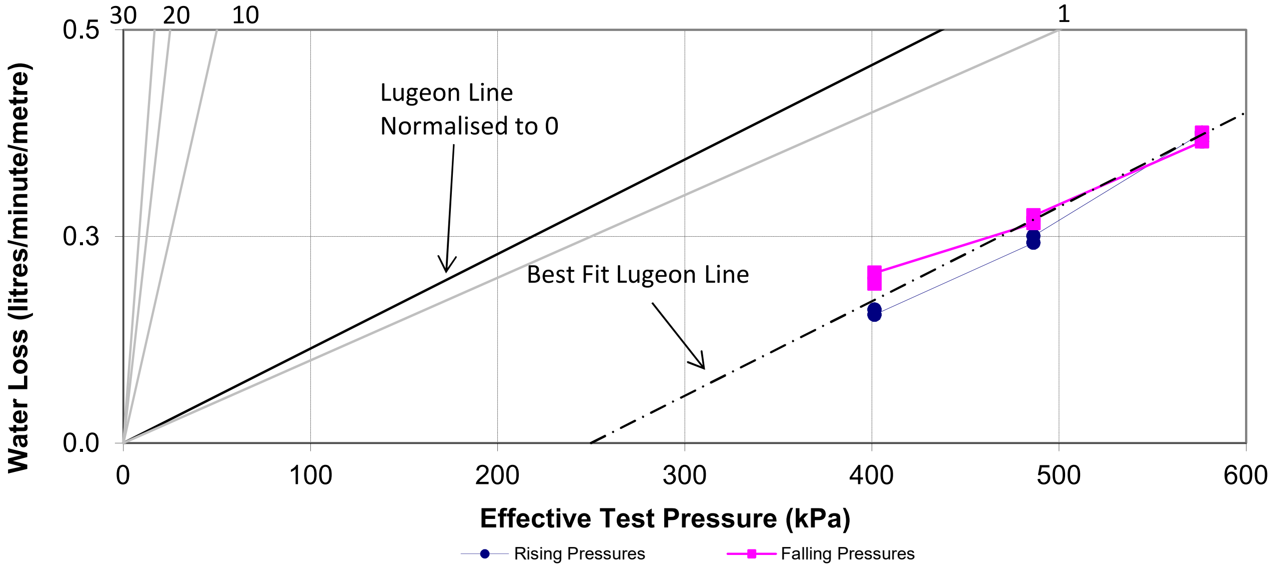
BHH152 - 32m - 49m

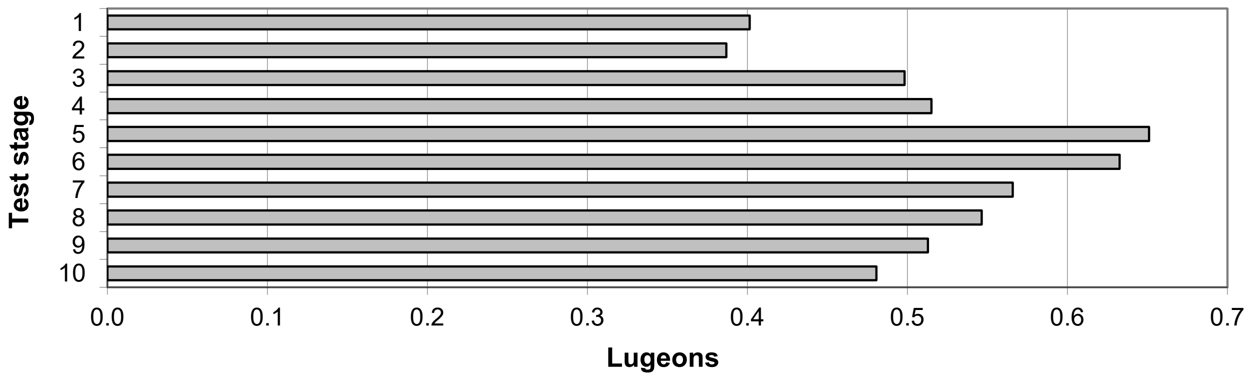
hole no:	BHH152	date:	9-Dec-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

section tested: from 32.00 m to 49.00 m length 17.00 m

relative level at surface:	139.1 m RL	centre of test section (downhole):	40.5 m
gauge height:	1.7 m	test depth converted to vertical:	38.1 m
inclination of borehole:	70 °	relative level at centre of test section:	101.0 m RL
inferred groundwater level:	108.5 m RL	correction to gauge pressure (1)(2):	316.5 kPa

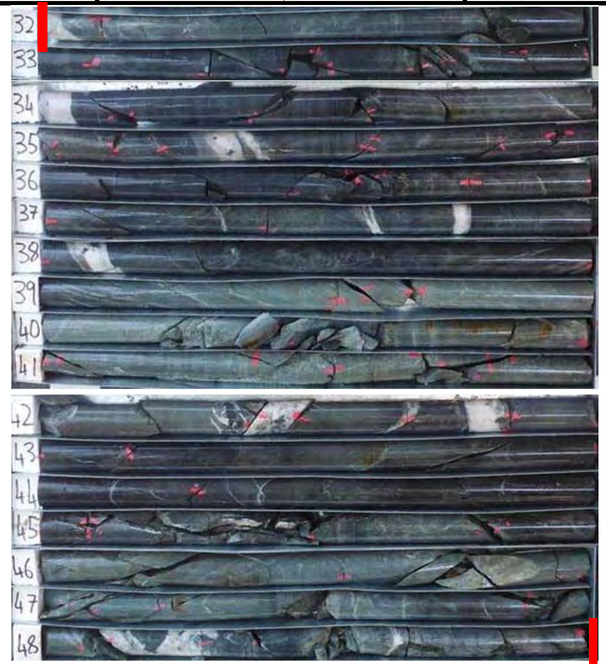
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	85	402	13.7	0.2	41%	0.4
5	85	402	13.2	0.2	41%	0.4
5	170	487	20.6	0.2	49%	0.5
5	170	487	21.3	0.3	49%	0.5
5	260	577	31.9	0.4	58%	0.7
5	260	577	31.0	0.4	58%	0.6
5	170	487	23.4	0.3	49%	0.6
5	170	487	22.6	0.3	49%	0.5
5	85	402	17.5	0.2	41%	0.5
5	85	402	16.4	0.2	41%	0.5





Interpreted flow mechanism: Laminar Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size: HQ3

Lithology: Argillite

Primary Defect Set: Joints

Lugeon (uL) Assessment

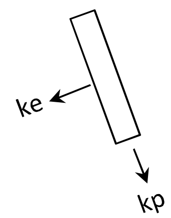
uL (Fell et al., 2005) - upper chart	1.1
uL (Houlsby, 1976) - lower chart	0.5

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	1.1.E-02	m/day
ke/kp = 100	9.5.E-03	m/day
ke/kp = 10	8.2.E-03	m/day
ke/kp = 1	6.8.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 7.2.E-03 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

BHH152 - 51m - 62m

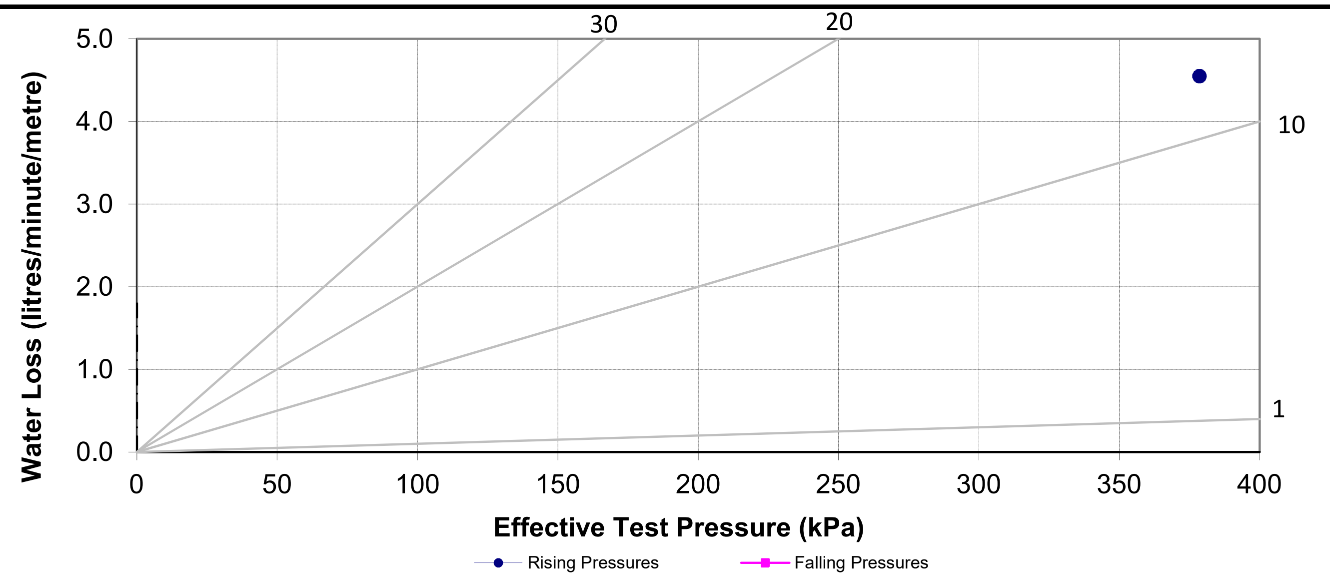
hole no: BHH152 | date: 13-Dec-16
project: Coffs Harbour Bypass
location: Gatelys Road | by: J Everitt

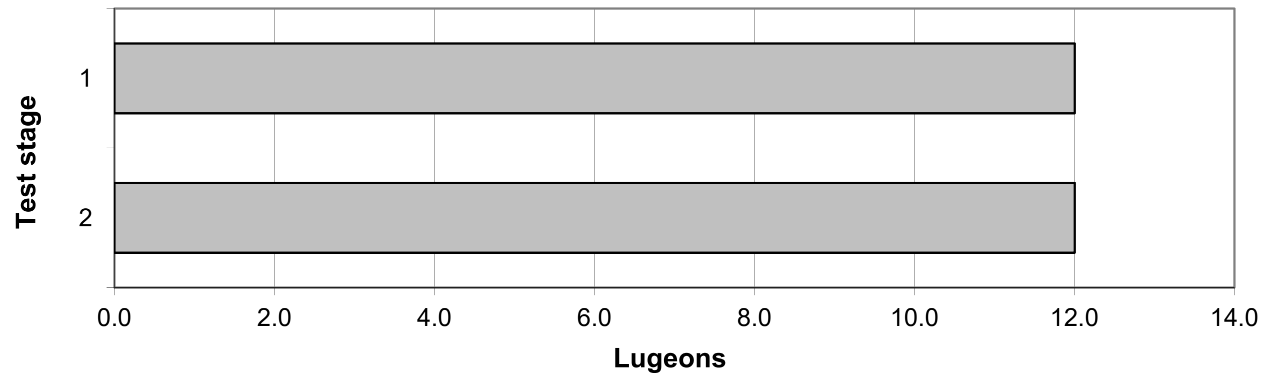
section tested: from 51.00 m to 62.00 m length 11.00 m

relative level at surface: 139.1 m RL | centre of test section (downhole): 56.5 m
gauge height: 0.9 m | test depth converted to vertical: 53.1 m
inclination of borehole: 70 ° | relative level at centre of test section: 86.0 m RL
inferred groundwater level: 108.5 m RL | correction to gauge pressure (1)(2): 308.7 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	70	379	250.0	4.5	27%	12.0
5	70	379	250.0	4.5	27%	12.0

70kPa maximum pressure achievable during test





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size: HQ3
Lithology: Argillite
Primary Defect Set: Joints

Lugeon (uL) Assessment

uL (Fell et al., 2005) - upper chart n/a

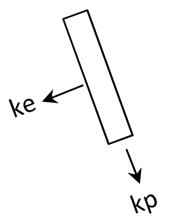
uL (Houlsby, 1976) - lower chart n/a

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	2.4.E-01	m/day
ke/kp = 100	2.1.E-01	m/day
ke/kp = 10	1.8.E-01	m/day
ke/kp = 1	1.5.E-01	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 1.5.E-01 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

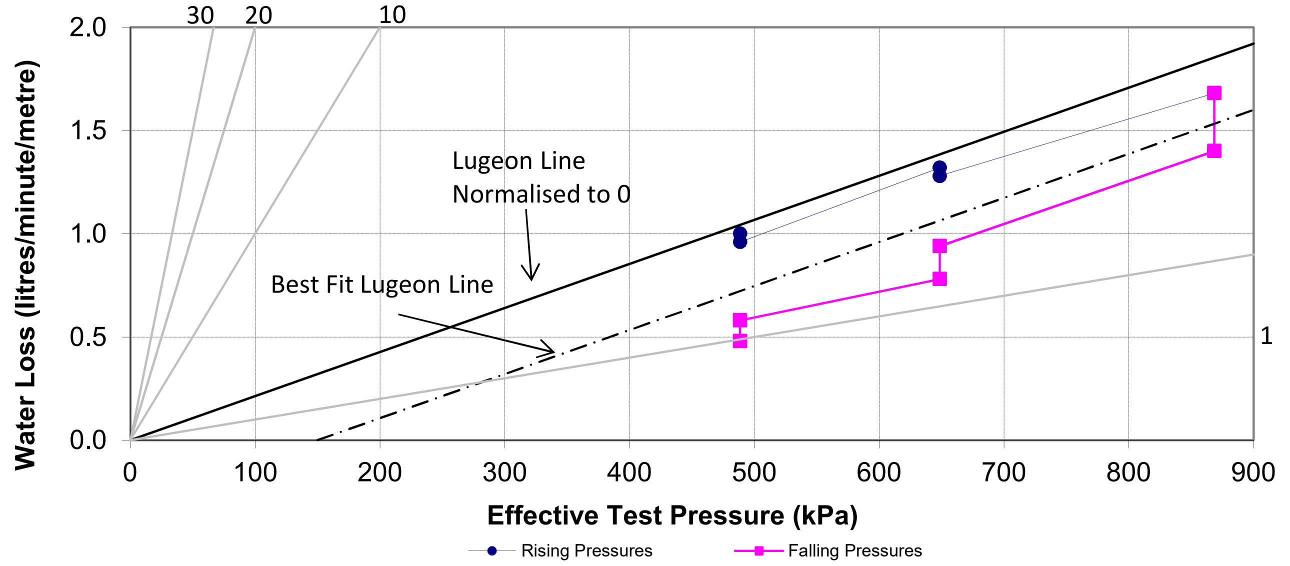
BHH152 - 57m - 62m

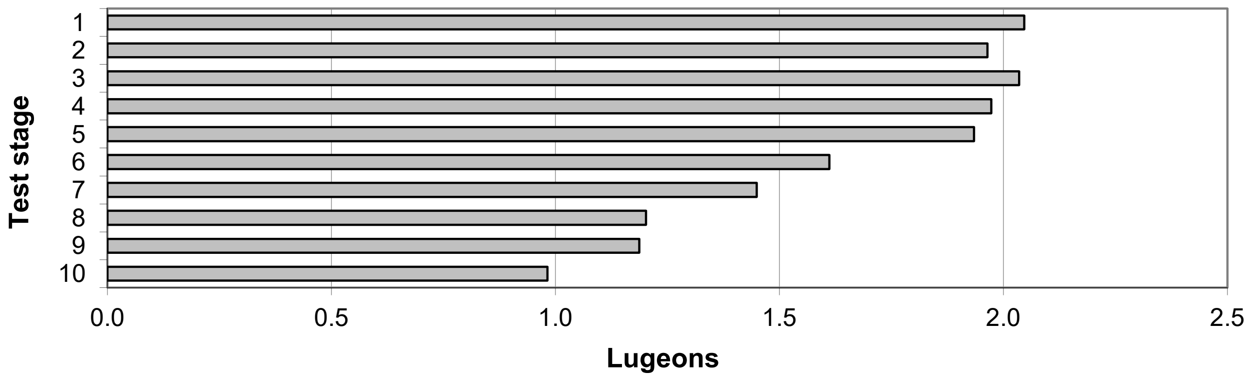
hole no:	BHH152	date:	13-Dec-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	J Everitt

section tested: from 57.00 m to 62.00 m length 5.00 m

relative level at surface:	139.1 m RL	centre of test section (downhole):	59.5 m
gauge height:	0.9 m	test depth converted to vertical:	55.9 m
inclination of borehole:	70 °	relative level at centre of test section:	83.2 m RL
inferred groundwater level:	108.5 m RL	correction to gauge pressure (1)(2):	308.7 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	180	489	25.0	1.0	34%	2.0
5	180	489	24.0	1.0	34%	2.0
5	340	649	33.0	1.3	45%	2.0
5	340	649	32.0	1.3	45%	2.0
5	560	869	42.0	1.7	60%	1.9
5	560	869	35.0	1.4	60%	1.6
5	340	649	23.5	0.9	45%	1.4
5	340	649	19.5	0.8	45%	1.2
5	180	489	14.5	0.6	34%	1.2
5	180	489	12.0	0.5	34%	1.0





Interpreted flow mechanism: Void filling

Image of packer test section:

LEGEND:

Packer zone boundary

Core size: HQ3

Lithology: Argillite

Primary Defect Set: Joints

Lugeon (uL) Assessment

uL (Fell et al., 2005) - upper chart 2.1

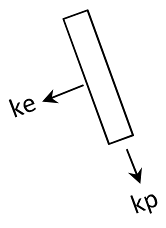
uL (Houlsby, 1976) - lower chart 1.1

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	2.0.E-02	m/day
ke/kp = 100	1.7.E-02	m/day
ke/kp = 10	1.4.E-02	m/day
ke/kp = 1	1.1.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 1.2.E-02 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

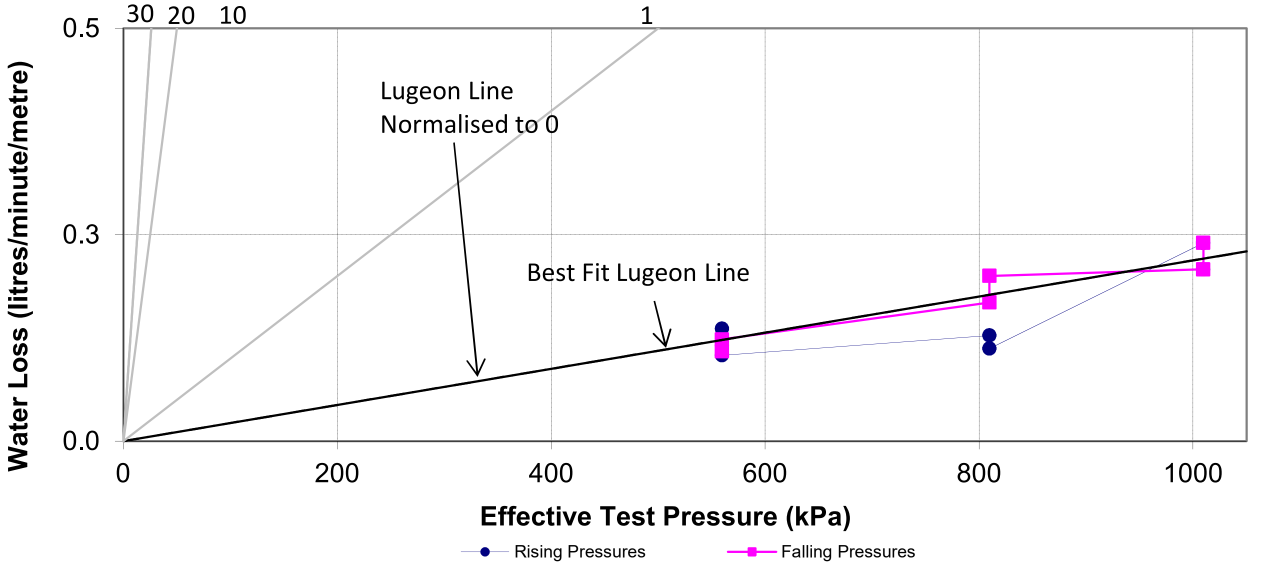
BHH152 - 62.26m - 74.76m

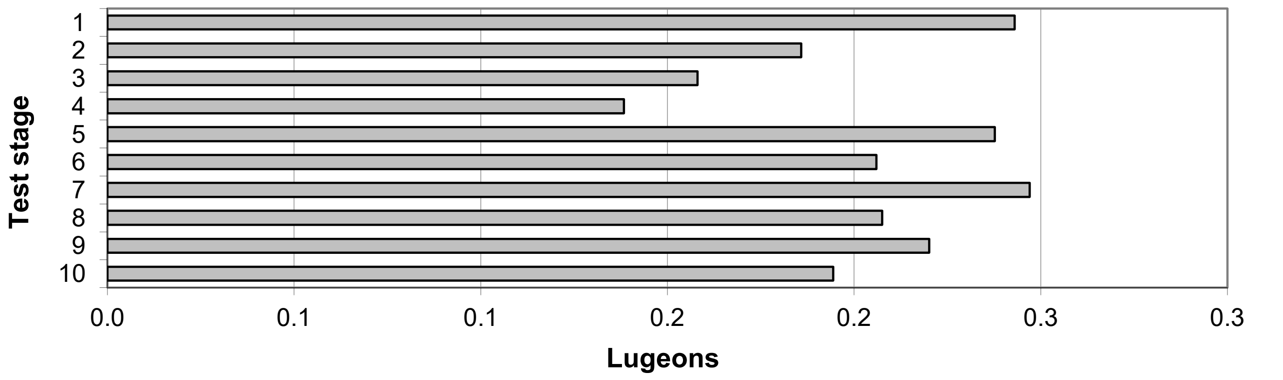
hole no:	BHH152	date:	15-Dec-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	J Everitt

section tested: from 62.26 m to 74.76 m length 12.50 m

relative level at surface:	139.1 m RL	centre of test section (downhole):	68.5 m
gauge height:	1.0 m	test depth converted to vertical:	64.4 m
inclination of borehole:	70 °	relative level at centre of test section:	74.7 m RL
inferred groundwater level:	108.5 m RL	correction to gauge pressure [(1)(2)]:	309.7 kPa

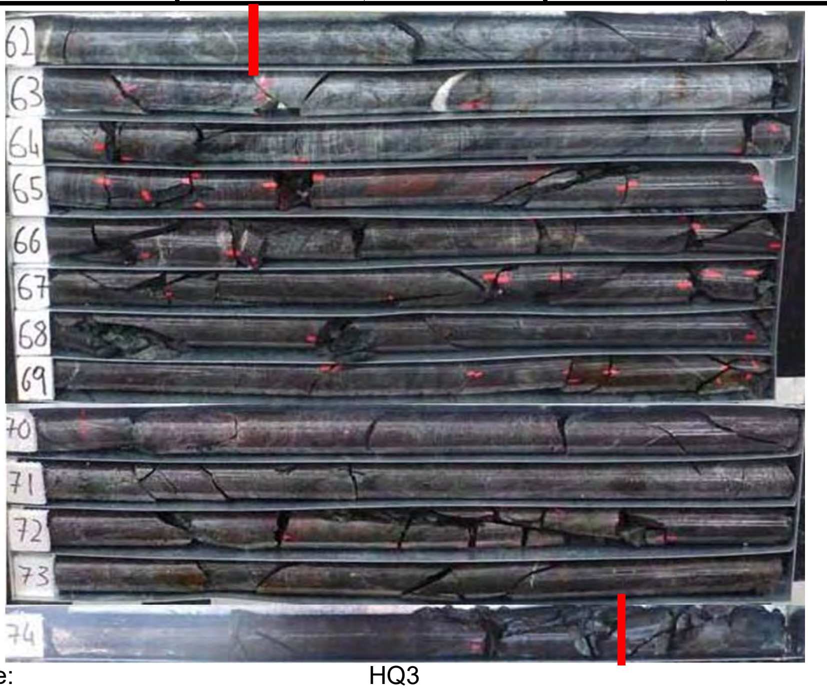
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure [(3)]	lugeons litres / min / m / 1000kPa
5	250	560	8.5	0.1	33%	0.2
5	250	560	6.5	0.1	33%	0.2
5	500	810	8.0	0.1	48%	0.2
5	500	810	7.0	0.1	48%	0.1
5	700	1010	15.0	0.2	60%	0.2
5	700	1010	13.0	0.2	60%	0.2
5	500	810	12.5	0.2	48%	0.2
5	500	810	10.5	0.2	48%	0.2
5	250	560	7.7	0.1	33%	0.2
5	250	560	6.8	0.1	33%	0.2





Interpreted flow mechanism: Laminar Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size: HQ3

Lithology: Argillite

Primary Defect Set: Joints

Lugeon (uL) Assessment

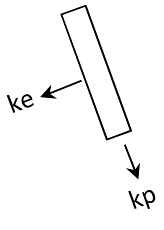
uL (Fell et al., 2005) - upper chart	0.2
uL (Houlsby, 1976) - lower chart	0.2

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	4.1.E-03	m/day
ke/kp = 100	3.6.E-03	m/day
ke/kp = 10	3.1.E-03	m/day
ke/kp = 1	2.5.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.7.E-03 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

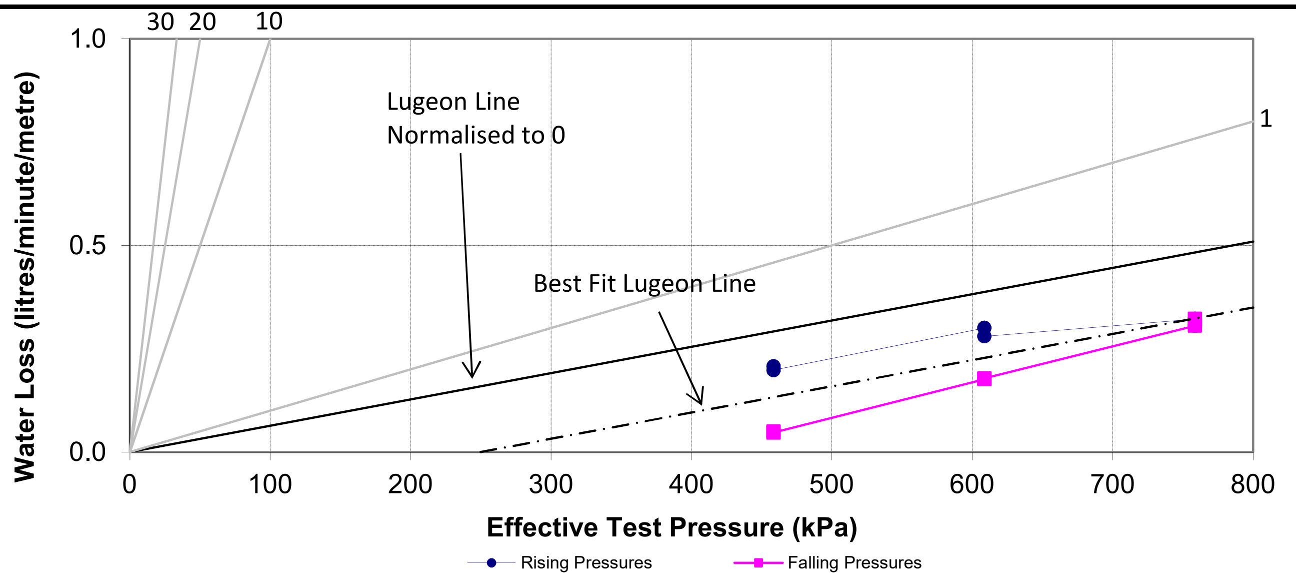
BHH152 - 74.96m - 85.96m

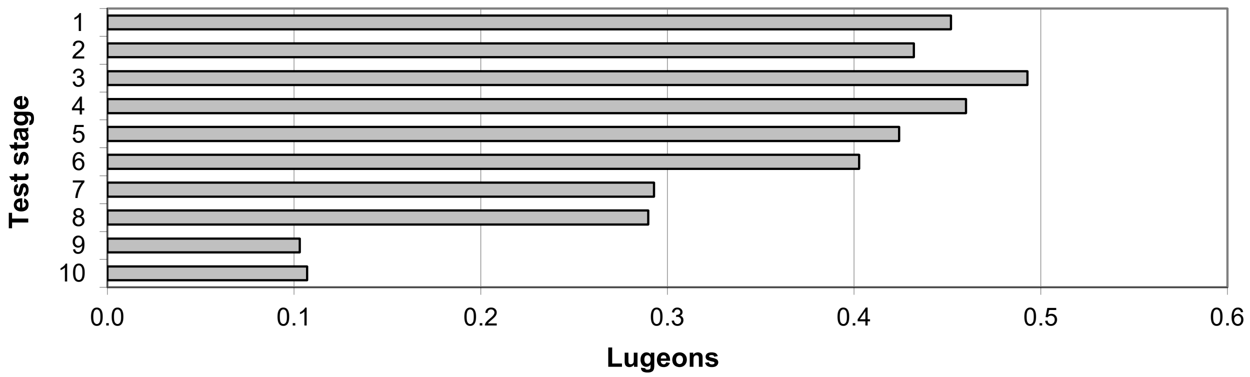
hole no:	BHH152	date:	19-Dec-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

section tested: from 74.96 m to 85.96 m length 11.00 m

relative level at surface:	139.1 m RL	centre of test section (downhole):	80.5 m
gauge height:	0.9 m	test depth converted to vertical:	75.6 m
inclination of borehole:	70 °	relative level at centre of test section:	63.5 m RL
inferred groundwater level:	108.5 m RL	correction to gauge pressure (1)(2):	308.7 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	459	11.4	0.2	23%	0.5
5	150	459	10.9	0.2	23%	0.4
5	300	609	16.5	0.3	31%	0.5
5	300	609	15.4	0.3	31%	0.5
5	450	759	17.7	0.3	39%	0.4
5	450	759	16.8	0.3	39%	0.4
5	300	609	9.8	0.2	31%	0.3
5	300	609	9.7	0.2	31%	0.3
5	150	459	2.6	0.0	23%	0.1
5	150	459	2.7	0.0	23%	0.1





Interpreted flow mechanism: Void Filling

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

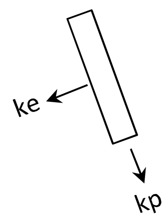
uL (Fell et al., 2005) - upper chart	0.6
uL (Houlsby, 1976) - lower chart	0.1

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	2.1.E-03	m/day
ke/kp = 100	1.8.E-03	m/day
ke/kp = 10	1.6.E-03	m/day
ke/kp = 1	1.3.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 1.4.E-03 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

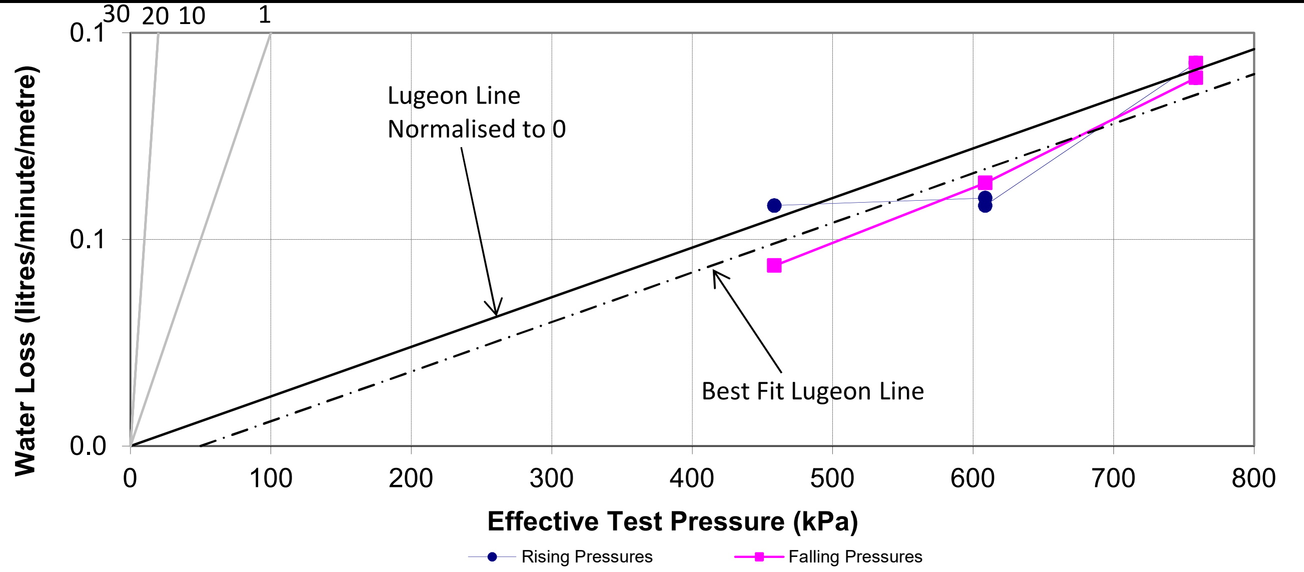
BHH152 - 87.2m - 98.2m

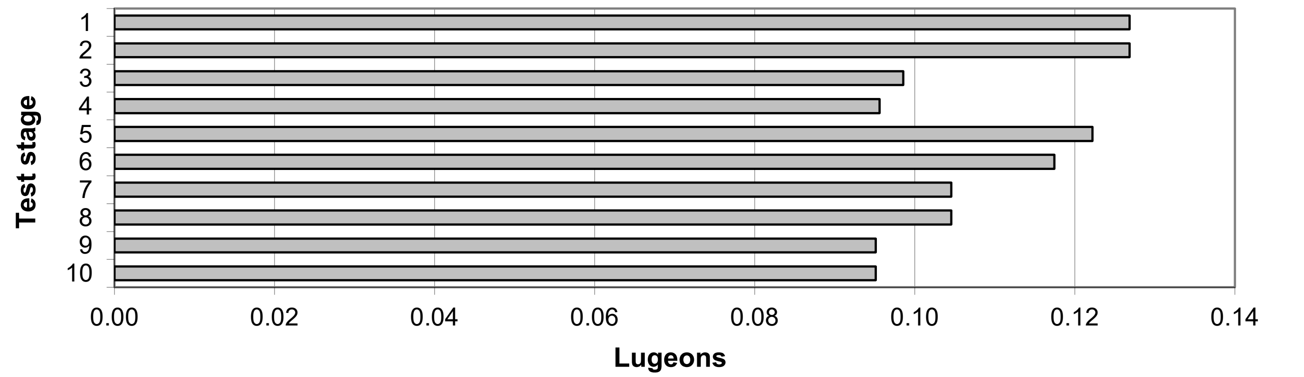
hole no:	BHH152	date:	21-Dec-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

section tested: from 87.20 m to 98.20 m length 11.00 m

relative level at surface:	139.1 m RL	centre of test section (downhole):	92.7 m
gauge height:	0.9 m	test depth converted to vertical:	87.1 m
inclination of borehole:	70 °	relative level at centre of test section:	52.0 m RL
inferred groundwater level:	108.5 m RL	correction to gauge pressure (1)(2):	308.7 kPa

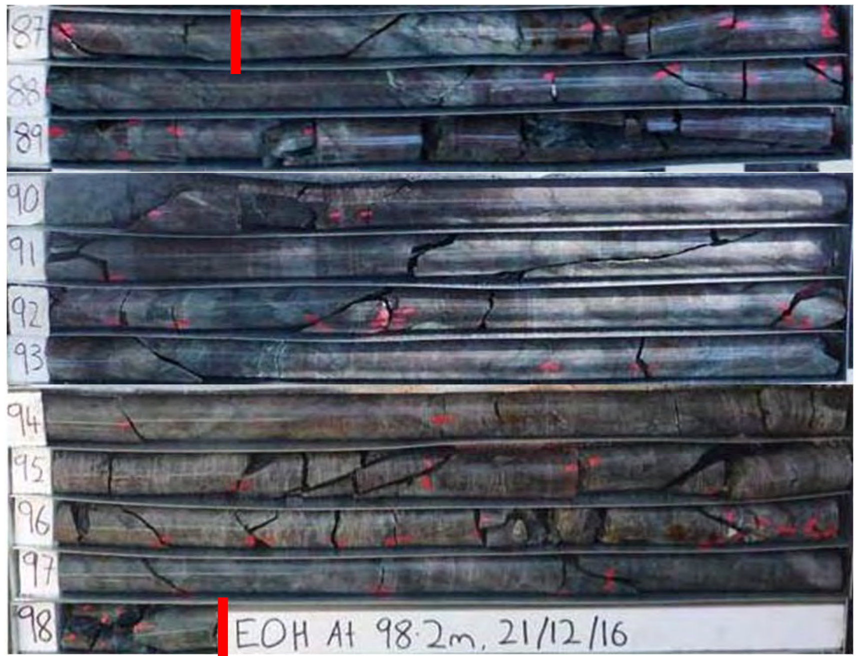
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	459	3.2	0.1	20%	0.13
5	150	459	3.2	0.1	20%	0.13
5	300	609	3.3	0.1	27%	0.10
5	300	609	3.2	0.1	27%	0.10
5	450	759	5.1	0.1	33%	0.12
5	450	759	4.9	0.1	33%	0.12
5	300	609	3.5	0.1	27%	0.10
5	300	609	3.5	0.1	27%	0.10
5	150	459	2.4	0.0	20%	0.10
5	150	459	2.4	0.0	20%	0.10





Interpreted flow mechanism: Laminar Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size: HQ3

Lithology: Argillite

Primary Defect Set: Joints

Lugeon (uL) Assessment

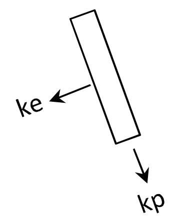
uL (Fell et al., 2005) - upper chart	0.12
uL (Houlsby, 1976) - lower chart	0.11

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	2.2.E-03	m/day
ke/kp = 100	1.9.E-03	m/day
ke/kp = 10	1.6.E-03	m/day
ke/kp = 1	1.3.E-03	m/day
ke value using Moye Method (1967) - equivalent to ke/kp = 1:	1.4.E-03	m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Test undertaken above water table

Job no. PSM2876

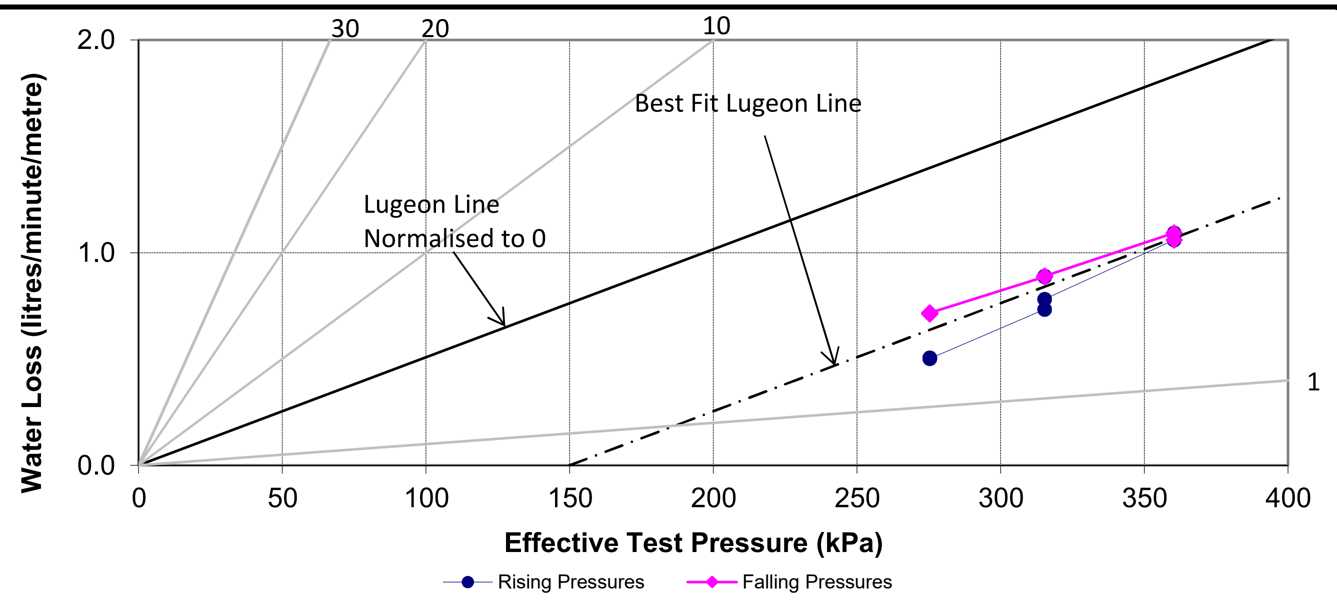
BHH153 - 16m - 27m

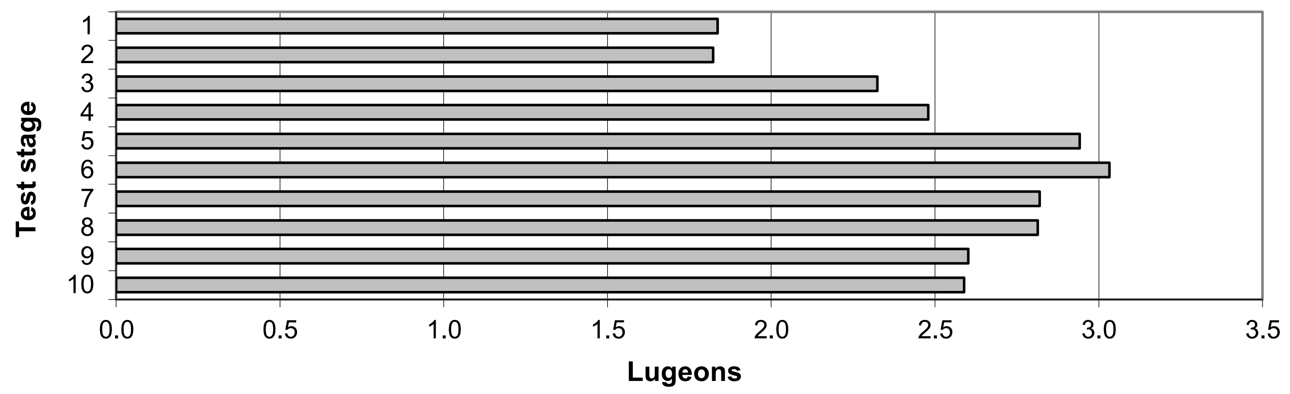
hole no:	BHH153	date:	29-Nov-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

section tested: from 16.0 m to 27.0 m length 11.0 m

relative level at surface:	136.2 m RL	centre of test section (downhole):	21.5 m
gauge height:	2.0 m	test depth converted to vertical:	21.5 m
inclination of borehole:	90 °	relative level at centre of test section:	114.7 m RL
inferred groundwater level:	106.9 m RL	correction to gauge pressure (1)(2):	230.3 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	45	275	27.8	0.5	49%	1.8
5	45	275	27.6	0.5	49%	1.8
5	85	315	40.3	0.7	56%	2.3
5	85	315	43.0	0.8	56%	2.5
5	130	360	58.3	1.1	64%	2.9
5	130	360	60.1	1.1	64%	3.0
5	85	315	48.9	0.9	56%	2.8
5	85	315	48.8	0.9	56%	2.8
5	45	275	39.4	0.7	49%	2.6
5	45	275	39.2	0.7	49%	2.6





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size: HQ3

Lithology: Argillite

Primary Defect Set: Joints / No Core

Lugeon (uL) Assessment

uL (Fell et al., 2005) - upper chart 5.1

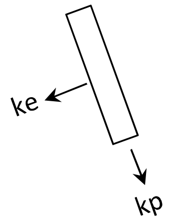
uL (Houlsby, 1976) - lower chart 2.5

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	5.0.E-02	m/day
ke/kp = 100	4.4.E-02	m/day
ke/kp = 10	3.7.E-02	m/day
ke/kp = 1	3.1.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 3.3.E-02 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Job no. PSM2876

Borehole Water Pressure Test Summary

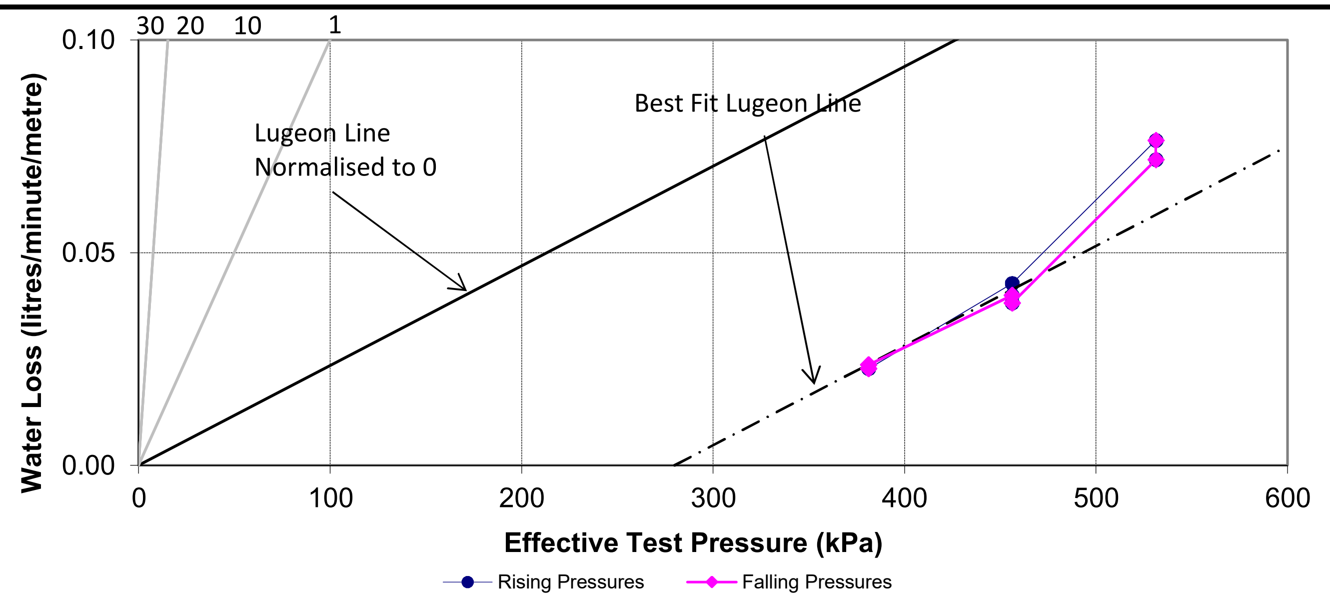
BHH153 - 27.07m - 38.07m

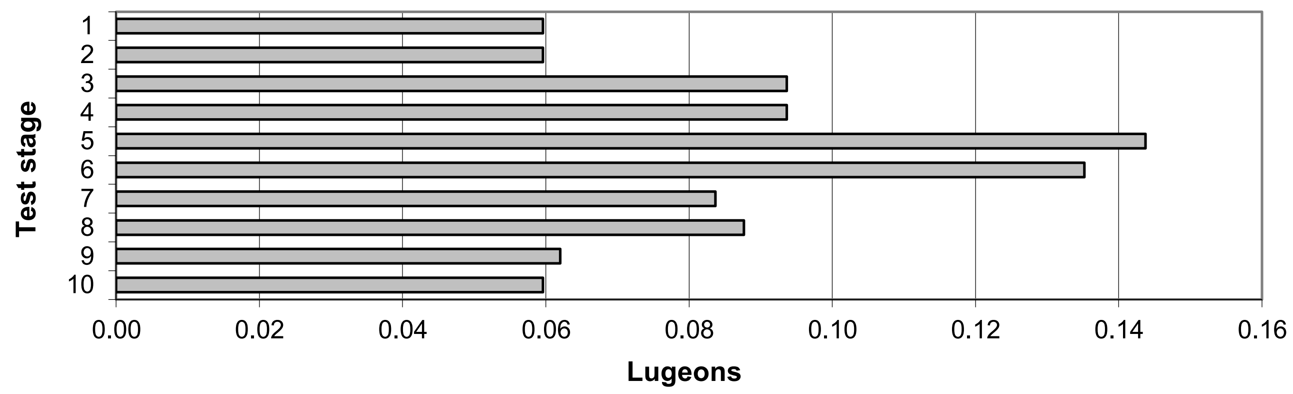
hole no:	BHH153	date:	1-Dec-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

section tested: from 27.1 m to 38.1 m length 11.0 m

relative level at surface:	136.2 m RL	centre of test section (downhole):	32.6 m
gauge height:	2.0 m	test depth converted to vertical:	32.6 m
inclination of borehole:	90 °	relative level at centre of test section:	103.6 m RL
inferred groundwater level:	106.9 m RL	correction to gauge pressure (1)(2):	306.3 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	75	381	1.3	0.0	45%	0.06
5	75	381	1.3	0.0	45%	0.06
5	150	456	2.4	0.0	54%	0.09
5	150	456	2.4	0.0	54%	0.09
5	225	531	4.2	0.1	63%	0.14
5	225	531	4.0	0.1	63%	0.14
5	150	456	2.1	0.0	54%	0.08
5	150	456	2.2	0.0	54%	0.09
5	75	381	1.3	0.0	45%	0.06
5	75	381	1.3	0.0	45%	0.06





Interpreted flow mechanism: Dilation

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

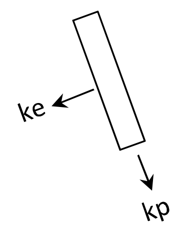
uL (Fell et al., 2005) - upper chart	0.23
uL (Houlsby, 1976) - lower chart	0.07

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	1.5.E-03	m/day
ke/kp = 100	1.3.E-03	m/day
ke/kp = 10	1.1.E-03	m/day
ke/kp = 1	9.1.E-04	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 9.7.E-04 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink Job no. PSM2876

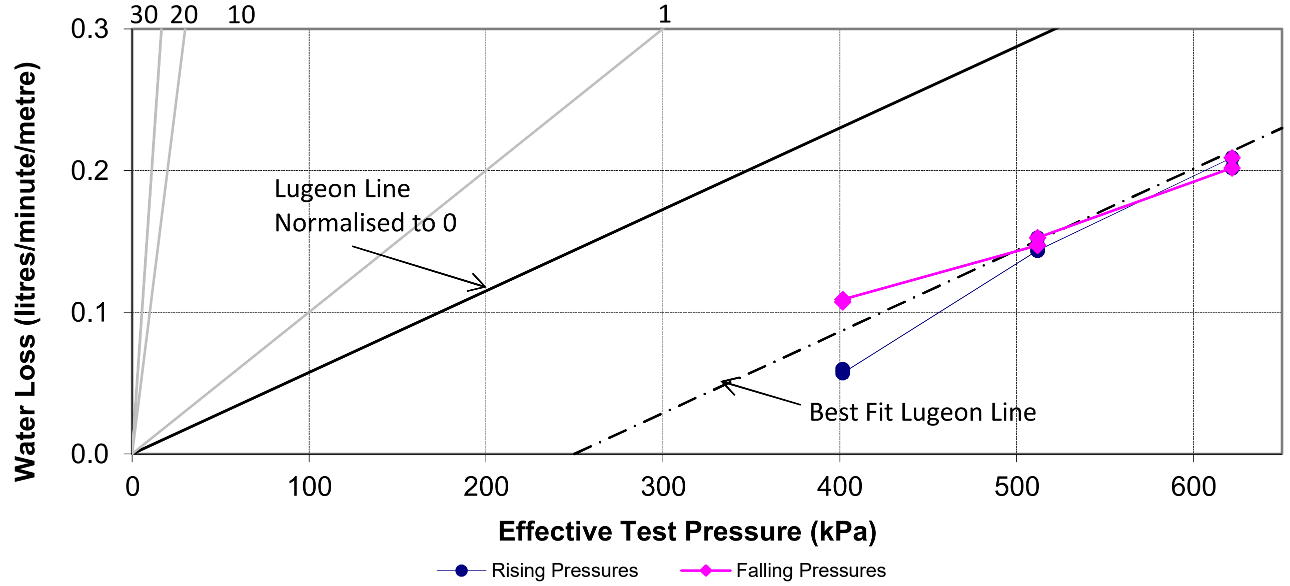
Borehole Water Pressure Test Summary **BHH153 - 39m - 50m**

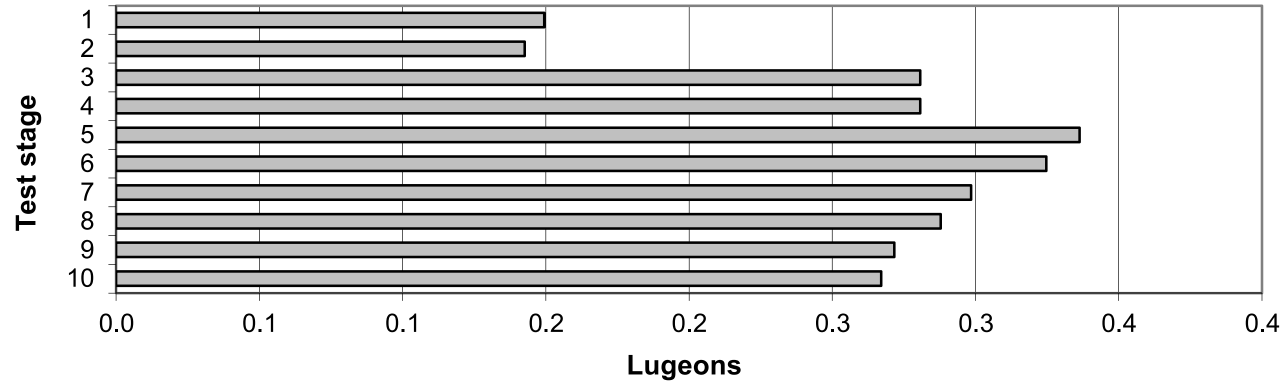
hole no:	BHH153	date:	1-Dec-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

section tested: from 39.0 m to 50.0 m length 11.0 m

relative level at surface:	136.2 m RL	centre of test section (downhole):	44.5 m
gauge height:	0.0 m	test depth converted to vertical:	44.5 m
inclination of borehole:	90 °	relative level at centre of test section:	91.7 m RL
inferred groundwater level:	106.9 m RL	correction to gauge pressure (1)(2):	286.7 kPa

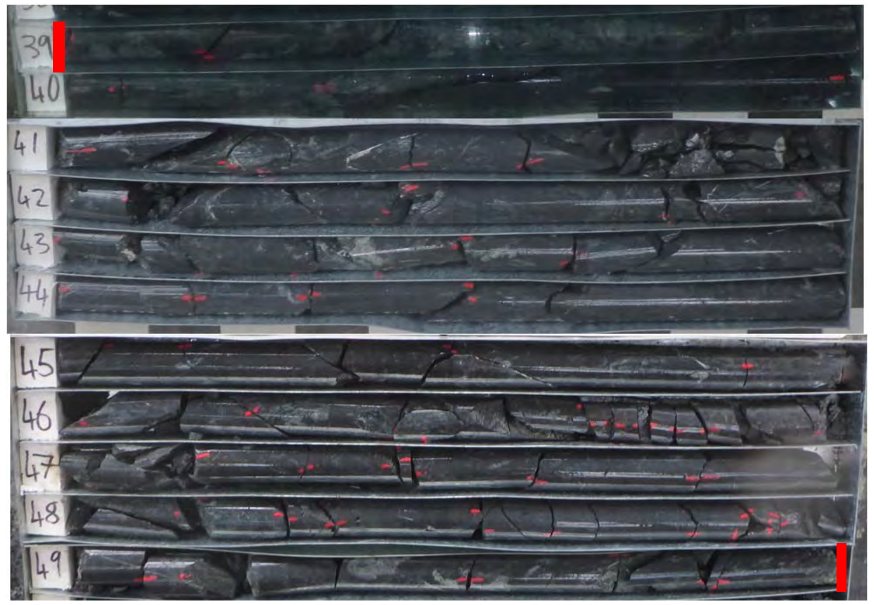
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	115	402	3.3	0.1	35%	0.1
5	115	402	3.2	0.1	35%	0.1
5	225	512	7.9	0.1	44%	0.3
5	225	512	7.9	0.1	44%	0.3
5	335	622	11.5	0.2	54%	0.3
5	335	622	11.1	0.2	54%	0.3
5	225	512	8.4	0.2	44%	0.3
5	225	512	8.1	0.1	44%	0.3
5	115	402	6.0	0.1	35%	0.3
5	115	402	5.9	0.1	35%	0.3





Interpreted flow mechanism: Laminar Flow

Image of packer test section:



LEGEND:

Packer zone boundary

Core size: HQ3
Lithology: Argillite
Primary Defect Set: Joints

Lugeon (uL) Assessment

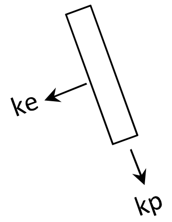
uL (Fell et al., 2005) - upper chart	0.6
uL (Houlsby, 1976) - lower chart	0.3

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	5.3.E-03	m/day
ke/kp = 100	4.6.E-03	m/day
ke/kp = 10	3.9.E-03	m/day
ke/kp = 1	3.2.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 3.4.E-03 m/day

Notes 1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

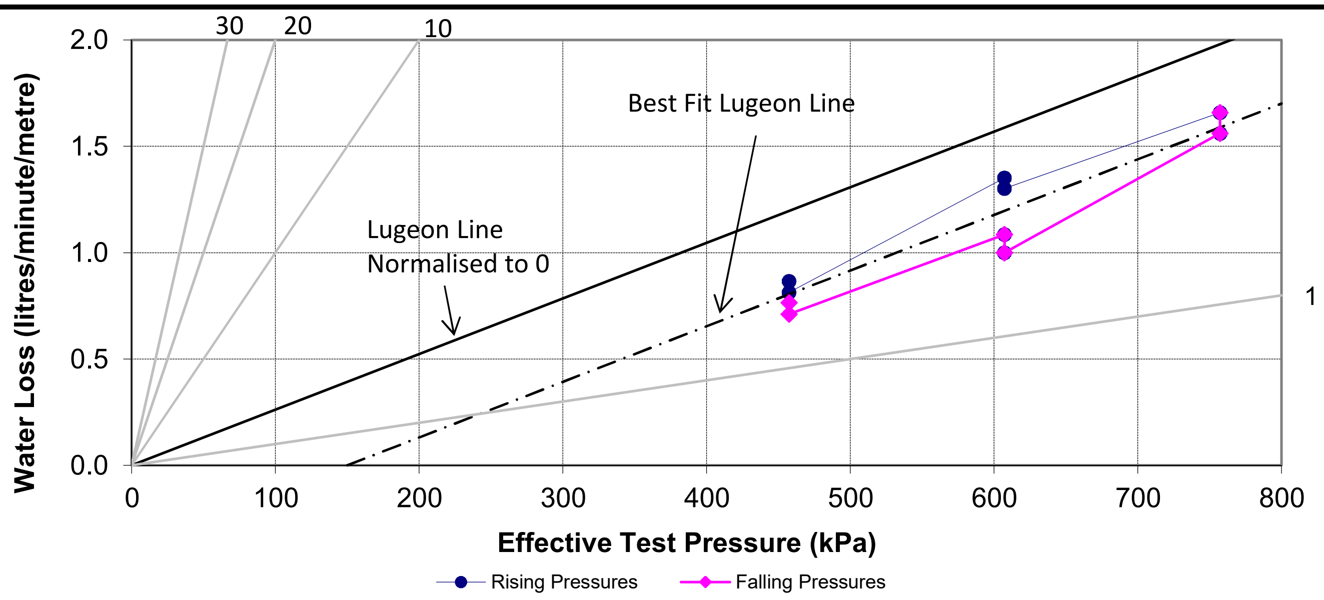
BHH153 - 51m - 62m

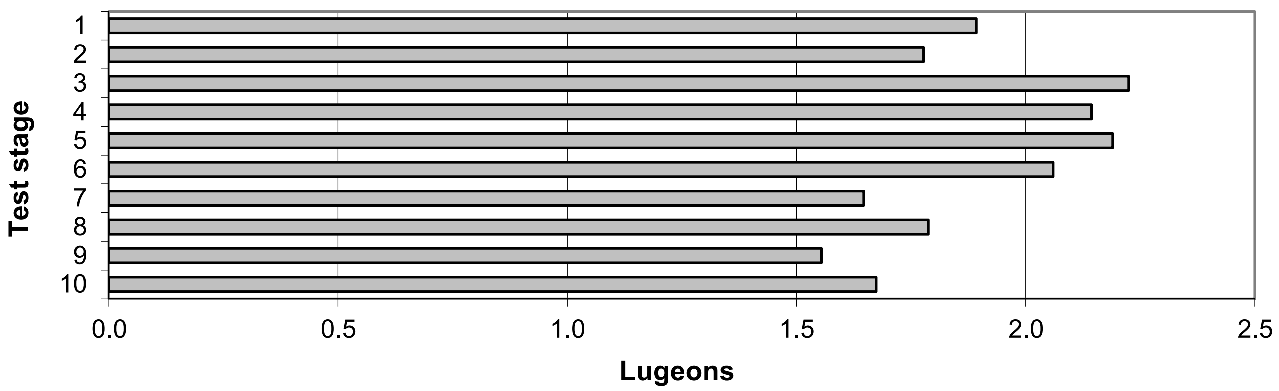
hole no:	BHH153	date:	2-Dec-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

section tested: from 51.0 m to 62.0 m length 11.0 m

relative level at surface:	136.2 m RL	centre of test section (downhole):	56.5 m
gauge height:	2.1 m	test depth converted to vertical:	56.5 m
inclination of borehole:	90 °	relative level at centre of test section:	79.7 m RL
inferred groundwater level:	106.9 m RL	correction to gauge pressure (1)(2):	307.2 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	457	47.6	0.9	31%	1.9
5	150	457	44.7	0.8	31%	1.8
5	300	607	74.3	1.4	41%	2.2
5	300	607	71.6	1.3	41%	2.1
5	450	757	91.2	1.7	52%	2.2
5	450	757	85.8	1.6	52%	2.1
5	300	607	55.0	1.0	41%	1.6
5	300	607	59.7	1.1	41%	1.8
5	150	457	39.1	0.7	31%	1.6
5	150	457	42.1	0.8	31%	1.7





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

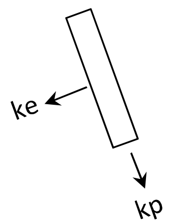
uL (Fell et al., 2005) - upper chart	2.6
uL (Houlsby, 1976) - lower chart	1.9

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	3.8.E-02	m/day
ke/kp = 100	3.3.E-02	m/day
ke/kp = 10	2.8.E-02	m/day
ke/kp = 1	2.3.E-02	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 2.4.E-02 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink — Job no. PSM2876

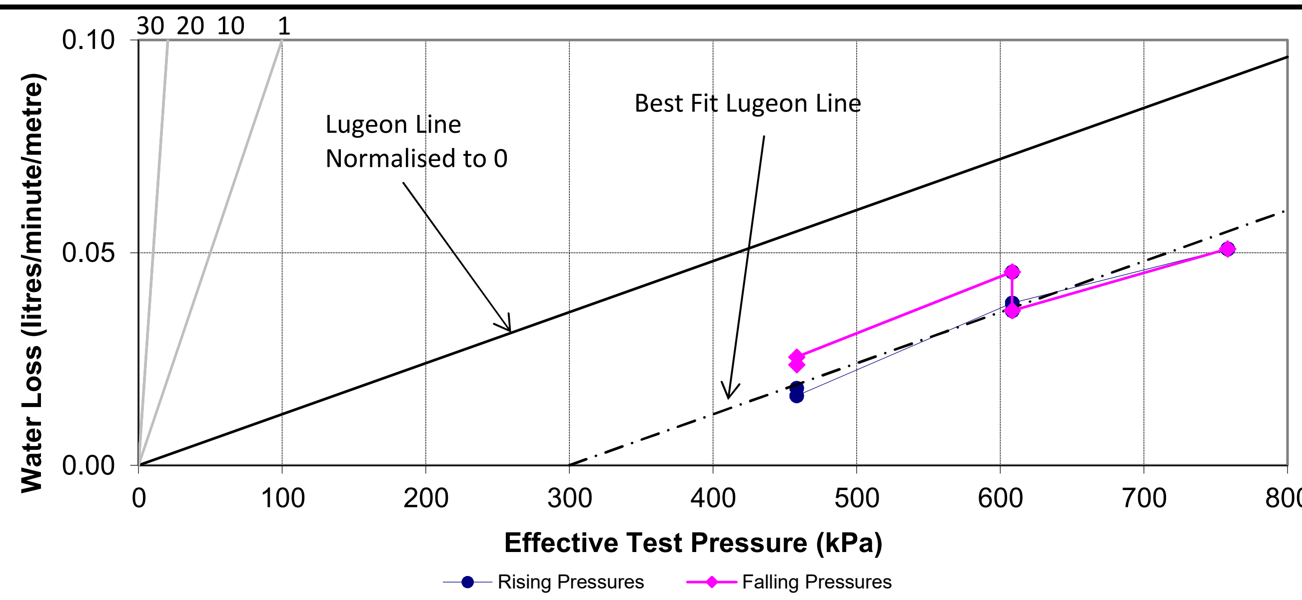
Borehole Water Pressure Test Summary — **BHH153 - 63m - 74m**

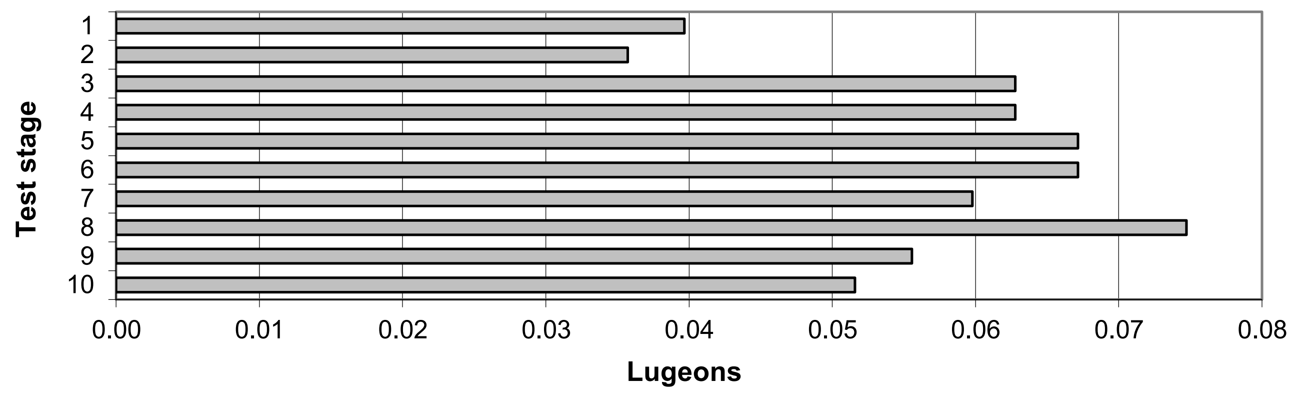
hole no:	BHH153	date:	2-Dec-16
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	D Cooper

section tested: from 63.0 m to 74.0 m length 11.0 m

relative level at surface:	136.2 m RL	centre of test section (downhole):	68.5 m
gauge height:	2.2 m	test depth converted to vertical:	68.5 m
inclination of borehole:	90 °	relative level at centre of test section:	67.7 m RL
inferred groundwater level:	106.9 m RL	correction to gauge pressure (1)(2):	308.2 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	150	458	1.0	0.0	26%	0.04
5	150	458	0.9	0.0	26%	0.04
5	300	608	2.1	0.0	34%	0.06
5	300	608	2.1	0.0	34%	0.06
5	450	758	2.8	0.1	43%	0.07
5	450	758	2.8	0.1	43%	0.07
5	300	608	2.0	0.0	34%	0.06
5	300	608	2.5	0.0	34%	0.07
5	150	458	1.4	0.0	26%	0.06
5	150	458	1.3	0.0	26%	0.05





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints

Lugeon (uL) Assessment

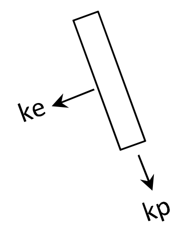
uL (Fell et al., 2005) - upper chart	0.12
uL (Houlsby, 1976) - lower chart	0.06

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	1.2.E-03	m/day
ke/kp = 100	1.0.E-03	m/day
ke/kp = 10	8.5.E-04	m/day
ke/kp = 1	7.0.E-04	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 7.4.E-04 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink

Borehole Water Pressure Test Summary

Job no. PSM2876

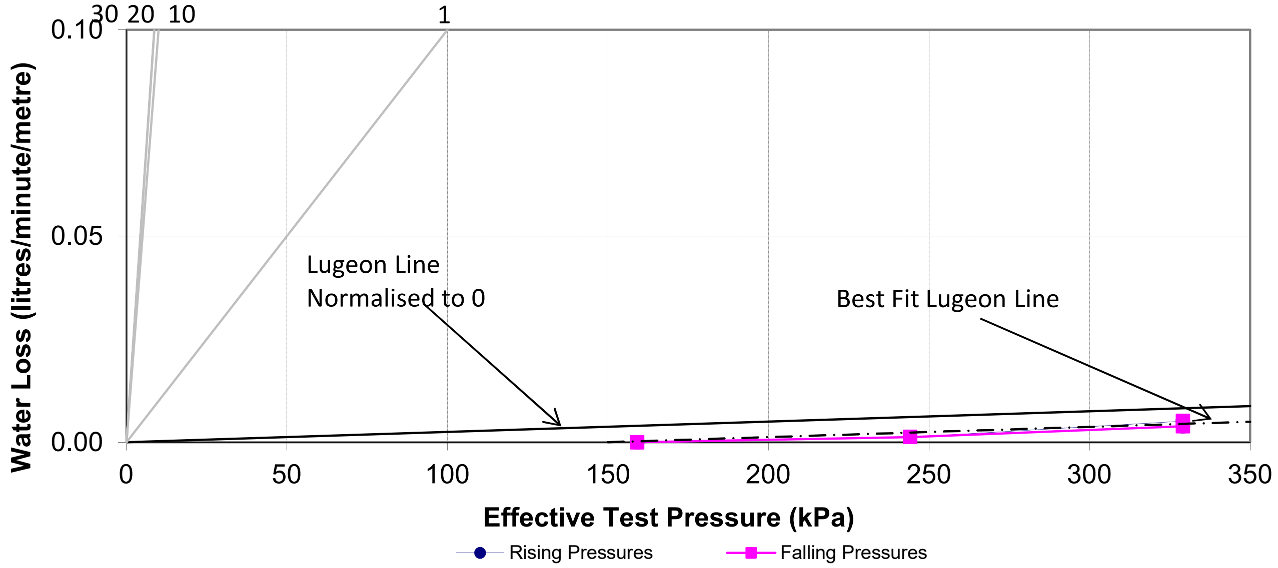
BHH154 - 29m - 44.55m

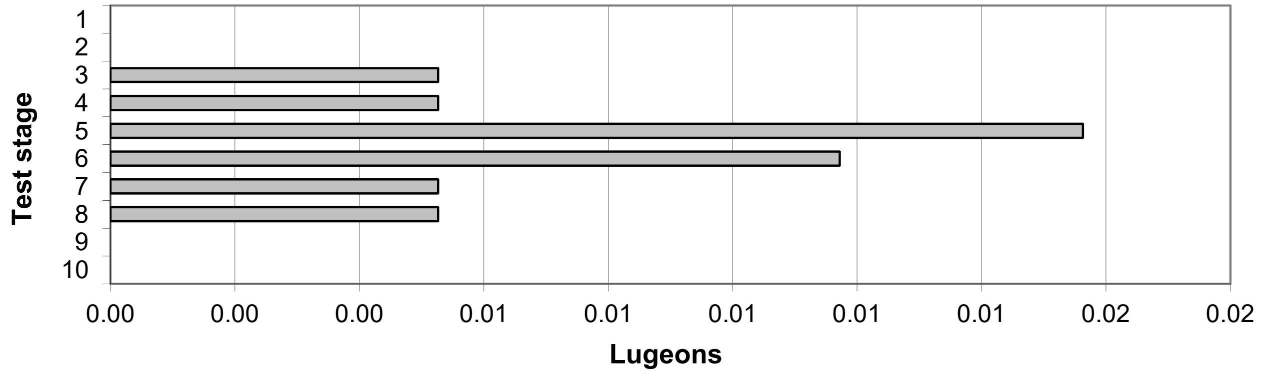
hole no:	BHH154	date:	17-Jan-17
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	T Hosking

section tested: from 29.00 m to 44.55 m length 15.55 m

relative level at surface:	86.8 m RL	centre of test section (downhole):	36.8 m
gauge height:	1.0 m	test depth converted to vertical:	36.8 m
inclination of borehole:	90 °	relative level at centre of test section:	50.0 m RL
inferred groundwater level:	80.3 m RL	correction to gauge pressure (1)(2):	74.2 kPa

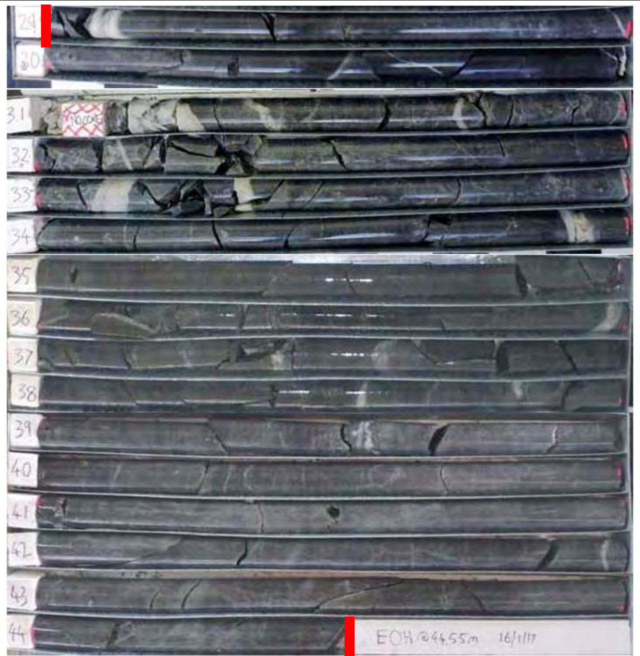
test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	85	159	0.0	0.0	17%	0.00
5	85	159	0.0	0.0	17%	0.00
5	170	244	0.1	0.0	26%	0.01
5	170	244	0.1	0.0	26%	0.01
5	255	329	0.4	0.0	34%	0.02
5	255	329	0.3	0.0	34%	0.01
5	170	244	0.1	0.0	26%	0.01
5	170	244	0.1	0.0	26%	0.01
5	85	159	0.0	0.0	17%	0.00
5	85	159	0.0	0.0	17%	0.00





Interpreted flow mechanism: Dilation

Image of packer test section:



LEGEND:

Packer zone boundary

Core size:	HQ3
Lithology:	Argillite
Primary Defect Set:	Joints / No core

Lugeon (uL) Assessment

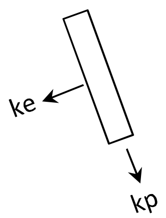
uL (Fell et al., 2005) - upper chart	0.03
uL (Houlsby, 1976) - lower chart	0.00

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	5.5.E-05	m/day
ke/kp = 100	4.8.E-05	m/day
ke/kp = 10	4.1.E-05	m/day
ke/kp = 1	3.4.E-05	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 3.6.E-05 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

Pells Sullivan Meynink | Job no. PSM2876

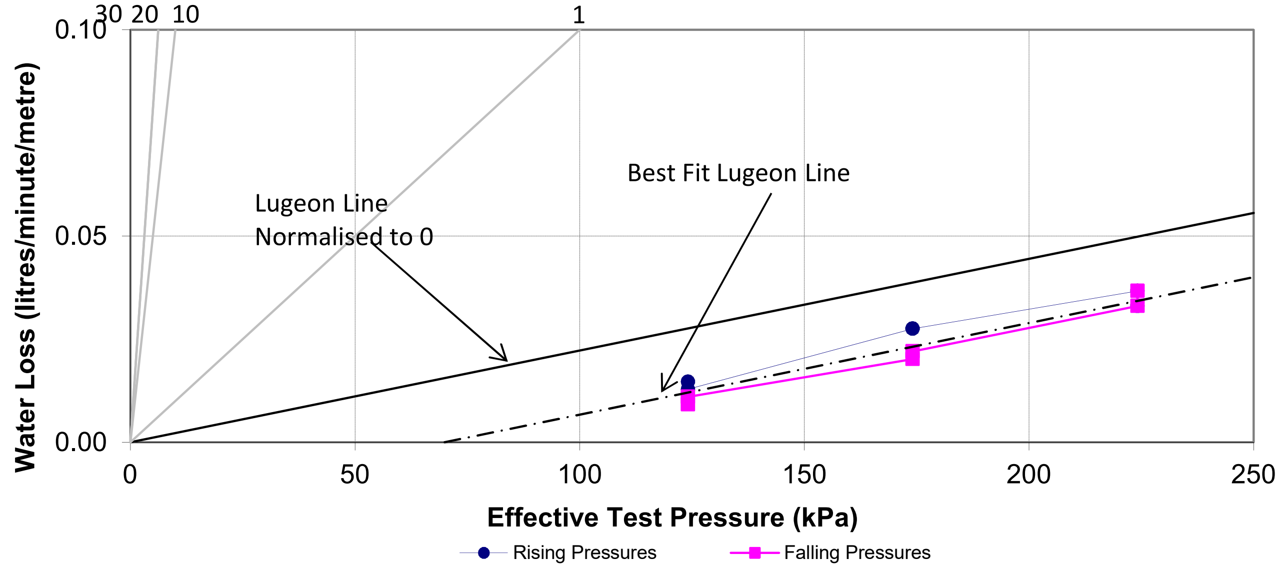
Borehole Water Pressure Test Summary | **BHH154 - 17m - 27.9m**

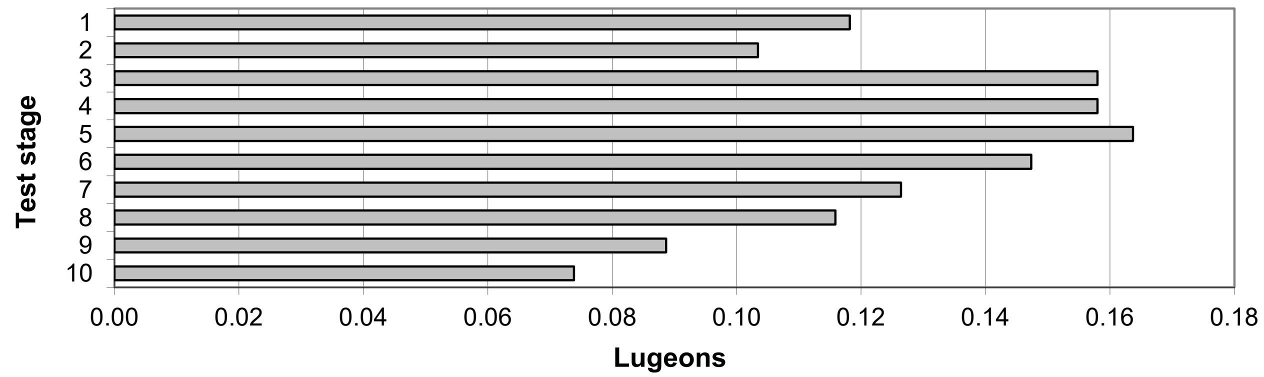
hole no:	BHH154	date:	12-Jan-17
project:	Coffs Harbour Bypass		
location:	Gatelys Road	by:	T Hosking

section tested: from 17.00 m to 27.90 m length 10.90 m

relative level at surface:	86.8 m RL	centre of test section (downhole):	22.5 m
gauge height:	1.0 m	test depth converted to vertical:	22.5 m
inclination of borehole:	90 °	relative level at centre of test section:	64.4 m RL
inferred groundwater level:	80.3 m RL	correction to gauge pressure (1)(2):	74.2 kPa

test duration (min)	gauge pressure (kPa)	corrected test pressure (kPa)	average water loss (L)	water loss litres/min/m	corrected / overburden pressure (3)	lugeons litres / min / m / 1000kPa
5	50	124	0.8	0.0	21%	0.12
5	50	124	0.7	0.0	21%	0.10
5	100	174	1.5	0.0	30%	0.16
5	100	174	1.5	0.0	30%	0.16
5	150	224	2.0	0.0	38%	0.16
5	150	224	1.8	0.0	38%	0.15
5	100	174	1.2	0.0	30%	0.13
5	100	174	1.1	0.0	30%	0.12
5	50	124	0.6	0.0	21%	0.09
5	50	124	0.5	0.0	21%	0.07





Interpreted flow mechanism: Laminar Flow

Image of packer test section:

LEGEND:

Packer zone boundary

Core size: HQ3
Lithology: Argillite
Primary Defect Set: Joints

Lugeon (uL) Assessment

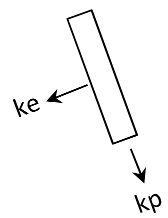
uL (Fell et al., 2005) - upper chart	0.22
uL (Houlsby, 1976) - lower chart	0.13

Lugeon Classification

Lugeon	Range	Condition
<1	Low	Joints tight
1-5	Low/Mod	Small joint openings
5-50	Mod/High	Some open joints
>50	High	Many open joints

Estimated permeability (ke):

ke value using Hoek & Bray Method (1981) for:



ke/kp = 1000	2.5.E-03	m/day
ke/kp = 100	2.2.E-03	m/day
ke/kp = 10	1.9.E-03	m/day
ke/kp = 1	1.5.E-03	m/day

ke value using Moye Method (1967) - equivalent to ke/kp = 1: 1.6.E-03 m/day

Notes
1 - If the inferred groundwater level is above the centre of the test section, the correction is taken as the head difference between the groundwater level and the gauge height.
2 - If the inferred groundwater level is below the centre of test section, the correction is taken as the head difference between the centre of the test section and the gauge height
3 - Ratio of corrected test pressure to overburden pressure is taken relative to the centre of test section

A4 – Hydraulic Conductivity Summary Tables

A4 – Hydraulic Conductivity Summary Tables

RMU	Hydraulic Conductivity (m/day)		
	Minimum	Geometric Mean	Max
Soil	0.03		0.4
RMU-A	0.03		0.4
RMU-B1 (MW)	NA	NA	NA
RMU-B1 (SW)	1.1×10^{-5}	9.4×10^{-4}	2.2×10^{-2}
RMU-B2	2.0×10^{-4}	5.9×10^{-4}	5.4×10^{-3}
RMU-C1 (MW)	1.3×10^{-2}	1.4×10^{-2}	1.5×10^{-2}
RMU-C1 (SW)	1.8×10^{-3}	7.3×10^{-3}	5.6×10^{-2}
RMU-C2	3.6×10^{-5}	2.2×10^{-3}	4.9×10^{-2}
RMU-D1 (MW)	3.3×10^{-2}	1.4×10^{-1}	4.3×10^{-1}
RMU-D1 (SW)	9.7×10^{-4}	3.4×10^{-2}	4.8×10^{-1}
RMU-D2	3.9×10^{-4}	6.1×10^{-3}	1.8×10^{-1}

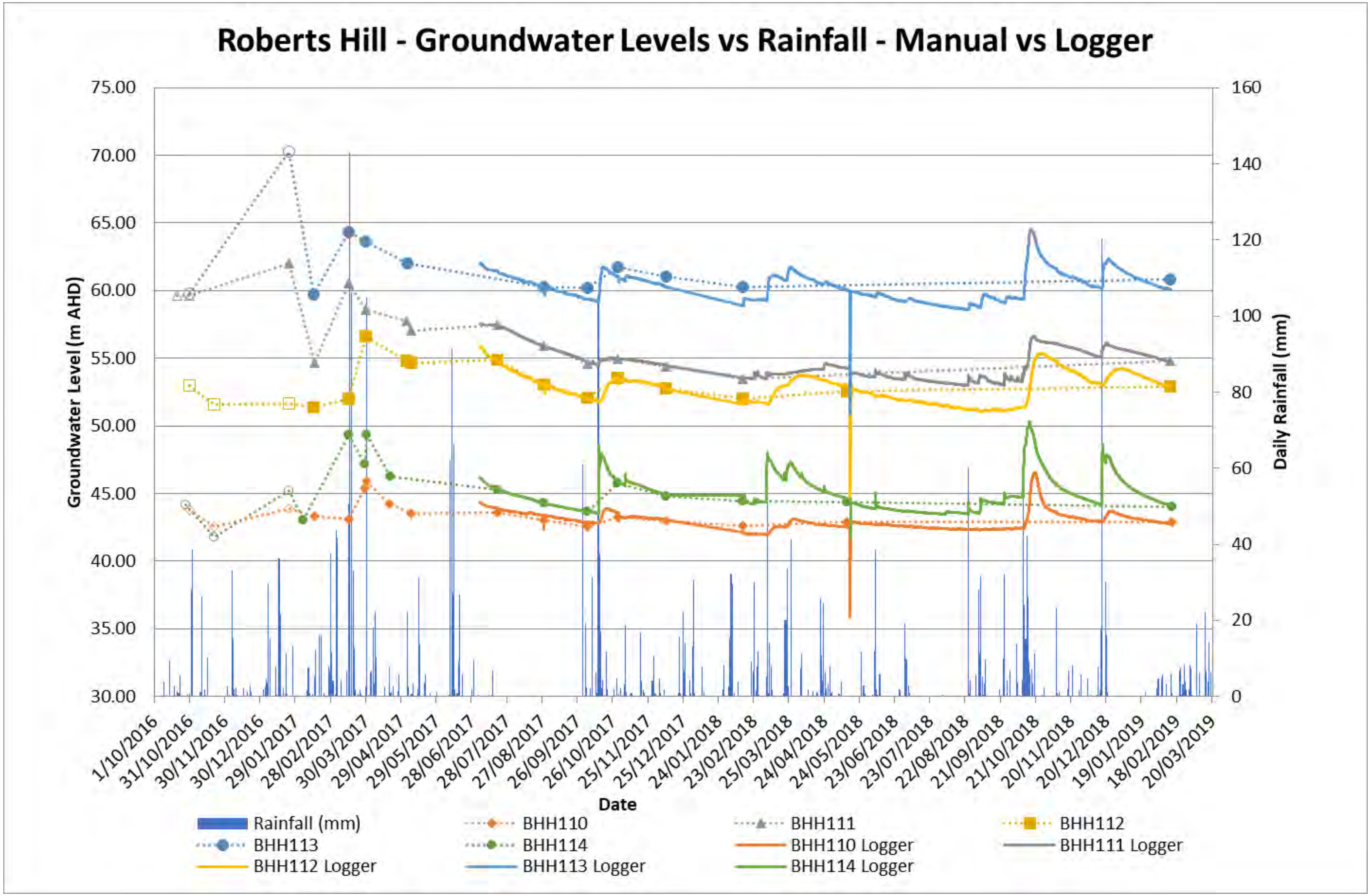
Geometric Mean Hydraulic Conductivity by Site

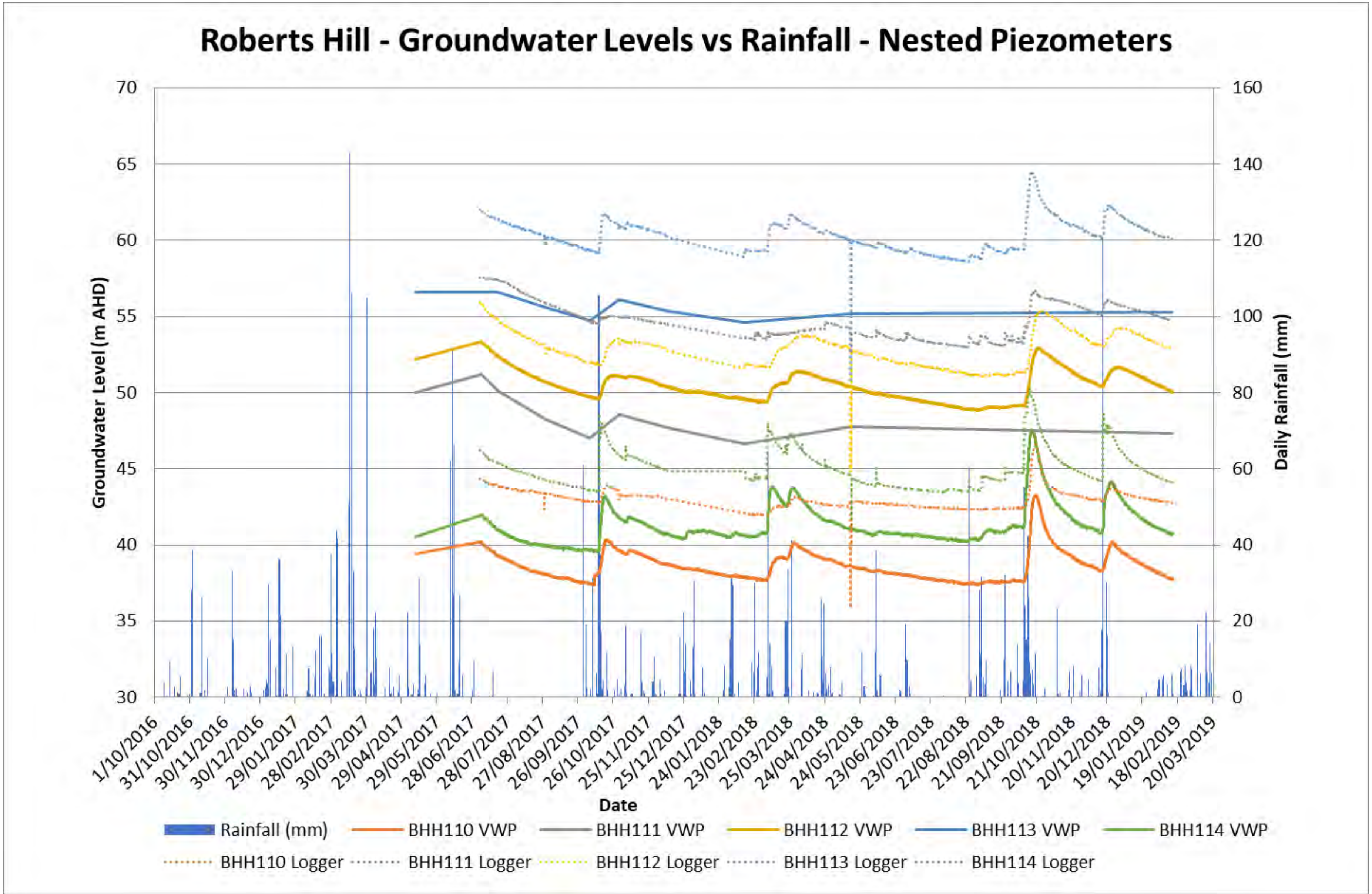
RMU	Hydraulic Conductivity (m/day)			
	Roberts Hill	Shephards Lane	Gatelys Road	Combined
# Test Bores (RMU)	1 (C)	3 (C) & 4 (B)	5 (D) & 1 (C)	MW (7), SW (17) & Fresh (56)
MW	1.4×10^{-1}	NA	1.4×10^{-1}	1.4×10^{-1}
SW	3.9×10^{-3}	2.1×10^{-3}	3.4×10^{-2}	1.1×10^{-2}
FRESH	3.0×10^{-3}	1.4×10^{-3}	5.1×10^{-3}	3.4×10^{-3}

Appendix B
Groundwater Monitoring

- B1 – Roberts Hill Groundwater Monitoring Data
- B2 – Shephards Lane Groundwater Monitoring Data
- B3 – Gatelys Road Groundwater Monitoring Data
- B4 – Groundwater Chemistry

B1 – Roberts Hill Groundwater Monitoring Data


Roberts Hill - Groundwater Levels vs Rainfall - Manual vs Logger
Groundwater Level (m AHD)
Daily Rainfall (mm)
Date
75.00
70.00
65.00
60.00
55.00
50.00
45.00
40.00
35.00
30.00
160
140
120
100
80
60
40
20
0
1/10/2016
31/10/2016
30/11/2016
30/12/2016
29/01/2017
28/02/2017
30/03/2017
29/04/2017
29/05/2017
28/06/2017
28/07/2017
27/08/2017
26/09/2017
26/10/2017
25/11/2017
25/12/2017
24/01/2018
23/02/2018
25/03/2018
24/04/2018
24/05/2018
23/06/2018
23/07/2018
22/08/2018
21/09/2018
21/10/2018
20/11/2018
20/12/2018
19/01/2019
18/02/2019
20/03/2019
Rainfall (mm)
BHH110
BHH111
BHH112
BHH113
BHH114
BHH110 Logger
BHH111 Logger
BHH112 Logger
BHH113 Logger
BHH114 Logger



Roberts Hill - Groundwater Levels vs Rainfall - Nested Piezometers
Groundwater Level (m AHD)
Daily Rainfall (mm)
70
65
60
55
50
45
40
35
30
160
140
120
100
80
60
40
20
0
1/10/2016
31/10/2016
30/11/2016
30/12/2016
29/01/2017
28/02/2017
30/03/2017
29/04/2017
29/05/2017
28/06/2017
28/07/2017
27/08/2017
26/09/2017
26/10/2017
25/11/2017
25/12/2017
24/01/2018
23/02/2018
25/03/2018
24/04/2018
24/05/2018
23/06/2018
23/07/2018
22/08/2018
21/09/2018
21/10/2018
20/11/2018
20/12/2018
19/01/2019
18/02/2019
20/03/2019
Date
Rainfall (mm)
BHH110 VWP
BHH111 VWP
BHH112 VWP
BHH113 VWP
BHH114 VWP
BHH110 Logger
BHH111 Logger
BHH112 Logger
BHH113 Logger
BHH114 Logger

Table B1-1 Roberts Hill Manual Groundwater Level Measurements

Depths Below Ground (uncorrected)

Borehole / Piezometer ID	Easting	Northing	Surface RL (m)	Incline (°)	Bore Completion	Piezo Install Date	Borehole Depth (m)	Measured Groundwater - depth below ground (uncorrected for incline) (m)							
								21/10/2016	28/10/2016	31/10/2016	21/11/2016	24/01/2017	5/02/2017	14/02/2017	15/02/2017
BHH110	508250.04	6648447.56	51.54	64	18/10/16	15/02/17	40.10			8.60	10.00	8.50			9.16
BHH111	508318.73	6648533.3	83.827	70	13/10/16	15/02/17	20.50	25.74		25.74		23.22			31.00
BHH112	508238.04	6648546.74	84.028	90	20/10/16	14/02/17	71.85			31.02	32.47	32.40		32.69	
BHH113	508150.17	6648548.32	83.942	70	18/10/16	14/02/17	56.00			25.70		14.55		25.76	
BHH114	508175.41	6648669.64	51.585	66	27/10/16	5/02/17	41.60		8.13		10.70	6.95	9.35		

Borehole / Piezometer ID	Measured Groundwater - depth below ground (uncorrected for incline) (m)																	
	16/03/2017	29/03/2017	30/03/2017	31/03/2017	19/04/2017	20/04/2017	4/05/2017	5/05/2017	8/05/2017	20/07/2017	29/08/2017	5/10/2017	31/10/2017	11/12/2017	14/02/2018	14/05/2018	13/02/2019	14/02/2019
BHH110	9.37	6.81		6.27	8.17				8.92	8.85	9.46	10.01	9.26	9.57	9.92	9.63		9.62
BHH111	24.77		26.85				27.75		28.47	28.09	29.73	31.08	30.76	31.34	32.27		30.89	
BHH112	32.00		27.37				29.22		29.4	29.13	30.98	31.98	30.52	31.26	32.06	31.46	31.15	
BHH113	20.90		21.6					23.34			25.2	25.3	23.64	24.4	25.23		24.62	
BHH114	2.45	4.83		2.48		5.8				6.88	8.01	8.7	6.33	7.47	7.82	7.93		8.29

Depths Below Ground (corrected for incline)

