Australia Pacific LNG Project

Volume 5: Attachments

Attachment 22: Surface Water and Watercourses – Gas Fields

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Executive Summary

WorleyParsons has been commissioned by Australia Pacific LNG to undertake surface water investigations across the gas fields study area associated with the gas field element of the Australia Pacific LNG project (the Project). The scope of work for the surface water and watercourses assessments was based upon the need to address the Terms of Reference (ToR), as set out by the Coordinator-General. Executive Summary Table 1 details these requirements.

Executive Summary Table 1 Terms of Reference requirements and relevant report section

Relevant Report Section	ToR Section
Section 4	3.4.1.1 Description of environmental values
Section 5	3.4.1.1 Description of environmental values
	3.4.1.2 Potential impacts and mitigation measures
	7. Cumulative impacts
Section 6	2.5.1 Stormwater drainage
	3.4.1.2 Potential impacts and mitigation measures
	8. Environmental management plan
Section 7	3.4.1.2 Potential impacts and mitigation measures
Section 8	3.4.1.2 Potential impacts and mitigation measures

In order to meet these requirements, the following scope of work was undertaken:

- Consideration was given to the relevant legislative framework
- A methodology was adopted for each technical assessment to specifically address the ToRs
- Descriptions of the existing environment and environmental values were prepared
- Potential impacts upon these environmental values were then identified
- Mitigation measures for these potential impacts were then proposed
- A risk assessment was conducted that factored in both potential impacts and mitigation measures

Some requirements of the Terms of Reference as listed in Section 3.4.1 have been addressed in the Aquatic Ecology, Water Quality and Geomorphology Impact Assessment – Gas Fields (Hydrobiology 2009) (Volume 5, Attachment 20). This includes but is not limited to water quality (including monitoring), geomorphic, and descriptions of environmental values of the major waterways within the gas fields study area.

The gas field element of the Project covers an area of approximately 570,000ha and is comprised of:

- Up to 10,000 gas wells
- A network of gas and water gathering pipelines

- Six new water treatment facilities (WTFs) and one upgrade
- Twenty three gas processing facilities (GPF)
- Thirty three water transfer stations (WTS)
- Three brine ponds (BP) and one upgrade

The investigations into surface water related aspects of the project have included the following:

Regional scale flooding investigation

There are a number of catchments and associated river systems which the gas fields study area intersects.

The Condamine River is a major tributary of the Darling River, located in the upper Murray-Darling catchment. Its boundaries to the east and north are formed by the Great Dividing Range, approximately 1,400m above sea level, near Toowoomba and Warwick. Its southern boundaries comprise the much lower Herries Range, which is approximately 800m above sea level. The western boundaries comprise the Dogwood Creek sub-catchment which flows into the Condamine River, where it becomes the Balonne River.

The Dawson River catchment is a sub-catchment of the Fitzroy Basin. It has a total area of about 50,800km^2 and is bordered by the Auburn, Calliope, Ulam and Dee Ranges to the east. The Great Dividing Range lies to the west and south, and the Lynd and Canarvon, Expedition and Bigge ranges to the north west (Telfer 1995). The south western headwaters of the Dawson River flow easterly through relatively narrow valleys until about the Nathan Gorge constriction. From there the channel alters direction, flowing north, with a gradual downstream broadening of the valley to wide alluvial plains.

The Border Rivers catchment is located on the Queensland – New South Wales (NSW) border and covers about 50 000 km^2. The south eastern headwaters border the Great Dividing Range in NSW, whereas the north west headwaters border the southern section of the Condamine River catchment near Millmerran.

An analysis of catchment response and flooding behaviour within the gas fields study area was undertaken using the Runoff Analysis and Flow Training Simulation (XP-RAFTS) and the Two Dimensional Unsteady Flow (TUFLOW) modelling packages.

Existing case (pre-project) modelling results showed varying degrees of inundation within the gas fields study area. Areas where significant or expansive inundation was predicted to occur were typically in watercourses with large contributing catchments and poorly defined waterways with expansive floodplains. Examples of expansive flooding include the Dogwood Creek and Condamine River systems.

Most local roads were shown to be inundated at waterway crossings within the gas fields study area. These crossings are typically causeways or natural crossings, and are hence subject to frequent inundation. Some major road corridors, including the Warrego Highway and Leichardt Highway, were also shown to suffer from inundation in the 10-year Average Recurrence Interval (ARI) rainfall event at a number of discrete locations.

The majority of proposed infrastructure locations were predicted to be flood free during all of the modelled rainfall events (10, 20, 100, 500-year ARI design rainfall events). However, eleven of the proposed infrastructure locations were shown to be located within the existing case (pre-project) flood extents. At the majority of these locations, inundation was predicted to be at the edge of the

infrastructure site, however there are two locations with tributaries flowing through the middle of the proposed infrastructure area. All infrastructure locations predicted to be inundated are summarised in Executive Summary Table 2 with the respective rainfall event in which inundation occurs. The remaining water treatment facilities and gas processing facilities are not expected to be inundated during the flood events modelled as part of this study.

Executive Summary Table 2 Infrastructure location within modelled flood extents

Infrastructure type	Infrastructure name	Rainfall event in which regional flood inundation occurs*
Water treatment facility	WTF_MEL_01	10-year ARI
Brine pond	BP_MEL_01	10-year ARI
Water transfer station	WTS_COM_04	10-year ARI
Water treatment facility	WTF_RCK_01a	10-year ARI
Brine pond	BP_RCK_01a	10-year ARI
Water transfer station	WTS_PHS_07	10-year ARI
Gas processing plant	GPF_HCK_01a	500-year ARI
Water treatment facility	WTF_CON_01	10-year ARI
Gas processing plant	GPF_CON_02b	10-year ARI
Water transfer station	WTS_TAL_00	10-year ARI
Gas processing plant	GPF_WAA_03	10-year ARI

* Smaller rainfall events than the 10-year ARI event were not assessed as part of this investigation.

It is envisaged in the majority of cases, individual infrastructure items could be located away from the areas of inundation within the proposed infrastructure locations identified in Executive Summary Table 2. If this is not achievable, movement of the proposed infrastructure site clear of the predicted 100 year ARI flood extent may be required. Although gas well locations are yet to be confirmed, it is envisaged that a number of wells will be located within the flood extents of all the major waterways investigated as part of this study. Appropriate well designs to deal with possible inundation at these locations will be required.

While at this stage it is not envisaged to be necessary for operational purposes, flood immunity upgrades for access roads, (to provide 10-year ARI immunity of access roads) have been reviewed. The review indicated immunity is achievable for most local roads. However, more intensive crossing augmentations were shown to be required at major watercourse crossings such as the Warrego Highway crossing of Dogwood Creek, Miles. Summaries of the major roads which would be inundated within the study area, based on baseline topographic data used in the investigation are provided in Executive Summary Table 3.

Executive Summary Table 3 Major roads inundated in the gas fields study area

Road name	Location	Rainfall event in which regional flooding overtopping occurs*	Depth of Inundation (m)	Period of inundation (hours)
Warrego Highway	Dogwood Creek, Miles (EPP692)	10-year ARI	0.2	>23
Leichardt Highway	South of Miles (EPP692)	10-year ARI	0.1	11
Jackson-Wandoan Road	North of Jackson (EPP972)	10-year ARI	5.9	>37
Jackson-Wandoan Road	South of Wandoan (PL209)	10-year ARI	2.2	>32
Kogan-Condamine Road	Kogan (SEP692)	10-year ARI	1.4	>19
Dalby-Kogan Road	Kogan (SEP692)	10-year ARI	1.0	>24
Tara-Kogan Road	Kogan (SEP692)	10-year ARI	0.2	4
Warra-Kogan Road	Kogan (SEP692)	10-year ARI	0.7	>21
Chinchilla-Tara Road	South of Chinchilla	10-year ARI	1.4	>59
Kogan-Condamine Road	East of Condamine (multiple crossings)	10-year ARI	5.3	>108

* smaller rainfall events than the 10-year ARI event were not assessed as part of this investigation.

Given the nature of the creek crossings by minor roads within the study areas, as well as the number of major roads predicted to be inundated during the 10-year ARI event, it is envisaged that the majority of infrastructure sites will be inaccessible at some point during a major rainfall event (≥10 year ARI event) if no augmentation of creek crossings occurs. This is of particular importance to the Warrego Highway which is the major transport link to the western tenement areas. Local roads, however, will typically have shorter periods of inundation due to the smaller catchments upstream of these crossings.

Stormwater Management

Stormwater Management Plans have been prepared to mitigate potential adverse waterway impacts relating to the construction and on-going operation of the various gas processing and water treatment facilities throughout the gas fields.

Approximately half of sites are located on unconstrained areas requiring no diversion of stormwater runoff from upstream around the facilities. The remaining sites will require varying degrees of diversion techniques. The detailed design of the diversion systems will be undertaken during the design phases

of the project. Executive Summary Table 4 summarises the assessed stormwater and runoff management requirements for each site. Some sites have been assessed as requiring two management ratings, as separated within the table.