Borehole / Piezometer ID	Easting	Northing	Surface RL (m)	Incline (°)	Bore Completion	Piezo Install Date	Borehole Depth (m)	Measured Groundwater - depth below ground (corrected for incline) (m)							
								21/10/2016	28/10/2016	31/10/2016	21/11/2016	24/01/2017	5/02/2017	14/02/2017	15/02/2017
BHH110	508250.04	6648447.56	51.54	64	18/10/16	15/02/17	40.10			7.73	8.99	7.64			8.23
BHH111	508318.73	6648533.3	83.827	70	13/10/16	15/02/17	20.50	24.19		24.19		21.82			29.13
BHH112	508238.04	6648546.74	84.028	90	20/10/16	14/02/17	71.85			31.02	32.47	32.40		32.69	
BHH113	508150.17	6648548.32	83.942	70	18/10/16	14/02/17	56.00			24.15		13.67		24.21	
BHH114	508175.41	6648669.64	51.585	66	27/10/16	5/02/17	41.60		7.43		9.77	6.35	8.54		

Borehole / Piezometer ID	Measured Groundwater - depth below ground (corrected for incline) (m)																	
	16/03/2017	29/03/2017	30/03/2017	31/03/2017	19/04/2017	20/04/2017	4/05/2017	5/05/2017	8/05/2017	20/07/2017	29/08/2017	5/10/2017	31/10/2017	11/12/2017	14/02/2018	14/05/2018	13/02/2019	14/02/2019
BHH110	8.42	6.12		5.64	7.34				8.02	7.95	8.50	9.00	8.32	8.60	8.92	8.66		8.65
BHH111	23.28		25.23				26.08		26.75	26.40	27.94	29.21	28.90	29.45	30.32		29.03	
BHH112	32.00		27.37				29.22		29.40	29.13	30.98	31.98	30.52	31.26	32.06	31.46	31.15	
BHH113	19.64		20.30					21.93			23.68	23.77	22.21	22.93	23.71		23.14	
BHH114	2.24	4.41		2.27		5.30				6.29	7.32	7.95	5.78	6.82	7.14	7.24		7.57

Depths (RL)

Borehole / Piezometer ID	Easting	Northing	Surface RL (m)	Incline (°)	Bore Completion	Piezo Install Date	Borehole Depth (m)	Measured Groundwater - RL (m)							
								21/10/2016	28/10/2016	31/10/2016	21/11/2016	24/01/2017	5/02/2017	14/02/2017	15/02/2017
BHH110	508250.04	6648447.56	51.54	64	18/10/16	15/02/17	40.10			43.81	42.55	43.90			43.31
BHH111	508318.73	6648533.3	83.827	70	13/10/16	15/02/17	20.50	59.64		59.64		62.01			54.70
BHH112	508238.04	6648546.74	84.028	90	20/10/16	14/02/17	71.85			53.01	51.56	51.63		51.34	
BHH113	508150.17	6648548.32	83.942	70	18/10/16	14/02/17	56.00			59.79		70.27		59.74	
BHH114	508175.41	6648669.64	51.585	66	27/10/16	5/02/17	41.60		44.16		41.81	45.24	43.04		

Borehole / Piezometer ID	Measured Groundwater - RL (m)																	
	16/03/2017	29/03/2017	30/03/2017	31/03/2017	19/04/2017	20/04/2017	4/05/2017	5/05/2017	8/05/2017	20/07/2017	29/08/2017	5/10/2017	31/10/2017	11/12/2017	14/02/2018	14/05/2018	13/02/2019	14/02/2019
BHH110	43.12	45.42		45.90	44.20				43.52	43.59	43.04	42.54	43.22	42.94	42.62	42.88		42.89
BHH111	60.55		58.60				57.75		57.07	57.43	55.89	54.62	54.92	54.38	53.50		54.80	
BHH112	52.03		56.66				54.81		54.63	54.90	53.05	52.05	53.51	52.77	51.97	52.57	52.88	
BHH113	64.30		63.64					62.01			60.26	60.17	61.73	61.01	60.23		60.81	
BHH114	49.35	47.17		49.32		46.29				45.30	44.27	43.64	45.80	44.76	44.44	44.34		44.01

Open hole reading

Standpipe piezometer reading

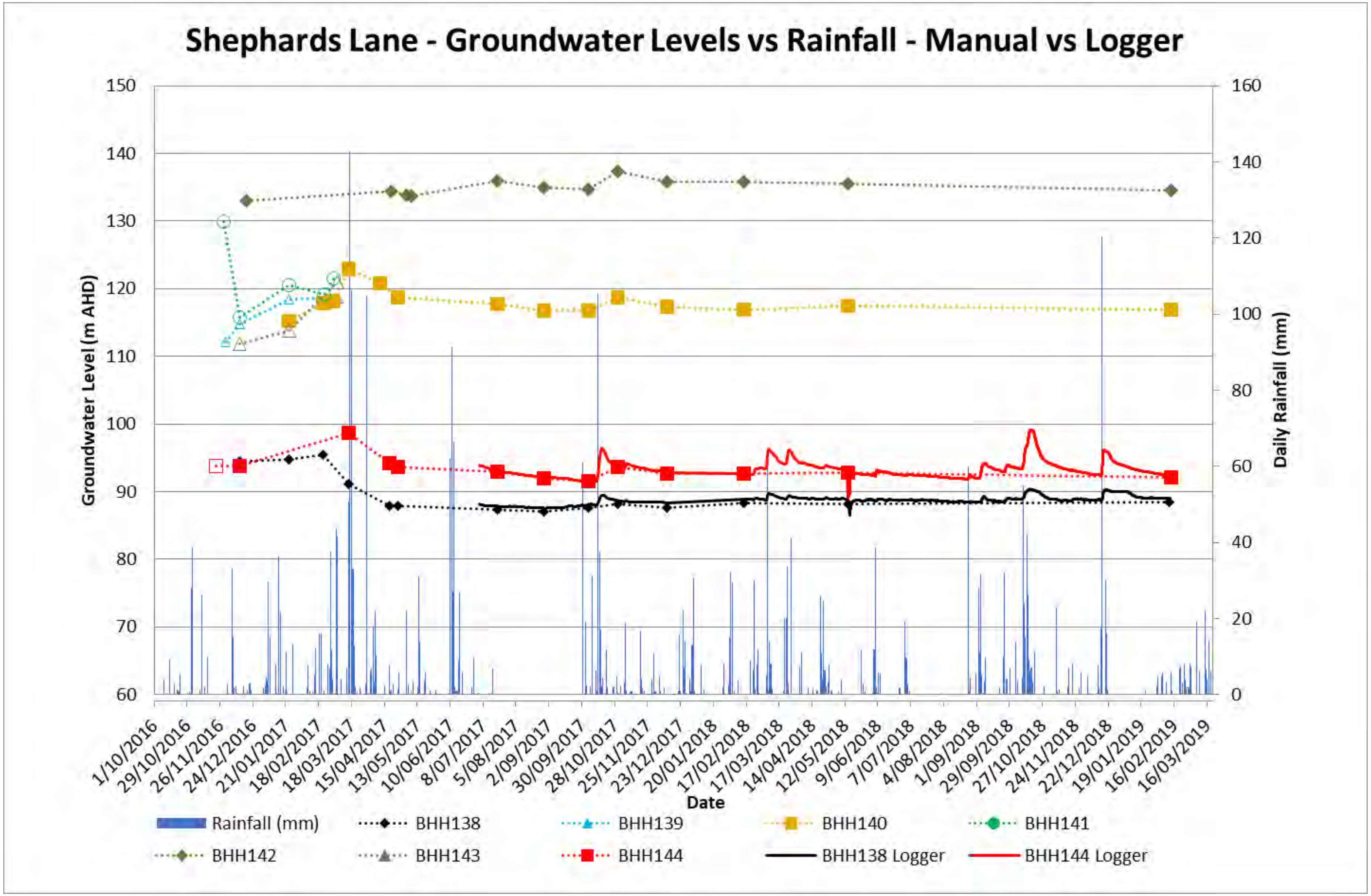
Table B1-2 Roberts Hill VWP Measurements

									Installation measurement						Additional measurement				
										In Water Zero		Reading prior to Grouting					Readings		
Bore ID	Location	Depth of VWP (m)	Depth to water at installation (uncorrected for incline) (m)*	Incline (°)	Depth to water at installation (corrected for incline)	Piezo Ref No	Pressure Coefficient (C_P)	Thermal Coefficient (C_T)	Date	kHz^2x10^{-3} (F_0)	Deg (T_0)	kHz^2x10^{-3} (F_1)	Deg (T_1)	Pore Pressure	Date	Depth to water (m)**	kHz^2x10^{-3} (F_1)	Deg (T_1)	Pore Pressure
BHH110	Roberts Hill	31	9.16	64	8.23	23105	0.1652	0.04108	15/02/2017	8721.7	20.7	7534.5	18.7	**196.0433**	11/05/2017	8.92	7601.9	18.4	**184.89648**
BHH111	Roberts Hill	54	31	70	29.13	23106	0.1617	-0.01566	15/02/2017	8825.9	25.3	7577.5	19.1	**201.9634**	11/05/2017	28.88	7597.8	19	**198.68243**
BHH112	Roberts Hill	60	32.69	90	32.69	23108	0.1634	0.0463	14/02/2017	8801	25.7	7144.7	19	**270.3292**	11/05/2017	29.80	7107.9	18.9	**276.3377**
BHH113	Roberts Hill	54	25.76	70	24.21	23107	0.1694	0.01162	14/02/2017	8832.7	24.4	7283.3	19.7	**262.4137**	11/05/2017	23.39	7288.5	19	**261.52473**
BHH114	Roberts Hill	31	9.35	66	8.54	23104	0.1748	-0.00292	15/02/2017	8897.8	26.9	7772.4	20.3	**196.7392**	11/05/2017	6.97	7781.2	19.2	**195.20416**

* water depth measured in open hole may not represent true groundwater level

** water depth measured in standpipe above VWP

B2 – Shephards Lane Groundwater Monitoring Data



Shephards Lane - Groundwater Levels vs Rainfall - Manual vs Logger
Groundwater Level (m AHD)
Daily Rainfall (mm)
150
140
130
120
110
100
90
80
70
60
160
140
120
100
80
60
40
20
0
1/10/2016
29/10/2016
26/11/2016
24/12/2016
21/01/2017
18/02/2017
18/03/2017
15/04/2017
13/05/2017
10/06/2017
8/07/2017
5/08/2017
2/09/2017
30/09/2017
28/10/2017
25/11/2017
23/12/2017
20/01/2018
17/02/2018
17/03/2018
14/04/2018
12/05/2018
9/06/2018
7/07/2018
4/08/2018
1/09/2018
29/09/2018
27/10/2018
24/11/2018
22/12/2018
19/01/2019
16/02/2019
16/03/2019
Date
Rainfall (mm)
BHH138
BHH139
BHH140
BHH141
BHH142
BHH143
BHH144
BHH138 Logger
BHH144 Logger

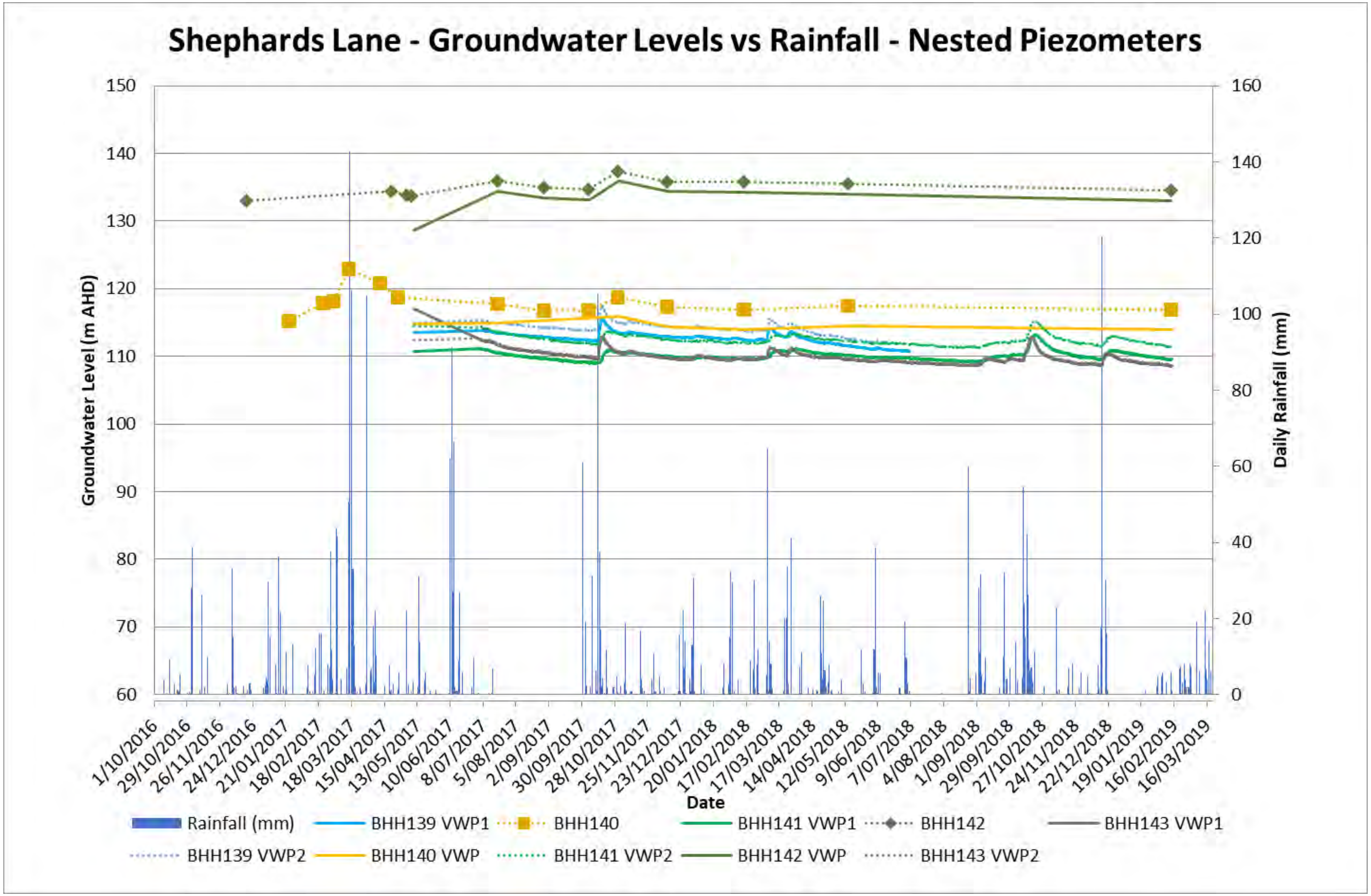

Shephards Lane - Groundwater Levels vs Rainfall - Nested Piezometers
Groundwater Level (m AHD)
Daily Rainfall (mm)
Date
150
140
130
120
110
100
90
80
70
60
160
140
120
100
80
60
40
20
0
1/10/2016
29/10/2016
26/11/2016
24/12/2016
21/01/2017
18/02/2017
18/03/2017
15/04/2017
13/05/2017
10/06/2017
8/07/2017
5/08/2017
2/09/2017
30/09/2017
28/10/2017
25/11/2017
23/12/2017
20/01/2018
17/02/2018
17/03/2018
14/04/2018
12/05/2018
9/06/2018
7/07/2018
4/08/2018
1/09/2018
29/09/2018
27/10/2018
24/11/2018
22/12/2018
19/01/2019
16/02/2019
16/03/2019
Rainfall (mm)
BHH139 VWP1
BHH140
BHH141 VWP1
BHH142
BHH143 VWP1
BHH139 VWP2
BHH140 VWP
BHH141 VWP2
BHH142 VWP
BHH143 VWP2

Table B2-1 Shephards Lane Manual Groundwater Level Measurements

Depths Below Ground (uncorrected)

Borehole / Piezometer ID	Easting	Northing	Surface RL (m)	Incline (°)	Bore Completion	Piezo Install Date	Borehole Depth (m)	Measured Groundwater - depth below ground (uncorrected for incline) (m)											
								23/11/2016	25/11/2016	29/11/2016	1/12/2016	13/12/2016	19/12/2016	24/01/2017	22/02/2017	23/02/2017	3/03/2017	6/03/2017	7/03/2017
BHH138	508715.15	6651133.57	96.44	90	24/11/2016	8/03/2017	30.00					2.00		1.70	Above 1.0				
BHH139	508828.40	6651178.55	127.16	90	1/12/2016	8/03/2017	60.10				15.00	12.30		8.75					8.53
BHH140	508906.71	6651147.33	160.94	70	20/12/2016	9/03/2017	82.65							48.60	45.89		45.50		
BHH141	508886.47	6651189.66	151.98	90	2/12/2016	8/03/2017	84.10			22.10		36.20		31.43		32.85	30.50		
BHH142	508870.20	6651256.34	157.98	70	11/12/2016	12/04/2017	80.70						26.60						
BHH143	508944.30	6651214.88	122.02	90	8/12/2016	8/03/2017	55.52					10.10		8.25				1.00	
BHH144	509029.79	6651219.67	106.04	90	17/11/2016	10/03/2017	40.00	12.31				12.20							

Borehole / Piezometer ID	Measured Groundwater - depth below ground (uncorrected for incline) (m)															
	16/03/2017	11/04/2017	19/04/2017	21/04/2017	27/04/2017	4/05/2017	8/05/2017	20/07/2017	29/08/2017	6/10/2017	31/10/2017	12/12/2017	15/02/2018	15/05/2018	12/02/2019	14/02/2019
BHH138	5.35		8.48		8.5			9.14	9.42	8.89	8.28	8.76	8.1	8.29	8.04	
BHH139																
BHH140	40.52	42.75			45			45.91	47	47	44.98	46.46	46.92	46.26		46.91
BHH141																
BHH142				25.07		25.7	25.82	23.45	24.44	24.75	21.9	23.55	23.6	23.94		24.97
BHH143																
BHH144	7.30		11.78		12.44			13.04	14.03	14.56	12.46	13.37	13.41	13.22		13.88

Depths Below Ground (corrected for incline)

Borehole / Piezometer ID	Easting	Northing	Surface RL (m)	Incline (°)	Bore Completion	Piezo Install Date	Borehole Depth (m)	Measured Groundwater - depth below ground (corrected for incline) (m)											
								23/11/2016	25/11/2016	29/11/2016	1/12/2016	13/12/2016	19/12/2016	24/01/2017	22/02/2017	23/02/2017	3/03/2017	6/03/2017	7/03/2017
BHH138	508715.15	6651133.57	96.44	90	24/11/2016	8/03/2017	30.00					2.00		1.70	Above 1				
BHH139	508828.40	6651178.55	127.16	90	1/12/2016	8/03/2017	60.10				15.00	12.30		8.75					8.53
BHH140	508906.71	6651147.33	160.94	70	20/12/2016	9/03/2017	82.65							45.67	43.12		42.76		
BHH141	508886.47	6651189.66	151.98	90	2/12/2016	8/03/2017	84.10			22.10		36.20		31.43		32.85	30.50		
BHH142	508870.20	6651256.34	157.98	70	11/12/2016	12/04/2017	80.70						25.00						
BHH143	508944.30	6651214.88	122.02	90	8/12/2016	8/03/2017	55.52					10.10		8.25				1.00	
BHH144	509029.79	6651219.67	106.04	90	17/11/2016	10/03/2017	40.00	12.31				12.20							

Borehole / Piezometer ID	Measured Groundwater - depth below ground (corrected for incline) (m)															
	16/03/2017	11/04/2017	19/04/2017	21/04/2017	27/04/2017	4/05/2017	8/05/2017	20/07/2017	29/08/2017	6/10/2017	31/10/2017	12/12/2017	15/02/2018	15/05/2018	12/02/2019	14/02/2019
BHH138	5.35		8.48		8.50			9.14	9.42	8.89	8.28	8.76	8.10	8.29	8.04	
BHH139																
BHH140	38.08	40.17			42.29			43.14	44.17	44.17	42.27	43.66	44.09	43.47		44.08
BHH141																
BHH142				23.56		24.15	24.26	22.04	22.97	23.26	20.58	22.13	22.18	22.50		23.46
BHH143																
BHH144	7.30		11.78		12.44			13.04	14.03	14.56	12.46	13.37	13.41	13.22		13.88

Depths (RL)

Borehole / Piezometer ID	Easting	Northing	Surface RL (m)	Incline (°)	Bore Completion	Piezo Install Date	Borehole Depth (m)	Measured Groundwater - RL (m)											
								23/11/2016	25/11/2016	29/11/2016	1/12/2016	13/12/2016	19/12/2016	24/01/2017	22/02/2017	23/02/2017	3/03/2017	6/03/2017	7/03/2017
BHH138	508715.15	6651133.57	96.44	90	24/11/2016	8/03/2017	30.00					94.44		94.74	95.44				
BHH139	508828.40	6651178.55	127.16	90	1/12/2016	8/03/2017	60.10				112.16	114.86		118.41					118.63
BHH140	508906.71	6651147.33	160.94	70	20/12/2016	9/03/2017	82.65							115.27	117.82		118.18		
BHH141	508886.47	6651189.66	151.98	90	2/12/2016	8/03/2017	84.10			129.88		115.78		120.55		119.13	121.48		
BHH142	508870.20	6651256.34	157.98	70	11/12/2016	12/04/2017	80.70						132.98						
BHH143	508944.30	6651214.88	122.02	90	8/12/2016	8/03/2017	55.52					111.92		113.77				121.02	
BHH144	509029.79	6651219.67	106.04	90	17/11/2016	10/03/2017	40.00	93.73				93.84							

Borehole / Piezometer ID	Measured Groundwater - RL (m)															
	16/03/2017	11/04/2017	19/04/2017	21/04/2017	27/04/2017	4/05/2017	8/05/2017	20/07/2017	29/08/2017	6/10/2017	31/10/2017	12/12/2017	15/02/2018	15/05/2018	12/02/2019	14/02/2019
BHH138	91.09		87.96		87.94			87.30	87.02	87.55	88.16	87.68	88.34	88.15	88.40	
BHH139																
BHH140	122.86	120.77			118.65			117.80	116.77	116.77	118.67	117.28	116.85	117.47		116.86
BHH141																
BHH142				134.42		133.83	133.72	135.94	135.01	134.72	137.40	135.85	135.80	135.48		134.52
BHH143																
BHH144	98.74		94.26		93.60			93.00	92.01	91.48	93.58	92.67	92.63	92.82		92.16

Open hole reading
Standpipe piezometer reading

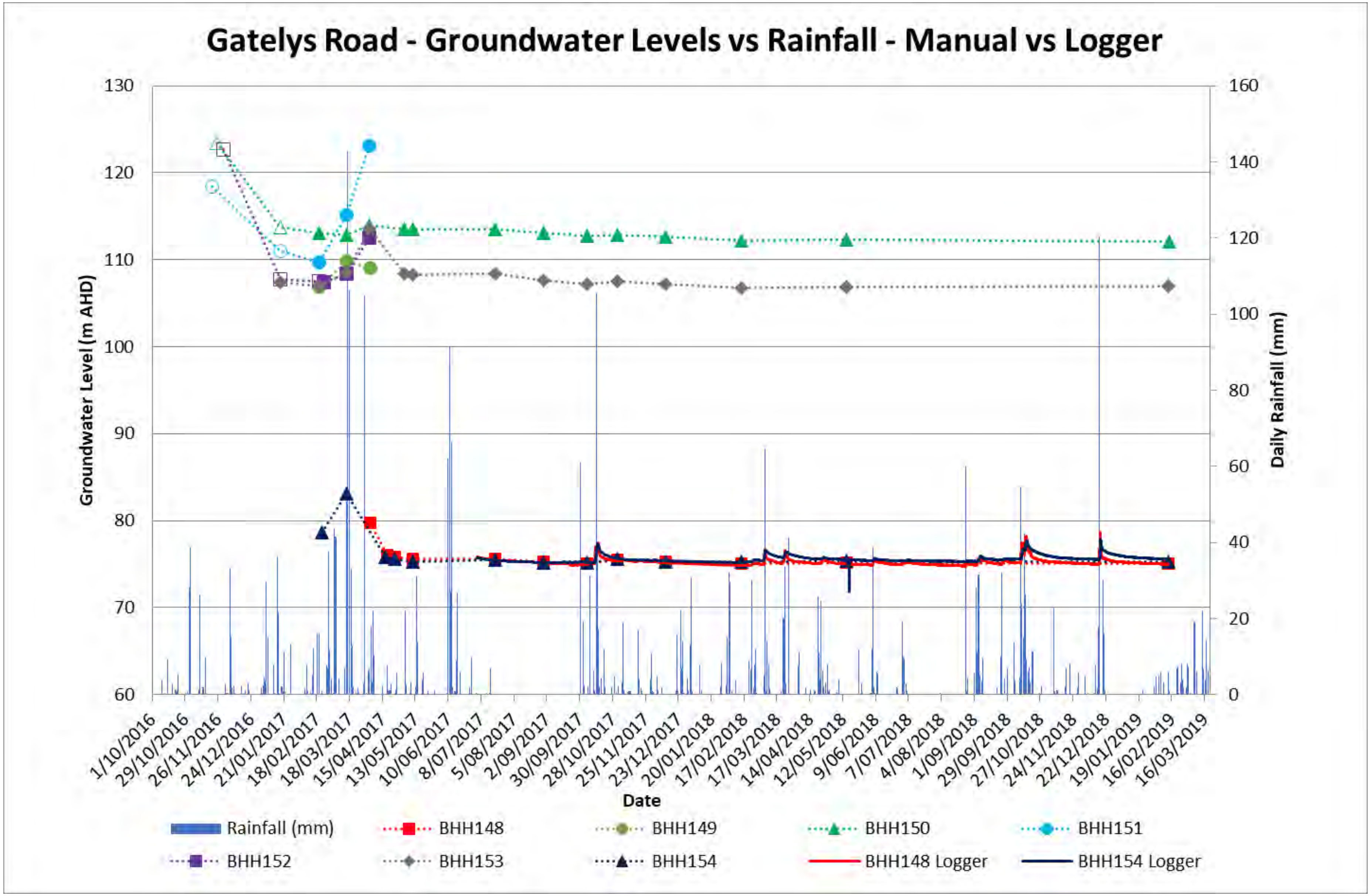
Table B2-2 Shephards Lane VWP Measurements

Bore ID	Location	Depth of VWP (m)	Depth to water at installation (uncorrected for incline) (m)*	Incline (°)	Depth to water at installation (corrected for incline)	Piezo Ref No	Pressure Coefficient (C_P)	Thermal Coefficient (C_T)	Installation measurement						Additional measurement				
									Date	In Water Zero		Reading prior to Grouting		Pore Pressure	Date	Depth to water (m)**	Readings		Pore Pressure
										kHz^2x10^{-3} (F_0)	Deg (T_0)	kHz^2x10^{-3} (F_1)	Deg (T_1)				kHz^2x10^{-3} (F_1)	Deg (T_1)	
BHH139	Shephards Lane	48	8.53	90	8.53	23130	0.1691	-0.00053	7/03/2017	9071.4	25.9	6798.4	18.7	**384.3681**	10/05/2017	-	7111.9	18.6	**331.35532**
BHH139	Shephards Lane	28	8.53	90	8.53	23129	0.1644	-0.01419	7/03/2017	8896.5	26.2	7729.5	19.3	**191.9527**	10/05/2017	-	7989.4	18.7	**149.23367**
BHH140	Shephards Lane	82.5	45.5	70	42.76	23162	0.2566	0.01325	6/03/2017	8974.8	27.6	7641.2	19.2	**342.0905**	10/05/2017	46.07	7723.4	19	**320.99529**
BHH141	Shephards Lane	72.4	30.5	90	30.5	23163	0.2507	-0.01724	3/03/2017	9101.7	20.7	7803.1	19.3	**325.5832**	10/05/2017	-	7868.8	19	**309.11734**
BHH141	Shephards Lane	52.4	30.5	90	30.5	23131	0.1598	-0.02722	3/03/2017	8973	27.2	7946.2	19.3	**164.2977**	10/05/2017	-	8034.4	19.1	**150.20876**
BHH142	Shephards Lane	75.8	21.9	70	20.58	23164	0.2601	0.04756	14/04/2017	8803.2	16.7	7045.9	18.9	**457.1784**	10/05/2017	29.23	7670.8	18.8	**294.63712**
BHH143	Shephards Lane	42	1	90	1	23133	0.1594	-0.01561	6/03/2017	8867.4	28.2	6362.2	21.9	**399.4272**	1/06/2017	-	6887.8	19.3	**315.68717**
BHH143	Shephards Lane	22	1	90	1	23132	0.1504	-0.02881	6/03/2017	9061.1	28.6	7708.7	21.6	**203.6026**	1/06/2017	-	8254.9	19.3	**121.52041**

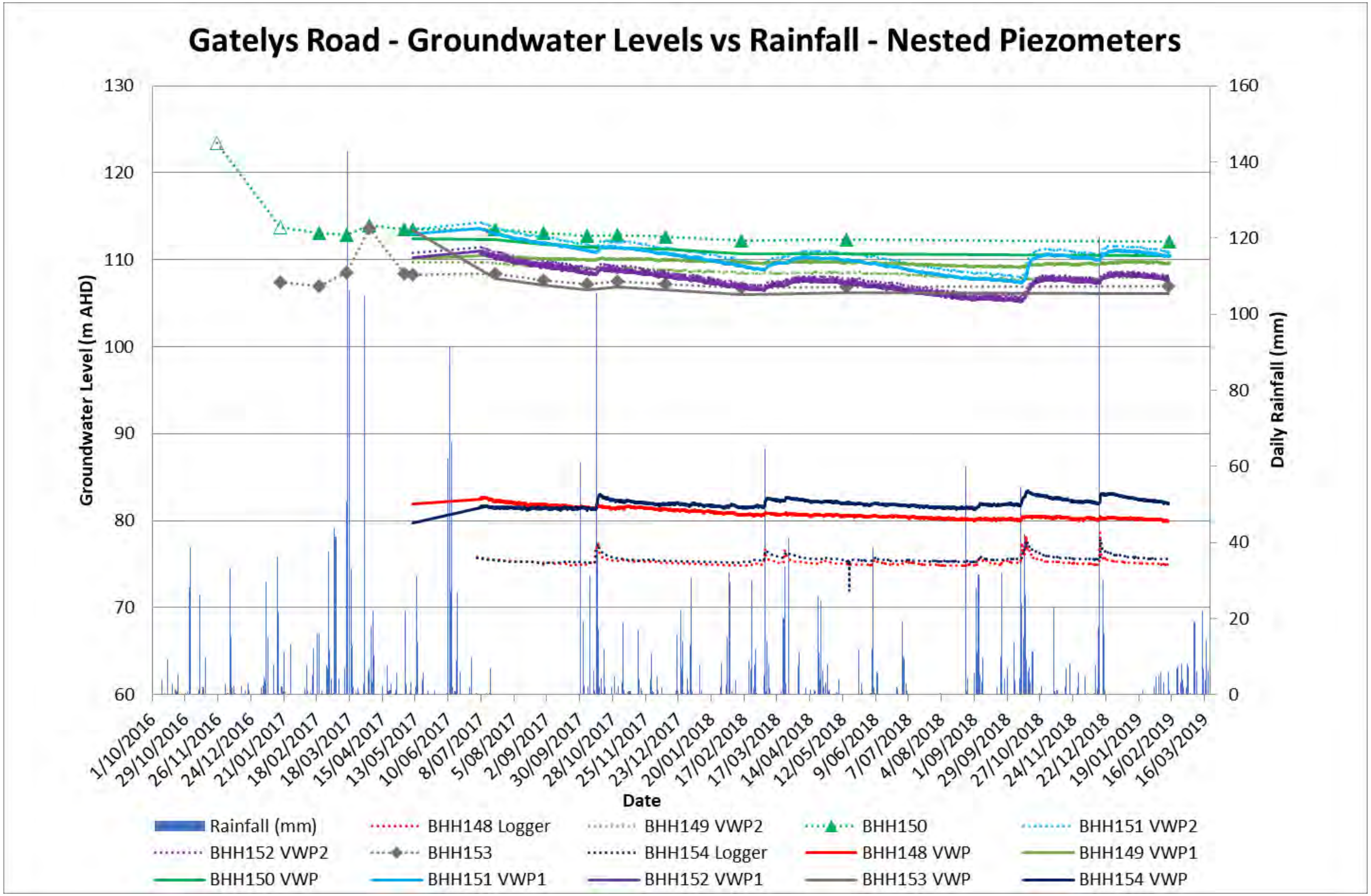
* water depth measured in open hole may not represent true groundwater level

** water depth measured in standpipe above VWP

B3 – Gatelys Road Groundwater Monitoring Data



Gatelys Road - Groundwater Levels vs Rainfall - Manual vs Logger
Groundwater Level (m AHD)
Daily Rainfall (mm)
130
120
110
100
90
80
70
60
160
140
120
100
80
60
40
20
0
1/10/2016
29/10/2016
26/11/2016
24/12/2016
21/01/2017
18/02/2017
18/03/2017
15/04/2017
13/05/2017
10/06/2017
8/07/2017
5/08/2017
2/09/2017
30/09/2017
28/10/2017
25/11/2017
23/12/2017
20/01/2018
17/02/2018
17/03/2018
14/04/2018
12/05/2018
9/06/2018
7/07/2018
4/08/2018
1/09/2018
29/09/2018
27/10/2018
24/11/2018
22/12/2018
19/01/2019
16/02/2019
16/03/2019
Date
Rainfall (mm)
BHH148
BHH149
BHH150
BHH151
BHH152
BHH153
BHH154
BHH148 Logger
BHH154 Logger



Gatelys Road - Groundwater Levels vs Rainfall - Nested Piezometers
Groundwater Level (m AHD)
Daily Rainfall (mm)
Date
130
120
110
100
90
80
70
60
160
140
120
100
80
60
40
20
0
1/10/2016
29/10/2016
26/11/2016
24/12/2016
21/01/2017
18/02/2017
18/03/2017
15/04/2017
13/05/2017
10/06/2017
8/07/2017
5/08/2017
2/09/2017
30/09/2017
28/10/2017
25/11/2017
23/12/2017
20/01/2018
17/02/2018
17/03/2018
14/04/2018
12/05/2018
9/06/2018
7/07/2018
4/08/2018
1/09/2018
29/09/2018
27/10/2018
24/11/2018
22/12/2018
19/01/2019
16/02/2019
16/03/2019
Rainfall (mm)
BHH148 Logger
BHH149 VWP2
BHH150
BHH151 VWP2
BHH152 VWP2
BHH153
BHH154 Logger
BHH148 VWP
BHH149 VWP1
BHH150 VWP
BHH151 VWP1
BHH152 VWP1
BHH153 VWP
BHH154 VWP

Table B3-1 Gatelys Road Manual Groundwater Level Measurements

Depths Below Ground (uncorrected)

Borehole / Piezometer ID	Easting	Northing	Surface RL (m)	Incline (°)	Bore Completion	Piezo Install Date	Borehole Depth (m)	Measured Groundwater - depth below ground (uncorrected for incline) (m)										
								21/11/2016	25/11/2016	29/11/2016	1/12/2016	18/01/2017	20/02/2017	23/02/2017	24/02/2017	16/03/2017	4/04/2017	5/04/2017
BHH148	510585.23	6651112.70	79.70	69	2/11/10	7/04/17	35.03											0
BHH149	510741.95	6651131.89	138.77	70	23/01/17	7/04/17	98.00						34.00			30.80		31.64
BHH150	510775.14	6651164.81	154.17	90	25/11/16	7/04/17	91.27		30.70			40.46	41.06			41.30	40.18	
BHH151	510816.42	6651116.12	148.50	90	15/11/16	4/04/17	101.00	30.05				37.48	38.76			33.30	25.35	
BHH152	510878.33	6651125.65	137.70	68	21/12/16	5/04/17	98.20				16.20	32.30	32.60		32.71	31.60	27.17	
BHH153	510891.56	6651075.38	136.19	90	5/12/16	7/04/17	74.40					28.80	29.25			27.72	22.53	
BHH154	511013.98	6651126.49	86.82	90	17/01/17	12/04/17	44.00							8.15		3.73		

Borehole / Piezometer ID	Measured Groundwater - depth below ground (uncorrected for incline) (m)														
	18/04/2017	19/04/2017	26/04/2017	27/04/2017	4/05/2017	11/05/2017	20/07/2017	30/08/2017	6/10/2017	1/11/2017	12/12/2017	15/02/2018	15/05/2018	13/02/2019	14/02/2019
BHH148		3.95	4.2			4.4	4.44	4.78	4.96	4.56	4.8	5	4.83	5	
BHH149															
BHH150					40.67	40.67	40.7	41.14	41.45	41.3	41.56	41.92	41.9		42.04
BHH151															
BHH152															
BHH153					27.77	27.96	27.75	28.55	28.96	28.63	29.04	29.48	29.35	29.23	
BHH154	11		11.25			11.52	11.37	11.71	11.67	11.24	11.54	11.6	11.48	11.52	

Depths Below Ground (corrected for incline)

Borehole / Piezometer ID	Easting	Northing	Surface RL (m)	Incline (°)	Bore Completion	Piezo Install Date	Borehole Depth (m)	Measured Groundwater - depth below ground (corrected for incline) (m)										
								21/11/2016	25/11/2016	29/11/2016	1/12/2016	18/01/2017	20/02/2017	23/02/2017	24/02/2017	16/03/2017	4/04/2017	5/04/2017
BHH148	510585.23	6651112.70	79.70	69	2/11/10	7/04/17	35.03											0.00
BHH149	510741.95	6651131.89	138.77	70	23/01/17	7/04/17	98.00						31.95			28.94		29.73
BHH150	510775.14	6651164.81	154.17	90	25/11/16	7/04/17	91.27		30.70			40.46	41.06			41.30	40.18	
BHH151	510816.42	6651116.12	148.50	90	15/11/16	4/04/17	101.00	30.05				37.48	38.76			33.30	25.35	
BHH152	510878.33	6651125.65	137.70	68	21/12/16	5/04/17	98.20				15.02	29.95	30.23		30.33	29.30	25.19	
BHH153	510891.56	6651075.38	136.19	90	5/12/16	7/04/17	74.40					28.80	29.25			27.72	22.53	
BHH154	511013.98	6651126.49	86.82	90	17/01/17	12/04/17	44.00							8.15		3.73		

Borehole / Piezometer ID	Measured Groundwater - depth below ground (corrected for incline) (m)														
	18/04/2017	19/04/2017	26/04/2017	27/04/2017	4/05/2017	11/05/2017	20/07/2017	30/08/2017	6/10/2017	1/11/2017	12/12/2017	15/02/2018	15/05/2018	13/02/2019	14/02/2019
BHH148		3.69	3.92			4.11	4.15	4.46	4.63	4.26	4.48	4.67	4.51	4.67	
BHH149															
BHH150					40.67	40.67	40.70	41.14	41.45	41.30	41.56	41.92	41.90		42.04
BHH151															
BHH152															
BHH153					27.77	27.96	27.75	28.55	28.96	28.63	29.04	29.48	29.35	29.23	
BHH154	11.00		11.25			11.52	11.37	11.71	11.67	11.24	11.54	11.60	11.48	11.52	

Depths (RL)

Borehole / Piezometer ID	Easting	Northing	Surface RL (m)	Incline (°)	Bore Completion	Piezo Install Date	Borehole Depth (m)	Measured Groundwater - RL (m)										
								21/11/2016	25/11/2016	29/11/2016	1/12/2016	18/01/2017	20/02/2017	23/02/2017	24/02/2017	16/03/2017	4/04/2017	5/04/2017
BHH148	510585.23	6651112.70	79.70	69	2/11/10	7/04/17	35.03											79.70
BHH149	510741.95	6651131.89	138.77	70	23/01/17	7/04/17	98.00						106.82			109.83		109.04
BHH150	510775.14	6651164.81	154.17	90	25/11/16	7/04/17	91.27		123.47			113.71	113.11			112.87	113.99	
BHH151	510816.42	6651116.12	148.50	90	15/11/16	4/04/17	101.00	118.45				111.02	109.74			115.20	123.15	
BHH152	510878.33	6651125.65	137.70	68	21/12/16	5/04/17	98.20				122.68	107.75	107.47		107.37	108.40	112.51	
BHH153	510891.56	6651075.38	136.19	90	5/12/16	7/04/17	74.40					107.39	106.94			108.47	113.66	
BHH154	511013.98	6651126.49	86.82	90	17/01/17	12/04/17	44.00							78.67		83.09		

Borehole / Piezometer ID	Measured Groundwater - RL (m)														
	18/04/2017	19/04/2017	26/04/2017	27/04/2017	4/05/2017	11/05/2017	20/07/2017	30/08/2017	6/10/2017	1/11/2017	12/12/2017	15/02/2018	15/05/2018	13/02/2019	14/02/2019
BHH148		76.01	75.78			75.59	75.55	75.24	75.07	75.44	75.22	75.03	75.19	75.03	
BHH149															
BHH150					113.50	113.50	113.47	113.03	112.72	112.87	112.61	112.25	112.27		112.13
BHH151															
BHH152															
BHH153					108.42	108.23	108.44	107.64	107.23	107.56	107.15	106.71	106.84	106.96	
BHH154	75.82		75.57			75.30	75.45	75.11	75.15	75.58	75.28	75.22	75.34	75.30	

Open hole reading

Standpipe piezometer reading

Table B3-2 Gatelys Road VWP Measurements

Bore ID	Location	Depth of VWP (m)	Depth to water at installation (uncorrected for incline) (m)*	Incline (°)	Depth to water at installation (corrected for incline)	Piezo Ref No	Pressure Coefficient (C_P)	Thermal Coefficient (C_T)	Installation measurement						Additional measurement				
									Date	In Water Zero		Reading prior to Grouting		Pore Pressure	Date	Depth to water (m)**	Readings		Pore Pressure
										kHz^2x10^{-3} (F_0)	Deg (T_0)	kHz^2x10^{-3} (F_1)	Deg (T_1)				kHz^2x10^{-3} (F_1)	Deg (T_1)	
BHH148	Gatelys Road	20	0	69	0	23323	0.1123	-0.05905	5/04/2017	8782.2	19.6	7185.1	18.9	**179.3957**	11/05/2017	4.40	6948	19.12	**206.009**
BHH149	Gatelys Road	60	31.64	70	29.73	23294	0.1541	-0.03281	5/04/2017	8893	19.1	7174.4	18.7	**264.8494**	11/05/2017	-	7153.2	18.8	**268.11302**
BHH149	Gatelys Road	80	31.64	70	29.73	23301	0.2569	0.1	5/04/2017	8893	19	7149.5	19.5	**447.9552**	11/05/2017	-	7112.2	18.8	**457.46752**
BHH150	Gatelys Road	89.8	40.18	90	40.18	23304	0.2562	0.204	4/04/2017	8991.1	18.9	7111.8	19.1	**481.5175**	11/05/2017	4.67	7146.6	19.1	**472.6017**
BHH151	Gatelys Road	70	25.35	90	25.35	23296	0.1637	-0.0163	4/04/2017	8871.4	18.3	6220.8	18.1	**433.9065**	11/05/2017	-	6897.5	19.1	**323.11439**
BHH151	Gatelys Road	88.5	25.35	90	25.35	23303	0.2725	0.1504	4/04/2017	8943.3	18.2	6680.8	18.3	**616.5463**	11/05/2017	-	7109.4	18.9	**499.84303**
BHH152	Gatelys Road	62.5	27.17	68	25.19	23295	0.1609	-0.00464	5/04/2017	8976.5	18.9	6988.3	19.1	**319.9005**	11/05/2017	-	7164.5	19.1	**291.54987**
BHH152	Gatelys Road	82.5	27.17	68	25.19	23302	0.265	0.2062	4/04/2017	8979.2	19.4	7098.5	19.2	**498.3443**	11/05/2017	-	7213.9	19.4	**467.8045**
BHH153	Gatelys Road	73.4	22.53	90	22.53	23297	0.1557	0.011	4/04/2017	8782.4	18.7	5585.5	19	**497.7606**	11/05/2017	27.96	5592.1	19	**496.73301**
BHH154	Gatelys Road	28	7.6	90	7.6	23324	0.1036	-0.06494	12/04/2017	8873.5	20.8	6943	20.3	**200.0323**	11/05/2017	11.52	6892.8	19.7	**205.27195**

* water depth measured in open hole may not represent true groundwater level

** water depth measured in standpipe above VWP

B4 – Groundwater Chemistry

Table B4-1 Roberts Hill – May 2018 Groundwater Chemistry Analysis

Method Name	Analyte Name	Units	Monitoring Bore	BHH106	BHH109	BHH110	BHH111	BHH112	BHH113	BHH114	BHH115	BHH117	BHH119
			Sample Date	16/5/18	16/5/18	16/5/18	16/5/18	16/5/18	16/5/18	16/5/18	16/5/18	16/5/18	16/5/18
			Reporting Limit	Result	Result	Result	Result	Result	Result	Result	Result	Result	Result
Anions by Ion Chromatography in Water	Chloride	mg/L	0.05	17	20	24	22	44	28	15	54	19	26
Anions by Ion Chromatography in Water	Sulfate, SO4	mg/L	1	34	36	17	87	42	16	32	39	64	17
Metals in Water (Total) by ICPOES	Total Calcium	mg/L	0.1	4.2	5.8	8.7	68	38	5.7	23	47	16	40
Metals in Water (Total) by ICPOES	Total Magnesium	mg/L	0.1	3.1	5.2	6.7	25	18	4.6	20	8.7	10	7.8
Metals in Water (Total) by ICPOES	Total Sodium	mg/L	0.1	35	30	24	39	49	27	19	37	41	51
Metals in Water (Total) by ICPOES	Total Potassium	mg/L	0.2	1.8	1.6	2.7	6.7	4.9	2.0	8.1	2.8	3.7	5.3
Trace Metals (Total) in Water by ICPMS	Total Aluminium	µg/L	5	81	230	1,800	1,500	1,700	880	39,000	140	4,200	170
Trace Metals (Total) in Water by ICPMS	Total Iron	µg/L	5	390	320	3,100	1,800	2,100	440	34,000	580	4,200	790

Table B4-2 Shephards Lane – May 2018 Groundwater Chemistry Analysis

Method Name	Analyte Name	Units	Monitoring Bore	BHH131	BHH132	BHH140	BHH144
			Sample Date	16/5/2018	16/5/2018	15/5/2018	15/5/2018
			Reporting Limit	Result	Result	Result	Result
Anions by Ion Chromatography in Water	Chloride	mg/L	0.05	16	31	20	15
Anions by Ion Chromatography in Water	Sulfate, SO4	mg/L	1	18	76	17	15
Metals in Water (Total) by ICPOES	Total Calcium	mg/L	0.1	4.4	81	22	26
Metals in Water (Total) by ICPOES	Total Magnesium	mg/L	0.1	16	13	6.3	4.9
Metals in Water (Total) by ICPOES	Total Sodium	mg/L	0.1	20	28	25	19
Metals in Water (Total) by ICPOES	Total Potassium	mg/L	0.2	9.1	4.5	1.6	2.6
Trace Metals (Total) in Water by ICPMS	Total Aluminium	µg/L	5	17,000	2,000	470	4,200
Trace Metals (Total) in Water by ICPMS	Total Iron	µg/L	5	26,000	8,100	440	1,900

Table B4-3 Gatelys Road – May 2018 Groundwater Chemistry Analysis

Method Name	Analyte Name	Units	Monitoring Bore	BHH147	BHH153	BHH158	BHH160
			Sample Date	15/5/2018	15/5/2018	15/5/2018	15/5/2018
			Reporting Limit	Result	Result	Result	Result
Anions by Ion Chromatography in Water	Chloride	mg/L	0.05	21	46	60	96
Anions by Ion Chromatography in Water	Sulfate, SO4	mg/L	1	61	15	46	22
Metals in Water (Total) by ICPOES	Total Calcium	mg/L	0.1	16	52	16	85
Metals in Water (Total) by ICPOES	Total Magnesium	mg/L	0.1	10	10	9.8	13
Metals in Water (Total) by ICPOES	Total Sodium	mg/L	0.1	42	39	41	74
Metals in Water (Total) by ICPOES	Total Potassium	mg/L	0.2	3.0	7.4	2.1	2.0
Trace Metals (Total) in Water by ICPMS	Total Aluminium	µg/L	5	3,000	250	170	250
Trace Metals (Total) in Water by ICPMS	Total Iron	µg/L	5	8,700	400	420	840

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NATA Accredited Laboratory 9811
Corporate Site Number 18077
Accredited for compliance with ISO/IEC 17025

Robert Carr & Associates
92 Hill Street
CarringtonNSW2294

Attention: Robert Carr

Project:	RCA ref 11717-1601/1		
Date:	30/04/2017		
Client reference:	Coffs Harbour Bypass		
Received date:	21/04/2017	**Number of samples:**	2
Client order number:	N/A	**Testing commenced:**	21/04/2017

CERTIFICATE OF ANALYSIS

1 ANALYTICAL TEST METHODS

ANALYSIS	METHOD	UNITS	ANALYSING LABORATORY	NATA ANALYSIS / NON NATA	Measurement of Uncertainty Coverage Factor 2
pH	ENV-LAB006*	pH	RCA Laboratories - Environmental	NATA	±0.54
Total Dissolved Solids	ENV-LAB020*	mg/L	RCA Laboratories - Environmental	NATA	±11.48
Conductivity	ENV-LAB010*	µS/cm	RCA Laboratories - Environmental	NATA	±1.32
Turbidity	ENV-LAB037*	NTU	RCA Laboratories - Environmental	NATA	±4.88
Alkalinity**	ENV-LAB112	mg $CaCO_3$/L	RCA Laboratories - Environmental	NATA	±6.97
Sulphate**	ENV-LAB108	mg/L	RCA Laboratories - Environmental	NATA	±4.78
Dissolved Oxygen	ENV PC040	mg/L	RCA Laboratories - Environmental	NON NATA	-
Salinity	ENV PC040	%	RCA Laboratories - Environmental	NON NATA	-

* The analytical procedures used by RCA Laboratories - Environmental are based on established internationally recognised procedures such as APHA and Australian Standards

** Indicates NATA accreditation does not cover the performance of this service

This report cancels and supersedes the report No.11717-1601/0 issued by RCA Environmental Laboratory due to removal of a sample.

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2 RESULTS

ANALYSIS	UNITS	BHH114	BHH169
Water			
Sample Number	-	041711717002	041711717003
Date Sampled	-	20/4/2017	20/4/2017
Sampled By		JH	JH
pH Value	pH unit	6.37	6.56
Total Dissolved Solids	mg/L	199	
Conductivity	µS/cm	259	1174
Turbidity	NTU	2	<1
Hydroxide Alkalinity as CaCO3	mg/L	<1	<1
Carbonate Alkalinity as CaCO3	mg/L	<1	<1
Bicarbonate Alkalinity as CaCO3	mg/L	45	571
Total Alkalinity as CaCO3	mg/L	45	571
Sulphate as SO4 - Turbidimetric	mg/L	35	122
Dissolved Oxygen	mg/L		6.5
Salinity	%		0.08

** Indicates NATA accreditation does not cover the performance of this service

Shaded cells indicate analysis not required

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Water

NATA Scope of Accreditation does not cover the sampling of surface and ground waters by the client or by RCA.

Analysis on samples is on an as received basis.

3 QUALITY CONTROL RESULTS

Water Quality Control Sample Results

DATE	ANALYSIS	METHOD	UNITS	QUALITY CONTROL STANDARD VALUE	QUALITY CONTROL ACCEPTANCE CRITERIA	QUALITY CONTROL STANDARD RESULT
21/4/2017	pH	ENV-LAB006	pH	7.00	6.95 - 7.05	6.98
26/4/2017	Total Dissolved Solids	ENV-LAB020	mg/L	35	31.5 – 38.5	37
21/4/2017	Conductivity	ENV-LAB010	µS/cm	1413	1385 - 1441	1411
21/4/2017	Turbidity	ENV-LAB037	NTU	400	380 - 420	401
21/4/2017	Alkalinity	ENV-LAB023	mg $CaCO_3$/L	100	80 - 120	102
27/4/2017	Sulphate	ENV-LAB108	mg/L	25	17.5 – 32.5	30

Water Duplicate Analysis Results

SAMPLE NUMBER	DATE	ANALYSIS	METHOD	UNITS	LOR	SAMPLE RESULT	SAMPLE DUPLICATE RESULT
041711717001 BATCH	21/4/2017	pH	ENV-LAB006	pH	-	9.04	9.07
041711717001 BATCH	26/4/2017	Total Dissolved Solids	ENV-LAB020	mg/L	5	184	195
041711717001 BATCH	21/4/2017	Conductivity	ENV-LAB010	µS/cm	1	264	267
041711717001 BATCH	21/4/2017	Turbidity	ENV-LAB037	NTU	1	25	25
041711717002	21/4/2017	Hydroxide Alkalinity as CaCO3	ENV-LAB023	mg/L	1	<1	<1
041711717002	21/4/2017	Carbonate Alkalinity as CaCO3	ENV-LAB023	mg/L	1	<1	<1
041711717002	21/4/2017	Bicarbonate Alkalinity as CaCO3	ENV-LAB023	mg/L	1	45	47
041711717002	21/4/2017	Total Alkalinity as CaCO3	ENV-LAB023	mg/L	1	45	47
041711717001 BATCH	27/4/2017	Sulphate	ENV-LAB108	mg/L	1	25	26
041711717003	21/4/2017	Dissolved Oxygen	ENV PC040	mg/L	1	6.5	6.5
041711717003	21/4/2017	Salinity	ENV PC040	%	-	0.08	0.08

Please contact the undersigned if you have any queries.

Yours sincerely

Laura Schofield
Environmental Laboratory Manager
Robert Carr & Associates Pty Ltd Trading as
RCA Laboratories - Environmental
Environmental
Approved Signatory

Chad South
Environmental Technician
Robert Carr & Associates Pty Ltd Trading as
RCA Laboratories -
Approved Signatory

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RCA Internal Quality Review

General

1. Laboratory QC results for Method Blanks, Duplicates and Laboratory Control Samples are included in this QC report where applicable. Additional QC data maybe available on request.
2. RCA QC Acceptance / Rejection Criteria are available on request.
3. Proficiency Trial results are available on request.
4. Actual PQLs are matrix dependant. Quoted PQLs may be raised where sample extracts are diluted due to interferences.
5. When individual results are qualified in the body of a report, refer to the qualifier descriptions that follow.
6. Samples were analysed on an 'as received' basis.
7. Sampled dates in this report are those listed on the COC or sample jars; if no sample dates are noted, the date the samples are received at the laboratory have been used.
8. All soil results are reported on a dry basis, unless otherwise stated. (ACID SULPHATE SOILS)
9. This report replaces any interim results previously issued.

Holding Times.

For samples received on the last day of holding time, notification of testing requirements should have been received at least 6 hours prior to sample receipt deadlines as stated on the Sample

Receipt Acknowledgment.

If the Laboratory did not receive the information in the required timeframe, and regardless of any other integrity issues, suitably qualified results may still be reported.

Holding times apply from the date of sampling, therefore compliance to these may be outside the laboratory's control.

##NOTE: pH duplicates are reported as a range NOT as RPD

QC - ACCEPTANCE CRITERIA

RPD Duplicates: Global RPD Duplicates Acceptance Criteria is 30% however the following acceptance guidelines are equally applicable:

Results <10 times the LOR: No Limit

Results between 10-20 times the LOR: RPD must lie between 0-50%

Results >20 times the LOR: RPD must lie between 0-30%

QC DATA GENERAL COMMENTS

1. Where a result is reported as a less than (<), higher than the nominated LOR, this is due to either matrix interference, extract dilution required due to interferences or contaminant levels within the sample, high moisture content or insufficient sample provided.

2. Duplicate data shown within this report that states the word "BATCH" is a Batch Duplicate from outside of your sample batch, but within the laboratory sample batch at a 1:10 ratio. The Parent and Duplicate data shown is not data from your samples.

3. Duplicate RPD's are calculated from raw analytical data thus it is possible to have two sets of data.

Glossary

UNITS

mg/kg: milligrams per Kilogram

ug/L: micrograms per litre

ppm: Parts per million

ppb: Parts per billion

%: Percentage

org/100ml: Organisms per 100 millilitres

NTU: Units

MPN/100mL: Most Probable Number of organisms per 100 millilitres

mg/L: milligrams per Litre

TERMS

Dry Where moisture has been determined on a solid sample the result is expressed on a dry basis.

LOR Limit of Reporting.

RPD Relative Percent Difference between two Duplicate pieces of analysis can be obtained upon request.

QCS Quality Control Sample - reported as value recovery

Method Blank In the case of solid samples these are performed on laboratory certified clean sands.

In the case of water samples these are performed on de-ionised water.

Duplicate A second piece of analysis from the same sample and reported in the same units as the result to show comparison.

Batch Duplicate A second piece of analysis from a sample outside of the clients batch of samples but run within the laboratory batch of analysis.

USEPA United States Environment Protection Authority

APHA American Public Health Association

COC Chain of Custody

CP Client Parent - QC was performed on samples pertaining to this report

NCP Non-Client Parent - QC performed on samples not pertaining to this report, QC is representative of the sequence or batch that client samples were analysed within

< indicates less than

> Indicates greater than

ND Not Detected

RCA LABORATORIES
• ENVIRONMENTAL •
CONSTRUCTION MATERIALS TESTING

Ph: (02) 4902 9200 Fax: 02 4902 9299
92 Hill Street, Carrington NSW 2294
www.rca.com.au Email: labenviro@rca.com.au

ENV-F103-4

Client Name: RCA
Client Site: Coffs Harbour Bypass
Contact Name: RJC
Phone Number: 490294220
Email Report To: bobc@rca.com.au
Project Manager:

Turnaround Required: ☐ Urgent ☐ Standard (5 Day)
Date Required: standard
Expected Reporting Date:
(Laboratory Use Only)

ANALYSIS REQUIRED

Page of

RCA Job Number: 11717

SAMPLE INFORMATION

RCA Laboratories Environmental Sample Number	Client ID / Description	Date	Matrix	Total Samples	PH, EC DO Turbidiyt, Salinity	pH EC TDS Turbidity Alkalinity	Anions Cl SO4	Zn, Mn, Sulphite	Cations Ca, Mg, Na, K, Al, Fe	Alkalinity	Notes: invoice and report to above and administrator@rca.com.au
	BHH101				x		x		x	x	
	BHH104					x	x	x	x		
	BHH106				x		x		x	x	
	BHH109				x		x		x	x	
	BHH110	20/04/17	w	3		x	x	x	x		
	BHH111				x		x		x	x	
	BHH112				x		x		x	x	
	BHH113				x		x		x	x	
	BHH114	20/04/17	w	3		x	x	x	x		
	BHH115				x		x		x	x	
	BHH117				x		x		x	x	
	BHH119				x		x		x	x	
	BHH121				x		x		x	x	
	BHH123				x		x		x	x	
	BHH125				x		x		x	x	

RELINQUISHED BY		RECEIVED BY		Laboratory use only (circle appropriate)
Name:	Date: 21/4/17	Name: UOG	Date: 21/4/17	**Received in good condition:** Yes No
Of: RCA	Time:	Of: RCAVE	Time:	**Chilled:** Yes No

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CONSTRUCTION MATERIALS TESTING

Ph: (02) 4902 9200 Fax: 02 4902 9299
92 Hill Street, Carrington NSW 2294
www.rca.com.au Email: labenviro@rca.com.au

ENV-F103-4

Client Name: RCA
Client Site: Coffs Harbour Bypass

Contact Name: RJC
Phone Number: 490294220

Email Report To: bobc@rca.com.au
Project Manager:

Turnaround Required: ☐ Urgent ☐ Standard (5 Day)

Date Required:

Expected Reporting Date:
(Laboratory Use Only)

Page of

RCA Job Number: 11717

SAMPLE INFORMATION

ANALYSIS REQUIRED

RCA Laboratories Environmental Sample Number	Client ID / Description	Date	Matrix	Total Samples	PH, EC DO Turbidiyt, Salinity	pH EC TDS Turbidity Alkalinity	Anions Cl SO4	Zn, Mn, Sulphite	Cations Ca, Mg, Na, K, Al, Fe	Alkalinity							Notes: invoice and report to above and administrator@rca.com.au
	BHH151				x		x		x	x							
	BHH152				x		x		x	x							
	BHH153				x		x		x	x							
	BHH154					x	x	x	x								
	BHH158				x		x		x	x							
	BHH160				x		x		x	x							
	BHH163				x		x		x	x							
	BHH169	20/04/17	w	3	x		x		x	x							

RELINQUISHED BY		RECEIVED BY		Laboratory use only (circle appropriate)
Name:	Date:	Name:	Date:	**Received in good condition:** Yes No
Of:	Time:	Of:	Time:	**Chilled:** Yes No

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Accredited for compliance with ISO/IEC 17025

Robert Carr & Associates
92 Hill Street
CarringtonNSW2294

Attention: Robert Carr

Project:	RCA ref 11717-1602/0		
Date:	2/05/2017		
Client reference:	Coffs Harbour Bypass		
Received date:	28/04/2017	**Number of samples:**	4
Client order number:	N/A	**Testing commenced:**	28/04/2017

CERTIFICATE OF ANALYSIS

1 ANALYTICAL TEST METHODS

ANALYSIS	METHOD	UNITS	ANALYSING LABORATORY	NATA ANALYSIS / NON NATA	Measurement of Uncertainty Coverage Factor 2
pH	ENV-LAB006*	pH	RCA Laboratories - Environmental	NATA	±0.54
Total Dissolved Solids	ENV-LAB020*	mg/L	RCA Laboratories - Environmental	NATA	±11.48
Conductivity	ENV-LAB010*	µS/cm	RCA Laboratories - Environmental	NATA	±1.32
Turbidity	ENV-LAB037*	NTU	RCA Laboratories - Environmental	NATA	±4.88
Alkalinity**	ENV-LAB112	mg $CaCO_3$/L	RCA Laboratories - Environmental	NATA	±6.97
Sulphate**	ENV-LAB108	mg/L	RCA Laboratories - Environmental	NATA	±4.78
Dissolved Oxygen	ENV PC040	mg/L	RCA Laboratories - Environmental	NON NATA	-
Salinity	ENV PC040	%	RCA Laboratories - Environmental	NON NATA	-

***** The analytical procedures used by RCA Laboratories - Environmental are based on established internationally recognised procedures such as APHA and Australian Standards

****** Indicates NATA accreditation does not cover the performance of this service

2 RESULTS

ANALYSIS	UNITS	BHH104	BHH140	BHH144
Water				
Sample Number	-	041711717004	041711717006	041711717007
Date Sampled	-	27/4/2017	27/4/2017	27/4/2017
Sampled By		JH	JH	JH
pH Value	pH unit	6.42	7.17	7.87
Total Dissolved Solids	mg/L	437		210
Conductivity	µS/cm	707	370	279
Turbidity	NTU	<1	9	<1
Hydroxide Alkalinity as CaCO3	mg/L	<1	<1	<1
Carbonate Alkalinity as CaCO3	mg/L	<1	<1	11
Bicarbonate Alkalinity as CaCO3	mg/L	84	121	63
Total Alkalinity as CaCO3	mg/L	84	121	74
Sulphate as SO4 - Turbidimetric	mg/L	40	27	26
Dissolved Oxygen	mg/L		9.6	
Salinity	%		0.33	

ANALYSIS	UNITS	BHH147
Water		
Sample Number	-	041711717008
Date Sampled	-	27/4/2017
Sampled By		JH
pH Value	pH unit	6.37
Total Dissolved Solids	mg/L	
Conductivity	µS/cm	410
Turbidity	NTU	<1
Hydroxide Alkalinity as CaCO3	mg/L	<1
Carbonate Alkalinity as CaCO3	mg/L	<1
Bicarbonate Alkalinity as CaCO3	mg/L	69
Total Alkalinity as CaCO3	mg/L	69
Sulphate as SO4 - Turbidimetric	mg/L	121
Dissolved Oxygen	mg/L	2.5
Salinity	%	0.02

** Indicates NATA accreditation does not cover the performance of this service

Shaded cells indicate analysis not required

Water

NATA Scope of Accreditation does not cover the sampling of surface and ground waters by the client or by RCA.

Analysis on samples is on an as received basis.

3 QUALITY CONTROL RESULTS

Water Quality Control Sample Results

DATE	ANALYSIS	METHOD	UNITS	QUALITY CONTROL STANDARD VALUE	QUALITY CONTROL ACCEPTANCE CRITERIA	QUALITY CONTROL STANDARD RESULT
28/4/2017	pH	ENV-LAB006	pH	7.00	6.95 - 7.05	6.98
28/4/2017	Total Dissolved Solids	ENV-LAB020	mg/L	35	31.5 – 38.5	36
28/4/2017	Conductivity	ENV-LAB010	µS/cm	1413	1385 - 1441	1414
28/4/2017	Turbidity	ENV-LAB037	NTU	400	380 - 420	400
28/4/2017	Alkalinity	ENV-LAB023	mg $CaCO_3$/L	100	80 - 120	103
2/5/2017	Sulphate	ENV-LAB108	mg/L	25	17.5 – 32.5	27

Water Duplicate Analysis Results

SAMPLE NUMBER	DATE	ANALYSIS	METHOD	UNITS	LOR	SAMPLE RESULT	SAMPLE DUPLICATE RESULT
041711717004	28/4/2017	pH	ENV-LAB006	pH	-	6.42	6.45
041711717004	28/4/2017	Total Dissolved Solids	ENV-LAB020	mg/L	5	437	425
041711717004	28/4/2017	Conductivity	ENV-LAB010	µS/cm	1	707	704
041711717004	28/4/2017	Turbidity	ENV-LAB037	NTU	1	<1	<1
041711717006	28/4/2017	Hydroxide Alkalinity as CaCO3	ENV-LAB023	mg/L	1	<1	<1
041711717006	28/4/2017	Carbonate Alkalinity as CaCO3	ENV-LAB023	mg/L	1	<1	<1
041711717006	28/4/2017	Bicarbonate Alkalinity as CaCO3	ENV-LAB023	mg/L	1	121	121
041711717006	28/4/2017	Total Alkalinity as CaCO3	ENV-LAB023	mg/L	1	121	121
041711717005	2/5/2017	Sulphate	ENV-LAB108	mg/L	1	16	17
041711717004	28/4/2017	Dissolved Oxygen	ENV PC040	mg/L	1	4.2	4.4
041711717004	28/4/2017	Salinity	ENV PC040	%	-	0.03	0.03

Please contact the undersigned if you have any queries.

Yours sincerely

Laura Schofield
Environmental Laboratory Manager
Robert Carr & Associates Pty Ltd Trading as
RCA Laboratories - Environmental
Approved Signatory

Chad South
Environmental Technician
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RCA Internal Quality Review

General

1. Laboratory QC results for Method Blanks, Duplicates and Laboratory Control Samples are included in this QC report where applicable. Additional QC data maybe available on request.
2. RCA QC Acceptance / Rejection Criteria are available on request.
3. Proficiency Trial results are available on request.
4. Actual PQLs are matrix dependant. Quoted PQLs may be raised where sample extracts are diluted due to interferences.
5. When individual results are qualified in the body of a report, refer to the qualifier descriptions that follow.
6. Samples were analysed on an 'as received' basis.
7. Sampled dates in this report are those listed on the COC or sample jars; if no sample dates are noted, the date the samples are received at the laboratory have been used.
8. All soil results are reported on a dry basis, unless otherwise stated. (ACID SULPHATE SOILS)
9. This report replaces any interim results previously issued.

Holding Times.

For samples received on the last day of holding time, notification of testing requirements should have been received at least 6 hours prior to sample receipt deadlines as stated on the Sample

Receipt Acknowledgment.

If the Laboratory did not receive the information in the required timeframe, and regardless of any other integrity issues, suitably qualified results may still be reported.

Holding times apply from the date of sampling, therefore compliance to these may be outside the laboratory's control.

##NOTE: pH duplicates are reported as a range NOT as RPD

QC - ACCEPTANCE CRITERIA

RPD Duplicates: Global RPD Duplicates Acceptance Criteria is 30% however the following acceptance guidelines are equally applicable:

Results <10 times the LOR: No Limit

Results between 10-20 times the LOR: RPD must lie between 0-50%

Results >20 times the LOR: RPD must lie between 0-30%

QC DATA GENERAL COMMENTS

1. Where a result is reported as a less than (<), higher than the nominated LOR, this is due to either matrix interference, extract dilution required due to interferences or contaminant levels within the sample, high moisture content or insufficient sample provided.

2. Duplicate data shown within this report that states the word "BATCH" is a Batch Duplicate from outside of your sample batch, but within the laboratory sample batch at a 1:10 ratio. The Parent and Duplicate data shown is not data from your samples.

3. Duplicate RPD's are calculated from raw analytical data thus it is possible to have two sets of data.

Glossary

UNITS

mg/kg: milligrams per Kilogram

ug/L: micrograms per litre

ppm: Parts per million

ppb: Parts per billion

%: Percentage

org/100ml: Organisms per 100 millilitres

NTU: Units

MPN/100mL: Most Probable Number of organisms per 100 millilitres

mg/L: milligrams per Litre

TERMS

Dry Where moisture has been determined on a solid sample the result is expressed on a dry basis.

LOR Limit of Reporting.

RPD Relative Percent Difference between two Duplicate pieces of analysis can be obtained upon request.

QCS Quality Control Sample - reported as value recovery

Method Blank In the case of solid samples these are performed on laboratory certified clean sands.

In the case of water samples these are performed on de-ionised water.

Duplicate A second piece of analysis from the same sample and reported in the same units as the result to show comparison.

Batch Duplicate A second piece of analysis from a sample outside of the clients batch of samples but run within the laboratory batch of analysis.

USEPA United States Environment Protection Authority

APHA American Public Health Association

COC Chain of Custody

CP Client Parent - QC was performed on samples pertaining to this report

NCP Non-Client Parent - QC performed on samples not pertaining to this report, QC is representative of the sequence or batch that client samples were analysed within

< indicates less than

> Indicates greater than

ND Not Detected

ANALYTICAL REPORT

Accreditation No. 2562

CLIENT DETAILS		LABORATORY DETAILS	
Contact	Laura Schofield	Manager	Huong Crawford
Client	RCA AUSTRALIA	Laboratory	SGS Alexandria Environmental
Address	PO BOX 175 NSW 2294	Address	Unit 16, 33 Maddox St Alexandria NSW 2015
Telephone	61 2 49029200	Telephone	+61 2 8594 0400
Facsimile	61 2 49029299	Facsimile	+61 2 8594 0499
Email	lauras@rca.com.au	Email	au.environmental.sydney@sgs.com
Project	**11717 - Coffs Harbour Bypass**	SGS Reference	**SE179432 R0**
Order Number	(Not specified)	Date Received	22 May 2018
Samples	26	Date Reported	28 May 2018

COMMENTS

Accredited for compliance with ISO/IEC 17025 - Testing. NATA accredited laboratory 2562(4354).

SIGNATORIES

Dong Liang
Metals/Inorganics Team Leader

Kamrul Ahsan
Senior Chemist

Parameter	Units	LOR	SE179432.001	SE179432.002	SE179432.003	SE179432.004
Sample Matrix			Water	Water	Water	Water
Sample Date			17 May 2018	14 May 2018	16 May 2018	16 May 2018
Sample Name			051811717001 BHH101	051811717002 BHN104	051811717003 BHH106	051811717004 BHH109
Anions by Ion Chromatography in Water Method: AN245 Tested: 24/5/2018						
Chloride	mg/L	0.05	25	20	17	20
Sulfate, SO4	mg/L	1	8.9	14	34	36
Metals in Water (Total) by ICPOES Method: AN022/AN320 Tested: 24/5/2018						
Total Calcium	mg/L	0.1	19	64	4.2	5.8
Total Magnesium	mg/L	0.1	13	6.9	3.1	5.2
Total Potassium	mg/L	0.2	5.3	1.7	1.8	1.6
Total Sodium	mg/L	0.1	51	20	35	30
Trace Metals (Total) in Water by ICPMS Method: AN022/AN318 Tested: 23/5/2018						
Total Aluminium	μg/L	5	3200	390	81	230
Total Iron	μg/L	5	7100	1600	390	320

Parameter	Units	LOR	SE179432.005	SE179432.006	SE179432.007	SE179432.008
Sample Matrix			Water	Water	Water	Water
Sample Date			16 May 2018	16 May 2018	16 May 2018	16 May 2018
Sample Name			051811717005 BHH110	051811717006 BHH111	051811717007 BHH112	051811717008 BHH113

Anions by Ion Chromatography in Water Method: AN245 Tested: 24/5/2018

Chloride	mg/L	0.05	24	22	44	28
Sulfate, SO4	mg/L	1	17	87	42	16

Metals in Water (Total) by ICPOES Method: AN022/AN320 Tested: 24/5/2018

Total Calcium	mg/L	0.1	8.7	68	38	5.7
Total Magnesium	mg/L	0.1	6.7	25	18	4.6
Total Potassium	mg/L	0.2	2.7	6.7	4.9	2.0
Total Sodium	mg/L	0.1	24	39	49	27

Trace Metals (Total) in Water by ICPMS Method: AN022/AN318 Tested: 23/5/2018

Total Aluminium	µg/L	5	1800	1500	1700	880
Total Iron	µg/L	5	3100	1800	2100	440

Parameter	Units	LOR	SE179432.009	SE179432.010	SE179432.011	SE179432.012
Sample Matrix			Water	Water	Water	Water
Sample Date			16 May 2018	16 May 2018	16 May 2018	16 May 2018
Sample Name			051811717009 BHH114	051811717010 BHH115	051811717011 BHH117	051811717012 BHH119
Anions by Ion Chromatography in Water Method: AN245 Tested: 24/5/2018						
Chloride	mg/L	0.05	15	54	19	26
Sulfate, SO4	mg/L	1	32	39	64	17
Metals in Water (Total) by ICPOES Method: AN022/AN320 Tested: 24/5/2018						
Total Calcium	mg/L	0.1	23	47	16	40
Total Magnesium	mg/L	0.1	20	8.7	10	7.8
Total Potassium	mg/L	0.2	8.1	2.8	3.7	5.3
Total Sodium	mg/L	0.1	19	37	41	51
Trace Metals (Total) in Water by ICPMS Method: AN022/AN318 Tested: 23/5/2018						
Total Aluminium	μg/L	5	39000	140	4200	170
Total Iron	μg/L	5	34000	580	4200	790

Parameter	Units	LOR	SE179432.013	SE179432.014	SE179432.015	SE179432.016
Sample Matrix			Water	Water	Water	Water
Sample Date			16 May 2018	17 May 2018	17 May 2018	17 May 2018
Sample Name			051811717013 BHH121	051811717014 BHH123	051811717015 BHH125	051811717016 BHH127
Anions by Ion Chromatography in Water Method: AN245 Tested: 24/5/2018						
Chloride	mg/L	0.05	32	39	13	25
Sulfate, SO4	mg/L	1	17	44	32	39
Metals in Water (Total) by ICPOES Method: AN022/AN320 Tested: 24/5/2018						
Total Calcium	mg/L	0.1	5.6	9.1	8.7	11
Total Magnesium	mg/L	0.1	5.5	8.8	9.4	11
Total Potassium	mg/L	0.2	1.3	3.9	9.5	5.6
Total Sodium	mg/L	0.1	37	38	30	45
Trace Metals (Total) in Water by ICPMS Method: AN022/AN318 Tested: 23/5/2018						
Total Aluminium	µg/L	5	1400	1200	6100	4100
Total Iron	µg/L	5	3400	1900	10000	6300

Parameter	Units	LOR	SE179432.017	SE179432.018	SE179432.019	SE179432.020
Sample Matrix			Water	Water	Water	Water
Sample Date			16 May 2018	16 May 2018	16 May 2018	15 May 2018
Sample Name			051811717017 BHH130	051811717018 BHH131	051811717019 BHH132	051811717020 BHH140
Anions by Ion Chromatography in Water Method: AN245 Tested: 24/5/2018						
Chloride	mg/L	0.05	16	16	31	20
Sulfate, SO4	mg/L	1	35	18	76	17
Metals in Water (Total) by ICPOES Method: AN022/AN320 Tested: 24/5/2018						
Total Calcium	mg/L	0.1	3.7	4.4	81	22
Total Magnesium	mg/L	0.1	13	16	13	6.3
Total Potassium	mg/L	0.2	8.0	9.1	4.5	1.6
Total Sodium	mg/L	0.1	22	20	28	25
Trace Metals (Total) in Water by ICPMS Method: AN022/AN318 Tested: 23/5/2018						
Total Aluminium	μg/L	5	15000	17000	2000	470
Total Iron	μg/L	5	20000	26000	8100	440

Parameter	Units	LOR	SE179432.021	SE179432.022	SE179432.023	SE179432.024
Sample Matrix			Water	Water	Water	Water
Sample Date			15 May 2018	15 May 2018	15 May 2018	15 May 2018
Sample Name			051811717021 BHH144	051811717022 BHH147	051811717023 BHH153	051811717024 BHH158
Anions by Ion Chromatography in Water Method: AN245 Tested: 24/5/2018						
Chloride	mg/L	0.05	15	21	46	60
Sulfate, SO4	mg/L	1	15	61	15	46
Metals in Water (Total) by ICPOES Method: AN022/AN320 Tested: 24/5/2018						
Total Calcium	mg/L	0.1	26	16	52	16
Total Magnesium	mg/L	0.1	4.9	10	10	9.8
Total Potassium	mg/L	0.2	2.6	3.0	7.4	2.1
Total Sodium	mg/L	0.1	19	42	39	41
Trace Metals (Total) in Water by ICPMS Method: AN022/AN318 Tested: 23/5/2018						
Total Aluminium	µg/L	5	4200	3000	250	170
Total Iron	µg/L	5	1900	8700	400	420

Parameter	Units	LOR	SE179432.025	SE179432.026
Sample Matrix			Water	Water
Sample Date			15 May 2018	14 May 2018
Sample Name			051811717025 BHH160	051811717026 BHH169
Anions by Ion Chromatography in Water Method: AN245 Tested: 24/5/2018				
Chloride	mg/L	0.05	96	120
Sulfate, SO4	mg/L	1	22	120
Metals in Water (Total) by ICPOES Method: AN022/AN320 Tested: 24/5/2018				
Total Calcium	mg/L	0.1	85	48
Total Magnesium	mg/L	0.1	13	88
Total Potassium	mg/L	0.2	2.0	37
Total Sodium	mg/L	0.1	74	150
Trace Metals (Total) in Water by ICPMS Method: AN022/AN318 Tested: 23/5/2018				
Total Aluminium	µg/L	5	250	2800
Total Iron	µg/L	5	840	3000

QC SUMMARY

MB blank results are compared to the Limit of Reporting
LCS and MS spike recoveries are measured as the percentage of analyte recovered from the sample compared the the amount of analyte spiked into the sample.
DUP and MSD relative percent differences are measured against their original counterpart samples according to the formula : *the absolute difference of the two results divided by the average of the two results as a percentage.* Where the DUP RPD is 'NA' , the results are less than the LOR and thus the RPD is not applicable.

Anions by Ion Chromatography in Water   Method: ME-(AU)-[ENV]AN245

Parameter	QC Reference	Units	LOR	MB	DUP %RPD	LCS %Recovery
Chloride	LB148551	mg/L	0.05	<0.05	1%	97%
Sulfate, SO4	LB148551	mg/L	1	<1.0	1%	95%

Metals in Water (Total) by ICPOES   Method: ME-(AU)-[ENV]AN022/AN320

Parameter	QC Reference	Units	LOR	MB	DUP %RPD	LCS %Recovery	MS %Recovery
Total Calcium	LB148516	mg/L	0.1	<0.1	0 - 1%	100%	100%
Total Magnesium	LB148516	mg/L	0.1	<0.1	0 - 1%	101%	101%
Total Potassium	LB148516	mg/L	0.2	<0.2	0 - 2%	101%	108%
Total Sodium	LB148516	mg/L	0.1	<0.1	0 - 2%	105%	121%

Trace Metals (Total) in Water by ICPMS   Method: ME-(AU)-[ENV]AN022/AN318

Parameter	QC Reference	Units	LOR	MB	DUP %RPD	LCS %Recovery	MS %Recovery
Total Aluminium	LB148485	μg/L	5	<5	11%	113%	452 - 983%
Total Iron	LB148485	μg/L	5	<5	5%	NA	NA

METHOD SUMMARY

METHOD	METHODOLOGY SUMMARY
AN022	The water sample is digested with Nitric Acid and made up to the original volume similar to APHA3030E.
AN022/AN318	Following acid digestion of un filtered sample, determination of elements at trace level in waters by ICP-MS technique, in accordance with USEPA 6020A.
AN022/AN320	Total (acid soluble) Metals by ICP-OES: Samples are digested in nitric or nitric and hydrochloric acids prior to analysis for a wide range of metals and some non-metals. This solution is measured by Inductively Coupled Plasma. Solutions are aspirated into an argon plasma at 8000-10000K and emit characteristic energy or light as a result of electron transitions through unique energy levels. The emitted light is focused onto a diffraction grating where it is separated into components.
AN245	Anions by Ion Chromatography: A water sample is injected into an eluent stream that passes through the ion chromatographic system where the anions of interest ie Br, Cl, NO2, NO3 and SO4 are separated on their relative affinities for the active sites on the column packing material. Changes to the conductivity and the UV-visible absorbance of the eluent enable identification and quantitation of the anions based on their retention time and peak height or area. APHA 4110 B
AN320	Photomultipliers or CCDs are used to measure the light intensity at specific wavelengths. This intensity is directly proportional to concentration. Corrections are required to compensate for spectral overlap between elements. Reference APHA 3120 B.

FOOTNOTES

IS	Insufficient sample for analysis.	LOR	Limit of Reporting
LNR	Sample listed, but not received.	↑↓	Raised or Lowered Limit of Reporting
*	NATA accreditation does not cover the performance of this service.	QFH	QC result is above the upper tolerance
**	Indicative data, theoretical holding time exceeded.	QFL	QC result is below the lower tolerance
		-	The sample was not analysed for this analyte
		NVL	Not Validated

Samples analysed as received.
Solid samples expressed on a dry weight basis.

Where "Total" analyte groups are reported (for example, Total PAHs, Total OC Pesticides) the total will be calculated as the sum of the individual analytes, with those analytes that are reported as <LOR being assumed to be zero. The summed (Total) limit of reporting is calcuated by summing the individual analyte LORs and dividing by two. For example, where 16 individual analytes are being summed and each has an LOR of 0.1 mg/kg, the "Totals" LOR will be 1.6 / 2 (0.8 mg/kg). Where only 2 analytes are being summed, the " Total" LOR will be the sum of those two LORs.

Some totals may not appear to add up because the total is rounded after adding up the raw values.

If reported, measurement uncertainty follow the ± sign after the analytical result and is expressed as the expanded uncertainty calculated using a coverage factor of 2, providing a level of confidence of approximately 95%, unless stated otherwise in the comments section of this report.

Results reported for samples tested under test methods with codes starting with ARS-SOP, radionuclide or gross radioactivity concentrations are expressed in becquerel (Bq) per unit of mass or volume or per wipe as stated on the report. Becquerel is the SI unit for activity and equals one nuclear transformation per second.
Note that in terms of units of radioactivity:

a. 1 Bq is equivalent to 27 pCi
b. 37 MBq is equivalent to 1 mCi

For results reported for samples tested under test methods with codes starting with ARS-SOP, less than (<) values indicate the detection limit for each radionuclide or parameter for the measurement system used. The respective detection limits have been calculated in accordance with ISO 11929.

The QC criteria are subject to internal review according to the SGS QAQC plan and may be provided on request or alternatively can be found here : http://www.sgs.com.au/~/media/Local/Australia/Documents/Technical%20Documents/MP-AU-ENV-QU-022%20QA%20QC%20Plan.pdf

Appendix O

Flooding and hydrology assessment

Roads and Maritime Services

Coffs Harbour Bypass

Flooding and Hydrology assessment

FLD01

Issue |

Job number 248379

Arup Pty Ltd ABN 18 000 966 165

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Level 4, 108 Wickham Street
Fortitude Valley
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GPO Box 685 Brisbane QLD 4001
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ARUP

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1 Introduction

1.1 Project overview

Roads and Maritime is seeking approval for the project under Division 5.2 of the EP&A Act as critical State significant infrastructure (CSSI).

The project includes a 12 km bypass of Coffs Harbour from south of Englands Road to Korora Hill in the north and a two-kilometre upgrade of the existing highway between Korora Hill and Sapphire. The project would provide a four-lane divided highway that bypasses Coffs Harbour, passing through the North Boambee Valley, Roberts Hill ridge and then traversing the foothills of the Coffs Harbour basin to the west and north to Korora Hill. Figure 1 illustrates the project extents.

The key features of the project include:

- Four-lane divided highway from south of Englands Road roundabout to the dual carriageway highway at Sapphire
- Bypass of the Coffs Harbour urban area from south of Englands Road intersection to Korora Hill
- Upgrade of the existing Pacific Highway between Korora Hill and the dual carriageway highway at Sapphire
- Grade-separated interchanges at Englands Road, Coramba Road and Korora Hill
- A one-way local access road along the western side of the project between the southern tie-in and Englands Road, connecting properties to the road network via Englands Road
- A new service road, located east of the project, connecting Solitary Islands Way with James Small Drive and the existing Pacific Highway near Bruxner Park Road
- Three tunnels through ridges at Roberts Hill (around 190 m long), Shephards Lane (around 360 m long), and Gatelys Road (around 450 m long)
- Structures to pass over local roads and creeks as well as a bridge over the North Coast Railway
- A series of cuttings and embankments along the project
- Tie-ins and modifications to the local road network to enable local road connections across and around the alignment
- Pedestrian and cycling facilities, including a shared path along the service road tying into the existing shared path on Solitary Islands Way, and a new pedestrian bridge to replace the existing Luke Bowen footbridge with the name being retained
- Relocation of the Kororo Public School bus interchange

- Noise attenuation, including low noise pavement, noise barriers and at-property treatments as required
- Fauna crossing structures including glider poles, underpasses and fencing
- Ancillary work to facilitate construction and operation of the project, including:
  - Adjustment, relocation and/or protection of utilities and services
  - New or adjusted property accesses as required
  - Operational water quality measures and retention basins
  - Temporary construction facilities and work including compound and stockpile sites, concrete/asphalt batching plant, sedimentation basins and access roads (if required).

Legend

- Construction footprint
- Alignment
- North Coast Railway
- Sensitive receiver
- Watercourse

Coffs Harbour Bypass
Key features of the project
Figure 1-1

NOTE: Elements are indicative only for the purpose of project planning and may change prior to construction.

0 0.5 1 1.5 km

Scale @A4: 1:50,000
GDA 1994 MGA Zone 56

N

1.2 Purpose of this report

This technical report has been prepared to provided details of the methods and processes undertaken to address specific Secretary's Environmental Assessment Requirements (SEARs) for flooding and hydrology and to provide a detailed analysis for input into the EIS.

The SEARs relevant to hydrology and flooding are contained within **Table 1**. A number of these requirements also require assessment with regard to groundwater and surface water quality.

Table 1: Relevant SEARs

Key Issue & Requirement	Location
11. Water - Hydrology 1. The Proponent must describe (and map) the existing hydrological regime for any surface and groundwater resource (including reliance by users and for ecological purposes) likely to be impacted by the project, including stream orders, as per the FBA.	Section 2.1 Section 3 Section 4 Section 5
2. The Proponent must assess (and model if appropriate) the impact of the construction and operation of the project and any ancillary facilities (both built elements and discharges) on surface and groundwater hydrology in accordance with the current guidelines, including:	
(a) natural processes within rivers, wetlands, estuaries, marine waters and floodplains that affect the health of the fluvial, riparian, estuarine or marine system and landscape health (such as modified discharge volumes, durations and velocities), aquatic connectivity and access to habitat for spawning and refuge;	Section 4.1 Section 5.1
(d) direct or indirect increases in erosion, siltation, destruction of riparian vegetation or a reduction in the stability of river banks or watercourses;	Section 4 Section 5.1
(e) minimising the effects of proposed stormwater and wastewater management during construction and operation on natural hydrological attributes (such as volumes, flow rates, management methods and re - use options) and on the conveyance capacity of existing stormwater systems where discharges are proposed through such systems; and	Section 4 Section 5
12. Flooding 1. The Proponent must assess (and model where required) the impacts from the project on flood behaviour, in particular Coffs Creek, during the construction and operation for a full range of flood events up to the probable maximum flood (taking into account sea level rise and storm intensity due to climate change) including:	
(a) Any detrimental increases in the potential flood affectation of the project infrastructure and other properties, assets and infrastructure;	Section 4.1 Section 5.3 Section 6
(b) Consistency (or inconsistency) with applicable Council floodplain risk management plans;	Section 5.3
(c) Compatibility with the flood hazard of the land;	Section 4 Section 5.3 Section 6

Key Issue & Requirement	Location
(d) Compatibility with the hydraulic functions of flow conveyance in flood ways and storage areas of the land;	Section 4 Section 5.3 Section 6
(e) Whether there will be adverse effect to beneficial inundation of the floodplain environment, on, or adjacent to or downstream of the site;	Section 5.3 Section 6
(f) Downstream velocity and scour potential;	Section 5.3
(g) Impacts the project may have upon existing community emergency management arrangements for flooding, including Council's upper catchment detention basins. These matters must be discussed with the State Emergency Services and Coffs Harbour City Council;	Section 4.5 Section 5.3
(h) Any impacts the project may have on the social and economic costs to the community as consequence of flooding;	Section 5.3
(i) Whether there will be direct or indirect increase in erosion, siltation, destruction of riparian vegetation or a reduction in the stability of river banks or watercourses; and	Section 4.1 Section 4.2 Section 5.3
(j) Any mitigation measures required to offset potential flood risks attributable to the project.	Section 4 Section 5

1.3 Study area

The project is located within the Coffs Harbour City Council (CHCC) local government area (LGA).

Key drainage features of the study area are two topographic zones. These include, a hillside zone (areas above the 50m contour) and the lowland area (areas below 50m contour).

The hillside zone comprises steep slopes and ridges which rise to about 150-250m AHD. Major ridge lines project from the Great Dividing Range such as the prominent ridge to the south of Coramba that ends at Roberts Hill. Numerous drainage channels that typically flow east to the lowland area, incise the hillside area. Most of the steep slopes and ridges are either forested or used for banana cultivation.

The lowland area is characterised by low undulating residual hills with gentle gradients and alluvial floodplains including backswamps and dunes.

The project covers several catchments which predominantly drain from the western ridges of the Great Dividing Range towards the Pacific Ocean, as illustrated in **Figure 2.**

The catchments have been grouped by locality and relate to the creeks and watercourses to which they drain. They also relate to catchments as they are defined within existing flood models.

The project catchments as listed below are referred to throughout the report as, North Boambee Valley, Coffs Creek and northern creeks. The primary waterways within each catchment are:

- North Boambee Valley:
  - Tributary of Boambee Creek
  - Newports Creek.
- Coffs Creek:
  - Coffs Creek
  - Treefern Creek.
- Northern creeks:
  - Jordans Creek
  - Kororo Basin – Kororo Basin is a catchment located south east of the Pine Brush Creek, it is not related to the Korora, which is located in the upper catchment area of Pine Brush Creek
  - Pine Brush Creek
  - Sapphire Beach – this relates to an unnamed waterway at this location.

1.3.1 North Boambee valley

The catchment drains from the west to the Pacific Ocean via Boambee Creek and Newports Creek. The combined Boambee and Newports Creek catchment area is about 50 km^2.

The existing Pacific Highway crosses Newports Creek and is affected by the 1 per cent AEP flood event (GHD, 2016).

The upper catchment to the west is primarily steep and densely vegetated. The middle and lower catchment areas are characterised by a large floodplain and become more urbanised towards the coastline in the east.

1.3.2 Coffs Creek

The catchment drains from the west to the Pacific Ocean via Coffs Creek, Treefern Creek and other unnamed tributaries. It generally drains through natural channels surrounded by urban areas. Coffs Creek converges west of the Pacific Highway and forms an estuary at the coast.

The catchment area is about 25 km^2 and consists of a flat coastal floodplain from the Pacific Ocean to the east rising to a steep escarpment in the west. This terrain is conducive to orographic effects, quickly rising from 10 to 500 mAHD. About 23 per cent of the catchment is densely vegetated, 33 per cent grazing and farmland, with the remainder urban (GeoLINK, 2015).

The Coffs Creek catchment is prone to flash flooding due to the steep upper terrain and a relatively high level of urban development within the floodplain (BMT WBM 2018).

1.3.3 Northern Creeks

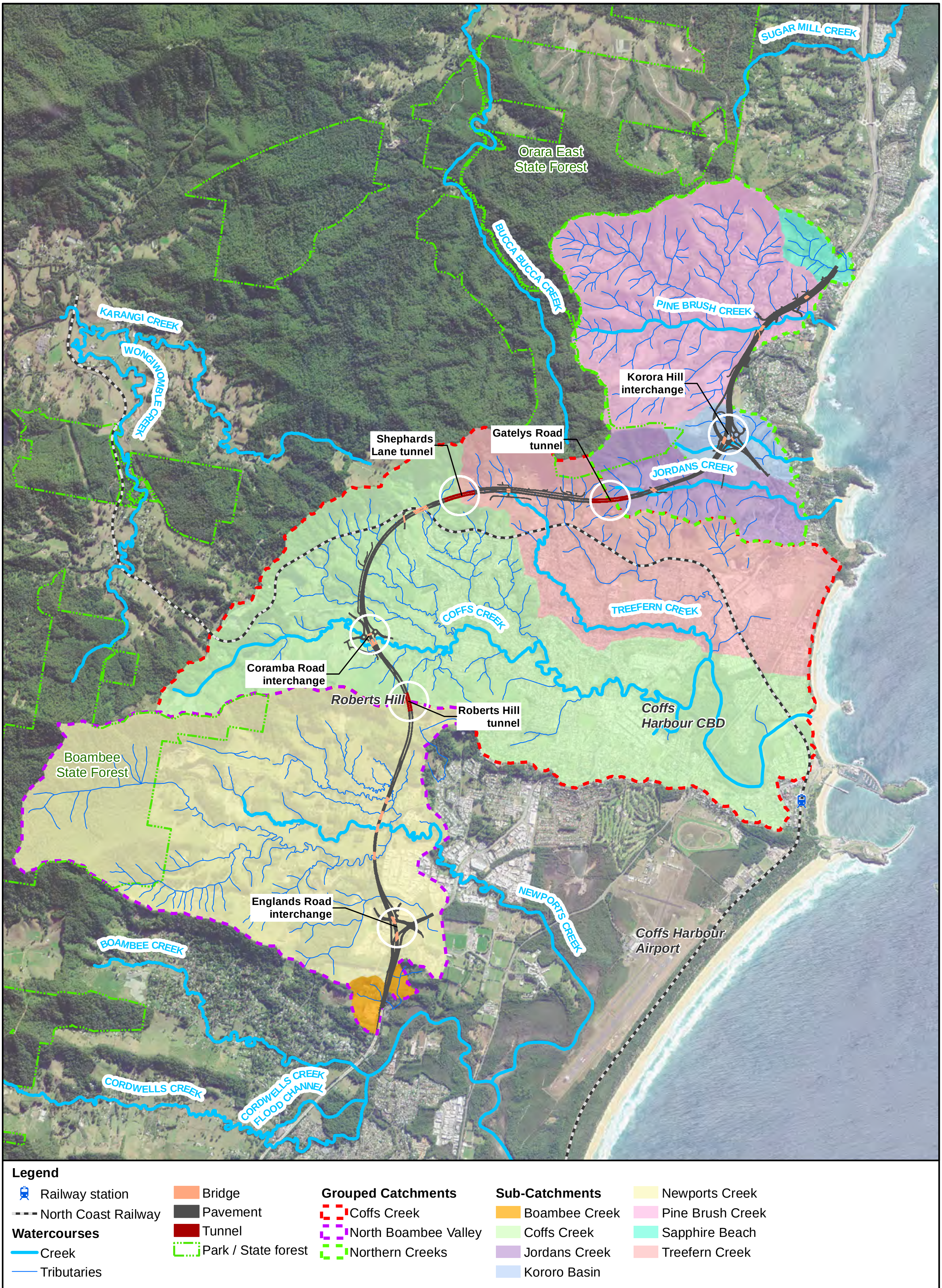
The combined catchment named Northern creeks drains to the Pacific Ocean via a number of creeks and watercourses. These are, Kororo Basin, Jordans Creek, Pine Brush Creek and an unnamed waterway at Sapphire Beach. The total area of the Northern Catchments is about 13 $km^{2.}$

The catchment is divided into four sub catchments, which reflect the creeks and waterways to which they drain. The defined sub-catchments and their areas are listed in **Table 2**.