Executive Summary Table 4 Summary of stormwater management requirements

Rating	Requirement	GPF Sites	WTF Sites
Minimal	Onsite stormwater system only	PHS_05a, RCK_04a, LUK_02a, CAR_01a, DAL_02, CON_01b, CON_01c, CNS_03, OAN_04, MUG_06, COM_03a, DAL_01b, CNN_04, CON_02b, ZIG_06	MEL_01, BYM_01, GIL_01
Moderate	Onsite stormwater system plus some diversion of uphill runoff.	RAM_01b, HCK_01a, BYM_03, GIL_02, WAA_03, WAA_04, ZIG_05, ORA_03b	RCK_01a, HCK_01, WOL_01
Focused	Onsite stormwater system plus significant diversion of uphill runoff.	NGA_02, NGA_04, WOL_02, CAS_05, KIA_01a, WOL_01	CON_01, GIL_01a,
Erosion management	Dedicated erosion management for stormwater flows downstream of the site, eg scour protection	MUG_06, COM_03a, DAL_01b, CNN_04, WOL_01, CON_02b, ZIG_06, ORA_03b, NGA_04, DAL_02	WOL_01

Generic designs have been used for the purpose of the Environmental Impact Statement in relation to 23 gas processing and six water treatment facilities and these will be further adapted to specific site conditions during the design phase of the project. The current generic, conceptual stormwater treatment designs include a swale system diverting on-site stormwater into a sedimentation basin proposed to be constructed at each gas processing and water treatment facility. Following standard design guidelines the proposed basins are designed to hold stormwater runoff up to the one year Average Recurrence Interval (ARI), 24hr duration event.

The proposed basins have been assessed using the Model for Urban Stormwater Improvement Conceptualisation (MUSIC) to verify the suitability of the proposed treatment capacities. The assessment indicated the proposed basins are capable of removing over 85% of sediments from stormwater leaving the facilities.

Hydraulic stream flow impact assessment

The process by which project coal seam gas (CSG) will be extracted from the coal seam requires the depressurisation of the seam via the extraction of water. This water is generally referred to as associated water and is defined by the Environmental Impact Statement Terms of Reference as "'*Underground water taken by a petroleum tenure holder from a gas well*". This water is typically of varying quality but is generally unsuitable for direct potable, livestock or irrigation use.

The management of coal seam gas associated water is a major issue for the Australia Pacific LNG Project. The Queensland Government (Queensland Government, 2009) has identified that the volumes of associated water expected to be generated by the coal seam gas industry pose a significant challenge for the industry. However, there are significant opportunities for beneficial use of coal seam gas associated water that range from aquifer injection to irrigation of biodiesel fuel crops.

The Associated Water Management Strategy (Volume 5, Attachment 24) for the project includes, as one option, the disposal of treated associated water through discharges to watercourses across the study area.

The potential impacts that the discharge of associated water may have on the hydraulic characteristics of the watercourses proposed as discharge locations has been assessed at a preliminary level which is suitable for this stage of the project. In addition, the existing flow regimes have been characterised at each location and recommendations developed as to potentially suitable/beneficial discharge regimes.

Discharges from the Origin operated Talinga Water Treatment Facility have been investigated previously (EECO Consulting 2008; EECO Consulting, 2009) and application is currently being made for a 35 ML/d discharge to the Condamine River. As such further analysis of this discharge location is not included in this study.

The hydraulic stream flow impact assessment concludes that the watercourses at all proposed discharge locations are non-perennial in nature with no flow periods extending between 5% and 70% of the simulated time. The Condamine River at Chinchilla shows a significant trend in decreasing flow and increasing probabilities of no flow conditions from the 1950s to present. The statistical analysis of flows from mid 1950s to the construction of the Chinchilla Weir indicate that the flow pattern consisted of no flow conditions 5% of the time, whereas the long term statistical analysis indicates that no flow conditions occurred 20% of the recorded period. Simulated long term flow series analysed for the other discharge locations indicated that these tributaries are likely to exist under no flow conditions for between 55% and 70% of the time.

The watercourses were also shown to be dominated by summer and autumn flows with 45% and 30% of flows occurring in the summer and autumn months respectively.

The watercourses were estimated to exhibit low velocity, stream power and stream stress at all discharge locations under flow conditions up to bank full. It is concluded that at all proposed locations the watercourses had the hydraulic capacity to accept the addition of permeate discharges. If discharge occurred under normal flow conditions, up to bank full, there was unlikely to be a significant alteration of the geomorphic characteristics of the watercourse. Discharge should be avoided during periods of low or no flow conditions in order to reduce the total sediment transport impacts.

A significant number of water users are identified along the reach of the Condamine River between the proposed discharge location and the upstream extent of the St George Water Supply Scheme. The Condamine-Balonne system also exhibits a relatively low utilisation of un-supplemented water supply allocations indicating that additional discharges may, if released under appropriate conditions, act to increase the water availability to these users. Preliminary IQQM modelling supports this assertion.

Based on the outcomes of this investigation, the following approaches are recommended:

- Discharge to the Condamine River is preferred with discharges to the other proposed locations limited to opportunistic releases during flow periods
- Additional hydrologic modelling and streamflow monitoring would be required in order to develop a similar discharge regime for tributary locations
- A discharge regime based on Option 3, a discharge factored on averaged seasonal flows or Option 4, discharge triggered by watercourse flows, is to be adopted

- Discharge volumes to be limited to 50ML/day constant release with a 3 month period of no release from August to October from the Talinga discharge location or an alternative release arrangement which complies with the ROP to achieve a medium impact
- Discharge volumes to be limited to 100 ML/day discharge from the Talinga discharge location with a no flow condition for up to 30% of the water year triggered by a flow of less than 6 ML/day within the Condamine River or an alternative release arrangement which complies with the ROP and produces a flow duration pattern which mimics the predevelopment flow regime to achieve a low impact
- Design of discharge infrastructure be undertaken such that localised velocity and scour is minimised and appropriate mixing of discharge is achieved
- An ongoing program of monitoring be developed which includes regular inspection of discharge locations and cross section survey as well as monitoring of aquatic and riparian ecosystems. (Refer to Volume 5, Attachment 24 for details of monitoring for aquatic ecosystems)
- Investigation and implementation of alternative disposal/beneficial uses be undertaken to reduce the volume of storage/discharge required

Dam failure impact assessment

WorleyParsons undertook an assessment of the hazards associated with the various water storages within the gas fields study area. This has included assessment of the proposed water treatment facility, brine pond and water transfer station storages.

Hazard classification of the ponds has been carried out in accordance with the relevant former Department of Natural Resources and Mines (DNRM) and Environmental Protection Agency (EPA) guidelines and manuals. The former DNRM and EPA now reside under the Department of Environment and Resource Management (DERM).

The hazard classification of each of the water storages is based on both the physical and chemical composition of the stored water, as well as identification of impacts associated with a dam breach (failure impact assessment). The failure impact assessments of the respective water treatment facility and brine pond storages was carried out by way of a combination of techniques as described in the DNRM guidelines, including advanced two dimensional modelling techniques where available.

The proposed water treatment facility, brine pond and water transfer station storages are within the high hazard category, in accordance with DNRM and EPA guidelines due to the characteristics of the storages. Four of the proposed brine ponds had significant hazard ratings with respect to possible harm to humans or general economic loss as a result of a hypothetical dam failure. Typically, significant hazards associated with the failure of the water treatment facility storages were not as common due to the lower storage volumes and associated peak flows, however five of the water treatment facility ponds were still predicted to have potential impact on areas of possible human inhabitancy. Modelling of water transfer station failures was not necessary as these facilities were typically of a low storage volume (less than 250ML). Desktop review of the proposed locations of the water transfer stations indicated that the majority of ponds are to be located a significant distance from areas of possible frequent human inhabitancy.

The determination of the hazards associated with the various storage facilities within the gas fields study area has been undertaken. This resulted in a definitive classification of the hazard rating for each storage facility, and provides Australia Pacific LNG with set performance criteria with respect to storage design requirements. Due to all the water transfer station, water treatment facility and brine

pond storages being classified as high hazard, Australia Pacific LNG will be required to comply with the Environmental Authority Streamlined conditions for high hazard dams containing high hazard waste, and the Code of Compliance for High Hazard Dams Containing High Hazard Waste (EPA, 2002) in relation to the storages.

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Appendices

1. Introduction

Australia Pacific LNG proposes to develop its substantial coal seam gas (CSG) resources in Queensland. The CSG reserves occur in the Surat and Bowen Basins with the main development planned for the Walloons gas fields area.

As a result of this planned expansion, WorleyParsons have been commissioned by Australia Pacific LNG to undertake an investigation relating to a number of surface water aspects associated with the development of the CSG resources in the gas fields study area. This investigation of the surface water and water management components of the project has been undertaken in support of the Environmental Impact Statement (EIS). These components include:

Regional scale flooding investigation

The analysis of flooding for the major watercourses within the gas fields study area has been undertaken to determine existing flooding conditions within the major watercourses in the tenement areas, as well as determine any impacts on these watercourses resulting from the development of project infrastructure.

The regional flooding analysis has been undertaken using the latest industry practices and techniques, including the use of the XP-STORM hydrologic and TUFLOW hydraulic modelling packages to determine flood extents and behaviour.

This represents a complete regional scale investigation into catchment response and waterway behaviour for both the existing and base cases for the 10, 20, 100, 500 year ARI design rainfall events.

Local flooding impacts on project infrastructure sites are discussed in Section 6.

Stormwater management plans

Stormwater Management Plans have been developed as part of this study for the various infrastructure sites within the gas fields study area. These plans address local stormwater management during the construction and ongoing operation of the various gas processing and water treatment facilities.

The plans do not apply to the upgrading of the Talinga gas processing and water treatment facilities, located within the Talinga gas field west of Chinchilla, as stormwater management plans currently exist for these facilities.