Table 2: Northern creeks sub-catchments

Sub-catchment	Total area (km^2)
Jordans Creek	2.7
Kororo Basin	1.4
Pine Brush	8.4
Sapphire Beach	0.5

All sub-catchments flow from steep terrain in the west, in an easterly direction towards the coastline. Land use within the catchment area consist of about 40 per cent dense bushland, 50 per cent pastural and the remainder urban (primarily in the lower regions of the catchment).



Coffs Harbour Bypass
Study area
Figure 2

1.4 Terminology

Specific flooding and hydrology terms used in this report are defined in **Table 3**.

Table 3: Glossary of terms

Term	Definition
Afflux	Predicted increase in developed peak flood level relative to the existing condition
Australian Height Datum (AHD)	Standard height above the average sea level at which a flood level is measured
Average recurrence interval (ARI)	Average number of years between exceedances of a flood event of the same size
Annual exceedance probability (AEP)	Percent likelihood a flood event of a certain size will occur within any one year
Climate change	Predicted future rainfall intensities and sea levels affecting flood behaviour
BMT WBM	The developers of the TUFLOW flood modelling software
Developed case	Operational phase with the project in place (post-construction).
Detention basin	Excavated (or bunded) land to increase floodplain storage, with an outlet designed to attenuate flows and decrease flooding downstream
Existing case	Existing conditions without the project in place (pre-construction).
Finished Floor Level (FFL)	Existing internal floor elevation of a structure
Hydrologic model	Represents catchment rainfall-runoff processes. Runoff generation are modelled at the sub-catchment scale and resulting runoff hydrographs are routed along catchment stream reaches and storages
Hydraulic model	Simulates conveyance to predict characteristics such as flood level and velocity, based on hydrologically derived inflows
Intensity frequency duration (IFD)	Design event storm parameters provided by BoM based on statistical analysis of historic events
Manning's '*n*' roughness	An empirically derived coefficient, generally representative of the hydraulic roughness of a surface ($s/m^{1/3}$)
Orographic effect (or rainfall gradient)	The influence of mountainous topography on rainfall patterns, dependant on surface gradients, wind direction and storm sources, which may concentrate rainfall
Probable maximum flood (PMF)	The worst-case flood event that could possibly occur based on Probable maximum precipitation (PMP) and the most extreme catchment conditions
TUFLOW	The name of the hydraulic (flood) modelling software used in this study
XP-RAFTS	The name of the hydraulic (flood) modelling software used in this study

1.5 Design event nomenclature

The report adopts design flood nomenclature in terms of AEP, as detailed in *Australian Rainfall and Runoff* (ARR) (Ball, et al., 2016). **Table 4** presents the relationship between ARI and AEP for a range of design events.

Table 4: Design event nomenclature

AEP (%)	AEP (1 in x)	ARI (year)
50	2	1.44
39.35	2.54	2
20	5	4.48
18.13	5.52	5
10	10	9.49
5	20	20
2	50	50
1	100	100
0.5	200	200
0.2	500	500
0.05	2000	2000

1.6 Policy context and legislative framework

In addition to the SEARs set out in Section 1.2, there are local, State and National legislation, policies and guidelines which are relevant to the project.

The policies, guidelines and legislation used for the assessment of hydrology and flooding are summarised in **Table 5**. The table also details the relevance of each document to the project and this report.

Table 5: Relevant legislation, policies, and guidelines

Level	Legislation/Policy/Guideline	Relevance
National	*Australian Rainfall and Runoff* (ARR) (Pilgrim, 1987) (Ball, et al., 2016)	National guideline for design flood estimation.
	Managing the Floodplain: A Guide to Best Practice in Flood Risk Management in Australia (AIDR, 2017)	Developed with consideration of the *National Strategy for Disaster Resilience* (COAG, 2011) and intended to provide broad guidance on all aspects of managing flood risk.

Level	Legislation/Policy/Guideline	Relevance
State	*Floodplain Development Manual* (DIPNR, 2005)	This manual details methods which aim to reduce the impact of flooding and flood liability while recognising the benefits of the use, occupation, and development of flood prone land. It does this by promoting a merit approach to balance social, economic, environmental, and flood risk parameters. The manual defines the categorisation of flood risk in NSW. This manual is nominated under the project SEAR for flooding as relevant for consideration. The methods contained with the manual have been used to inform the development of the project specific Flood Plane Management Objectives against which the project impacts have been assessed. Details of this method are contained within Section 1.7 of this report.
	Practical Consideration of Climate Change – Flood Risk Management Guideline (DECC, 2007)	Assists flood consultants and councils in the preparation and implementation of flood risk management plans with climate change considerations. This guideline has also been nominated under the project SEARs for flooding as relevant for consideration. The methods contained with the manual have been used to inform the development of the project specific Flood Plane Management Objectives against which the project impacts have been assessed. Details of this method are contained within Section 1.7 of this report.
	NSW 2021: A Plan to Make NSW Number One (DPC, 2011)	Presents the strategy for the decade, including priority actions to increase the capacity to prepare for, prevent, respond to, and recover from future extreme weather events and hazards. The methods contained with the manual have been used to inform the development of the project specific Flood Plane Management Objectives against which the project impacts have been assessed. Details of this method are contained within Section 1.7 of this report.
	Upgrading the Pacific Highway – Design Guidelines (Roads and Maritime, 2015)	Detail of design guidelines relevant to the project, including hydraulic design criteria The methods contained with the manual have been used to inform the development of the project specific Flood Plane Management Objectives against which the project impacts have been assessed. Details of this method are contained within Section 1.7 of this report.

Level	Legislation/Policy/Guideline	Relevance
	North Coast Regional Plan 2036 (DPE, 2017)	Encompasses goals aimed towards delivering greater prosperity in the region. It specifically aims to manage natural hazards and climate change by identifying, avoiding, and managing vulnerable areas and hazards. It also calls for action to review and update floodplain risk, particularly where urban growth is being considered.
Local	*Coffs Creek Floodplain Risk Management Plan* (Bewsher Consulting, 2005)	A result of the Floodplain Risk Management Study commissioned by CHCC, recommends floodplain management improvements for the Coffs Creek Floodplain. This management plan has also been nominated under the project SEARs for flooding as relevant for consideration. The methods contained with the manual have been used to inform the development of the project specific Flood Plane Management Objectives against which the project impacts have been assessed. Details of this method are contained within Section 1.7 of this report.
	Coffs Harbour Local Environmental Plan 2013 (NSW Government, 2013)	Aims to make local environmental planning provisions for land in Coffs Harbour in accordance with the relevant standard environmental planning instrument, and specifically to minimize the exposure of development to natural hazards and natural risks.
	Floodplain Development and Management Policy (CHCC, 2017)	Standard for flood assessment in the Coffs Harbour LGA and is supported by the EPA Act. Sets policy to minimise flood risk and effects of development
	Coffs Harbour Local Flood Plan (SES, 2017)	Details the flood preparedness, response and recovery procedures for the occurrence of a significant storm event.

1.7 Project floodplain management objectives

Based on the documents referenced in Section 1.6, the project SEARs, project floodplain management objectives have been developed similar to objectives established for other Pacific Highway upgrade projects and other major Roads and Maritime projects.

The project floodplain management objectives have been defined for two areas of project infrastructure management objectives (elements within the project construction boundary) and external to the construction footprint management objectives. The objectives are listed in **Table 6**.

Table 6: Project floodplain management objectives

Project infrastructure	
Element	**Criteria**
Alignment	1% AEP flood immunity for proposed main carriageway and 5% AEP for ramps and interchanges
Tunnel portals	Above the PMF or the 1% AEP flood level +0.5 m (whichever is greater), where ingress of floodwaters would collect at the sag in the tunnel
Waterway crossings	Bridge soffits >0.5 m above 1% AEP flood level. Appropriate scour protection designed for areas at risk of scour due to the project to ensure long term bed and bank stability
Construction	Potential impact of ancillary site locations is identified, to ensure appropriate flood risk assessment of vulnerable sites and to inform a future construction flood management plan
External to construction footprint	
Element	**Criteria**
Level	A merit-based approach, considering the relative impact to peak flood level, hazard, extent and potential damages. In general, the following afflux criteria is applied for design events up to the 1% AEP: <10 mm for residential, commercial and industrial areas and buildings affected by FFL inundation; <50 mm for agricultural land; and <250 mm pastural, forest and recreational areas.
Scour	No adverse increase in peak flood velocity for design events (up to 1% AEP)
Access	All affected existing local and access roads are to be ultimately configured (where feasible during construction) such that the existing level of flood immunity, inundation duration and available evacuation time is maintained or improved (subject to CHCC and stakeholder consultation)
Direction	No change to flow direction / receiving catchment except for constriction into and expansion out of discrete openings (culverts and bridges) and constructed diversions.
Critical infrastructure	No adverse modifications to flood behaviour or hazard on critical or vulnerable infrastructure such as hospitals, nursing homes, child care facilities and schools (up to PMF).
Emergency management	No adverse impact upon community flood emergency management plans - unless alternate risk mitigation is proposed.

Section 4 of this report details an assessment of the above objectives for the project during construction of the project.

The project has been assessed against the floodplain management objectives, noting that a merit-based approach has been adopted for the flood level objectives as outlined in **Table 6** (refer to **Section 5**).

2 Hydrology and flooding methodology

This assessment has been carried out in line with the NSW Floodplain Development Manual (DIPNR, 2005) with reference to the Coffs Creek Floodplain Risk Management Plan (Bewsher Consulting, 2005) and the Boambee Newports Creek Floodplain Risk Management Plan (GHD, 2016). The following process has been carried out for the assessment:

- Review all relevant information and data applicable to the project including availability of existing hydrological and hydraulic models, digital terrain data, aerial imagery, survey data, project design components and any other relevant information
- Review documentation in relation to applicable guidelines, floodplain risk management plans and establish project objectives and floodplain management objectives and design criteria for the project
- Review the flood risk of the existing environment for the study area, understanding the key flooding mechanisms, and reviewing information for historical flood events
- Refining and updating the existing flood models and developing new flood models for areas where no previous flood modelling had been undertaken
- Ensuring orographic rainfall effects were included in the flood models
- Carry out model validation for the new flood models and for those that had been refined and updated
- Simulate and establish the existing case scenario to understand the current flooding conditions for a range of rainfall events
- Consultation with NSW State Emergency Service (SES) and CHCC about flooding and the potential impacts of the project and proposed mitigation measures
- Assess the potential flooding impacts during construction of the project and identify environmental management measures to avoid, minimise and/or mitigate potential flood impacts on the project or because of the project
- Assess the potential operational impacts of the project and identify and recommend mitigation measures which have been incorporated into the design of the project to reduce and manage potential flood impacts
- Provide environmental management measures to manage residual operational impacts following the implementation of the flood mitigation measures.

2.1 Background information

2.1.1 Historic floods

Coffs Harbour has historically been affected by significant flooding, with the largest flooding events on record detailed below.

November 1996 event

The most significant flood event in Coffs Harbour's history which resulted in declaration of a natural disaster zone. About 500 mm of rainfall fell in six hours, with the most intense rainfall falling in the upper catchments (Maddocks & Rowe, 2004). The flood affected 800 properties, with inundation above floor level of over 250 residential and 210 commercial and public properties (CHCC, 2018). Coffs Creek peaked at a record 5.4 m (Speer, Phillips, & Hanstrum, 2011), over one metre greater than the predicted 1 per cent AEP event and caused $31 million in claimed damages. This event resulted in CHCC commissioning a revised flood study to investigate the orographic rainfall effects of the catchment, resulting in predicted peak flood level increases of 0.5 m or more in many areas (Maddocks & Rowe, 2004).

Figure 3: Flooded commercial areas of Coffs Harbour in 1996 flood (Maddocks & Rowe, 2004)

March 2009 event

About 440 mm of rainfall was recorded within 24 hours (ABC, 2009). Coffs Creek peaked at 5.1 m (0.7 m above 1 per cent AEP), isolating 3200 people (Speer, Phillips, & Hanstrum, 2011). The flood event affected key rail infrastructure, causing closure landslides just north of Coramba.

Figure 4: Flooded tracks north-west of Coffs Harbour on April 1, 2009 (ABC, 2012)

2.1.2 Current flood mitigation

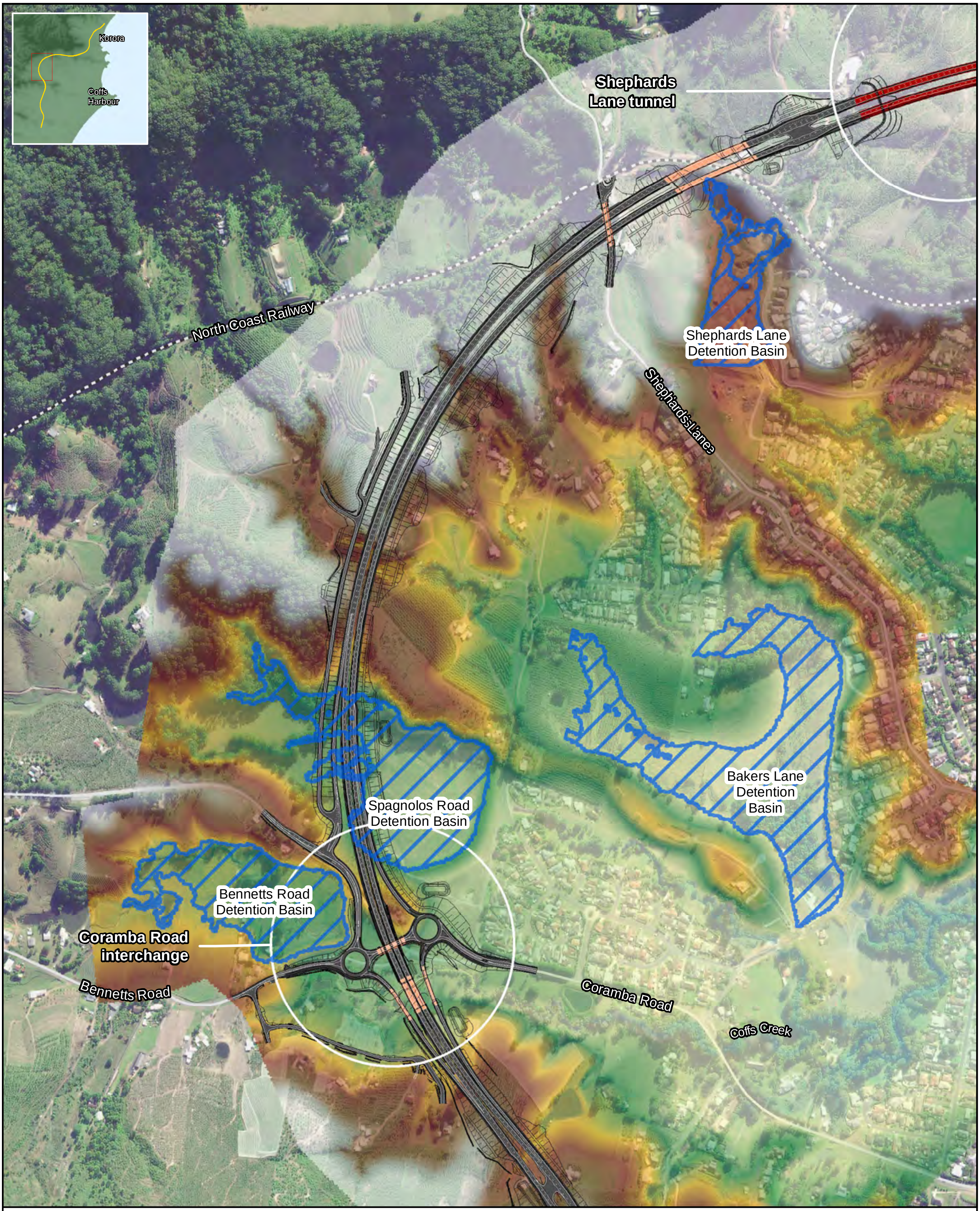
Several detention basins have been constructed to mitigate the flood risk to the community, including:

- The upper tributaries of Coffs Creek near Goodenough Terrace
- Isles Drive Industrial Estate (WMAwater, 2011)
- Several agricultural dams in the upper catchment.

The CHCC Flood Mitigation Programme (CHCC, 2018) incorporated additional detention basins at the following locations:

- Bakers Lane detention basin at William Sharpe Drive, West Coffs
- Bennetts Road detention basin
- Spagnolos Road detention basin
- Shephards Lane detention basin.

Figure 5 illustrates the above basins interaction with the project.



Legend

Detention basin
Pavement
Bridge
Tunnel
North Coast Railway

Elevation (mAHD)

Value

High : 60

Low : 10

Coffs Harbour Bypass
Figure 5
Operational detention basins

0 0.1 0.2 0.3 km
Scale @A4: 1:10,000
GDA 1994 MGA Zone 56
N

2.1.3 Previous flood studies

Relevant existing flood studies were identified and reviewed as part of the assessment of hydrologic and flooding impacts for the project. These are summarised in **Table 7**.

Table 7: Relevant flood studies

Flood study	Summary
Coffs Creek Flood Study (Webb, McKeown & Associates, 2001)	• RORB hydrology with application of rainfall gradients • RUBICON hydraulic model calibrated to historic events • Assessment of previously constructed flood mitigation work, catchment development and tailwater variability.
Coffs Creek Floodplain Risk Management Plan (Bewsher Consulting, 2005)	• Updates to previous flood models with assessment of potential mitigation measures • Provides recommendations based on cost-benefit analysis of flood mitigation options.
***Coffs Creek and Park Beach Flood Study* (BMT WBM, 2018)	• XPRafts hydrologic modelling with application of rainfall zones based on recorded events • 2D TUFLOW hydraulic modelling with linked 1D elements • Calibration and validation to 2009 and 1996 events respectively • Sensitivity testing of climate change, blockage, roughness and rainfall gradients.
Boambee Creek and Newports Creek Flood Study (WMAwater, 2011)	• WBNM hydrologic model with weighted catchment zones to represent orographic effects • MIKE 11 / 2D TUFLOW hydraulic models to represent the upper and lower catchment areas respectively • Calibrated to 1996 event.
***North Boambee Valley (West) Flood Study* (de Groot & Benson, 2014)	• Finer delineation of sub-catchments of the previous hydrology model • 2D TUFLOW hydraulic model with linked 1D elements of upper catchment • Validated to previous results.
Boambee Newports Creek Floodplain Risk Management Study (GHD, 2016)	• Minor updates to the flood models (WMAwater, 2011) with assessment of potential mitigation measures • Provides recommendations based on cost-benefit analysis of flood mitigation options.

** Denotes flood studies models which have been adopted for the assessment of flooding and hydrology for the project. Section 2.2 details the methodology for adoption and use of these models.

2.2 Adopted flood models

After consultation with CHCC, it was agreed to adopt the previously established flood models of North Boambee Valley (de Groot & Benson, 2014) and Coffs Creek (BMT WBM, 2018) as the basis for this assessment.

No previously completed studies were available for the northern creeks catchment. Flood models for the northern creeks were developed and established for the purposes of this assessment.

A description of the hydrology and hydraulic models used for the project for each of the three catchments (as outlined in **Section 1.3**) is provided in **Section 2.4** and **Section 2.5** respectively.

The previously established hydrologic models were developed in accordance with the established practice at the time of their development, which was detailed within ARR (Pilgrim, 1987) (referred to as ARR 1987) – ie single design storm temporal patterns. These models included modifications to account for orographic effects (effects of mountains forcing moist air to rise) of the Coffs Harbour region.

At the time of EIS commencement an update to ARR (Ball, et al., 2016) (referred to as ARR 2016) was developed and is still in draft form. The differences between the design storm depths (IFDs) as contained in the established hydrologic models (ARR 1987) and design storm depths contained within ARR 2016 were compared to determine if the existing models were suitable to assess the impact of the project. Details of this comparison are provided in **Table 8**.

Table 8: Design storm data 2016 vs 1987

Duration (hour)	2016 Rainfall depth difference (%)					
	39% AEP	18% AEP	10% AEP	5% AEP	2% AEP	1% AEP
1	-9.7	-0.8	5.7	8.1	11.7	14.8
2	-13.1	-3.8	3.5	6.7	11.5	15.4
3	-14.7	-5.5	2.7	5.4	10.9	15.0
6	-15.7	-7.6	0.3	3.6	8.2	11.6
12	-15.7	-8.8	-1.8	0.1	3.1	5.5

The above comparison indicated differences of ±15% between the design storm depths from ARR 1987 and ARR 2016.

The orographic effects of the Coffs Harbour region were incorporated into the established hydrologic models for the North Boambee Valley and Coffs Cree catchments. These include calibrated orographic patterns of up to +60% for the ARR 1987 design storm depths.

The design storm depths from ARR 2016 are extracted from a data grid of around 2.6 km^2, which BoM has noted care should be used in areas of steep rainfall gradients when using these design storm depths – such as the Coffs Harbour coastal escarpment.

Based on the comparison of the design storm depths between ARR 1987 and ARR 2016 and the details of the application of orographic effects to the ARR 1987 design storm depths, the hydrological models within previously established flood models were adopted for the project. These include the design storm depths from the established flood models combined with the established orographic effects and consistency with the temporal pattern methodology.

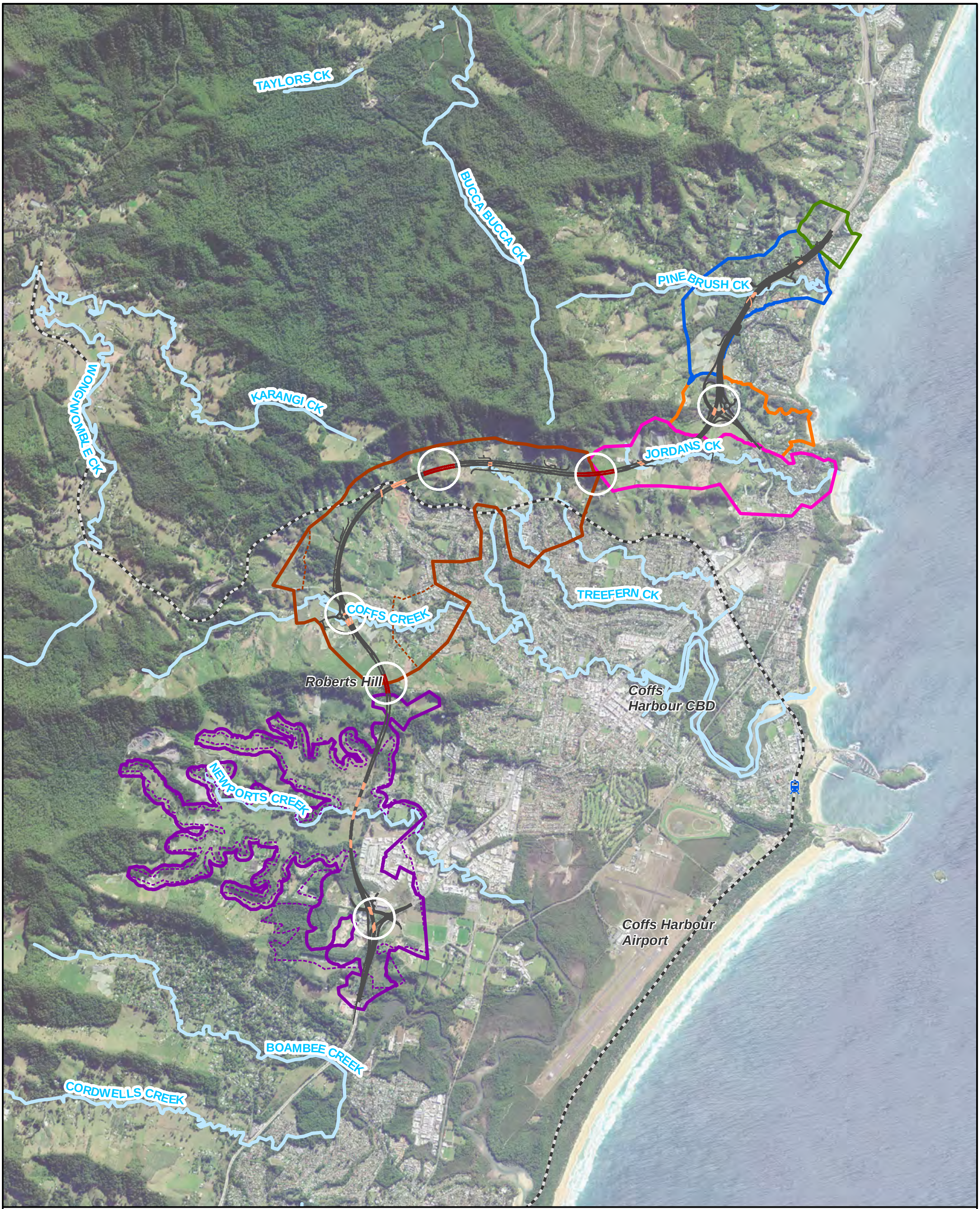
The extents of the hydraulic (flood model) and hydrologic models are shown in **Figure 6** and **Figure 7** respectively.

2.3 Design storm events

The following design storm events were assessed:

- 18, 10, 5, 2 and 1 per cent AEP and PMF
- 1 per cent AEP climate change sensitivity tests (DECC, 2007):
    - 2050 climate: +0.4 m sea level and +10 per cent rainfall intensity
    - 2100 climate: +0.9 m sea level and +30 per cent rainfall intensity.

It is noted the 2050 and 2100 climate change rainfall intensity increases are roughly equivalent to the 0.5 and 0.1 per cent AEP events respectively.



Legend

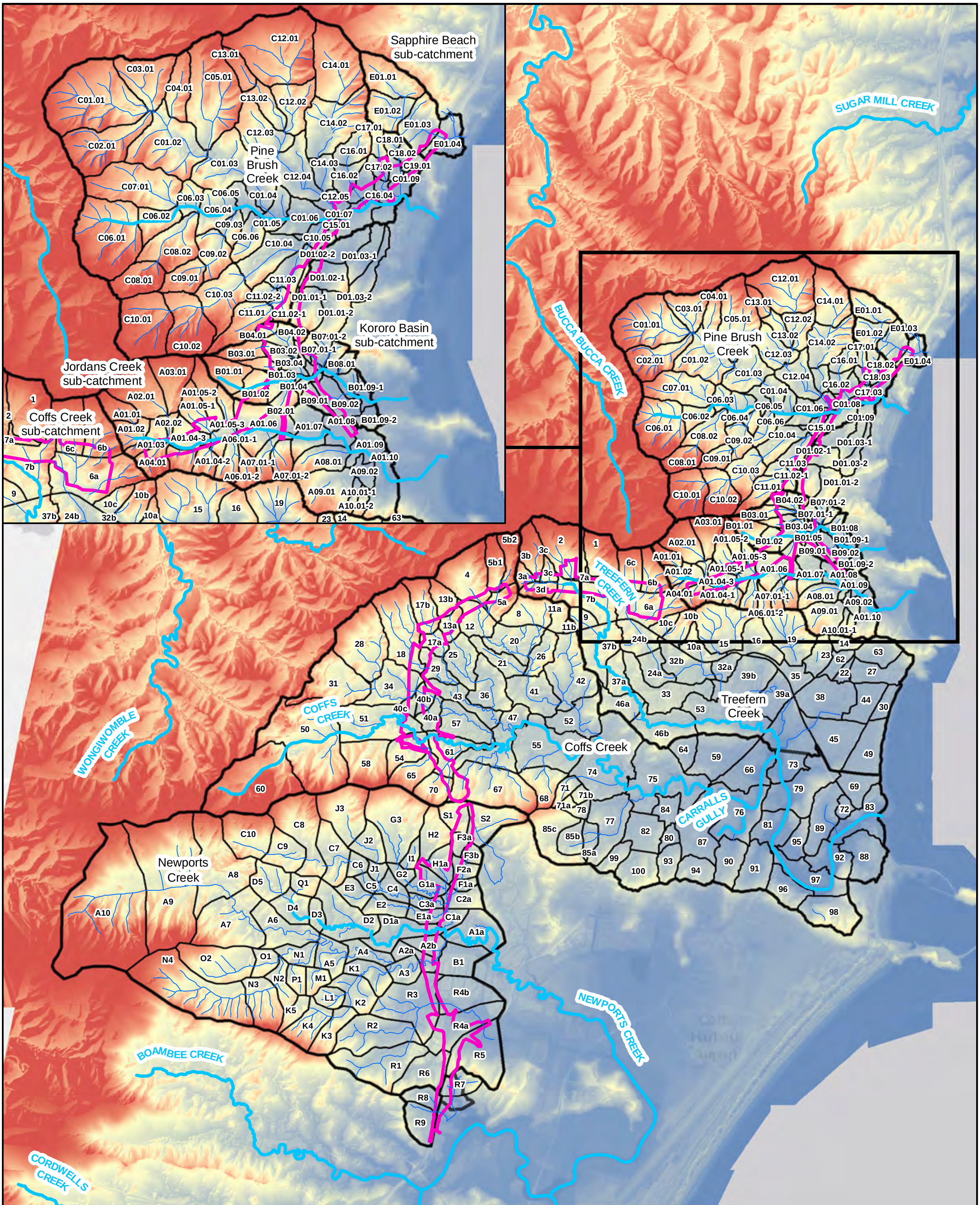
Railway station
North Coast Railway
River
Water body
Bridge
Pavement
Tunnel
Coffs Creek old extent
North Boambee Valley old extent

Current flood model extents

Coffs Creek
North Boambee Valley
Northern Creeks: Domain 1
Northern Creeks: Domain 2
Northern Creeks: Domain 4
Northern Creeks: Domain 5

Coffs Harbour Bypass
Figure 6
Flood model extents

0 0.6 1.2 1.8 km
N
Scale @A4: 1:60,000
GDA 1994 MGA Zone 56



Legend

- Creek
- Tributaries
- Sub-Catchment
- Construction footprint

Topography

High : 588.484

Low : -1.88012

Coffs Harbour Bypass

Figure 7

Hydrological model subcatchments

0 0.6 1.2 1.8 km

Scale @A4: 1:60,000

GDA 1994 MGA Zone 56

N

Service Layer Credits: Esri, HERE, Garmin, (c) OpenStreetMap contributors, and the GIS user community

2.4 Hydrology

2.4.1 North Boambee Valley

The WBNM model developed by (de Groot & Benson, 2014) was adopted for this assessment with the following changes:

- Sub-catchments along the project were adjusted/split where necessary
- An additional model (adopting the applicable parameters and orographic factors) was created to capture an additional nine sub-catchments to the south and two to the north affected by the project
- Addition of the PMF storms, as per Generalised Short Duration Method (GSDM) (BoM, 2003) and climate change intensity increases, as detailed in **Section 2.3**.

The model parameters are detailed within **Table 32** of **Appendix A1**.

Existing scenario hydrologic flows were adopted for the developed hydraulic analysis, based on the following:

- There is an insignificant increase in impervious area (0.4 per cent) between the existing and developed scenarios
- The response time of the upstream catchment (nine hours) is significantly divergent relative to local project runoff response time (10 minutes).

2.4.2 Coffs Creek

The XPRafts model developed for the *Coffs Creek and Park Beach Flood Study* (BMT WBM, 2018) was adopted for this assessment with the following changes:

- Sub-catchments along the project were adjusted/split where necessary with applicable model parameters and orographic factors applied
- Sub-catchment delineation was provided in pdf format rather GIS format, which is used in the project models, hence there are fractional discrepancies based on minor redefinition differences
- The developed scenario was updated to reflect changes in flow direction and fraction impervious in accordance with the project
- Model validation for the Coffs Creek model is detailed in **Section 2.6.2**.

The models are summarised in **Table 33** and **Table 34** of **Appendix A1**.

2.4.3 Northern Creeks

A new XP-Rafts hydrologic model was established for the northern creeks catchments. Key model aspects are summarised below:

- The model parameters adopted are as per the project Coffs Creek model. This was done because of the absence of available rainfall or stream gauge data for calibration
- The adopted sub-catchment roughness values (PERN) range from 0.015 to 0.12, with an average of 0.08, in accordance with recommended values of typical catchment land uses. The values were determined based on the average within each sub-catchment (accounting for surface area taken up by roughness values)
- Pervious initial and continuing losses of 0 mm and 2.5 mm/hour respectively (no impervious losses)
- Orographic effects were applied using methodology of the Coffs Creek study (BMT WBM, 2018)
- Validation of the model was performed to the Rational Method, refer to **Section 2.6.3**.

The developed scenario was updated to reflect changes in flow direction and fraction impervious in accordance to the project.

The models are summarised in **Table 35** and **Table 36** of **Appendix A1**.

2.5 Hydraulics

TUFLOW HPC (version 2018-03-AC) was adopted for all models with the following approach:

- Model topography has been constructed from a range of supplied Aerial Laser Survey (ALS) datasets. The priority of terrain data applied in the model, is as follows:
  - Project ALS (captured May 2016)
  - Regional ALS (captured September 2013).
- Incorporation of initial water levels such that storages are assumed as full (up to drainage invert) before an event
- Simulation of a range of durations initially to determine critical storm(s)
- Bridges were schematised as layered flow constrictions and culverts as linked 1D elements. Applied blockage factors are in accordance with ARR (Ball, et al., 2016). Refer to Table 37 and Table 38 for existing and developed structure parameters respectively, based on the following:
  - Structure information was obtained from CHCC or Roads and Maritime unless otherwise stated:
    - CHCC supplied depth to invert to the nearest 5 mm. Adopted invert levels were based on the topographic data minus depth to invert
    - Roads and Maritime supplied depth of cover provided to the nearest 100 mm. Adopted invert levels were based on topographic data minus depth of cover and structure dimensions.
  - 1.5 m bridge deck depth assumed for all developed bridges for the purposes of hydraulic assessment.

2.5.1 North Boambee Valley

Existing scenario

The TUFLOW model (de Groot & Benson, 2014) was adopted for this assessment with the following changes:

- Four metre grid resolution, including trimming of extent to relevant study area and extension to include the proposed Englands Road interchange and its approaches
- Adjustment of the inflow locations and boundary conditions to match the amended hydrology and extents, as per **Table 9**
- Updates / additional structures as summarised in **Table 37**
- Updates to model roughness in accordance to latest aerial imagery at the time of model development, as per **Table 10**.

Table 9: Model boundaries – North Boambee Valley

Boundary	Schematisation
Upstream	6 QT boundaries
Local	61 Source A inflows
Downstream	5 HT boundaries extracted (GHD, 2016)

Table 10: Model roughness – North Boambee Valley

Land use	Manning's 'n' value
Roads / easements	0.022
Waterways / ponds / entrance transition	0.030
Pasture	0.060
Vegetated / upper creek	0.080
Buildings	3.000

Developed scenario

The developed scenario hydraulic model as was used to inform the design response of key flood design elements including:

- The optimising of bridge locations throughout the design process to achieve conveyance for low and high flow events, as well as for biodiversity objectives for flora and fauna
- The sizing and positioning of longitudinal and transverse drainage channels and culverts
- The potential realignment of a northern tributary of Newports Creek and addition of free draining storage areas
- The optimisation throughout the design development of the road embankments to reduce impact on floodplain storage

- Provision of table drains along either side of North Boambee Road to provide sufficient drainage for low flow events.

The above elements were incorporated into the developed model via:

- Application of the project to model topography and roughness
- Incorporation of structures as summarised in **Table 38**
- Definition of drain inverts and bund crests.

2.5.2 Coffs Creek

Existing scenario

The TUFLOW model (BMT WBM, 2018) was adopted for this assessment with the following changes:

- Four metre grid resolution, extension to include the project and trimming where appropriate
- Enforcement of critical hydraulic controls (such as key crests and gullies)
- Adjustment of the inflow locations and boundary conditions to match the amended hydrology and extents (including changes to SA polygons for stability and application of orographic factors), as per **Table 11**
- Updates / additional structures as summarised in **Table 37**
- Updates to model roughness in accordance to latest aerial imagery at the time of model development, as per **Table 12**

Table 11: Model boundaries – Coffs Creek

Boundary	Schematisation
Local	45 SA inflows
Downstream	5 HT extracted from BMT WBM (2018) model.

Table 12: Model roughness – Coffs Creek

Land use	Manning's 'n' value
Roads / easements	0.030
Waterways / ponds / entrance transition	0.030
Industrial	0.040
Creek transition	0.045
Pasture / Urban / mid creek	0.060
Light vegetation / upper creek	0.080
Moderate vegetation	0.100
Forest	0.120
Buildings	1.000

Developed scenario

The developed scenario hydraulic model as was used to inform the design response of key flood design elements including:

- The optimising of bridge locations throughout the design process to achieve conveyance for low and high flow events as well as for biodiversity objectives for flora and fauna
- The sizing and positioning of longitudinal and transverse drainage channels and culverts
- Extension of the Bennetts Road basin outlet to incorporate the proposed interchange with minimal changes to basin performance
- Ensuring increased runoff does not adversely impact flood levels external to the project
- The optimising of the location of proposed water quality treatment basins throughout the design process
- Provision of table drains to capture flows and maintain drainage.

The above elements were incorporated into the developed model via:

- Application of the project to model topography and roughness
- Incorporation of structures as summarised in **Table 38**
- Definition of drain inverts and bund crests
- Application of developed case flows at downstream water quality treatment basins or drainage lines. Proportional flows of alignment catchments were applied at locations in accordance to the linear drainage design.

2.5.3 Northern creeks

Existing scenario

Four new models were developed for the assessment of the northern creeks area, with the following conditions:

- 2.5 m grid resolution, apart from domain one and four where a 2.0 m grid was used
- Enforcement of critical hydraulic controls (such as key crests and gullies)
- Adopted model roughness values as per

- **Table** 13
- Incorporation of structures as summarised in **Table 37**
- Model boundaries as per **Table 14**.

Table 13: Adopted hydraulic roughness (Manning's 'n') values

Land use	Northern creeks
Roads / road easements	0.030
Waterways / ponds / entrance transition	0.030
Pasture / Urban	0.060
Forested / dense vegetation	0.120
Buildings	1.000

Table 14: Northern creeks model boundary conditions

<table>
<tr><th>Parameter</th><th>Domain 1</th><th>Domain 2</th><th>Domain 4</th><th>Domain 5</th></tr>
<tr><td>Inflows</td><td>3 SA</td><td>22 SA
7 QT</td><td>26 SA
1 QT</td><td>25 SA</td></tr>
<tr><td>Downstream boundaries</td><td>As below</td><td>As below and 1 automatic QH normal depth boundary based 1% slope</td><td>As below and 4 automatic QH normal depth boundaries based 1% slope</td><td>As below</td></tr>
<tr><td></td><td colspan="4">All domains: HT boundary as per BMT WBM (2018). presented in Figure 8 and as follows:
<table>
<tr><th>Local event</th><th>Ocean event</th><th>Peak ocean WL (mAHD)</th></tr>
<tr><td>18% AEP</td><td>HHWS(SS)</td><td>1.13</td></tr>
<tr><td>5% AEP</td><td>HHWS(SS)</td><td>1.13</td></tr>
<tr><td>2% AEP</td><td>5% AEP</td><td>2.0</td></tr>
<tr><td>1% AEP</td><td>5% AEP</td><td>2.0</td></tr>
<tr><td>PMF</td><td>1% AEP</td><td>2.1</td></tr>
<tr><td>1% AEP year 2050 climate change</td><td>5% AEP (+0.4m)</td><td>2.4</td></tr>
<tr><td>1% AEP year 2100 climate change</td><td>5% AEP (+0.9m)</td><td>2.9</td></tr>
</table></td></tr>
</table>

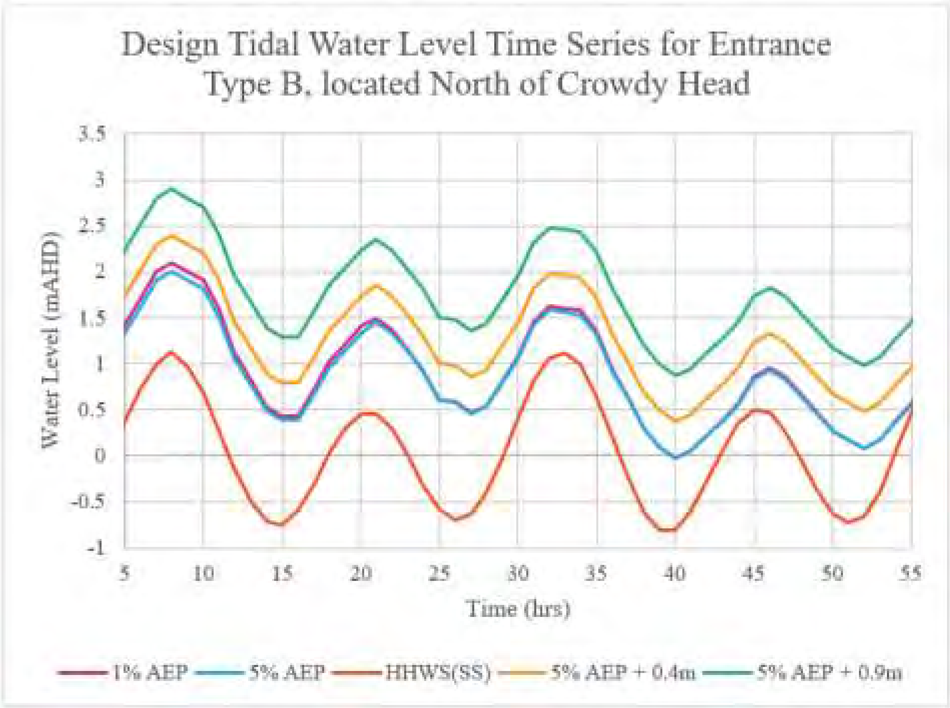


Figure 8: Design tides for entrance Type B, north of Crowdy Head (OEH, 2015)

Developed scenario

The developed scenario hydraulic model as was used to inform the design response of key flood design elements including:

- The optimising throughout the design development of the bridge openings to achieve conveyance for low and high flow events as well as for biodiversity objectives for fora and fauna
- The sizing and positioning of longitudinal and transverse drainage channels and culverts
- Managing overland flows from small steep upstream catchments
- Ensuring any increased runoff volumes do not adversely impact flood levels external to the project footprint
- Optimising the location of water quality treatment to not adversely impact flood flows
- Provision of table drains and appropriate scour protection to capture flows and minimise the risk of adverse impact to waterways and bank stability
- Design coordination and optimisation to ensure appropriate management of Korora Hill interchange runoff.

The above elements were incorporated into the developed model via:

- Application of the project to model topography and roughness
- Incorporation of structures as summarised in **Table 38**
- Definition of drain inverts and bund crests
- Application of developed case flows at downstream water quality treatment basins or drainage lines.

2.6 Validation

2.6.1 North Boambee Valley

The hydrologic model results were checked to previous results (WMAwater, 2011) to ensure comparable flows are adopted as presented in **Table 15**.

Table 15: Peak 1 per cent AEP flow comparison

Location	WMA 2011 (m^3/s)	Current (m^3/s)
Newports Creek - SW of Keona Circuit	240	244
Englands Road Tributary – Upstream of Isles Drive	24	26

The peak flood levels of the hydraulic model were checked to the previous results (GHD, 2016) where the two model domains overlap – as presented in **Table 16**.

Table 16: North Boambee Valley hydraulic model comparison

Location	Peak Flood Level Difference (1% AEP 9hr)
Downstream of Pacific Highway	±0.10 m
Downstream Isles Drive	±0.02 m
Upstream Isles Drive	+0.50 m

The predicted differences were considered reasonable and are likely due to the following:

- New ALS data – particularly elevations of the new Highlander Drive development area including floodplain fill
- Model schematisation (TUFLOW vs. MIKE/TUFLOW combination) and boundary effects.

2.6.2 Coffs Creek

The revised Coffs Creek model was checked to previous results (BMT WBM, 2018). The critical 1 per cent AEP event noted negligible peak flood level differences for at least 95 per cent of the model area.

Localised differences are noted of up to 0.25 m, generally these are considered to be due to adjustments of inflow sources and the change in software engine (classic to HPC). Overall, the revised model was considered reasonable.

2.6.3 Northern Creeks

In the absence of historical data, validation was performed to the Rational Method (with weighted orographic factors applied). **Table 17** presents the critical 1 per cent AEP event peak flow comparison.

Table 17: Northern Creeks 1 per cent AEP validation

Domain	Sub-catchment ID	Critical duration (min)	XPRafts peak flow (m^3/s)	Rational peak flow (m^3/s)	Difference (%)
1	E01.03	120	12.9	13.3	-3.0
2	Pine Brush (C10.04)	60	27.7	29.5	-6.1
4	B01.02	120	11.4	12.5	-8.8
5	Jordans (A02.02)	60	18.1	18.2	-0.5

Flows derived from the XPRafts model exhibit a good fit when compared to the Rational Method and considered suitable for use in the hydraulic model.

3 Existing conditions

The flood models were simulated for the existing case for the range of flood events listed in **Section 2.3**. General results are discussed below, with peak flood level, depth, velocity and hazard maps presented in **Appendix B**. Hazard categories have been defined in accordance with Figure L2 of the NSW Floodplain Development Manual (DIPNR, 2005). Figure L2 has been recreated in **Figure 9** below.

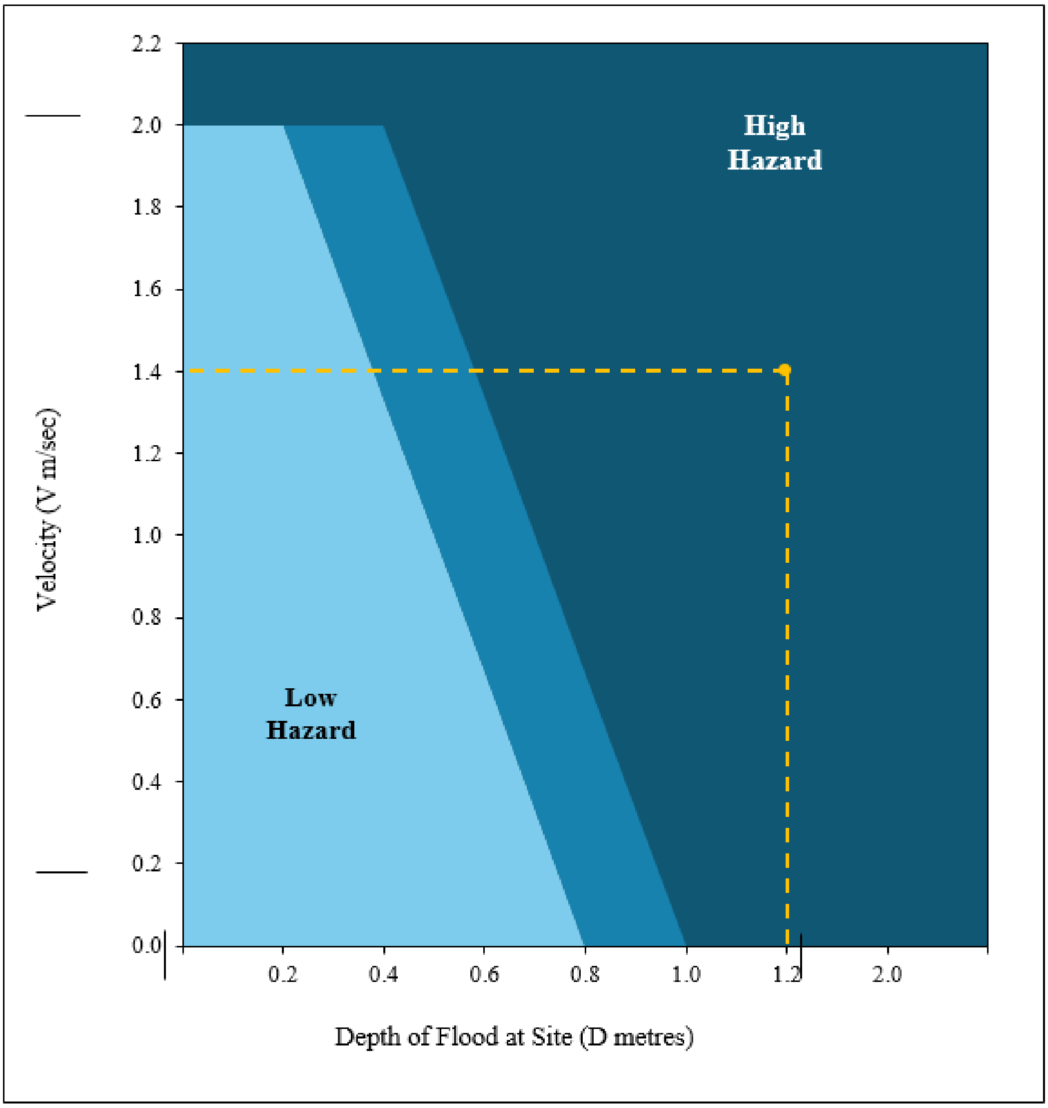


Figure 9: Flood hazard categorisation (DIPNR, 2005)

3.1 North Boambee Valley

The following observations are noted:

- The project is located within the lower reaches of the floodplain of Newports Creek, hence flooding is characterised by relatively low velocity flows outside the main creek channels
- A significant hydraulic control between Bishop Druitt College and Industrial Drive dictates Newports Creek flood levels. **Appendix B1.1.5** indicates a three metre peak head drop for the 1 per cent AEP event
- North Boambee Road is overtopped during an 18 per cent AEP event with a peak flood depth of 0.81 m
- Several North Boambee Road rural properties and the northern extent of Highlander Drive are affected by the 18 per cent AEP event. However, no existing structures are affected by high hazard flooding during the 1 per cent AEP event
- Flooding of the unnamed drainage line south of the Isles Drive Industrial Estate is generally controlled by the road crossings
- Englands Road and Isles Drive overtop during the 18 per cent AEP, with predicted peak depths of 0.13 m and 0.7 m respectively. There is minor inundation of the upstream Pacific Highway shoulder during the 1 per cent AEP event
- Inundation of the road network and the north-west lots of Isles Drive Industrial Estate occurs during the five per cent AEP, with much of the remaining industrial lots flooded during the PMF
- The listed critical infrastructure within the model extents are PMF immune. This includes Coffs Harbour GP Super Clinic and Bishop Druitt College
- The Coffs Harbour Health Campus is outside the model extents, as shown in **Appendix D**. Under current conditions, access to the Coffs Harbour Health Campus from the south via the existing Pacific Highway is unrestricted for all events up to and including the 1 per cent AEP. Access to the Coffs Harbour Health Campus from the north along the existing Pacific Highway. In the PMF event, the existing Pacific Highway south of the Coffs Harbour Health Campus is not trafficable on both the northbound and southbound lanes.
- The North Boambee Valley (west) urban release area includes extensive high hazard PMF areas throughout the Newports Creek floodplain, as illustrated in **Appendix B3.1.8**
- Critical design storm durations (ie producing maximum flood levels) over the project are:
  - Design AEP events: Nine hours
  - PMF: Two hours.

3.2 Coffs Creek

The following observations are noted:

- Existing flooding through the project is characterised by high velocity flow paths generally contained to the established tributaries of the western escarpment
- The North Coast Railway is overtopped during the PMF event north of Brennan Court, with a peak overtopping depth of 0.9 m
- Railway cross-drainage structures east of the project are not modelled (due to insufficient survey available at the time of writing). Due to this, modelling indicates relatively dispersive flooding to the immediate downstream localities of Rigoni Crescent, Baringa Private Hospital (critical infrastructure) and Abel Tasman Drive before converging back into the main conveyance channels. Whilst model accuracy of these areas is limited, they are not expected to be impacted by the project
- The listed critical infrastructure Cow & Koala Professional Child Care is within the Coffs Creek model extents. Cow & Koala Professional Child Care is immune in the 1 per cent AEP event but inundated in the PMF event
- 1 per cent AEP event inundation of existing structures are noted in the following areas (generally outside of PMF high hazard):
    - Within Shephards Lane basin and Bennetts Road
    - Several Coramba Road properties backing onto Coffs Creek
    - Immediately downstream of Spagnolos Road Basin
    - Several properties around Roselands and Coriedale Drives.
- The CHCC Flood Mitigation Programme detention basins were designed to achieve efficient flood protection of downstream properties for a variety of storm events (CHCC, 2018). Maximum flow attenuation is generally achieved if the basin flood level remains below the spillway crest. The minimum overtopping (ie spillway engagement) design storm event and corresponding peak flood level for each basin potentially affected by the project are listed below:
    - Bennetts Road basin: 1 per cent AEP / 28.71 mAHD
    - Spagnolos Road basin: 1 per cent AEP / 23.58 mAHD
    - Bakers Road basin: PMF / 18.67 mAHD
    - Shephards Lane basin: 18 per cent AEP / 43.46 mAHD).
- Critical design storm durations are:
    - Design AEP events: Two and nine hours
    - PMF: One hour.

3.3 Northern creeks

The following observations are noted:

- Flooding is generally characterised by numerous, relatively small flow paths draining off the western escarpment, controlled by the existing Pacific Highway drainage structures
- There is a significant hydraulic control upstream of the Pacific Highway / Bruxner Park Road intersection (ES61) resulting in peak flood depths up to seven meters in the 1 per cent AEP event (attenuating flooding to the downstream Pacific Bay resort)
- The existing Pacific Highway is above the 1 per cent AEP peak flood level, except for the Jordans Creek crossing (<18 per cent AEP immunity) and minor inundation of northbound lanes just west of Opal Boulevard
- There are several urban areas next to the project currently affected by 1 per cent AEP flooding (these are generally affected by PMF high hazard) including:
  - Nautilus Villas
  - Residential lots between Coachmans Close and Pine Brush Crescent
  - James Small Drive residential lots backing onto Pine Brush Creek
  - Bananacoast Caravan Park
  - Various rural lots immediately upstream of the project.
- The listed critical infrastructure of Kororo Public School and Coffs Harbour Montessori Preschool are PMF immune
- Critical design storm durations:
  - Design AEP events: two hours
  - PMF:
    - Domain One: 1.5 hours
    - Domain Two, Four and Five: one hour.

4 Assessment of construction impacts

This section of the report details the aspects relating to construction objectives.

As detailed within **Section 1.7**. the project floodplain management objectives have been divided into sub criteria objectives for specific measurable elements.

The objectives for project infrastructure are:

- **Alignment** - 1% AEP flood immunity for proposed main carriageway and 5% AEP for ramps and interchanges
- **Tunnel portals** - Above the PMF or the 1% AEP flood level +0.5 m (whichever is greater), where ingress of floodwaters would collect at the sag in the tunnel
- **Waterway crossings** - Bridge soffits >0.5 m above 1% AEP flood level. Appropriate scour protection designed for areas at risk of scour due to the project to ensure long term bed and bank stability
- **Construction** - Potential impact of ancillary site locations is identified, to ensure appropriate flood risk assessment of vulnerable sites and to inform a future construction flood management plan.

An assessment of the relative hydraulic and hydrologic impacts, the flood risk and potential impact of the predicted construction activities to construct the project infrastructure and the impacts on ancillary sites to support construction activities was conducted. These activities and ancillary site locations are reflective of the anticipated uses and activities at this concept design stage.

Construction of the project is anticipated to take four years and would likely be built using conventional methods used on most highway projects. The methods may be modified during the detailed design or construction stages to address site-specific environmental or engineering constraints.

The detailed uses and particular construction methodologies would be refined within the detailed design of the project and by the construction contractor, prior to and during construction, based on the site constraints and in accordance with any conditions of approval.

This report identifies the potential flood impact which would inform a future Construction Flood Management Plan (CFMP) to be completed as part of the detailed design phases of the project. A CFMP should include the following:

- Stockpiles, site compounds, plant machinery, elevated haul roads and construction facilities should be located outside defined streams and/or low-lying areas subject to frequent flooding
- Where stockpiling or haul roads within the floodplain cannot be avoided, low velocity locations or appropriate materials should be utilised to minimise loss of material during flooding
- Flood monitoring and response measures should be implemented to mitigate flood risks to life, equipment and property. Given the flash nature of flooding

there would be limited warning time and hence monitoring may largely rely on forecasts issued by BOM

- When a storm/flood warning forecast is issued and it safe to do so, the developed protocols to relocate site materials and machinery to flood immune (or less hazardous) locations should be undertaken
- Procedures for safe site evacuation should be implemented
- Induction of all staff and visitors to brief emergency response procedures.

The potential hydrology and flooding impact of the following construction activities have been assessed, individually within this section of the report.

- Ancillary facilities
- Temporary waterway crossings
- Earthworks
- Catchment drainage.

The assessment of each of these potential construction impacts have been developed to address the specific requirements of the project SEARS and the project floodplain management objectives.

4.1 Ancillary sites

Several ancillary sites have been identified to facilitate construction of the project. These sites may be used for various construction activities and may include, site compounds, the stockpiling or laydown of materials, crushing and screening facilities, concrete batching plants, haul roads to and from the main construction works, temporary access roads to and from the ancillary sites and the storage of plant.

The assessment of ancillary facilities considers potential facilities located within the 5 per cent AEP flood extent because these sites would have a higher risk of potential flood impacts than sites located outside the 5 per cent AEP flood extent. The peak flood extents for the 5 per cent AEP flood, 1 per cent AEP flood and PMF events (1 per cent AEP and PMF flood extents were used to provide an indication of the flood risks for the proposed ancillary facilities), and construction zones (including ancillary facilities) are shown in **Appendix C**.

Ten of the 14 potential sites for ancillary facilities identified for the project are located within potential flood hazard areas (areas within the 5 per cent AEP flood extent). These sites are subject to flooding in the 5 per cent AEP event. The flood extents and construction zones (including ancillary facilities) are shown in **Appendix C**.

An assessment has been carried out to identify the potential flood risk of each ancillary site considering the 5 and 1 per cent AEP, and PMF events. The assessment also considers the ancillary sites that are at risk of frequent (18 per cent AEP) high flood depths and velocities. **Table 18** presents the potential hydrology and flooding impacts of the proposed ancillary facility sites.

Table 18: Flood affected ancillary sites

Site	Flood risk and potential impact	Management measure
1D	The northern portion of this site is part of the Newport Creek floodplain, is within the 5% AEP flood extent and at risk of frequent (18% AEP) high flood depths and velocities. Because Isles Drive industrial area is immediately downstream of this site, locating site compounds or other facilities within the area of frequent impact could cause higher risk of impacts to Isles Drive.	A CFMP will be prepared to manage potential flood risk. Site compounds, stockpiling and plant machinery should be placed outside of the flood hazard area.
1G	This area is predominately flood immune apart from a small area in the north east and a small area on the southern boundary which are part of the Newports Creek floodplain. The areas of risk are part of Newports Creek floodplain, so locating site compounds or other facilities within the areas of risk could cause displacement of existing flood storage / attenuation and have downstream impacts.	A CFMP will be prepared to manage potential flood risk. Site compounds, stockpiling and plant machinery should be placed outside of the flood hazard area.
2A	This site is predominately above 1% AEP flood level and is subject to flooding during a PMF event. Use of this area for ancillary facilities has a relative low risk of potential impacts on flooding and hydrology. The consequence of inundation is high because of proximity of residential properties downstream of the site.	Management of the site uses outside of the PMF event are not required because of the low probability of flooding.
2C	This area is predominately flood immune apart from a tributary which originates in the site. The redirection of this tributary and its flows may cause previously flood free areas to be impacted, however, because the site is in the upper reaches of the catchment, potential impacts on flooding and hydrology are expected to be minimal.	A CFMP will be prepared to manage potential flood risk. Conveyance of existing small tributary within the site and its associated flows should be maintained.
2D	An existing farm dam upstream of the site controls inundation of this area and the site is impacted by the 5% AEP flood event. Ancillary facilities may result in redirection of flows and may cause previously flood free areas to be impacted, however, because the site is in the upper reaches of the catchment, potential impacts on flooding and hydrology are expected to be minimal.	A CFMP will be prepared to manage potential flood risk. Inspection of the dam existing condition before construction activities. Inspection of the dam should also be carried out, after storm events during construction. Site compounds, stockpiling and plant machinery should be placed outside of the flood hazard area.
2E	The southern portion of this site is in the upper reaches of Treefern Creek and is impacted in a 5% AEP flood event. Locating ancillary facilities in areas affected by flooding may result in redirection of flows and may cause previously flood free areas to be impacted, however, because the site is in the upper reaches of	A CFMP will be prepared to manage potential flood risk. Site compounds, stockpiling and plant machinery should be placed outside of the flood hazard area.

Site	Flood risk and potential impact	Management measure
	the catchment, potential impacts on flooding and hydrology are expected to be minimal. Because of the proximity of residences at Abel Tasman Drive, locating ancillary facilities within the areas of flood risk could cause higher risk of impacts to Abel Tasman Drive.	
2G	Most of this site is within the 5% AEP flood extents and is at risk of frequent (18% AEP) high flood depths and velocities. Because of agricultural land uses and a residential property, locating ancillary facilities within the area of frequent flood impact could cause higher risk of impacts to these lands.	A CFMP will be prepared to manage potential flood risk. Site compounds, stockpiling and plant machinery should be placed outside of the flood hazard area.
3C	The south eastern portion of the site contains a tributary discharging into Kororo Basin, is within the 5%AEP flood extents and is at risk of frequent (18% AEP) high flood depths and velocities. The redirection of flows may cause previously flood free areas to be impacted and may increase flooding of upstream areas, with potential impacts on Bruxner Park Road.	A CFMP will be prepared to manage potential flood risk. Site compounds, stockpiling and plant machinery should be placed outside of the flood hazard area. The existing small tributary within the site and its associated flows should be maintained.
3E	Most of the site is within the 5% AEP flood extent and is at risk of frequent (18% AEP) high flood depths and velocities. Consequence of inundation is potential high because of the relative proximity of properties.	A CFMP will be prepared to manage potential flood risk. Site compounds, stockpiling and plant machinery should be placed outside of the flood hazard area.
3G	Most of this site is flood free apart from an area along the southern boundary which is at risk of frequent (18% AEP) high flood depths and velocities. Locating ancillary facilities in areas affected by flooding may result in redirection of flows and may cause previously flood free areas to be impacted, potentially impacting nearby residences.	A CFMP will be prepared to manage potential flood risk. Site compounds, stockpiling and plant machinery should be placed outside of the flood hazard area.

Locating ancillary facilities in areas of high flood risk or in areas subject to flood has the potential to impact on existing flooding and hydrology. Key ancillary site plant and facilities should be positioned to the least flood affected site areas to reduce potential impacts.

4.2 Temporary waterway crossings

There is a potential that the construction and operation (during construction) of temporary waterway crossings, including temporary structures, may impact the existing flooding and hydrology of the study area. These temporary crossings have the potential to impact on the hydraulic function of the waterway, aquatic environment and bank stability, causing water levels to rise upstream of the crossing during a flood event.

Temporary crossing structures may be required to cross Newports Creek, Coffs Creek, Treefern Creek, Jordans Creek, Pine Brush Creek and other small unnamed drainage lines and watercourses to enable materials to be hauled within the construction footprint (rather than using the existing road network) while the adjacent culvert or bridge is being built.

Once final culverts or bridges are suitable for trafficking those structures would be used as haul routes for the project.

To avoid potential flood impacts from temporary creek crossings, the works would be designed, constructed and maintained in accordance with the following mitigation requirements:

- Affected waterway crossings are to be constructed so that natural flow conditions are maintained as much as possible and carried out in accordance with environmental and fish conservation requirements. Existing flow areas are to be maintained as much as possible to minimise potential flood impact of storm events during construction
- Flood modelling may be required to determine the extent of potential impacts and to aid in the appropriate sizing and location of temporary culverts or structures. This is particularly likely for temporary crossings of Newports Creek, Coffs Creek, Jordans Creek, Pine Brush Creek
- Erosion and sediment control measures (including scour protection) are to be implemented immediately around the affected watercourses
- Realigned channels (if required) are to be constructed offline and generally remain free of external flows to allow adequate establishment of vegetation, prior to initiation of the ultimate waterway arrangement
- Temporary haul road crossing structures may also be required in areas of overland flows and are to be constructed such that low and high flows are maintained and fine sediment materials are avoided or contained within the haul road formation
- Following construction completion, affected waterway crossing areas are to be rehabilitated to existing (or improved) conditions.

4.3 Earthworks

Significant earthworks would be required for the construction of the road embankments and cuttings and tunnels within all construction zones.

Earthwork activities during construction would include, the stripping and temporary stockpiling of topsoil, bridge pier foundation works, geotechnical investigations, landscaping, drainage channels, swales, temporary and permanent water quality basins.

Primarily, construction earthworks activities would comprise of temporary stockpiles, temporary water quality basins, construction of embankments, cuttings and tunnels.

The assessment of construction earthworks impacts has been carried out by reference to the final arrangement of the earthworks following construction of the project.

Details of the assessment for the developed scenario are contained in **Section 5**. The assessment examines the impacts of flows, velocity and duration and the assessment is based on the earthworks within their operational / final position. To align the mitigation measures proposed for the operational earthwork arrangements, construction earthworks within flood affected areas are to be constrained to the ultimate (developed scenario) condition of the project to avoid adverse impact.

Development of the detailed design (or operation of the construction plan), may result in earthworks within flood affected areas extending beyond the final arrangement of the earthworks considered in this assessment. Revised flood modelling would be carried out to assess the impact of this change in earthworks if this were to occur.

4.4 Catchment drainage

Construction activities will be required to establish and construct project infrastructure. These works would include, clearing of vegetation, earthworks for embankments, cuttings, temporary haul roads, local road construction and structures. These construction activities have the potential to impact on the surface water quality, volume and velocity discharging to adjacent waterways and hydrological process during and after rainfall events.

Catch drains and cross drainage structures would be built to divert overland flows away from the project and to convey overland flows under the project. Construction of the project would require diversion and management of overland flows to drain new works as they are being built and these activities would have the potential to impact on flooding and hydrology.

The construction of the catch drains and cross drainage structures (including pits, pipes, culverts and open drains/swales) would occur progressively in conjunction with temporary, staged and permanent road drainage to enable continuity of natural watercourses and hydrological processes.

To further support the continuity of natural water courses and hydrological processes, catchment drainage would be designed to divert flows from entering areas of construction. This is to minimise the erosion effect of such flows and also minimise the subsequent requirement to treat any flows which are discharging from the works.

Emergency management

The project would maintain hydrologically dependant environmental values of affected waterways, by ensuring:

- Natural processes, aquatic habitat and connectivity within waterways is maintained
- Environmental water availability and flows are maintained
- Erosion and sedimentation processes are managed
- The effects of proposed stormwater management are minimised.

These environmental issues can be assessed via a comparison of existing and design case for the follow elements:

- Flows
- Velocities
- Durations of inundation.

Flows

Peak flow rates are largely related to the size of the catchment area and the proportion of impervious areas within the catchment. A comparison of the proportion of impervious areas and the peak flow rates between the existing and developed case flood conditions at several points of interest (POI) downstream of the project, for the 1 per cent AEP flood event, is provided in **Table 19**.

Points of interest downstream of the project demonstrate the impact of the project on existing flow conditions. Points downstream of the project were selected to assess whether the impacts of the project would be localised to areas close to the construction footprint, or if there would be changes in the downstream flow conditions. The points of interest for each catchment are shown in the maps in **Appendix D**.

Table 19: Hydrologic comparison

Catchment	POI	Scenario	Catchment (ha)	Impervious area (%)	1% AEP peak flow (m^3/s)
North Boambee Valley^	D	Existing	195.4	23.7	51.4
		Developed	195.4	23.7	52.2
		Difference	0.0	0.0	1.7%
	BA	Existing	1485.6	1.6	239.9
		Developed	1485.6	1.6	241.1
		Difference	0.0	0.0	0.5%
Coffs Creek	BB	Existing	676.9	4.1	84.1
		Developed	678.2	6.1	84.4
		Difference	1.2	2.0	0.4%
	BC	Existing	156.8	16.9	46.7
		Developed	155.6	18.0	46.7
		Difference	-1.2	1.1	0.0%
	AP	Existing	127.9	11.7	61.7
		Developed	135.0	16.4	58.6
		Difference	7.1	4.7	5.0%
	BD	Existing	78.2	13.3	18.6
		Developed	80.1	15.4	19.7
		Difference	1.9	2.0	6.0%

Catchment	POI	Scenario	Catchment (ha)	Impervious area (%)	1% AEP peak flow (m^3/s)
Northern Creeks	P	Existing	149.6	0.2	76.9
		Developed	150.4	4.5	72.6
		Difference	0.8	4.2	5.6%
	Q	Existing	95.7	7.1	37.3
		Developed	95.5	21.3	44.5
		Difference	-0.2	14.2	19.1%
	T	Existing	729.5	2.4	245.0
		Developed	779.2	4.3	244.7
		Difference	49.7	1.9	0.1%
	V	Existing	50.2	6.0	13.7
		Developed	50.1	8.5	14.1
		Difference	-0.1	2.6	2.9%

^ Existing hydrologic flows were adopted for the developed hydraulic analysis for the North Boambee Valley Catchment for the reasons listed below. This results in the per cent impervious areas in individual catchments being equal within the model:

- The increase in impervious areas within the catchment because of the project would be relatively small (about 0.4 per cent)
- The response time for flows from the upper reaches of the catchment (where impervious areas would be unchanged because of the project) would be significantly longer (nine hours) when compared with the response time for flows from the project (ten minutes) (where the impervious areas would be increased). This means runoff from impervious areas of the project during a storm event would be discharged downstream long before flows from the upper reaches of the catchment reach the project, and as such would not affect peak flood levels.

The assessment is based on the comparison between the existing case and the developed case flood conditions. Conditions would change progressively during construction of the project. To be consistent with the floodplain management objectives outlined in **Section 1.7**, flood conditions during construction would be expected to be no worse than the developed case flood.

The assessment indicates peak flow rates in the developed case would generally be within five per cent of the existing flow rates downstream of the project. The exception would be at point of interest Q, which is downstream of the Korora Hill interchange. There would be a moderate increase in the peak flow rates at this location because of the increase in impervious area from the proposed interchange (refer to **Section 5.2.3** for the assessment of operational impacts at this location).

No adverse impacts to natural processes within waterways and floodplains, including the availability of water for ecological purposes, would be expected. The minor changes in peak flow rates would not be anticipated to adversely impact on existing stormwater infrastructure.

No adverse impacts to the environmental availability of water or natural processes within the waterways would be expected. In addition, the minor changes would not be anticipated to adversely impact on the existing stormwater infrastructure.

If during detailed design construction impacts are predicted to be worse than the developed case flood impacts, mitigation measures will be developed in accordance with the flood plain management objectives and the CFMP.

Velocities

Peak flood levels and flow velocities provide an indication of the potential change in natural processes within waterways. The locations where the most change would be expected is at the waterway crossings where flows would be constricted to pass beneath bridges at those locations. A comparison of the peak flood levels and flow velocities between the existing and developed case flood conditions at the major creek crossings, for the 1 per cent AEP flood event, is provided in **Table 20**.

Table 20: Flood conditions of waterway crossings

Waterway	ID	Peak 1% AEP Level (mAHD)			Peak 1% AEP Velocity (m/s)		
		Existing	Design	Impact	Existing	Design	Impact
Newports	DS10	10.88	10.86	-0.02	1.39	1.45	0.06
	DS12	10.32	10.55	0.23	0.53	0.90	0.37
	DS13	10.28	10.28	0.00	0.67	0.45	-0.22
	DS14	10.28	10.30	0.02	0.66	0.46	-0.20
Coffs	DS34	21.44	21.50	0.06	0.74	0.75	0.01
	DS45	60.29	60.31	0.02	0.12	0.49	0.37
Jordans	DS66	52.19	51.35	-0.84	0.53	0.92	0.39
Pine Brush	DS85	11.63	11.67	0.04	2.54	2.48	-0.06

The assessment is based on the comparison between the existing case and the developed case flood conditions. Conditions would change progressively during construction of the project. To be consistent with the floodplain management objectives outlined in **Section 1.7**, flood conditions during construction would be expected to be no worse than the developed case flood.

The differences in flood conditions between the existing and developed case shown in **Table 20** indicates there would be limited change in peak flood conditions at these waterway crossings. The exceptions are at DS12 over Newports Creek and DS66 near Jordans Creek where there would be a 230 mm flood level increase and an 840 mm flood level decrease respectively. The extent of flood level impacts at these two locations are shown on the flood maps within **Appendix D1.1** and **Appendix D2**. These maps show these impacts would be localised.

Natural waterway processes would be maintained or improved following rehabilitation of the waterways affected by construction of the project.

If during detailed design construction impacts are predicted to be worse than the developed case flood impacts, mitigation measures will be developed in accordance with the flood plain management objectives and the CFMP.

Duration

The time of inundation for flood events may be increased immediately upstream of the project because of the location of the project in relation to the contributing catchments. An assessment of the predicted flood impacts has been carried out to identify the locations that would be most flood affected because of the project as these are the locations where the greatest change in time of inundation could be expected.

A comparison of the time of inundation between the existing and developed case flood conditions at these locations for the 1 per cent AEP flood event is provided in **Table 21**.

Table 21: Impacts to flood duration of inundation

POI	1% AEP Flood duration (hr:min)		
	Existing	Design	Difference
B	10:35	10:40	0:05
E	3:15	5:15	2:00
J	6:00	6:55	0:55

The assessment indicates the worst-case changes in time of inundation are in the order of hours (at point of interest E), which would be unlikely to adversely impact the surrounding natural processes.

5 Assessment of operation impacts

Flood modelling was carried out during development of the design for the project to identify areas of impact and recommend mitigation measures which have been incorporated into the design of the project to reduce and manage potential flood impacts, which represents the developed case assessed in the following sections.

The flood models were simulated for the range of storm events listed in **Section 2.3** for the developed case (ie with the project) and compared to the existing case (ie without the project) flood conditions. The flood impacts were reviewed against the floodplain management objectives in **Table 6** and the outcomes are summarised in the following sections.

5.1 Project infrastructure

As detailed within **Section 1.7**. Project floodplain management objectives for project infrastructure has been set for the alignment, tunnel portal and waterway crossing elements of the design as described below:

Alignment

All areas of the proposed alignment achieve required flood immunity criteria of 1 per cent AEP flood immunity for proposed main carriageway and 5% AEP for ramps and interchanges

Bridge soffits >0.5 m above 1 per cent AEP flood level. Appropriate scour protection designed for areas at risk of scour due to the project to ensure long term bed and bank stability.

Tunnel portals

All tunnel portals achieve required flood immunity criteria of being the PMF or the 1 per cent AEP flood level +0.5 m (whichever is greater), where ingress of floodwaters would collect at the sag in the tunnel.

In addition, there are no sags located within any of the project tunnels.

Waterway crossings

The project bridge soffits have been developed to meet a design criteria to be set at least 0.5 m above 1 per cent AEP flood level to provide potential debris clearance, as presented in **Table 22**.

All bridge decks within the project flood models were set at 1.5 m depth, this conforms with the design development of the bridge structures for this concept stage.

Table 22: Project bridge soffit flood clearance

ID	Bridge Soffit (mAHD)	Peak 1% AEP Level (mAHD	Clearance (m)
DS10	11.75	10.86	0.89
DS12	15.46	10.55	4.91
DS13	16.04	10.28	5.76
DS14	12.99	10.30	2.69
DS32	25.75	21.55	4.20
DS33^	22.28	21.52	1.76
DS34^	22.60	21.50	1.10
DS35	26.38	21.47	4.91
DS44	74.42	60.31	14.11
DS45	76.11	60.31	15.80
DS66	56.26	51.35	4.91
DS67	56.16	51.02	5.14
DS79	37.17	32.27	4.90
DS85	13.42	11.67	1.75

^ - Bridges DS33 and DS34 are designed with a deck thickness of 600 mm. The clearances shown above for these two structures are based on a 600 mm deck thickness.

All project bridges achieve required clearances above the 1 per cent AEP level.

Final bridge deck depths may develop further during detailed design. The criteria to maintain clearance from bridge soffits above the 1 per cent AEP would be adopted for detailed design stages of the project.

During detailed design of the project all structures would be designed with appropriate scour protection and velocity dissipation treatments as required. Typical treatments would include rock protection, rip rap and stilling basins. These treatments would be identified at the detailed design phase of the project (once structure arrangements are confirmed).

5.2 Operational impact

Assessment of the potential operational impacts of the project on flooding and hydrology against the design criteria and flooding objectives outlined in **Section 1.7** are outlined in the following sections.

5.2.1 North Boambee Valley

Key elements of the project relating to flooding and hydrology for North Boambee Valley catchment which have been incorporated into the design of the project include:

- Optimising the bridge locations to achieve conveyance for low and high flow events as well as for biodiversity objectives for fauna
- Appropriate sizing and positioning of longitudinal and transverse drainage culverts and channels
- Realignment of a northern tributary of Newports Creek (beneath Bridge 05[DS14]) and addition of free draining storage areas beneath the bridge over North Boambee Road (Bridge 04 [DS13]) and Bridge 05 (DS14) to provide compensatory flood storage
- Optimisation of the road embankment design to minimise impact on floodplain storage while still providing noise mounds where required
- Provision of table drains along either side of North Boambee Road to provide sufficient drainage for low flow events.

Level

Peak flood levels for the 1 per cent AEP flood event in the North Boambee Valley catchment are shown in **Appendix D1** and potential impacts of the project in terms of flood levels for representative points of interest (POI) in the catchment are summarised in **Table 23**.

Bridges, culverts and additional floodplain storage (north of North Boambee Road) have been incorporated into the project to mitigate potential flood impacts.

All areas external to the project in the North Boambee Valley catchment achieve required flood afflux criteria (as summarised in **Table 23**) except for at the Newports Creek floodplain upstream of the project (points of interest E and Z) because of the reduced flood conveyance and storage at this location. Point of interest B exceeds the afflux criteria, however this is on land owned by Roads and Maritime (refer to **Table 23**).

Table 23: Predicted flood levels for the 1 per cent AEP flood event in the North Boambee Valley catchment and potential impacts

POI	Potential flood impact	Mitigation measures included in the design
A	The project widens the road embankment into the low-lying area currently drained by the existing culvert (ES01) and the driveway access of Lot 232 DP740659. Afflux up to 120 mm in the 1% AEP event is noted over the current dam.	The existing culvert (ES01) has been lengthened to match the width of the widened road embankment. A new culvert (DS02) has been included adjacent to ES01 to alleviate potential flood level increases upstream. New culverts (DS03) have also been included and raising of the affected driveway crest is proposed to maintain flood access.

POI	Potential flood impact	Mitigation measures included in the design
B	The project has the potential to impact the tributary adjacent to Englands Road at point of interest B. Afflux up to 850 mm is predicted in the 1% AEP event which would be contained on land owned by Roads and Maritime between the project and Englands Road. The afflux is contained to the heavily vegetated floodplain with no impact to Englands Road flood immunity. Time of inundation is predicted to increase from 10 hours 35 minutes to 10 hours 40 minutes and as such this minor increase in duration is not expected impact environmental processes.	The approach of attenuating flood flows upstream of the project via the proposed culvert (DS09) results in peak flood level reductions to the downstream areas.
C	Stormwater drainage from the Englands Road interchange discharges to the existing drainage channel adjacent to the existing Pacific Highway, resulting in a change in flow distribution over Lot 61 DP1026815.	The proposed culvert (DS05) discharges directly into the downstream channel generally resulting in peak flood level reductions.
D	The tie-in with the existing Pacific Highway slightly modifies the road profile and embankment width affecting flood conveyance. There is a localised increase in flow velocities downstream of the culverts because of the project.	Extension of cross-drainage culverts has been included to match the width of road embankment (DS07, DS08).
E	The project traverses the Newports Creek floodplain at this location and the project embankments affect flood storage and conveyance to the main creek channels. Localised afflux of up to 0.5 m in the 1% AEP event is predicted immediately upstream of the project. Afflux reduces to around 0.2 m as the extent of flood depth increase extends upstream to: • The existing agricultural/forested areas • The residential property adjacent to North Boambee Road (property is owned by Roads and Maritime). Flood depth increase by 0.2 m in the 1% AEP event • Towards North Boambee Road. There is no change to the PMF flood hazard category upstream of the project throughout the North Boambee Valley (West) urban release area.	The proposed bridge and culvert structures (DS10 (BR03) to DS12(BR23)) have been included to provide for flood flow conveyance but do not eliminate afflux upstream.

POI	Potential flood impact	Mitigation measures included in the design
F / Z	The project traverses the Newports Creek floodplain. Embankments reduce floodplain storage in this area resulting in afflux up to 35 mm in the 1% AEP event on the surrounding pastural/forested areas and the northern extent of Highlander Drive. Afflux of up to 18 mm is predicted at the residential property of Lot 1 DP711234 – on the north side of North Boambee Road near point of interest: Z	The proposed bridges (DS13 (BR04) and DS14 (BR05)) and excavation areas provide mitigating flood conveyance and provide compensatory flood storage. Excavation of the floodplain beneath the bridges increases flood storage and is needed to reduce predicted afflux.
G	The project traverses the northern upper sub-catchments of Newports Creek requiring conveyance.	Proposed culverts (DS16-21) provide conveyance of upstream flows. The outlets of these culverts would require at detailed design stage, design of sufficient scour protection /dissipation to address the high velocities which are predicted here.

Mitigation measures for residual impacts

The following design options will be investigated before construction of the project, to reduce the predicted afflux in those areas where afflux is forecast to be greater than the floodplain management objectives (refer to **Section 1.7**):

- Increased bridge lengths: This would provide increased conveyance and reduce the impact to floodplain storage by reducing the size of road embankments
- Downstream channel works: Minor modifications to the channel of Newports Creek downstream of the project could be considered in consultation with CHCC, to reduce predicted afflux
- Additional storage areas: Compensatory excavation of floodplain areas to mitigate the storage loss from embankments for the project. There is limited available area within the project footprint and maintenance of free drainage of low-lying areas may be difficult
- Cross-drainage: Mitigation measures incorporated into the project would hold back flood waters upstream of the project (point of interest B), on heavily vegetated areas on land owned by Roads and Maritime. This would result in a decrease in the flood levels downstream of the project in the 1 per cent AEP flood event, improving flood conditions downstream of the project. Refinement of the cross-drainage design during detailed design could provide a better balance between holding water upstream of the project and managing downstream flood levels consistent with the floodplain management objectives in **Section 1.7**
- Whole of government approach: Through discussions with CHCC and DPIE (Environment, Energy and Science), a whole of government approach would be investigated which considers the relationship between the project and North Boambee Valley (West) URA and what reasonable and feasible options could be implemented to assist in managing potential flood impacts.

Investigation of the potential mitigation measures listed above would need to be carried out in consultation with CHCC and other relevant stakeholders.

This may result in a requirement to increase the current flood model extents to more fully assess the potential benefits of these mitigation measures.

Scour and velocity

The peak velocity difference maps presented in **Appendix D2** illustrate relatively stable developed flood velocities except for increased flows concentrated through the proposed structures. The flows upstream of proposed bridge structures DS12, DS13 and DS14 increase by approximately 0.8 m/s in the 1 per cent AEP event. Increase in velocity of approximately 0.5 m/s was also forecast downstream of proposed culverts DS07, DS08 and DS20 in events above the 5 per cent AEP. Adequate revegetation and scour protection would be required through and around these areas (subject to further mitigation design as above).

As there are no significant peak velocity impacts, no notable adverse impacts to the adjacent riparian vegetation are expected via increases in erosion or sedimentation.

Access

Table 24 presents the predicted minimum design flood event road closure and overtopping depth for the existing and developed scenarios. For this assessment, a road or access point is considered non-trafficable where there would be 100 mm or more water over the crest of the road or access point. There are some cases where there would be a minor increase or decrease in the depth of flooding with the project in place, however the predicted flood depth would remain greater than 100 mm. Despite a minor change in flood depth, the access would be non-trafficable and would remain as such because there would be more than 100 mm over the road or access point.

Table 24: North Boambee Valley flood access

POI	Affected road / driveway	Minimum event closure (AEP) / crest depth (m)		Description
		Existing	Developed	
A	Lot 232 DP740659	<18% / 0.52	>1% / 0	Under current conditions, the driveway access of Lot 232 DP740659 is not trafficable in the 18% AEP event with a depth of up to 520 mm on the road. With the project in place the flood immunity of the driveway access is achieved for the 1% AEP event.
B	Englands Road	<18% / 130	<18% / 130	No change to flood immunity. Note there would be a minor reduction in the time of inundation by 2 minutes from 1 hour 58 minutes to 2 hours.

POI	Affected road / driveway	Minimum event closure (AEP) / crest depth (m)		Description
		Existing	Developed	
D	Pacific Hwy Newports Ck	>1% / 0	>1% / 0	The tie-in with the existing Pacific Highway slightly modifies the road profile and embankment width affecting flood conveyance. However, the immunity of the road remains unchanged.
W	Isles Drive	<18% / 0.57	<18% / 0.16	Under current conditions, Isles Drive would not be trafficable in the 18% AEP event with a depth of up to 570 mm on the road. With the project in place the trafficability of Isles Drive remains unchanged however the maximum depth of overtopping is reduced to 160 mm.
X	Engineering Drive	2% / 0.11	2% / 0.11	Under current conditions, Engineering Drive would not be trafficable in the 2% AEP event with a peak flood level depth of up to 110 mm. With the project in place the trafficability remains unchanged with the peak flood level depth remaining at 110 mm in the 2 per cent AEP event.
Y	North Boambee Rd	<18% / 0.78	<18% / 0.78	Under current conditions, North Boambee Road has a flood immunity of less than the 18% AEP event with a depth of up to 780 mm on the road. Although the project results in minor increases of afflux at some locations along North Boambee Road, it does not worsen the immunity of the road, change the duration of inundation that this road would be closed for, or cause adverse flood impacts in this area when compared to existing conditions.
AA	Highlander Dr North	<18% / 0.54	<18% / 0.55	With the project in place, the trafficability remains unchanged for Highlander Drive North with the maximum depth increasing only by 10 mm in the 18% AEP event. Note there would be a minor reduction in the time of inundation.
AA	Glengyle Cl	<18% / 0.51	<18% / 0.52	With the project in place, the trafficability remains unchanged for Glengyle Close with the maximum depth increasing only by 10 mm in the 18% AEP event. Note there would be a minor reduction in the time of inundation.
Z	Lot 2 DP711234	<18% / 0.28	<18% / 0.28	Under current conditions driveway access to Lot 2 DP711234 would not be trafficable in the 18% AEP event with a peak flood level depth of up to 280 mm.

POI	Affected road / driveway	Minimum event closure (AEP) / crest depth (m)		Description
		Existing	Developed	
Z	Lot 100 DP1145073	<18% / 0.19	<18% / 0.20	Under current conditions driveway access to Lot 100 DP1145073 is not trafficable in the 18% AEP event with a peak flood levels depth of up to 190 mm. With the project in place the trafficability remains unchanged with peak flood depths increased only by 10 mm in the 18% AEP event.

Table 24 demonstrates the project is not predicted to adversely impact currently flood affected access routes and no additional mitigation would be required for access in the North Boambee Valley catchment.

Consultation with CHCC indicates North Boambee Road could be upgraded to improve flood immunity. The project provides sufficient vertical clearance to North Boambee Road to enable it to be raised in the future.

Direction

The project results in minimal changes to surface water source and direction where possible, except for constriction into and expansion out of structures and constructed diversions, in line with the project floodplain management objectives.

Hazard

The project is predicted to increase the flood hazard on the upstream side of the project (POI:E) to high, over an area of around 1.5 hectares for design flood events.

An increase of flood hazard is also predicted on the upstream side of Englands Road within pasture and forested land during the PMF event. No changes to flood hazard classifications are predicted over existing buildings.

Critical infrastructure

Project results of critical infrastructure within the flood model extents shown in **Appendix D** maps are presented in **Table 25**.

Table 25: Critical infrastructure impact in North Boambee Valley

Location	Potential flood impact
Bishop Druitt College	All buildings are outside flood extents. A portion of carpark and sporting fields are inundated but not impacted by the project. No change anticipated.
Coffs Harbour GP Super Clinic	Outside flood extents. No change anticipated.

Emergency management

Newports Creek and its tributaries are current flooding concerns for the SES. Flooding around Newports Creek, adjacent the Coffs Harbour Health Campus, is a current issue and SES rely on a stream gauge adjacent the Isles Drive industrial estate to provide flood levels.

Peak flood level difference maps within Appendix D illustrate no adverse impact to the identified evacuation routes and assembly areas surrounding the North Boambee Valley flood model. Access to the Coffs Harbour Health Campus from the south is maintained for events up to and including the 1 per cent AEP event.

The project provides additional routes and connections above predicted flood levels resulting in potentially more effective flood evacuation procedures. This includes improved access to the Coffs Harbour Health Campus from the north via the bypass, the Englands Road interchanges and the section of the existing Pacific Highway north of the Englands Road interchange, for events up to and including the 1 per cent AEP event.

Consultation with SES and CHCC will be carried out during detailed design if there are any changes to the existing flood evacuation routes or associated roads which may be impacted during operation.

Boambee Newports Creek Floodplain Risk Management Plan

The current management plan (CHCC, 2016) indicates a development control plan is currently drafted to provide detailed flood planning controls. This includes a high priority to reduce the flooding on the approaches to the Coffs Harbour Health Campus. The project does not impact the flood immunity of the existing Pacific Highway approach, in addition to providing an alternate route.

5.2.2 Coffs Creek

Key elements of the project relating to flooding and hydrology for Coffs Creek catchment which have been incorporated into the design of the project include:

- Optimising the bridge openings to achieve conveyance for low and high flow events, biodiversity objectives for fauna and constructability
- Appropriate sizing and positioning of longitudinal and transverse drainage culverts and channels
- Modification of the Bennetts Road detention basin and outlet arrangement. Excavation of the base of the Bennetts Road detention basin is proposed to increase the storage of the basin by 26,600 m^3 while maintaining the existing low flow channel
- Mitigating adverse impacts by optimising the location of proposed water quality treatment basins to not impact on existing flow paths
- Provision of table drains and appropriate scour protection along either side of the project to capture flows and minimise the risk of adverse impacts on the existing waterway and bank stability.

Level

Peak flood levels for the 1 per cent AEP flood event in the Coffs Creek catchment are shown in **Appendix D1** and potential impacts of the project in terms of flood levels for representative points of interest (POI) in the catchment are summarised in **Table 26**.

Bridges, culverts and additional flood storage (upstream of the project near Coramba Road and within the Bennetts Road detention basin) have been incorporated into the project to mitigate potential flood impacts.

All areas external to the project achieve required flood afflux criteria (as summarised in **Table 6**) except for Coffs Creek downstream of the Coramba Road interchange (points of interest I and AQ). This is because of impacts of the project on the outlet from the Bennetts Road detention basin and the increased pavement area resulting in more stormwater runoff entering the creek from the project.

Despite the mitigation works incorporated into the project, downstream residential properties backing onto Coffs Creek (point of interest AQ) are predicted to experience peak flood level increases. It is unconfirmed if this predicted afflux would affect existing structures, as finished floor level survey of these properties has not been conducted. A finished floor level survey of the properties identified at point of interest AQ will be carried out during detailed design to confirm whether predicted afflux would affect the existing structures.

Table 26: Predicted flood levels for the 1 per cent AEP flood event in the Coffs Creek catchment and potential impacts

POI	Potential flood impact	Mitigation measures included in the design
H	The project traverses the southern upper sub-catchment of Coffs Creek requiring conveyance.	Proposed culverts (DS27) provide conveyance of upstream flows. The outlet of these culverts would require the detailing and design of sufficient scour protection/dissipation measures during detailed design as high velocities are predicted.
I	Predicted afflux in the 1% AEP flood event is 18 mm within the Bennetts Road detention basin because of the Coramba Road interchange immediately downstream of the basin and the impact this has on the outlet from the basin.	The basin outlet pipe has been extended to daylight (DS37), the spillway flows are routed through a proposed culvert (DS36) and the proposed bridges (DS32 to 35 (bridges BR06, BR07 and BR08)) would provide conveyance to Coffs Creek. Excavation of the basin floor is proposed to increase storage in the basin by about 26,600 m^3.
AQ	Predicted afflux in the 1% AEP flood event is 50 mm within Coffs Creek downstream of the project. The increase in flood level at this location is because of the increased area of impervious surfaces (the project pavement), resulting in additional stormwater runoff entering the creek.	Alignment drainage allows for a proportion of flood flows (10% AEP) to discharge at the various tributary crossings upstream of Coffs Creek to reduce the volume of stormwater runoff from the project, discharging directly to Coffs Creek. Excavation of the base of the Bennetts Road detention basin to increase the storage of the basin and balance the volume of flows downstream in Coffs Creek.

POI	Potential flood impact	Mitigation measures included in the design
J	The project extends into the existing Spagnolos Road detention basin, decreasing storage volume and attenuation effectiveness. Predicted afflux upstream of the project and the Spagnolos Road detention basin in the 1% AEP flood event would be greater than 500 mm. This afflux is contained to the heavily vegetated areas on land owned by Roads and Maritime. There would be a decrease in flood levels within the Spagnolos Road detention basin in the 1% AEP flood event.	The approach of attenuating flood flows upstream of the project via the proposed culvert (DS38) results in peak flood level reductions to the downstream areas.
K	The project traverses the upper sub-catchments of Coffs Creek requiring conveyance.	Proposed structures (DS39-46) provide conveyance of upstream flows with minor afflux upstream within objectives. The outlets of these culverts would require the design and detailing of sufficient scour protection/dissipation to address the high velocities which are predicted here.
L	It is proposed to reconfigure the access road resulting in modification of flood flow distribution.	Proposed structures (DS47-60) are sized to ensure no adverse impact to access flood immunity
M	Afflux of up to 400 mm during the 1% AEP flood event is predicted within the Treefern Creek area downstream of project near point of interest M. The concept design for the project includes measures to direct flows crossing the main carriageway (via a proposed culvert DS55) away from Mackays Road to improve local access and reduce potential scour effects. Afflux is contained to vegetated creek areas and the proposed design results in no adverse flood impact to access.	
N	The project traverses the upper sub-catchments of Coffs Creek requiring conveyance.	Proposed culverts (DS61,63) provide conveyance of upstream flows with afflux contained to vegetated creek areas. The outlets of these culverts would require the design and detailing of sufficient scour protection/dissipation to address the high velocities which are predicted here.

Mitigation measures for residual impacts

The following design options will be investigated before construction of the project, to reduce the predicted afflux in those areas where afflux is forecast to be greater than the floodplain management objectives (refer to **Section 1.7**):

- Main carriageway drainage: The Coffs Creek crossing forms the longitudinal low point of the alignment between the Roberts Hill and Shephards Lane tunnels. The design of the main carriageway for the project in this area includes a drainage system which would collect stormwater from the main carriageway (up to the 10 per cent AEP event) and discharge the flows at the various tributary crossings north of Coramba Road interchange. For storm events greater than a 10 per cent AEP event, stormwater collected on the main carriageway up to the 10 per cent AEP event flows would be collected in the drainage system, and the remaining flows would bypass the drainage system and discharge to Coffs Creek. Refinement of the drainage system to carry flows greater than the 10 per cent AEP event could reduce the total amount of runoff from the main carriageway entering Coffs Creek at Coramba Road interchange, and potentially reduce downstream impacts along Coffs Creek

- Downstream channel works: In areas where afflux is predicted, modifications to the Coffs Creek channel may reduce potential impacts to adjacent properties and could be considered in consultation with CHCC. These works may however shift afflux further downstream and would impact existing established vegetation along the existing creek channel
- Southern tributary: The proposed culvert (DS27) could be modified to further hold back flood flows or a new detention storage could be included within the construction footprint to provide additional storage upstream of the project to reduce impacts downstream of the project and reduce flood levels at point of interest AQ
- Cross-drainage: The project as proposed would hold back flood waters upstream of the project (point of interest J), on heavily vegetated areas on land currently owned by Roads and Maritime. This would cause the road formation to act as a detention basin and potentially result in a decrease in flood levels within the Spagnolos Road detention basin in the 1 per cent AEP flood event. While this would potentially improve flood conditions downstream of the project, there would be greater operational and management risks for the main carriageway as well as ongoing maintenance and management requirements for this location. Refinement of the cross-drainage design in this location will be carried out during detailed design in consultation with CHCC and DPIE (Environment, Energy and Science). Refinement of the cross-drainage design would aim to maintain the existing flooding / hydrological regime by providing a better balance between holding water upstream of the project and managing downstream flood levels consistent with the floodplain management objectives in **Section 1.7**
- Local property mitigation: There may be opportunities to carry out localised mitigation work over affected properties (point of interest AQ), including flood barriers / levees to protect existing structures and confine flows to the main channel. A finished floor level survey is required to confirm any adverse impacts to existing structures
- Culvert duplication: The culvert under Coramba Road (ES19) could be modified in consultation with CHCC to reduce the predicted afflux. Further investigation would be required to ensure afflux does not result further downstream.

Investigation of the potential mitigation measures listed above would need to be carried out in consultation with CHCC and other relevant stakeholders. The investigation and further consultation may also result in additional mitigation options to those identified above.

As such, the final design solution may involve combinations of the above mitigation options and the design response developed as part of the concept design.

Scour and velocity

The peak velocity difference maps presented in **Appendix D** illustrate relatively stable developed flood velocities with the exception of the locations detailed below:

- **Coffs Creek**: Minor (up to +0.2 m/s) peak velocity increases are predicted within Coffs Creek downstream of Bennetts Road basin that may result in localised scour instances during peak events. This assessment is subject to further refinement during detailed design
- **Treefern Creek**: The proposed Mackays Road bund (POI: M) redistributes flows and hence increases peak flood velocities (up to 0.5 m/s) to the vegetated area to the east. Absolute velocities are still relatively low in the 18 per cent AEP event, increasing from 1.4 m/s in existing conditions to 2.1 m/s post-project conditions
- **Minor tributaries**: Downstream of design culverts DS41 and DS61, increases were observed of up to 0.3 m/s in events above the 5 per cent AEP. As is noted in other areas of increased velocity downstream of culverts outlet scour protection is to be refined in the detailed design stage.

Based on the above it is considered that there are no significant peak velocity impacts and no notable adverse impacts to the adjacent riparian vegetation are expected via increases in either erosion or siltation.

Access

Table 27 presents the predicted minimum design flood event road closure and overtopping depth for the existing and developed scenarios. For this assessment, a road or access point is considered non-trafficable where there would be 100 mm or more water over the crest of the road or access point. There are some cases where there would be a minor increase or decrease in the depth of flooding with the project in place, however the predicted flood depth would remain greater than 100 mm. Despite a minor change in flood depth, the access would be non-trafficable and would remain as such because there would be more than 100 mm over the road or access point.

Table 27: Coffs Creek flood access

POI	Affected road / driveway	Minimum event closure (AEP) / crest depth (m)		Description
		Existing	Developed	
AD	Lot 60 DP586574	<18% / 0.33	>1% / 0.05	Driveway access would currently be closed during the 18% AEP event with a peak flood depth of 330 mm. The project would be expected to improve flood access to a 1% AEP event standard.
AD	Lot 730 DP1066743	<18% / 0.36	10% / 0.13	Driveway access would currently be closed during the 18% AEP event with a peak depth of 360 mm. The project is predicted to improve flood access almost to a 10% AEP event standard.
AE	William Sharp Dr West	<18% / 0.11	10% / 0.19	William Sharp Drive West would currently be closed during the 18% AEP event with a peak flood depth of 110 mm. The project is predicted to improve flood access almost to a 10% AEP event standard.
AF	Rosalee Cl	<18% / 0.43	<18% / 0.41	The project is anticipated to provide a minor flood depth reduction (20 mm) to Rosalee Close, currently would be closed during the 18% AEP event with a peak depth of 430 mm.
AK	Roselands Dr near Spagnolos Rd	10% / 0.13	5% / 0.12	Roselands Drive (near Spagnolos Road) would currently be closed during the 10% AEP flood event with a peak depth of 130 mm. The project is predicted to improve flood access almost to a 5% AEP event standard.
AL	Roselands Dr near Barnet St	5% / 0.14	5% / 0.11	The project is anticipated to provide a minor food depth reduction (30 mm) to Roselands Drive (near Barnet Street), currently would be closed during the 5% AEP event with a peak depth of 140 mm.
AM	Gillon St	5% / 0.16	1% / 0.18	Gillon Street would currently be closed during the 5% AEP event with a peak depth of 160 mm. The project is predicted to improve the flood access almost to a 1% AEP event standard.
AN	Polwarth Drive	<18% / 0.18	<18% / 0.16	The project is anticipated to provide a minor flood depth reduction (20 mm) to Polwarth Drive, currently would be closed during the 18% AEP event with a peak depth of 180 mm. Note there would be a minor reduction in the time of inundation.

POI	Affected road / driveway	Minimum event closure (AEP) / crest depth (m)		Description
		Existing	Developed	
AG	Spagnolos Rd	1% / 0.12	>1% / 0.02	Spagnolos Road would currently be closed during the 1% AEP event with a peak depth of 120 mm. The project is predicted to improve flood access to above a 1% AEP event standard.
AI	Lot 5 DP1104404	<18% / 0.23	<18% / 0.21	The project is anticipated to provide a minor flood depth reduction (20 mm) to the driveway, currently closed during the 18% AEP event with a peak depth of 230 mm. Note there would be a minor reduction in the time of inundation.
AH	Lot 102 DP1150637	<18% / 0.64	<18% / 0.60	The project is anticipated to reduce flooding (40 mm) over the driveway, currently would be closed during the 18% AEP event with a peak depth of 640 mm. Note there would be a minor reduction in the time of inundation.
AJ	Lot 4 DP1157157	<18% / 0.59	<18% / 0.59	Access would remain unchanged.
M	Mackays Rd Treefern Ck North	<18% / 0.52	<18% / 0.42	The project is anticipated to reduce flooding (100 mm) over Mackays Road, currently would be closed during the 18% AEP event with a peak depth of 520 mm. Note there would be a minor reduction in the time of inundation.
AP	Mackays Rd Treefern Ck South (Bray St)	<18% / 0.26	<18% / 0.15	The project is anticipated to reduce flooding (110 mm) over Mackays Road, currently would be closed during the 18% AEP event with a peak depth of 260 mm. Note there would be a minor increase in the time of inundation.

Table 27 demonstrates the project is not predicted to adversely impact currently flood affected access routes and in some cases access is improved, no additional mitigation is required for access in the Coffs Creek catchment.

Direction

The project results in minimal changes to surface water source and direction where possible, except for constriction into and expansion out of structures and constructed diversions, in line with the project floodplain management objectives.

Hazard

Hazard in the Coffs Creek model typically remains unchanged with the exception of:

- Increases in hazard classification in vegetated and open pasture areas in events between 5 per cent AEP and PMF near POI:L and east of POI:M
- Hazard levels have been adversely impacted upstream of the existing Spagnolos Road detention basin (near point of interest J). Under current conditions, the existing Spagnolos Road detention basin provides a level of flood storage. With the project in place this flood storage is reduced. The project as proposed would hold back flood waters upstream of the project (point of interest J), on heavily vegetated areas on land currently owned by Roads and Maritime. This would cause the road formation to act as a detention basin and potentially result in a decrease in flood levels within the Spagnolos Road detention basin in the 1 per cent AEP flood event. While this would potentially improve flood conditions downstream of the project, there would be greater operational and management risks for the main carriageway as well as ongoing maintenance and management requirements for this location. Refinement of the cross-drainage design in this location will be carried out during detailed design in consultation with CHCC and DPIE (Environment, Energy and Science). Refinement of the cross-drainage design would aim to maintain the existing flooding / hydrological regime by providing a better balance between holding water upstream of the project and managing downstream flood levels consistent with the floodplain management objectives in **Section 1.7**
- There would be increases in hazard in localised areas within Baringa Private Hospital in the PMF event, however there are no changes to hazard in smaller rainfall events
- Baringa Private Hospital: Predicted reduction in peak flood levels for all events except the PMF, which predicts a minor increase of 18 mm with the project in place, with a peak flood depth 954 mm
- There would be a decrease in hazard near Cow & Koala Professional Child Care in the PMF event only, other events remained unchanged
- Considerable decrease in hazard was forecast in the PMF near POI: AK and POI: AG.

Critical infrastructure

Project results of critical infrastructure within the flood model extents shown in **Appendix D** maps are presented in **Table 28**.

Table 28: Critical infrastructure impact in Coffs Creek

Location	Potential flood impact
Baringa Private Hospital	Peak flood level reductions for all events except minor PMF increases of up to 18 mm, with a peak flood depth 954 mm. It is noted the accuracy of this location is limited without the upstream railway cross-drainage (refer **Section 3.2**).
Cow & Koala Professional Child Care	Cow & Koala Professional Child Care remains immune in events up to and including the 1% AEP event. Peak flood levels are reduced in the PMF event by up to 11 mm.

Mitigation measures for residual impacts

The following measures will be investigated before construction of the project, to confirm potential impacts and reduce the predicted afflux if required:

- Additional survey data will be collected, including the existing culverts beneath the North Coast Railway, and incorporated into the flood models. Additional flood modelling will be carried out to confirm the potential impacts at Baringa Private Hospital
- If additional modelling indicates a potential impact at Baringa Private Hospital, finished floor level surveys will be carried out to confirm whether predicted afflux affects the existing structures. A finished floor level survey of the properties identified at point of interest R will be carried out during detailed design to confirm whether predicted afflux affects the existing structures. If required, there may be opportunities to incorporate additional mitigation measures within the construction footprint to reduce potential downstream impacts.

Investigation of the potential mitigation measures listed above would need to be carried out in consultation with CHCC, ARTC and other relevant stakeholders.

Emergency management

Peak flood level difference maps within **Appendix D** illustrate no adverse impact to the identified evacuation routes and assembly areas surrounding the Coffs Creek flood model. Furthermore, the project provides additional routes and connections above predicted flood levels resulting in potentially more effective flood procedures.

Consultation with SES and CHCC will be carried out during detailed design if there are any changes to the existing flood evacuation routes or associated roads which may be impacted during operation.

Coffs Creek Floodplain Risk Management Plan

The recommended floodplain management measures within the Coffs Creek Floodplain Risk Management Plan (CHCC, 2005) are consistent with the project. All four detention basins have been incorporated in the hydraulic models used as part of this assessment. The project generally provides attenuation upstream providing additional flood protection to downstream urban areas.

The project is generally predicted to have a positive impact to the existing flood detention basins, modifying the peak 1 per cent AEP flood level as below:

- 0.06 m decrease of Spagnolos Road basin
- 0.03 m decrease of Bakers Road basin
- No change of Shephards Lane basin
- 0.02 m increase of Bennetts Road basin.

5.2.3 Northern Creeks

Key elements of the project relating to flooding and hydrology for northern creeks catchments which have been incorporated into the design of the project include:

- Optimising the bridge openings to achieve conveyance for low and high flow events as well as for biodiversity objectives for fauna
- Appropriate sizing and positioning of cross drainage culverts
- Managing overland flows from small steep upstream catchments to achieve the flood immunity objectives of the project within an urbanised environment
- Ensuring any increased stormwater runoff from the project did not adversely impact flood levels downstream of the project
- Mitigating adverse impacts by optimising the location of water quality treatment basins to not impact on existing flow paths
- Provision of table drains and appropriate scour protection to capture flows and minimise the risk of adverse impacts on the existing waterway and bank stability
- Design coordination and optimisation to ensure that the Korora Hill interchange road runoff catchments would be captured and outlet to manage downstream impacts.

Level

Peak flood levels for the 1 per cent AEP flood event in the northern creeks catchments are shown in **Appendix D1** and potential impacts of the project in terms of flood levels for representative points of interest (POI) in the catchment are summarised in **Table 29**.

Bridges and culverts have been incorporated into the project to mitigate potential flood impacts.

All areas external to the project achieve required flood afflux criteria (as summarised in **Table 6**) except for the following locations:

- Pacific Bay Eastern Lands (point of interest BI), where afflux up to 100 mm is predicted on lots proposed as part of the approved development in the 1 per cent AEP event
- Russ Hammond Close/James Small Drive (near point of interest R), where afflux up to 200 mm is predicted in the heavily vegetated creek areas in the 1 per cent AEP event. Afflux would be contained to the existing flood inundation extents downstream of the project near point of interest R
- Campbell Close Korora (point of interest U), where afflux up to 200 mm is predicted in the waterway of the unnamed tributary that drains to Sapphire Beach in the 1 per cent AEP event
- Nautilus Villas (point of interest V), where up to 11 mm of afflux is predicted to the downstream area of the Nautilus Villas, and 28 mm of afflux is

predicted on three residential properties adjacent to the waterway in the 1 per cent AEP event.

Table 29: Predicted flood levels for the 1 per cent AEP flood event in the northern creeks catchments and potential impacts

POI	Potential flood impact	Mitigation measures included in the design
O	The project and revised local access road traverses the northern sub-catchments of Jordans Creek requiring conveyance.	Proposed culverts (DS65-70) have been sized to ensure adequate flood conveyance and no adverse flood impact to local access.
P	Existing access to Lot 19 DP771618 via Bruxner Park Road is proposed to be provided via West Korora Road with a new connection provided across Jordans Creek. Predicted afflux in the 1% AEP flood event is 1200 mm within Jordans Creek next to the proposed access crossing.	Afflux is contained to vegetated creek areas and proposed culverts (DS71 and DS72) provide no adverse flood impact. Refer to **Table 30** for assessment of impacts on property access.
Q	The Korora Hill interchange results in the removal of the Bruxner Park Road intersection detention, increased road runoff and redistribution of flood flows to the downstream Pacific Bay Resort. Predicted afflux in the 1% AEP flood event is up to 200 mm within the vegetated creek and lakes, golf course and carpark areas.	Afflux is generally contained to non-adverse areas with no adverse flood impact to Resort Drive.
BI	Increased runoff is predicted with the approved development area of Pacific Bay Eastern Lands from the interchange at Korora Hill. Predicted afflux in the 1% AEP flood event is up to 100 mm on Lot 14 of the approved development. New flow paths are predicted through Lots 14 to 16 and Lots 18 to 21 with depths of 30 mm and 50 mm respectively in the 1% AEP flood event. Previous consultation with the proponent of the Pacific Bay Eastern Lands during preparation of the EIS has indicated that the future proposals are also being investigated within the area subject to flooding impact.	
R	The project reconfigures the existing Pacific Highway Pine Brush Creek crossings (ES71) including additional bridges and embankment work. Predicted afflux in the 1% AEP flood event is up to 200 mm and 70 mm over upstream and downstream heavily vegetated creek areas. No adverse flood impact is predicted to the existing Old Coast Road (ES69 and ES72) or James Small Drive (ES74) bridges.	Proposed bridges (DS85 (BR21)) have been sized to ensure adequate flood conveyance.
S	The project and revised local access road traverses the northern sub-catchments of Pine Brush Creek requiring conveyance.	Proposed culverts (DS86-101) have been sized to ensure adequate flood conveyance and no adverse flood impact to local access.

POI	Potential flood impact	Mitigation measures included in the design
T	The Opal Boulevard access has been reconfigured, resulting in a modified flood distribution. Localised afflux of up to 300 mm is predicted in the 1% AEP event immediately upstream and downstream of the Opal Boulevard crossing of Pine Brush Creek.	Proposed roadside channels generally provide conveyance of upstream flood flows to the main creek channel. Afflux is contained to the vegetated creek areas with no adverse flood impact to Opal Boulevard flood access.
U	The proposed water quality basins extend into the waterway of the main Sapphire Beach tributary, resulting in localised afflux of up to 200 mm over vegetated areas of a residential property located on Campbell Close, Korora. Existing buildings are not affected.	
V	The project tie-in is predicted to result in up to 11 mm of afflux to the downstream area of Nautilus Villas. Greater peak level impacts of up 28 mm are predicted on three residential properties immediately adjacent to the waterway.	

Mitigation measures for residual impacts

The following design options will be investigated before construction of the project, to reduce the predicted afflux in those areas where afflux is greater than then floodplain management objectives (refer to **Section 1.7**):

- Pacific Bay Eastern Lands (point of interest BI): There are opportunities to reduce potential impacts through further refinement of cross-drainage culverts and by raising the height of the approved residential development area to avoid inundation in the 1 per cent AEP event. Consultation with the proponent of Pacific Bay Eastern Lands development will be carried out during detailed design to develop a reasonable and feasible design solution to mitigate flood impacts on the approved residential areas and the main resort building. Consultation will also consider future proposals that are being investigated
- Russ Hammond Close/James Small Drive (near point of interest R): Afflux would be contained to the existing flood inundation extents downstream of the project near point of interest R. A finished floor level survey of the properties identified at point of interest R will be carried out during detailed design to confirm whether predicted afflux affects the existing structures. If required, there may be opportunities to carry out localised mitigation work over affected properties, including flood barriers / levees to protect existing structures and confine flows to the main channel. The final mitigation measures would be developed in consultation with the individual property owners
- Campbell Close Korora (point of interest U): Investigate opportunities to reduce the size of the water quality basins (or change to a proprietary spill capture unit) adjacent to the waterway next to the residential properties to reduce potential flooding impacts. Note existing buildings are not adversely impacted

- Nautilus Villas (point of interest V): Further investigation will be conducted to improve the accuracy of the model during detailed design stage. Detailed terrain survey will be carried out during detailed design to confirm impacts. Properties adjacent to the waterway will have a finished floor level survey carried out during detailed design to determine if existing buildings are adversely impacted. If required, there may be opportunities to carry out localised mitigation work over affected properties, including flood barriers / levees to protect existing structures and confine flows to the main channel.

Investigation of the potential mitigation measures listed above would need to be carried out in consultation with CHCC and other relevant stakeholders.

Scour and velocity

The peak velocity difference maps presented in **Appendix D** illustrate relatively stable developed flood velocities except for Pacific Bay Resort Golf Course. Minor (up to +0.2 m/s) peak velocity increases are predicted within the current course flow-paths and lakes. Increases are generally limited to existing vegetated creeks and paved areas, except the new flow path downstream of ES57, subject to predicted velocities of around 0.5 and 0.7 m/s in the 18 and 1 per cent AEP events respectively. It is noted that this will be reviewed during detailed design with a focus on water quality basin outlet location and possible outlet scour protection.

Within the Pacific Bay Eastern Lands (point of interest BI) there are minor increases in peak velocity on Lot 14 in the 1 per cent AEP of up to 0.2 m/s. Increases were also predicted in the PMF event of up to 0.3 m/s on lots 14-22.

Localised velocity increases were also predicted downstream of design culverts DS70, DS71 and DS72 of up to 0.5 m/s in events above the 5 per cent AEP. All culverts, including those mentioned above would be designed with appropriate outlet scour protection and velocity dissipation. This would be assessed at detailed design stage to mitigate any risks of erosion and bank stability.

Access

Potential flood impacts of the project on existing local and access roads in the northern creeks catchments are summarised in **Table 30**.

For this assessment, a road or access point is considered non-trafficable where there would be 100 mm or more water over the crest of the road or access point. There are some cases where there would be a minor increase or decrease in the depth of flooding with the project in place, however the predicted flood depth would remain greater than 100 mm. Despite a minor change in flood depth, the access would be non-trafficable and would remain as such because there would be more than 100 mm over the road or access point.

The proposed reconfiguration of all local roads and driveways affected by the project results in no adverse impact to access during flood events for most properties. The exceptions to this are for Lot 1 DP527497 (point of interest S) and Lot 19 DP771618 (point of interest P), the predicted flood increase to Opal Boulevard (point of interest T), and the predicted flood increases at the southern end of James Small Drive (point of interest AZ).

These access roads and driveways are proposed to be upgraded by the project. Detailed design will be developed so there is no flood access impact.

Table 30: Northern creeks flood access

POI	Affected road / driveway	Minimum event closure (AEP) / overtopping depth (mm)		Description
		Existing	Developed	
AR	West Korora Road, Jordans Creek^	<18% / 1020	<18% / 1380	The project is predicted to increase flooding (360 mm) over West Korora Road, currently closed during the 18% AEP event with a peak depth of 1020 mm. Note the existing West Korora Road and existing Pacific Highway intersection is affected by the 18% AEP event (refer to POI AS) in the existing and developed, however there would be no increase in flood depth.
AX/P	Lot 19 DP771618	>1% / 58	5% / 190^	Local access to Lot 19 DP771618 has been reduced. Existing access via Bruxner Park Road would not be affected by the 1% AEP event. With the project in place and access via West Korora Road, the local access would be overtopped in 5% AEP event flood conditions by 190 mm. New culverts have been incorporated into the design to reduce the extent of afflux at this location.
AS	Pacific Highway, Jordans Creek	<18% / 590	<18% / 590	Access via Pacific Highway remains unchanged and overtops in an 18% AEP event in both existing case and developed case flood conditions.
AY	Bruxner Park Road	<18% / 130	<18% / 110	Bruxner Park Road would not be trafficable in current conditions in the 18% AEP event with peak flood depths up to 130 mm. With the project in place the trafficability remains unchanged with peak flood depths reducing by 20 mm in the 18% AEP event. Access flood immunity is maintained.
AZ	James Small Drive	>1% / 75	<18% / 130	Local access via James Small Drive has been lowered and would now overtop in 18% AEP event flood conditions by 130 mm. Note James Small Drive is predicted to have water on the road at this location in 18% AEP event in the existing case, however it is less than 100 mm, and as such the road is considered accessible.
Q	Resort Drive	<18% / 580	<18% / 580	Access via Resort Drive (ES99) remains unchanged and would overtop in an 18% AEP event in both existing case and developed case flood conditions.

POI	Affected road / driveway	Minimum event closure (AEP) / overtopping depth (mm)		Description
		Existing	Developed	
AU	Langley Close	<18% / 680	<18% / 670	Access via Langley Close remains unchanged and would overtop in an 18% AEP event in both existing case and developed case flood conditions.
AT	Driftwood Court	<18% / 760	<18% / 760	Access via Driftwood Court remains unchanged and would overtop in an 18% AEP event in both existing case and developed case flood conditions.
AU	Cutter Drive	<18% / 520	<18% / 510	Access via Cutter Drive remains unchanged and would overtop in an 18% AEP event in both existing case and developed case flood conditions.
AT	Firman Drive	<18% / 830	< 18% / 820	Access via Firman Drive remains unchanged and would overtop in an 18% AEP event in both existing case and developed case flood conditions.
AZ	Ballantine Drive	>1% / 22	>1% / 49	Access via Ballentine Drive remains unchanged and would overtop in an 1% AEP event in both existing case and developed case flood conditions.
R	Old Coast Road, Pine Brush Creek	10% / 130	10% / 140	Local access via Old Coast Road remains the same, the road would overtop in both existing case and developed case flood conditions in the 10% AEP event.
T	Opal Boulevard	5% / 110	10% / 100	Existing flood immunity of Opal Boulevard would be reduced to 10% AEP flood event.
S	Lot 1 DP270147	<18% / 130	10% / 120	Local access to Lot 1 DP270147 would be improved, now overtops in 10% AEP event flood conditions.
S	Lot 100 DP1112799	<18% / 170	>1% / 27	Local access to Lot 100 DP1112799 would be improved, now overtops in 1% AEP event flood conditions.
S	Lot 1 DP527497	>1% / 37	<18% / 220	Flood immunity of local access to Lot 1 DP527497 would be reduced and would now overtop in 18% AEP flood conditions by 220 mm.
V	Ocean Dream	<18% / 510	<18% / 520	Access flood immunity for Ocean Dream would be maintained to less than an 18% AEP event. However, the road would overtop by an additional 10 mm.

^ The Pacific Highway / West Korora Road intersection is also affected by the existing 18 per cent AEP flood event, which also affects access at this location

Mitigation measures for residual impacts

The following design options will be investigated before construction of the project, to reduce the potential impacts on access where it is impacted by the project (refer to **Section 1.7**):

- Reconfiguration of property access: There are opportunities to reconfigure access to properties near point of interest S to reduce potential impacts on access during flood events
- Alternative property access design: There are opportunities to provide alternative property access locations for the property affected by flooding at point of interest P, instead of providing access via West Korora Road, to reduce impacts on access during flood events. This would be investigated in consultation with the property owner
- Flood increases to Opal Boulevard (point of interest T): The predicted increases are the result of the proposed adjacent drainage channels overtopping and extending longitudinally down the road shoulder. Detailed design of these channels will be developed to contain upstream flows to achieve no adverse flood access impact
- Refinement of drainage design: There are opportunities to reduce flood impacts at point of interest AZ through refinement of the drainage design, to reduce impacts on access to the southern end of James Small Drive during flood events.

Investigation of the potential mitigation measures listed above would need to be carried out in consultation with CHCC and other relevant stakeholders.

Direction

The project results in minimal changes to surface water source and direction where possible, except for constriction into and expansion out of structures and constructed diversions, in line with the project floodplain management objectives.

Hazard

Increases in flood hazard classifications are predicted over some areas immediately upstream of the project (DS67, DS69, DS70, DS86).

Localised increases are predicted around the Pacific Bay Resort and golf course (downstream of culverts ES57 and ES58) during the five and 1 per cent AEP events.

Critical infrastructure

Project results of critical infrastructure within the flood model extents shown in **Appendix D** maps are presented in

Table 31

Table 31: Critical infrastructure impact in Northern creeks

Location	Potential flood impact
Kororo Public School	Outside flood extents. No change anticipated.
Coffs Harbour Montessori Preschool	Outside flood extents. No change anticipated.

Kororo Public School bus interchange

The proposed Kororo Public School bus interchange is located adjacent to, and to the east of, the Pine Brush Creek catchment. It is located at the top of the catchment and as such a flood model has not been developed to assess the potential flooding impacts from construction of the bus interchange.

An assessment of the catchment hydrology immediately downstream of the bus interchange was carried out. The increased impervious area associated with the hardstand surface of the bus interchange is predicted to resulted in an increase in discharge of 0.6%. This was validated using a Rational Method desktop calculation which showed the resultant increase in discharge to be of very similar magnitude (0.8%).

In addition, hydraulic assessment showed an increase in discharge of <1% yields less than 10 mm increase in flood level during the 1 per cent AEP event.

The findings of this assessment demonstrate that the bus interchange would not have an appreciable impact on the flooding characteristics downstream. This assessment will be revisited should any changes in the design or assumptions occur at a later stage in the project. Note also that any alterations to the hydrologic regime of the bus interchange could also be mitigated through onsite detention, if it is determined during detailed design that this is necessary. This can take several forms which could be tailored to the design and contained within the proposed construction footprint.

Emergency management

Peak flood level difference maps within **Appendix D** illustrate no adverse impact to the identified evacuation routes and assembly areas. Furthermore, the project will provide additional routes and connections that may improve flood procedures. The current flood evacuation plan (SES, 2017) should be revised following completion of the project to ensure the most effective management strategy.

Consultation with SES and CHCC will be carried out during detailed design if there are any changes to the existing flood evacuation routes or associated roads which may be impacted during operation.

5.2.4 Social and economic cost

The project includes mitigation and management measures to minimise short and long-term impacts from flooding including consideration for future climate conditions (see **Section 17.6.8**). In many areas, the project would reduce peak water levels downstream.

The project would improve transport efficiency of the existing Pacific Highway through Coffs Harbour, relieve congestion on the wider Coffs Harbour road network and provide an alternative route for some local trips. The project would provide a route which is above 1 per cent AEP flood level from the north of Coffs Harbour to the south of Coffs Harbour, with additional access points for local traffic to access this flood free route (eg via Coramba Road interchange). There would be significant economic benefits from increasing the reliability of a major national freight route such as the Pacific Highway. The project would also improve the local emergency management procedures during storm events, reducing the social and economic impact of flooding to the local community.

There are several affected properties that are predicted to have design event peak flood level increases around buildings. Actual flood damages may occur if the project results in inundation above the finished floor level where it did not occur previously. Finished floor levels of these properties will be surveyed to determine actionable damage and impacts mitigated, wherever possible, through further design refinement during detailed design.

6 Climate change

Rainfall and sea level are the two predominant factors which determine the degree and severity of flood events. Climate change has the potential to significantly influence both factors, by increasing sea levels and causing an increase in the severity of extreme weather events.

The *Practical Consideration of Climate Change – Floodplain Risk Management Guideline* prescribes indicative changes in extreme rainfall. The indicative changes are sourced from the CSIRO report for *Climate Change in NSW Catchments* published in 2007 (DECC, 2007). That report has been superseded by *Climate Change in Australia - Projections for Australia's Natural Resource Management Regions* technical report published in 2015, which has been referenced for the climate change effects on the project.

The CSIRO predicts, with very high confidence, that mean sea level will continue to rise and the height of extreme sea-level events will also increase (CSIRO, 2015). Since the NSW Government announced its Stage One Coastal Management Reforms on 8 September 2012, it is no longer recommended to apply state-wide sea level rise benchmarks by local councils. Sea level rise has therefore been modelled as a sensitivity check on predicted flood levels.

The project is located at elevations high enough to be unaffected by potential sea level scenarios. Nevertheless, the 2050 and 2100 scenarios have been assessed by increasing the ocean boundary levels by 400 mm and 900 mm, respectively (CHCC, 2018).

The CSIRO predicts average rainfall will decrease and that wet years will become less frequent. Despite this they also predict, with high confidence, that intense rainfall events will become more frequent and extreme while the magnitude of the increases cannot be confidently projected (CSIRO, 2015). In conjunction with sea level rise, the sensitivity assessment was undertaken to include a 10 per cent and 30 per cent increase in rainfall for 2050 and 2100 scenarios.

In summary, two climate change scenarios have been modelled (DECC, 2007):

- 2050 climate: 400 mm sea level rise and 10 per cent increase in rainfall intensity
- 2100 climate: 900 mm sea level rise and 30 per cent increase in rainfall intensity.

The 1 per cent AEP event was used as the basis for the sensitivity assessment with impacts of peak flood level and velocity compared to the following scenarios:

- Predicted impact of the project during climate change events (ie developed compared to existing scenario under climate change events)
- Predicted climate change impact to the project (ie developed comparison of current to future climate conditions).

The impacts identified from the sensitivity assessment are detailed by project catchment in the flowing sections.

6.1 North Boambee Valley

Impact of the project

The peak flood level and velocity impacts in the North Boambee Valley catchment for the climate change scenarios are shown in **Appendix D1** and **Appendix D2**.

When compared with the velocity and peak flood level impact from the 1 per cent AEP (see **Figure D1.1.5** of **Appendix D1**), the afflux pattern under climate conditions in the North Boambee and Newports Creek study catchment would not be appreciably altered compared to the baseline conditions (see **Figure D1.1.6** and **Figure D1.1.7** of **Appendix D1**).

An increase in peak water level impact was observed in the 2100 climate scenario west of POI: B and south of ES17. The impact occurs where previously no peak water level impact was observed. The increase is contained within the existing extent of inundation which is within the waterway and open pasture/grass land.

Flood immunity outcomes for the project did not change from those reported in **Section 5.2**. ie the mainline of the project remains trafficable in the 1 per cent AEP event in the 2050 and 2100 climate scenarios.

Hazard classification for climate scenarios generally remains the same as the on per cent AEP event, except for increases in high hazard areas upstream of the project (see POI: B and POI: E).

Impact to the project

The 1 per cent AEP flood immunity is achieved under future climate scenarios within the North Boambee Valley. **Appendix E** contains details of the 1 per cent AEP (see **Figure E1.1** and **Figure E1.2**).

Flood immunity of the project does not change under the climate change scenarios, with the main carriageway remaining trafficable in the 1 per cent AEP event in the 2050 and 2100 climate scenarios within the North Boambee Valley catchment.

6.2 Coffs Creek

Impact of the project

When compared with **Figure D1.2.5** in **Appendix D1**, the water level afflux pattern under climate conditions (see **Figures D1.2.6** and **D1.2.7** of **Appendix D1**) in the Coffs Creek study catchment shows improvements in the conditions downstream of the project.

In many of the areas that were observed to be impacted in the climate change scenario, the project alignment either prevents inundation completely for the 1 per cent AEP event or decreases the peak water level of up to 400 mm (see POI: BB).

Increases in peak flood level were generally observed on the upstream side of the project following a generally consistent afflux pattern observed for the non-climate future scenarios modelled.

An increase in peak water level of up to 15 mm and 23 mm were predicted around the Baringa Private Hospital in both the 2050 and 2100 future climate conditions respectively. The accuracy of the peak water levels at this location are limited as details of the upstream railway cross-drainage are not currently contained within the project models (refer **Section 3.2**).

Hazard classifications for both 2050 and 2100 climate scenarios remains generally the same as the existing case, except for an increase in high hazard upstream of POI: J. This high hazard area is located within vegetated and open pasture area.

Impact to the project

Appendix E illustrates the peak flood level impact of future climate conditions to the project.

Flood immunity of the project does not change under the climate change scenarios, with the main carriageway remaining trafficable in the 1 per cent AEP event in the 2050 and 2100 climate scenarios within the Coffs Creek catchment.

6.3 Northern Creeks

Impact of the project

The peak flood level impacts in the northern creeks for the climate change scenarios are shown in **Appendix D1** and **Appendix D2**.

When compared with **Figure D1.3.5** in **Appendix D1**, the afflux pattern under climate conditions in the northern creeks study catchment shows an increase in peak water level impact downstream of POI: AZ and culvert ES58, particularly in the 2100 climate change scenario.

Downstream of POI: AZ the peak water level increase is in the order of 70 mm and 170 mm for the 2050 and 2100 future climate change scenarios, respectively. Noting that in the baseline case (without the project in place) the flood depth is around 1700 mm and 1800 mm for the 2050 and 2100 future climate change scenarios. There were no notable increases in extent of inundation downstream.

The peak flood level increases of 30 mm and 60 mm were predicted downstream at POI: Q surrounding the golf course conference centre for both the 2050 and 2100 climate scenarios respectively. As the project removes the Pacific Highway / Bruxner Road intersection detention and increases impervious runoff, increased peak flood levels are predicted downstream.

Peak water level increases in the order of 30 mm and 80 mm for the 2050 and 2100 future climate scenarios respectively were predicted on Lots 18-21 and 14-16 of the Pacific Bay Eastern Lands approved development (see POI: BI).

Flood immunity for the project mainline would be maintained in both future climate change scenarios. The targeted flood immunity for the Korora Hill interchange is the 5 per cent AEP event, and this may not be achieved under future climate change conditions with the predicted increases in rainfall intensity.

Hazard in the future climate scenarios follows the same pattern as the 1 per cent AEP in both existing and design scenarios.

Impact to the project

Illustrated in **Appendix E3.1** and **Appendix E3.2** are the predicted flood increases under future climate predictions.

Flood immunity of the project does not change under the climate change scenarios, with the main carriageway remaining trafficable in the 1 per cent AEP event in the 2050 and 2100 climate scenarios within the northern creeks catchments.

7 Conclusion

This flooding assessment technical report has been prepared to address the relevant SEARs for the project. The pertinent background information, applied methodology, flood model development and key outcomes have been detailed.

This assessment and established flood models will form the basis of future detailed design stages of the project.

Flood model results were used to inform the concept design and determine the required mitigation measures, including optimising bridge/culvert arrangements, drainage channels and detention, and appropriate structure outlet scour protection / velocity dissipation, to achieve the required objectives, including:

- Minimum 1 per cent AEP flood immunity for proposed main carriageway, five per cent AEP for ramps and interchanges and PMF for tunnel portals
- No adverse peak flood level impact external to the site
- Negligible impact to external waterway stability or riparian vegetation
- Minimal changes to flood flow direction
- No adverse change to affected local road access during flood events
- No adverse flood impact to local infrastructure or emergency management.

The project will maintain hydrologically dependant environmental values of affected waterways, by ensuring crossings are rehabilitated and protection provided where required. Peak flow rates are generally consistent, with minor increases in runoff volumes predicted due to the additional impervious area of the project.

It is noted there are several locations identified requiring further development to achieve the above objectives, potentially via alternative mitigation measures.

Further investigation of these measures requires consultation with CHCC and other applicable stakeholders and will be carried out during the detailed design of the project.

The predicted impacts of the project under future climate scenarios do not extend to any additional buildings relative to current climate conditions. Flood immunity objectives for the project are maintained in future climate change scenarios.

A conceptual assessment of the relative flood risk and potential impact of the predicted construction activities was also conducted. With sufficient measures in place the project can achieve required flood and hydrologic objectives throughout the construction phase.

The project would provide 1 per cent AEP flood immune thoroughfare and local connections not serviced by current roads. There are substantial economic benefits of increasing the reliability of a major national freight route such as the Pacific Highway. Furthermore, it is considered the project would reduce the social and economic impact of flooding to the local community.

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Appendix A

Hydrologic and hydraulic model parameters

A1 Hydrologic model parameters

Table 32: North Boambee Valley WBNM

Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Sub-catchment	Area (ha)	Impervious (%)	Orographic factor
A10	238.3	0	1.11	G2	5.7	0	1.19
A1a	26.7	15	1.15	G3	58.2	0	1.14
A2a	12.6	0	1.05	H1a	11.8	0	1.02
A2b	6.9	0	1.02	H2	20.0	0	1
A3	11.3	2	1	I1	5.6	0	1.25
A4	11.2	0	1	J1	4.8	0	1.08
A5	15.4	1	1.05	J2	15.3	0	1.55
A6	27.1	1	1	J3	45.2	0	1.2
A7	57.0	0	1	K1	14.4	10	1.08
A8	66.7	0	1.15	K2	18.5	2	1.08
A9	140.0	0	1	K3	17.9	0	1.02
B1	18.6	60	1.02	K4	23.1	0	1.075
C10	46.1	0	1.225	K5	9.7	0	1.65
C1a	8.0	15	1	L1	8.9	0	1.3
C2a	11.9	15	1	M1	8.6	2	1.2
C3a	6.2	0	1.05	N1	11.2	1	1.2
C4	9.8	0	1.05	N2	12.2	1	1.03

Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Sub-catchment	Area (ha)	Impervious (%)	Orographic factor
C5	4.0	0	1	N3	29.5	1	1
C6	6.4	0	1	N4	111.8	0	1
C7	23.7	0	1.375	O1	9.8	2	1
C8	50.7	0	1.08	O2	45.4	0	1
C9	21.5	0	1	P1	8.6	0	1
D1a	21.3	0	1.525	Q1	9.1	5	1
D2	32.6	0	1.15	R9	18.9	30	1.2
D3	6.7	0	1.3	R8	11.7	20	1.2
D4	16.0	0	1.09	R6	27.6	0	1.2
D5	20.1	0	1.02	R7	12.2	0	1
E1a	6.6	5	1	R1	47.0	0	1.2
E2	11.2	5	1	R2	42.1	0	1.2
E3	10.9	2	1	R3	50.9	15	1.2
F1a	4.6	0	1	R4	55.4	70	1
F2a	6.4	0	1	R5	37.7	0	1
F3a	4.0	0	1	S1	12.1	0	1
F3b	6.3	0	1.075	S2	23.9	0	1
G1a	13.4	0	1.2				

Table 33: Coffs Creek XPRafts - existing scenario

Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)	Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)
C1	20.9	0.0	1.2	25.5	C49	23.1	55.8	1	0.3
C2	34.6	0.0	1.2	20.3	C50	83.5	0.0	1.2	7.6
C3	33.5	0.0	1.2	13.5	C51	39.4	0.0	1.2	1.6
C4	34.3	0.0	1.2	13.2	C52	24.0	36.0	1.2	1.1
C5	37.7	0.1	1.2	12.9	C53	37.9	44.1	1	0.8
C6	57.3	0.0	1.2	7.1	C54	17.8	0.0	1.2	4.7
C7a	9.0	0.0	1.2	9.7	C55	45.2	48.5	1.2	0.7
C7b	28.4	0.3	1	2.3	C57	23.4	32.8	1.2	0.7
C8	19.2	0.5	1.6	18.4	C58	29.4	0.0	1.2	15.3
C9	7.1	1.1	1.6	9.0	C59	31.2	74.6	1	1.1
C10	20.6	61.9	1	0.9	C60	67.4	0.0	1.2	13.0
C11	9.2	0.8	1.25	13.5	C61	19.2	0.0	1.2	1.3
C12	16.1	13.4	1.2	6.0	C62	5.7	34.0	1	1.2
C13	18.0	0.0	1.2	8.8	C63	15.1	55.9	1	4.6
C14	6.0	24.6	1	21.7	C64	10.0	54.5	1	3.9
C15	32.9	0.0	1.2	7.9	C65	15.7	0.0	1.2	11.9
C16	17.6	0.1	1.2	9.2	C66	25.6	37.8	1	0.3
C17	27.0	0.0	1.2	7.2	C67	46.1	0.0	1	9.6
C18	41.5	0.0	1.2	6.3	C68	17.2	2.1	1	16.7
C19	29.4	3.3	1.2	5.6	C69	14.0	56.6	1	0.3

Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)	Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)
C20	23.4	53.7	1.6	2.1	C70	18.7	0.0	1	18.3
C21	13.0	35.0	1.6	2.2	C71	12.5	10.7	1	17.6
C22	5.9	47.1	1	1.7	C72	4.3	54.5	1	0.7
C23	11.3	27.9	1	6.0	C73	41.4	51.8	1	0.6
C24	29.9	49.7	1.2	2.0	C74	59.0	52.9	1	0.5
C25	19.8	29.4	1.6	2.6	C75	54.2	53.9	1	0.1
C26	13.0	54.2	1.6	2.8	C76	42.8	34.8	1	2.5
C27	19.7	28.1	1	1.2	C77	30.1	51.5	1.2	1.0
C28	44.0	0.0	1.2	6.6	C78	4.5	15.7	1.2	24.3
C29	11.9	0.3	1.6	2.5	C79	36.7	29.4	1	1.0
C30	7.6	37.0	1	0.3	C80	4.9	74.0	1	0.6
C31	37.7	0.0	1.2	7.7	C81	20.3	2.5	1	0.4
C32	30.9	38.8	1	0.9	C82	19.8	51.1	1	0.6
C33	31.0	56.5	1	1.5	C83	26.1	11.8	1	4.7
C34	26.5	0.0	1.6	2.9	C84	9.7	90.6	1	0.2
C35	9.9	82.4	1	2.3	C85	44.4	44.6	1.2	4.8
C36	23.4	28.9	1.6	2.3	C87	26.9	85.6	1	1.0
C37	38.3	57.4	1.6	1.6	C88	25.0	31.9	1	1.1
C38	30.6	73.1	1	1.6	C89	24.5	43.0	1	0.7
C39	49.6	54.6	1	0.5	C90	18.4	50.2	1	1.4
C40a	5.1	4.4	1.6	3.6	C91	22.0	59.0	1	1.2

Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)	Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)
C40b	11.7	0.0	1.6	0.7	C92	16.9	14.1	1	0.6
C40c	2.2	0.0	1.6	9.3	C93	11.0	63.8	1	2.2
C41	25.4	52.3	1.6	1.7	C94	13.4	63.1	1	2.2
C42	38.4	45.8	1.6	2.3	C95	10.6	15.7	1	0.5
C43	18.6	5.2	1.6	0.9	C96	16.7	50.1	1	3.6
C44	15.7	26.2	1	0.2	C97	24.0	10.6	1	1.9
C45	30.4	47.8	1	0.4	C98	26.0	65.0	1	3.9
C46	41.8	44.5	1.01	1.3	C99	9.0	65.0	1.2	2.2
C47	29.3	20.2	1.2	0.7	C100	20.6	61.9	1	0.9

Table 34: Coffs Creek XPRafts - developed modifications

Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)	Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)
C1	20.9	0.0	1.2	25.5	CC3	0.5	100.0	1.2	1.1
C2	34.6	0.0	1.2	20.3	CC4	1.7	100.0	1.2	1.6
C3	33.5	0.0	1.2	13.5	CC5	1.5	100.0	1.2	1.5
C4	33.5	0.0	1.2	13.2	CC6	2.8	100.0	1.2	2.2
C5	36.8	0.1	1.2	12.9	CC7	0.7	100.0	1.2	3.8
C6	57.3	0.0	1.2	7.1	CC8	0.4	100.0	1.2	3.2
C7a	9.0	0.0	1.2	9.7	CC9	0.02	100.0	1.2	1.2
C7b	28.4	0.3	1.2	2.3	CC10	0.1	100.0	1.2	0.1

Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)	Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)
C10	24.9	0.0	1.2	3.5	CC12	5.1	100.0	1.2	3.9
C12	15.2	14.2	1.2	6.0	CC13	0.05	100.0	1.2	0.6
C13	16.4	0.0	1.2	8.8	CC15	1.1	100.0	1.2	6.8
C17	25.9	0.0	1.2	7.2	CC16	0.8	100.0	1.6	5.4
C18	40.9	0.0	1.2	6.3	CC17	1.9	100.0	1.2	2.8
C29	11.2	0.4	1.6	2.5	CC18	3.8	100.0	1.2	0.1
C34	25.5	0.0	1.6	2.9	CC19	1.4	100.0	1.2	1.9
C40a	3.8	5.8	1.6	3.6	CC20	0.5	100.0	1.2	2.3
C40b	11.0	0.0	1.6	0.7	CC21	0.2	100.0	1.2	4.3
C40c	1.6	0.1	1.6	9.3	CC22	0.1	100.0	1.2	13
C51	39.1	0.0	1.2	1.6	CC23	0.1	100.0	1.2	1.9
C54	16.9	0.0	1.2	4.7	CC24	0.1	100.0	1.2	0.5
C61	16.5	0.0	1.2	1.3	CC25	0.4	100.0	1.2	0.5
C67	44.9	0.0	1	9.6	CC26	0.4	100.0	1.2	5.7
CC1	4.9	100.0	1.4	3.3	CC27	0.2	100.0	1.2	2.6
CC2	4.1	100.0	1.4	3.3					

Table 35: Northern Creeks XPRafts - existing scenario

Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)	Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)
A01.01	6.7	0.0	1.2	30.0	C02.01	24.4	0.0	1.0	28.5
A01.02	6.5	0.0	1.2	36.7	C03.01	17.9	0.0	1.0	50.0
A01.03	6.1	0.0	1.2	40.0	C04.01	10.9	0.0	1.0	34.2
A01.04	18.0	2.0	1.2	22.9	C05.01	38.5	0.0	1.0	31.2
A01.05	26.8	0.0	1.6	27.3	C06.01	38.0	0.0	1.3	41.0
A01.06	18.3	0.0	1.2	2.6	C06.02	8.6	0.0	1.3	6.4
A01.07	12.1	2.0	1.2	4.0	C06.03	5.9	0.0	1.3	22.1
A01.08	7.2	20.0	1.2	2.7	C06.04	10.7	0.0	1.3	22.0
A01.09	8.5	27.5	1.0	13.2	C06.05	5.7	0.0	1.3	6.9
A01.10	13.6	67.2	1.0	15.5	C06.06	8.4	0.0	1.0	12.9
A02.01	18.5	0.0	1.2	55.8	C07.01	28.6	0.0	1.3	47.5
A02.02	2.3	0.0	1.2	22.4	C08.01	12.4	0.0	1.3	40.0
A03.01	14.0	0.0	1.2	54.9	C08.02	17.9	0.0	1.3	13.0
A04.01	7.1	0.0	1.2	9.7	C09.01	9.1	0.0	1.3	29.7
A06.01	12.5	0.0	1.0	19.1	C09.02	10.9	0.0	1.3	24.0
A07.01	12.7	0.0	1.2	40.3	C09.03	4.6	0.0	1.3	8.5
A08.01	11.1	8.0	1.0	24.0	C10.01	26.4	0.0	1.0	40.0
A09.01	17.3	30.0	1.2	23.4	C10.02	21.0	0.0	1.0	28.1
A09.02	4.3	30.0	1.0	11.0	C10.03	18.5	0.0	1.0	7.0

Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)	Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)
A10.01	5.3	30.0	1.0	18.7	C10.04	15.7	0.0	1.0	4.6
B01.01	9.5	0.0	1.6	34.7	C10.05	4.0	10.0	1.0	12.3
B01.02	7.4	5.0	1.0	25.2	C11.01	13.8	0.0	1.0	30.6
B01.03	4.6	5.0	1.0	18.1	C11.02	8.9	0.0	1.0	16.8
B01.04	0.8	5.0	1.0	3.0	C11.03	9.4	0.0	1.0	3.7
B01.05	2.4	20.0	1.0	11.0	C12.01	43.4	0.0	1.2	41.6
B01.06	0.4	0.0	1.0	12.8	C12.02	12.9	5.0	1.2	4.6
B01.07	1.3	0.0	1.0	3.8	C12.03	17.0	0.0	1.2	20.6
B01.08	4.2	0.0	1.0	15.8	C12.04	18.4	5.0	1.2	3.5
B01.09	7.3	10.0	1.0	7.0	C12.05	4.7	5.0	1.0	1.0
B02.01	4.5	5.0	1.0	11.7	C13.01	5.8	0.0	1.2	41.5
B03.01	6.3	0.0	1.6	26.8	C13.02	7.7	5.0	1.2	15.0
B03.02	2.2	5.0	1.0	26.3	C14.01	26.8	0.0	1.1	50.0
B03.03	0.6	0.0	1.0	8.7	C14.02	25.0	10.0	1.1	3.4
B03.04	2.4	0.0	1.0	10.2	C14.03	6.0	5.0	1.1	3.6
B04.01	3.0	5.0	1.6	31.3	C15.01	4.7	5.9	1.0	11.9
B04.02	4.2	0.0	1.0	28.0	C16.01	8.2	5.0	1.0	26.0
B05.01	3.1	5.0	1.6	30.8	C16.02	6.0	0.0	1.0	4.5
B05.02	1.9	0.0	1.0	25.0	C16.03	5.8	15.0	1.0	20.0
B06.01	1.6	20.0	1.0	19.1	C16.04	0.4	30.0	1.0	2.2

Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)	Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)
B06.02	1.0	70.0	1.0	4.3	C17.01	3.8	0.0	1.0	19.6
B06.03	0.9	40.0	1.0	7.9	C17.02	3.6	5.0	1.0	7.5
B07.01	9.7	20.0	1.0	22.8	C17.03	2.1	20.0	1.0	15.2
B08.01	2.6	0.0	1.0	11.2	C18.01	2.5	0.0	1.0	18.3
B09.01	4.3	0.0	1.0	9.3	C18.02	2.8	30.0	1.0	8.3
B09.02	9.4	10.0	1.0	4.3	C18.03	2.4	30.0	1.0	18.2
C01.01	45.7	0.0	1.2	32.8	C19.01	2.1	70.0	1.0	4.0
C01.02	29.8	1.0	1.2	14.7	D01.01	19.9	35.5	1.0	16.0
C01.03	21.3	2.0	1.2	23.6	D01.02	18.8	63.8	1.0	2.0
C01.04	8.4	0.0	1.2	3.7	D01.03	16.4	68.0	1.0	3.0
C01.05	7.1	0.0	1.0	17.0	E01.01	15.3	0.0	1.2	20.0
C01.06	9.9	0.0	1.0	7.6	E01.02	11.6	0.0	1.2	18.8
C01.07	0.8	20.0	1.0	9.4	E01.03	13.7	0.0	1.0	3.2
C01.08	9.0	30.0	1.0	4.8	E01.04	9.5	31.5	1.0	7.4
C01.09	9.7	34.0	1.0	5.1					

Table 36: Northern Creeks XPRafts - developed modifications

Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)	Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)
A01.04	16.7	2.0	1.2	22.9	C18.03	1.3	30.0	1.0	18.2
A01.05	25.9	0.0	1.6	27.3	C19.01	0.7	70.0	1.0	4.0
A01.06	15.9	0.0	1.2	2.6	D01.01	18.8	35.5	1.0	16.0
A04.01	6.3	0.0	1.2	9.7	D01.02	17.6	63.8	1.0	2.0
B01.02	7.1	5.0	1.0	25.2	E01.03	13.6	0.0	1.0	3.2
B01.03	1.5	5.0	1.0	18.1	E01.04	7.7	31.5	1.0	7.4
B01.04	0.2	5.0	1.0	3.0	A1.05_IP1	1.0	3.2	1.6	0.0
B01.05	1.7	20.0	1.0	11.0	A01.06_IP1	0.3	0.3	1.0	0.0
B02.01	4.0	5.0	1.0	11.7	A01.06_IP2	3.7	3.2	1.0	0.0
B03.01	6.2	0.0	1.6	26.8	A04.01_IP1	1.4	0.0	1.2	0.0
B03.02	1.3	5.0	1.0	26.3	B02.01_IP1	1.2	1.6	1.0	0.0
B03.03	0.2	0.0	1.0	8.7	B04.01_IP1	0.2	7.7	1.6	0.0
B03.04	0.5	0.0	1.0	10.2	B05.01_IP1	0.5	7.0	1.6	0.0
B04.01	2.6	5.0	1.6	31.3	B05.02_IP1	1.7	2.2	1.0	0.0
B04.02	2.6	0.0	1.0	28.0	B05.02_IP2	1.6	0.7	1.0	0.0
B05.01	2.6	5.0	1.6	30.8	B06.03_IP1	2.0	2.9	1.0	0.0
B05.02	1.0	0.0	1.0	25.0	B07.01_IP1	0.2	4.9	1.0	0.0
B06.01	1.2	20.0	1.0	19.1	C01.07_IP1	5.5	5.2	1.0	0.0
B06.02	0.3	70.0	1.0	4.3	C01.08_IP1	0.6	0.8	1.0	0.0

Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)	Sub-catchment	Area (ha)	Impervious (%)	Orographic factor	Vectored slope (%)
B06.03	0.8	40.0	1.0	7.9	C01.08_IP2	1.5	0.1	1.0	0.0
B07.01	9.6	20.0	1.0	22.8	C01.09_IP1	0.4	3.3	1.0	0.0
B09.01	3.7	0.0	1.0	9.3	C10.05_IP1	0.3	6.6	1.0	0.0
B09.02	8.4	10.0	1.0	4.3	C11.02_IP1	0.8	1.1	1.0	0.0
C01.07	0.6	20.0	1.0	9.4	C11.03_IP1	0.8	1.8	1.0	0.0
C01.08	8.2	30.0	1.0	4.8	C16.03_IP1	0.5	4.0	1.0	0.0
C01.09	8.8	34.0	1.0	5.1	C16.03_IP2	1.1	0.6	1.0	0.0
C10.05	3.8	10.0	1.0	12.3	C16.04_IP1	4.7	3.5	1.0	0.0
C11.02	7.6	0.0	1.0	16.8	C19.01_IP1	0.1	1.9	1.0	0.0
C11.03	8.1	0.0	1.0	3.7	E01.03_IP3	0.8	1.8	1.0	0.0
C12.05	3.7	5.0	1.0	1.0	E01.03_IP4	0.8	0.7	1.0	0.0
C15.01	1.0	5.9	1.0	11.9	E01.03_IP2	0.1	2.9	1.0	0.0
C16.03	4.1	15.0	1.0	20.0	E01.03_IP1	0.1	0.0	1.0	0.0
C16.04	0.3	30.0	1.0	2.2	B06.03_IP2	2.0	2.9	1.0	0.0
C17.02	3.3	5.0	1.0	7.5	B09.02_IP2	4.5	4.4	1.0	0.0
C17.03	0.7	20.0	1.0	15.2	B09.02_IP1	0.6	2.1	1.0	0.0

A2 Hydraulic structure parameters

Refer to Figures within **Appendix B** and **Appendix D** for existing and developed structure IDs respectively. **Figure A2** below presents a reduced scale insert of the heavily populated Treefern Creek crossing location.

Table 37: Hydraulic structures – existing

ID	Arrangement	Length (m)	US/DS Invert level (mAHD)	Additional comments
ES01	1/1.5 m RCP	55	3.44 / 2.04	Developed extension (DS01)
ES02	1/0.9 m RCP	57	3.81 / 3.21	
ES03	1/0.45 m RCP	161	3.20 / 1.70	
ES04	4/1.05 m RCP	30	2.89 / 2.74	Developed extension (DS07). Adopted (GHD, 2016) arrangement.
ES05	16/2.1 x 0.9 m RCBC	28	2.95 / 2.37	Developed extension (DS08). Adopted (GHD, 2016) arrangement.
ES06	10/2.4 x 0.9 m RCBC	20	4.75 / 4.60	
ES07	2/1.5 m RCP	9	4.90 / 4.70	
ES08	1/1.8 m RCP	8	14.44 / 14.40	
ES09	1/1 x 0.5 m RCBC	4	12.20 / 12.20	
ES10	1/0.9 m RCP	10	24.50 / 23.25	
ES11	1/0.6 m RCP	9	24.17 / 24.05	
ES12	2/1.8 m RCP	9	28.38 / 26.26	
ES13	2/3.6 x 1.2 m RCBC	10	23.40 / 23.35	
ES14	1/0.45 m RCP	11	29.04 / 28.42	
ES15	1/0.6 m RCP	10	34.00 / 34.00	
ES16	1/0.45 m RCP	9	34.10 / 33.97	
ES17	1/15 m bridge spans	17	8.50	Loss coefficient 0.00
ES18	2/0.875 m RCP	5	9.50 / 9.50	
ES19	2/3.5 x 3.6 m RCBC	13	12.55 / 12.45	
ES20	2/1.8 m RCP	3	17.62 / 17.60	Developed removal
ES21	1/3 x 1.8 m RCBC	3	17.60 / 17.62	Developed removal
ES22	1/3.3 x 1.2 m RCBC	15	20.10 / 20.00	Developed removal
ES23	1/3.3 x 1.2 m RCBC	54	21.46 / 20.92	
ES24	2/3 x 2.1 m RCBC	12	12.70 / 12.70	
ES25	1/3 x 0.9 m RCBC	5	11.20 / 11.20	
ES26	1/3 x 2.4 m RCBC	35	11.20 / 11.20	
ES27	1/0.825 m RCP	13	17.25 / 17.05	
ES28	1/0.9 m RCP	13	16.70 / 16.50	
ES29	1/1.8 m RCP	9	13.50 / 13.49	
ES30	2/1.05 m RCP	77	18.00 / 16.40	
ES31	4/0.9 m RCP	22	20.00 / 19.68	
ES32	1/1.5 m RCP	15	18.17 / 18.14	
ES33	1/0.6 m RCP	15	22.90 / 22.64	
ES34	0/0.75 m RCP	24	26.69 / 26.04	
ES35	3/2.15 x 2.15 m RCBC	12	20.72 / 20.57	
ES36	2/1.2 m RCP	9	34.90 / 34.80	
ES37	1/0.61 x 0.61 m RCBC	27	65.76 / 62.25	
ES38	1/0.91 x 0.91 m RCBC	18	65.73 / 64.10	
ES39	1/1.52 x 1.52 m RCBC	41	53.26 / 51.51	
ES40	1/1.52 x 1.52 m RCBC	41	57.70 / 50.39	

ID	Arrangement	Length (m)	US/DS Invert level (mAHD)	Additional comments
ES41	1/1.5 m RCP	11	40.40 / 40.28	
ES42	1/10 m bridge spans	10	30.60	Loss coefficient 0.023
ES43	1/0.9 m RCP	10	41.58 / 41.47	Developed removal
ES44	1/1.35 m RCP	6	41.47 / 41.08	
ES45	1/1.8 m RCP	11	35.36 / 35.17	
ES46	1/0.45 m RCP	7	31.10 / 30.93	
ES47	1/0.9 m RCP	8	27.40 / 27.25	
ES48	1/0.75 m RCP	7	28.50 / 28.37	
ES49	1/0.9 m RCP	6	22.57 / 22.33	
ES50	2/0.525 m RCP	7	12.81 / 12.74	
ES51	1/13.5 x 3.3 m RCBC	26	12.21 / 9.58	15 % blockage. Unusual dimensions
ES52	1/0.375 m RCP	16	14.84 / 14.24	30 % blockage
ES53	1/1.5 m RCP	28	17.59 / 16.04	5 % blockage
ES54	1/0.375 m RCP	35	27.43 / 25.22	15 % blockage
ES55	1/0.375 m RCP	55	32.63 / 31.07	
ES56	1/0.375 m RCP	24	41.03 / 39.25	
ES57	1/0.45 m RCP	38	20.80 / 19.00	
ES58	1/4.8 x 3 m RCBC	50	18.69 / 17.33	
ES59	1/0.75 m RCP	10	29.12 / 28.83	Developed removal
ES60	1/1.05 m RCP	13	40.77 / 40.61	
ES61	1/2.4 x 2.4 m RCBC	57	22.00 / 20.88	
ES62	1/0.525 m RCP	9	54.77 / 53.41	Developed removal
ES63	1/0.75 m RCP	14	68.42 / 67.88	
ES64	1/0.375 m RCP	36	43.35 / 42.89	Developed removal
ES65	1/0.45 m RCP	14	47.40 / 47.30	Developed removal
ES66	1/0.45 m RCP	13	47.30 / 47.00	25 % blockage. Developed removal
ES67	1/2.5 x 2.86 m RCBC	35	47.92 / 47.01	15 % blockage. Developed removal
ES68	1/0.45 m RCP	13	44.11 / 43.79	Developed removal
ES69	1/18 m bridge spans	20	10.57	Loss coefficient 0
ES70	2/20 m bridge spans	40	12.88	Loss coef. 0.118. Developed removal
ES71	2/20 m bridge spans	40	13.45	Loss coefficient 0.0825
ES72	1/15 m bridge spans	15	11.46	Loss coefficient 0
ES73	1/0.45 m RCP	44	11.90 / 11.69	Developed removal
ES74	1/30 m bridge spans	30	9.77	Loss coefficient 0
ES75	1/30 m bridge spans	30	6.73	Loss coefficient 0
ES76	3/0.75 m RCP	44	10.10 / 10.10	Developed removal
ES77	1/0.45 m RCP	31	11.40 / 10.13	Developed removal
ES78	1/0.45 m RCP	41	13.41 / 11.95	Assumed from imagery and ALS. Developed removal.
ES79	3/0.75 m RCP	46	14.69 / 11.63	Developed removal.
ES82	1/2.1 x 1.2 m RCBC	62	17.68 / 13.46	15 % blockage. Assumed from imagery and ALS.

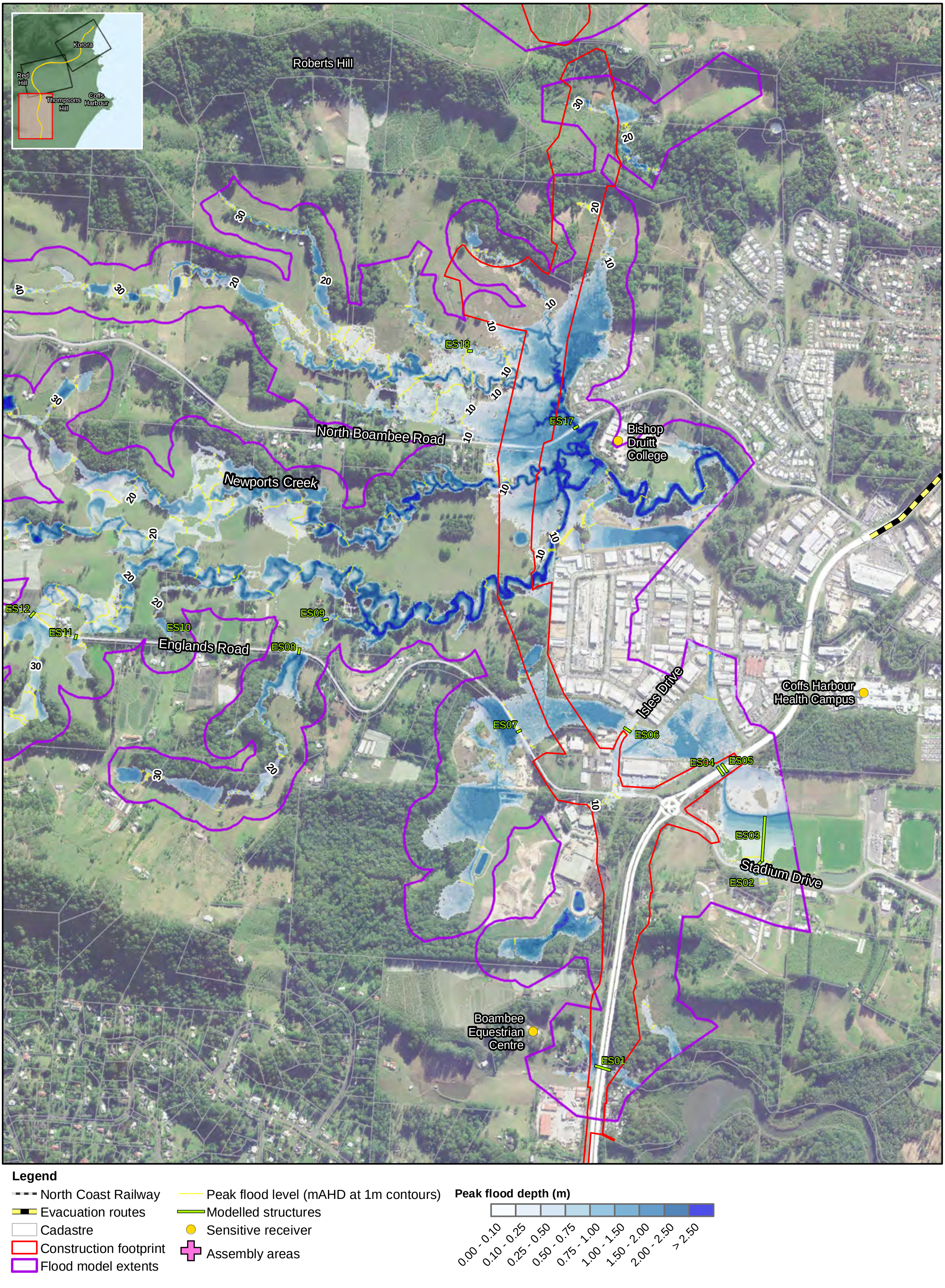
ID	Arrangement	Length (m)	US/DS Invert level (mAHD)	Additional comments
ES83	2/2.1 x 1.2 m RCBC	96	11.05 / 6.58	Assumed from imagery and ALS
ES84	1/0.9 x 0.6 m RCBC	27	14.76 / 14.12	Assumed from imagery and ALS
ES99	1/25 m bridge spans	25	3.15	Opening of bridge modelled
ES100	1/1.5 m RCP	25	7.96 / 7.54	Assumed 1.5 m RCP due to upstream dimension (0.9 m provided)
ES101	1/1.5 m RCP	25	6.47 / 6.40	
ES103	1/1.5 m RCP	23	9.79 / 7.96	
ES104	1/1.5 m RCP	26	6.40 / 4.79	
ES105	1/1.5 m RCP	22	7.54 / 7.15	Assumed 1.5 m RCP due to upstream dimension (1.05 m provided).
ES106	1/0.45 m RCP	7	7.43 / 6.48	
ES107	1/0.375 m RCP	10	8.29 / 7.55	
ES108	1/0.525 m RCP	20	6.48 / 6.46	
ES109	1/1.5 m RCP	12	7.07 / 7.00	
ES110	1/0.375 m RCP	28	7.74 / 7.61	
ES111	1/1.5 m RCP	19	7.09 / 7.07	Assumed 1.5 m RCP due to upstream dimension (1.35 m provided).
ES112	1/0.375 m RCP	29	7.61 / 7.43	
ES113	1/0.375 m RCP	46	13.97 / 10.83	
ES114	1/0.375 m RCP	8	12.46 / 13.97	
ES115	1/0.375 m RCP	23	15.77 / 13.97	
ES116	1/0.375 m RCP	8	7.70 / 7.61	
ES117	1/0.375 m RCP	20	6.50 / 4.80	
ES118	1/0.6 m RCP	14	7.65 / 7.10	
ES119	1/0.6 m RCP	10	7.89 / 7.65	
ES120	1/0.45 m RCP	35	10.83 / 7.89	
ES121	1/0.375 m RCP	12	13.01 / 12.48	
ES122	1/0.375 m RCP	29	12.40 / 9.33	
ES123	1/0.375 m RCP	19	9.33 / 7.53	
ES124	1/0.375 m RCP	20	7.43 / 6.50	
ES125	1/1.5 m RCP	6	10.04 / 9.79	
ES126	1/1.5 m RCP	18	12.93 / 10.04	
ES127	1/1.2 m RCP	11	11.42 / 10.80	
ES128	1/0.375 m RCP	31	17.10 / 12.79	
ES129	1/1.5 m RCP	10	4.43 / 4.30	
ES130	1/0.45 m RCP	17	7.26 / 7.00	
ES131	1/0.375 m RCP	20	13.47 / 9.79	
ES132	1/1.2 m RCP	21	5.96 / 3.54	
ES133	1/1.2 m RCP	77	10.72 / 6.67	
ES134	1/1.5 m RCP	12	4.46 / 4.34	
ES135	1/1.2 m RCP	20	6.67 / 5.96	
ES136	1/0.375 m RCP	23	7.06 / 3.77	

ID	Arrangement	Length (m)	US/DS Invert level (mAHD)	Additional comments
ES137	1/0.375 m RCP	9	7.72 / 6.40	
ES138	1/0.375 m RCP	2	6.32 / 5.96	
ES139	1/0.375 m RCP	8	7.13 / 4.79	
ES140	1/1.5 m RCP	13	4.77 / 4.46	

Appendix B

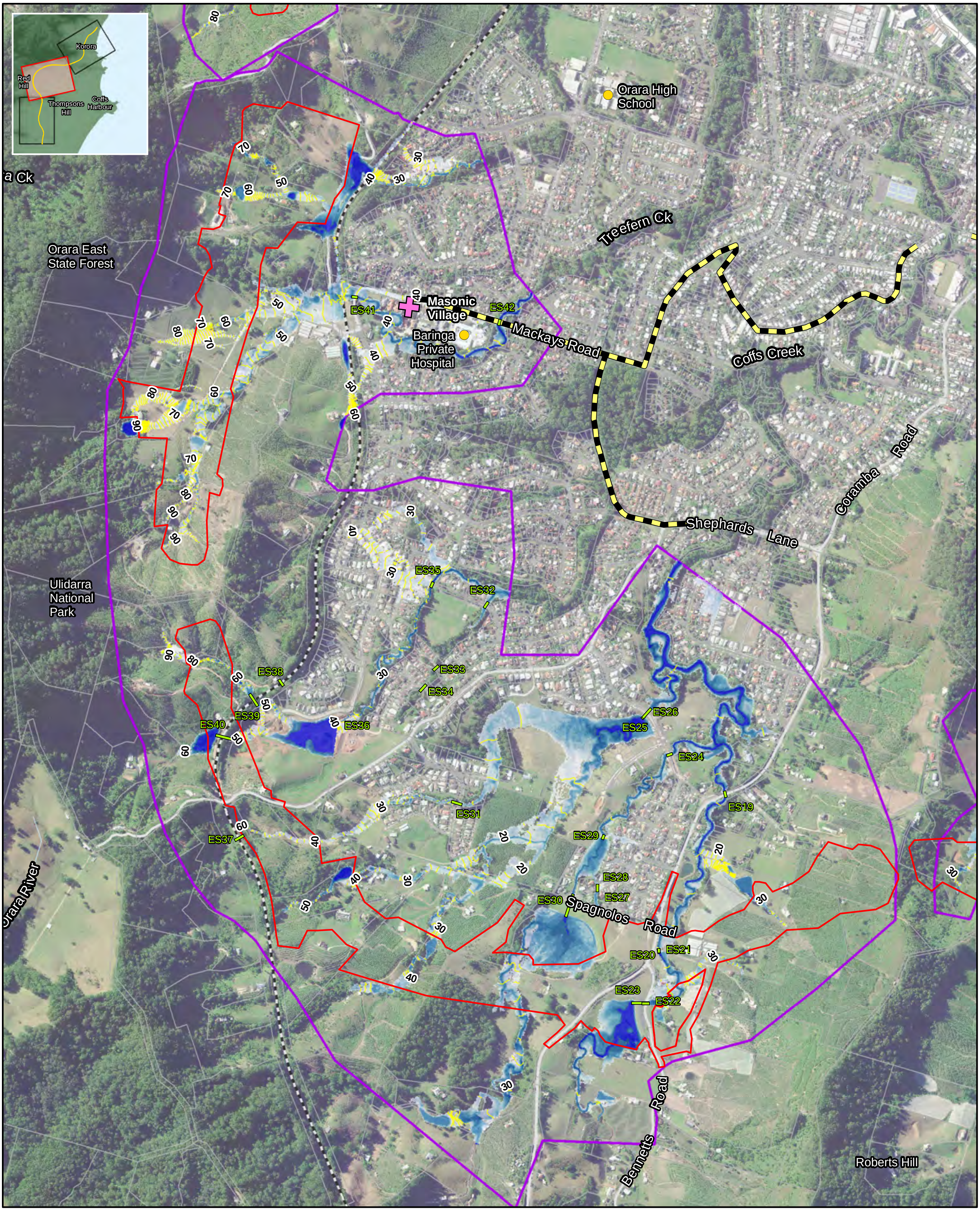
Existing flood maps

B1 Peak flood level and depth



Coffs Harbour Bypass
North Boambee Valley 18 % AEP peak flood level and depth
B1.1.1

Scale @A4: 1:17,500
GDA 1994 MGA Zone 56



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Peak flood level (mAHD at 1m contours)
- Modelled structures
- Sensitive receiver
- Assembly areas

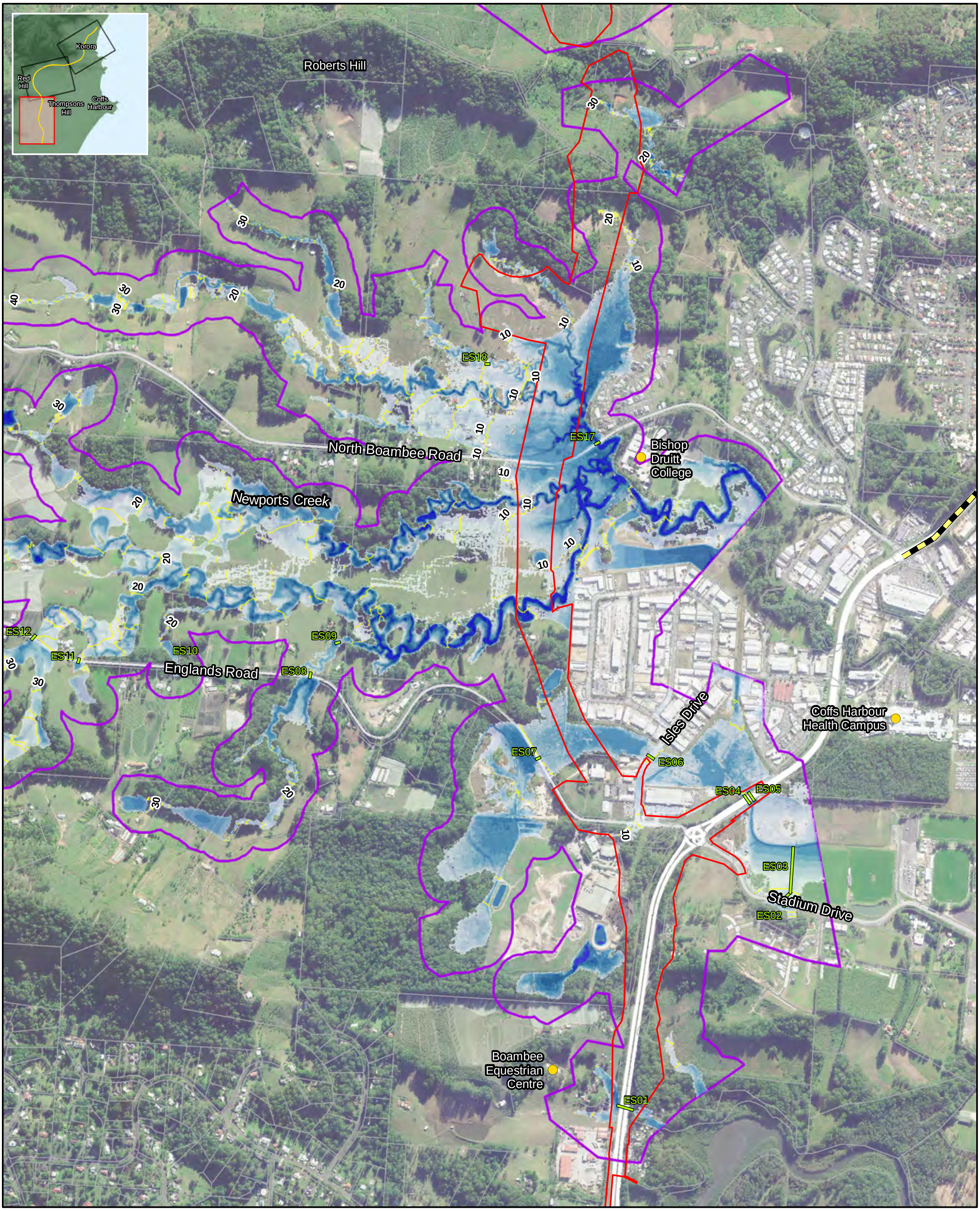
Peak flood depth (m)

0.00 - 0.10, 0.10 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.50, 1.50 - 2.00, 2.00 - 2.50, > 2.50

Coffs Harbour Bypass
Coffs Creek 18 % AEP peak flood level and depth
B1.2.1

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N

Coffs Harbour Bypass
Northern Creek 18 % AEP peak flood level and depth
B1.3.1



Legend

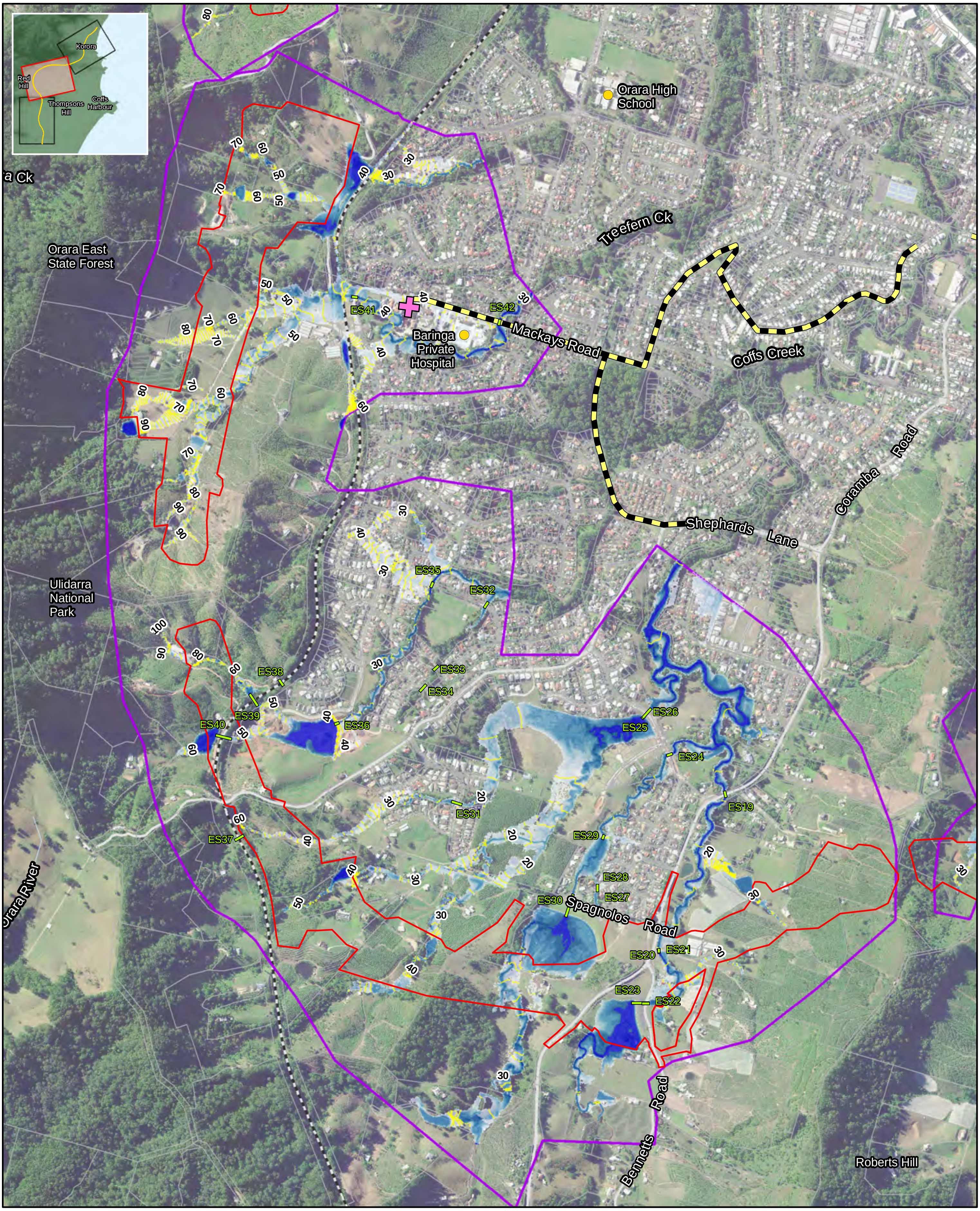
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Peak flood level (mAHD at 1m contours)
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood depth (m)

0.00 - 0.10, 0.10 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.50, 1.50 - 2.00, 2.00 - 2.50, > 2.50

Coffs Harbour Bypass
North Boambee Valley 10 % AEP peak flood level and depth
B1.1.2

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

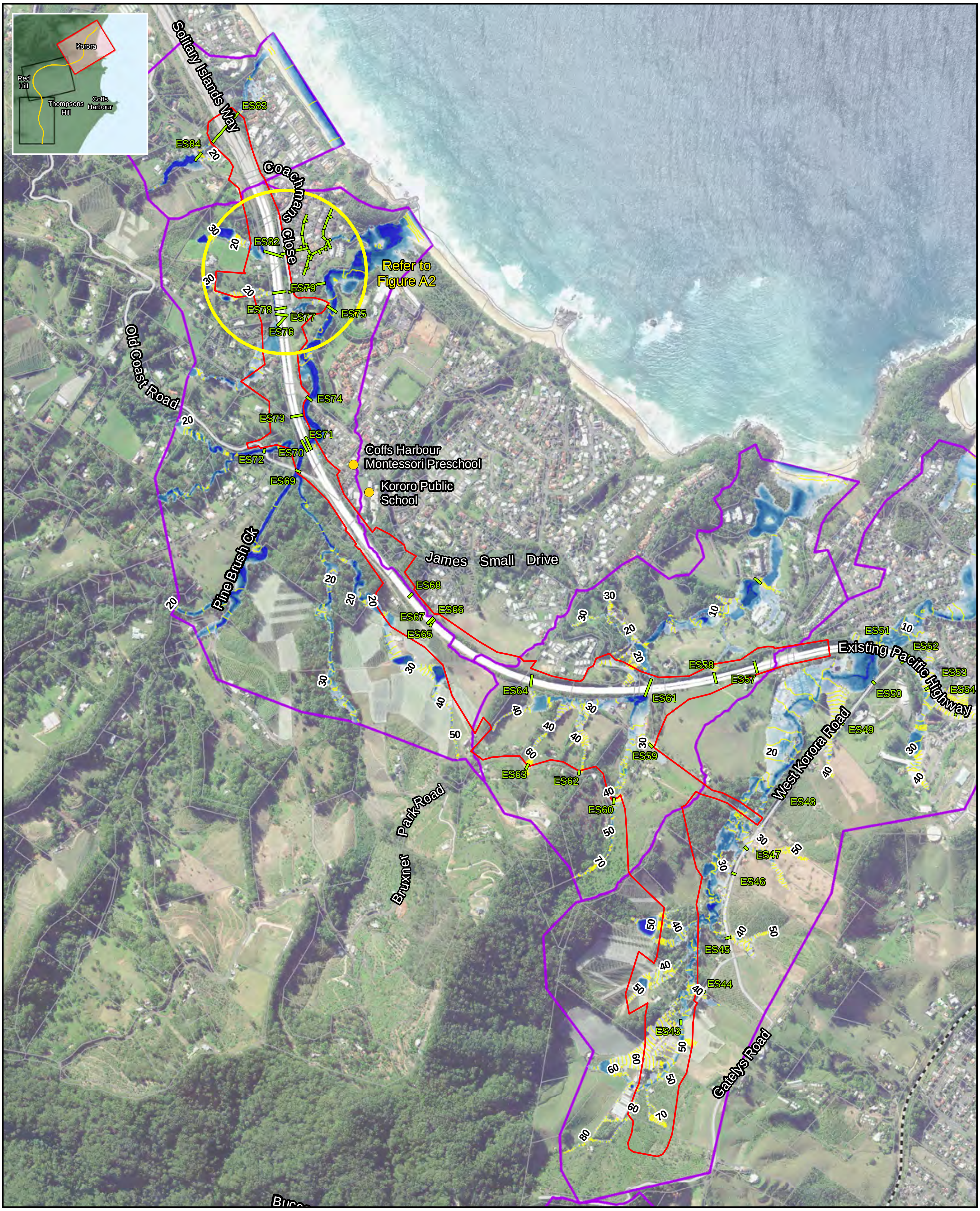
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Peak flood level (mAHD at 1m contours)
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood depth (m)

0.00 - 0.10, 0.10 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.50, 1.50 - 2.00, 2.00 - 2.50, > 2.50

Coffs Harbour Bypass
Coffs Creek 10 % AEP peak flood level and depth
B1.2.2

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56



Legend

- North Coast Railway
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- Sensitive receiver
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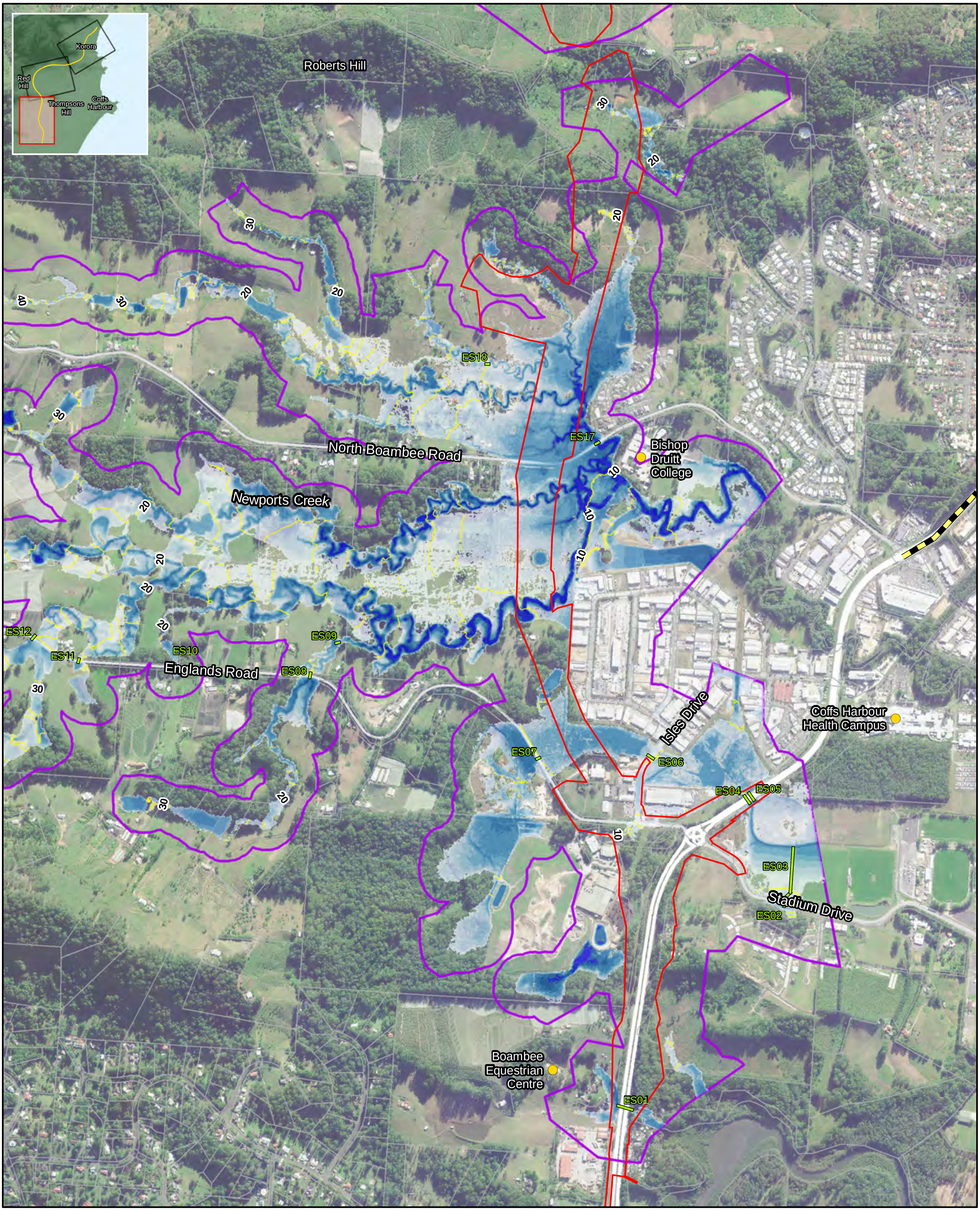
Peak flood depth (m)

0.00 - 0.10, 0.10 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.50, 1.50 - 2.00, 2.00 - 2.50, > 2.50

Coffs Harbour Bypass
Northern Creek 10 % AEP peak flood level and depth
B1.3.2

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56



Legend

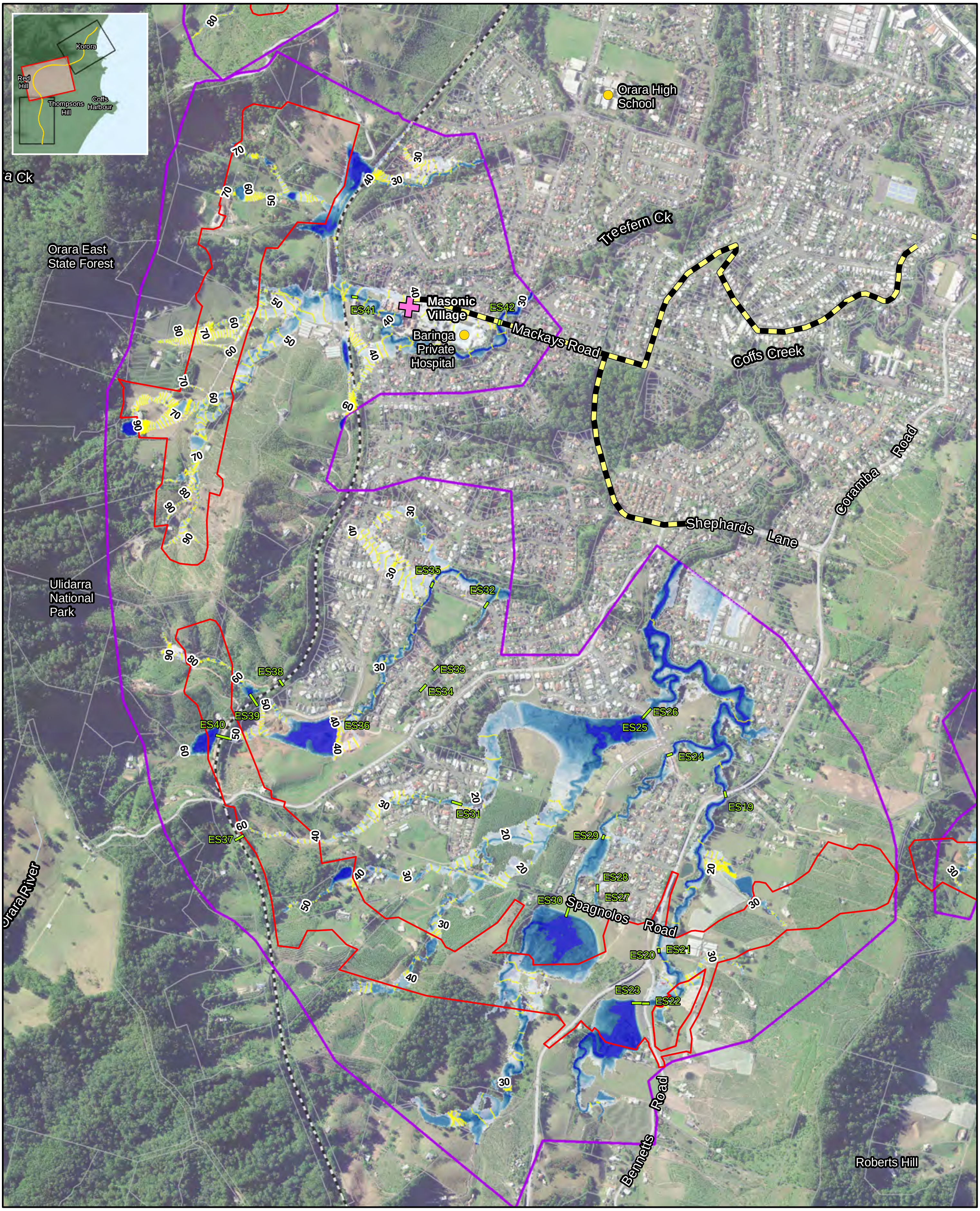
North Coast Railway
Evacuation routes
Cadastre
Construction footprint
Flood model extents
Peak flood level (mAHD at 1m contours)
Modelled structures
Sensitive receiver
Assembly areas

Peak flood depth (m)

0.00 - 0.10
0.10 - 0.25
0.25 - 0.50
0.50 - 0.75
0.75 - 1.00
1.00 - 1.50
1.50 - 2.00
2.00 - 2.50
> 2.50

Coffs Harbour Bypass
North Boambee Valley 5 % AEP peak flood level and depth
B1.1.3

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

North Coast Railway
Evacuation routes
Cadastre
Construction footprint
Flood model extents
Peak flood level (mAHD at 1m contours)
Modelled structures
Sensitive receiver
Assembly areas