Hydraulic stream flow impact assessment

In order for the GSG to be extracted it is necessary to depressurise the coal seam via the extraction of water. Once extracted, the associated water must be managed. The Adaptive Associated Water Management Plan (Volume 5, Attachment 12) outlines a range of options currently being considered to manage this water, including the opportunistic discharge of treated associated water to watercourses within the gas fields study area. This section assesses the potential impacts that discharge of treated associated water may have on the hydraulic characteristics of the watercourses proposed as discharge locations. In addition, the existing flow regimes are characterised at each location and recommendations provided as to potential discharge regimes.

The study area is predominately located within the Condamine-Balonne water resource planning catchment, however there are a number of proposed discharge locations where associated water may

be directed into the Fitzroy Basin catchment and the Border Rivers catchment. The watercourses which have been identified as potential locations for discharge based on their proximity to the WTFs include:

- Condamine River
- Yuleba Creek
- Tchanning Creek
- Dulacca Creek
- Woleebee Creek (Fitzroy Basin)
- Unnamed Tributary of Kangaroo Creek (Fitzroy Basin)
- Unnamed tributary of Weir River (Border Rivers).

Dam failure impact assessment

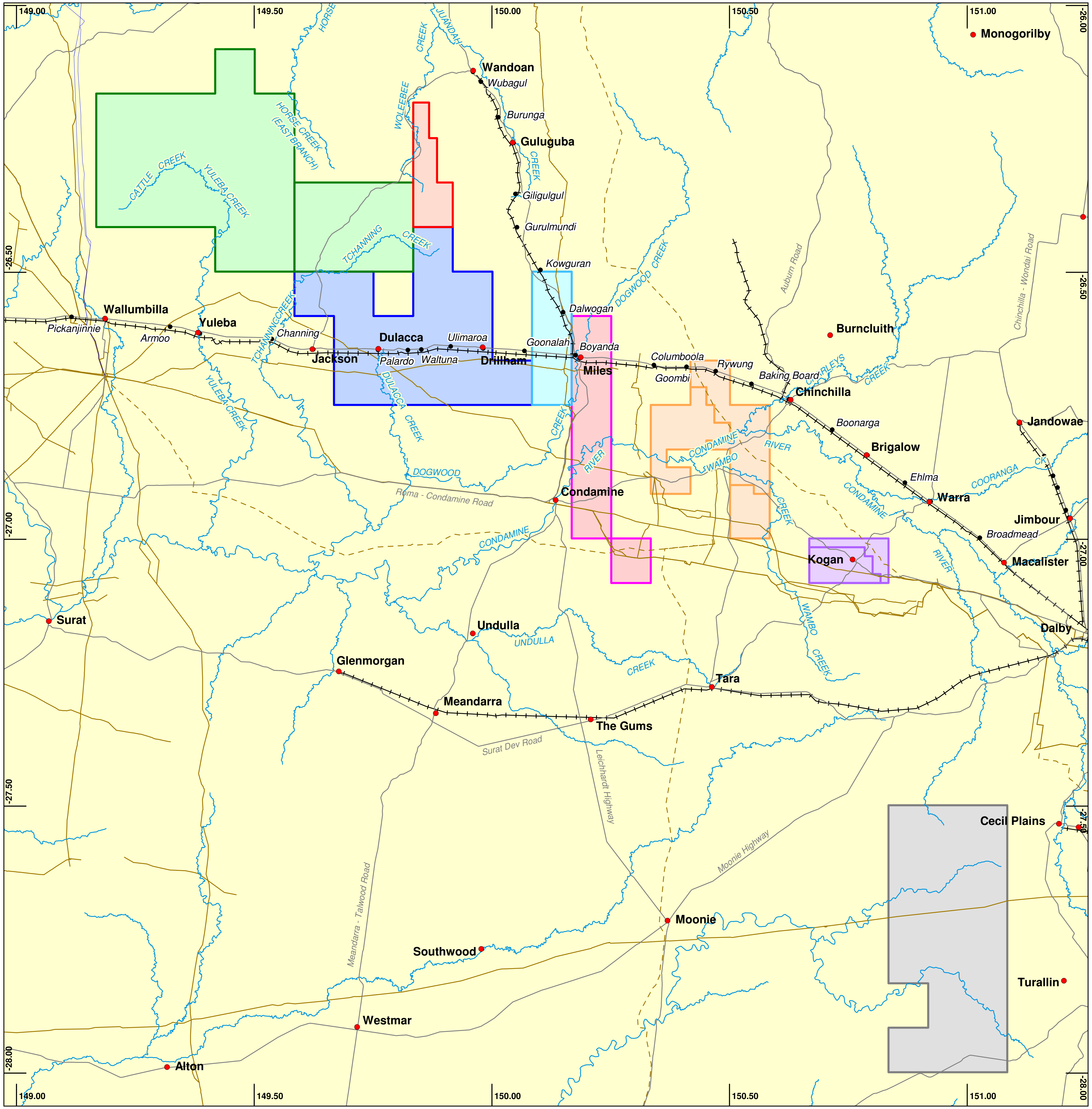
The dam failure analysis has involved hazard classification of water storages according to the Environmental Protection Agency (EPA) and Department of Environment and Resource Management (DERM) guidelines. These classifications define whether the ponds are hazardous dams due to their physical characteristics (e.g. height, size), and whether the ponds are hazardous due to their physical or chemical composition of the stored water. Based on these classifications the dam may be classified as referable or regulated and design requirements are determined for spillways, design storage allowance and mandatory reporting levels.

Hazard ratings resulting from hypothetical dam failure scenarios have also been assessed. Failure Impact Assessments are required to be carried out where there is concern for the potential of a water storage facility (dam, pond etc) to fail and cause environmental harm or threaten lives. The analysis of flow behaviour resulting from hypothetical dam breaches for each water storage facility have been carried out by way of advanced two dimensional modelling techniques. Analysis was carried out using guidelines specified by the DERM to determine if the impacts would be low, significant or high

1.1 Overall study objective

This assessment has aimed at providing a holistic assessment of existing surface water behaviour within the tenement areas, as well as providing both impact assessment and mitigation/management options to assist in the development of infrastructure items. Figure 1.1 shows the waterway systems within the tenements as well as other key areas that are commonly referenced within this report.

It is noted this report will not directly address geomorphology, water quality or aquatic ecology considerations as these aspects are addressed separately within the technical report entitled “Aquatic ecology, water quality and geomorphology impact assessment – gas fields (Hydrobiology, 2009)” (Volume 5, Attachment 20).



LEGEND

- Town
- Railway stop point
- Pipeline licence (Application)
- Pipeline licence (Granted)
- Existing railway
- Road
- Major watercourse

Walloons Gas Fields Development Areas

- Combabula / Ramyard
- Woleebee
- Carinya
- Condabri
- Talinga / Orana
- Dalwogan
- Kainama
- Gilbert Gully

0 25 50km

SCALE - 1:850,000 (at A3)

Latitude / Longitude
Geocentric Datum of Australia 1994

N W E S

Rev	Date	Revision Description	ORIG	CHK	ENG	APPD
0	15/12/2009	Issued for use	JM	DH		

WorleyParsons resources & energy

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Figure 1.1 Locality Map
Walloons Project Area

Project No: 301001-00448	Figure: 00448-00-EN-DAL-2207	Rev: 0

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2. Legislation

In accordance with Local and State Government legislation, proponents of development projects are required to ensure that potential adverse impacts to receiving environments are mitigated to protect the environmental values of waterways and catchments.

Relevant policy and legislation to the Project in relation to surface water resources are shown in Table 2.1. This list includes the main documents considered as part of this assessment but is not exhaustive.

Table 2.1 Relevant Policy and Legislation

Policy or Legislation	Description	Relevance
Water Act 2000	The *Water Act 2000* provides for the sustainable management of water and other resources. The Water Act regulates the use and allocation of water through water resource plans.	The supply of associated water to third party users is likely to require a water licence under this Act, where water will be supplied to outside of the tenements. Under this Act, a water licence is required for all operations that are not directly related to activities under the *Petroleum and Gas Act* that will interfere with surface water or watercourses.
The Water Supply (Safety and Reliability) Act 2008	The Water Supply (Safety and Reliability) Act 2008 aims to provide for the safety and reliability of water supply.	The *Water Supply (Safety and Reliability) Act 2008* defines the way in which dams are defined and regulated. The legislation puts the onus on particular dam owners to asses the impacts of dam failure by way of completing a 'dam failure impact assessment' if the dam meets criteria requiring assessment. The result of this assessment determines if the dam is referrable under the act. If the dam is classified as referrable, the chief executive of the DERM is responsible for the regulation of the referable dams in Queensland. If a dam contains hazardous waste it is not a referrable dam, and will be governed by the provisions of the *Environmental Protection Act 1994* instead of the *Water Supply Act.* The Water Supply Act requires that an owner of water infrastructure for the supply of water services be registered as a service provider. It is possible that should associated water be supplied to a third party via pipeline or through discharge to a watercourse that such registration may be required.
Sustainable Planning Act 2009	The *Sustainable Planning Act 2009* (SPA) provides the framework for Queensland 's planning and development assessment system.	Schedule 4 of the *Sustainable Planning Regulation 2009* provides that development for petroleum activities is exempt from assessment against a planning scheme. A development permit may be required under SPA for development that is assessable against a planning scheme and for which the petroleum activities exemption does not apply, or if the development is made