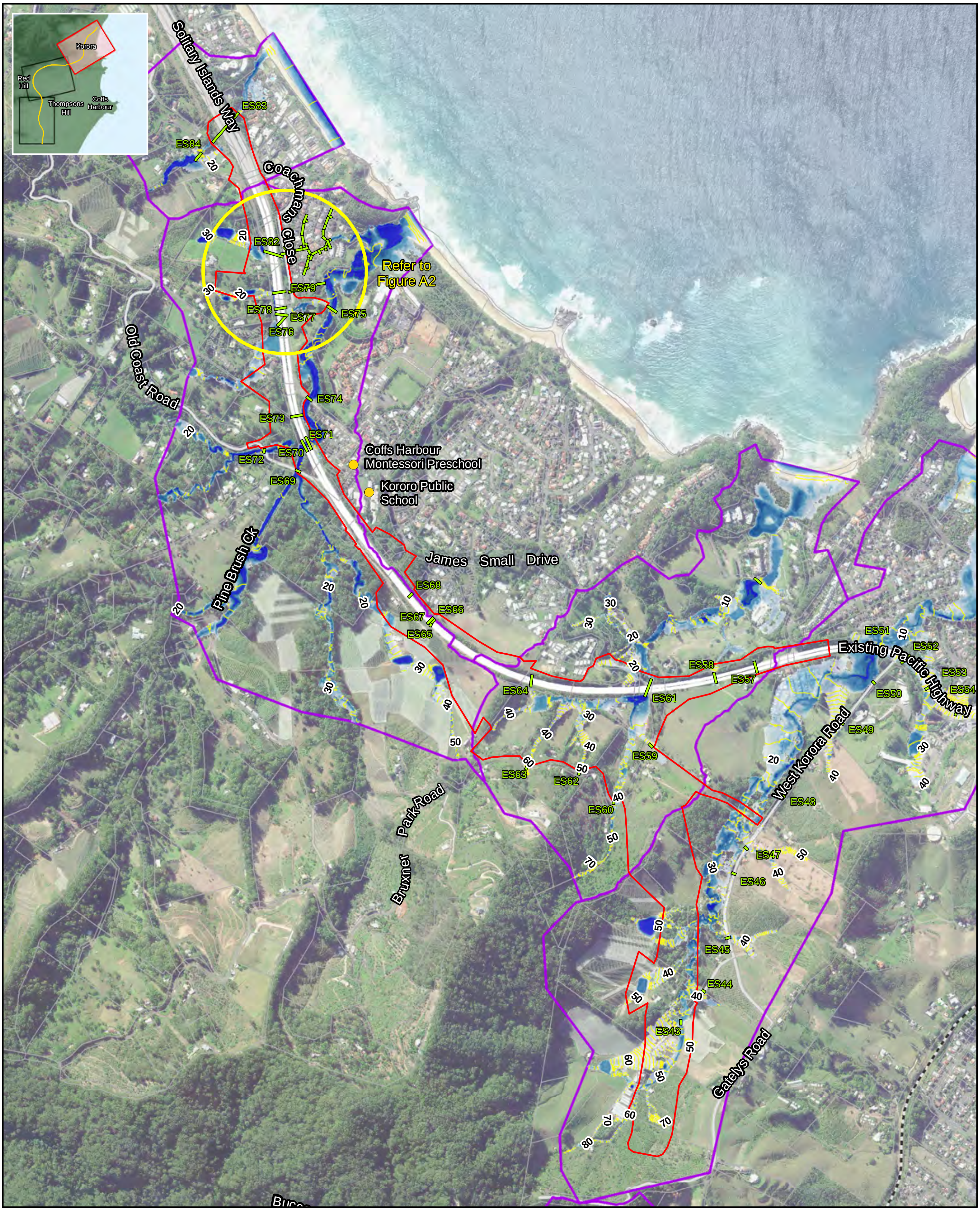
Peak flood depth (m)

0.00 - 0.10
0.10 - 0.25
0.25 - 0.50
0.50 - 0.75
0.75 - 1.00
1.00 - 1.50
1.50 - 2.00
2.00 - 2.50
> 2.50

Coffs Harbour Bypass
Coffs Creek 5 % AEP peak flood level and depth
B1.2.3

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N



Legend

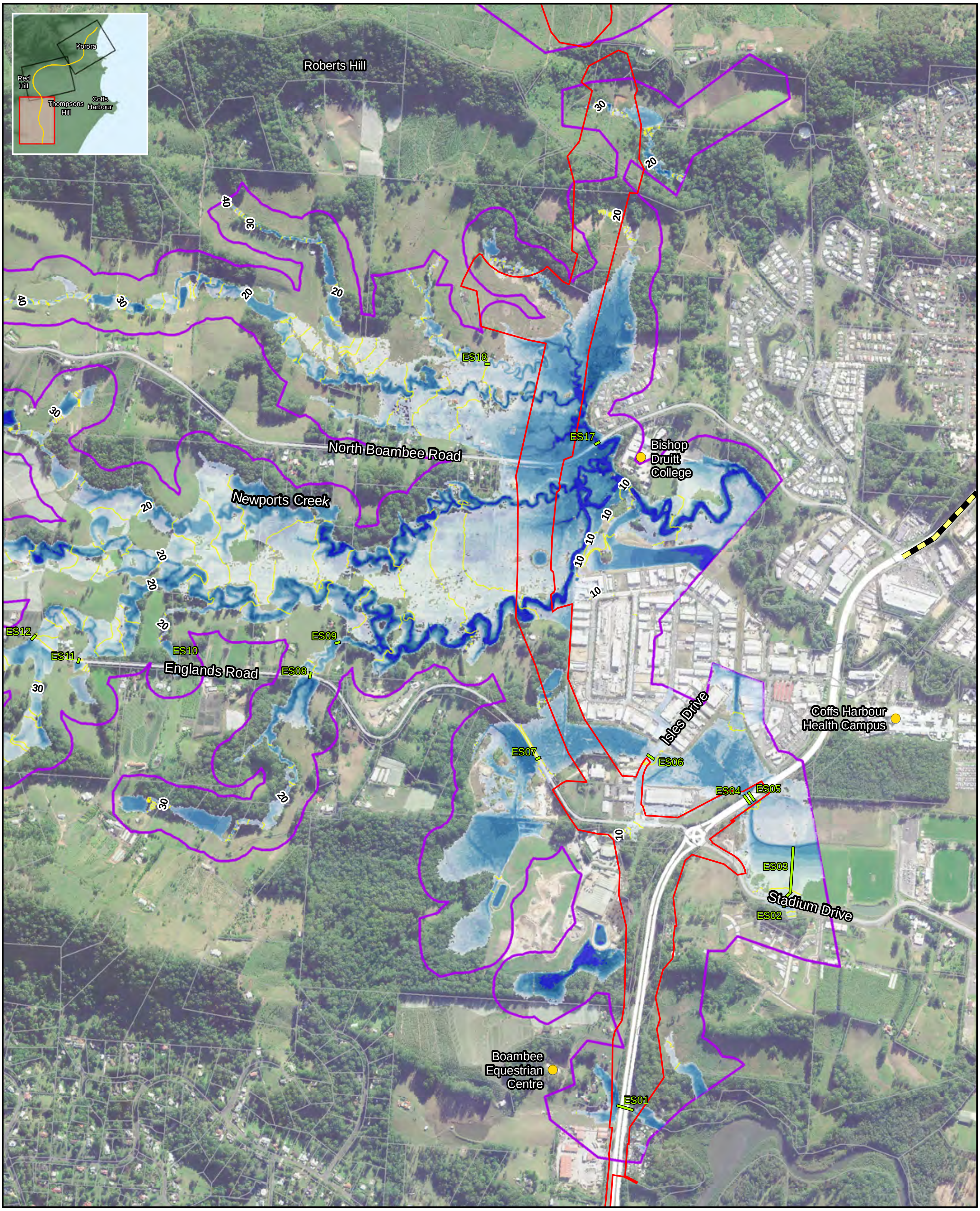
North Coast Railway
Evacuation routes
Cadastre
Construction footprint
Flood model extents
Peak flood level (mAHD at 1m contours)
Modelled structures
Sensitive receiver
Assembly areas

Peak flood depth (m)

0.00 - 0.10
0.10 - 0.25
0.25 - 0.50
0.50 - 0.75
0.75 - 1.00
1.00 - 1.50
1.50 - 2.00
2.00 - 2.50
> 2.50

Coffs Harbour Bypass
Northern Creek 5 % AEP peak flood level and depth
B1.3.3

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

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- Modelled structures
- Sensitive receiver
- Assembly areas

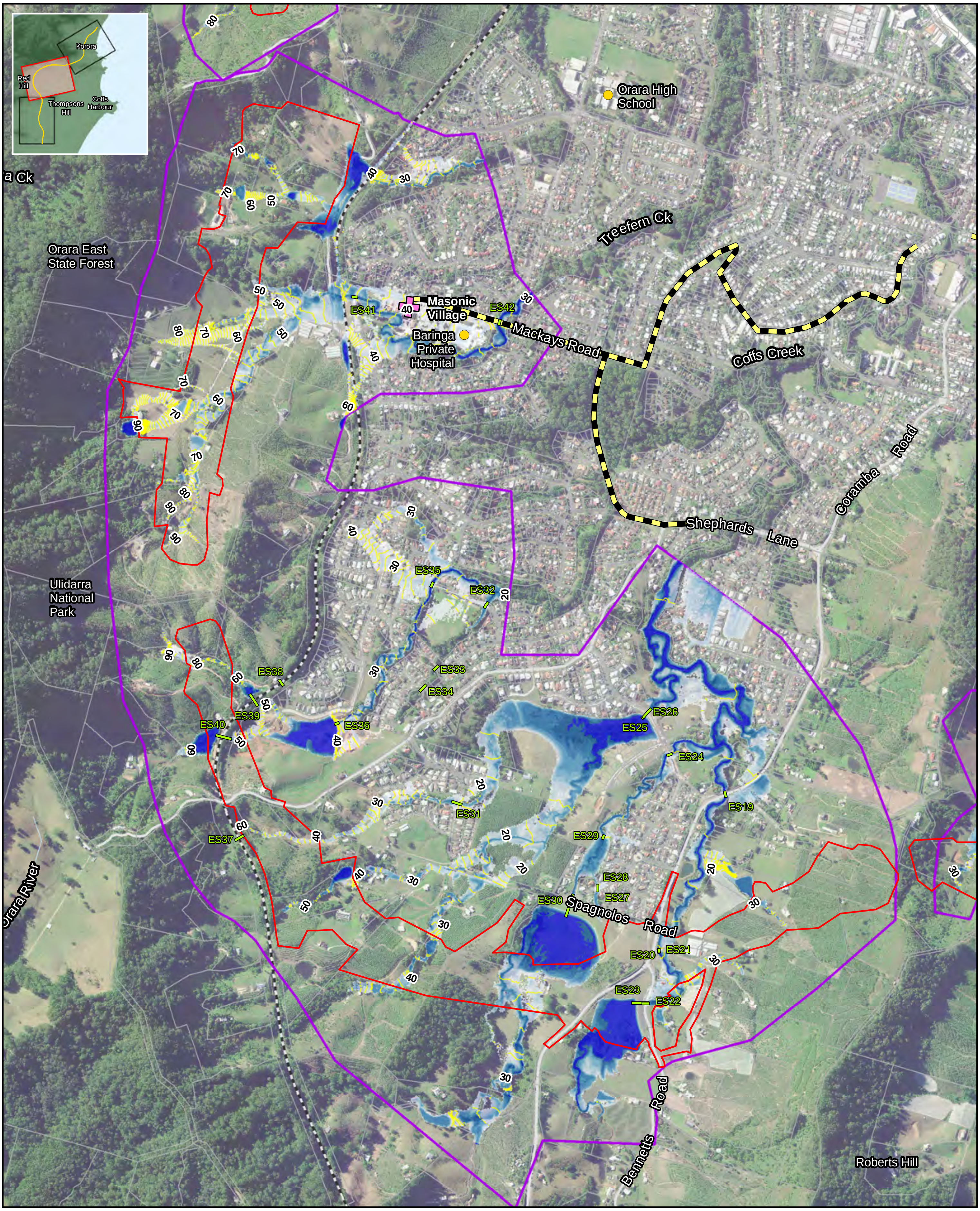
Peak flood depth (m)

0.00 - 0.10, 0.10 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.50, 1.50 - 2.00, 2.00 - 2.50, > 2.50

Coffs Harbour Bypass
North Boambee Valley 2 % AEP peak flood level and depth
B1.1.4

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N



Legend

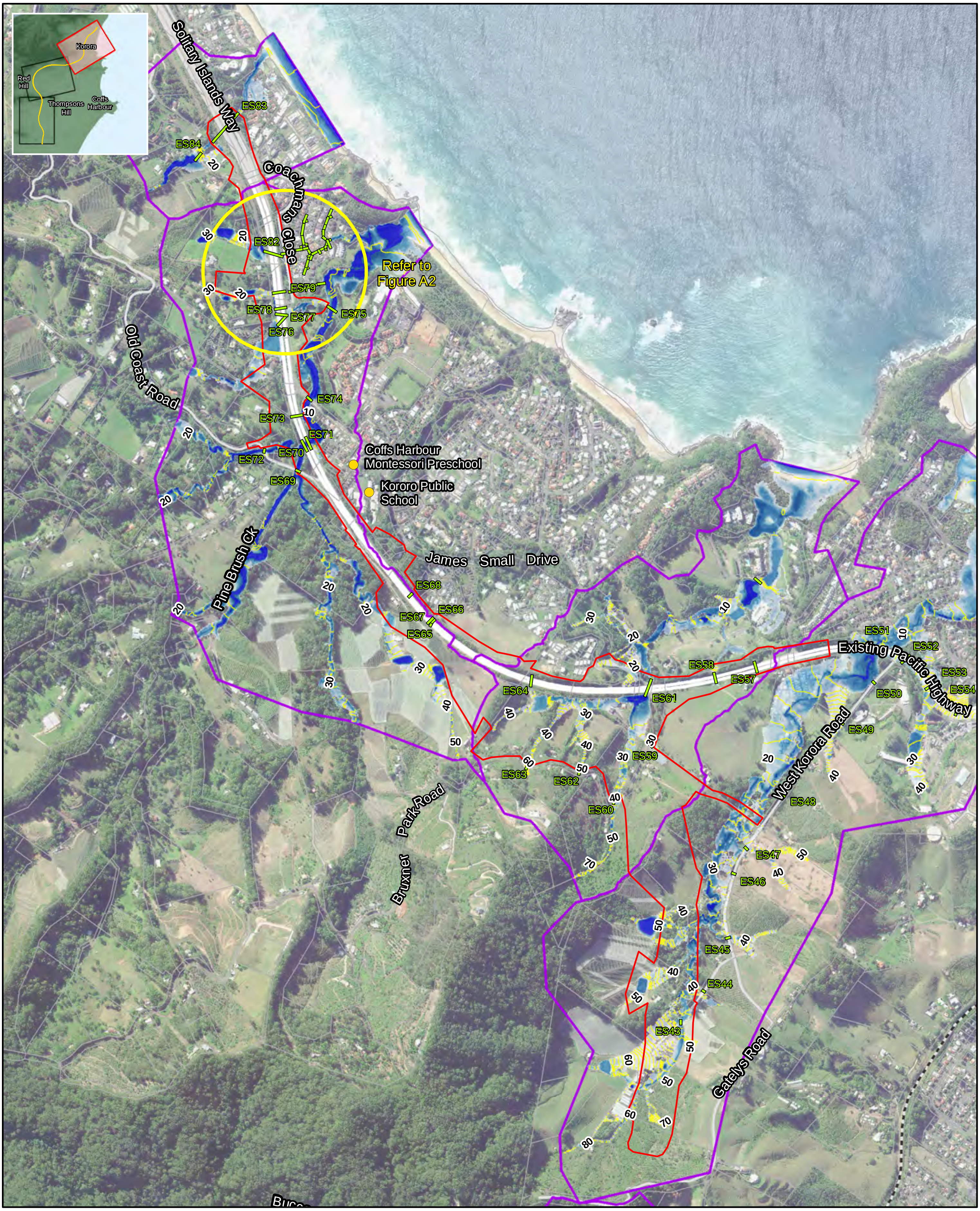
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Peak flood level (mAHD at 1m contours)
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood depth (m)

0.00 - 0.10, 0.10 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.50, 1.50 - 2.00, 2.00 - 2.50, > 2.50

Coffs Harbour Bypass
Coffs Creek 2 % AEP peak flood level and depth
B1.2.4

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

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- Evacuation routes
- Cadastre
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- Flood model extents
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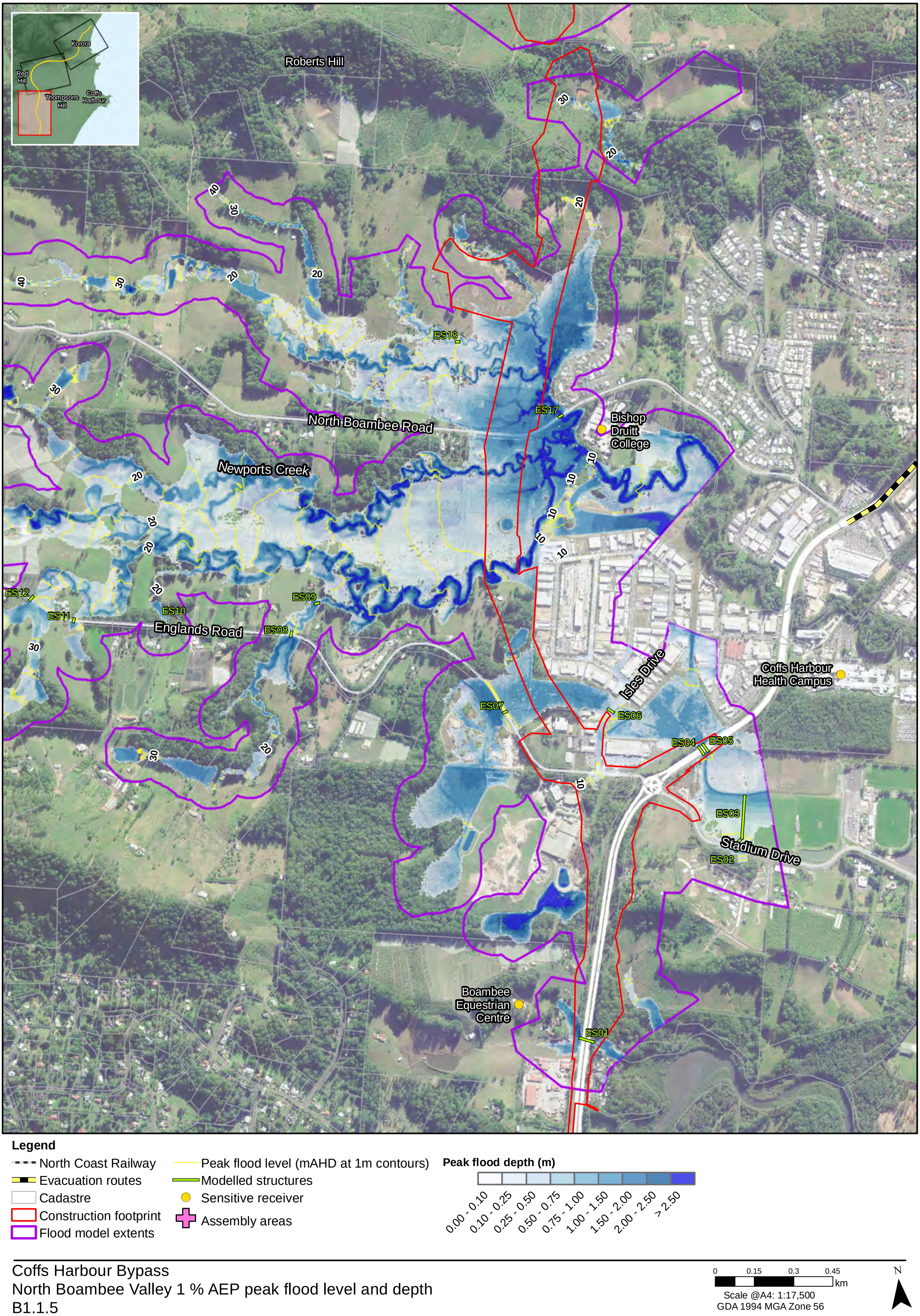
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood depth (m)

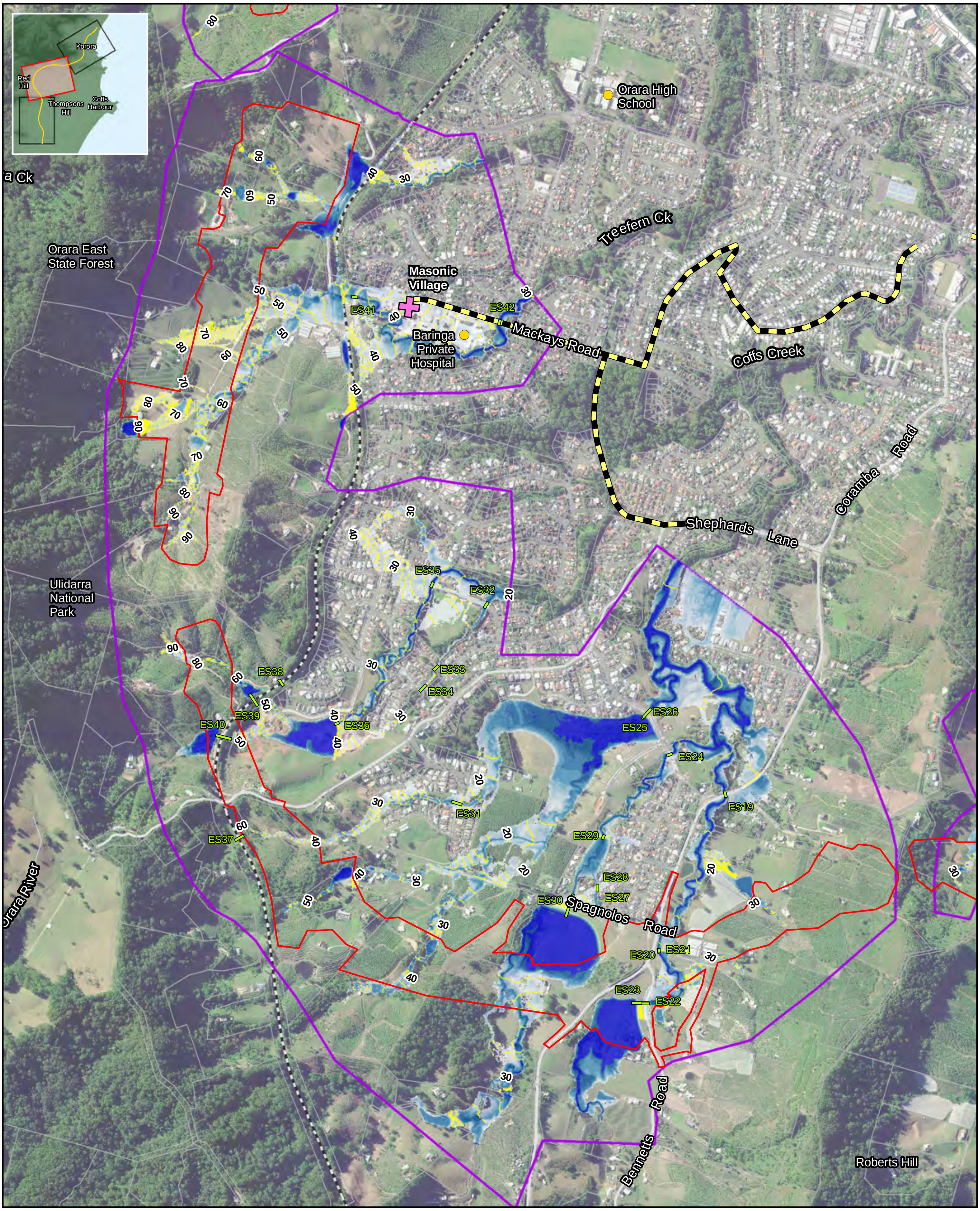
0.00 - 0.10, 0.10 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.50, 1.50 - 2.00, 2.00 - 2.50, > 2.50

Coffs Harbour Bypass
Northern Creek 2 % AEP peak flood level and depth
B1.3.4

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56



Coffs Harbour Bypass
North Boambee Valley 1 % AEP peak flood level and depth
B1.1.5



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Peak flood level (mAHD at 1m contours)
- Modelled structures
- Sensitive receiver
- Assembly areas

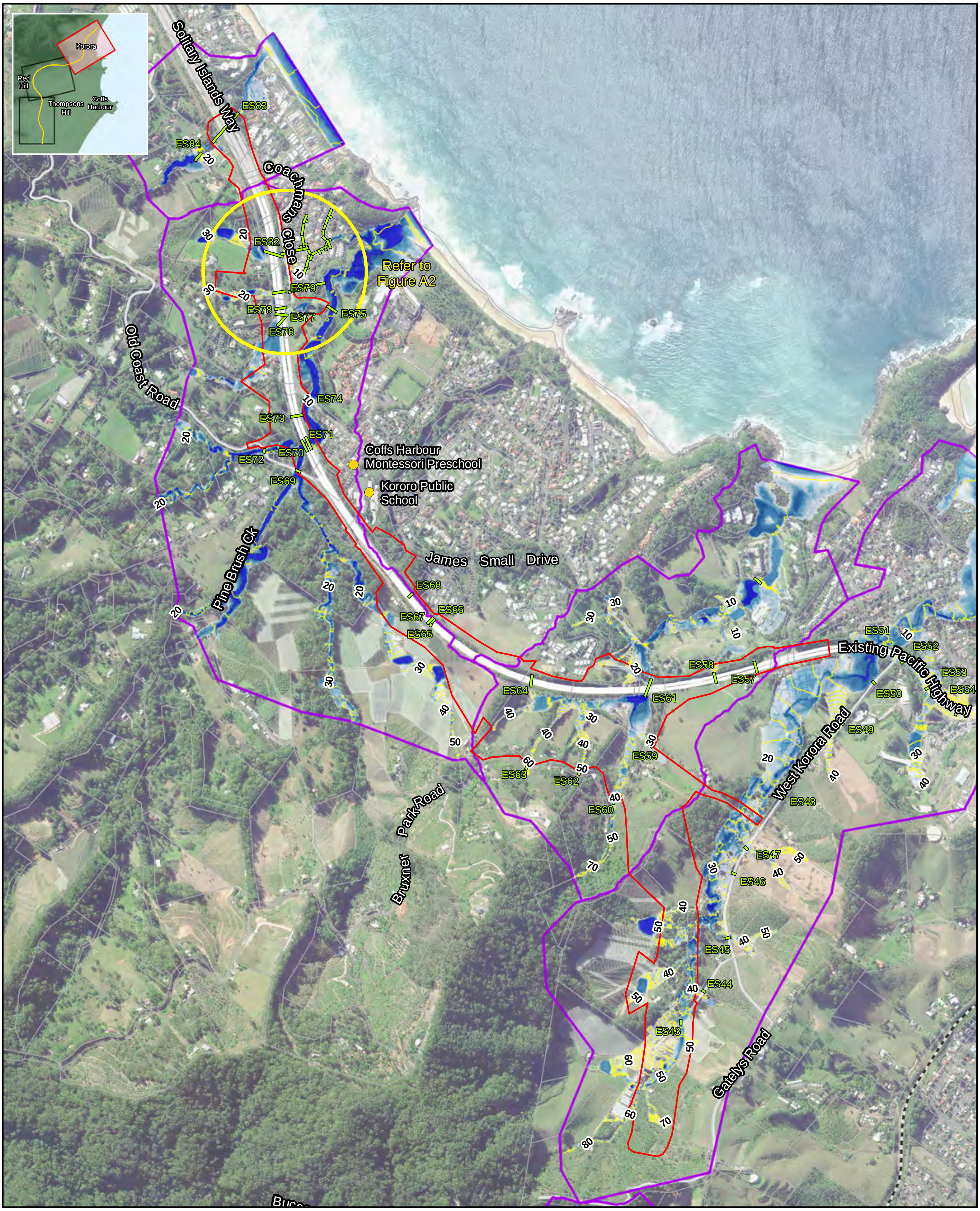
Peak flood depth (m)

0.00 - 0.10, 0.10 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.50, 1.50 - 2.00, 2.00 - 2.50, > 2.50

Coffs Harbour Bypass
Coffs Creek 1 % AEP peak flood level and depth
B1.2.5

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N



Legend

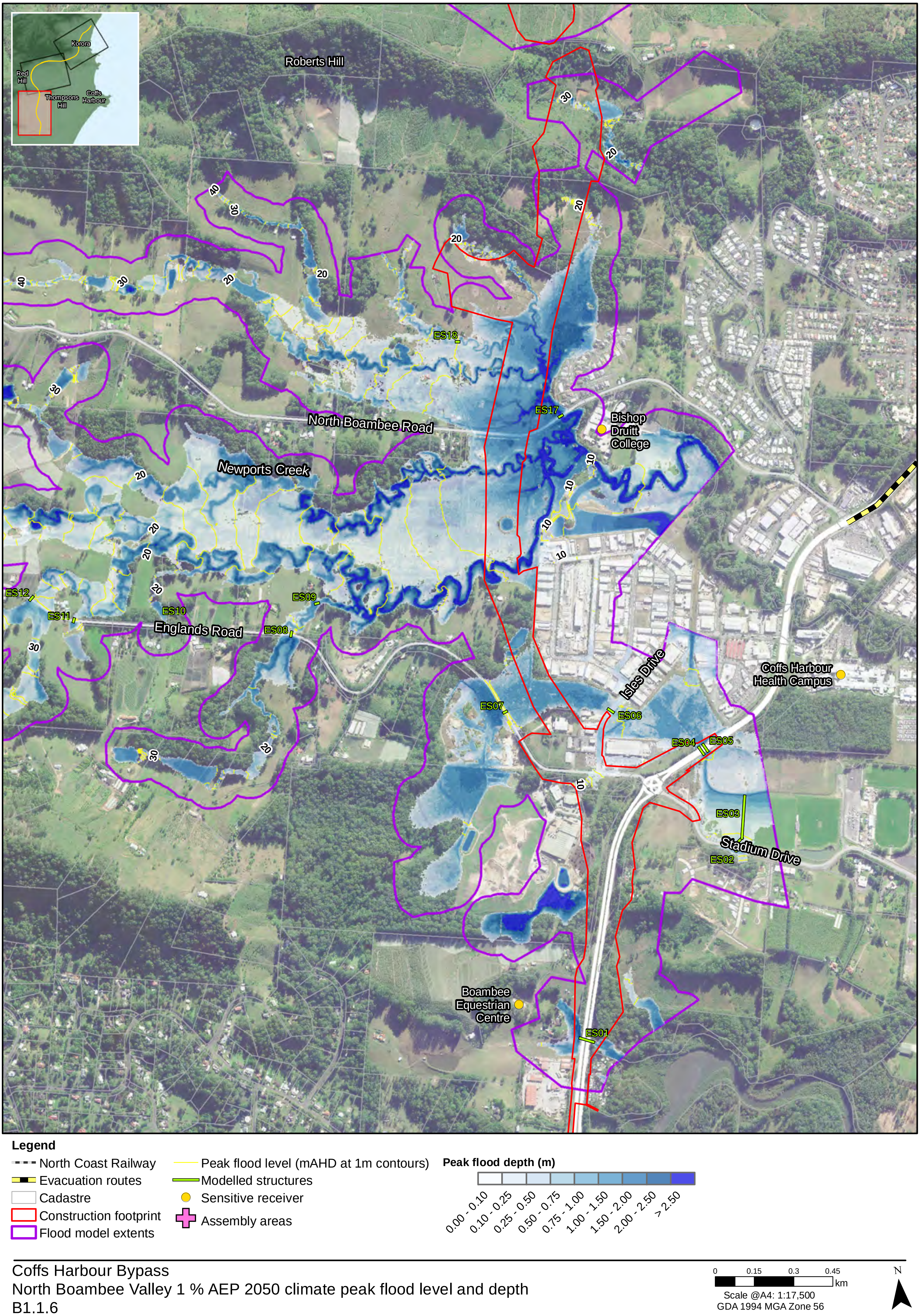
North Coast Railway
Evacuation routes
Cadastre
Construction footprint
Flood model extents
Peak flood level (mAHD at 1m contours)
Modelled structures
Sensitive receiver
Assembly areas

Peak flood depth (m)

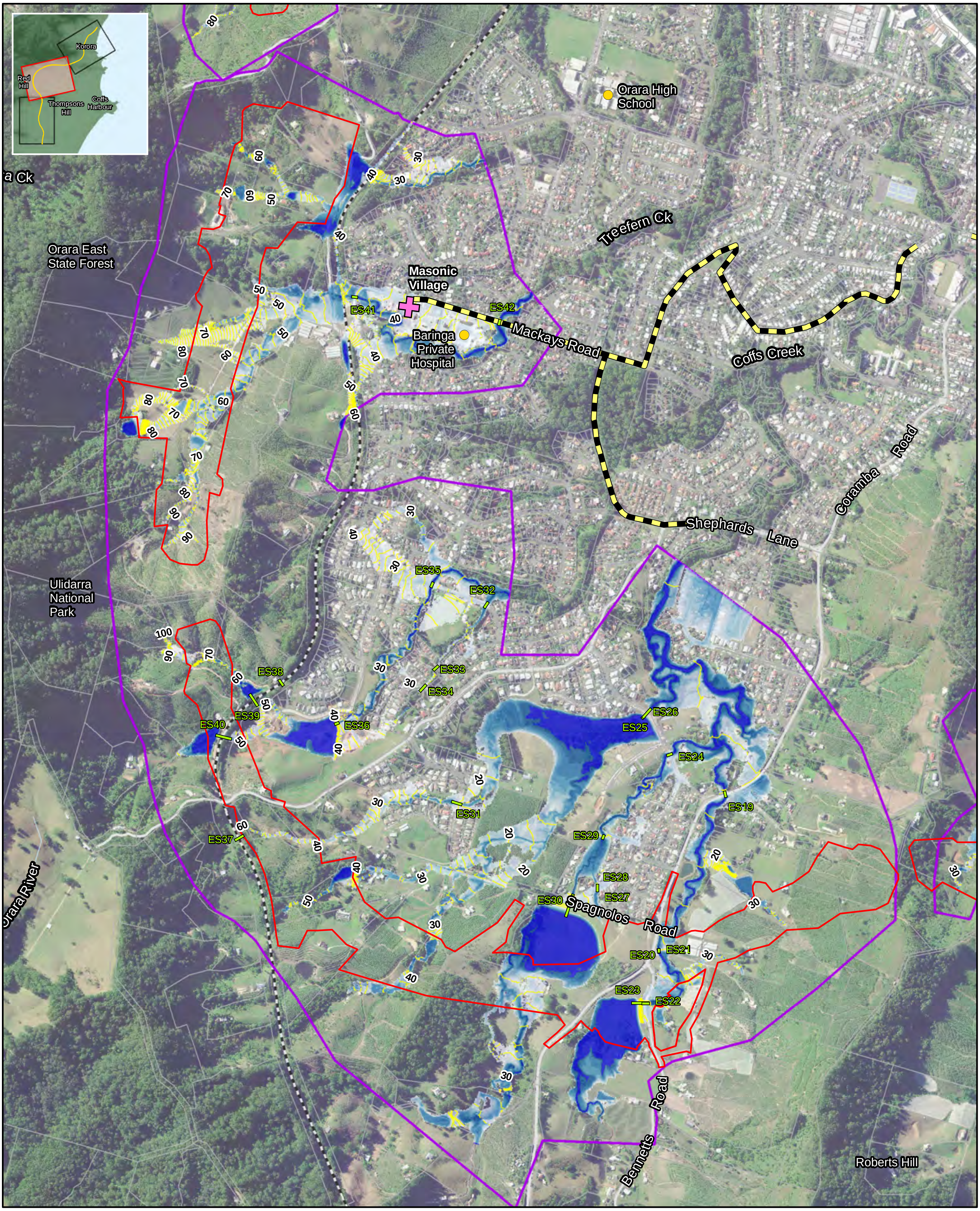
0.00 - 0.10
0.10 - 0.25
0.25 - 0.50
0.50 - 0.75
0.75 - 1.00
1.00 - 1.50
1.50 - 2.00
2.00 - 2.50
> 2.50

Coffs Harbour Bypass
Northern Creek 1 % AEP peak flood level and depth
B1.3.5

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56



Coffs Harbour Bypass
North Boambee Valley 1 % AEP 2050 climate peak flood level and depth
B1.1.6



Legend

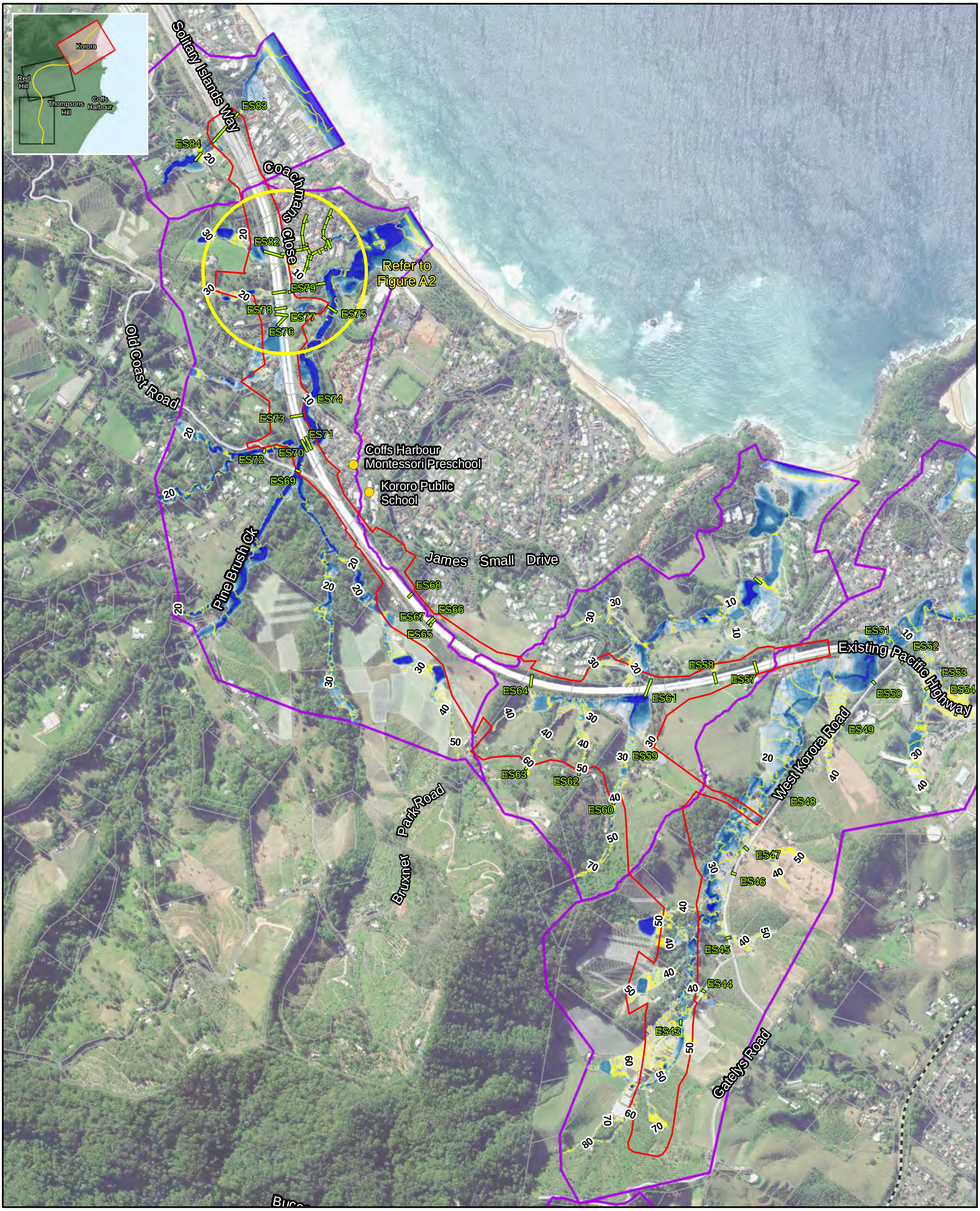
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Peak flood level (mAHD at 1m contours)
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood depth (m)

0.00 - 0.10, 0.10 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.50, 1.50 - 2.00, 2.00 - 2.50, > 2.50

Coffs Harbour Bypass
Coffs Creek 1 % AEP 2050 climate peak flood level and depth
B1.2.6

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

- North Coast Railway
- Evacuation routes
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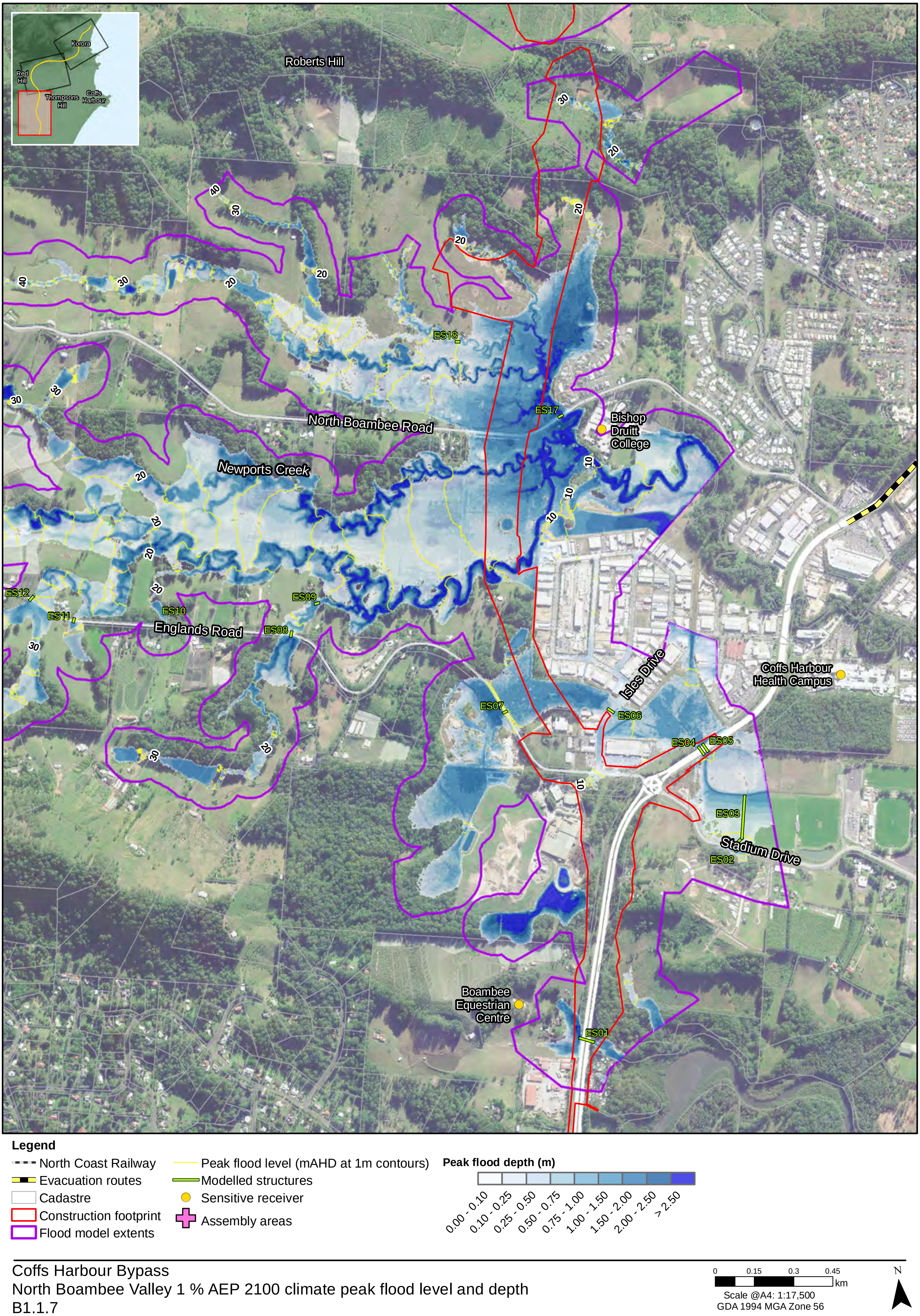
Peak flood depth (m)

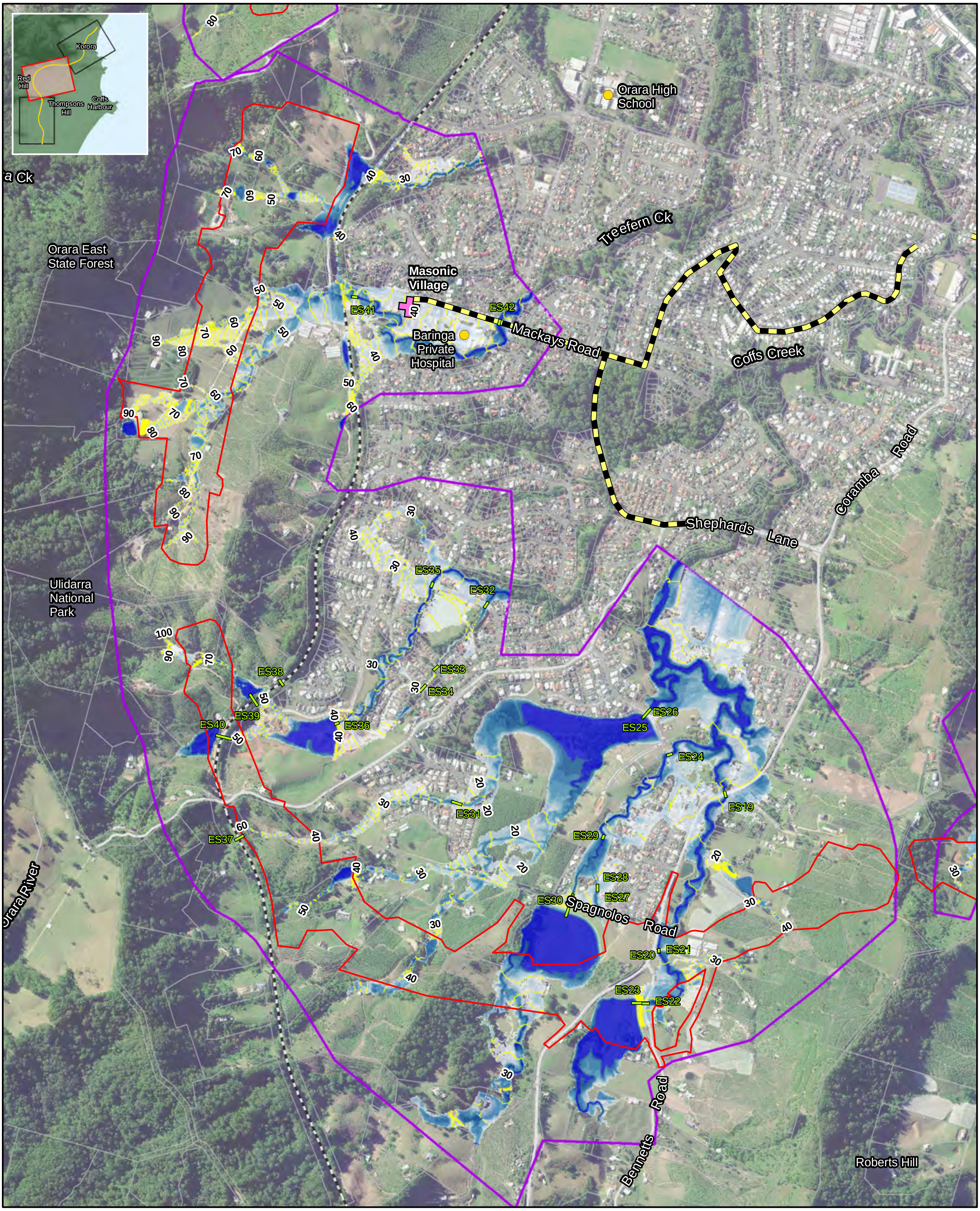
0.00 - 0.10, 0.10 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.50, 1.50 - 2.00, 2.00 - 2.50, > 2.50

Coffs Harbour Bypass
Northern Creek 1 % AEP 2050 climate peak flood level and depth
B1.3.6

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56



Coffs Harbour Bypass
North Boambee Valley 1 % AEP 2100 climate peak flood level and depth
B1.1.7



Legend

North Coast Railway
Evacuation routes
Cadastre
Construction footprint
Flood model extents
Peak flood level (mAHD at 1m contours)
Modelled structures
Sensitive receiver
Assembly areas

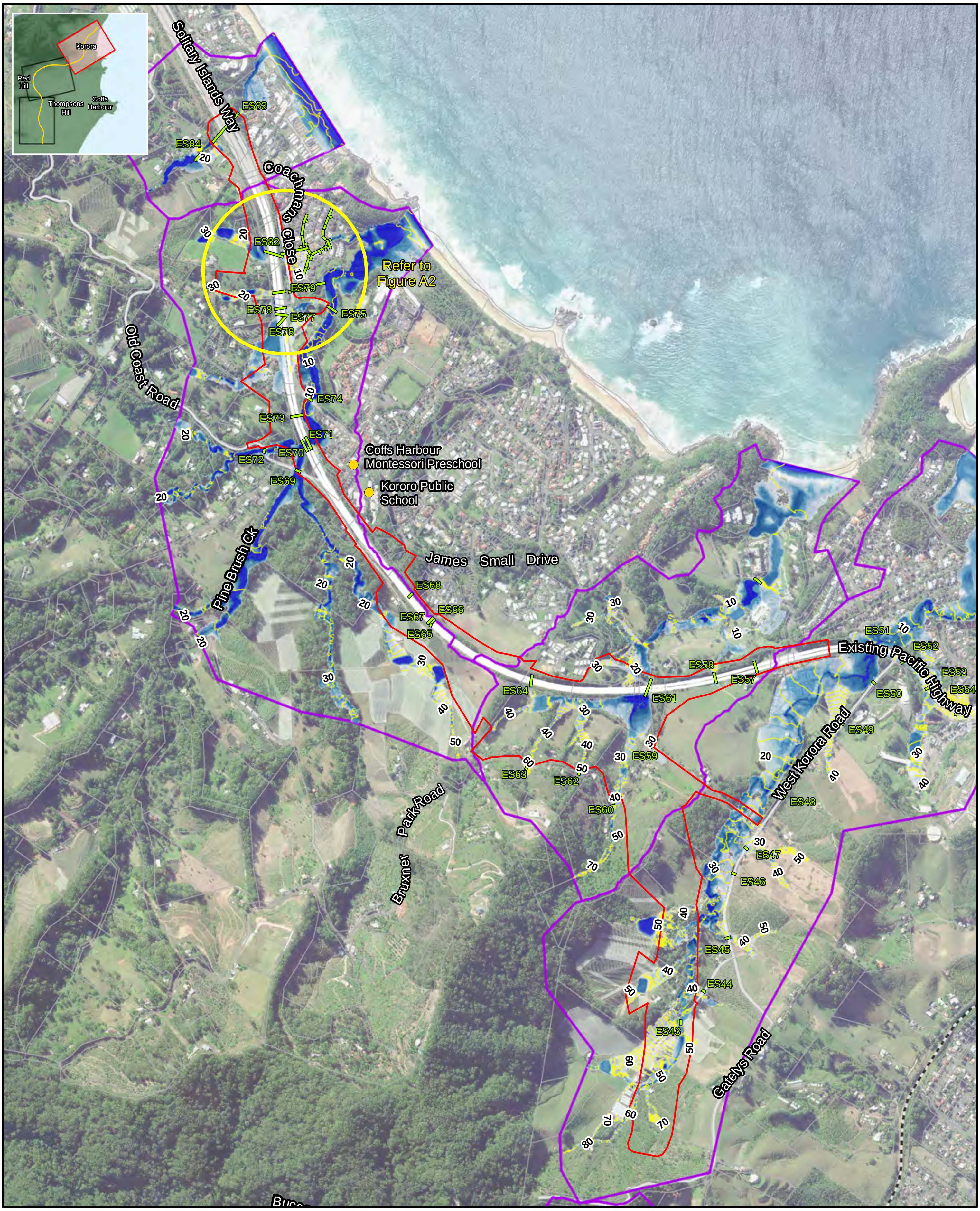
Peak flood depth (m)

0.00 - 0.10
0.10 - 0.25
0.25 - 0.50
0.50 - 0.75
0.75 - 1.00
1.00 - 1.50
1.50 - 2.00
2.00 - 2.50
> 2.50

Coffs Harbour Bypass
Coffs Creek 1 % AEP 2100 climate peak flood level and depth
B1.2.7

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N



Legend

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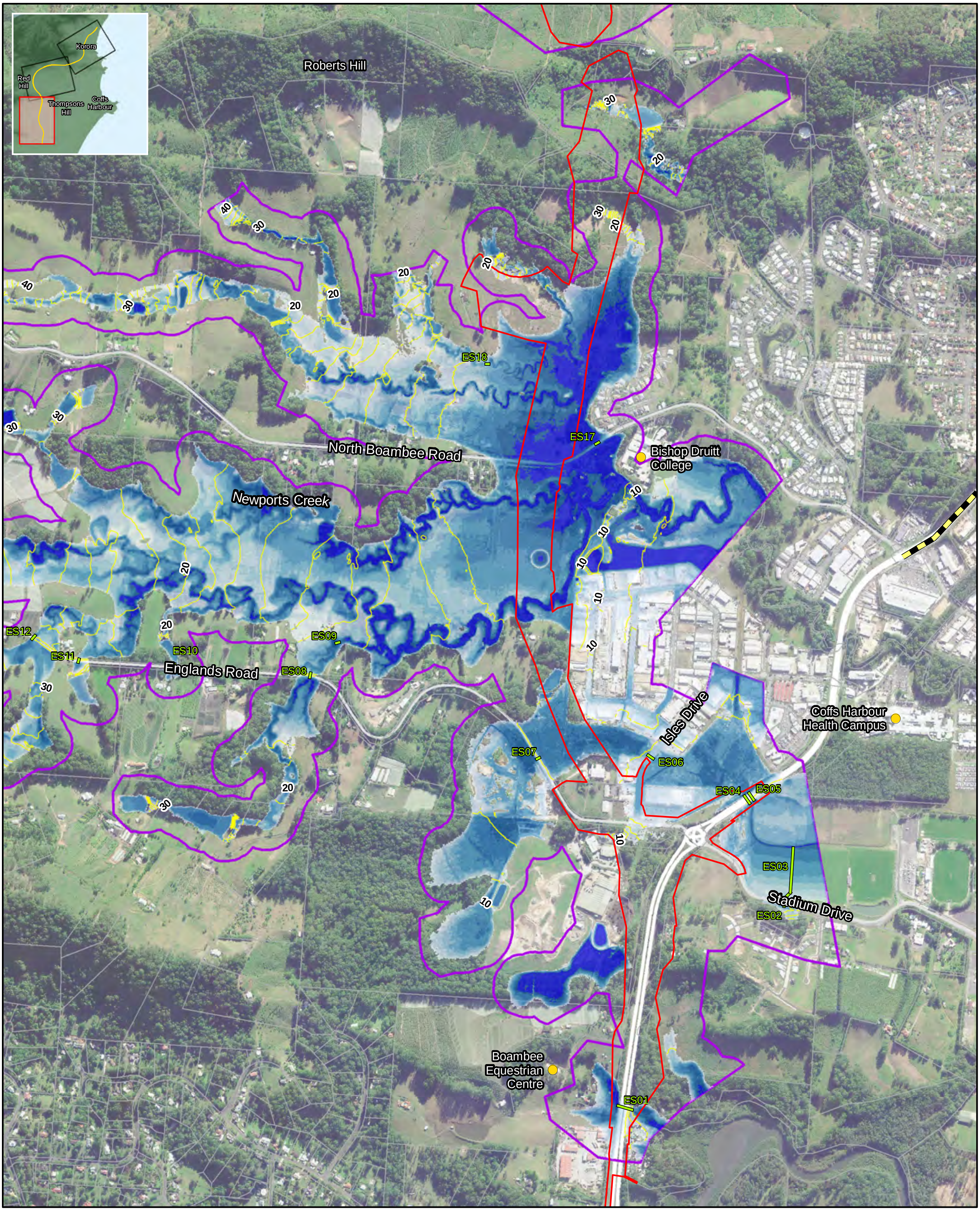
0.00 - 0.10, 0.10 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.50, 1.50 - 2.00, 2.00 - 2.50, > 2.50

Coffs Harbour Bypass
Northern Creek 1 % AEP 2100 climate peak flood level and depth
B1.3.7

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56



Legend

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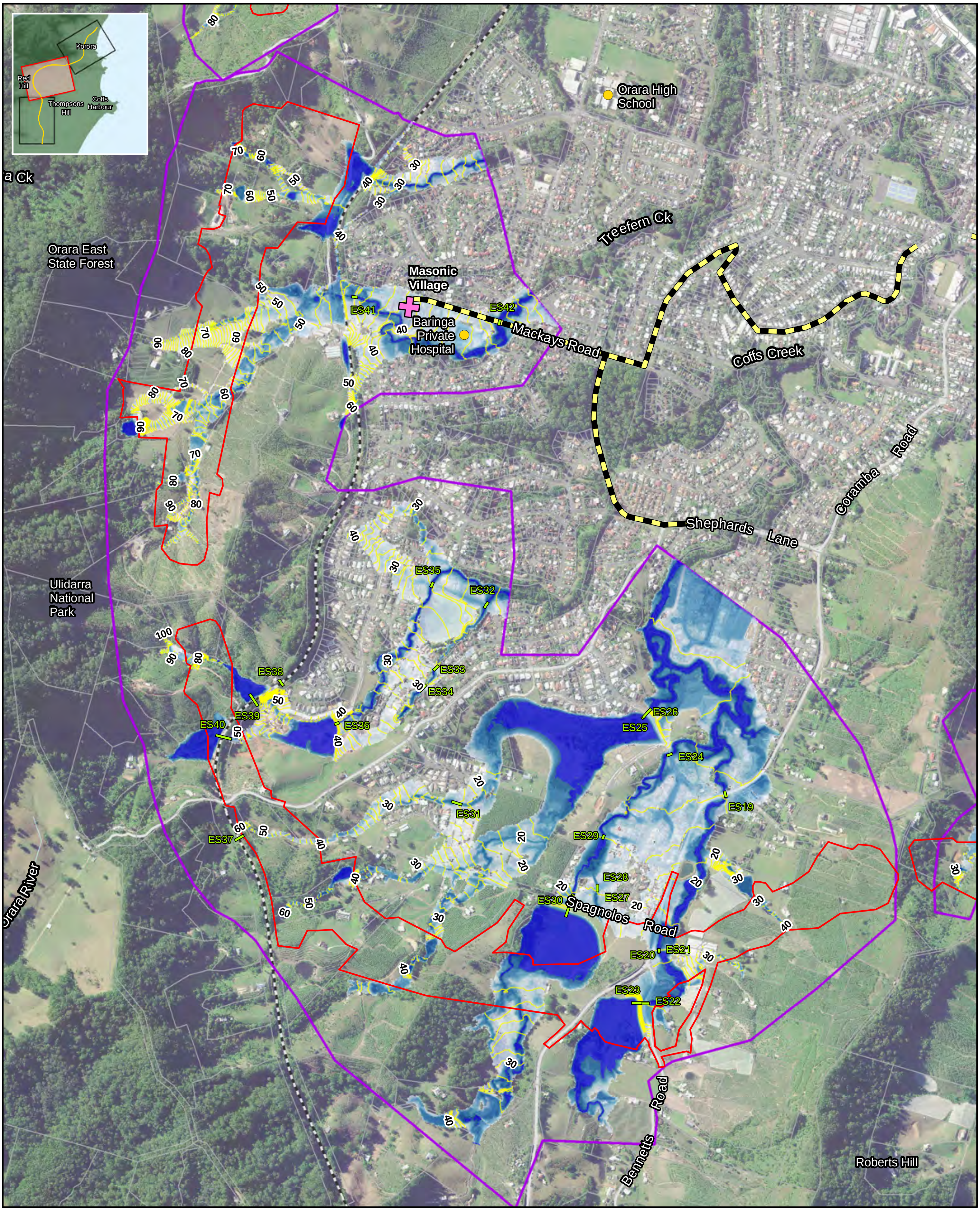
Peak flood depth (m)

0.00 - 0.10, 0.10 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.50, 1.50 - 2.00, 2.00 - 2.50, > 2.50

Coffs Harbour Bypass
North Boambee Valley PMF peak flood level and depth
B1.1.8

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

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- Cadastre
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- Flood model extents
- Peak flood level (mAHD at 1m contours)
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood depth (m)

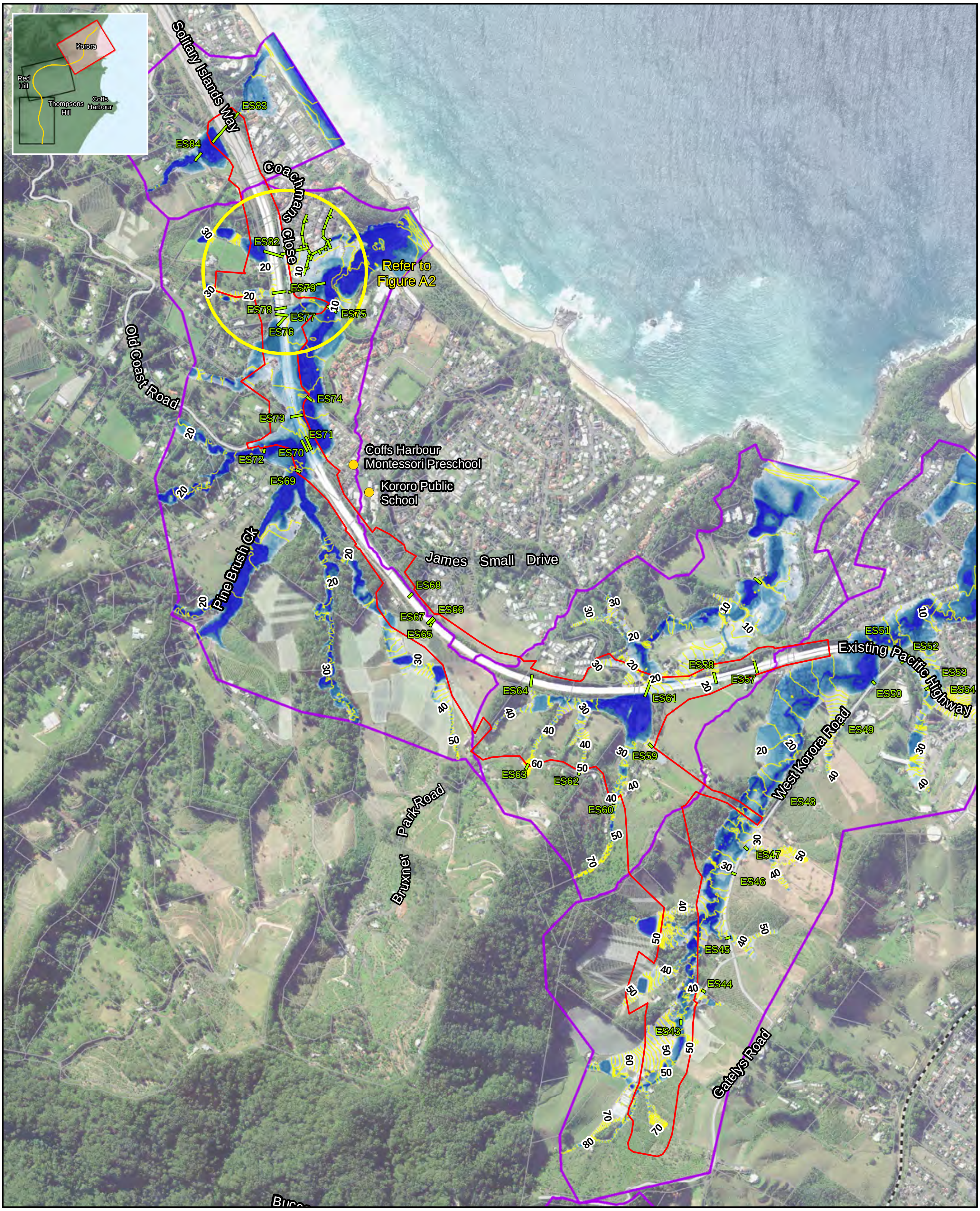
0.00 - 0.10, 0.10 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.50, 1.50 - 2.00, 2.00 - 2.50, > 2.50

Coffs Harbour Bypass
Coffs Creek PMF peak flood level and depth
B1.2.8

0 0.15 0.3 0.45 km

N

Scale @A4: 1:17,500
GDA 1994 MGA Zone 56



Legend

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- Evacuation routes
- Cadastre
- Construction footprint
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- Peak flood level (mAHD at 1m contours)
- Modelled structures
- Sensitive receiver
- Assembly areas

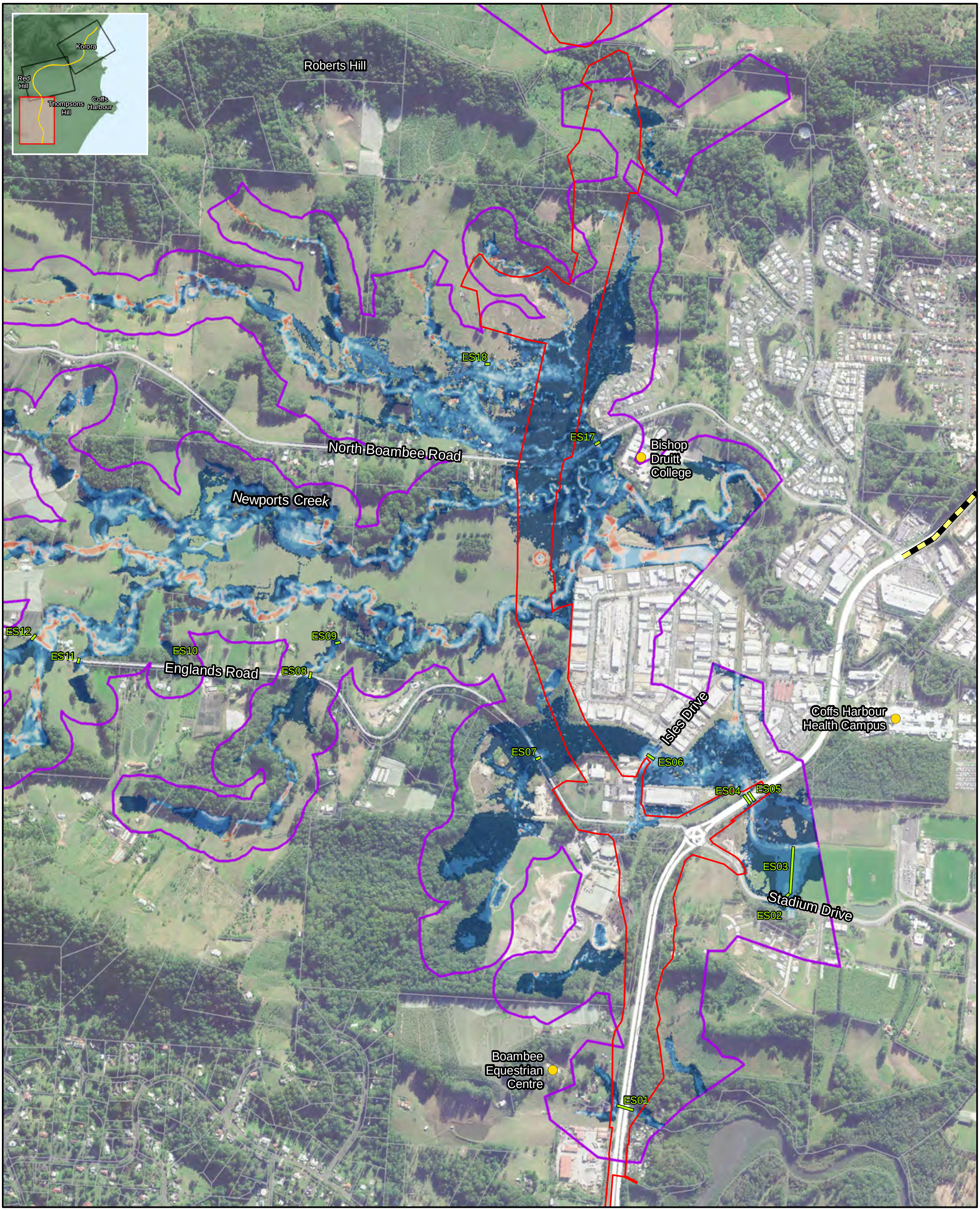
Peak flood depth (m)

0.00 - 0.10, 0.10 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.50, 1.50 - 2.00, 2.00 - 2.50, > 2.50

Coffs Harbour Bypass
Northern Creek PMF peak flood level and depth
B1.3.8

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N

B2 Peak flood velocity



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

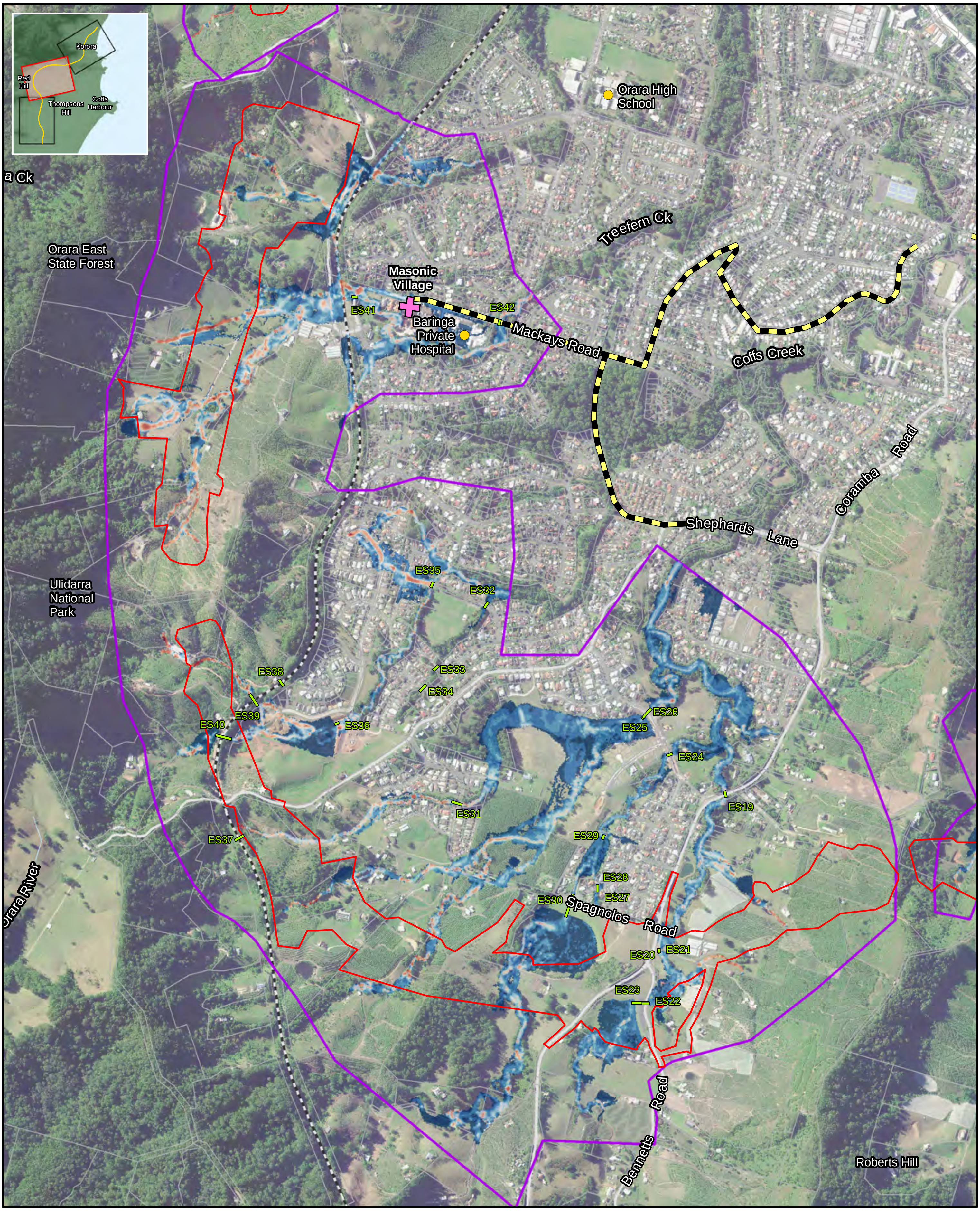
Peak flood velocity (m/s)

0.00 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.25, 1.25 - 1.50, 1.50 - 2.00, 2.00 - 3.00, 3.00 - 4.00, >4.00

Coffs Harbour Bypass
North Boambee Valley 18 % AEP peak flood velocity
B2.1.1

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N



Legend

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- Flood model extents
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- Sensitive receiver
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Peak flood velocity (m/s)

0.00 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.25, 1.25 - 1.50, 1.50 - 2.00, 2.00 - 3.00, 3.00 - 4.00, >4.00

Coffs Harbour Bypass
Coffs Creek 18 % AEP peak flood velocity
B2.2.1

0 0.15 0.3 0.45
km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N

Legend

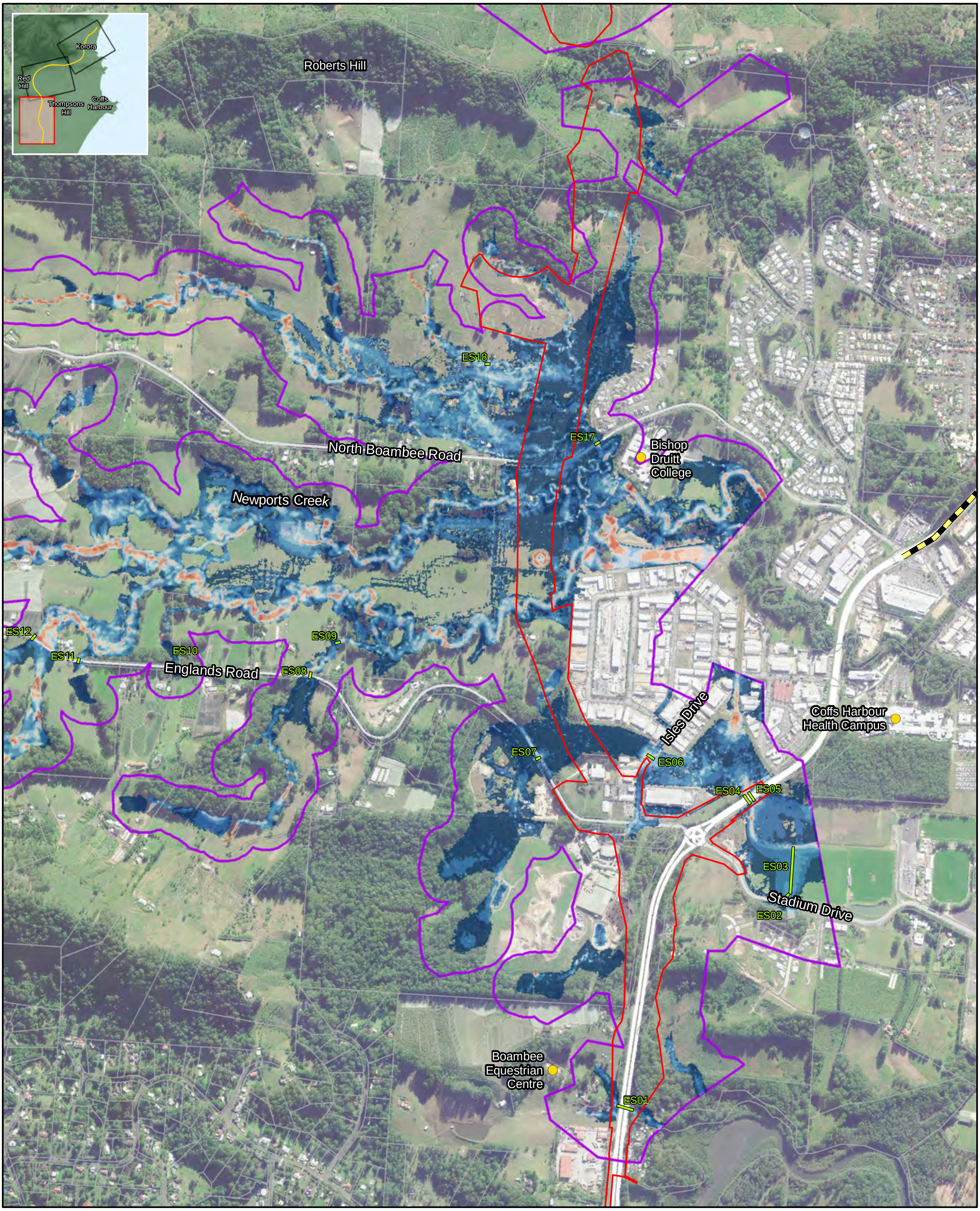
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood velocity (m/s)

0.00 - 0.25
0.25 - 0.50
0.50 - 0.75
0.75 - 1.00
1.00 - 1.25
1.25 - 1.50
1.50 - 2.00
2.00 - 3.00
3.00 - 4.00
>4.00

Coffs Harbour Bypass
Northern Creek 18 % AEP peak flood velocity
B2.3.1

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

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- Cadastre
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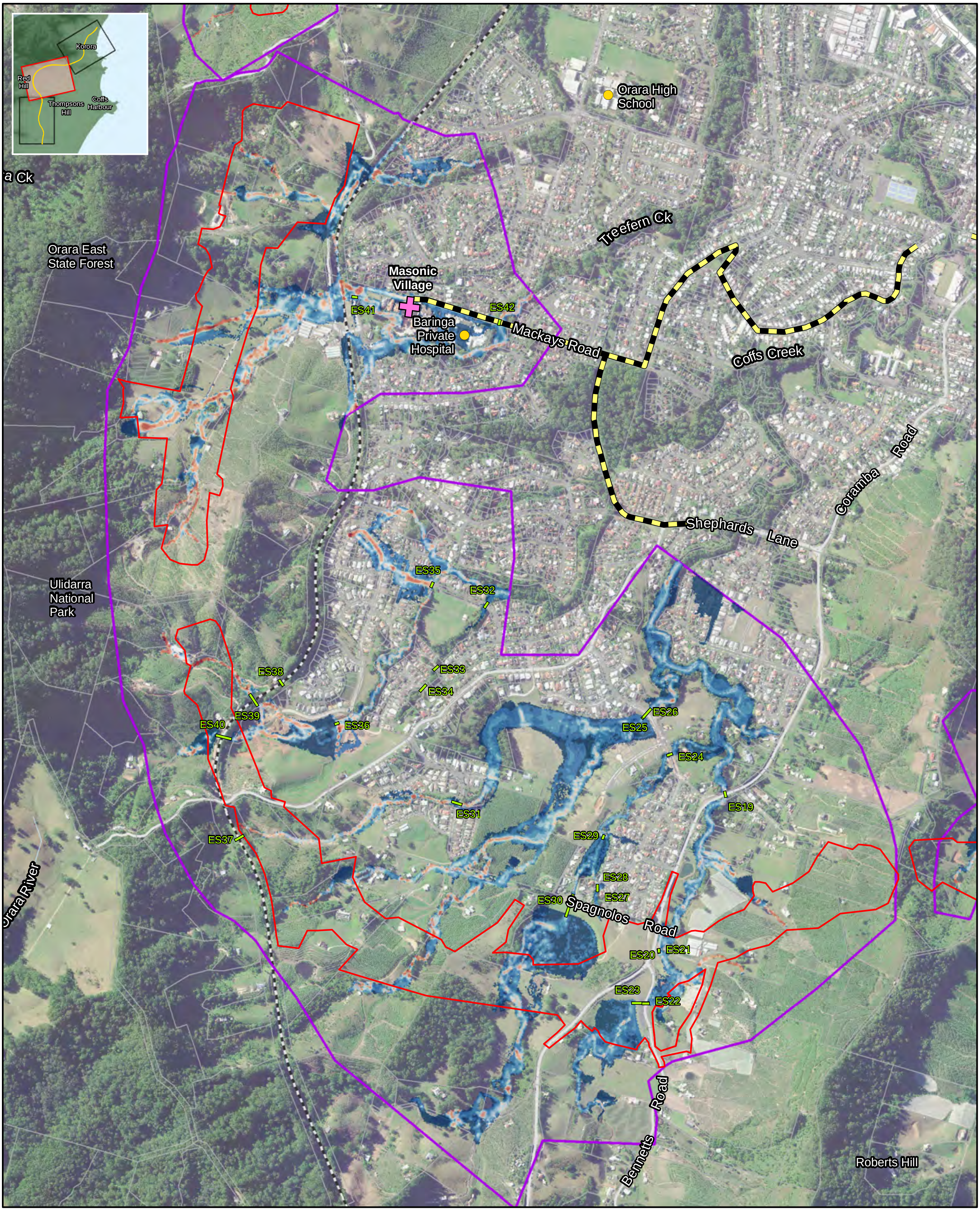
Peak flood velocity (m/s)

0.00 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.25, 1.25 - 1.50, 1.50 - 2.00, 2.00 - 3.00, 3.00 - 4.00, >4.00

Coffs Harbour Bypass
North Boambee Valley 10 % AEP peak flood velocity
B2.1.2

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

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- Evacuation routes
- Cadastre
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Peak flood velocity (m/s)

0.00 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.25, 1.25 - 1.50, 1.50 - 2.00, 2.00 - 3.00, 3.00 - 4.00, >4.00

Coffs Harbour Bypass
Coffs Creek 10 % AEP peak flood velocity
B2.2.2

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N

Legend

North Coast Railway
Evacuation routes
Cadastre
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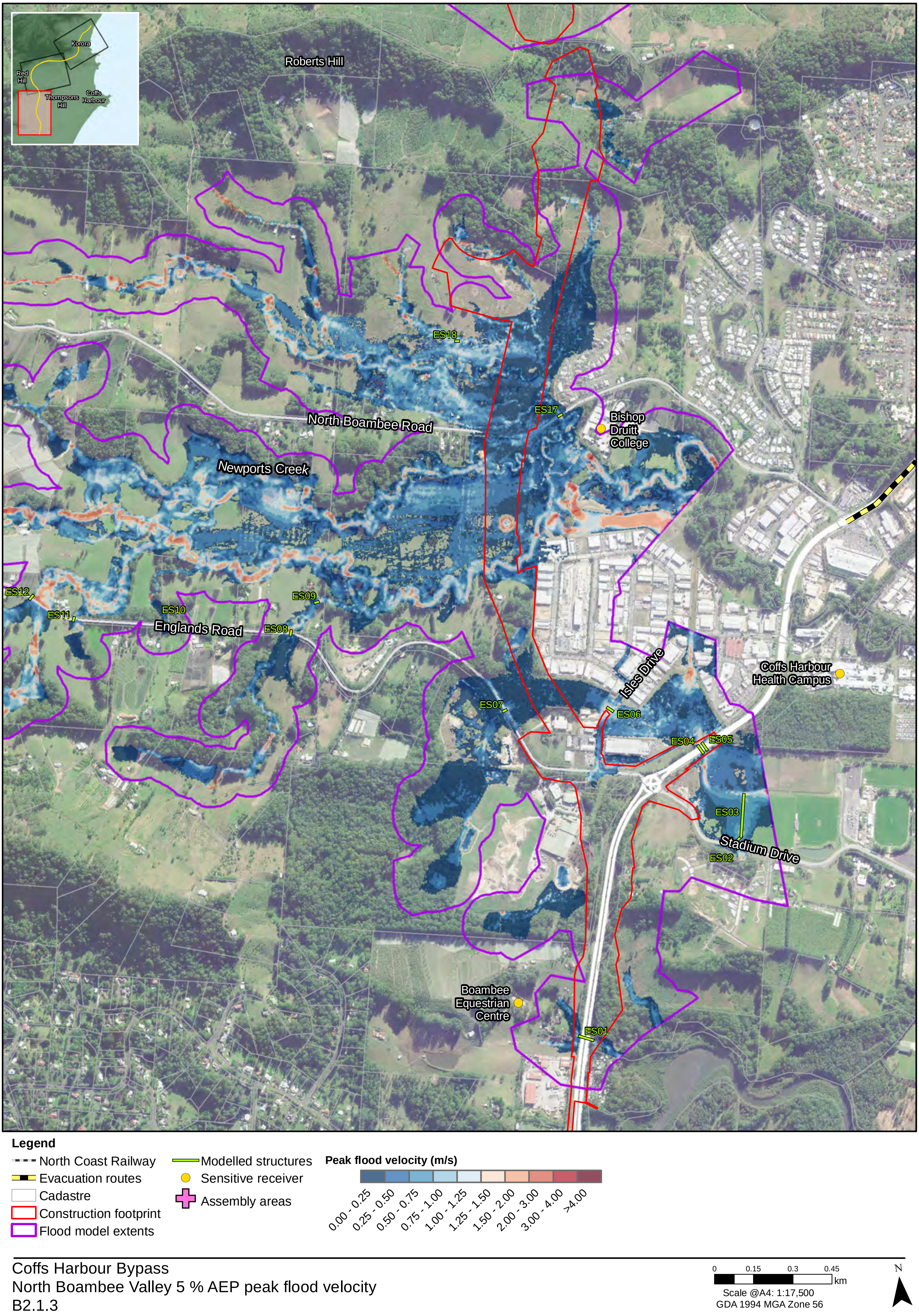
Peak flood velocity (m/s)

0.00 - 0.25
0.25 - 0.50
0.50 - 0.75
0.75 - 1.00
1.00 - 1.25
1.25 - 1.50
1.50 - 2.00
2.00 - 3.00
3.00 - 4.00
>4.00

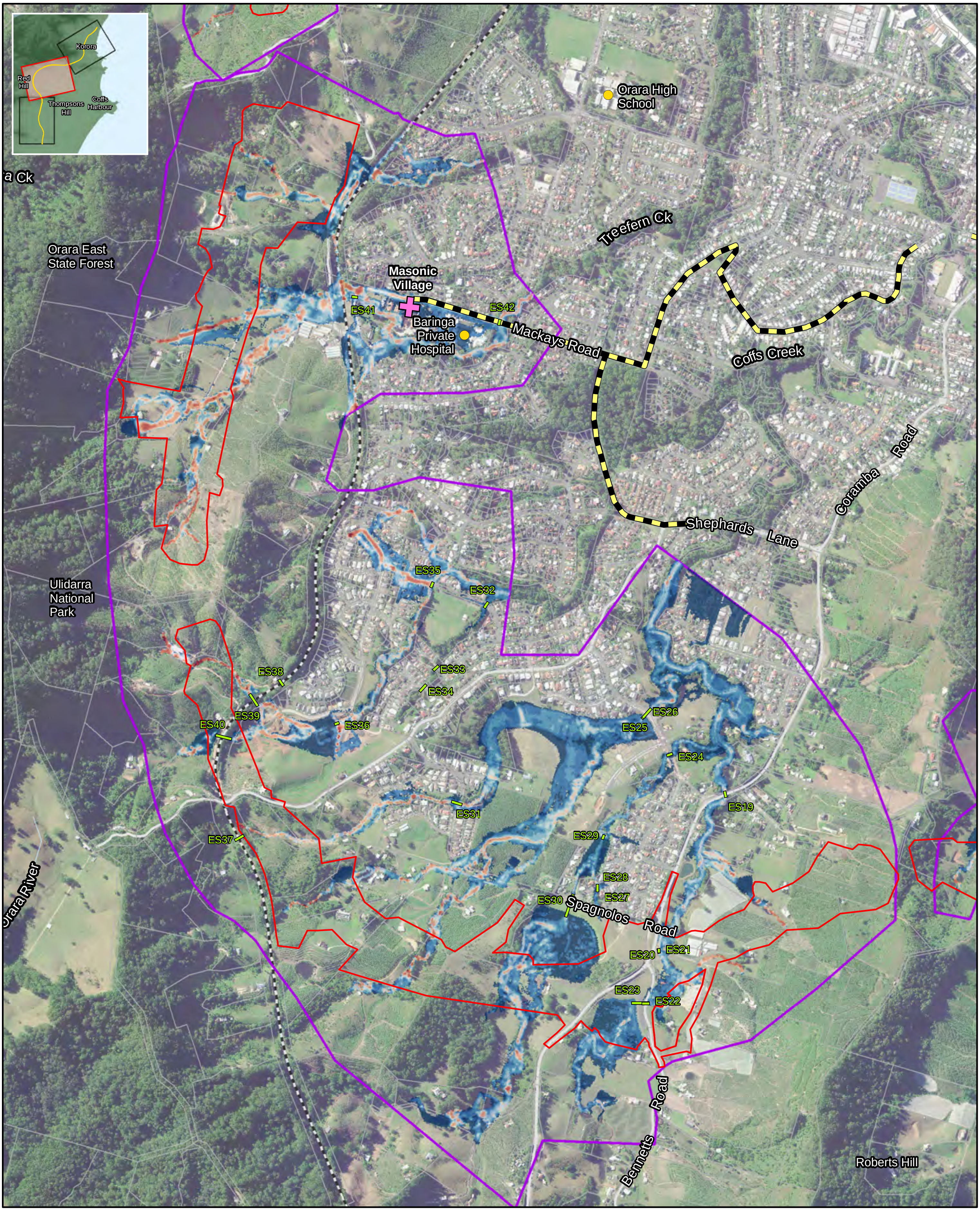
Coffs Harbour Bypass
Northern Creek 10 % AEP peak flood velocity
B2.3.2

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N



Coffs Harbour Bypass
North Boambee Valley 5 % AEP peak flood velocity
B2.1.3



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood velocity (m/s)

0.00 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.25, 1.25 - 1.50, 1.50 - 2.00, 2.00 - 3.00, 3.00 - 4.00, >4.00

Coffs Harbour Bypass
Coffs Creek 5 % AEP peak flood velocity
B2.2.3

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N

Legend

- North Coast Railway
- Evacuation routes
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- Modelled structures
- Sensitive receiver
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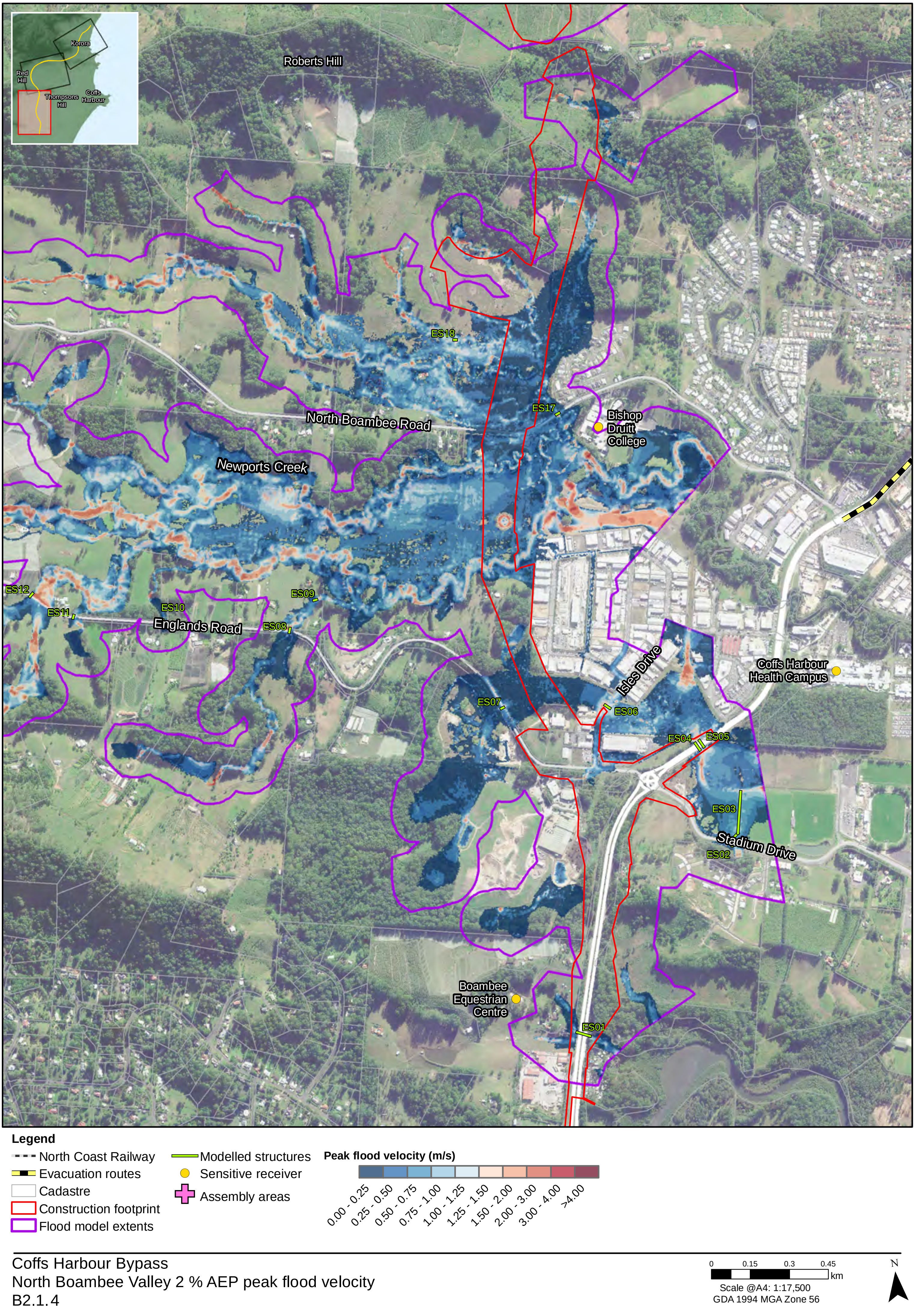
Peak flood velocity (m/s)

0.00 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.25, 1.25 - 1.50, 1.50 - 2.00, 2.00 - 3.00, 3.00 - 4.00, >4.00