Policy or Legislation	Description	Relevance
		assessable under Schedule 3 of the *Sustainable Planning Regulation 2009* , such as development that is operational work for taking or interfering with water or waterway barrier works.
Environmental Protection Act 1994 and Environmental Protection Regulation 2008	The Environmental Protection Act 1994 aims to protect Queensland's environment while allowing for development that improves the total quality of life, both now and in the future, in a way that maintains ecological processes. The Act regulates environmentally relevant activities and outlines procedures for environmental assessment.	The *Environmental Protection Act 1994* requires a project of this nature to operate under an Environmental Authority (Petroleum Activities) (EA). The Environmental Authority approvals process involves an examination of Environmentally Relevant Activities (ERA). It is anticipated that ERA 64 Water treatment and ERA 58 Regulated waste treatment may apply to the Project depending on the volumes and characteristics of associated water and the nature of residues requiring disposal.
Environmental Protection (Water) Policy 2009	The purpose of the Environmental Protection (Water) Policy 2009 is to achieve the object of the Environmental Protection Act 1994 in relation to Queensland waters. The object of the Environmental Protection Act 1994 is to protect Queensland's environment while allowing for development that improves the total quality of life, both now and in the future, in a way that maintains the ecological processes on which life depends.	The Environmental Protection Act 1994 requires a project of this nature to operate under an Environmental Authority (Petroleum Activities) (EA). Associated water is considered a regulated waste under the *Environmental Protection Regulation 2008* (EP Reg). Generally, the storage, treatment or disposal of regulated waste is an 'environmentally relevant activity' (ERA), subject to exceptions. The EA approvals process will involve an examination of ERAs that are petroleum activities. For any ERAs that are not petroleum activities, a development approval may be required. If associated water is treated in accordance with the conditions of a general or specific beneficial use approval, it will no longer be considered "waste" and will not trigger ERAs for dealing with waste / regulated waste. The disposal of hydro-test water is to be undertaken in accordance with the conditions set out in a permit issued by DERM under an Environmental Activity (Petroleum Activities) under the provisions of the Act.
Environmental Protection (Water)	The purpose of the Environmental Protection (Water) Policy 2009 is to achieve the object of the Environmental Protection	Section 6 of this policy describes the environmental values of waters to be enhanced or protected under the policy and Section 7 outlines the indicators and

Policy or Legislation	Description	Relevance
Policy 2009	Act 1994 in relation to Queensland waters. The object of the Environmental Protection Act 1994 is to protect Queensland's environment while allowing for development that improves the total quality of life, both now and in the future, in a way that maintains the ecological processes on which life depends.	water quality guidelines for environmental values. The direct release of waste water to waters is regulated according to section 13 of the Policy; a four step hierarchy of preferred procedures. The last (fourth) step stipulates that if waste water treatment and recycling does not, or is not likely to, eliminate the release of waste water or contaminants to waters, the following options must be evaluated (in order of priority): (i) appropriate treatment and release to waste facility or sewer (ii) appropriate treatment and release to land (iii) appropriate treatment and release to surface waters or groundwater
Environmental Protection (Waste Management) Regulation 2000 (Waste Reg) and Operational Policy - Management of water produced in association with petroleum activities (associated water)	The DERM Operational Policy on the Management of Associated Water (Operational Policy) promotes the beneficial use of associated water in accordance with the Environmental Protection (Waste Management) Policy 2000 (EPP Waste).	The operational policy applies to new applications for non-code compliant environmental authorities (petroleum activities). An application for a non-code compliant environmental authority (petroleum activities) must demonstrate a suitable method for managing associated water. The policy identifies a number of preferred management options (injection, direct use without treatment, treated water use) and non-preferred management options (disposal via evaporation ponds, injection after surface storage or into better quality groundwater, discharge to surface waters). Attached to the operational policy is the general approval for beneficial use of associated water issued under the Waste Regulation. Where associated water produced during the Project is used in accordance with the conditions of the general approval, the associated water will no longer be considered "waste" and may be reused for specific purposes outlined in the general approval. Alternatively, the Waste Reg provides that applications may be made for specific approval for the beneficial use of associated water.
Murray Darling Basin Agreement	Under the Commonwealth Water Act 2007, the Murray Darling Basin Authority (MDBA) is the entity responsible for the	The Project gas fields are predominantly located within the Murray Darling Basin and drain to the Condamine River.

Policy or Legislation	Description	Relevance
	management of water within this Basin and to this end is charged with the responsibility to prepare "The Basin Plan" by 2011. The Basin Plan is to address the following issues: • Limits on water (both surface and ground water) that can be taken • Identification of risks to Basin water resources (e.g. climate change) • A water quality and salinity management plan • An environmental watering plan to optimise environmental outcomes • Matter addressed by state water resource plans (eg water entitlement, environmental flow objectives) • Trading of water rights rules. Water-sharing arrangements that are provided for in existing water resource plans will remain in place until these plans cease, as outlined in the transitional arrangements set out in the Commonwealth Water Act. Under the previous Murray Darling Basin Commission (MDBC), a Basin Salinity Management Strategy for the Basin was put in place. This strategy established targets for river salinity for each tributary valley.	The Project includes release of associated water to the Condamine River. Under this MDBA, to release water into the Murray Darling Basin there will be a need to comply with current MDBA policies and State water resource plans.
DIP Discussion Paper - Management of	The government is yet to finalise its policy related to the disposal and aggregation of CSG water, and is in the process of working with industry and community groups to shape the	Australia Pacific LNG will manage associated water through their Adaptive Associated Water Management Plan. The Plan is discussed in Volume 2 Chapter 12. Management options of the associated water are defined in the plan including

Policy or Legislation	Description	Relevance
water produced from coal seam gas production	final policy framework. The CSG Associated Water Management policy is being developed to provide guidance on the management and disposal of CSG associated water. The draft policy is in response to concerns with current industry practice of using evaporation ponds to deal with associated water, and in particular, with the long-term legacy of the salt stored in the evaporation ponds.	the option to discharge to watercourses which is discussed in detail in the technical report Volume 5, Attachment 23 and summarised in this chapter.
Water Act 2000 - State Water Resource and Resource Operations Plans	Under the *Water Act 2000*, Water Resource Plans (WRPs) have been developed to define the availability and allocation of water and to ensure the sustainable management of water in Queensland. The objectives of the WRPs are to balance the needs of humans and the environment in a sustainable manner. Resource Operations Plans (ROPs) are developed to implement the outcomes and strategies contained within the WRP. The ROPs detail the day –to-day sharing and management of water within the system such that it meets the objective outlined in the WRP.	The proposed discharge locations fall within the bounds of three water resource planning catchments and are likely to be influenced by the following Water Resource Plans (WRPs) and Resource Operations Plans (ROPs): Water Resource (Condamine and Balonne) Plan 2004 and Condamine and Balonne Resource Operations Plan 2008 Water Resource (Fitzroy Basin) Plan 1999 and Fitzroy Basin Resource Operations Plan amended 2009 Water Resource (Border Rivers) Plan 2003 and Border Rivers Resource Operations Plan 2008 While the water resource planning process is primarily focused on the maintenance of flow within the systems and the management of extractions the WRPs do outline performance indicators and Environmental Flow Objectives (EFOs) throughout the river system.

3. Data

Data utilised to determine flooding behaviour within the tenements have been obtained from a variety of sources during the course of the assessment process. The following sub-sections summarise the data that have been used as part of this investigation and detail the sources and accuracy levels (if applicable).

3.1 Topographic data

During the initial stages of the project, topographic information was derived from contour information obtained from the DERM, 2009. The derived contours were at 10m intervals and were used to create a 25m Digital Elevation Model (DEM). The data were deemed adequate for hydrology assessment purposes, however were not deemed to be of sufficient accuracy or detail for the purposes of preparing regional-scale hydraulic models.

As a result, Photogrammetric data was collected by AAMHatch to supplement the topographic data obtained from DERM. The vertical accuracy of the data, as quoted by AAMHatch, is ±1m for measured points[1] and ±1.25m for derived points[2] on clear ground. These values represent standard error (68% confidence level or one sigma), in metres. It is noted that the definition of the ground under trees or in shadows may be less accurate. Given these considerations, the detail and accuracy of the photogrammetric data was considered to be appropriate for the purposes of preparing regional-scaled flooding information for the tenement areas. The photogrammetric data was manipulated by WorleyParsons to convert the raw datasets into various usable formats, including a series of discrete DEM of 5m resolution and traditional contour information of the tenement areas. The data were used as the base topographic dataset for the hydraulic models.

The overall gas fields study area transcends two separate horizontal datum zones. A horizontal datum forms the basis for computations of horizontal positions. As such, depending on the location of the modelling area, the datasets were based upon a horizontal datum either Map Grid of Australia 1994 (MGA94) Zone 55 or Zone 56. All datasets have used a vertical datum of Australian Height Datum (AHD).

3.2 Land use

Land use data for the study area has been based on review of both the Queensland Land Use Mapping Project (QLUMP, 1999) dataset and orthophoto imagery (detailed in Section 3.3). The QLUMP dataset was developed by DERM. Land use is classified according to the Australian Land Use and Management Classification (ALUMC) Version 5, February 2002.

Both of these datasets were used as the basis for development of catchment parameters as part of the hydrologic modelling undertaken for the catchments and tenement study areas.

3.3 Orthophoto imagery

Orthophoto imagery was obtained for the study area by AAMHatch. These datasets are classified as 0.5m Ground Sample Distance (GSD) resolution, and were supplied in the Earth Resource Mapping Enhanced Compression Wavelet (ECW) format.

[1] "Measured Points" are those observed directly.

[2] "Derived Points" are those interpolated from a digital elevation model (DEM).

The aerial photograph datasets were used to determine surface roughness throughout the study areas within the individual hydraulic models and to determine existing infrastructure parameters.

3.4 Rainfall

The design rainfall Intensity-Frequency Duration (IFD) data for various storm events were derived based upon the procedures outlined in Book 2 of Australian Rainfall and Runoff (AR&R) 2001 edition. Appendix A contains the adopted IFD datasets used for the different catchment areas in this study, while Section 5.2.2 summarises the procedures used to create the rainfall datasets.

3.5 Drainage structure details

Details of structures within waterways were obtained from site inspections. Where access to creek systems or roadway crossings was limited, detailed review of aerial photography was undertaken in conjunction with review of surrounding topography. This process identified that the majority of structures within the modelling areas servicing local roads were predominately partially blocked low flow features. As a result, these were deemed to have limited flow transference capabilities and hence negligible impact on flood levels for the regional scale 10, 20, 100 and 500 year ARI events investigated as part of this study. Structures identified as having significant flow transference capabilities and included in the modelling works are summarised in Section 5.3.2.

All large scale regional drainage infrastructure (i.e. bridges, large-scale culvert structures etc) on regional watercourses have been included in each hydraulic model where appropriate. These are summarised in Section 5.3.2.

3.6 Geographic Information System (GIS) Data

Base GIS information for the study area was sourced to aid in the completion of flooding investigations. This information has been utilised specifically for catchment hydrology, hydraulic analysis and mapping tasks. In particular, the following GIS information was used:

- GeoScience Australia – Native Vegetation Layers
- Queensland Land Use Mapping Project (QLUMP, 1999)
- General Detail – Towns, Roads, Watercourses, Tenement boundaries etc (various sources)

As the overall gas fields study area transcends two separate horizontal datum zones, depending on the extent and location of the datasets, the GIS data were based on a horizontal datum either Map Grid of Australia 1994 (MGA94) Zone 55 or Zone 56.

3.7 Previous flood study

Whilst a limited number of detailed flood studies have been undertaken for some discrete areas of the major watercourses upstream and downstream of the gas fields study area, there has been one recent flood study of relevance to the study area. This is summarised in Section 3.7.1.

3.7.1 Flood investigation for Talinga Coal Seam Gas Development, WorleyParsons (August 2008)

This study was undertaken by WorleyParsons to determine flood behaviour over the PL226 tenement.

Study results found the 100 year ARI event resulted in significant inundation within areas adjacent to the two major watercourses flowing through the tenement, namely the Condamine River and Wieambilla Creek.

Inundation typically resulted from regional flow breakouts form the aforementioned watercourses into surrounding low lying areas.

Local flows from smaller tributaries higher in the catchment and study area were also shown to contribute to inundation of significant areas during the 100 year ARI event, however this flooding was typically of shallow depth and low velocity.

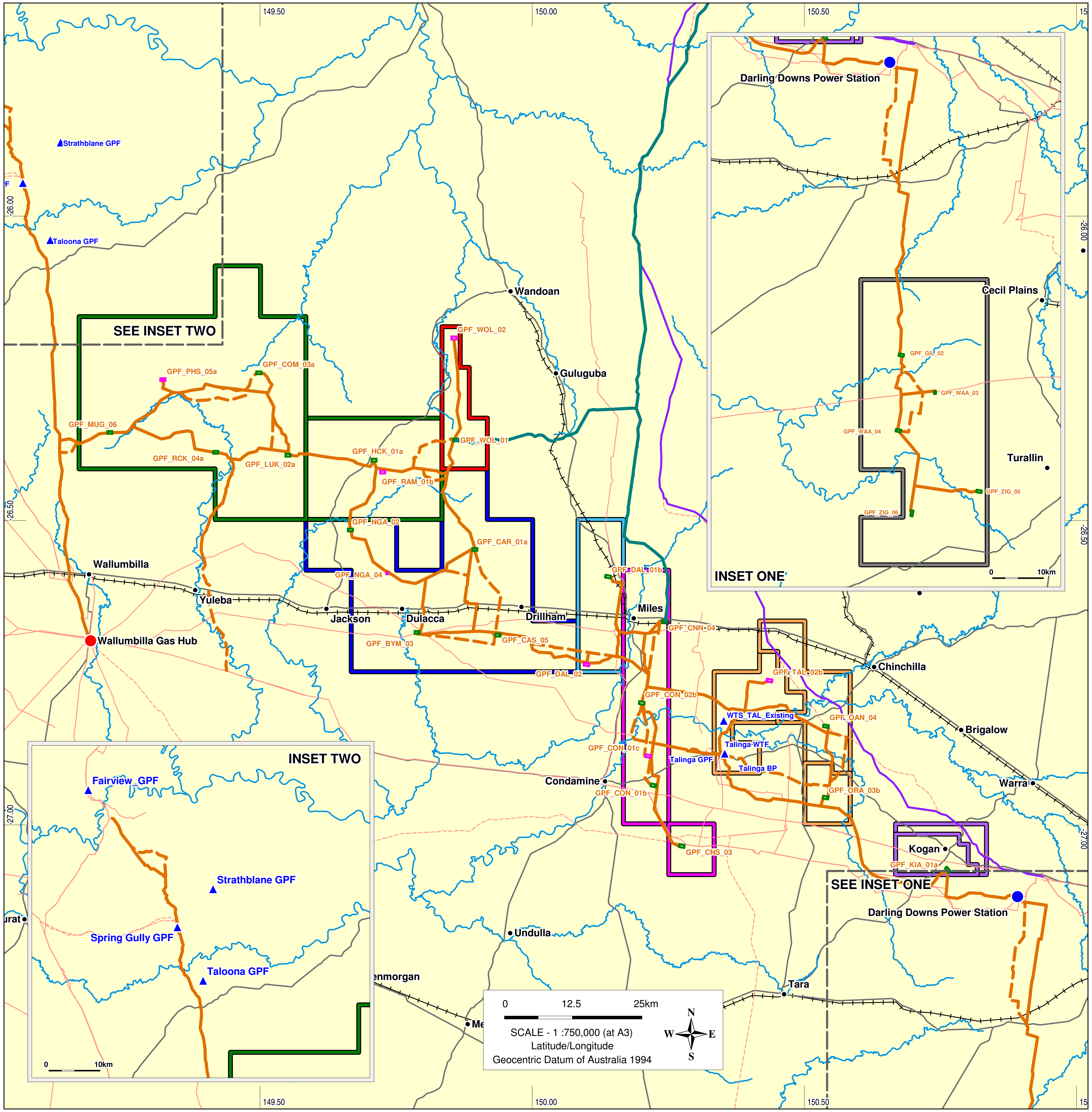
4. Existing environment description

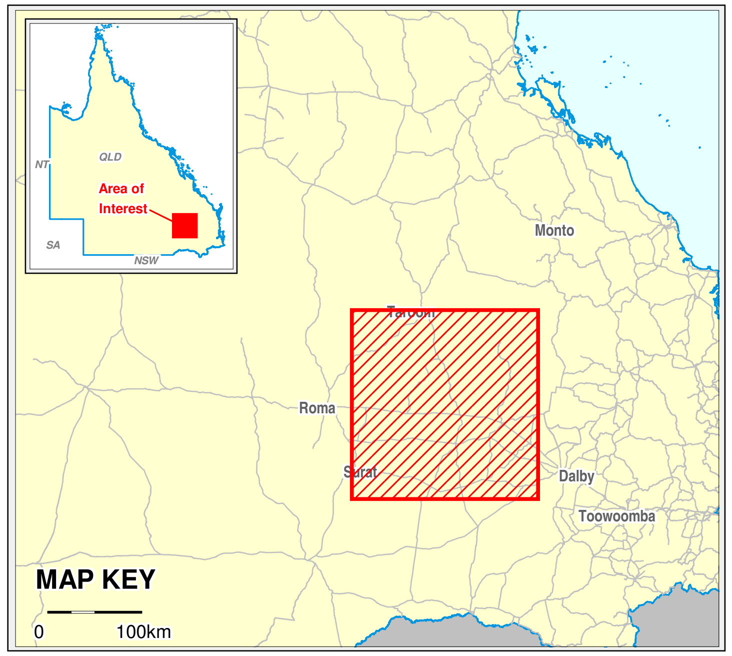
4.1 Tenement locations and infrastructure areas

The gas fields are located within the Miles - Wallumbilla - Wandoan region, with two smaller separate gas fields located at Kogan (south-east of Chinchilla) and the Gilbert Gully area (south-west of Cecil Plains). Twenty-three gas processing facilities, six water treatment facilities, thirty three water transfer stations and three brine ponds are identified as potential sites.

Each water treatment facility has a maximum anticipated footprint of 200m x 200m (4 ha) while the gas processing facilities have a maximum anticipated footprint of 1,000m x 500m (50ha).

The locations of the gas processing and water treatment facilities are shown in Figure 4.1 and Figure 4.2, respectively.