Coffs Harbour Bypass
Northern Creek 5 % AEP peak flood velocity
B2.3.3

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

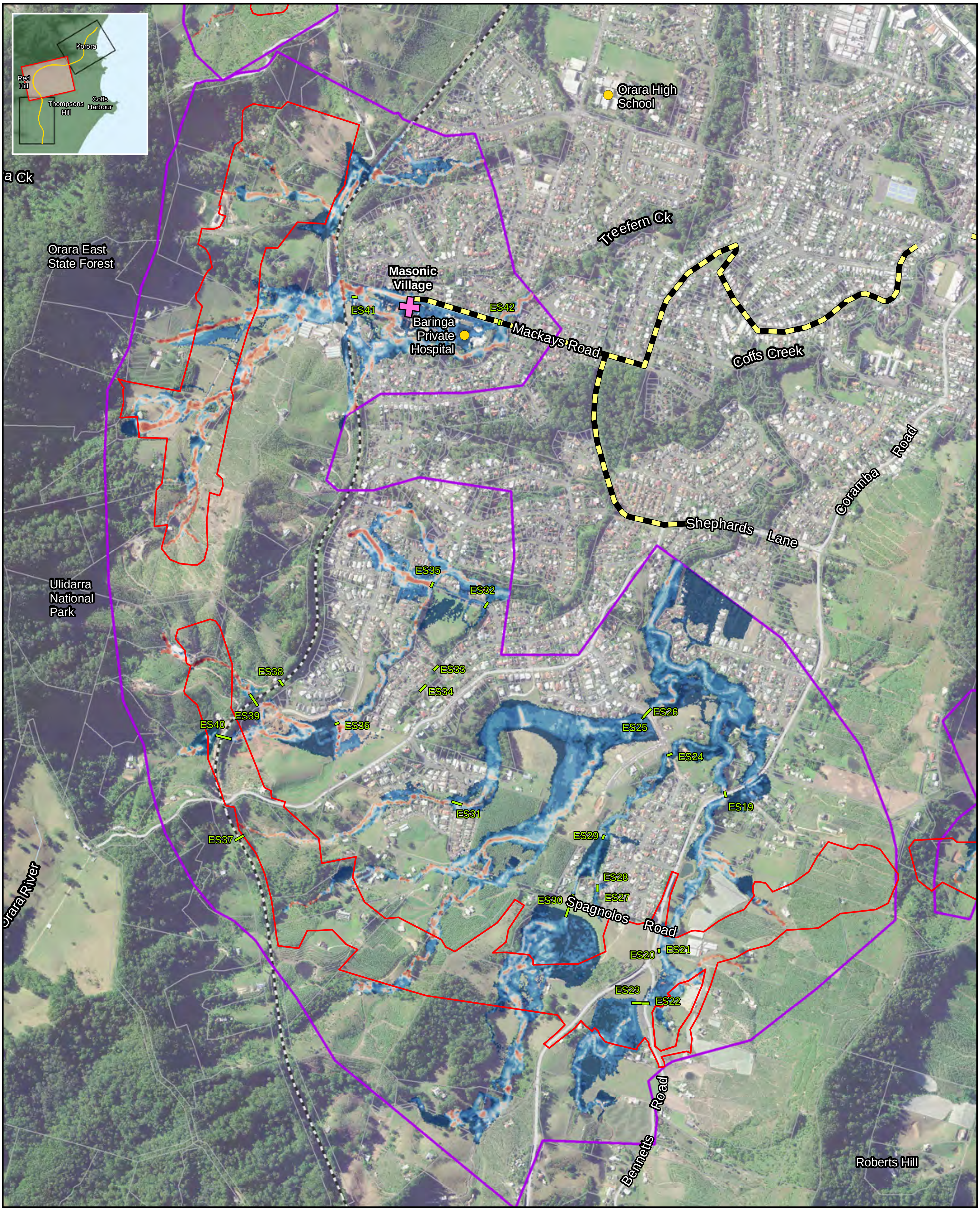
N



Coffs Harbour Bypass

North Boambee Valley 2 % AEP peak flood velocity

B2.1.4



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood velocity (m/s)

0.00 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.25, 1.25 - 1.50, 1.50 - 2.00, 2.00 - 3.00, 3.00 - 4.00, >4.00

Coffs Harbour Bypass
Coffs Creek 2 % AEP peak flood velocity
B2.2.4

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N

Legend

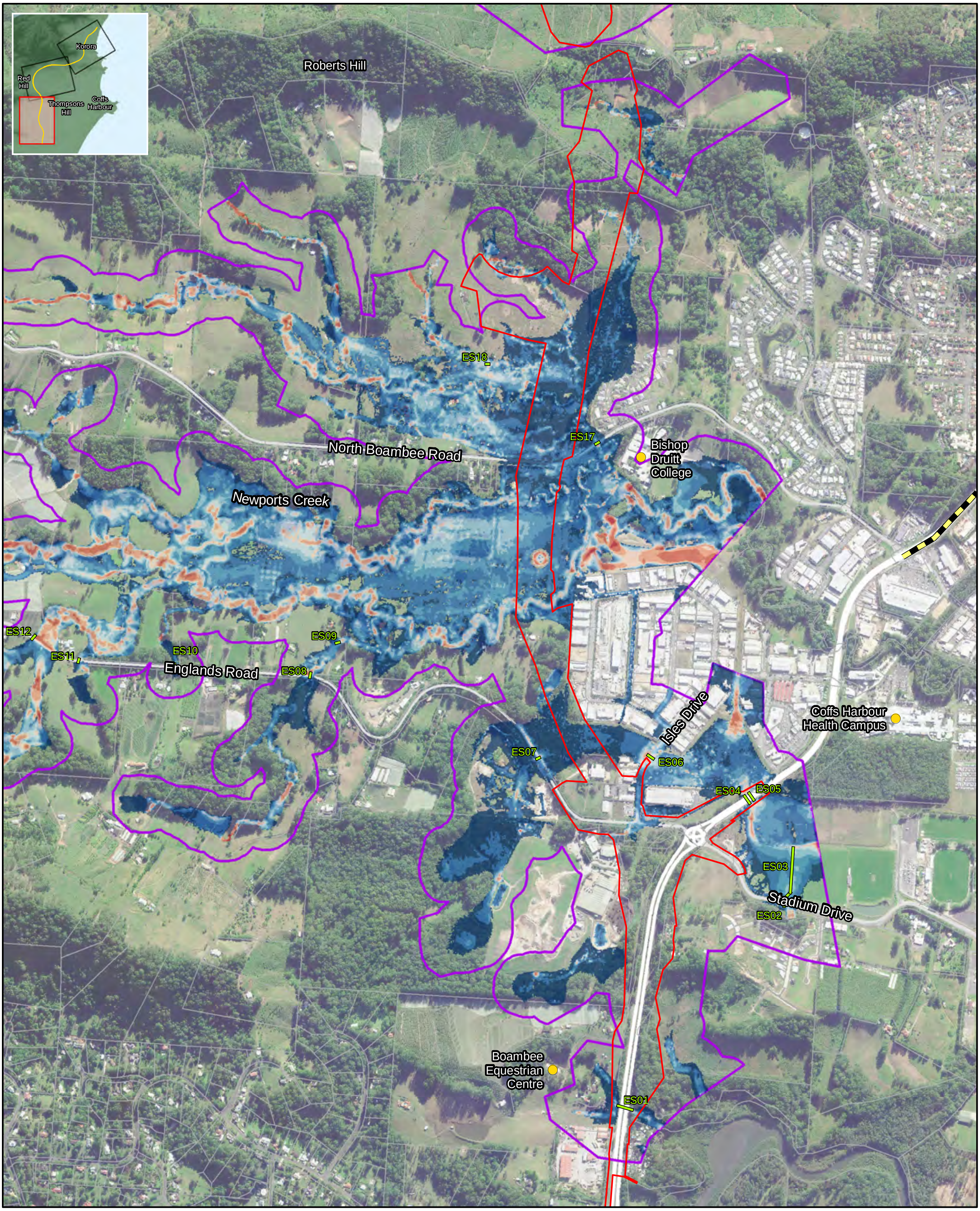
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood velocity (m/s)

0.00 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.25, 1.25 - 1.50, 1.50 - 2.00, 2.00 - 3.00, 3.00 - 4.00, >4.00

Coffs Harbour Bypass
Northern Creek 2 % AEP peak flood velocity
B2.3.4

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56



Legend

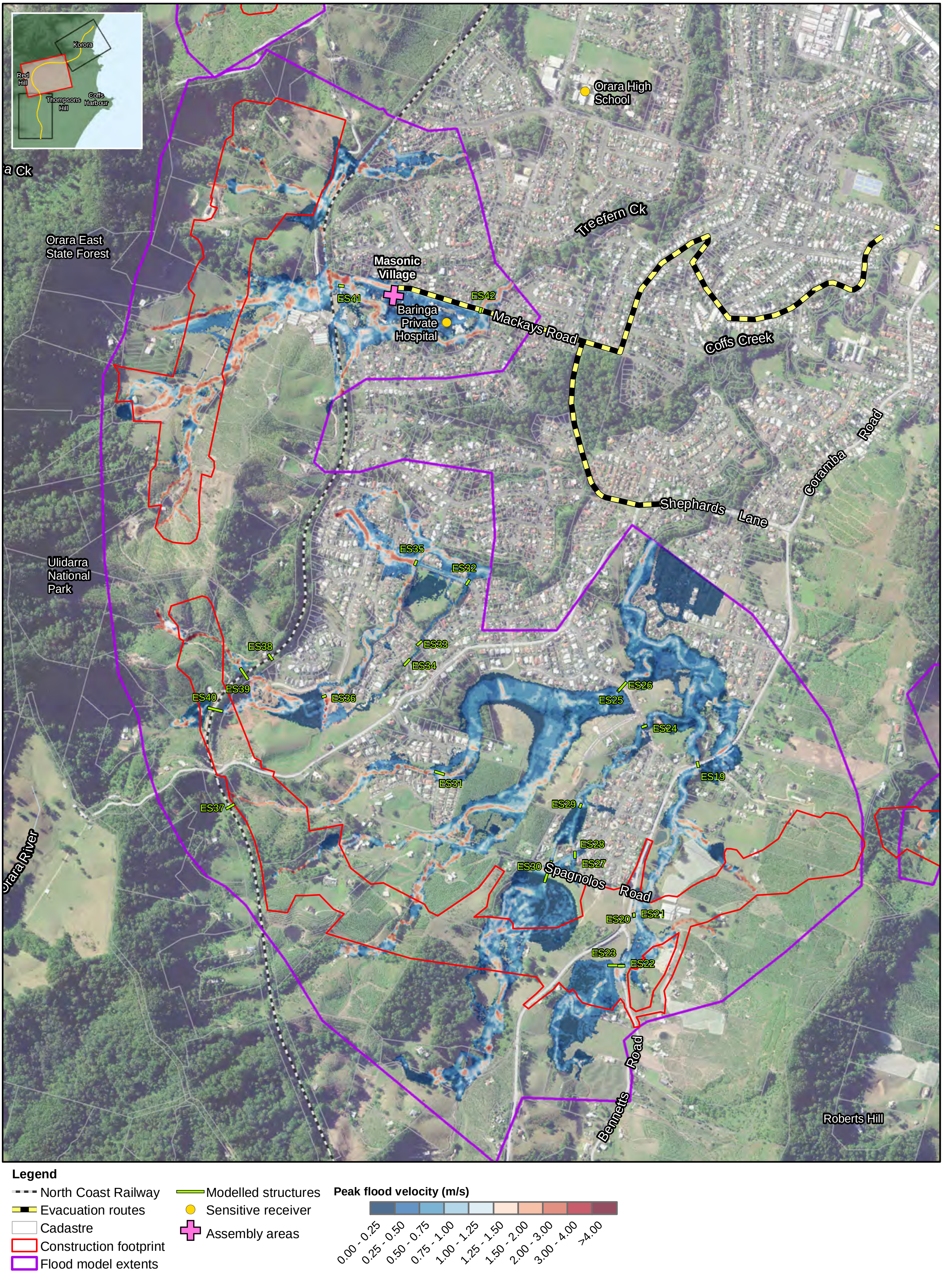
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood velocity (m/s)

0.00 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.25, 1.25 - 1.50, 1.50 - 2.00, 2.00 - 3.00, 3.00 - 4.00, >4.00

Coffs Harbour Bypass
North Boambee Valley 1 % AEP peak flood velocity
B2.1.5

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Coffs Harbour Bypass
Coffs Creek 1 % AEP peak flood velocity
B2.2.5

Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

Legend

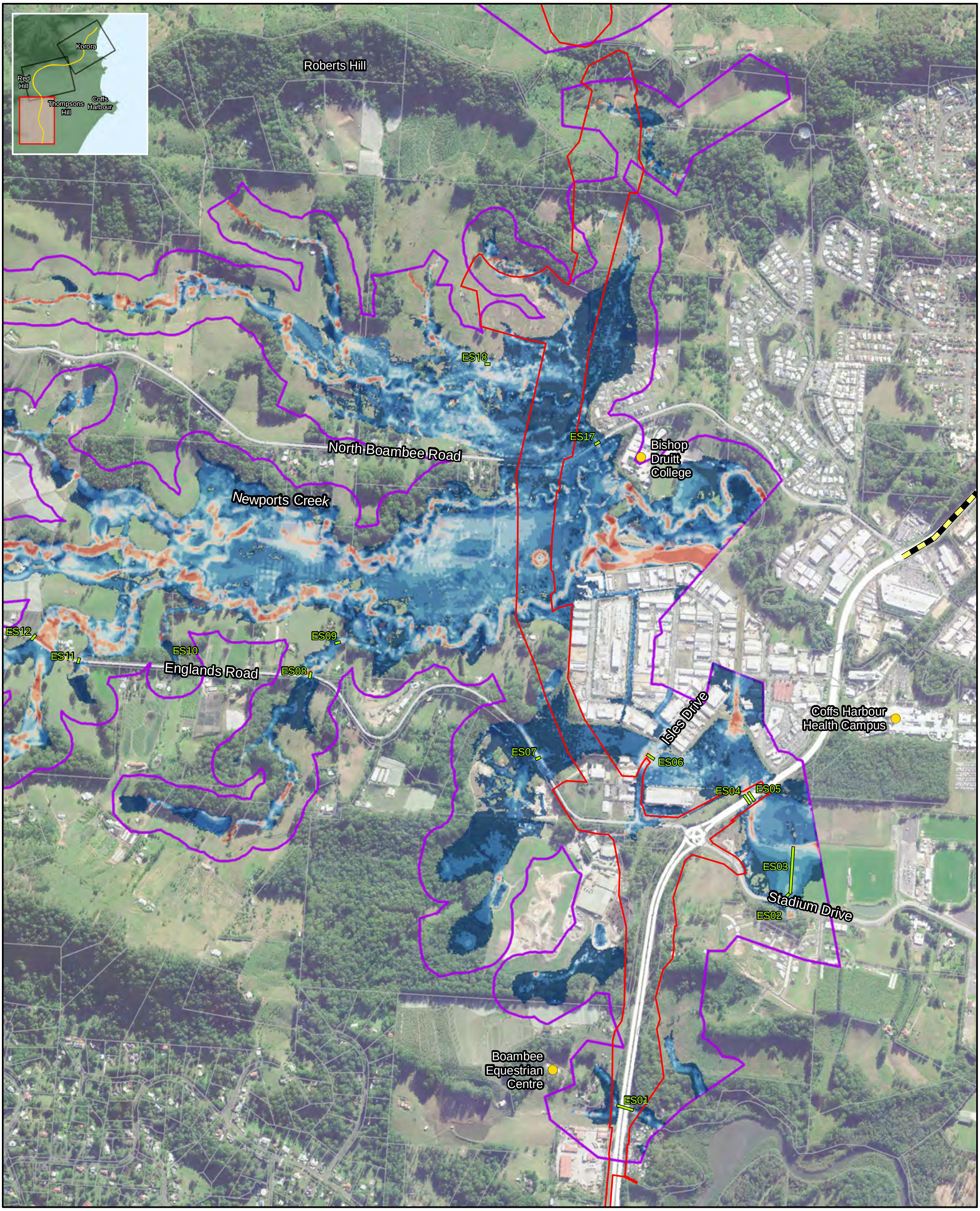
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood velocity (m/s)

0.00 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.25, 1.25 - 1.50, 1.50 - 2.00, 2.00 - 3.00, 3.00 - 4.00, >4.00

Coffs Harbour Bypass
Northern Creek 1 % AEP peak flood velocity
B2.3.5

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

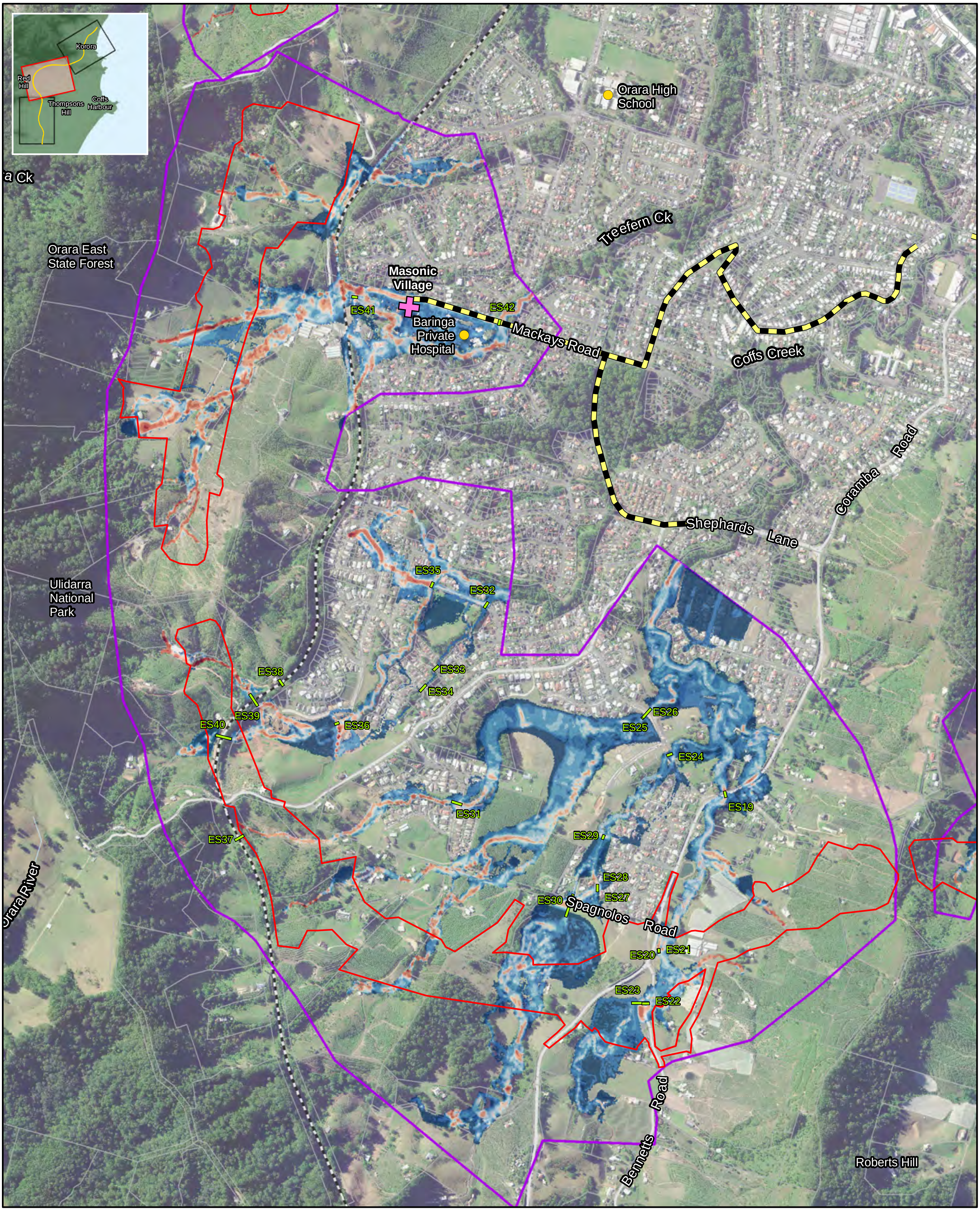
Peak flood velocity (m/s)

0.00 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.25, 1.25 - 1.50, 1.50 - 2.00, 2.00 - 3.00, 3.00 - 4.00, >4.00

Coffs Harbour Bypass
North Boambee Valley 1 % AEP 2050 climate peak flood velocity
B2.1.6

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N



Legend

North Coast Railway
Evacuation routes
Cadastre
Construction footprint
Flood model extents
Modelled structures
Sensitive receiver
Assembly areas

Peak flood velocity (m/s)

0.00 - 0.25
0.25 - 0.50
0.50 - 0.75
0.75 - 1.00
1.00 - 1.25
1.25 - 1.50
1.50 - 2.00
2.00 - 3.00
3.00 - 4.00
>4.00

Coffs Harbour Bypass
Coffs Creek 1 % AEP 2050 climate peak flood velocity
B2.2.6

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N

Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

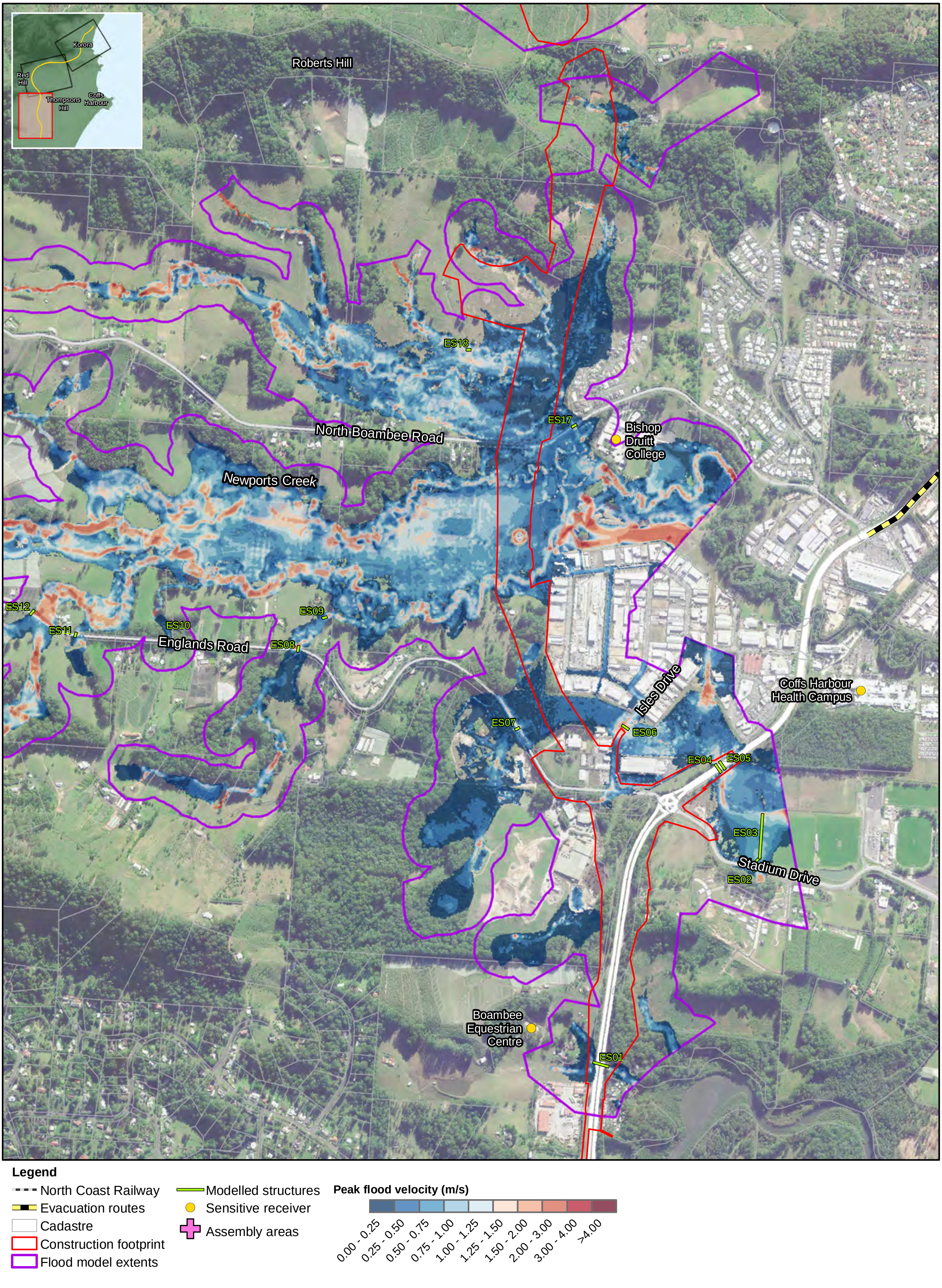
Peak flood velocity (m/s)

0.00 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.25, 1.25 - 1.50, 1.50 - 2.00, 2.00 - 3.00, 3.00 - 4.00, >4.00

Coffs Harbour Bypass
Northern Creek 1 % AEP 2050 climate peak flood velocity
B2.3.6

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

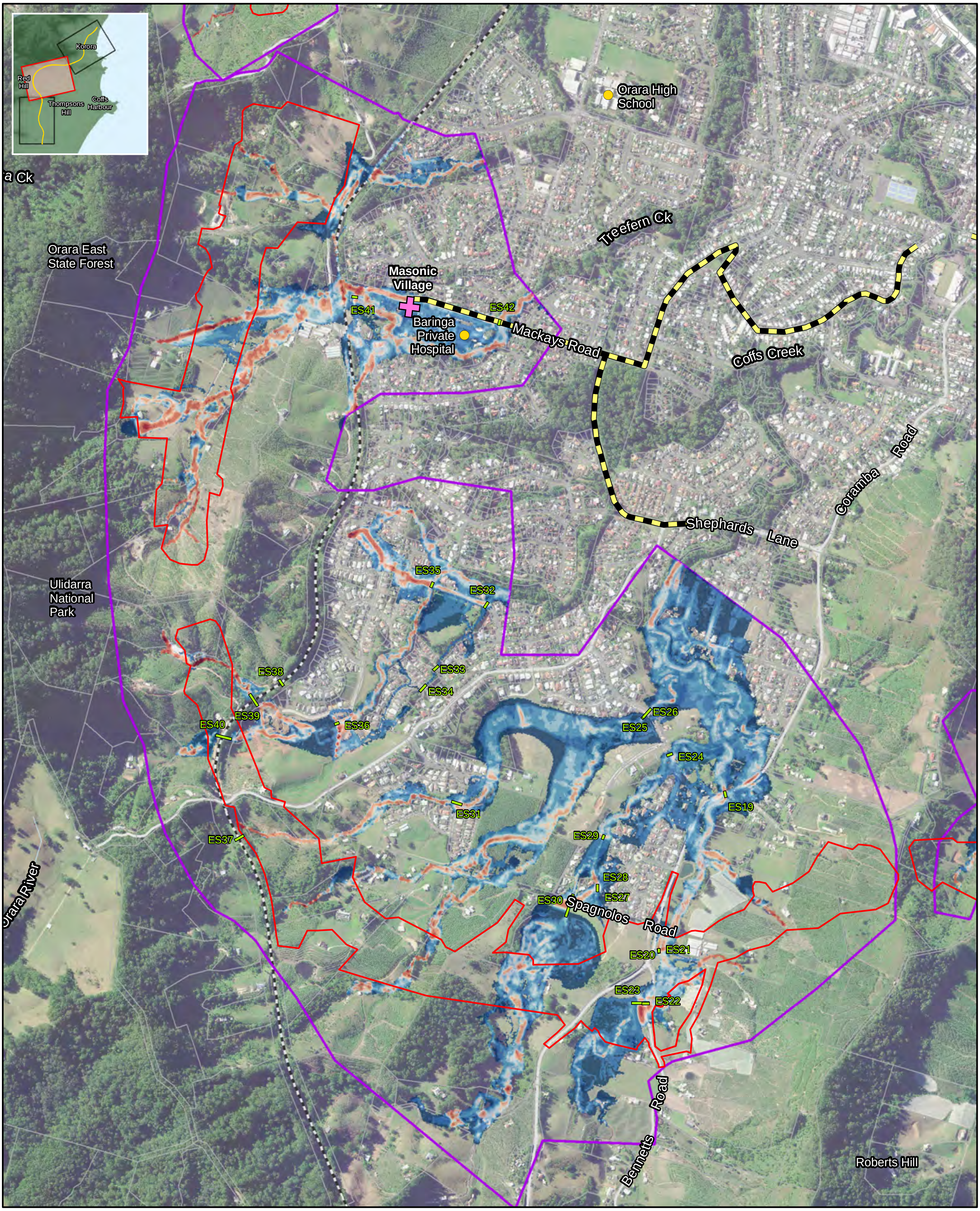
N



Coffs Harbour Bypass

North Boambee Valley 1 % AEP 2100 climate peak flood velocity

B2.1.7



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

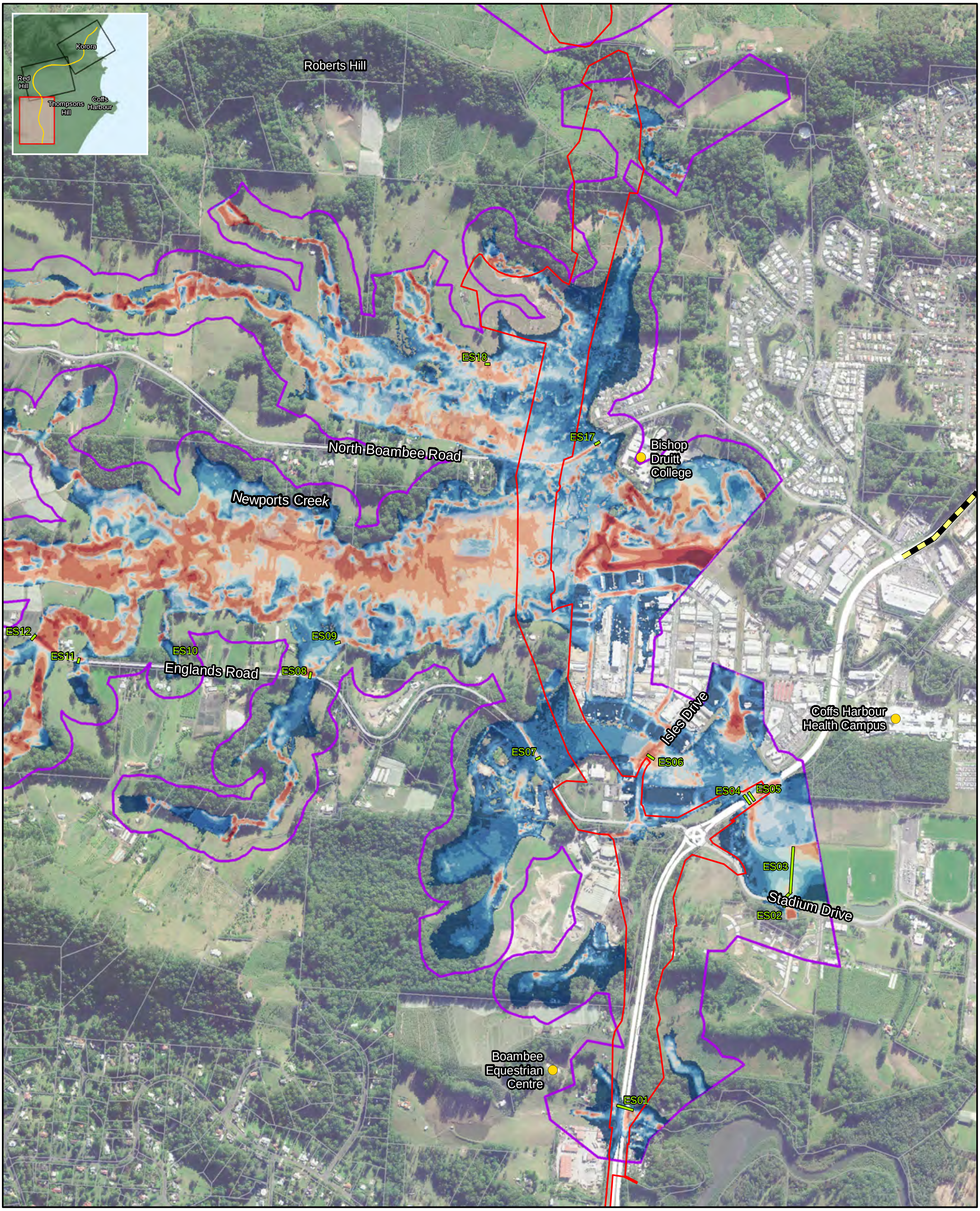
Peak flood velocity (m/s)

0.00 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.25, 1.25 - 1.50, 1.50 - 2.00, 2.00 - 3.00, 3.00 - 4.00, >4.00

Coffs Harbour Bypass
Coffs Creek 1 % AEP 2100 climate peak flood velocity
B2.2.7

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N

Coffs Harbour Bypass
Northern Creek 1 % AEP 2100 climate peak flood velocity
B2.3.7



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

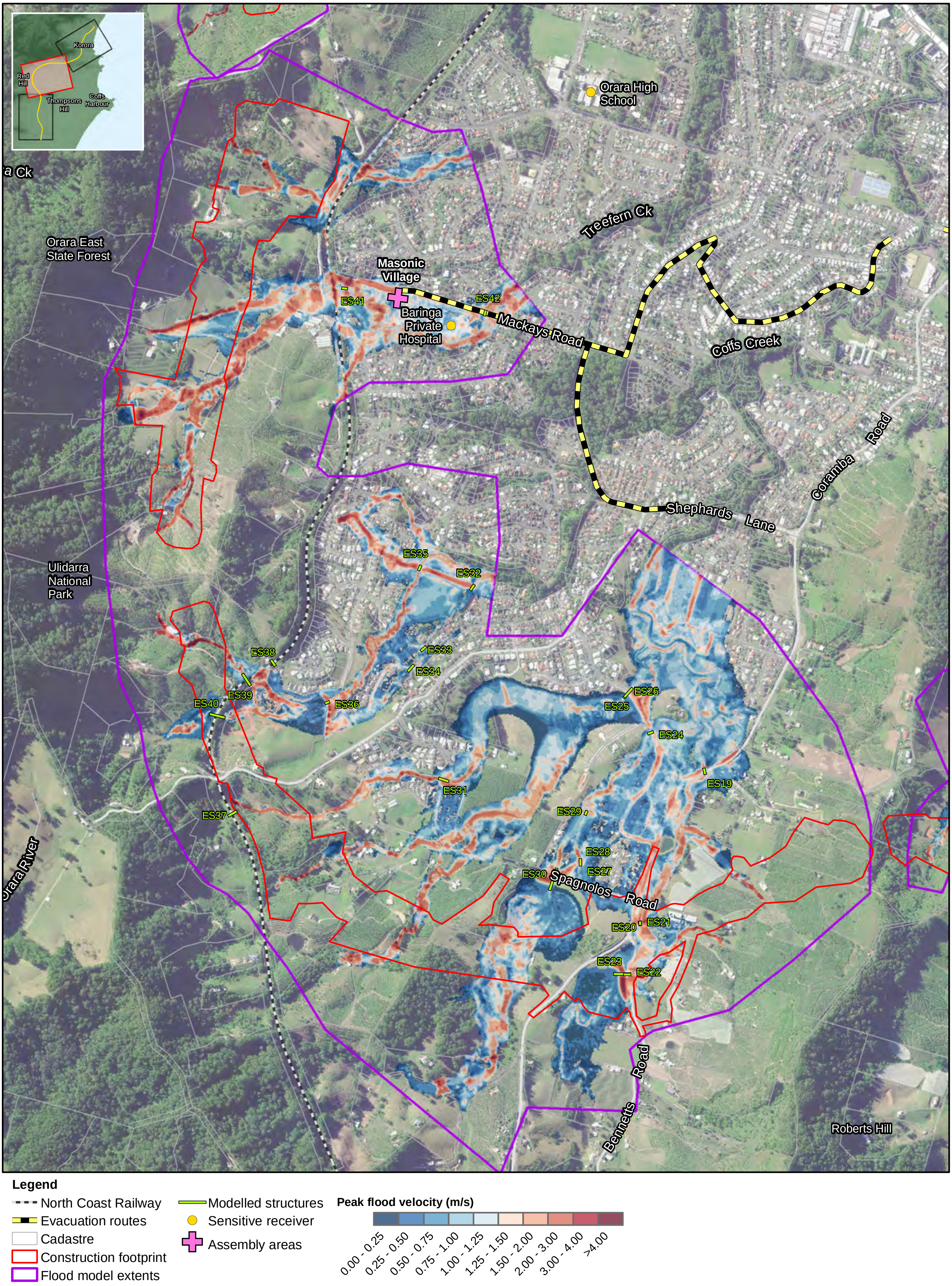
Peak flood velocity (m/s)

0.00 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.25, 1.25 - 1.50, 1.50 - 2.00, 2.00 - 3.00, 3.00 - 4.00, >4.00

Coffs Harbour Bypass
North Boambee Valley PMF peak flood velocity
B2.1.8

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N



Coffs Harbour Bypass
Coffs Creek PMF peak flood velocity
B2.2.8

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N

Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

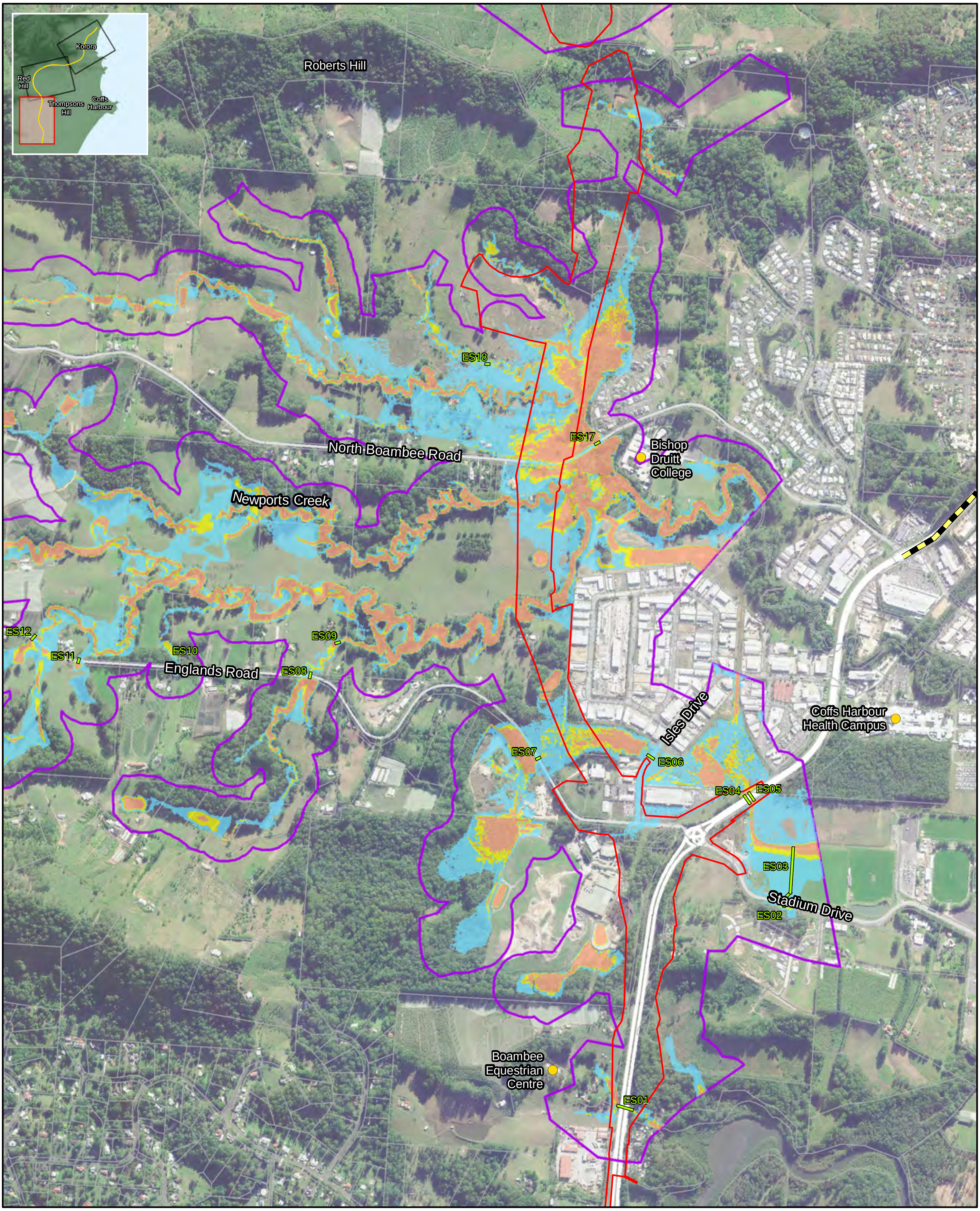
Peak flood velocity (m/s)

0.00 - 0.25, 0.25 - 0.50, 0.50 - 0.75, 0.75 - 1.00, 1.00 - 1.25, 1.25 - 1.50, 1.50 - 2.00, 2.00 - 3.00, 3.00 - 4.00, >4.00

Coffs Harbour Bypass
Northern Creek PMF peak flood velocity
B2.3.8

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

B3 Peak flood hazard



Legend

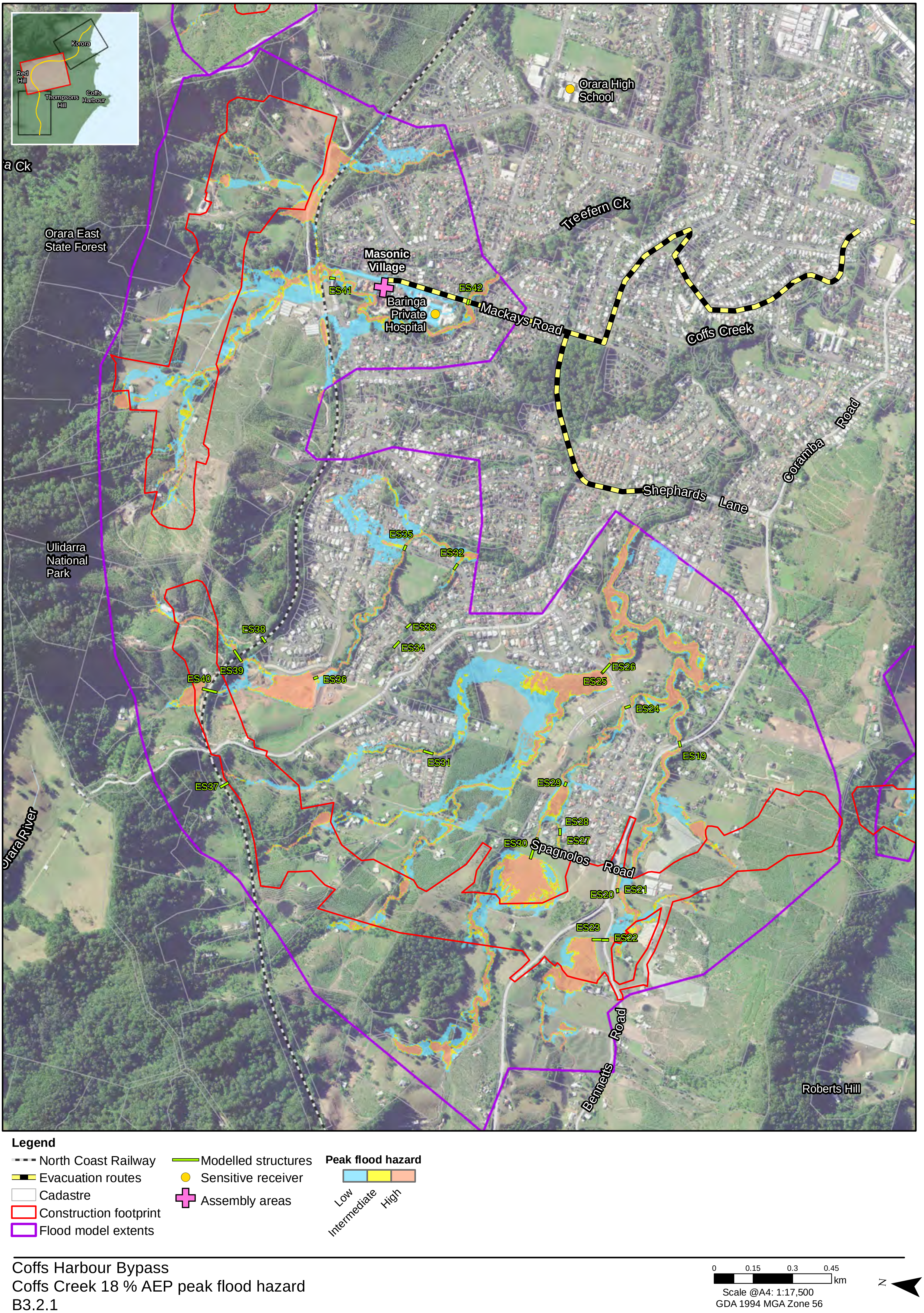
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood hazard

Low, Intermediate, High

Coffs Harbour Bypass
North Boambee Valley 18 % AEP peak flood hazard
B3.1.1

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Coffs Harbour Bypass
Coffs Creek 18 % AEP peak flood hazard
B3.2.1

Legend

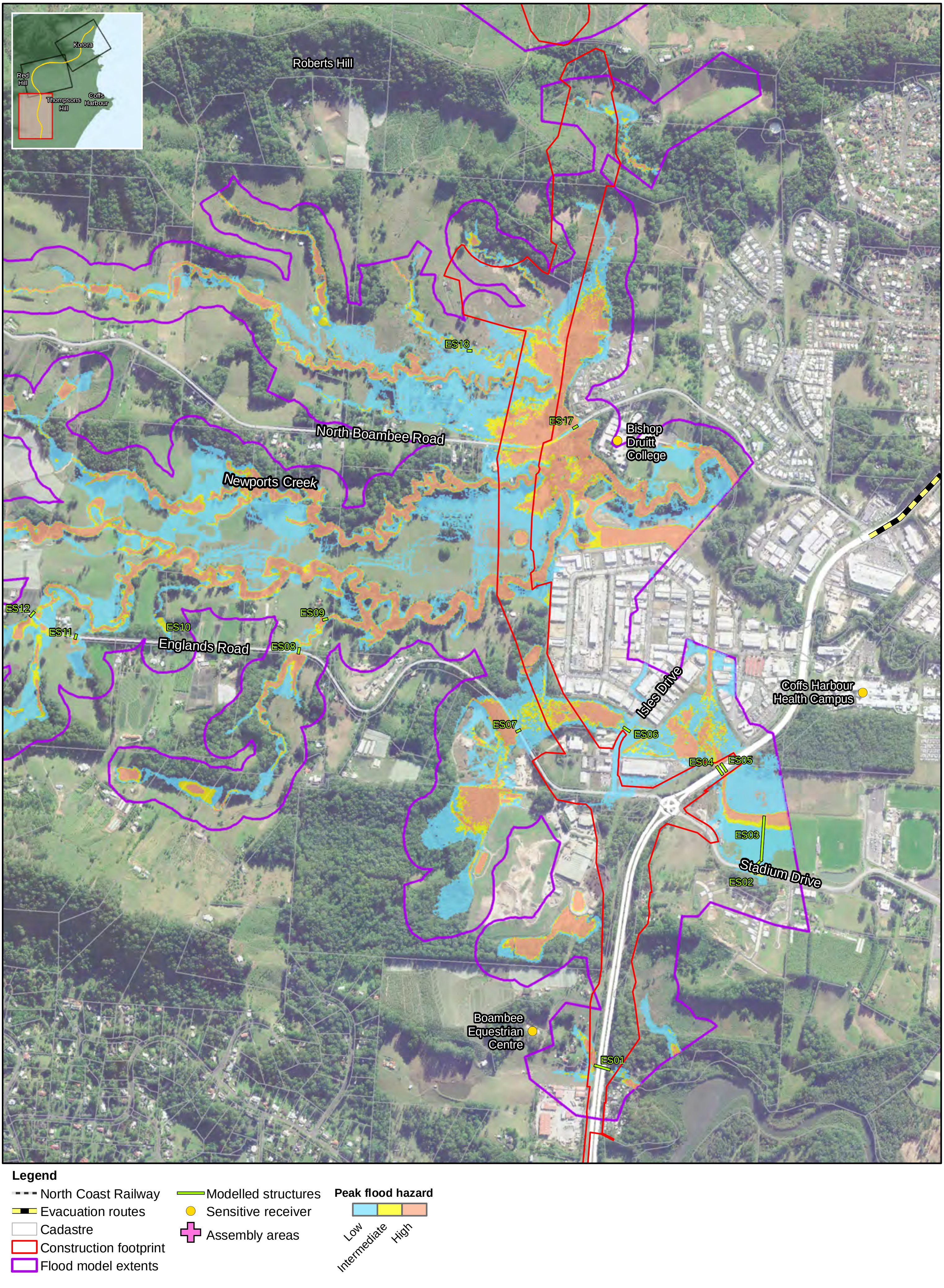
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood hazard

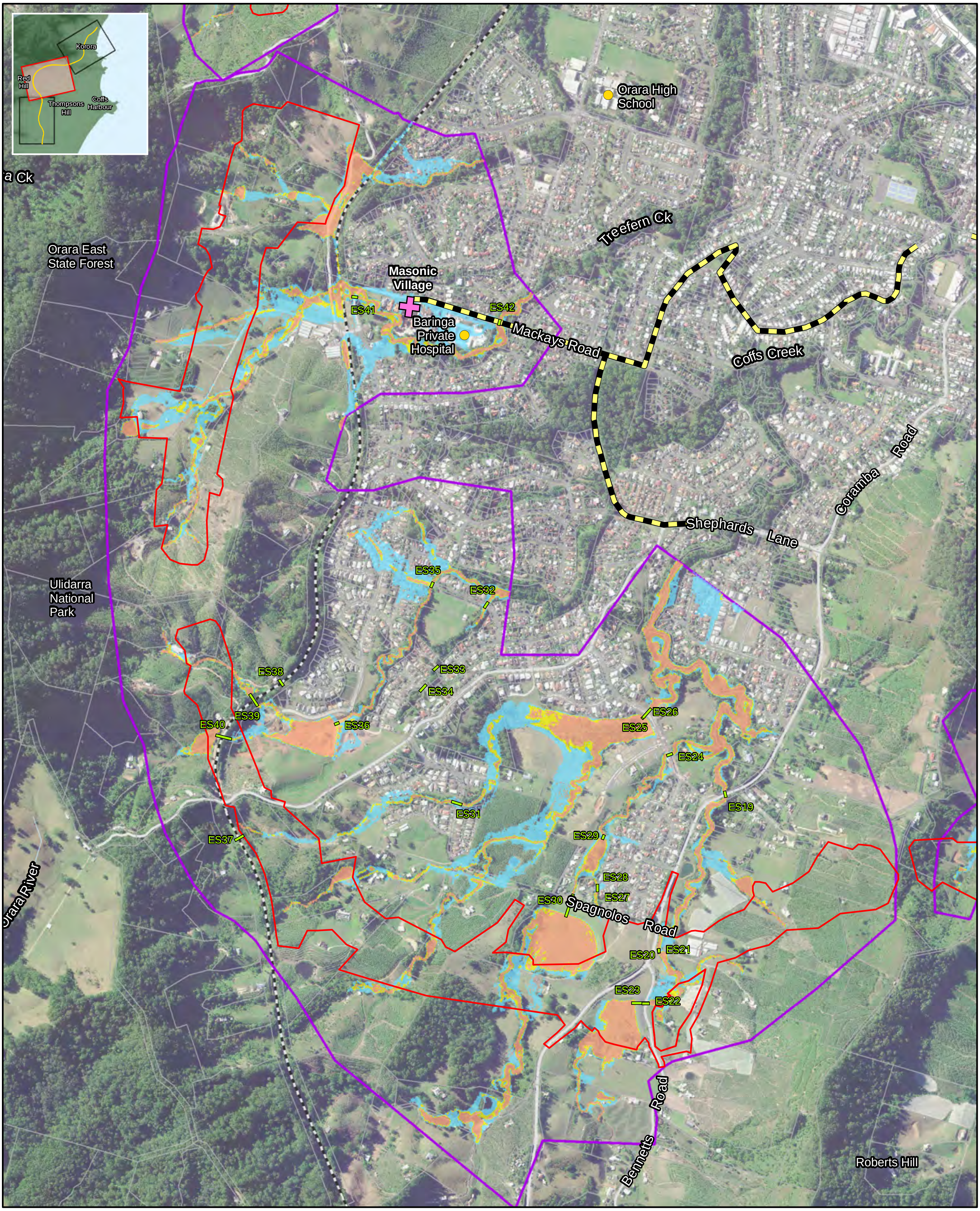
Low, Intermediate, High

Coffs Harbour Bypass
Northern Creek 18 % AEP peak flood hazard
B3.3.1

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Coffs Harbour Bypass
North Boambee Valley 10 % AEP peak flood hazard
B3.1.2



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood hazard

Low | Intermediate | High

0 0.15 0.3 0.45 km

Scale @A4: 1:17,500

GDA 1994 MGA Zone 56

N

Coffs Harbour Bypass

Coffs Creek 10 % AEP peak flood hazard

B3.2.2

Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

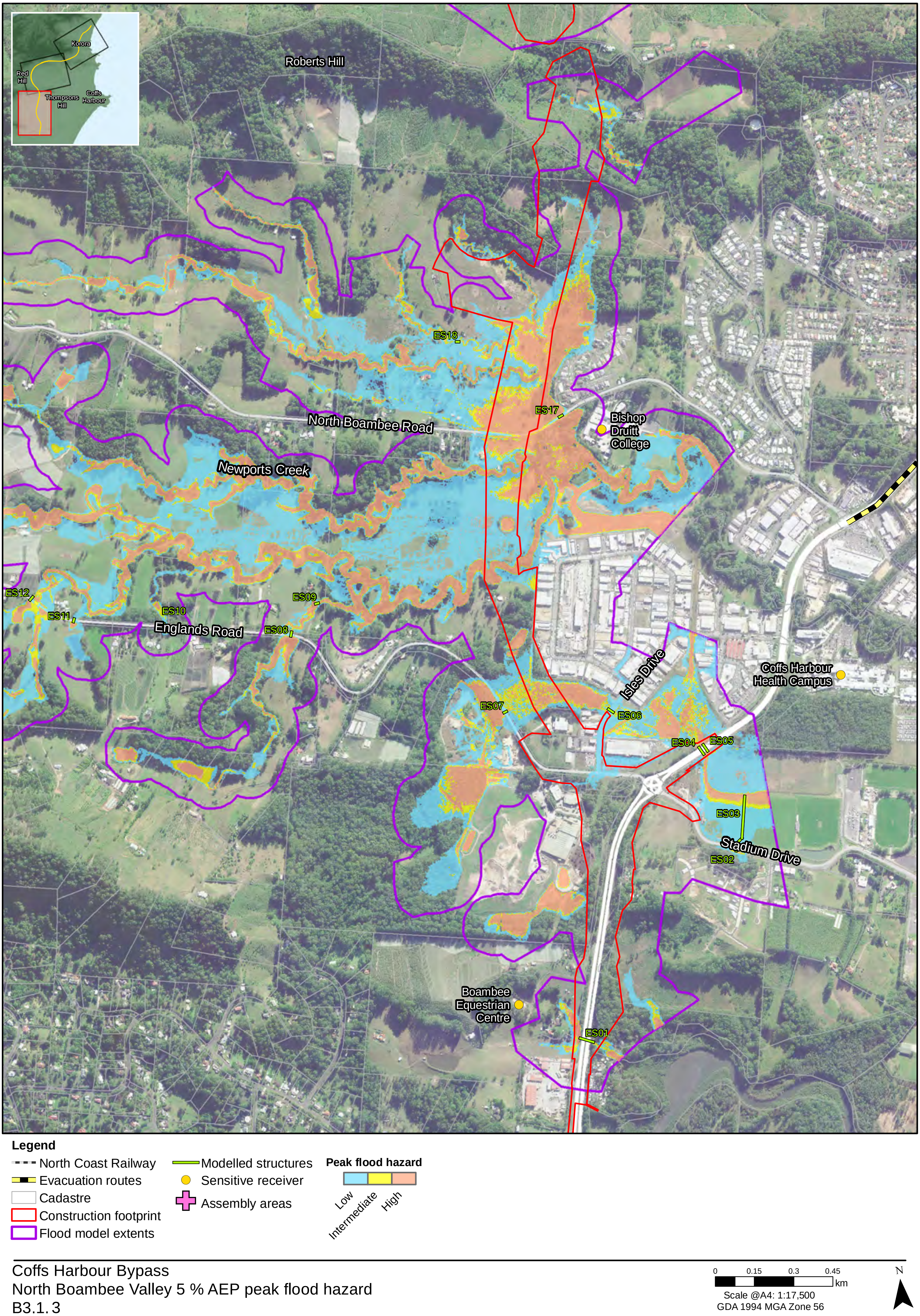
Peak flood hazard

Low | Intermediate | High

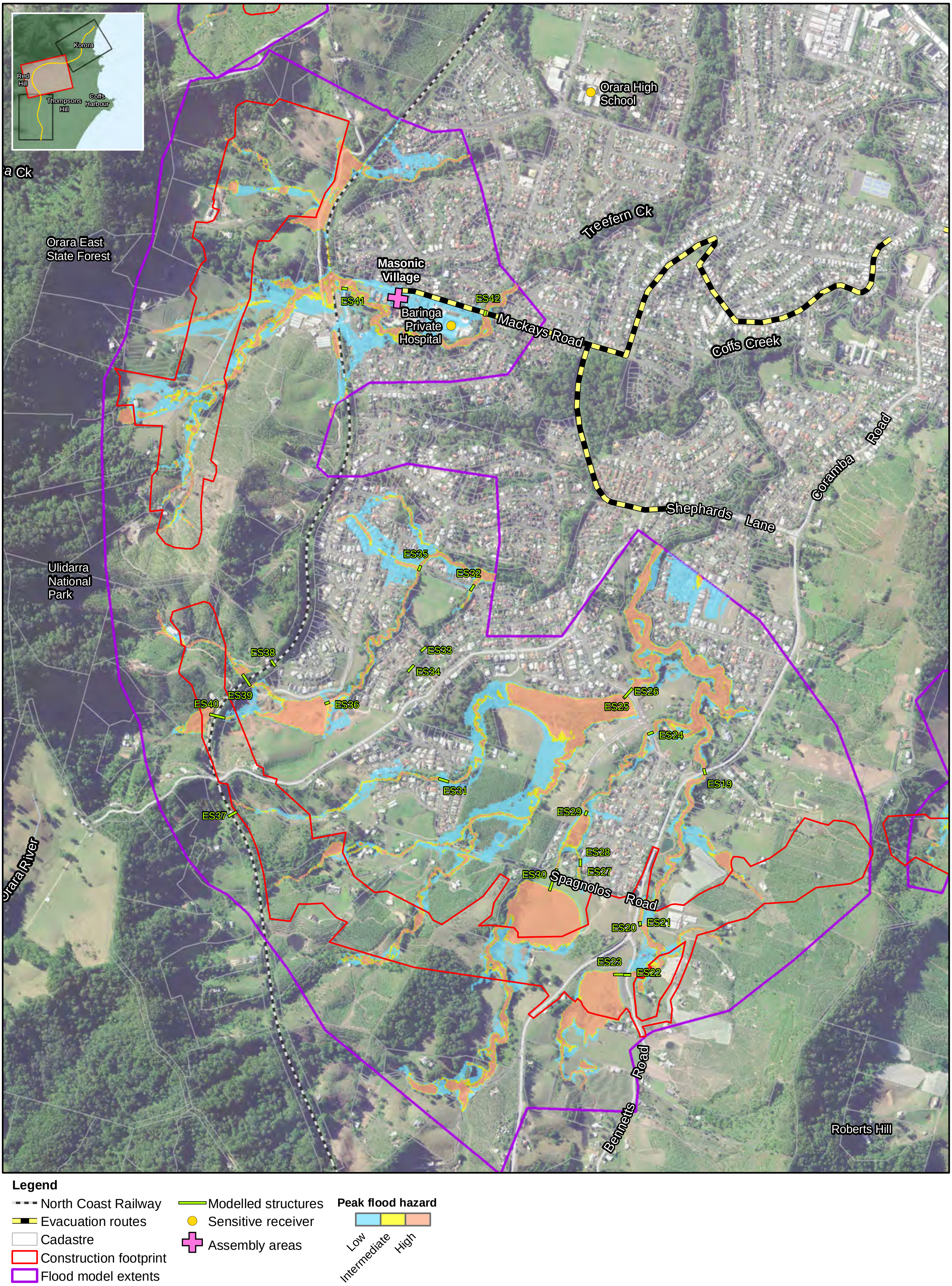
Coffs Harbour Bypass
Northern Creek 10 % AEP peak flood hazard
B3.3. 2

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N



Coffs Harbour Bypass
North Boambee Valley 5 % AEP peak flood hazard
B3.1.3



Coffs Harbour Bypass

Coffs Creek 5 % AEP peak flood hazard

B3.2.3

0 0.15 0.3 0.45 km

Scale @A4: 1:17,500

GDA 1994 MGA Zone 56

N

Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

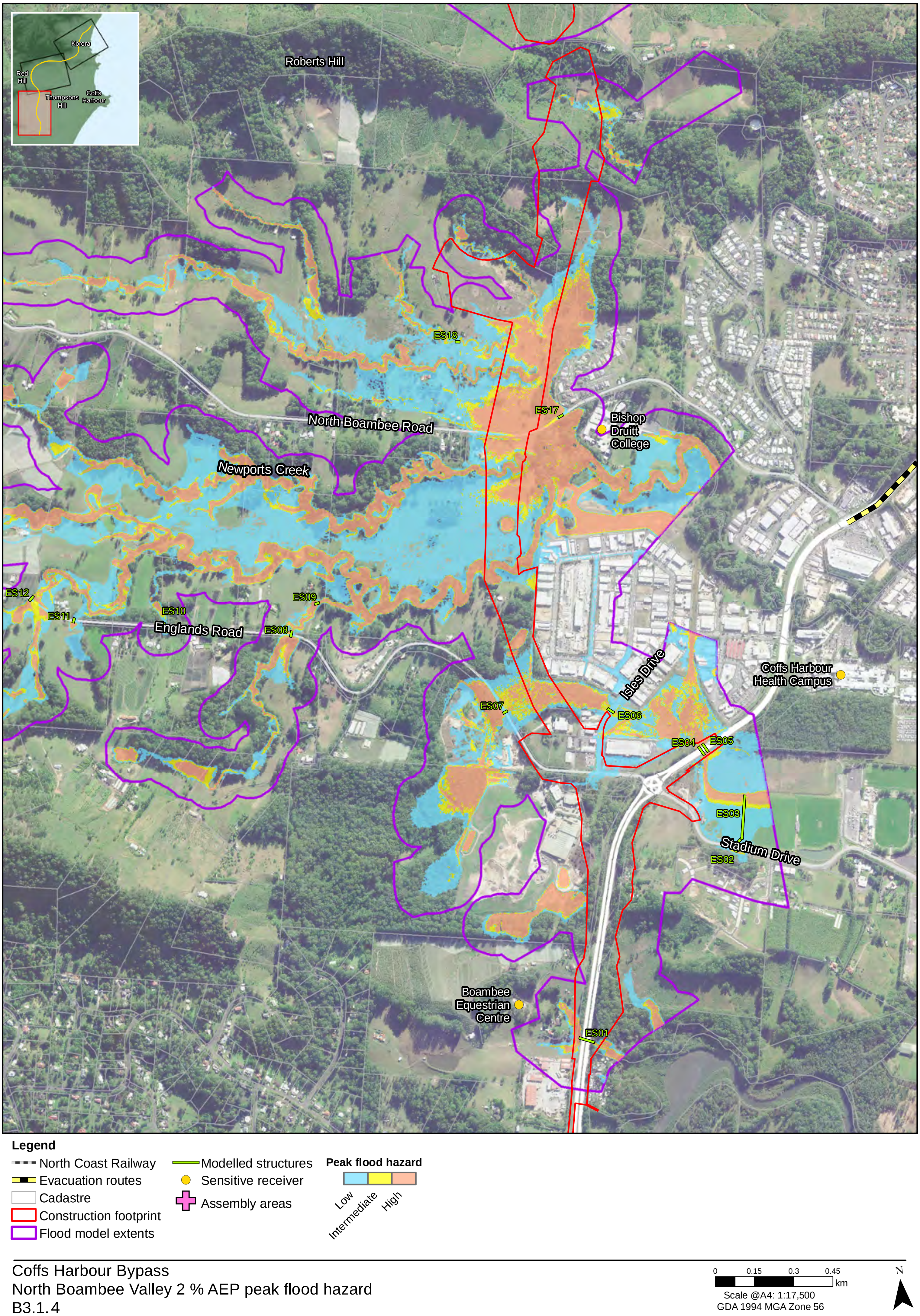
Peak flood hazard

Low | Intermediate | High

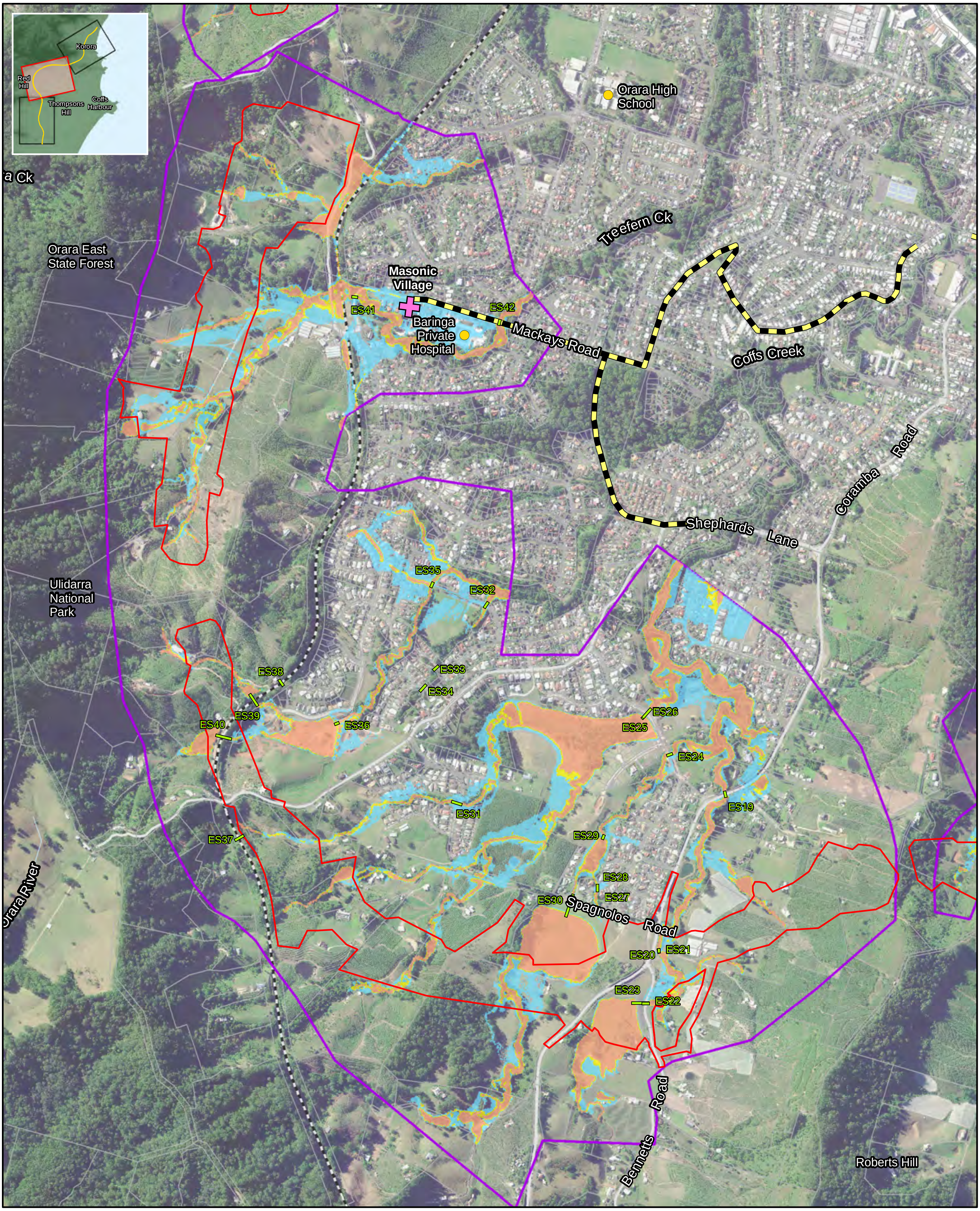
Coffs Harbour Bypass
Northern Creek 5 % AEP peak flood hazard
B3.3. 3

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N



Coffs Harbour Bypass
North Boambee Valley 2 % AEP peak flood hazard
B3.1.4



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood hazard

Low | Intermediate | High

Coffs Harbour Bypass
Coffs Creek 2 % AEP peak flood hazard
B3.2.4

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N

Legend

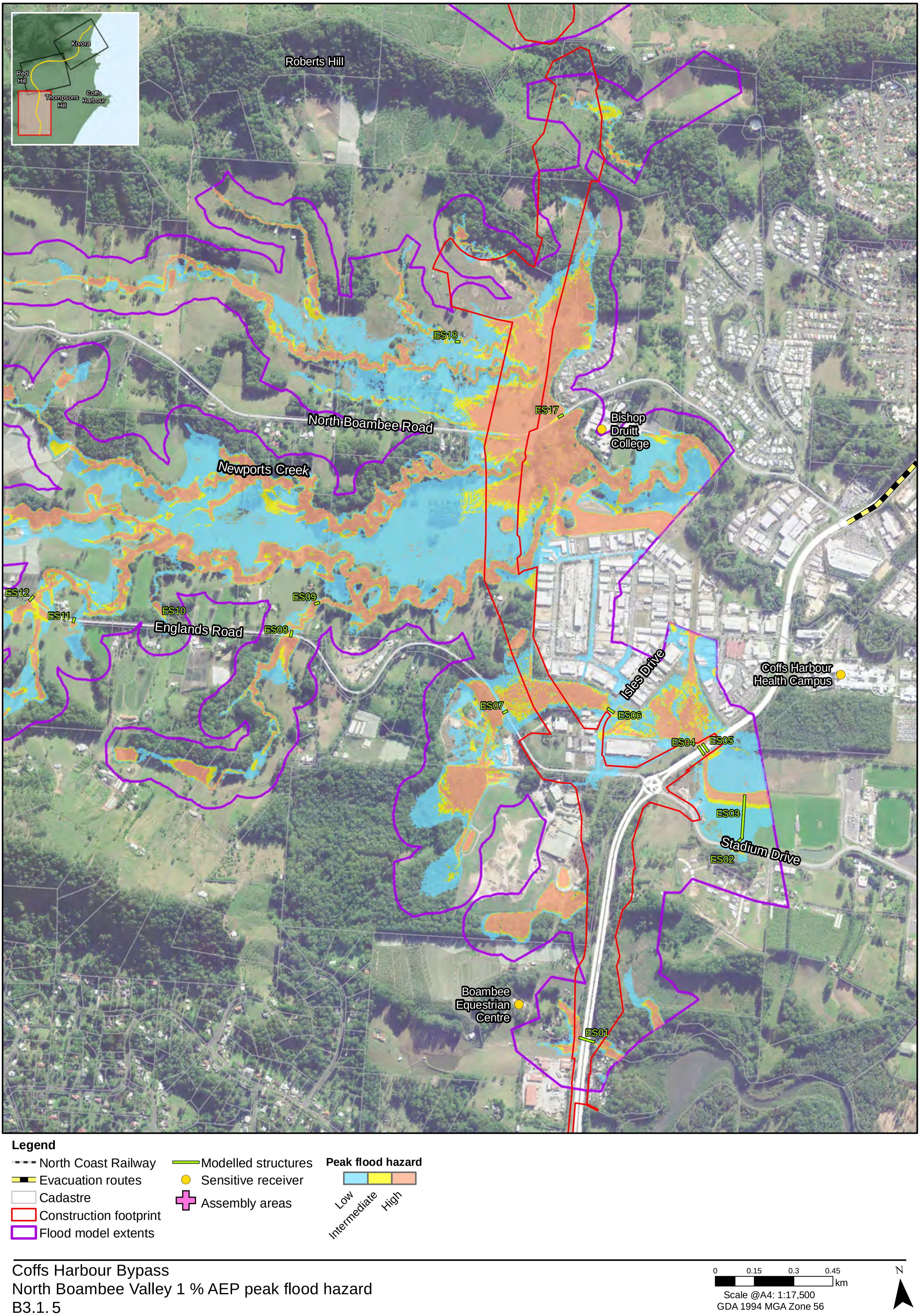
North Coast Railway
Evacuation routes
Cadastre
Construction footprint
Flood model extents
Modelled structures
Sensitive receiver
Assembly areas

Peak flood hazard

Low
Intermediate
High

Coffs Harbour Bypass
Northern Creek 2 % AEP peak flood hazard
B3.3.4

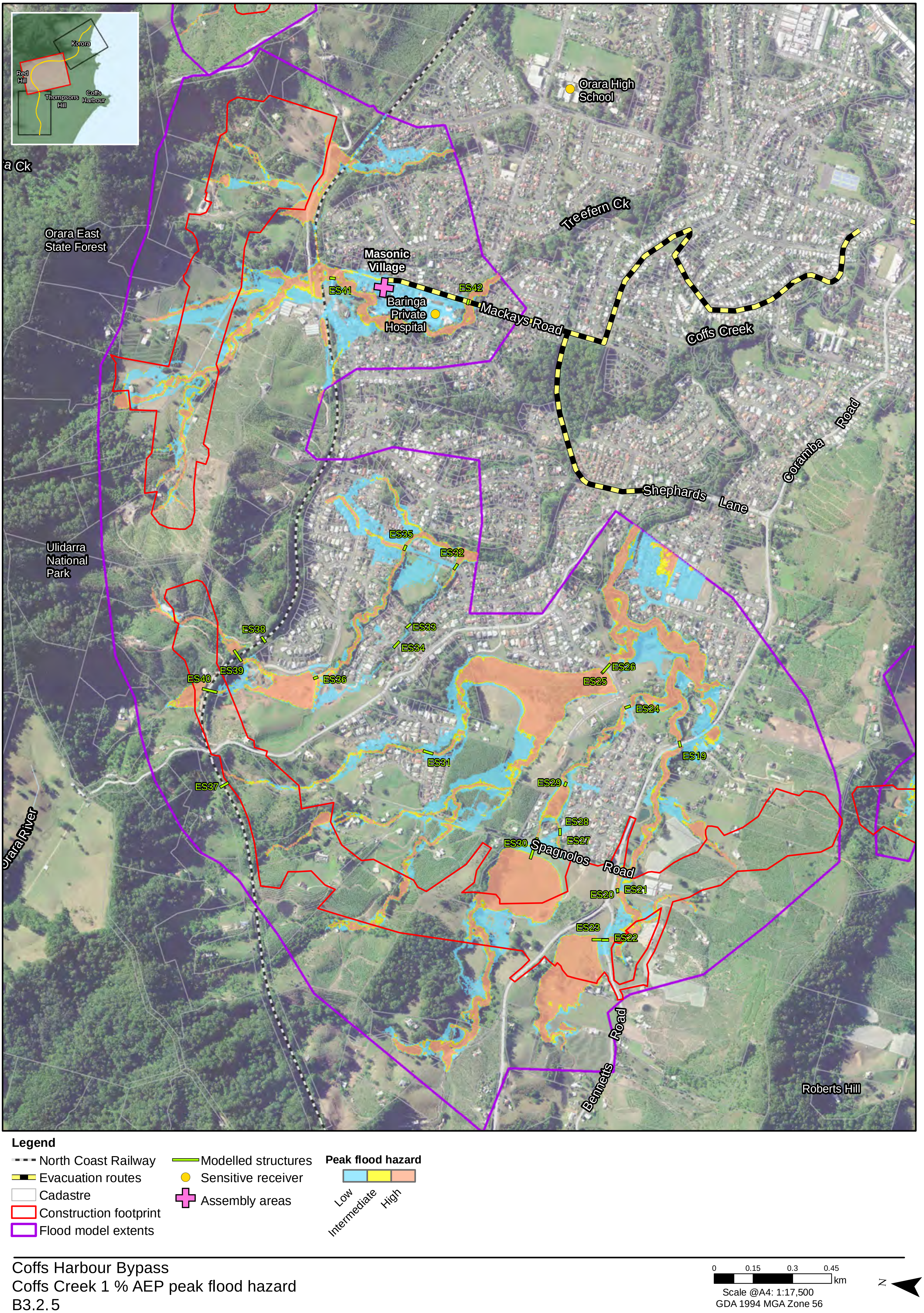
0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



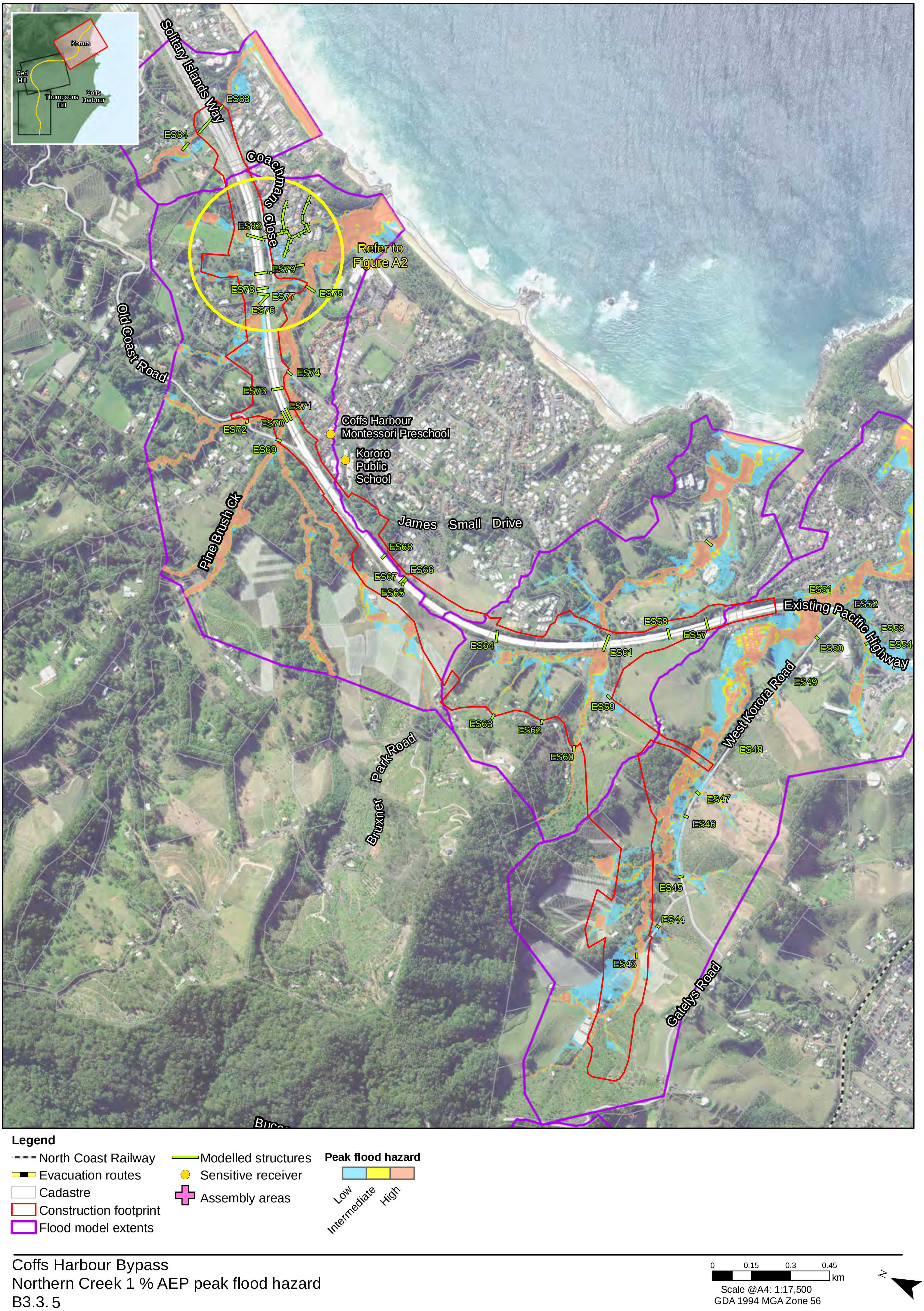
Coffs Harbour Bypass

North Boambee Valley 1 % AEP peak flood hazard

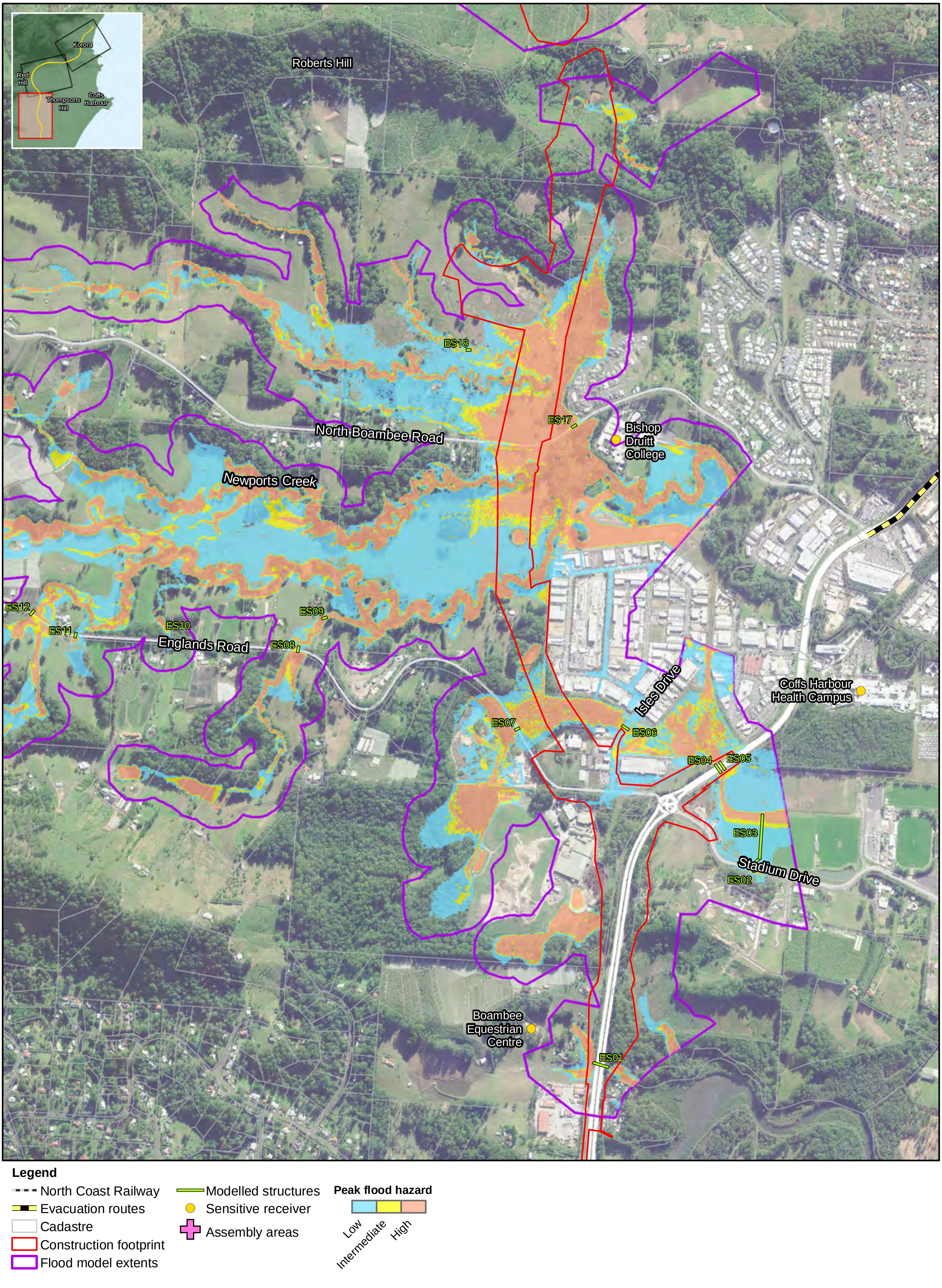
B3.1. 5



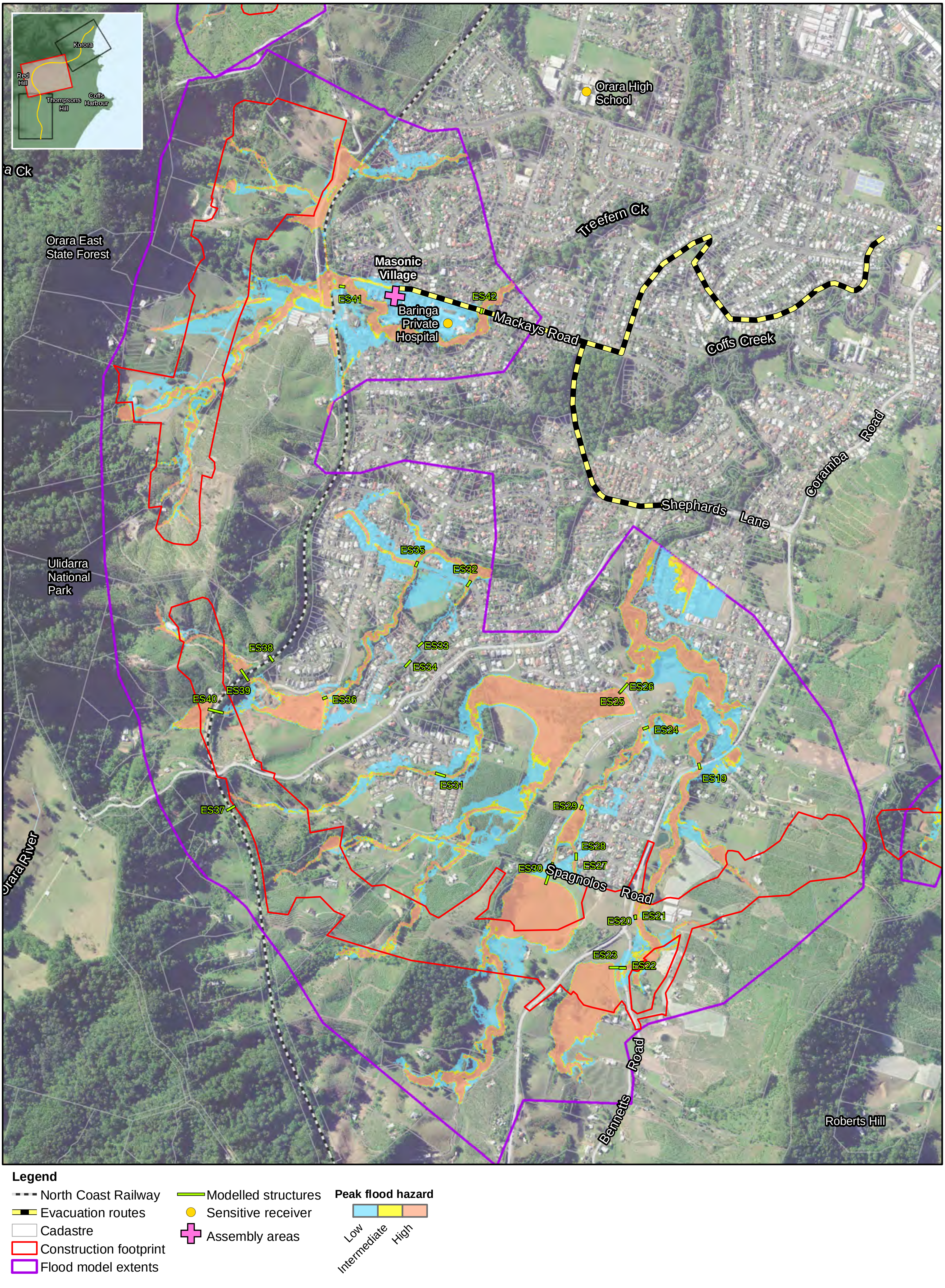
Coffs Harbour Bypass
Coffs Creek 1 % AEP peak flood hazard
B3.2.5



Coffs Harbour Bypass
Northern Creek 1 % AEP peak flood hazard
B3.3. 5



Coffs Harbour Bypass
North Boambee Valley 1 % AEP 2050 climate peak flood hazard
B3.1. 6



Coffs Harbour Bypass

Coffs Creek 1 % AEP 2050 climate peak flood hazard

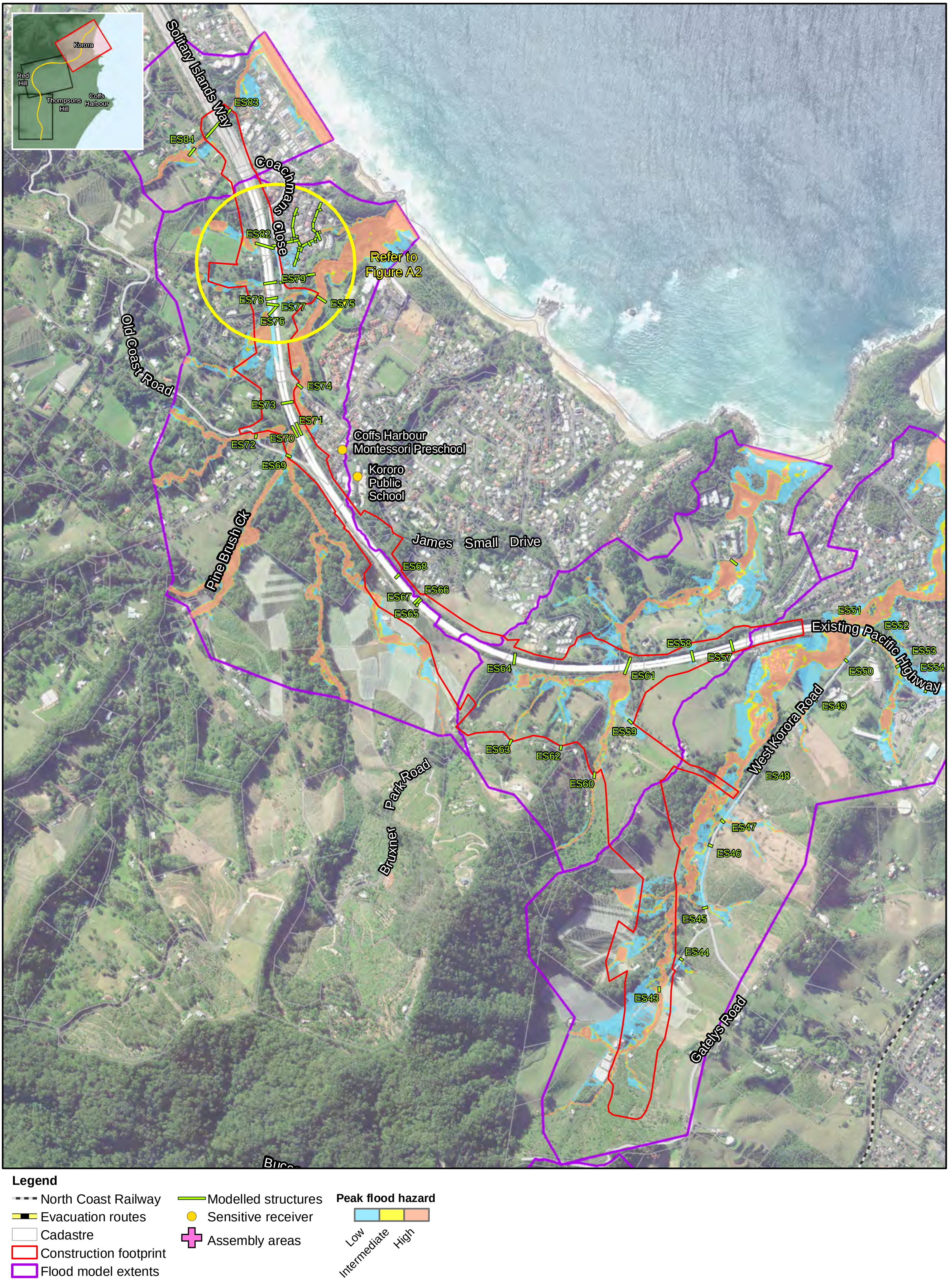
B3.2.6

0 0.15 0.3 0.45 km

Scale @A4: 1:17,500

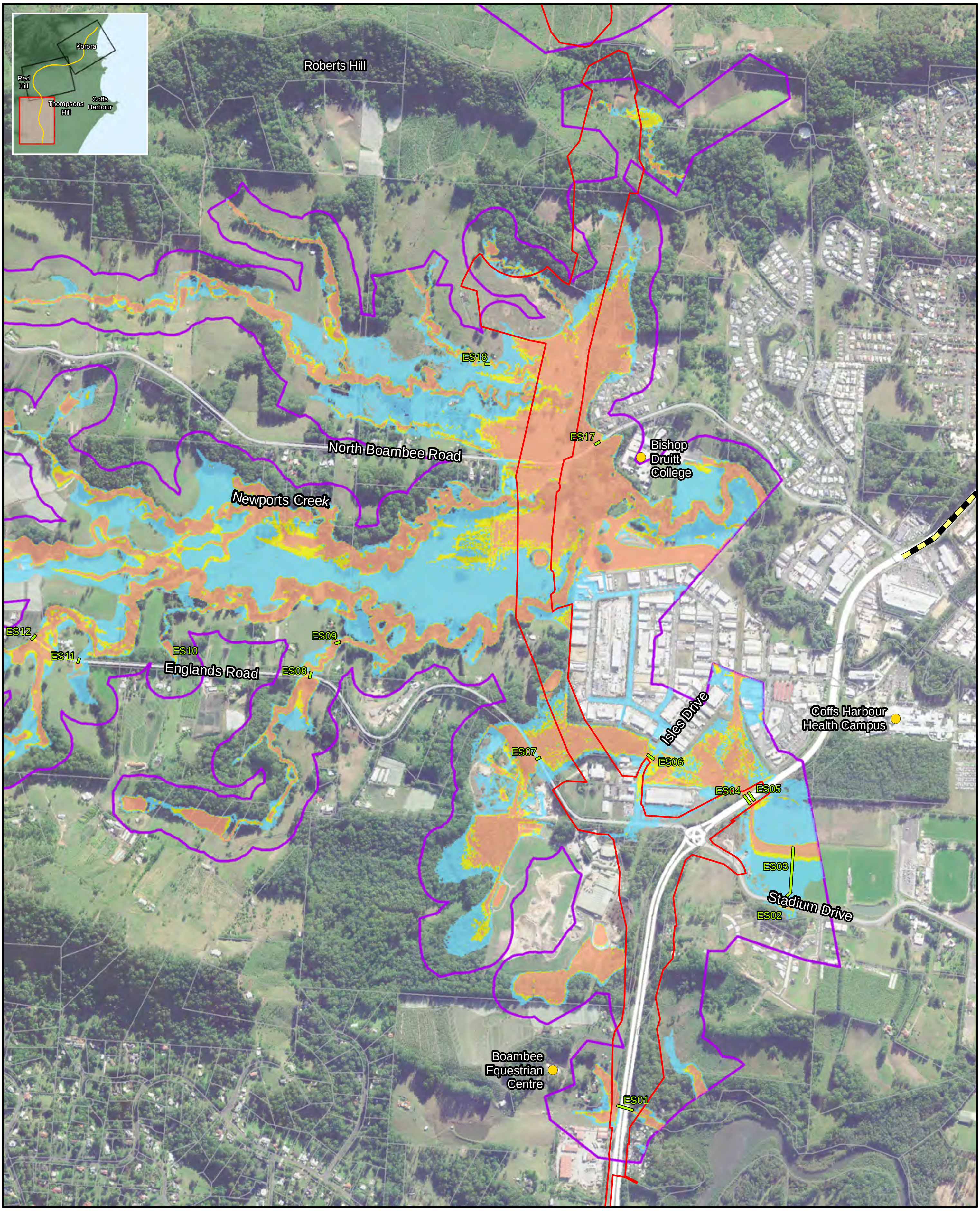
GDA 1994 MGA Zone 56

N



Coffs Harbour Bypass
Northern Creek 1 % AEP 2050 climate peak flood hazard
B3.3. 6