LEGEND

- Town
- Wallumbilla gas hub
- Major drainage
- Pipeline licence (Application)
- Pipeline licence (Granted)
- Existing infrastructure
- Darling Downs power station
- Road
- Railway

Walloons Gas Fields Development Areas

- Talinga / Orana
- Dalwogan
- Kainama
- Gilbert Gully
- Combabula / Ramyard
- Woleebee
- Carinya
- Condabri

Proposed Infrastructure

- High pressure gas pipelines
- Alternative high pressure gas pipelines
- Proposed Surat to Gladstone pipeline
- Gas pipeline route
- Gas processing facility
- Alternate gas processing facility

Rev	Date	Revision Description	ORIG	CHK	ENG	APPD
0	03/03/2010	Issued for use	GSB	MZ	AS	RB

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AUSTRALIA PACIFIC LNG PTY LIMITED

AUSTRALIA PACIFIC LNG PROJECT

Figure 4-1: Gas processing facility locations

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-0462 | Rev: 0

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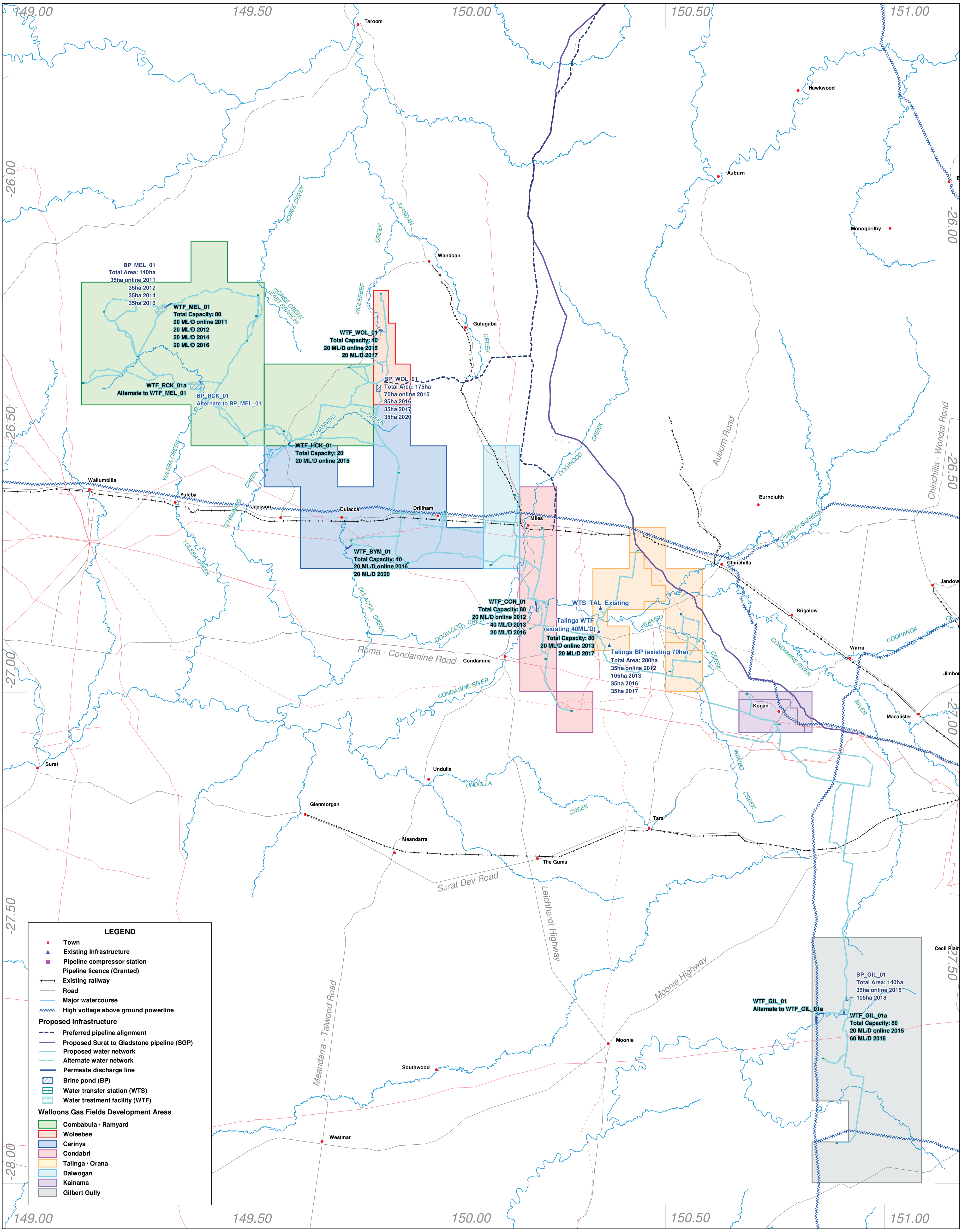

SGP pipeline route digitised from Initial Advice Statement dated December 2008
Preferred pipeline alignment provided by Origin Energy 03/11/2009.
Gas processing facility capacity data provided by A. Skelley on 07/09/2009.

0 12.5 25km
SCALE - 1 : 400,000 (at A1)
Latitude - Longitude
Geocentric Datum of Australia 1994

N W E S

Rev	Date	Revision Description	ORIG	CHK	ENG	APPD
0	14/09/2009	Issued for use	NA	KM	AS	RB

WorleyParsons
resources & energy

AUSTRALIA PACIFIC LNG PTY LIMITED

AUSTRALIA PACIFIC LNG PROJECT
Figure 4-2: Water treatment facility locations

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-0171 | Rev: 0

Compiled by BRISBANE INFRASTRUCTURE GIS SECTION

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4.2 Existing environment description

The landscape characteristics at all of the proposed gas processing and water treatment facility sites are summarised in Table 4.1. Around half of the sites are located on the crests of hills or on raised areas of land, and therefore will not receive runoff from an upstream catchment. A brief description of the area around each water treatment and gas processing facility is presented in Section 4.3.

The site descriptions include references to stream order. The Queensland AusRivAS Sampling and Processing Manual (DNRM, 2001) defines stream order as a hierarchical ordering system based on the degree of branching. For example:

a) A first-order stream does not have tributary branches.

b) Two first-order streams flow together to form a second-order stream.

c) Two second-order streams combine to make a third-order stream.

d) Each time a bifurcation node is encountered, the order is increased by one if both upstream branches have the same order, otherwise the larger order is used.

First through third order streams are also called headwater streams and constitute any waterways in the upper reaches of the watershed. Streams that are classified as fourth through sixth order are medium streams while anything larger (up to 12th order) is considered a river (Strahler, 1957).

The Aquatic Ecology, Water Quality and Geomorphology Impact Assessment – Gas Fields (Hydrobiology 2009) (Volume 5, Attachment 20) includes information regarding but is not limited to water quality (including monitoring), geomorphic, and descriptions of environmental values of the major waterways within the gas fields study area.

Table 4.1 Facility location description

Facility	Site	Figure	Catchment	Sub-catchment	Waterways	Primary Land Use	Topography
Water Treatment Facility	WTF_MEL_01	Figure 4.3	Dawson River	Kangaroo Creek	Appletree Creek (SE) Unnamed tributary of Kangaroo Creek (NW)	Grazing Cleared, sparse trees along creeks	Undulating-hills, 50-60m change in elevation
Water Treatment Facility	WTF_RCK_01a	Figure 4.6	Condamine River	Yuleba Creek	Ten Mile Creek (through N) Yuleba Creek (E)	Grazing Cleared, sparse trees along creeks Forestry in headwaters	Low-undulating with steeper headwater areas
Water Treatment Facility	WTF_HCK_01	Figure 4.9	Condamine River	Tchanning Creek	Noonga Creek (W) Tchanning Creek (S)	Cleared grazing, forestry in upper reaches	Low-undulating
Water Treatment Facility	WTF_BYM_01	Figure 4.11	Condamine River	Dulacca Creek	Seven Mile Creek (E) Dulacca Creek (W)	Cropping, grazing, cleared	Very low slope
Water Treatment Facility	WTF_WOL_01	Figure 4.13	Dawson River	Woleebee Creek	Unnamed tributary of Woleebee Creek (N) Woleebee Creek (W)	Grazing, cleared	Low-undulating, hills in headwaters
Water Treatment Facility	WTF_CON_01	Figure 4.20	Condamine River	Condamine River	Unnamed tributary of Condamine River (S through site)	Cleared grazing, irrigated pasture, cropping	Very low slope
Water Treatment	WTF_GIL_01a	Figure 4.26	Border Rivers	Weir River	Unnamed tributary of Weir River (E and W)	Cleared grazing, forestry	Low undulating (no contours)

Facility	Site	Figure	Catchment	Sub-catchment	Waterways	Primary Land Use	Topography
Facility					Weir River (S)		
Water Treatment Facility	WTF_GIL_01	Figure 4.28	Border Rivers	Weir River	Cattle Creek (W)	Cleared grazing	Low undulating (no contours)
Gas Processing Facility	GPF_PHS_05a	Figure 4.3	Dawson River	Kangaroo Creek	Spring Creek (SE) Appletree Creek (NW)	Grazing Cleared, sparse trees along creeks	Undulating-hills, 50-60m change in elevation
Gas Processing Facility	GPF_MUG_06	Figure 4.4	Condamine River	Yuleba Creek Blythe Creek	Cattle Creek (SW - distance) Unnamed tributary of Yuleba Creek (N – from site) Coxon Creek (W – from site)	Grazing Cleared land	Low, undulating land
Gas Processing Facility	GPF_COM_03a	Figure 4.5	Dawson River	Horse Creek (Main and West Branches)	Horse Creek West Branch (N) Unnamed tributary of Horse Creek Main Branch (S)	Plantation forestry (top of catchment only), grazing (cleared)	Undulating-hills, 50-60m change in elevation
Gas Processing Facility	GPF_RCK_04a	Figure 4.6	Condamine River	Yuleba Creek	Ten Mile Creek (N) Yuleba Creek (E) Unnamed tributary of	Grazing Cleared, sparse trees along creeks	Low-undulating with steeper headwater areas

Facility	Site	Figure	Catchment	Sub-catchment	Waterways	Primary Land Use	Topography
					Yuleba Creek (S)	Forestry in headwaters	
Gas Processing Facility	GPF_LUK_02a	Figure 4.7	Condamine River	Yuleba Creek	MacNally Creek (NW)	Grazing, plantation forestry	Low-undulating, 30-40m change in elevation
Gas Processing Facility	GPF_RAM_01b	Figure 4.8	Condamine River	Tchanning Creek	Clark Creek (W)	Grazing (cleared), Forestry in head waters	Low-undulating
Gas Processing Facility	GPF_HCK_01a	Figure 4.8	Condamine River	Tchanning Creek	Clark Creek (W)	Grazing (cleared), Forestry in head waters	Low-undulating
Gas Processing Facility	GPF_NGA_02	Figure 4.9	Condamine River	Tchanning Creek	Unnamed tributary of Tchanning Creek (N) Tchanning Creek (N)	Forestry, grazing	Undulating
Gas Processing Facility	GPF_NGA_04	Figure 4.10	Condamine River	Dulacca Creek	Wellcamp Creek (S)	Cropping, grazing	Very low-low undulating
Gas Processing Facility	GPF_BYM_03	Figure 4.11	Condamine River	Dulacca Creek	Seven Mile Creek (E) Dulacca Creek (W)	Cropping, grazing, cleared	Very low slope
Gas Processing Facility	GPF_WOL_02	Figure 4.12	Dawson River	Woleebee Creek	Unnamed tributary of Woleebee Creek (N)	Grazing, cleared	Very low undulating
Gas	GPF_WOL_01	Figure 4.14	Dawson River	Woleebee Creek	Unnamed tributary of	Grazing, cleared	Undulating-hills

Facility	Site	Figure	Catchment	Sub-catchment	Waterways	Primary Land Use	Topography
Processing Facility					Ramyard Creek (W) Unnamed tributary of Woleebee Creek (E)		
Gas Processing Facility	GPF_CAR_01a	Figure 4.15	Condamine River	Dogwood Creek	Wallan Creek (E)	Grazing, cleared	Very low slope
Gas Processing Facility	GPF_CAS_05	Figure 4.16	Condamine River	Dulacca Creek	Range Creek (W)	Forested, cleared grazing	Very low undulating
Gas Processing Facility	GPF_DAL_01b	Figure 4.17	Condamine River	Dogwood Creek	Unnamed tributary of Eleven Mile Creek (E)	Forestry, some grazing	Undulating
Gas Processing Facility	GPF_DAL_02	Figure 4.18	Condamine River	Dogwood Creek	Tomahan Gully (N and E)	Cleared grazing, some forestry in headwaters	Very low slope
Gas Processing Facility	GPF_CNN_04	Figure 4.19	Condamine River	Dogwood Creek	Unnamed tributary of Dogwood Creek (N) Unnamed tributary of Columboola Creek (S)	Township of Miles, grazing, forestry, gas fields (eastern headwaters)	Low-undulating
Gas Processing Facility	GPF_CON_02b	Figure 4.20	Condamine River	Condamine River	Unnamed tributary of Condamine River (S through site)	Cleared grazing, irrigated pasture, cropping	Very low slope
Gas Processing	GPF_CON_01b	Figure 4.21	Condamine River	Condamine River	Unnamed tributary of Condamine River (N)	Cleared grazing, forestry in headwaters	Very low slope

Facility	Site	Figure	Catchment	Sub-catchment	Waterways	Primary Land Use	Topography
Facility					Unnamed tributary of Cooloomala Creek (S)		
Gas Processing Facility	GPF_CON_01c	Figure 4.21	Condamine River	Condamine River	Unnamed tributary of Condamine River (N)	Cleared grazing, irrigated pasture, cropping	Very low slope
Gas Processing Facility	GPF_CNS_03	Figure 4.22	Condamine River	Condamine River	Unnamed tributary of Cooloomala Creek (W) Unnamed tributary of Cobbareena Creek (E)	Cleared grazing, forestry in headwaters	Flat-low slope
Gas Processing Facility	GPF_OAN_04	Figure 4.23	Condamine River	Condamine River	Unnamed tributary of Condamine River (N)	Cleared irrigated pastures, cropping	Flat
Gas Processing Facility	GPF_ORA_03b	Figure 4.24	Condamine River	Wieambilla Creek	Unnamed tributary of Nine Mile Creek (NW)	Forestry in headwaters, grazing in basin	Undulating at headwaters, flat-low slope in basin
Gas Processing Facility	GPF_KIA_01a	Figure 4.25	Condamine River	Kogan Creek	Unnamed tributary of Kogan Creek (W and N)	Forested, grazing	Undulating (no contours)
Gas Processing Facility	GPF_GIL_02	Figure 4.26	Border Rivers	Weir River	Unnamed tributary of Weir River (E and W) Weir River (S)	Cleared grazing, forestry	Low undulating (no contours)
Gas Processing	GPF_WAA_03	Figure 4.27	Border Rivers	Weir River	Teatree Gully (S)	Forested	Undulating (no contours)

Facility	Site	Figure	Catchment	Sub-catchment	Waterways	Primary Land Use	Topography
Facility							
Gas Processing Facility	GPF_WAA_04	Figure 4.29	Border Rivers	Weir River	Unnamed tributary of Jib Creek (NE)	Cleared grazing, irrigated pasture, forestry at mid-reaches	Low undulating (no contours)
Gas Processing Facility	GPF_ZIG_05	Figure 4.30	Border Rivers	Paddy Creek	Unnamed tributaries of Paddy Creek (N and S)	Forestry (mid-reaches), some cleared grazing (headwaters)	Low undulating (no contours)
Gas Processing Facility	GPF_ZIG_06	Figure 4.31	Border Rivers	Western Creek	Scrubby Creek (N)	Forestry	Undulating-hills (no contours)

4.3 Desktop assessment of infrastructure areas

Table 4.2 describes the various rates of stormwater infrastructure required. These rates are used as a guide for stormwater and runoff management at each site, based on the landscape characteristics at each location. Onsite stormwater systems will include bunding around chemical loading and storage areas to remove the risk of chemical contamination, plus swales and sediment basins to capture and treat onsite stormwater. These methods are detailed further in Section 6. Methods of diverting upstream runoff include constructing a suitably sized swale or interception trench along the upstream edge of the facility. These trenches are placed at an appropriate grade across the fall line of the sub-catchment to divert water around the facility. Where the facility is located in a focused runoff path, a larger version of the swale will be required to carry greater volumes.

Table 4.2 Stormwater management requirement ratings

Rate	Requirement	Reason
Minimal	Onsite stormwater system only	Top of hill or rise, no uphill runoff influence
Moderate	Onsite stormwater system plus some diversion of upstream runoff.	Located on the side of a gently sloping hill or grade, with some runoff expected
Focused	Onsite stormwater system plus significant diversion of uphill runoff.	Located at base of or within drainage line, with focused or large quantity of runoff expected
Erosion management	Dedicated erosion management for stormwater flows leaving the site, eg scour protection	Where slope may increase velocity of stormwater runoff from the facility and increase scour occurrence

The gas field tenements generally consist of grazing, cropping or vegetated land, with a network of small gullies, creeks and streams throughout the Condamine-Balonne and Dawson River catchments. Many of these creeks and streams are ephemeral in nature, being dry or containing unconnected standing pools of water on a regular basis in response to the rainfall patterns of the region.

WTF_MEL_01 and GPF_PHS_05a

There are two creeks adjacent to the proposed location of WTF_MEL_01; a stream order 1 watercourse to the north-west (unnamed, flowing into Kangaroo Creek), and a stream order 2 watercourse to the south-east (Appletree Creek) (refer Figure 4.3). The site is located on a ridge that predominately falls to the south-east. The creeks are ephemeral. The site has grass cover, with a few trees throughout the grazing properties and along riparian areas. As the site is located on top of the ridge it will not receive stormwater runoff from external catchments. *Stormwater management rating = minimal*.

There are two creeks adjacent to the proposed location of GPF_PHS_05a (refer Figure 4.3). There is a stream order 1 to the north-west (Appletree Creek), and a stream order 1 to the south-east (Spring Creek). There is a small ridge falling from south west to north east. The land is predominantly flat beyond the ridge. The creeks are ephemeral. The site has grass cover, with few trees throughout the grazing properties and along riparian areas. Stormwater drainage will need to be directed overland towards the two creeks, with Spring Creek likely to be the preferred direction of flow. *Stormwater management rating = minimal*.

GPF_MUG_06

Site GPF_MUG_06 has three stream order 1 waterways adjacent to or located nearby (refer Figure 4.4). Cattle Creek is to the south east (about 1.4km away). Coxon Creek flows from the site to the west. An unnamed creek flows from the site northwards to Yuleba Creek. These creeks are all ephemeral. The site is predominantly flat and consists of cleared grazing lands. Stormwater flows into either Coxon Creek or Yuleba Creek. Assessment of contour information indicates Coxon Creek is slightly steeper in slope than the Yuleba Creek tributary. Any changes to stormwater flow that occur would need to take into consideration the potential for erosion as a result of a concentrated flow, as opposed to a generalised sheetflow that occurs naturally on the site. *Stormwater management rating = minimal plus erosion management*.

GPF_COM_03a

There are two creeks adjacent to the proposed GPF_COM_03a site (refer Figure 4.5). The unnamed creek to the south is a stream order 1 waterway that flows into the main branch of Horse Creek, while Horse Creek (west branch) is a stream order 2 to the north of the site. Site slope falls at less than 1% both north and south of the proposed site. The creeks are ephemeral. The site is currently vegetated with closed eucalypt stands. The site will not receive stormwater runoff from an external catchment. Erosion control during vegetation clearing will be required to ensure no downstream impacts occur. *Stormwater management rating = minimal plus erosion management*.