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood hazard

Low, Intermediate, High

0 0.15 0.3 0.45 km

Scale @A4: 1:17,500

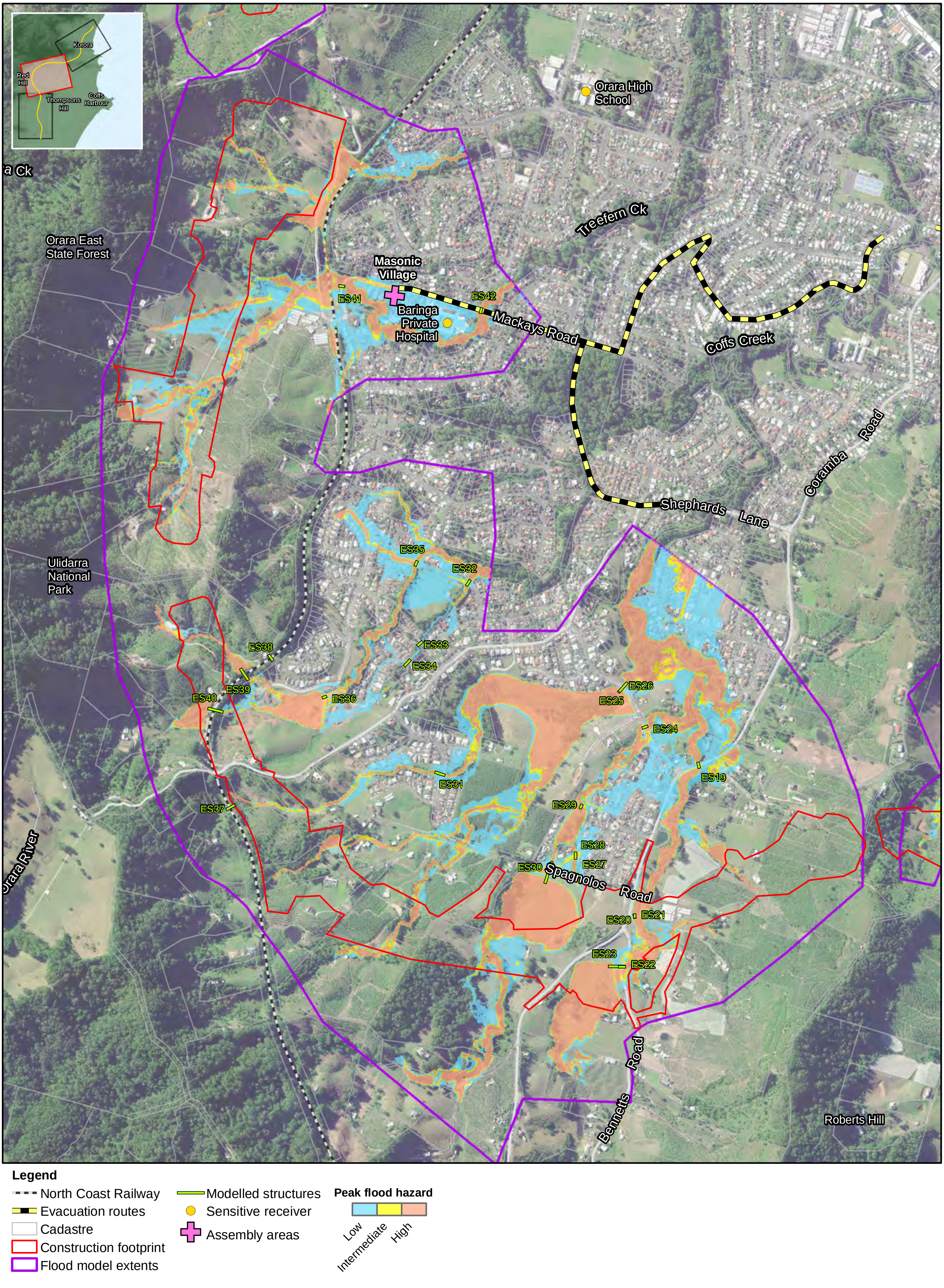
GDA 1994 MGA Zone 56

N

Coffs Harbour Bypass

North Boambee Valley 1 % AEP 2100 climate peak flood hazard

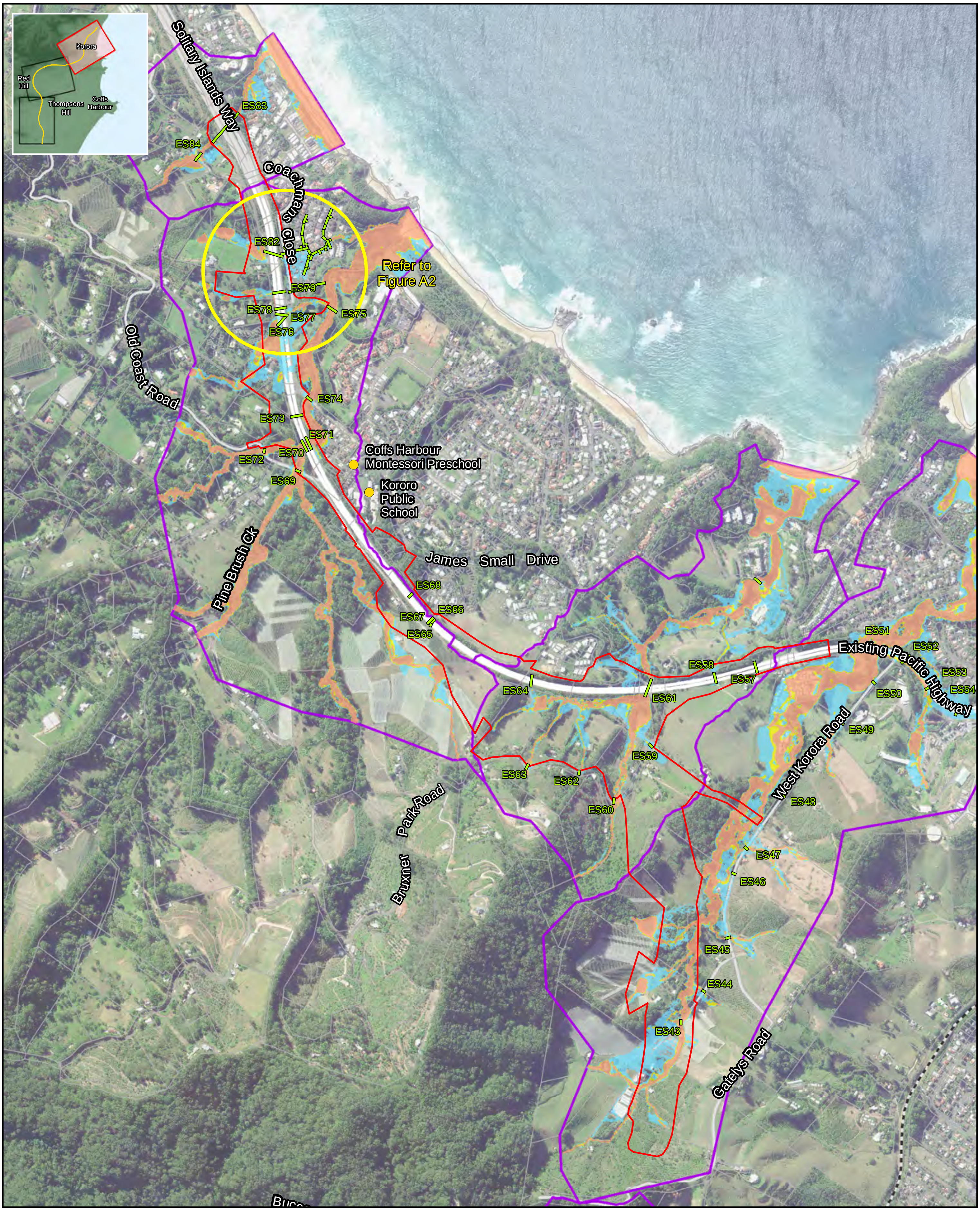
B3.1.7



Coffs Harbour Bypass

Coffs Creek 1 % AEP 2100 climate peak flood hazard

B3.2. 7



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

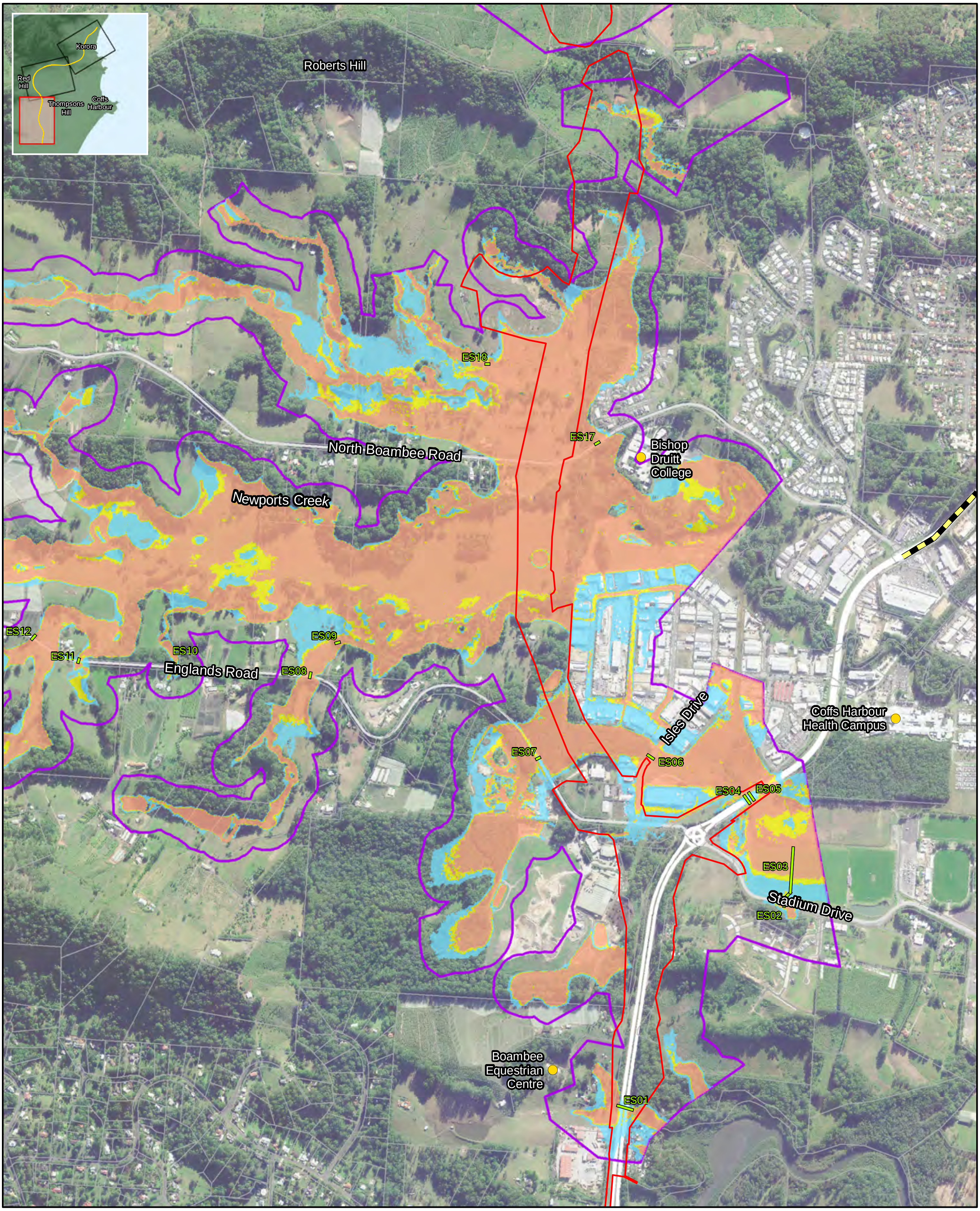
Peak flood hazard

Low | Intermediate | High

Coffs Harbour Bypass
Northern Creek 1 % AEP 2100 climate peak flood hazard
B3.3.7

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N



Legend

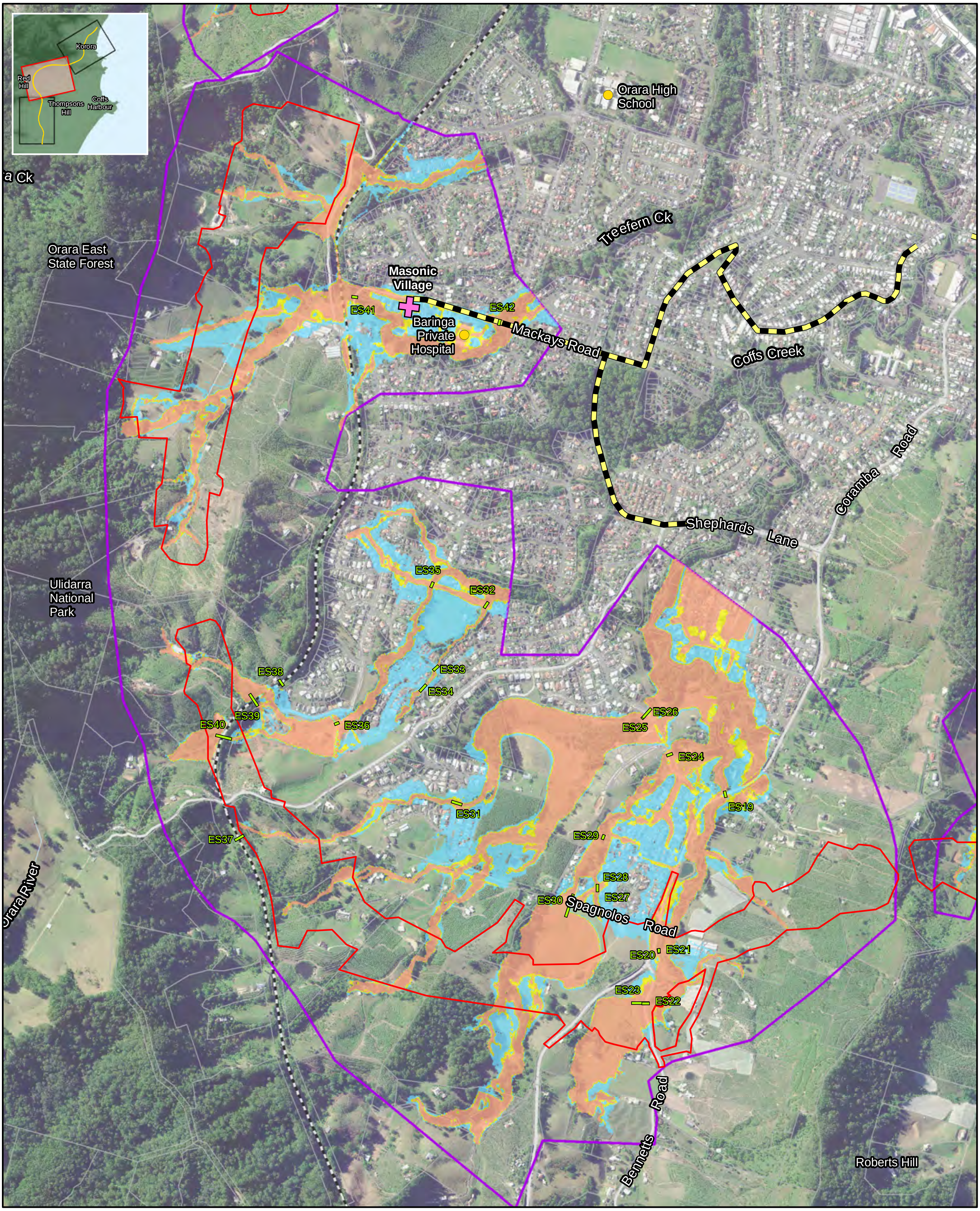
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood hazard

Low | Intermediate | High

Coffs Harbour Bypass
North Boambee Valley PMF peak flood hazard
B3.1.8

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

North Coast Railway
Evacuation routes
Cadastre
Construction footprint
Flood model extents
Modelled structures
Sensitive receiver
Assembly areas

Peak flood hazard

Low
Intermediate
High

Coffs Harbour Bypass
Coffs Creek PMF peak flood hazard
B3.2.8

0 0.15 0.3 0.45 km

Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N

Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas

Peak flood hazard

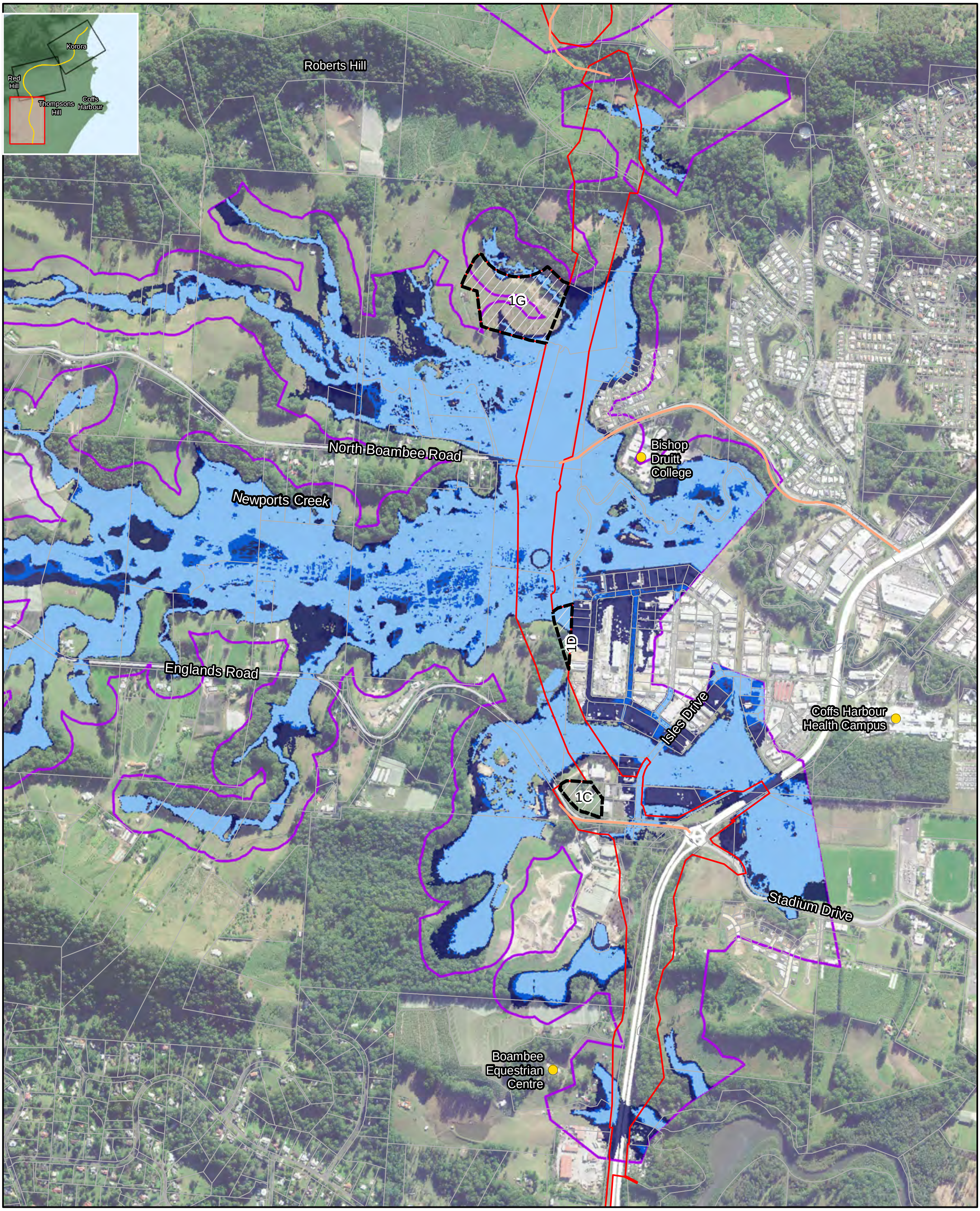
Low | Intermediate | High

Coffs Harbour Bypass
Northern Creek PMF peak flood hazard
B3.3.8

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

Appendix C

Conceptual construction flood maps

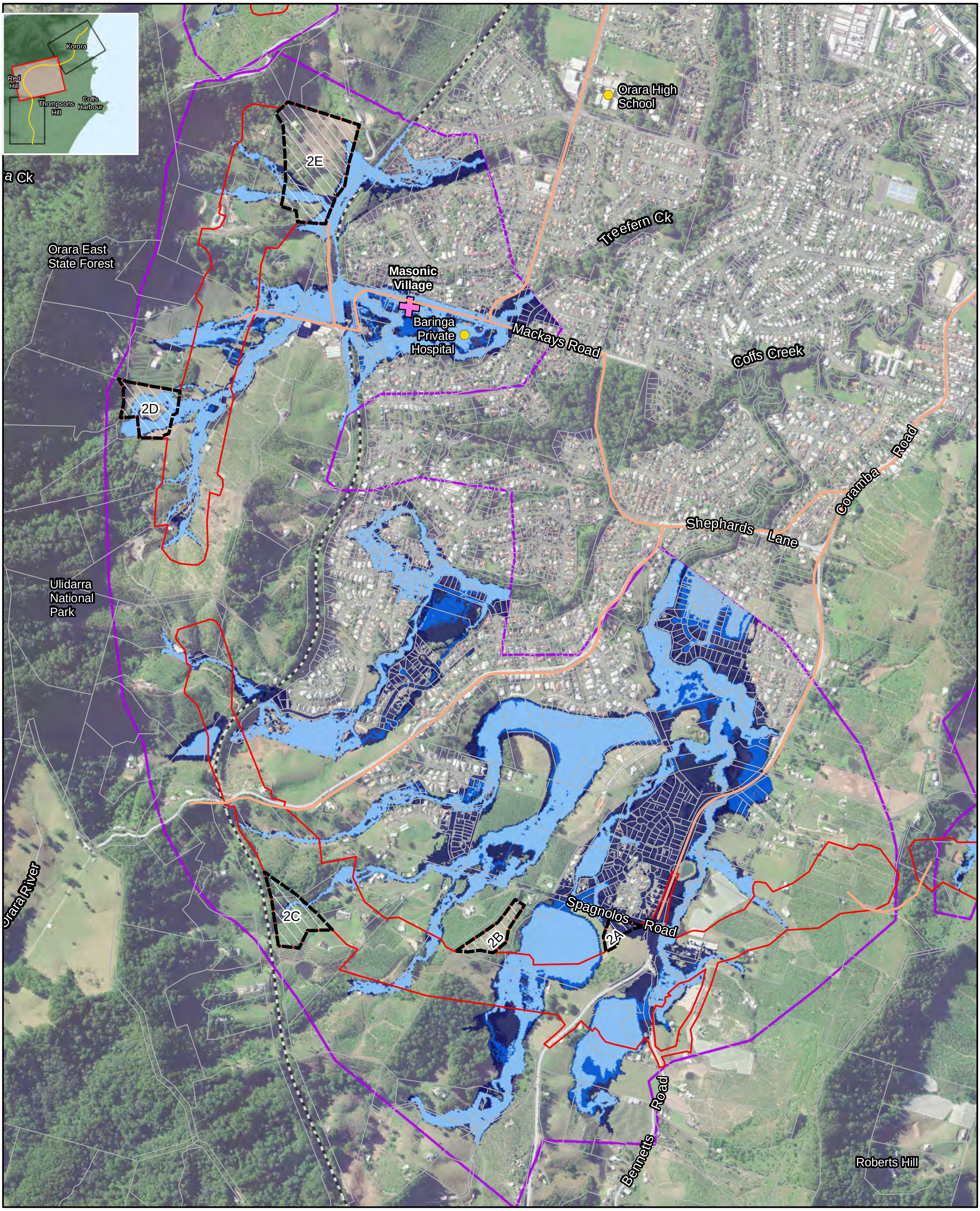


Legend

- North Coast Railway
- Cadastre
- Construction footprint
- Flood model extents
- Sensitive receiver
- Assembly areas
- Potential construction access
- Potential ancillary sites
- 5 % AEP flood extent
- 1 % AEP flood extent
- PMF flood extent

Coffs Harbour Bypass
North Boambee Valley ancillary sites peak flood inundation
C1

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N

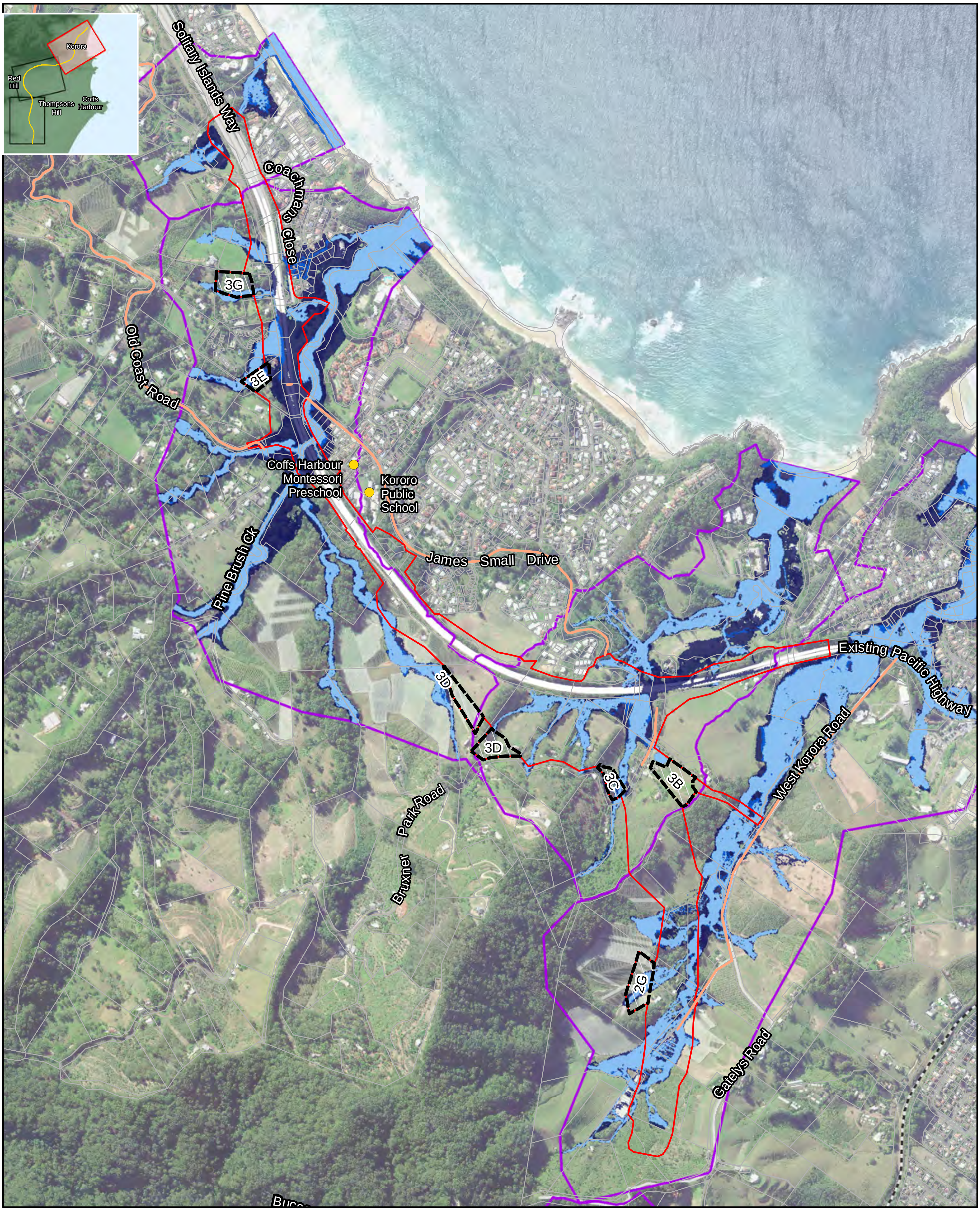


Legend

- North Coast Railway
- Cadastre
- Construction footprint
- Flood model extents
- Sensitive receiver
- Assembly areas
- Potential construction access
- Potential ancillary sites
- 5 % AEP flood extent
- 1 % AEP flood extent
- PMF flood extent

Coffs Harbour Bypass
Coffs Creek ancillary sites peak flood inundation
C2

0 0.15 0.3 0.45 km
N
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56



Legend

- North Coast Railway
- Cadastre
- Construction footprint
- Flood model extents
- Sensitive receiver
- Assembly areas
- Potential construction access
- Potential ancillary sites
- 5 % AEP flood extent
- 1 % AEP flood extent
- PMF flood extent

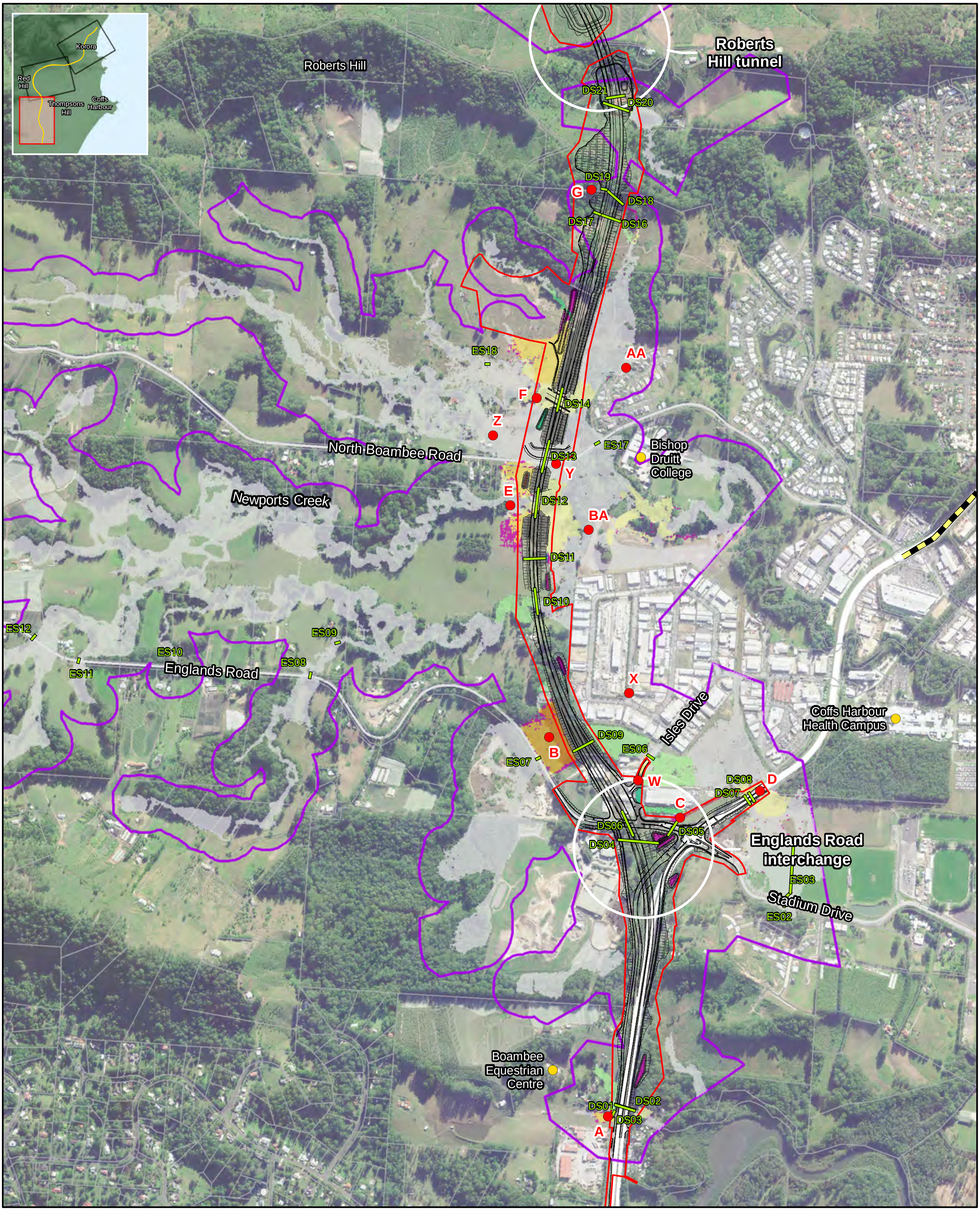
Coffs Harbour Bypass
Northern Creek ancillary sites peak flood inundation
C3

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N

Appendix D

Developed flood maps

D1 Peak flood level difference



Legend

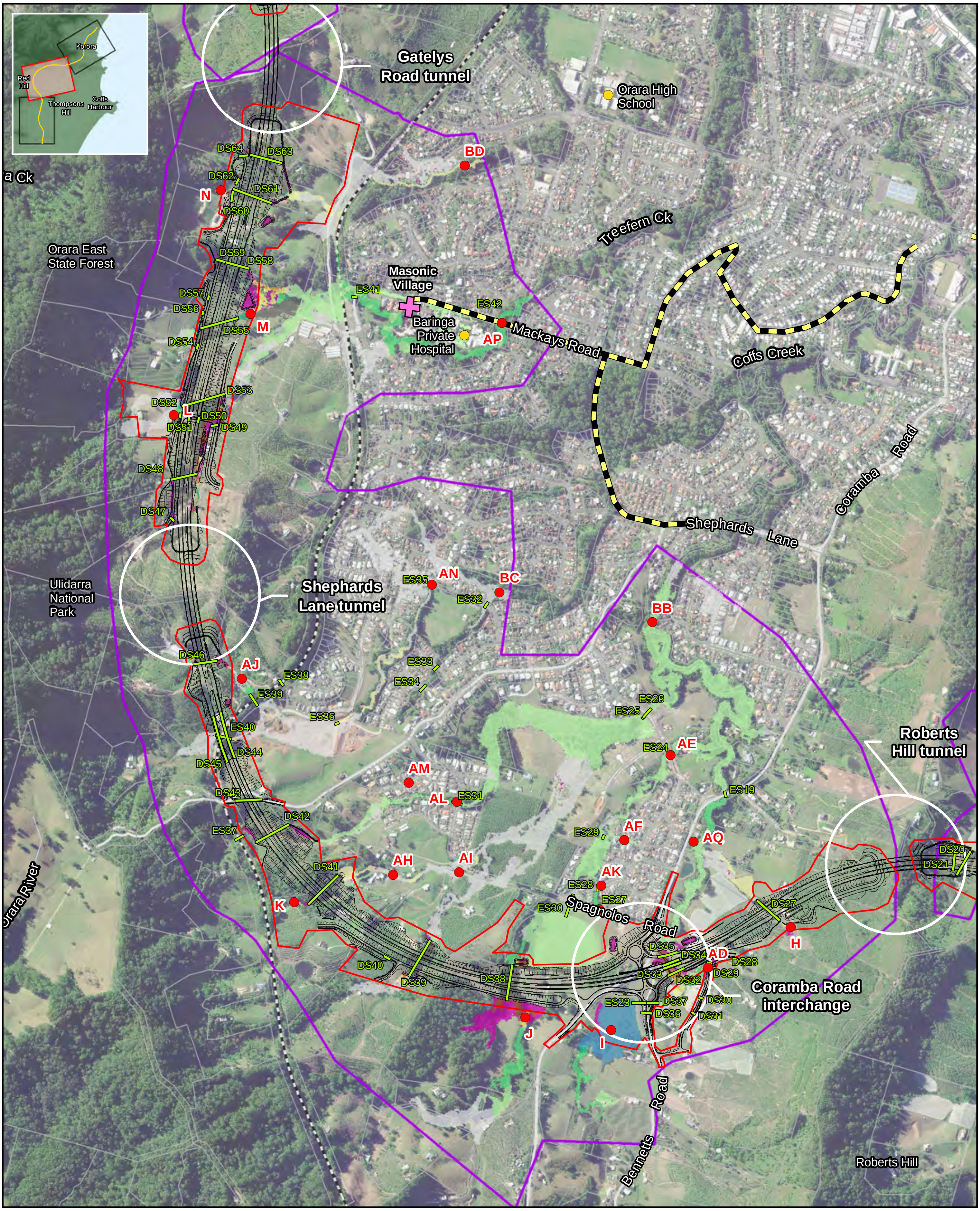
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

Peak level impact (m)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
North Boambee Valley 18 % AEP peak flood level difference
D1.1.1

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

Peak level impact (m)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
Coffs Creek 18 % AEP peak flood level difference
D1.2.1

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

Legend

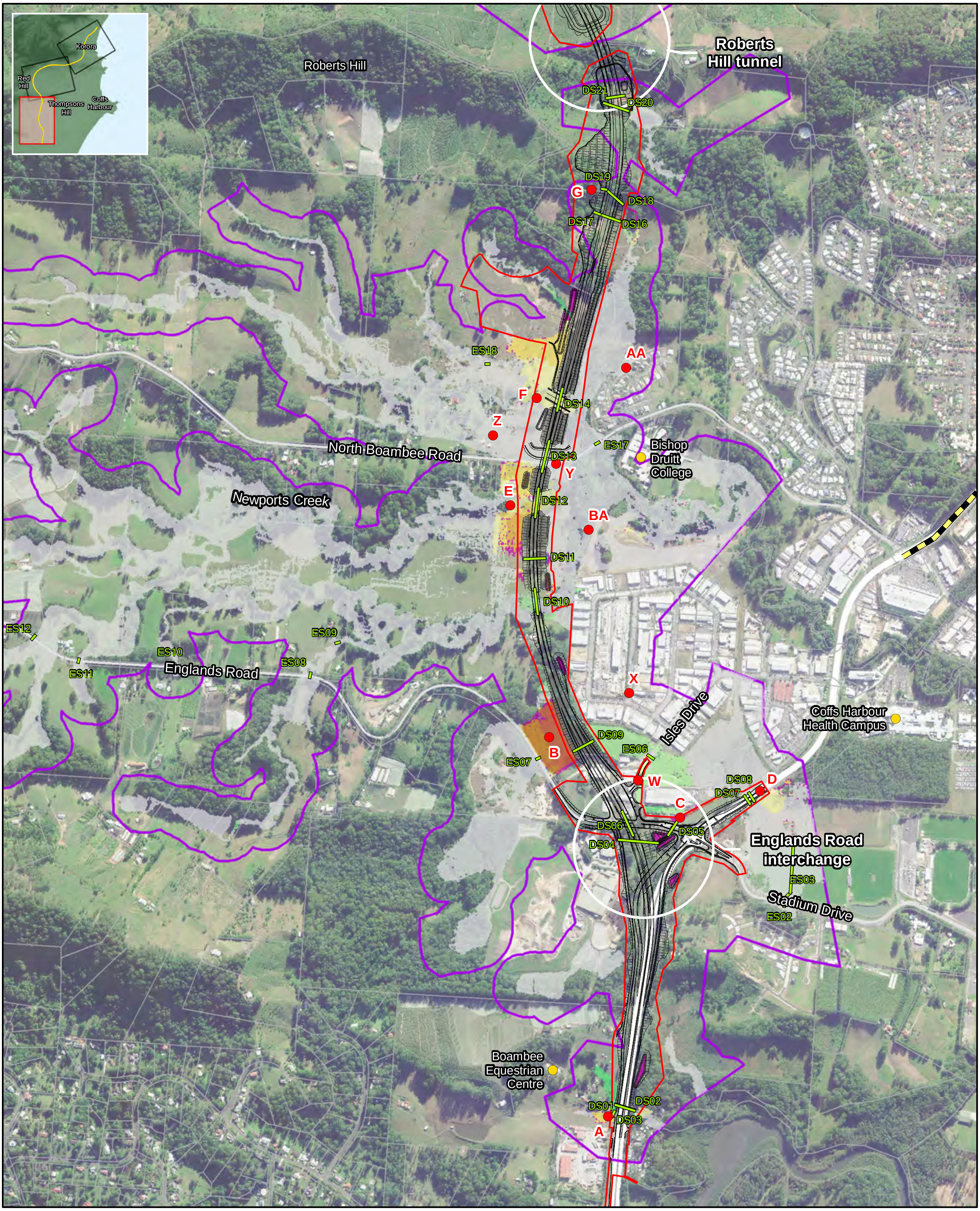
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

Peak level impact (m)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
Northern Creek 18 % AEP peak flood level difference
D1.3.1

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

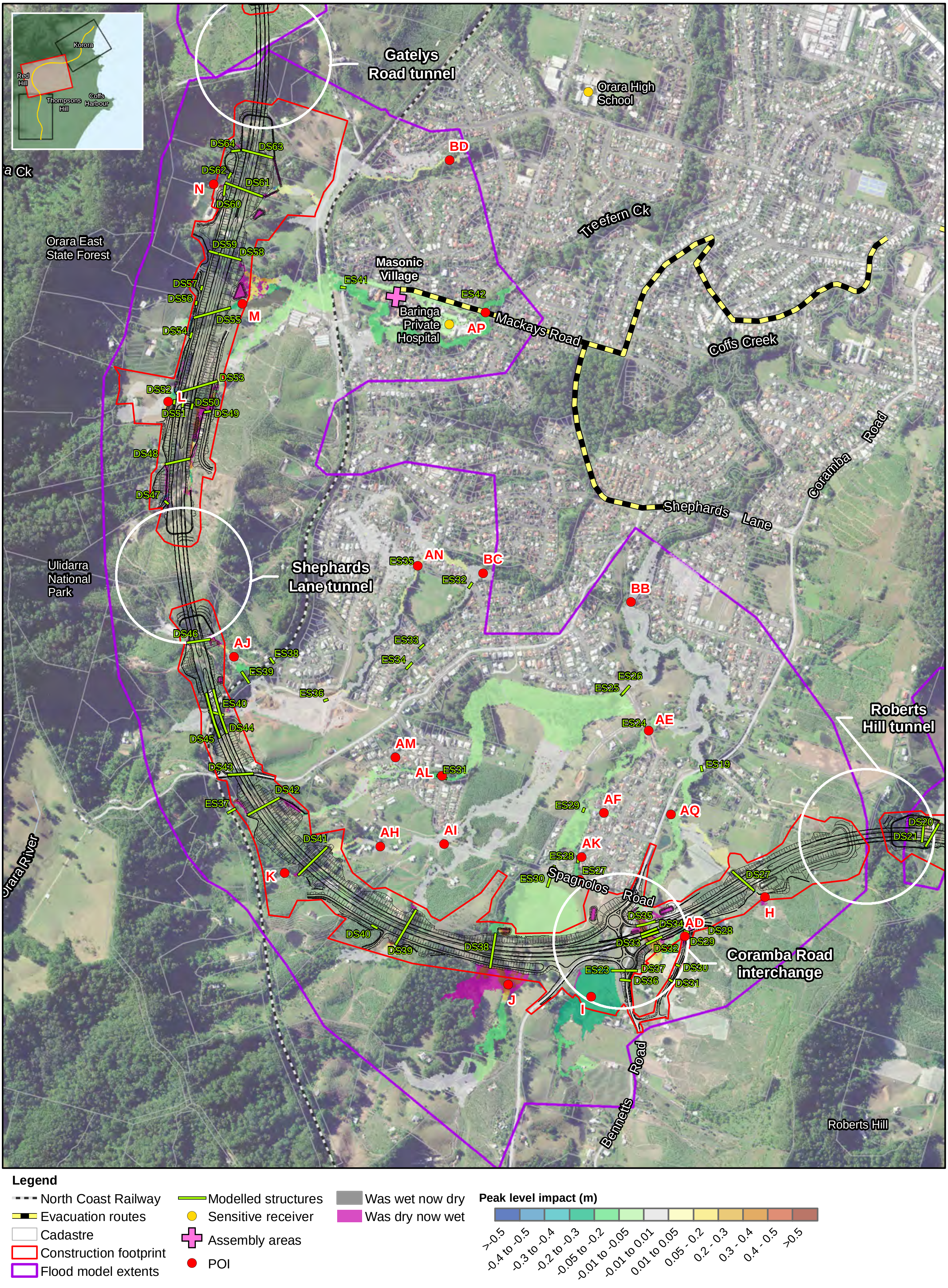
North Coast Railway
Evacuation routes
Cadastre
Construction footprint
Flood model extents
Modelled structures
Sensitive receiver
Assembly areas
POI
Was wet now dry
Was dry now wet

Peak level impact (m)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
North Boambee Valley 10 % AEP peak flood level difference
D1.1.2

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Coffs Harbour Bypass
Coffs Creek 10 % AEP peak flood level difference
D1.2.2

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

Legend

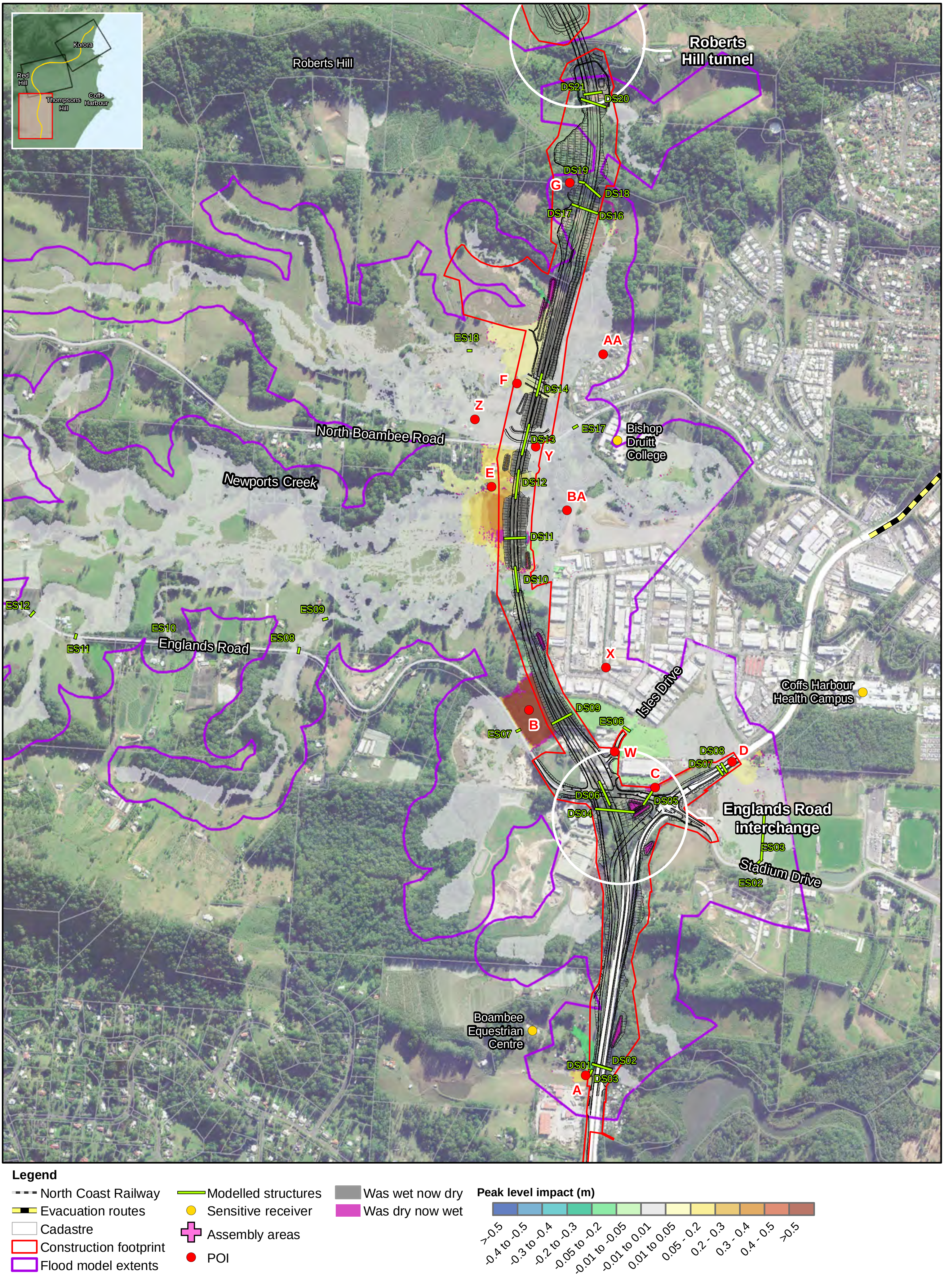
North Coast Railway
Evacuation routes
Cadastre
Construction footprint
Flood model extents
Modelled structures
Sensitive receiver
Assembly areas
POI
Was wet now dry
Was dry now wet

Peak level impact (m)

>0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

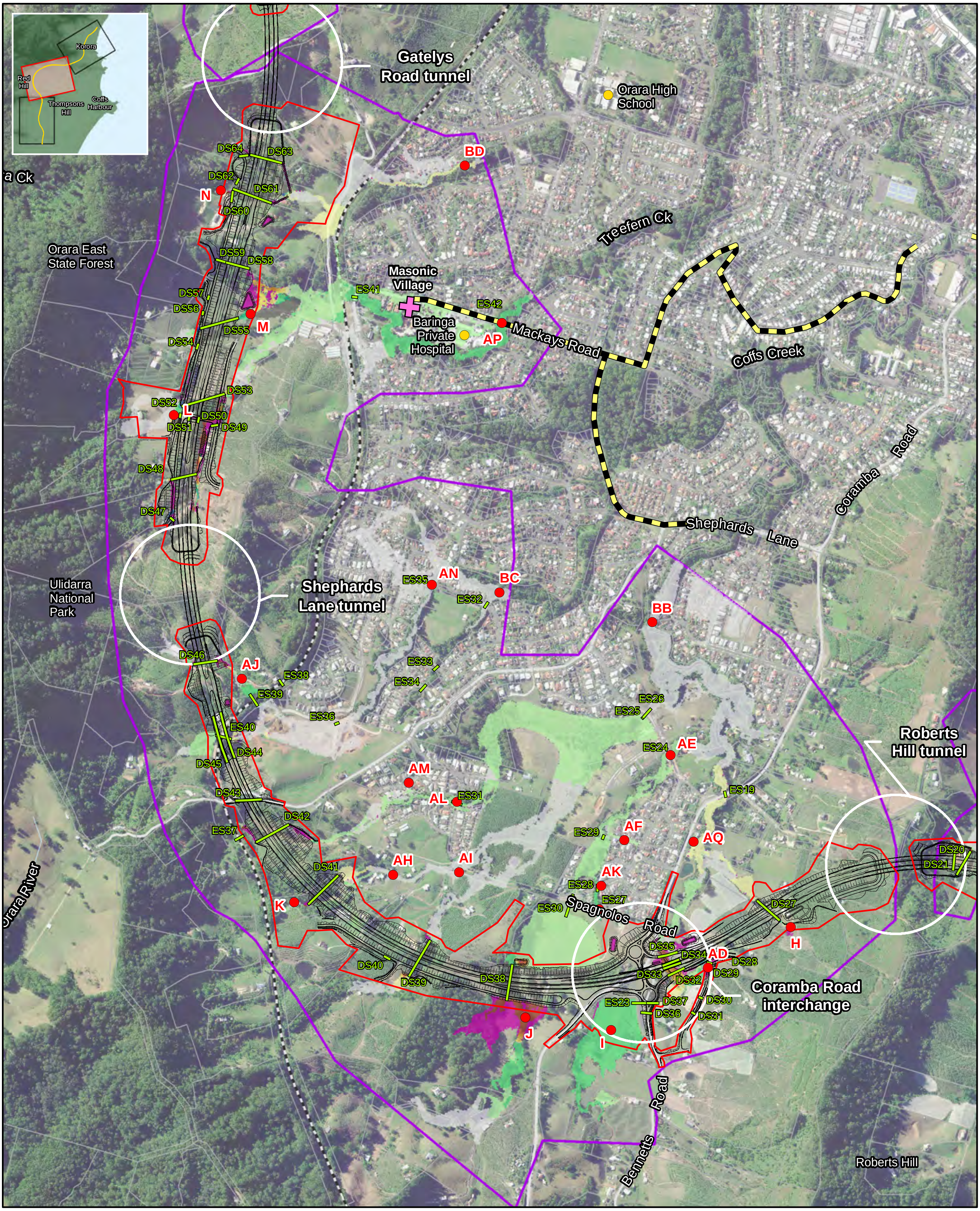
Coffs Harbour Bypass
Northern Creek 10 % AEP peak flood level difference
D1.3.2

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56



Coffs Harbour Bypass
North Boambee Valley 5 % AEP peak flood level difference
D1.1.3

Scale @A4: 1:17,500
GDA 1994 MGA Zone 56



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

Peak level impact (m)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
Coffs Creek 5 % AEP peak flood level difference
D1.2.3

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N

Legend

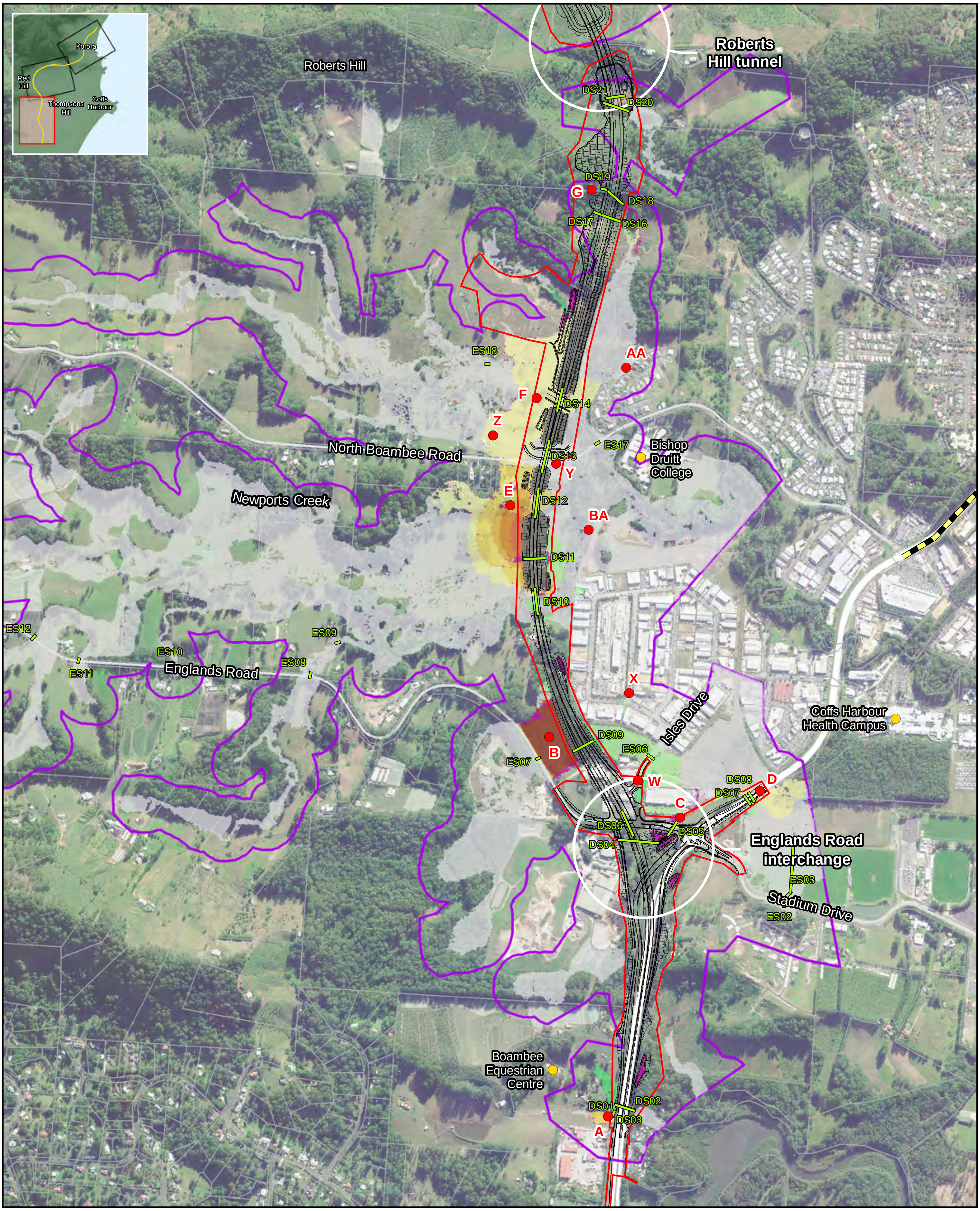
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

Peak level impact (m)

>-0.5 | -0.4 to -0.5 | -0.3 to -0.4 | -0.2 to -0.3 | -0.05 to -0.2 | -0.01 to -0.05 | -0.01 to 0.01 | 0.01 to 0.05 | 0.05 - 0.2 | 0.2 - 0.3 | 0.3 - 0.4 | 0.4 - 0.5 | >0.5

Coffs Harbour Bypass
Northern Creek 5 % AEP peak flood level difference
D1.3.3

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

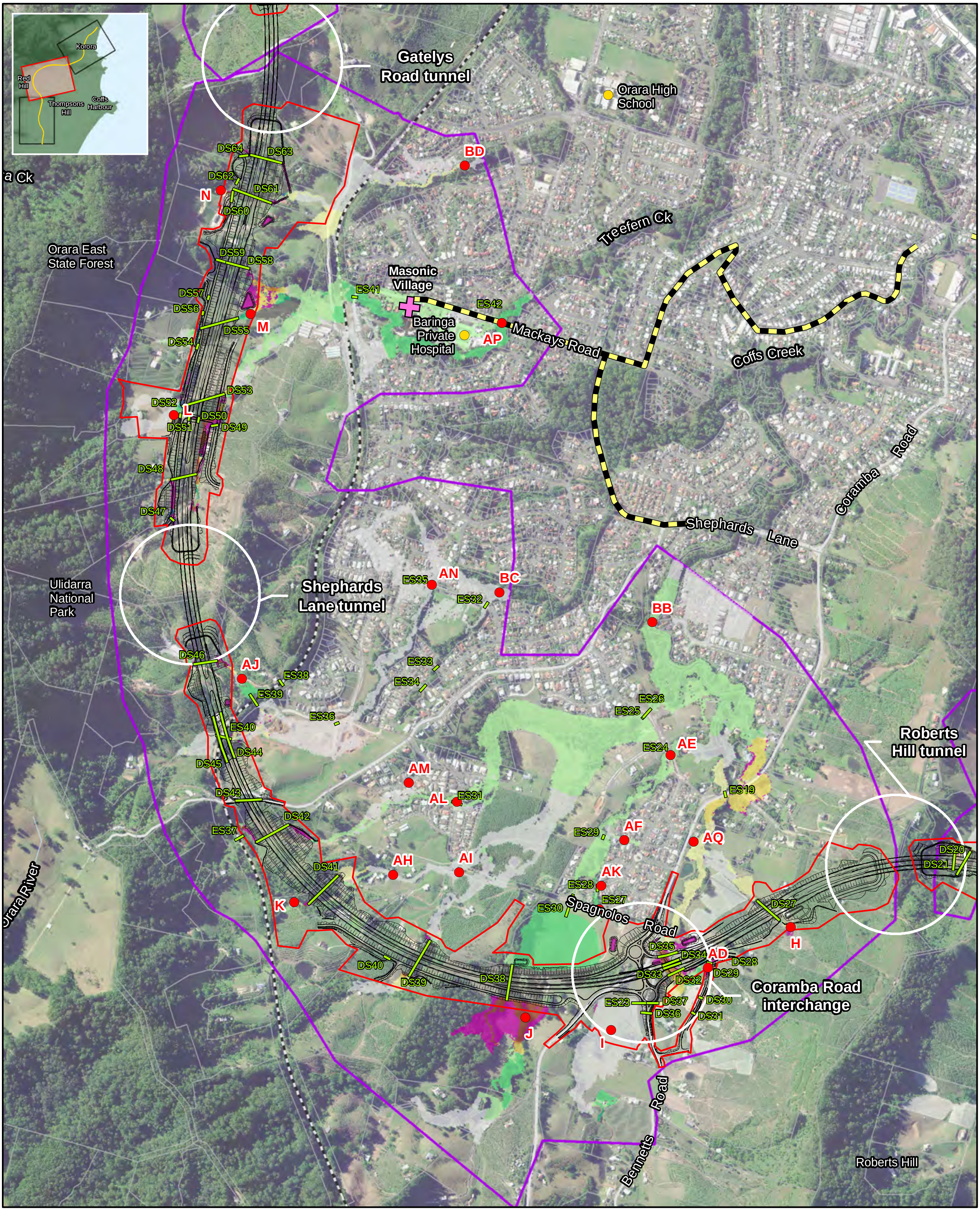
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

Peak level impact (m)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
North Boambee Valley 2 % AEP peak flood level difference
D1.1.4

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

Peak level impact (m)

>0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

0 0.15 0.3 0.45 km

N

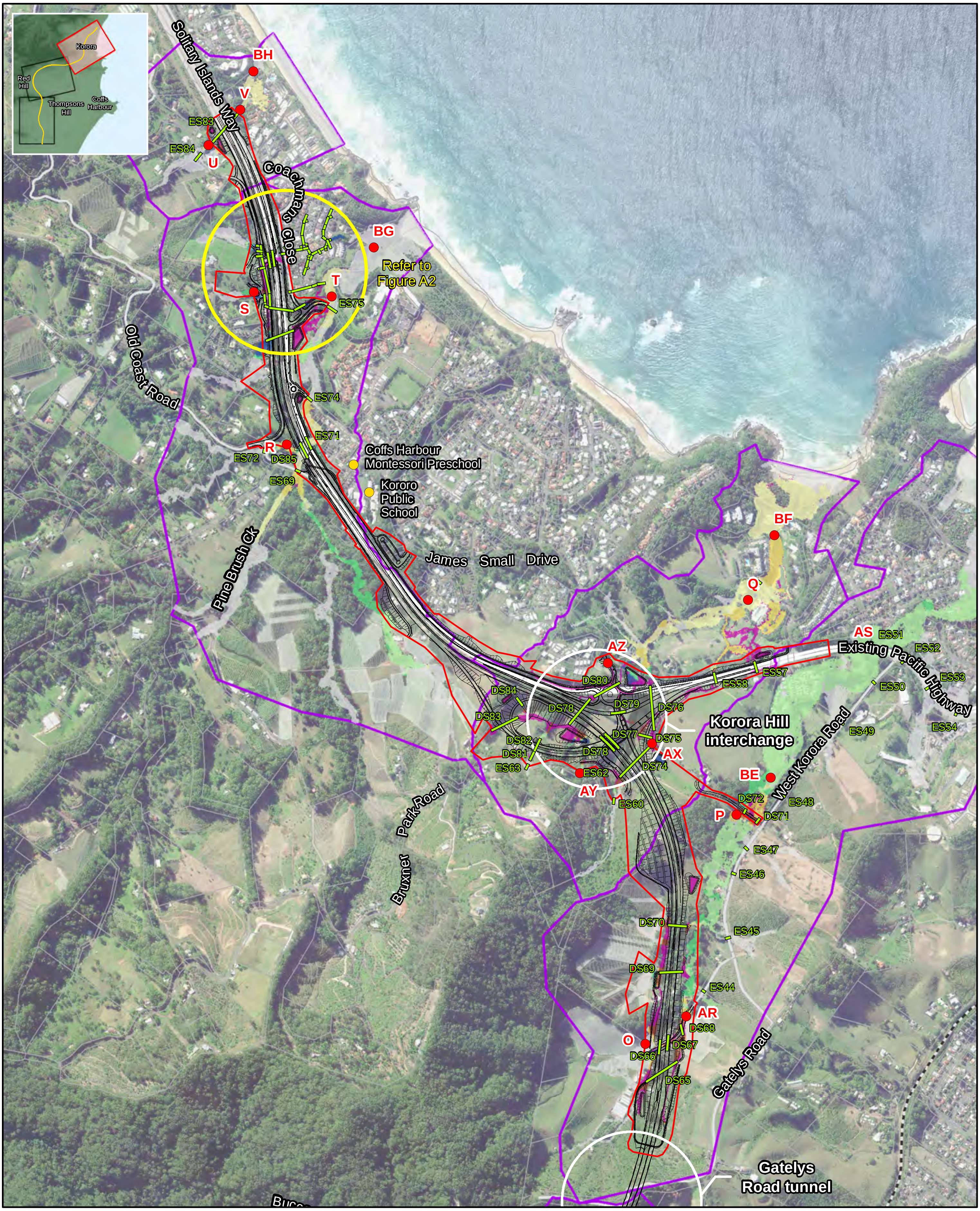
Scale @A4: 1:17,500

GDA 1994 MGA Zone 56

Coffs Harbour Bypass

Coffs Creek 2 % AEP peak flood level difference

D1.2.4



Legend

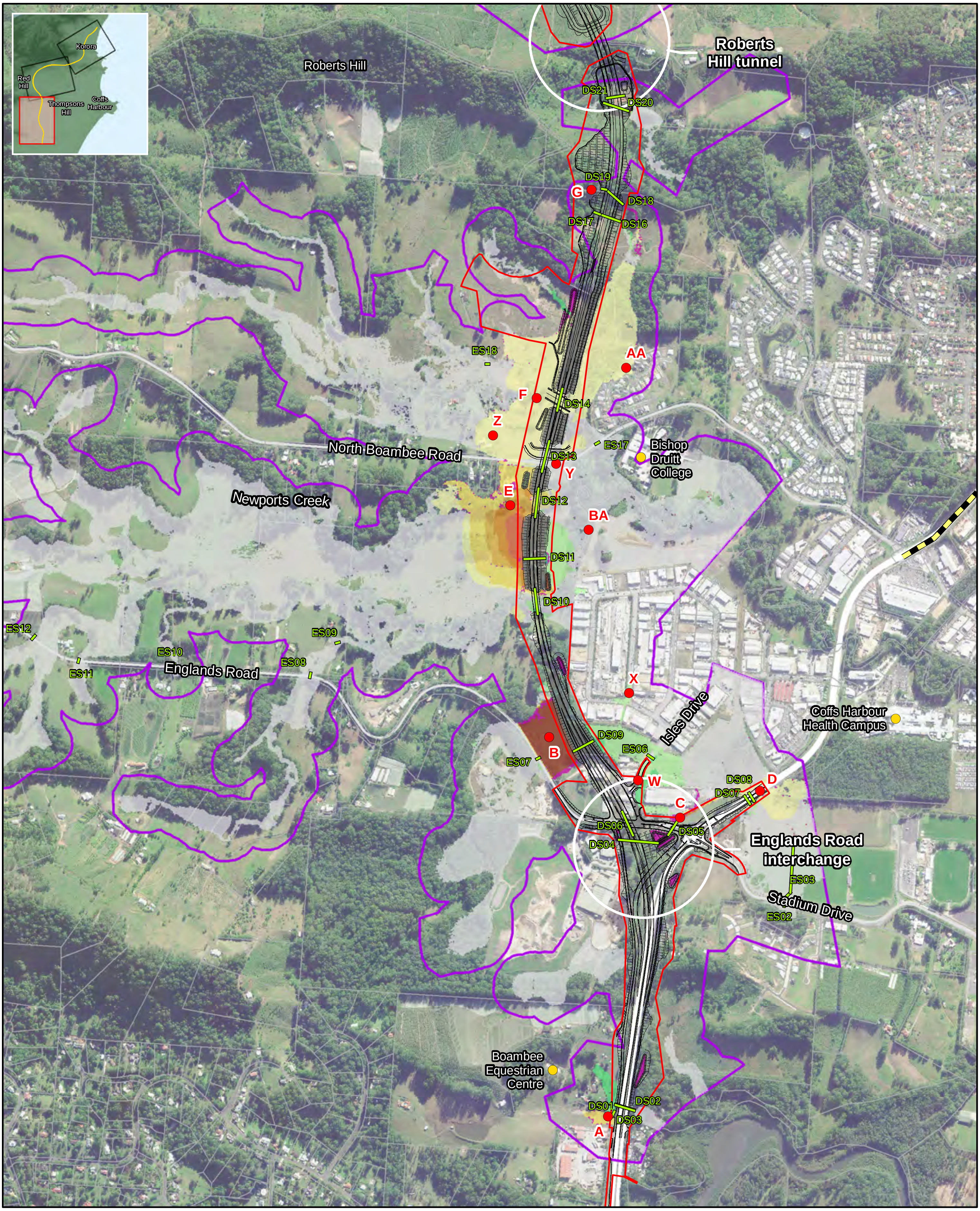
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

Peak level impact (m)

>-0.5 | -0.4 to -0.5 | -0.3 to -0.4 | -0.2 to -0.3 | -0.05 to -0.2 | -0.01 to -0.05 | -0.01 to 0.01 | 0.01 to 0.05 | 0.05 - 0.2 | 0.2 - 0.3 | 0.3 - 0.4 | 0.4 - 0.5 | >0.5

Coffs Harbour Bypass
Northern Creek 2 % AEP peak flood level difference
D1.3.4

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

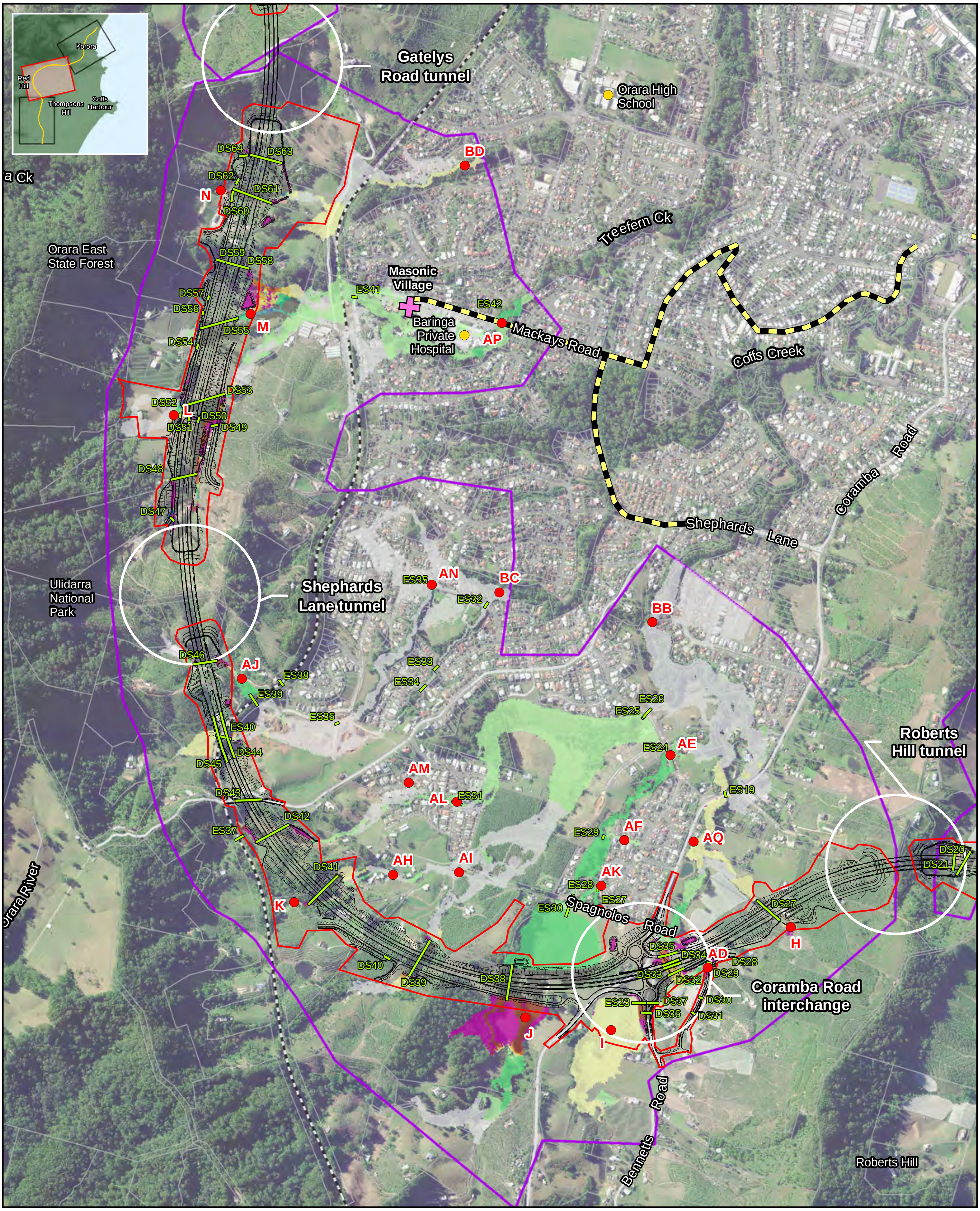
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

Peak level impact (m)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
North Boambee Valley 1 % AEP peak flood level difference
D1.1.5

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

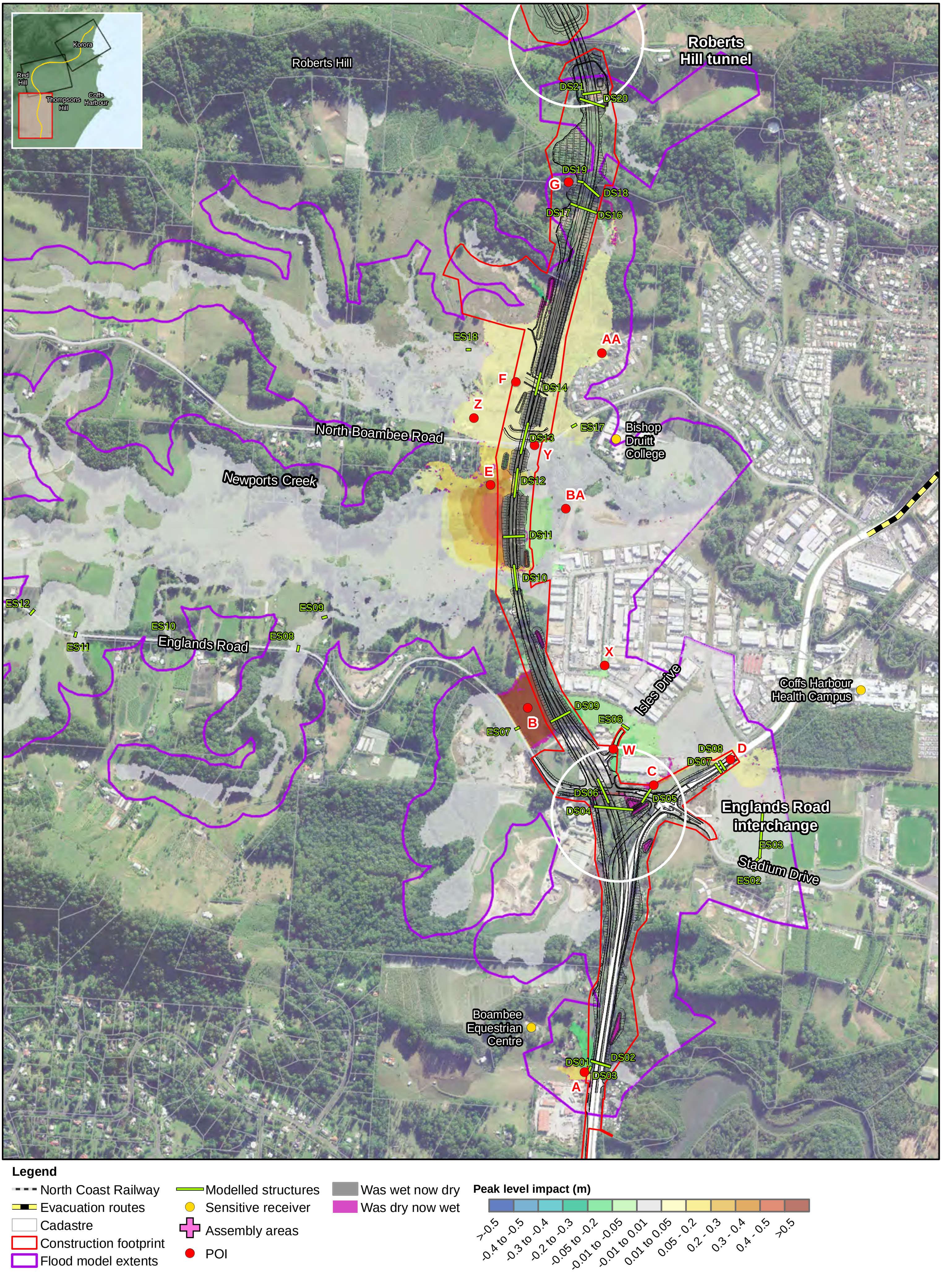
Peak level impact (m)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

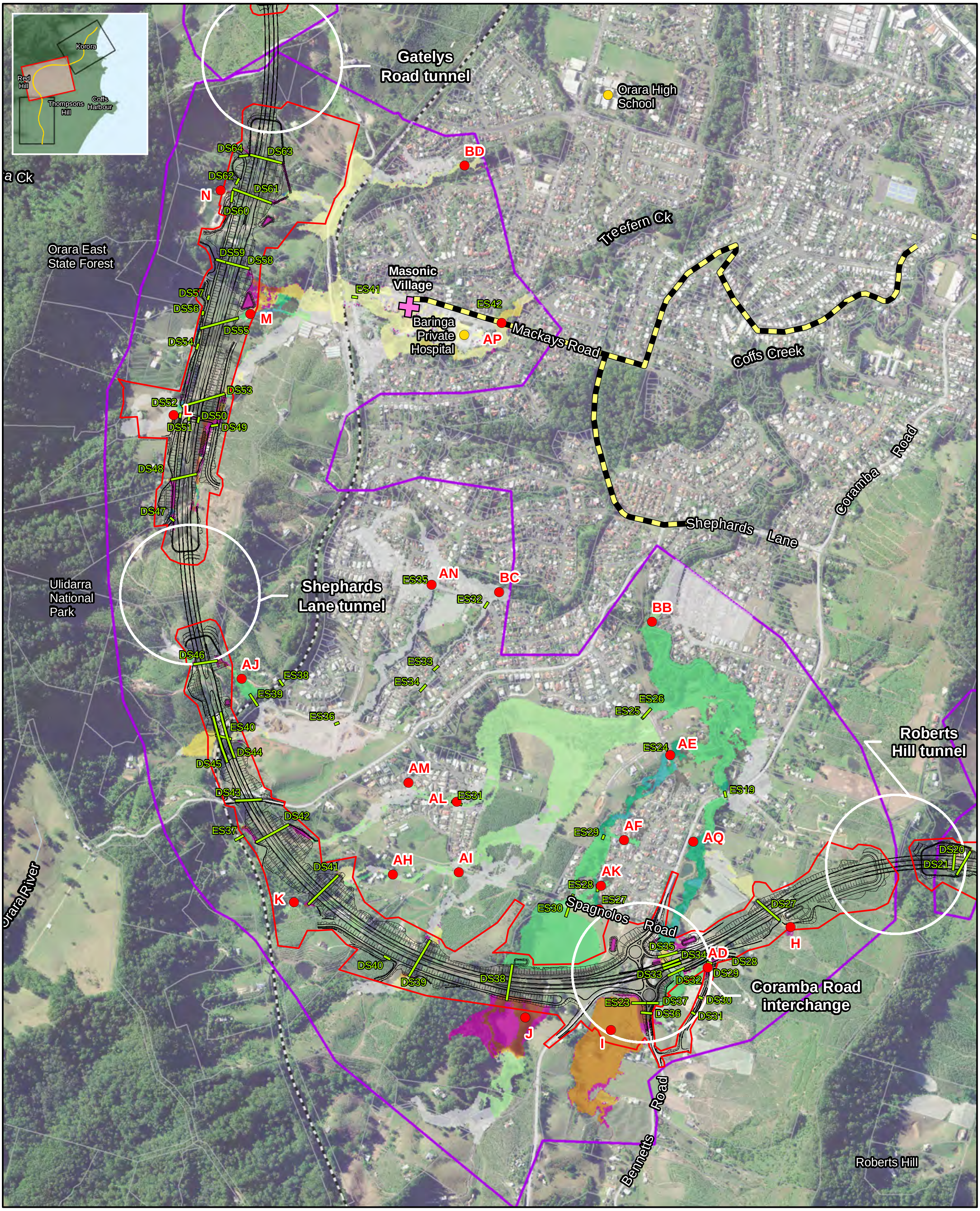
Coffs Harbour Bypass
Coffs Creek 1 % AEP peak flood level difference
D1.2.5

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

Coffs Harbour Bypass
Northern Creek 1 % AEP peak flood level difference
D1.3.5



Coffs Harbour Bypass
North Boambee Valley 1 % AEP 2050 climate peak flood level difference
D1.1.6



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

Peak level impact (m)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass

Coffs Creek 1 % AEP 2050 climate peak flood level difference

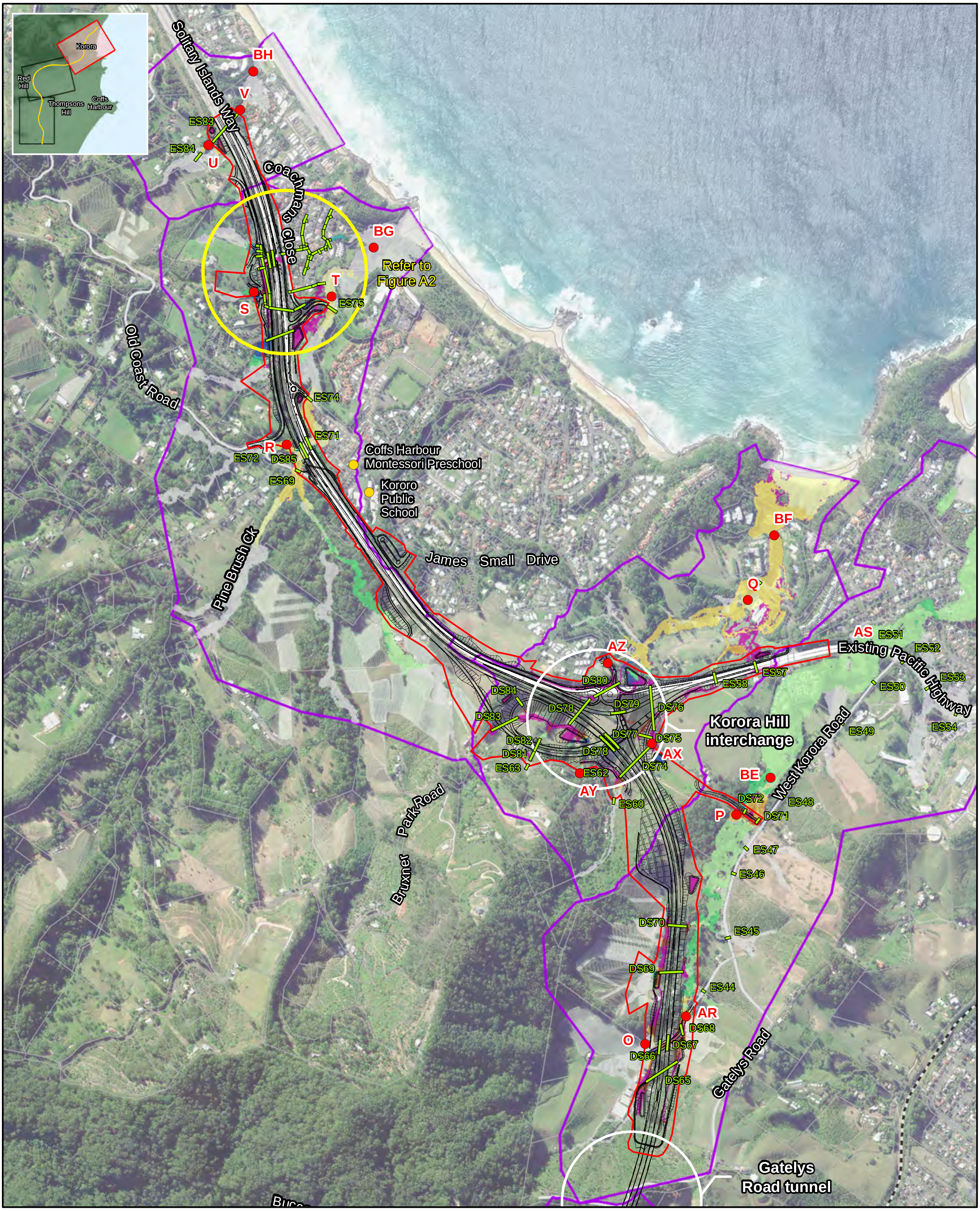
D1.2.6

0 0.15 0.3 0.45 km

Scale @A4: 1:17,500

GDA 1994 MGA Zone 56

N



Legend

North Coast Railway
Evacuation routes
Cadastre
Construction footprint
Flood model extents
Modelled structures
Sensitive receiver
Assembly areas
POI
Was wet now dry
Was dry now wet

Peak level impact (m)

>-0.5
-0.4 to -0.5
-0.3 to -0.4
-0.2 to -0.3
-0.05 to -0.2
-0.01 to -0.05
-0.01 to 0.01
0.01 to 0.05
0.05 - 0.2
0.2 - 0.3
0.3 - 0.4
0.4 - 0.5
>0.5

Coffs Harbour Bypass
Northern Creek 1 % AEP 2050 climate peak flood level difference
D1.3.6

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N

Coffs Harbour Bypass

North Boambee Valley 1 % AEP 2100 climate peak flood level difference

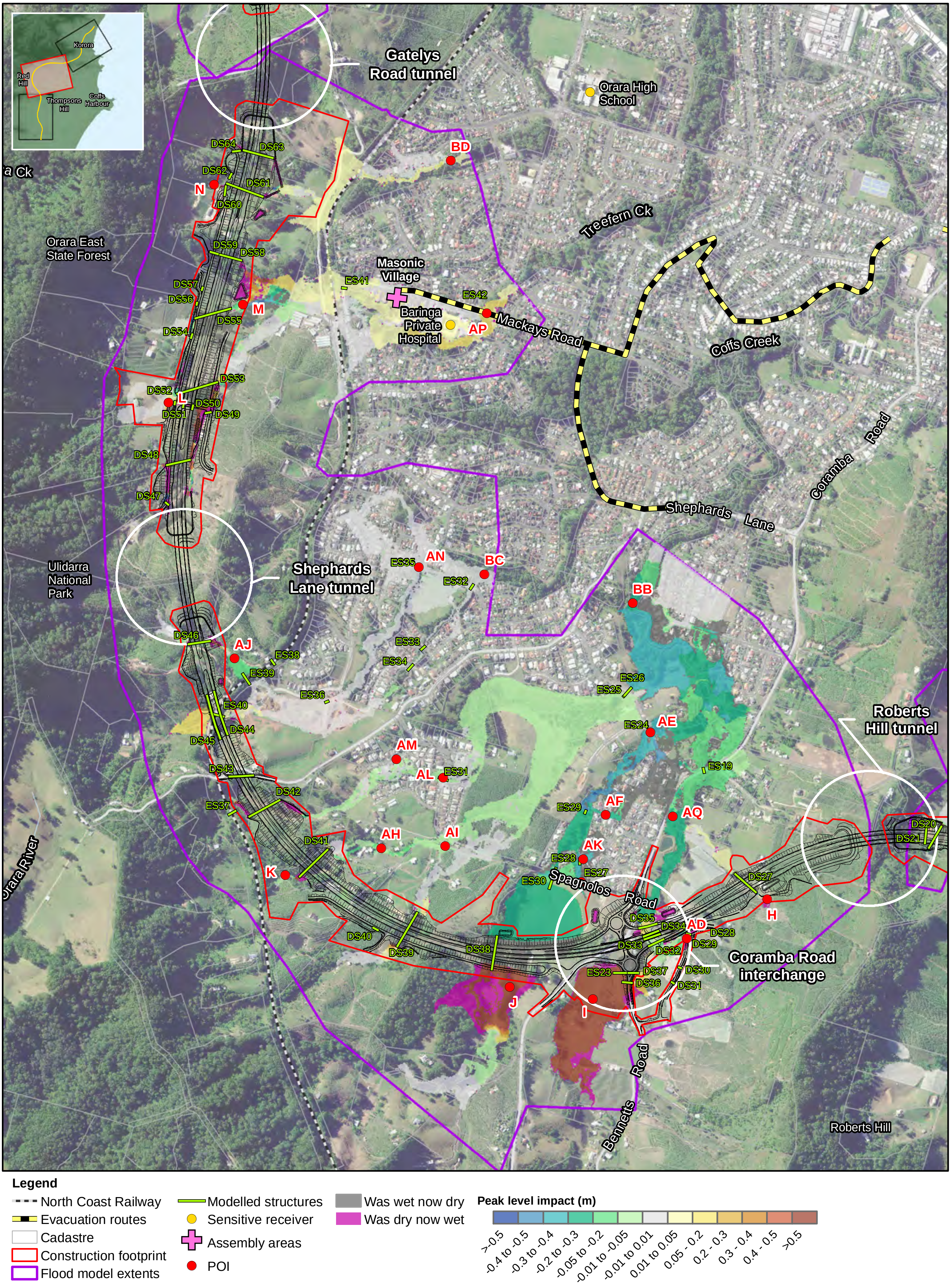
D1.1.7

0 0.15 0.3 0.45 km

Scale @A4: 1:17,500

GDA 1994 MGA Zone 56

N



Coffs Harbour Bypass
Coffs Creek 1 % AEP 2100 climate peak flood level difference
D1.2.7

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

Legend

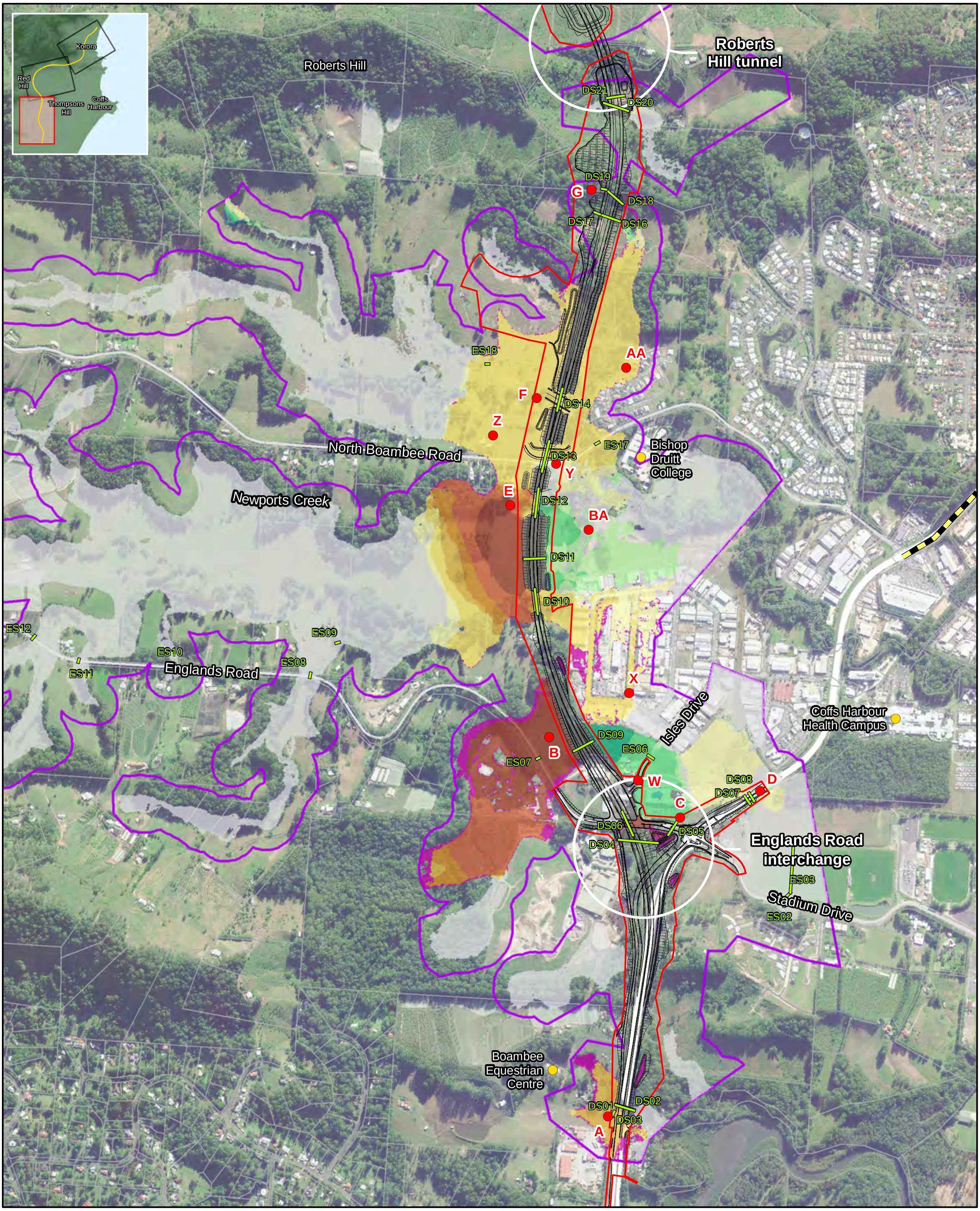
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

Peak level impact (m)

>-0.5 | -0.4 to -0.5 | -0.3 to -0.4 | -0.2 to -0.3 | -0.05 to -0.2 | -0.01 to -0.05 | -0.01 to 0.01 | 0.01 to 0.05 | 0.05 - 0.2 | 0.2 - 0.3 | 0.3 - 0.4 | 0.4 - 0.5 | >0.5

Coffs Harbour Bypass
Northern Creek 1 % AEP 2100 climate peak flood level difference
D1.3.7

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

Peak level impact (m)

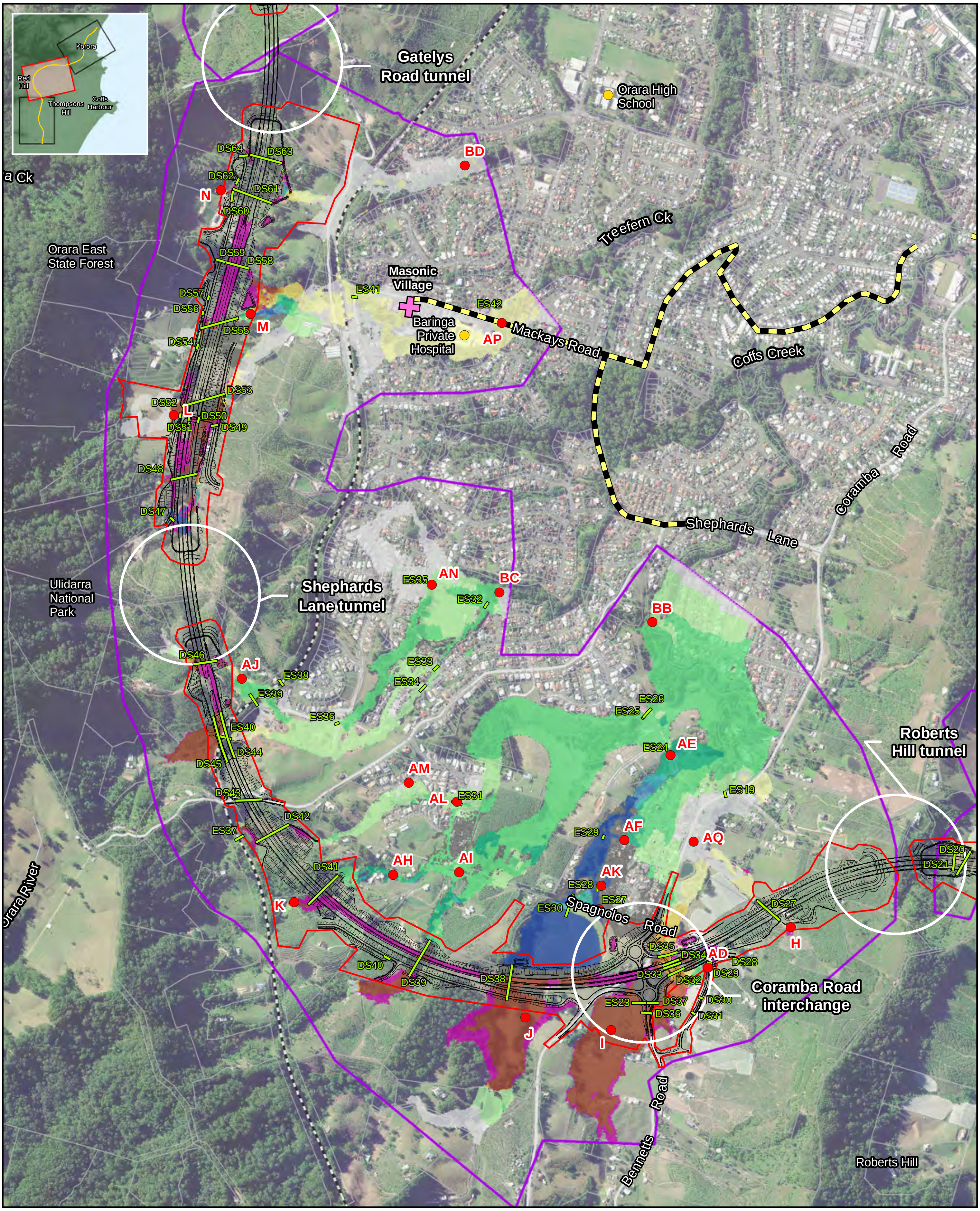
>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

0 0.15 0.3 0.45 km

Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N

Coffs Harbour Bypass
North Boambee Valley PMF peak flood level difference
D1.1.8



Legend

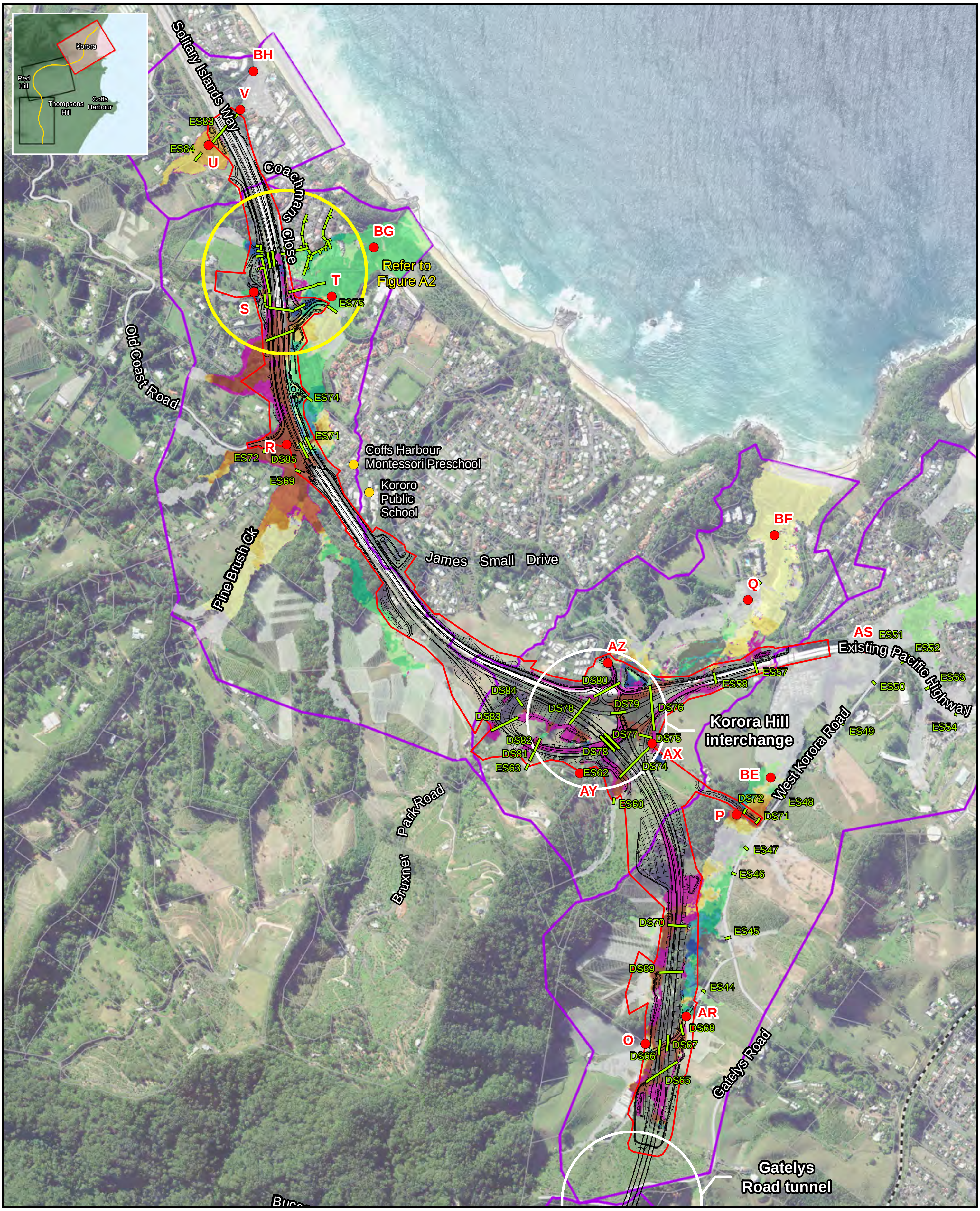
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

Peak level impact (m)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
Coffs Creek PMF peak flood level difference
D1.2.8

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

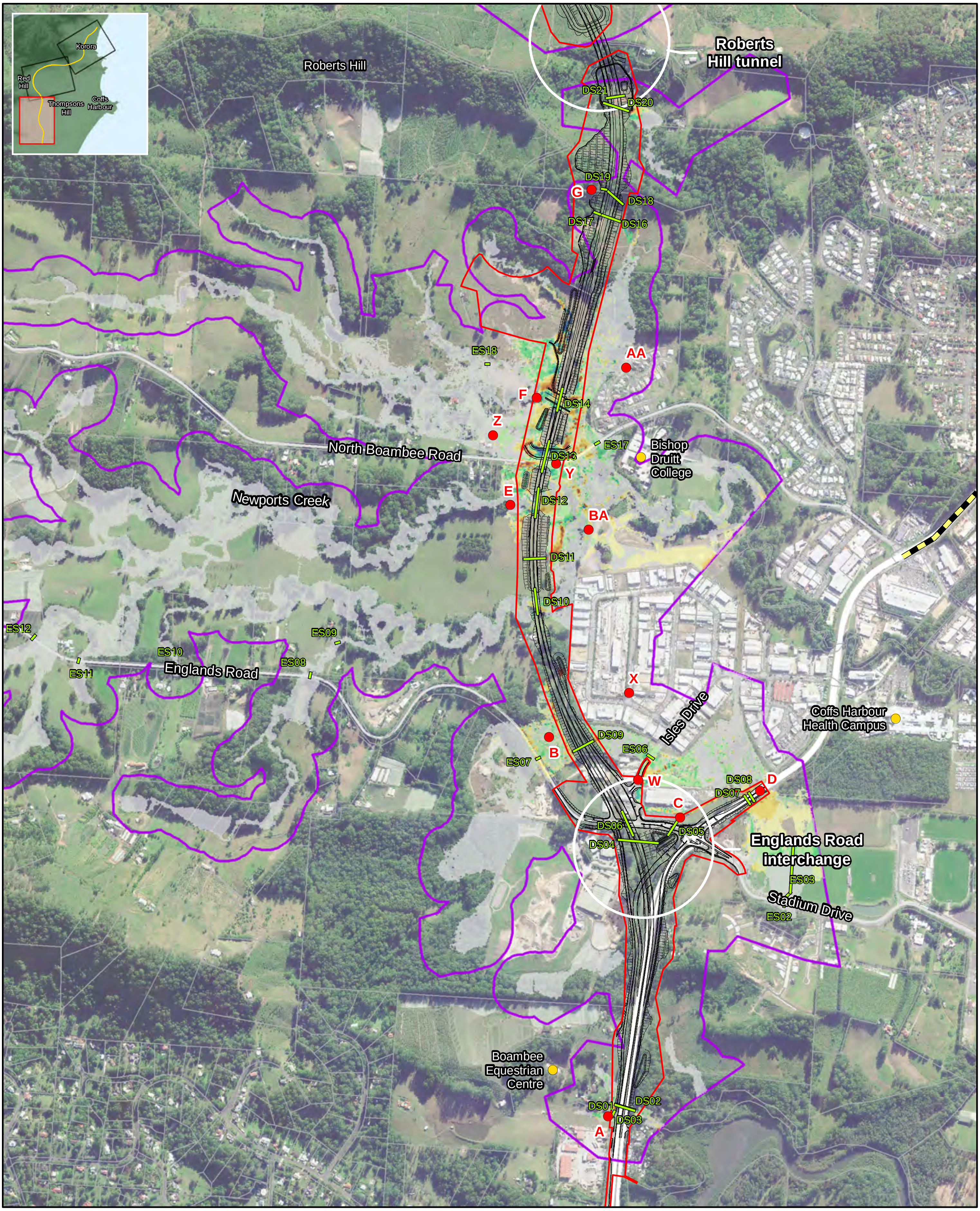
Peak level impact (m)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
Northern Creek PMF peak flood level difference
D1.3.8

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

D2 Peak flood velocity difference



Legend

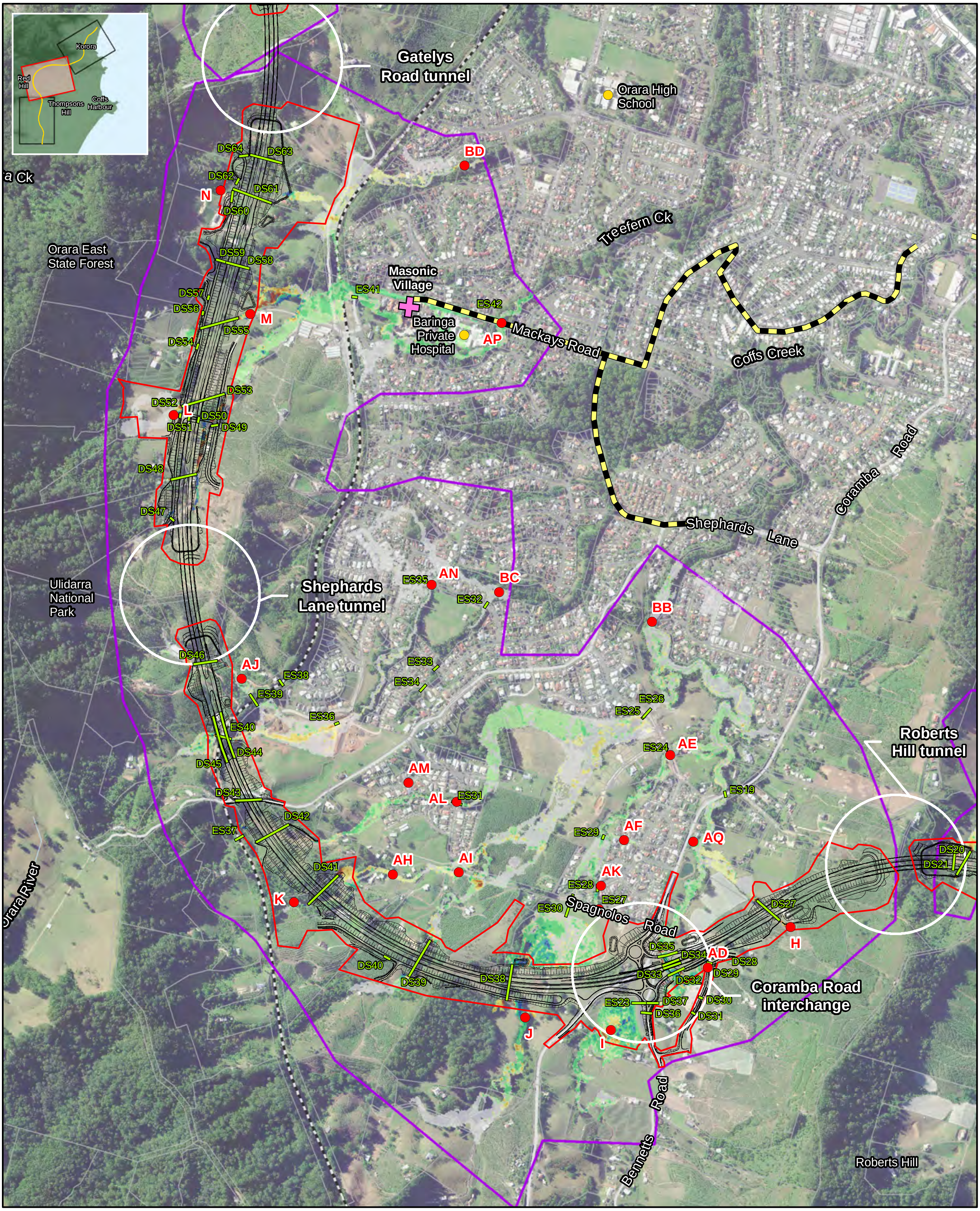
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
North Boambee Valley 18 % AEP peak flood velocity difference
D2.1.1

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

North Coast Railway
Evacuation routes
Cadastre
Construction footprint
Flood model extents
Modelled structures
Sensitive receiver
Assembly areas
POI

Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
Coffs Creek 18 % AEP peak flood velocity difference
D2.2.1

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N

Legend

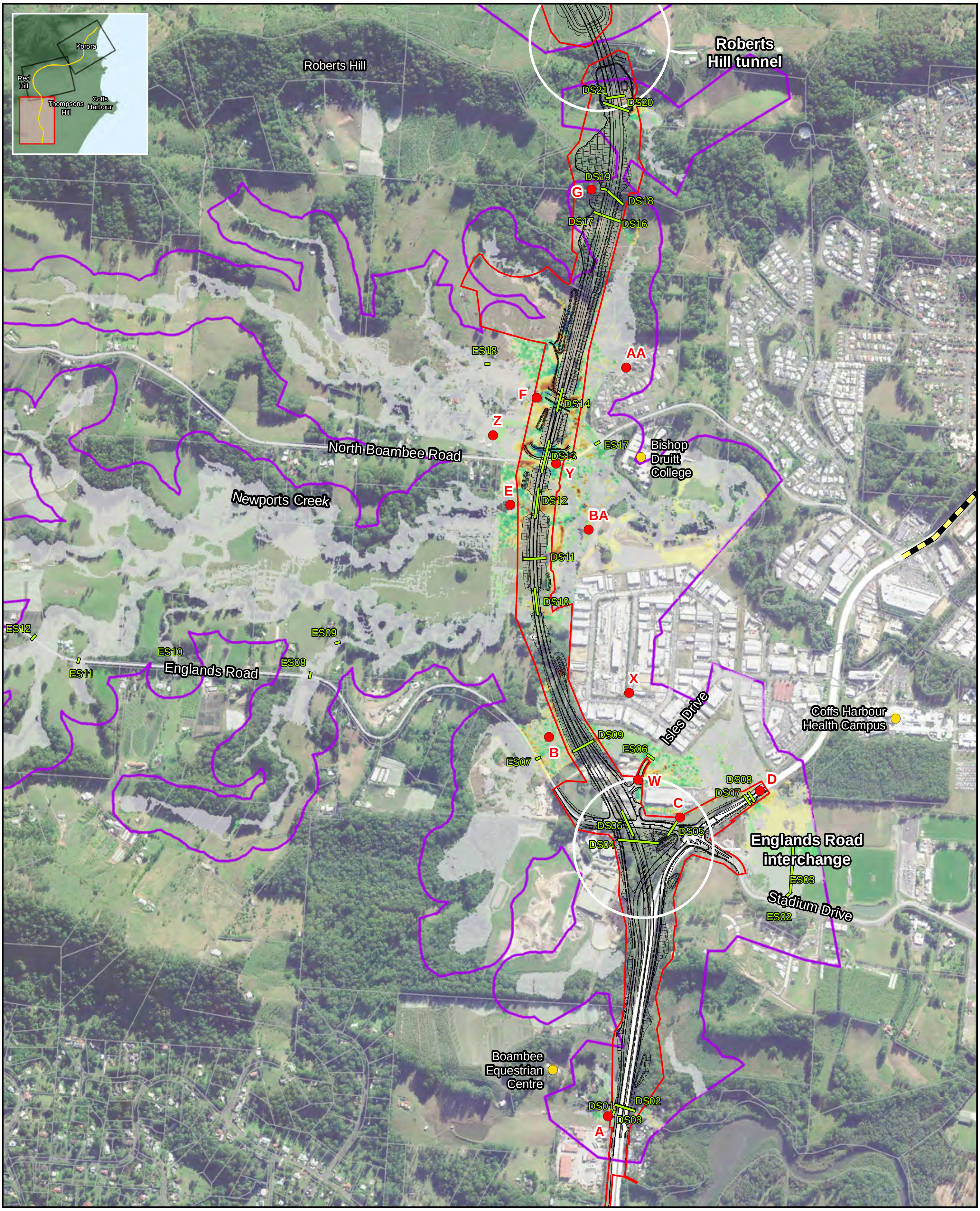
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
Northern Creek 18 % AEP peak flood velocity difference
D2.3.1

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

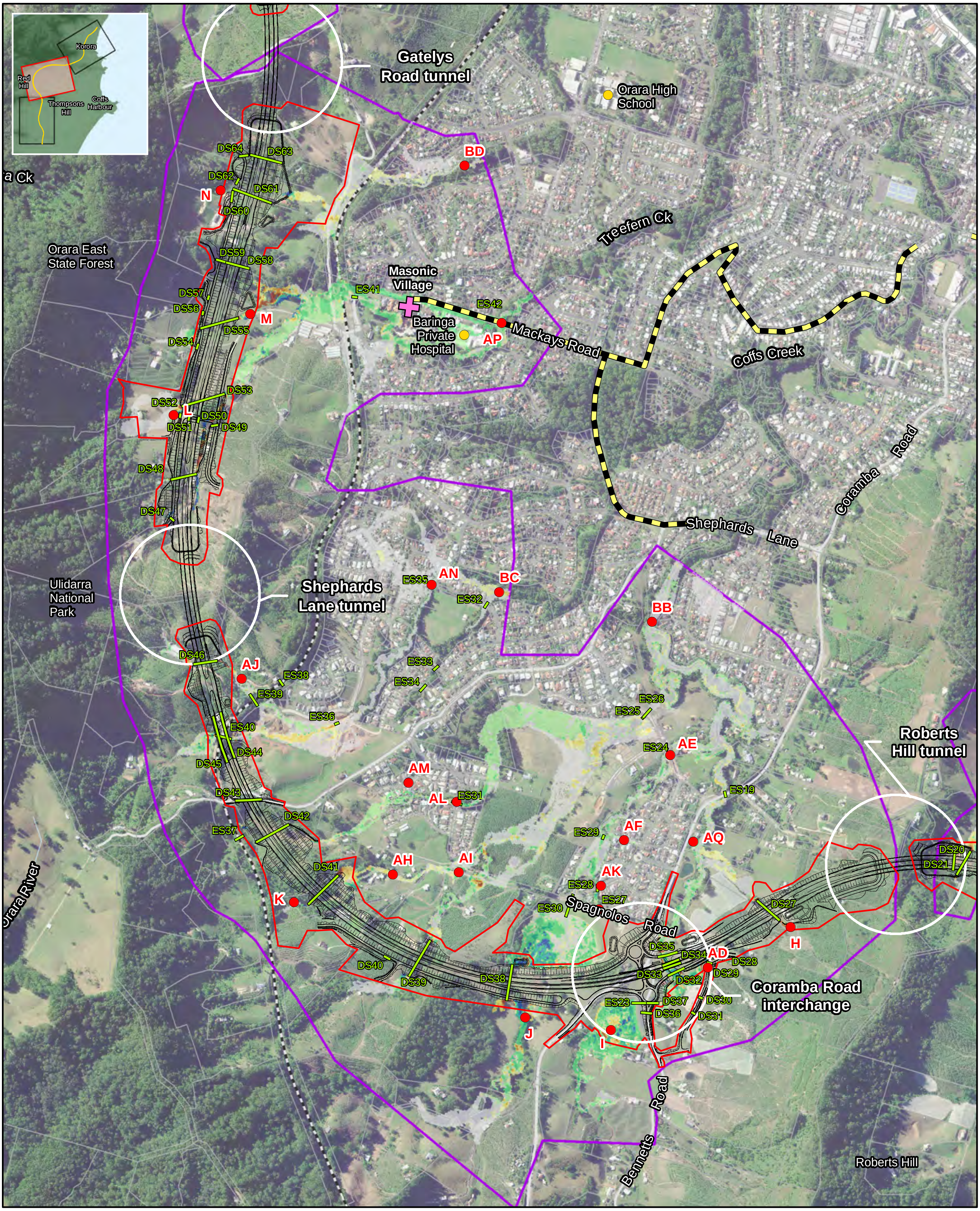
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
North Boambee Valley 10 % AEP peak flood velocity difference
D2.1.2

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

Peak velocity impact (m/s)

>-0.5 | -0.4 to -0.5 | -0.3 to -0.4 | -0.2 to -0.3 | -0.05 to -0.2 | -0.01 to -0.05 | -0.01 to 0.01 | 0.01 to 0.05 | 0.05 - 0.2 | 0.2 - 0.3 | 0.3 - 0.4 | 0.4 - 0.5 | >0.5

Coffs Harbour Bypass
Coffs Creek 10 % AEP peak flood velocity difference
D2.2.2

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

Legend

North Coast Railway
Evacuation routes
Cadastre
Construction footprint
Flood model extents
Modelled structures
Sensitive receiver
Assembly areas
POI

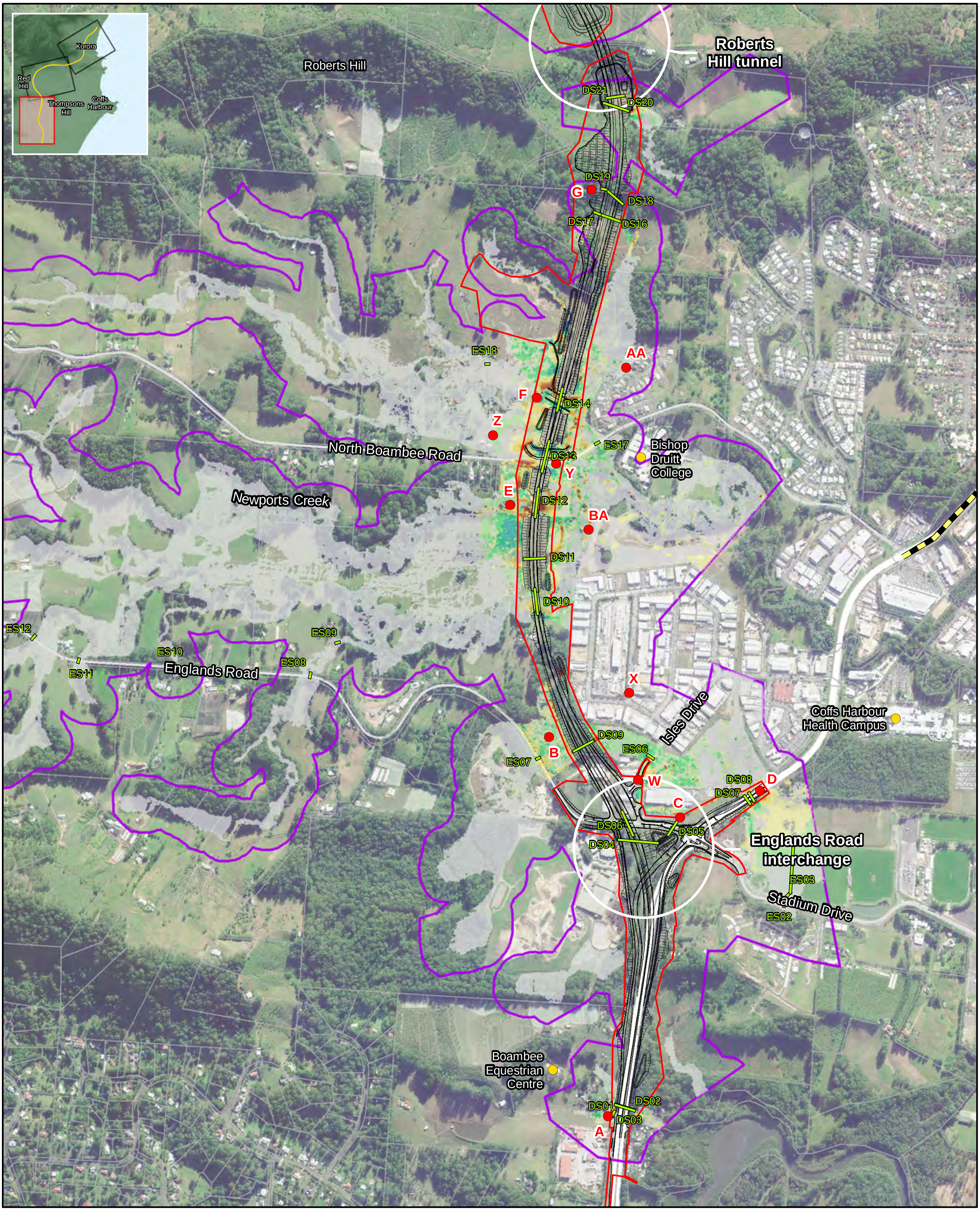
Peak velocity impact (m/s)

>-0.5
-0.4 to -0.5
-0.3 to -0.4
-0.2 to -0.3
-0.05 to -0.2
-0.01 to -0.05
-0.01 to 0.01
0.01 to 0.05
0.05 - 0.2
0.2 - 0.3
0.3 - 0.4
0.4 - 0.5
>0.5

Coffs Harbour Bypass
Northern Creek 10 % AEP peak flood velocity difference
D2.3.2

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N



Legend

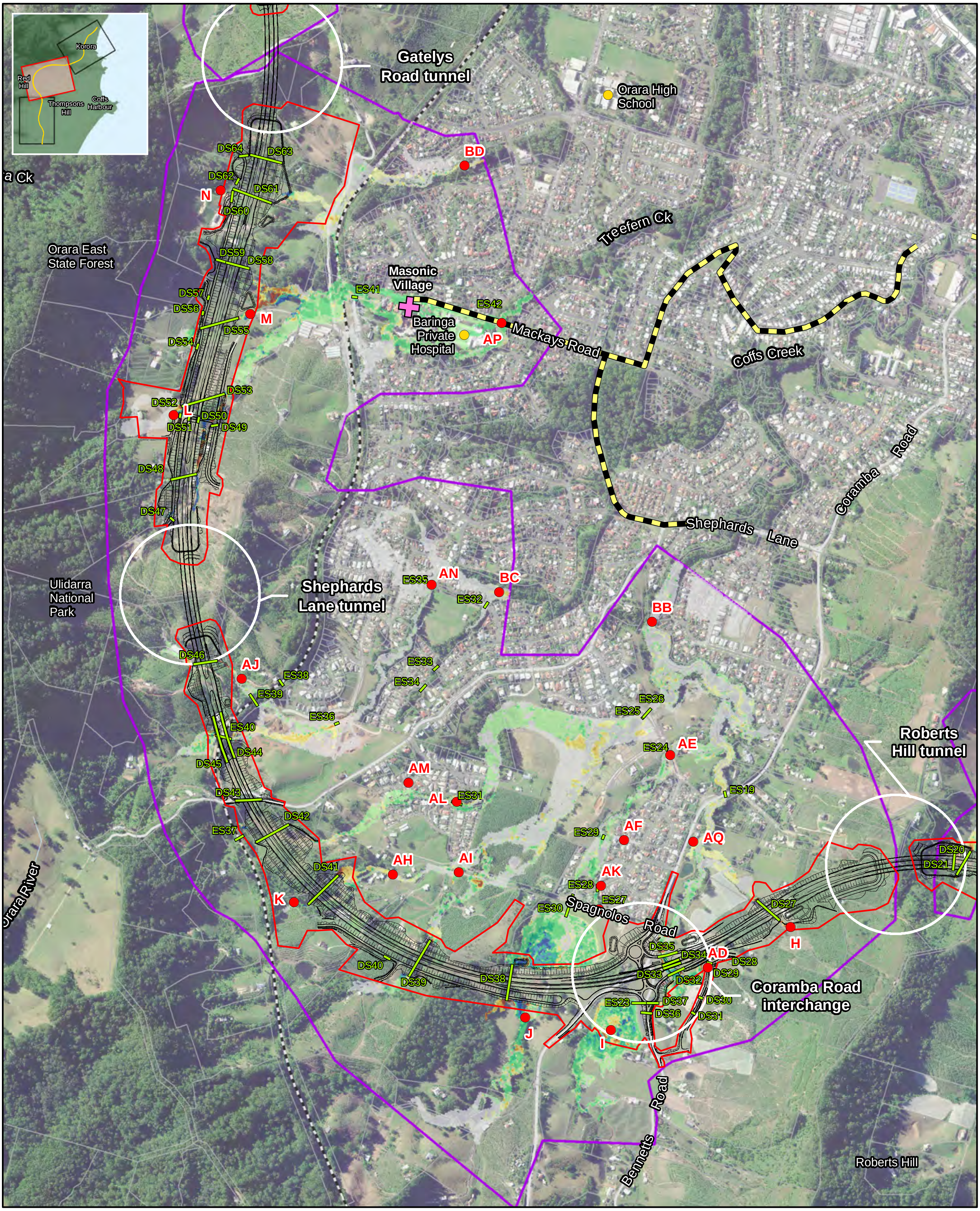
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
North Boambee Valley 5 % AEP peak flood velocity difference
D2.1.3

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

0 0.15 0.3 0.45 km

Scale @A4: 1:17,500

GDA 1994 MGA Zone 56

N

Coffs Harbour Bypass

Coffs Creek 5 % AEP peak flood velocity difference

D2.2.3

Legend

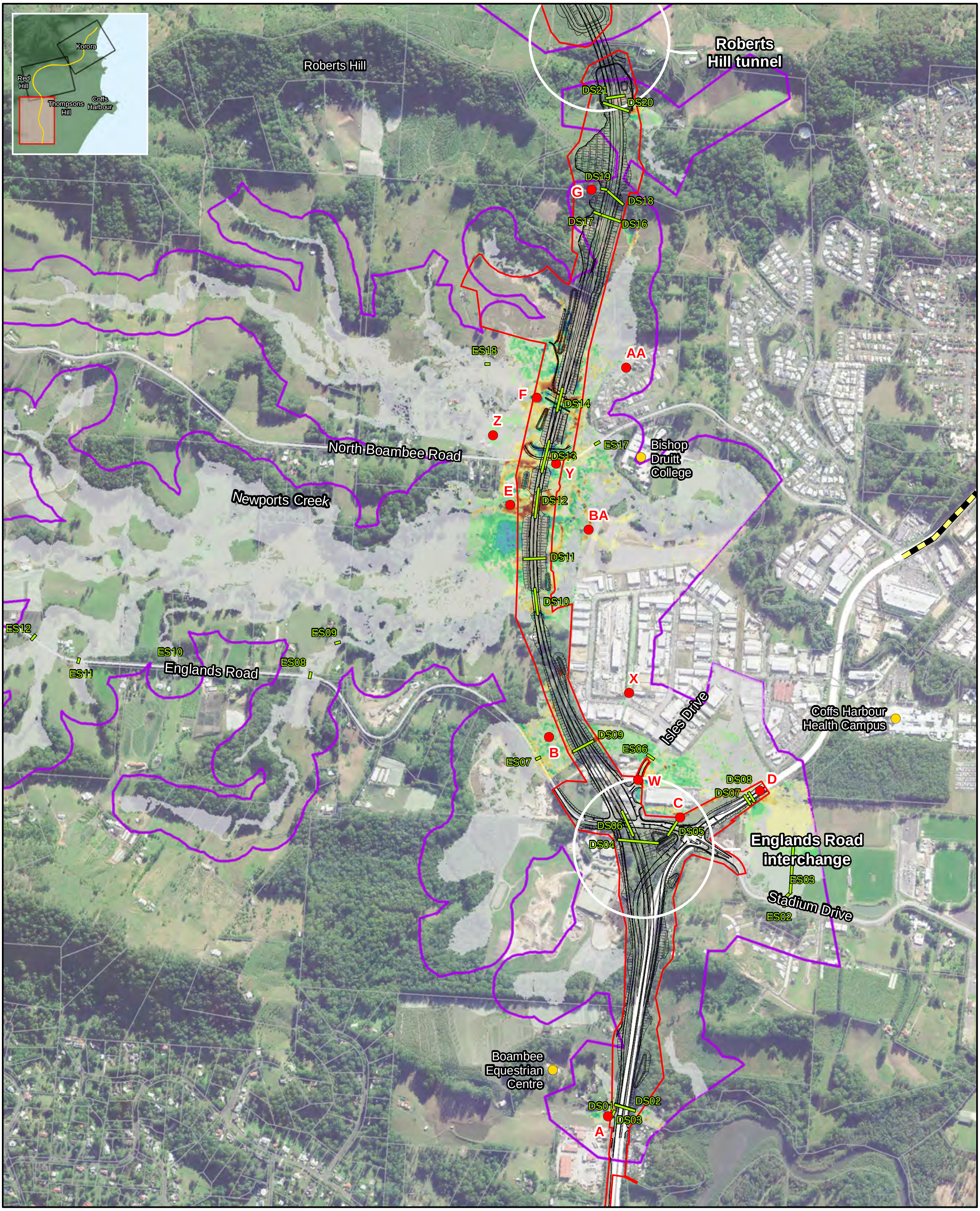
North Coast Railway
Evacuation routes
Cadastre
Construction footprint
Flood model extents
Modelled structures
Sensitive receiver
Assembly areas
POI

Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
Northern Creek 5 % AEP peak flood velocity difference
D2.3.3

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

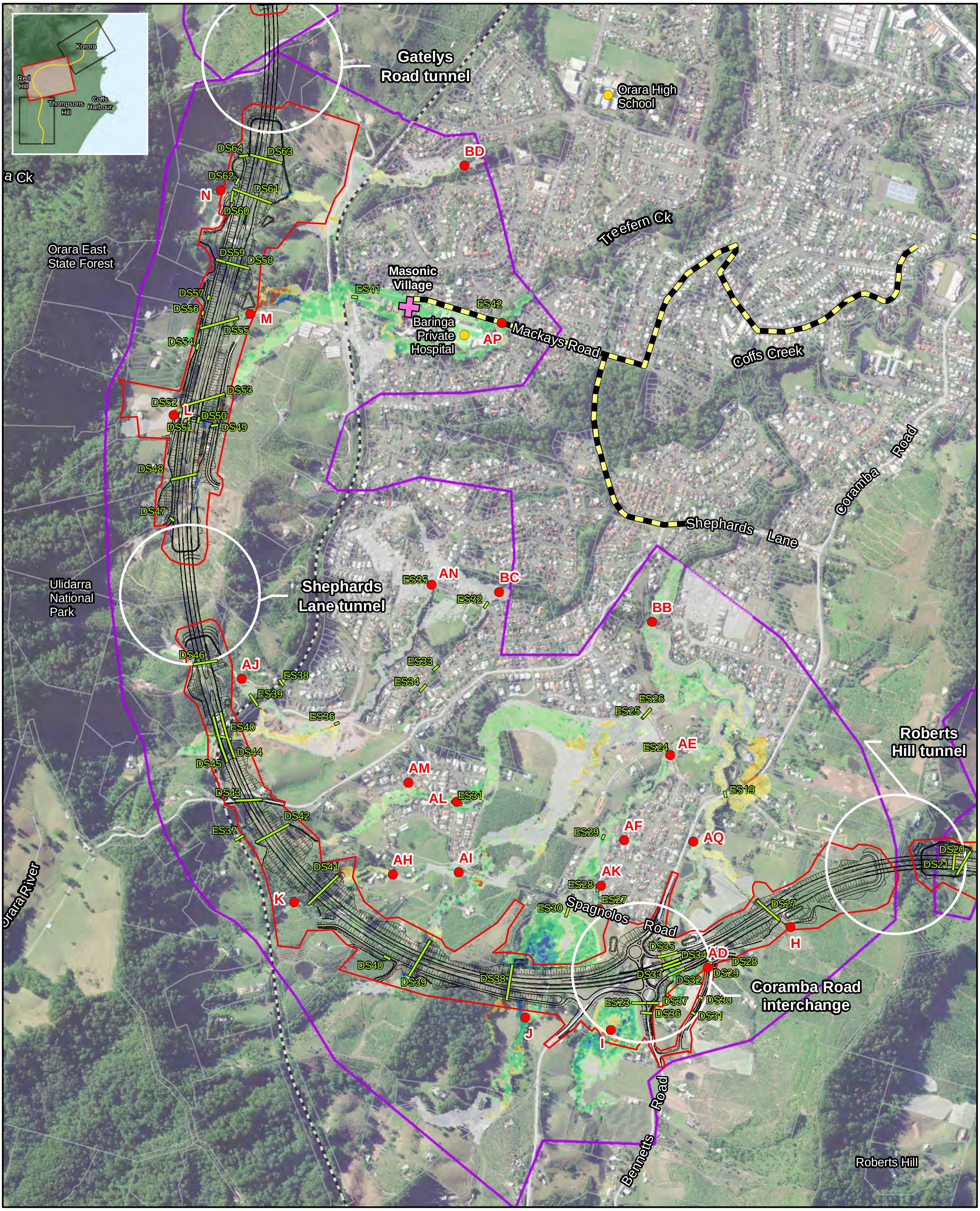
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

Peak velocity impact (m/s)

>0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
North Boambee Valley 2 % AEP peak flood velocity difference
D2.1.4

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
Coffs Creek 2 % AEP peak flood velocity difference
D2.2.4

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

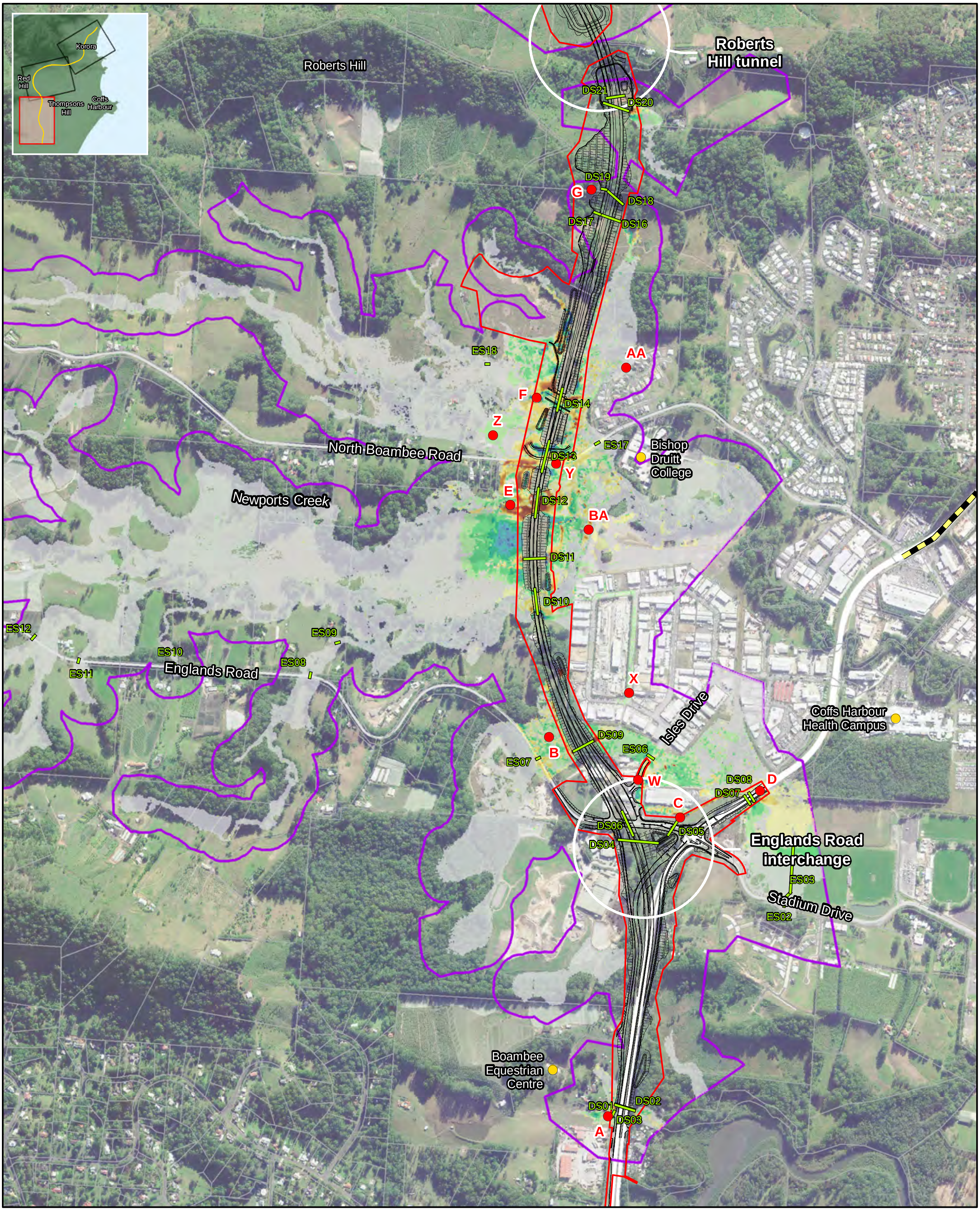
Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
Northern Creek 2 % AEP peak flood velocity difference
D2.3.4

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N



Legend

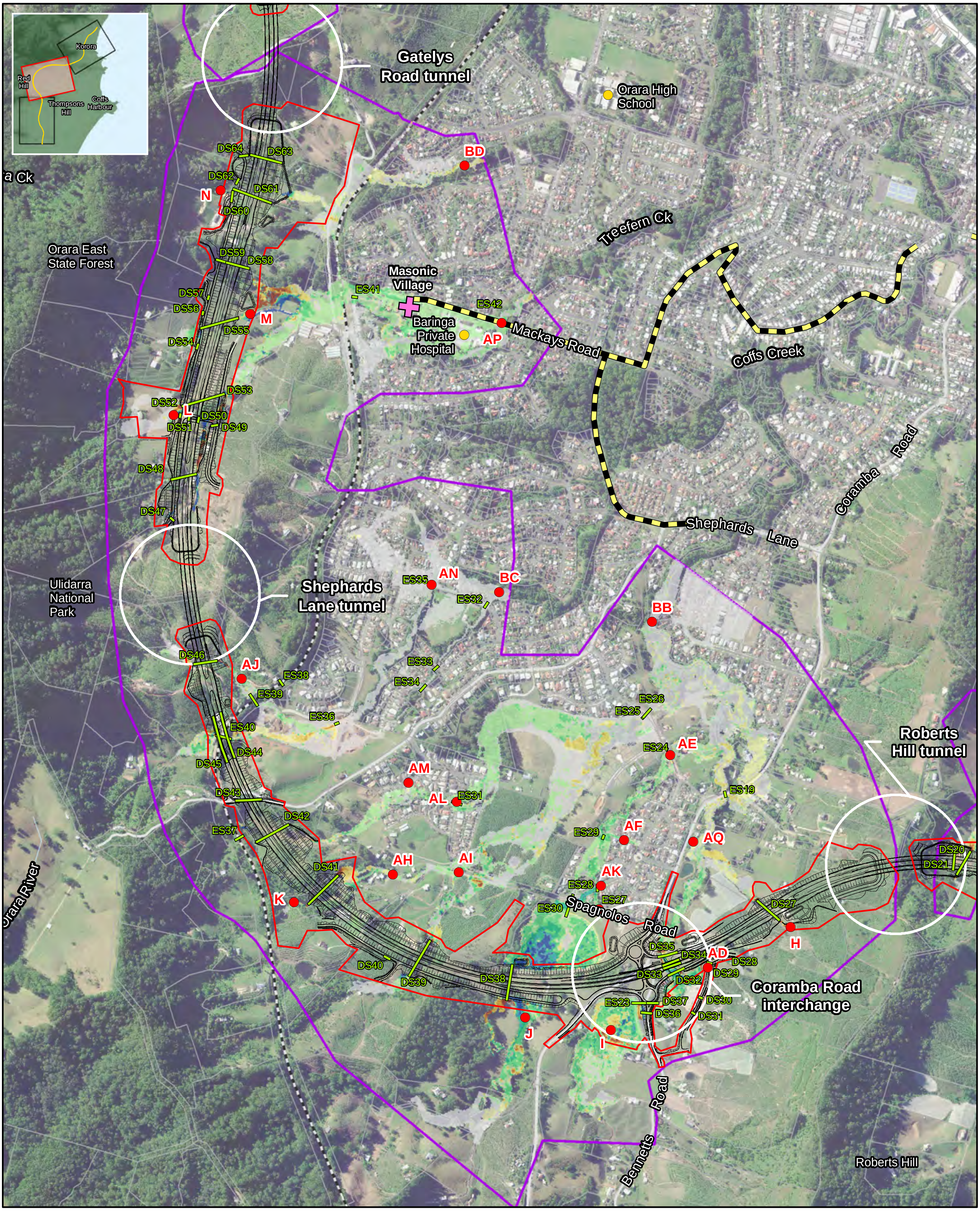
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
North Boambee Valley 1 % AEP peak flood velocity difference
D2.1.5

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
Coffs Creek 1 % AEP peak flood velocity difference
D2.2.5

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N

Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

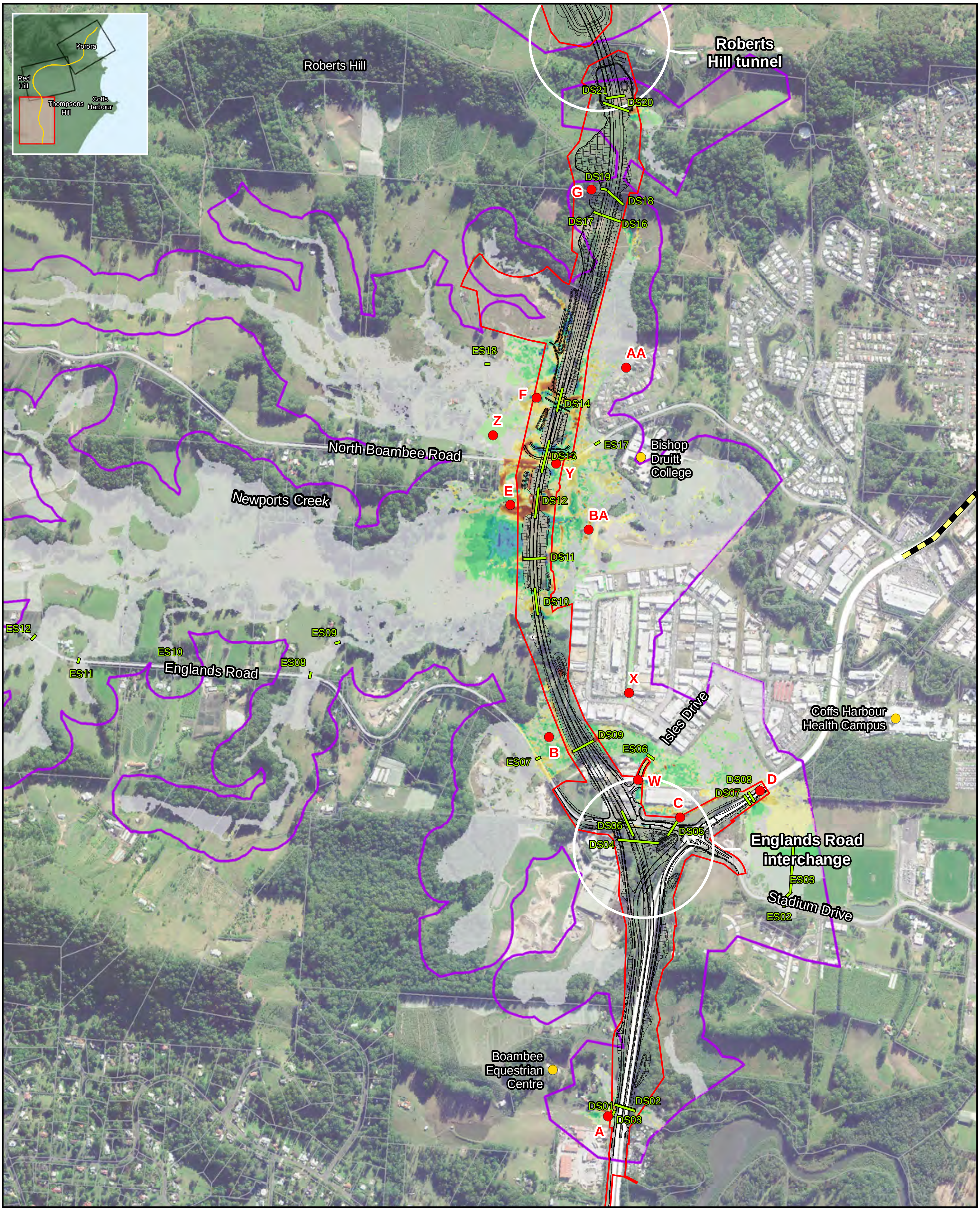
Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
Northern Creek 1 % AEP peak flood velocity difference
D2.3.5

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N



Legend

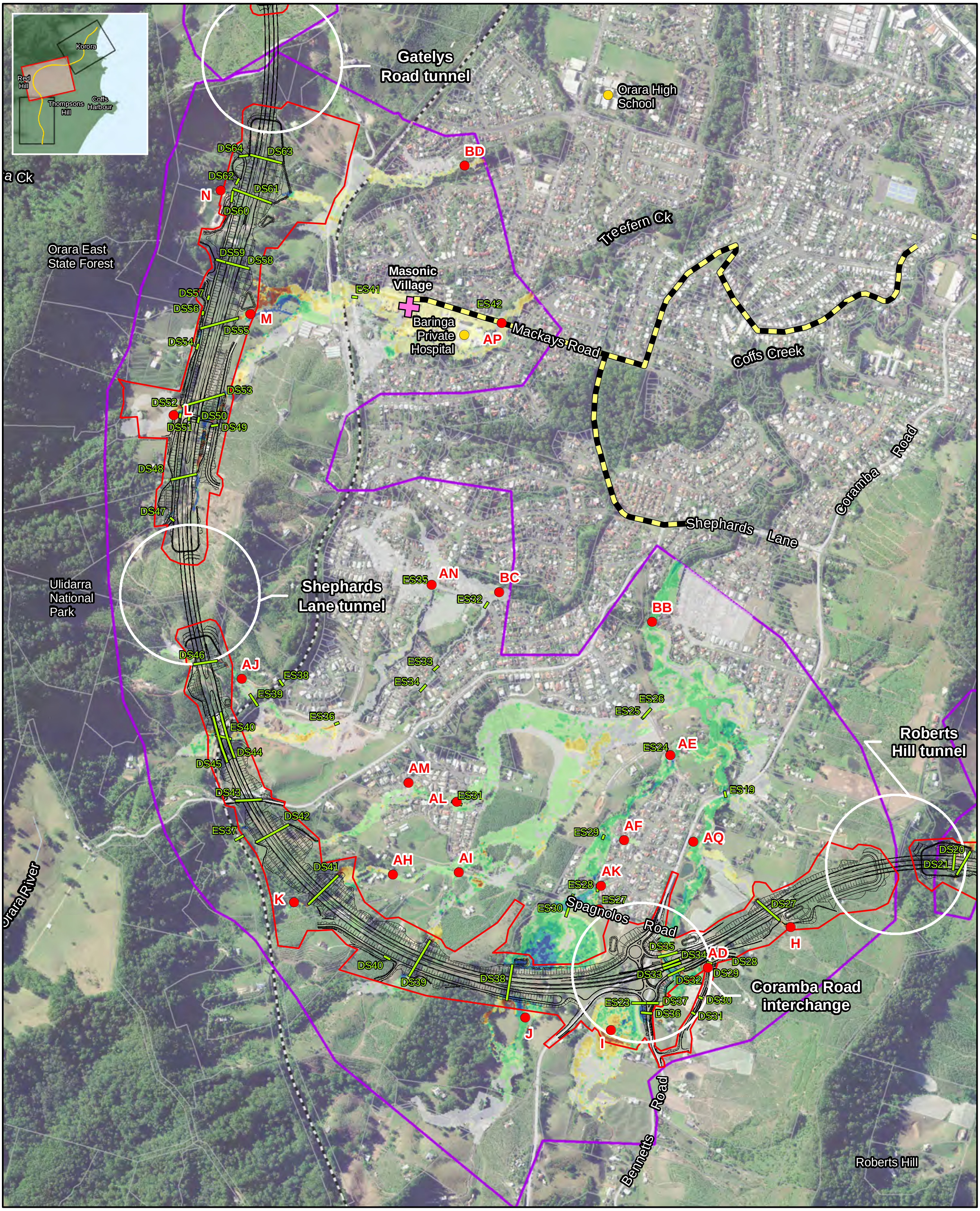
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
North Boambee Valley 1 % AEP 2050 climate peak flood velocity difference
D2.1.6

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
Coffs Creek 1 % AEP 2050 climate peak flood velocity difference
D2.2.6

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

Legend

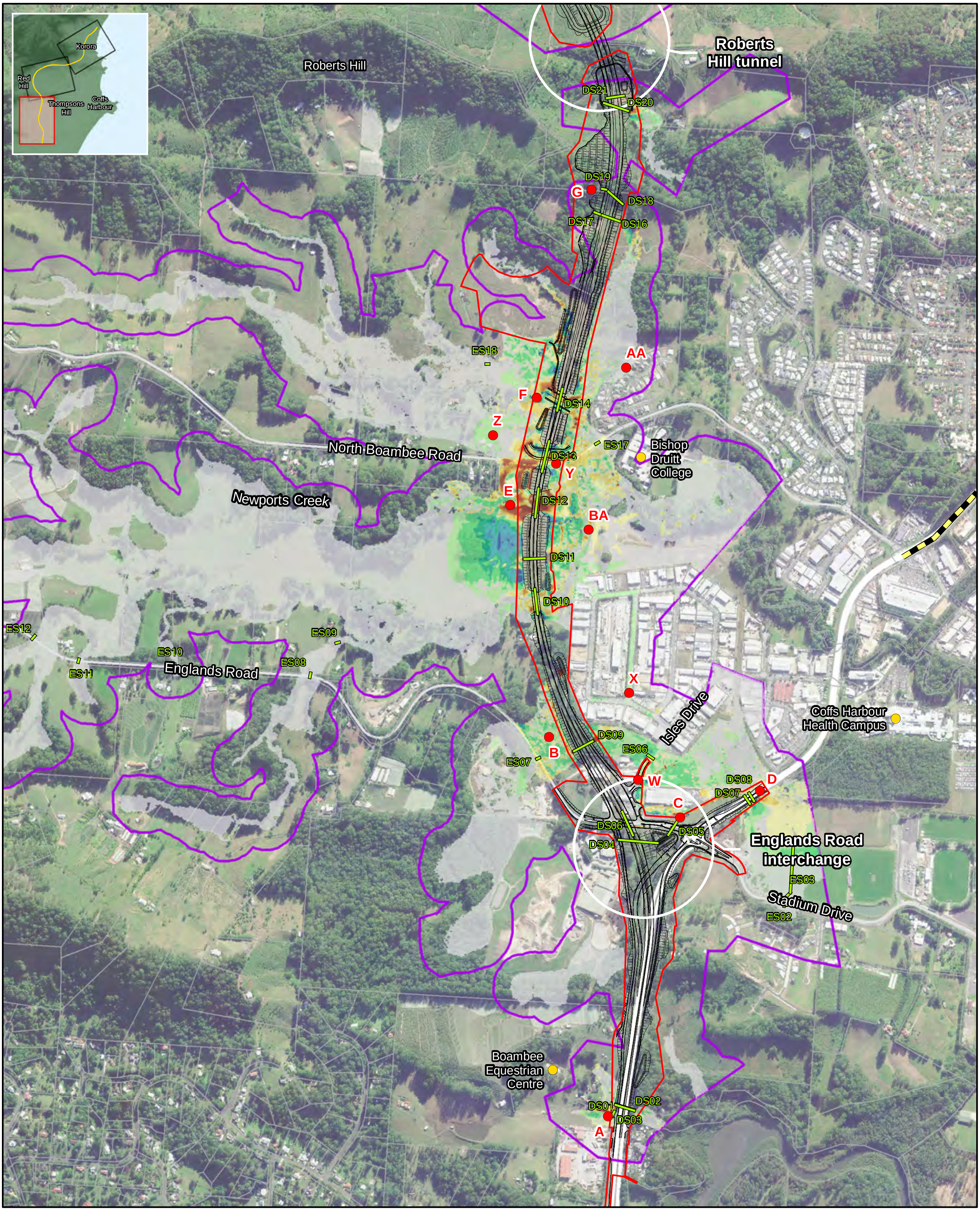
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
Northern Creek 1 % AEP 2050 climate peak flood velocity difference
D2.3.6

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

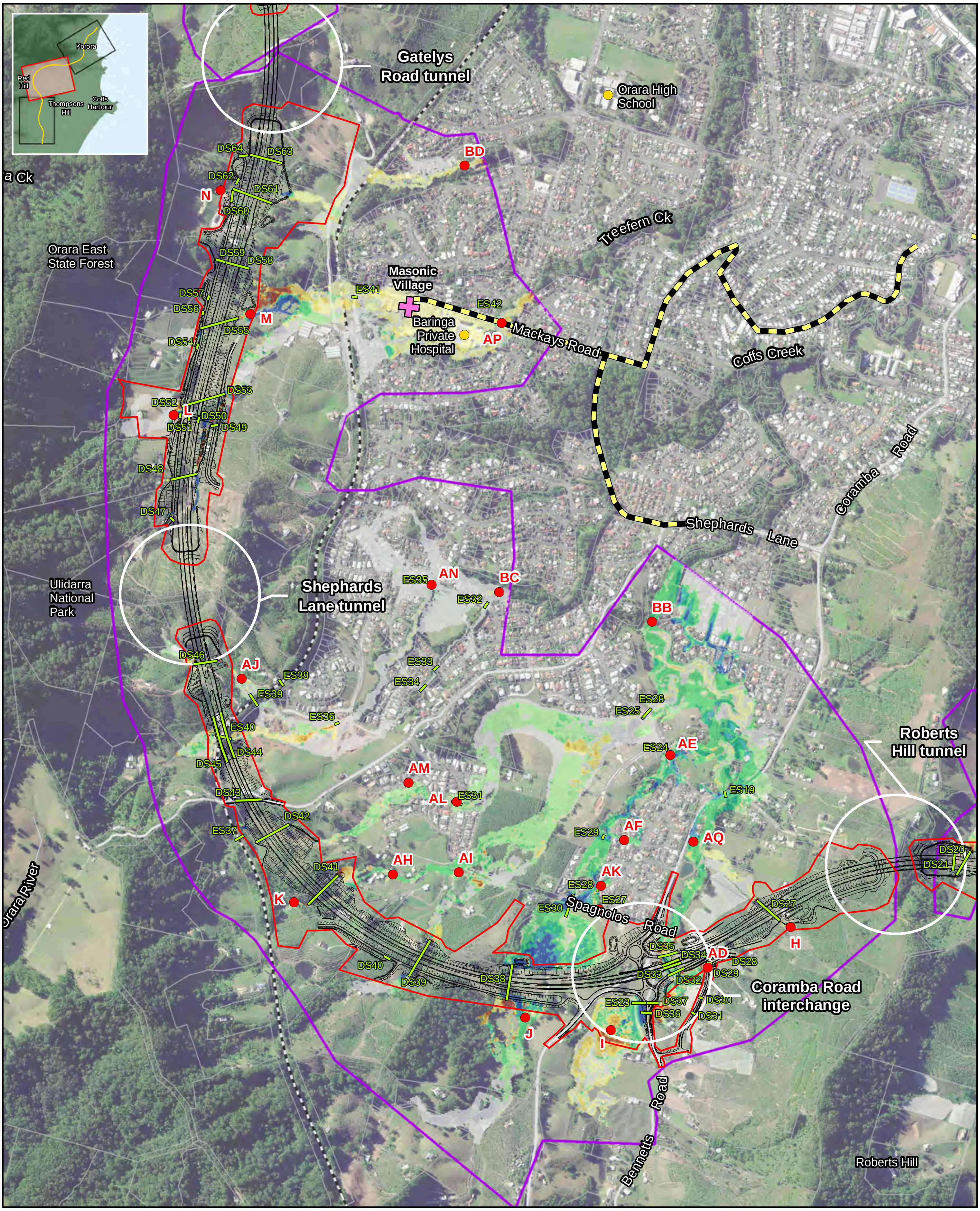
Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
North Boambee Valley 1 % AEP 2100 climate peak flood velocity difference
D2.1.7

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N



Legend

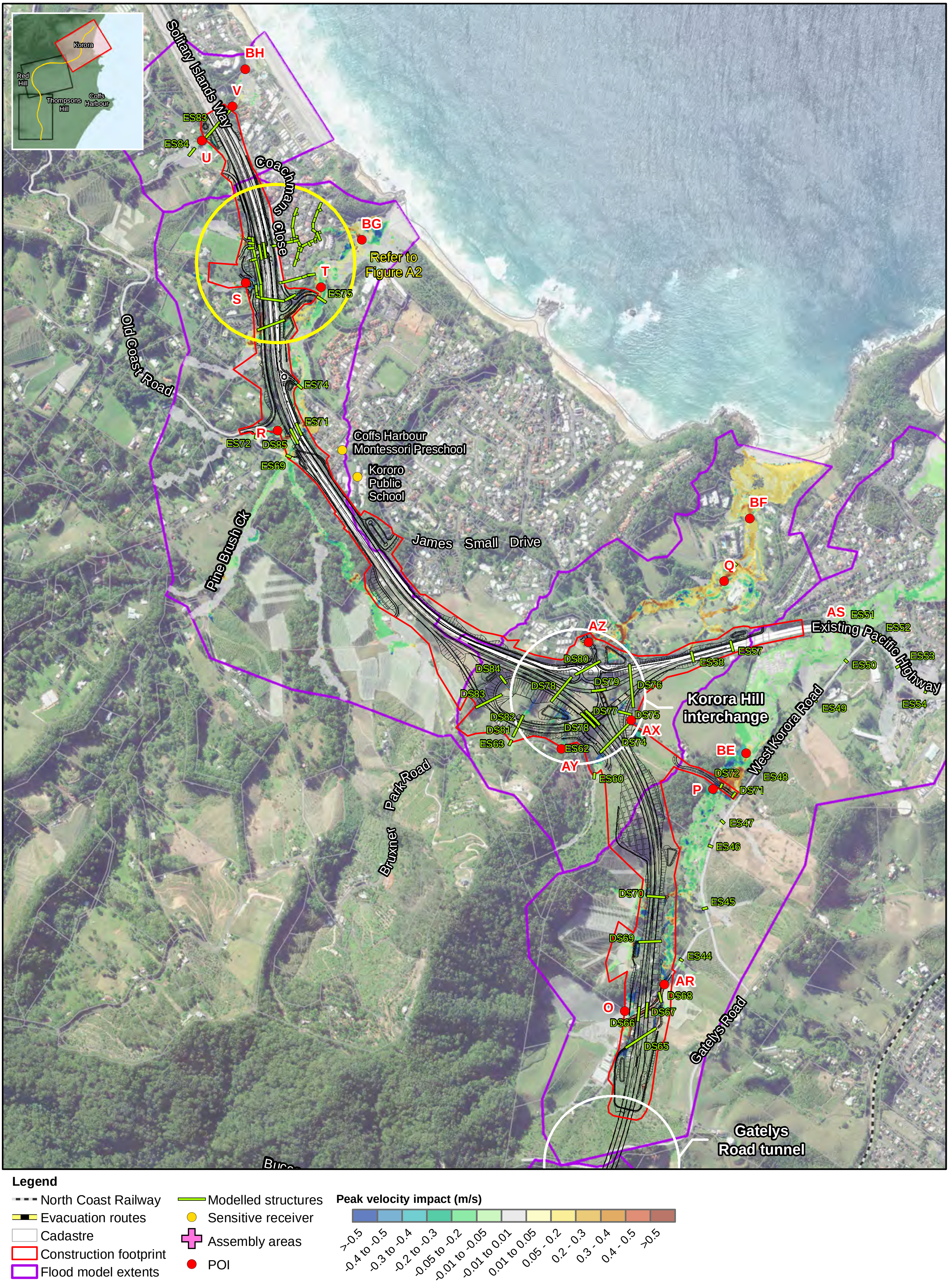
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

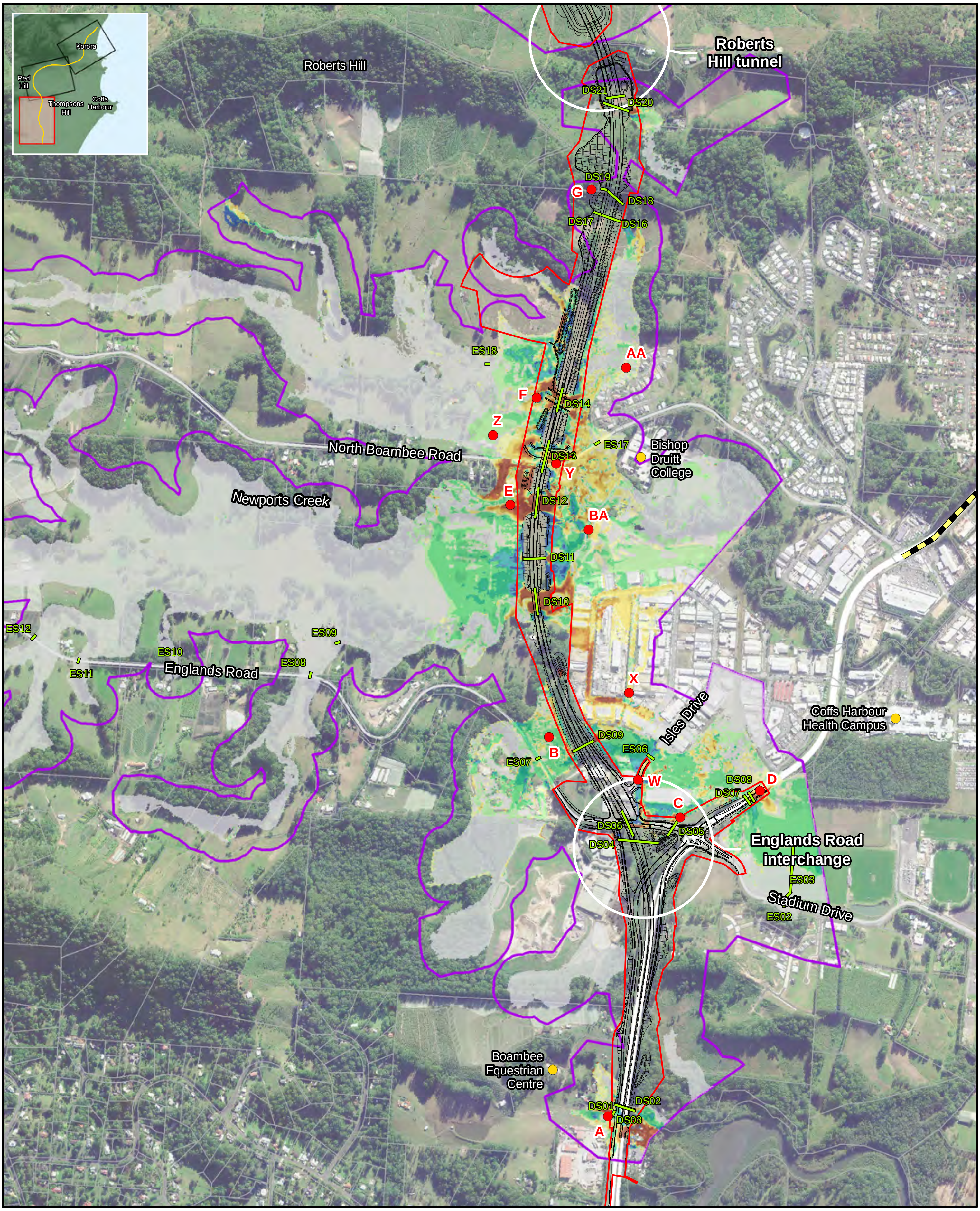
Coffs Harbour Bypass
Coffs Creek 1 % AEP 2100 climate peak flood velocity difference
D2.2.7

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56



Coffs Harbour Bypass
Northern Creek 1 % AEP 2100 climate peak flood velocity difference
D2.3.7

Scale @A4: 1:17,500
GDA 1994 MGA Zone 56



Legend

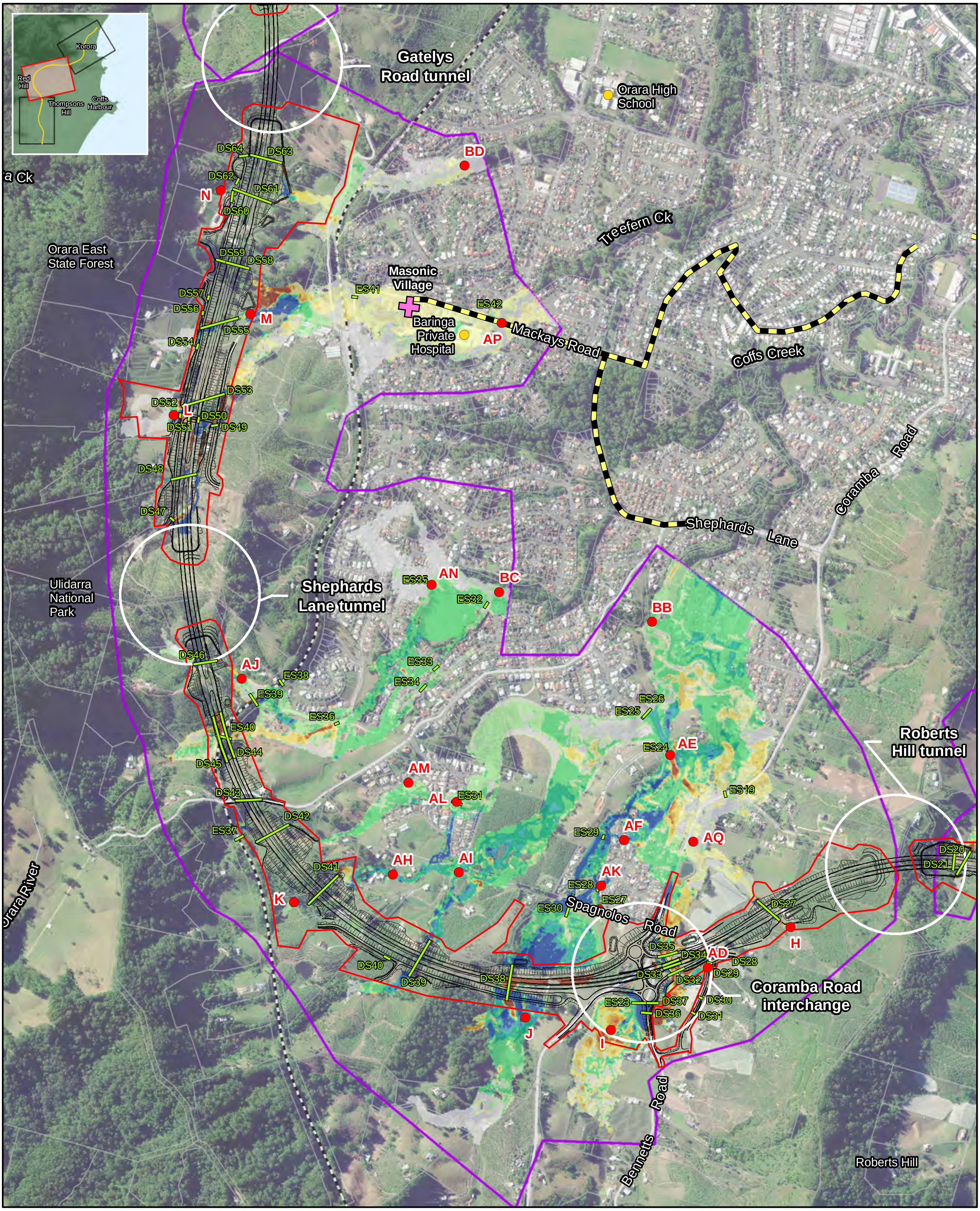
North Coast Railway
Evacuation routes
Cadastre
Construction footprint
Flood model extents
Modelled structures
Sensitive receiver
Assembly areas
POI

Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
North Boambee Valley PMF peak flood velocity difference
D2.1.8

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

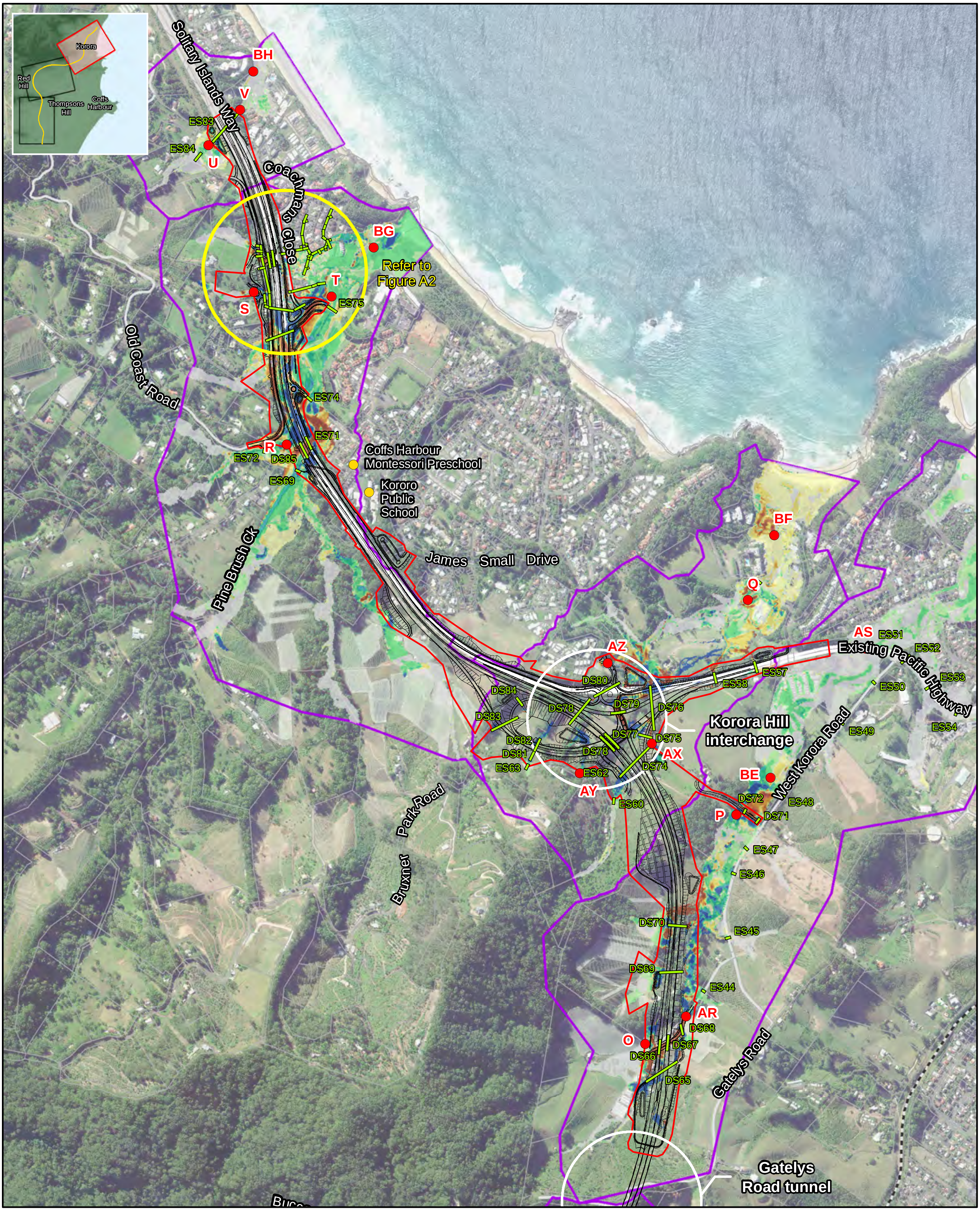
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

Peak velocity impact (m/s)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
Coffs Creek PMF peak flood velocity difference
D2.2.8

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI

Peak velocity impact (m/s)

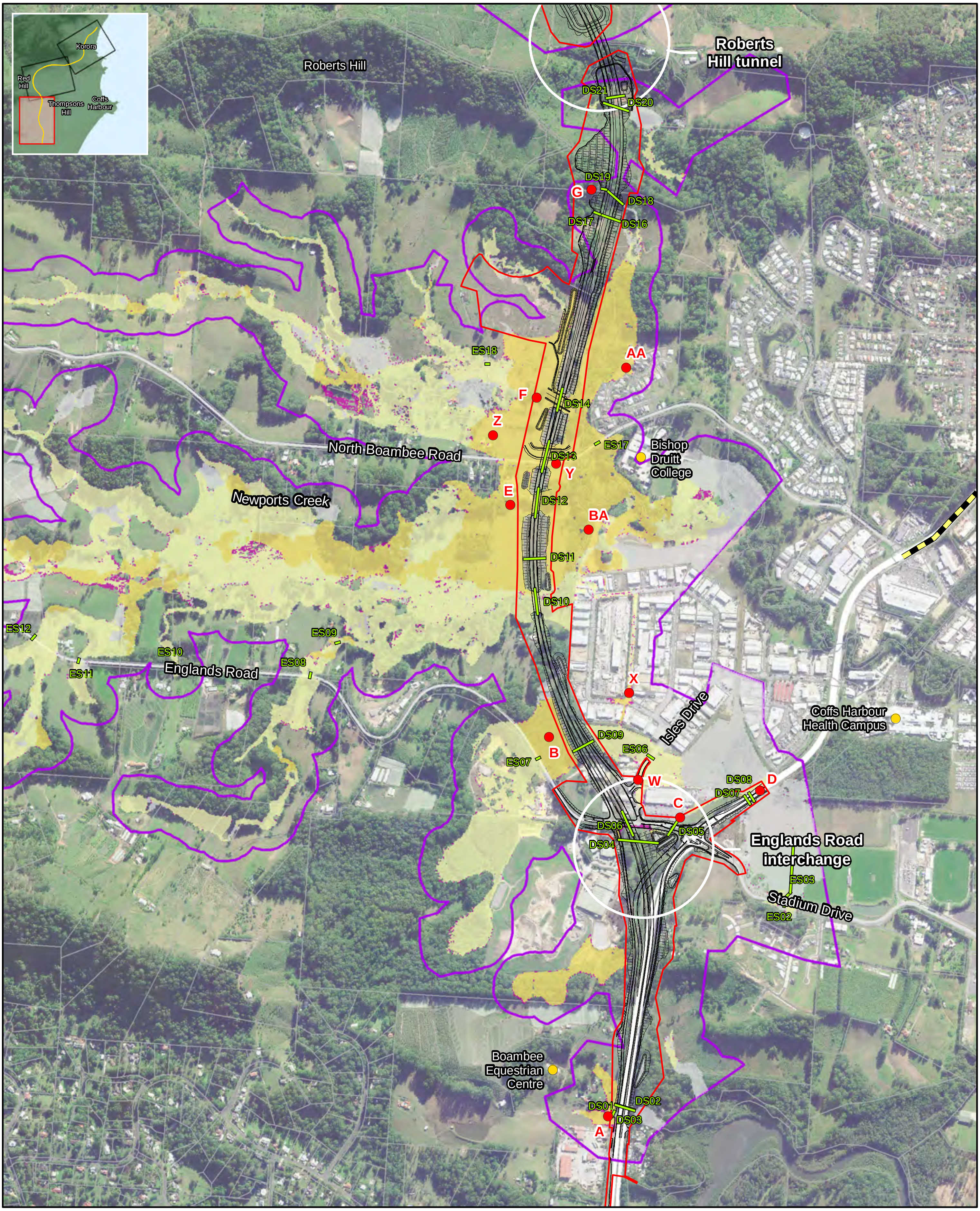
>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
Northern Creek PMF peak flood velocity difference
D2.3.8

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N

Appendix E

Climate change flood maps



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

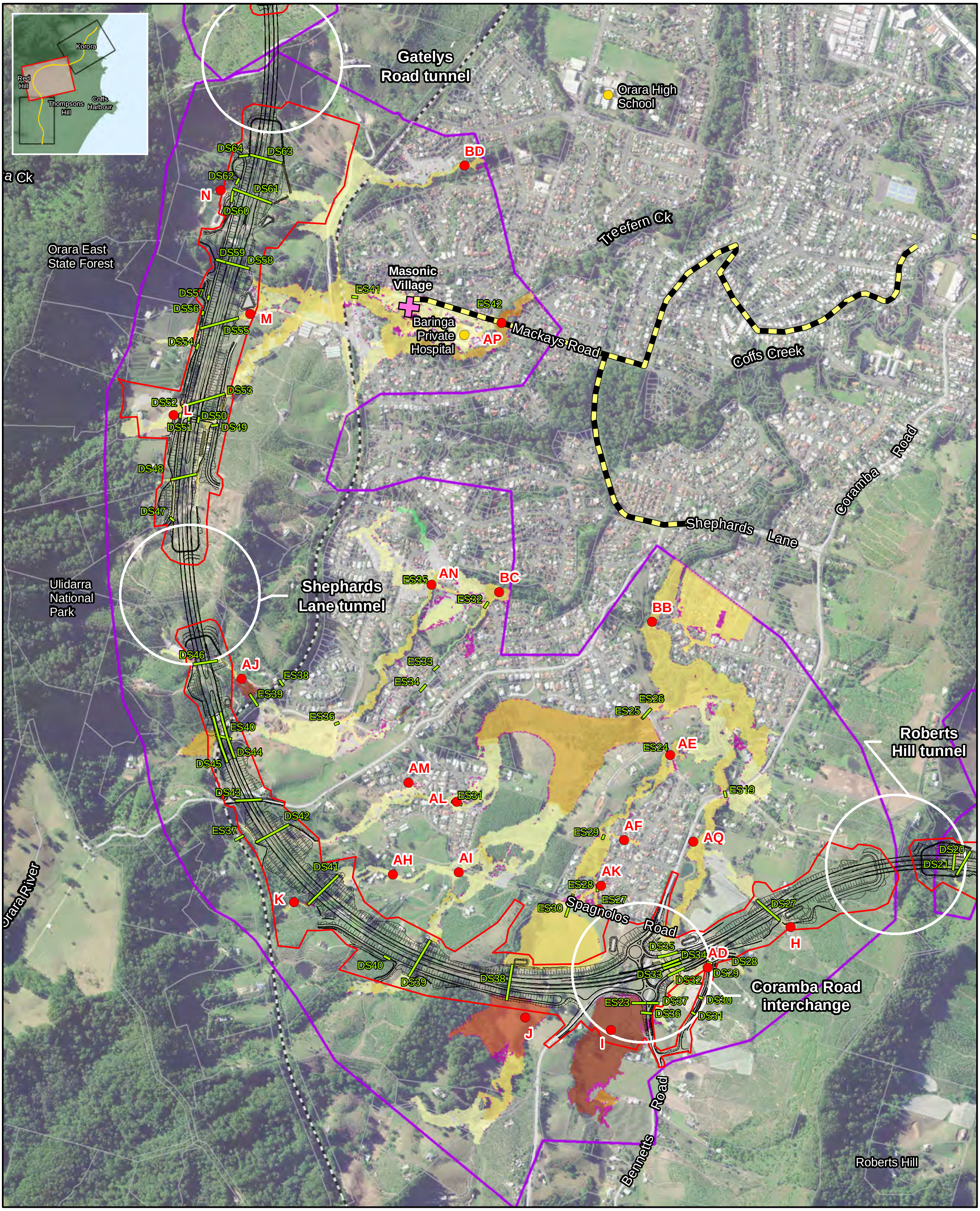
Peak level impact (m)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
North Boambee Valley 1 % AEP 2050 climate peak flood level difference from climate change
E1.1

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56

N



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

Peak level impact (m)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass

Coffs Creek 1 % AEP 2050 climate peak flood level difference from climate change

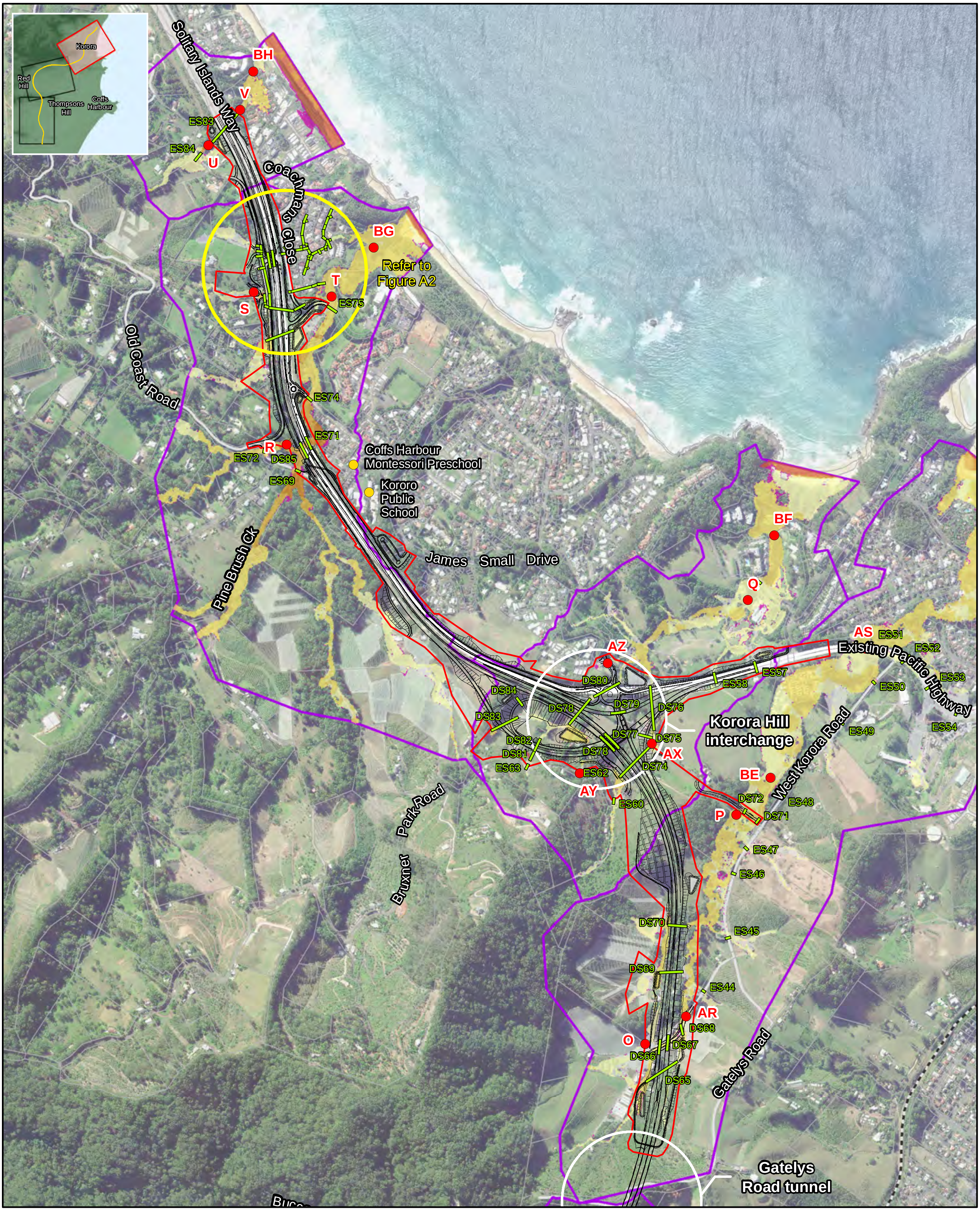
E2.1

0 0.15 0.3 0.45 km

Scale @A4: 1:17,500

GDA 1994 MGA Zone 56

N



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

Peak level impact (m)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

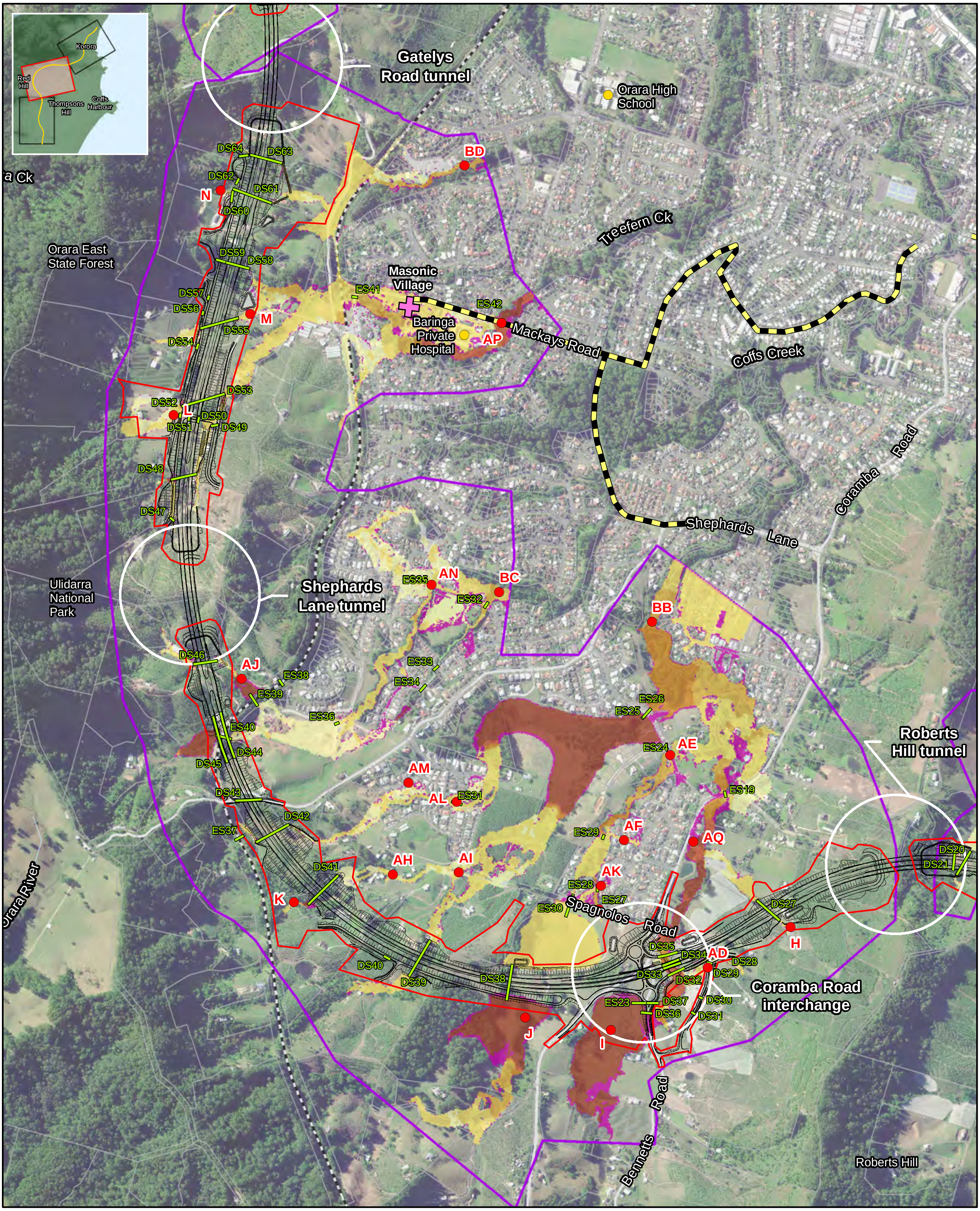
Coffs Harbour Bypass
Northern Creek 1 % AEP 2050 climate peak flood level difference from climate change
E3.1

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N

Coffs Harbour Bypass

North Boambee Valley 1 % AEP 2100 climate peak flood level difference from climate change

E1.2



Legend

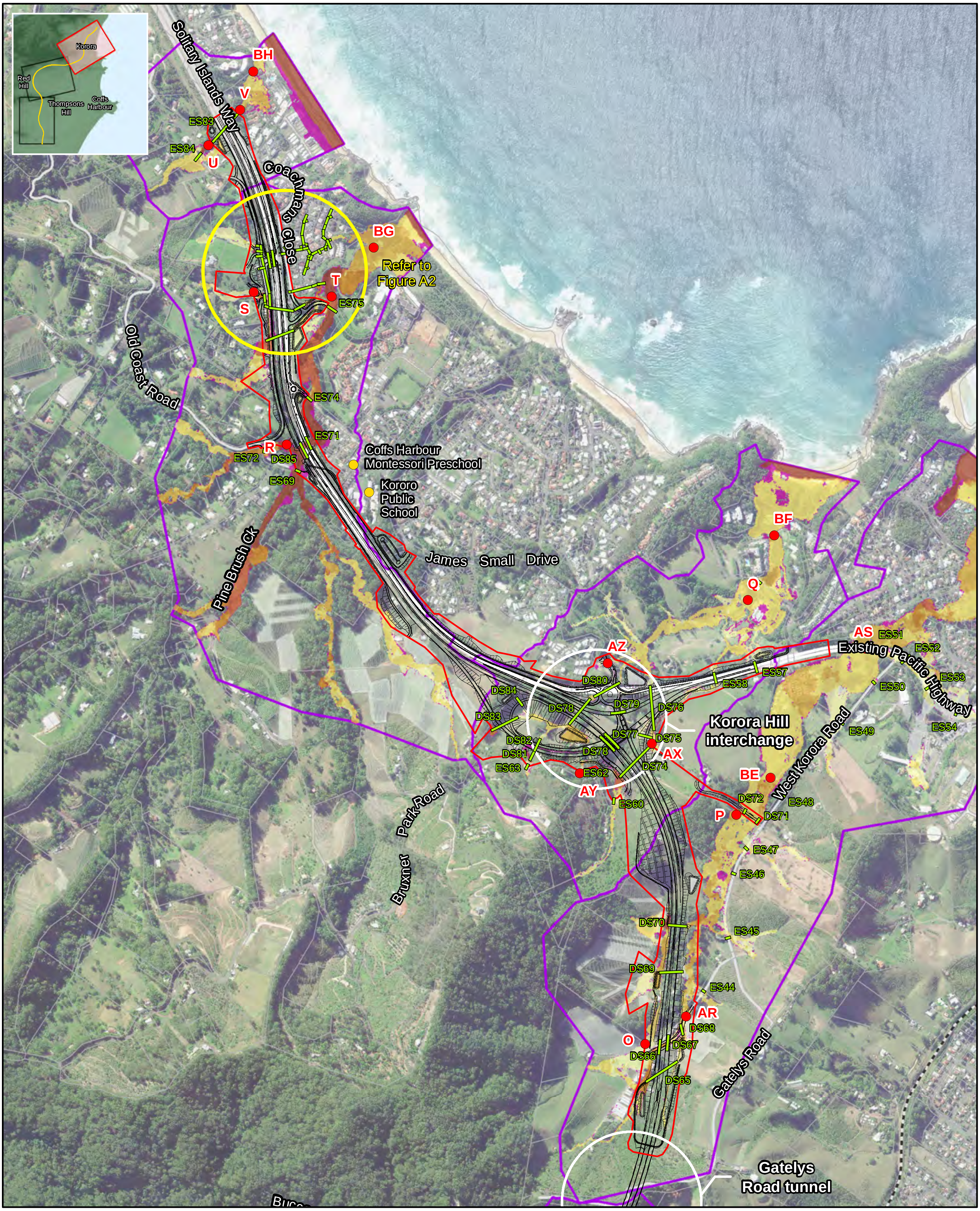
- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

Peak level impact (m)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
Coffs Creek 1 % AEP 2100 climate peak flood level difference from climate change
E2.2

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N



Legend

- North Coast Railway
- Evacuation routes
- Cadastre
- Construction footprint
- Flood model extents
- Modelled structures
- Sensitive receiver
- Assembly areas
- POI
- Was wet now dry
- Was dry now wet

Peak level impact (m)

>-0.5, -0.4 to -0.5, -0.3 to -0.4, -0.2 to -0.3, -0.05 to -0.2, -0.01 to -0.05, -0.01 to 0.01, 0.01 to 0.05, 0.05 - 0.2, 0.2 - 0.3, 0.3 - 0.4, 0.4 - 0.5, >0.5

Coffs Harbour Bypass
Northern Creek 1 % AEP 2100 climate peak flood level difference from climate change
E3.2

0 0.15 0.3 0.45 km
Scale @A4: 1:17,500
GDA 1994 MGA Zone 56
N