WTF_RCK_01a and GPF_RCK_04a

Ten Mile Creek (stream order 2) falls within the site allocated for the proposed WTF_RCK_01a flowing west to east. This site is an alternative site to WTF_MEL_01. Yuleba Creek is located east of the site. The proposed WTF site is located on the side of a ridge (refer Figure 4.6) with focused drainage through the western area. Due to its location stormwater runoff will need to be managed via diversion drains. The site currently consists of grazing lands. *Stormwater management rating = moderate to focused (subject to final location).*

There is an unnamed stream order 1 watercourse to the south of proposed GPF_RCK_04a flowing into Yuleba Creek. Stormwater generated from this facility will be directed into the Yuleba Creek tributary. *Stormwater management rating = minimal.*

GPF_LUK_02a

Two unnamed ephemeral stream order 1 creeks are located adjacent to GPF_LUK_02a, flowing into MacNally Creek to the west (Figure 4.7). One branch flows west-north-west from the site, while another branch is adjacent to the south-west boundary. The site itself slopes very gently (<1%) to the north, therefore the majority of surface runoff would enter the west-north-west flowing branch of MacNally Creek. The land consists of trees and cleared grazing areas. *Stormwater management rating = minimal*.

GPF_RAM_01b and GPF_HCK_01a

While Clark Creek is the closest mapped ephemeral watercourse to GPF_RAM_01b, at approximately 2.5km to the west-north-west, contour levels indicate that there is a small depression system leading from the proposed site to Clark Creek. The land slopes from east to west. Areas uphill of the site on the north and eastern boundaries, are heavily vegetated. To the south of the site is grazing land, with forest further up on the hill. *Stormwater management rating = moderate*.

GPF_HCK_01a is located north of GPF_RAM_01b (refer Figure 4.8). GPF_HCK_01a is bordered by two creeks. The stream order 3 Clark Creek runs along the western border, while an unnamed creek (stream order 1) borders the north of the site, flowing into Clark Creek. The creeks are ephemeral. The site lies on approximately a 1.3% slope from south east to north-west (into Clark Creek). Runoff from uphill is likely to occur and therefore is a potential issue which will require diversion around the proposed facility. Vegetation at the site consists of open grassland (grazing) with few trees along Clark Creek only. *Stormwater management rating = moderate*.

WTF_HCK_01 and GPF_NGA_02

WTF_HCK_01 lies on open grazing land, sloping south to Tchanning Creek. Noonga Creek lies to the west of the site, however from the contours it appears not to receive any runoff from the proposed site. The site is connected to Tchanning Creek by a small eroded depression. There will be some diversion of upslope runoff required. However, due to the flatness of the area, diversions should consist of only minor earthworks. The upslope area consists of open grazing and open eucalypt woodland. *Stormwater management rating = moderate*.

An unnamed stream order 1 watercourse traverses the proposed GPF_NGA_02 site, which is in the base of a horseshoe shaped ridge (refer Figure 4.9). All runoff from east, west and south of the site flows into and through the proposed facility area. The stream then flows north into Tchanning Creek. Stormwater runoff will need to be diverted around the proposed site. Vegetation higher in the local catchment is cleared grazing and eucalypt woodland. *Stormwater management rating = focused*.

GPF_NGA_04

A gully links the south of the proposed GPF_NGA_04 site to Wellcamp Creek, a stream order 1 watercourse approx 1.5km from the site (refer Figure 4.10). The site itself is in a slight depression between two small ridges, which would require diversion of uphill runoff from the east and west. The land is currently used for cropping, which would have a higher sediment transport in runoff than other sites. Stormwater runoff would flow from the site to the south into Wellcamp Creek. *Stormwater management rating = focused plus erosion management*.

WTF_BYM_01 and GPF_BYM_03

The proposed WTF_BYM_01 and GPF_BYM_03 sites are located on flat cropping land, with a few gullies leading to the south west into Dulacca Creek, about 3km from the site (refer Figure 4.11). An additional, unnamed, stream order 1 watercourse lies to the east of the site and flows into Seven Mile Creek; however the main direction of flow is towards Dulacca Creek. Both creeks are ephemeral. *Stormwater management rating = minimal (WTF) and moderate (GPF)*.

GPF_WOL_02

The proposed GPF_WOL_02 site is located on gently sloping grazing land, south of a stream order 1 tributary of Woleebee Creek (refer Figure 4.12). The tributary lies adjacent to the north border of the proposed site and receives stormwater runoff. Diversion of stormwater runoff is likely to be required to the east of the site to prevent contamination of clean stormwater. *Stormwater management rating = focused.*

WTF_WOL_01

An unnamed stream order 1 watercourse flows from south to north near the eastern boundary of the proposed WTF_WOL_01 site and eventually flows into Woleebee Creek (refer Figure 4.13). There is approximately a 3% slope of the land to the north-east into the watercourse. Diversions of stormwater

will be required to the south and west of the facility to ensure runoff from uphill of the facility does not become contaminated. Stormwater from the facility would also be directed into the tributary following treatment. *Stormwater management rating = moderate plus erosion management*.

GPF_WOL_01

Figure 4.14 shows the proposed GPF_WOL_01 site to be located on a saddle between two steep sections of a ridgeline. A tributary of Ramyard Creek falls to the west of the site, while a tributary of Woleebee Creek lies to the east. A majority of the proposed area falls within the Woleebee Creek catchment, and therefore it is likely that this would be the appropriate direction of stormwater runoff from the facility. Diversions of uphill runoff will be required to the north and south of the facility to ensure contamination of runoff from those areas does not occur. The southern ridgeline is heavily vegetated, while the remainder of the area is cleared grazing land. *Stormwater management rating = focused plus erosion management*.

GPF_CAR_01a

There is an unnamed stream order 1 watercourse to the north of the GPF_CAR_01a site, flowing east into Wallan Creek (refer Figure 4.15). The site is located on a hill with a 0.3% slope to the north, south and east, indicating that no diversion of external runoff will be required, and only site-sourced stormwater will require capture and treatment prior to release. The vegetation consists of cleared grazing land. *Stormwater management rating = minimal*.

GPF_CAS_05

Range Creek is a stream order 1 waterway to the west of the proposed GPF_CAS_05 site, with a small depression leading from the site to Range Creek (refer Figure 4.16). The site is located on open vegetated land downhill from a vegetated hill. Some diversion is required for runoff from the hill to the south of the site, which will be split to the east and west of the facility. *Stormwater management rating = focused*.

GPF_DAL_01b

The proposed GPF_DAL_01b site is located on the top of a vegetated ridge, with an ephemeral, unnamed stream order 1 watercourse taking runoff from the site to the north east (refer Figure 4.17), which flows into Eleven Mile Creek. *Stormwater management rating = minimal plus erosion management*.

GPF_DAL_02

The proposed GPF_DAL_02 site lies on cleared grazing areas in a flat (approximately 0.04% slope) location, sloping to the east (refer Figure 4.18). An unnamed ephemeral stream order 2 watercourse is located to the north of the site that flows to the east into Tomahan Gully (stream order 3). There are several observed erosion areas within the watercourses and the surrounding lands. Any stormwater management for this site will need to ensure that runoff from the facility does not exacerbate the erosion. *Stormwater management rating = minimal plus erosion management*.

GPF_CNN_04

The proposed GPF_CNN_04 site lies on the top of a forested ridge, with a 3% slope from the edge of the site to the north and south-west (refer Figure 4.19). There is an un-named ephemeral stream order 1 watercourse to the south of the site that flows south-west into a soak area, eventually making its way into Columboola Creek (7km from the site). An unnamed stream order 1 watercourse also lies 1.3km

to the north of the site, flowing into Dogwood Creek. Any runoff flowing north from the site would need to cross an unsealed road and stormwater controls, for example a culvert or causeway, may need to be constructed to minimise erosion and sediment transport. Both creeks are ephemeral. *Stormwater management rating = minimal plus erosion management*.

WTF_CON_01 and GPF_CON_02b

The WTF_CON_01 site lies downstream from GPF_CON_02b, with an unnamed ephemeral stream order 1 watercourse flowing through the facility area towards the Condamine River (refer Figure 4.20). The WTF site is located at the base between two hills, and therefore the final design of the facility will be further assessed so as to not interfere with either the watercourse or the immediate floodplain. GPF_CON_02b is located on flat open grazing land, at the headwaters of the tributary to the Condamine River. Erosion is evident within the area. The land consists of open grazing and few trees lining the watercourse. *Stormwater management rating = focused (WTF) and minimal plus erosion management (GPF)*.

GPF_CON_01b and GPF_CON_01c

There is an unnamed ephemeral stream order 2 watercourse to the north of site GPF_CON_01b flowing west into the Condamine River and a tributary of the Cooloomala Creek located to the south of the site (refer Figure 4.21). The site is positioned on very low sloping, open grazing land, while upstream areas are vegetated. Minor diversions of runoff from the east may be required. Additionally, there is an unnamed ephemeral stream order 1 watercourse to the north of the proposed GPF_CON_01c site that flows into the Condamine River. The gas processing site is located on open grazing lands with a 1% slope to the north. There appears to be no runoff from uphill areas into the proposed gas processing site. *Stormwater management rating = minimal*.

GPF_CSN_03

There is an unnamed stream order 1 ephemeral watercourse to the east of the proposed GPF_CSN_03 site, flowing from open grazing land into vegetated areas to the north of the site and into Cobbareena Creek (refer Figure 4.22). Additionally, there is an unnamed tributary of Cooloomala Creek to the west of the proposed site. The GPF site is on flat land, with no runoff diversion required. *Stormwater management rating = minimal*.

GPF_OAN_04

GPF_OAN_04 is located on flat, open grazing and cropping land, with a stream order 1 watercourse flowing from the site north-west into the Condamine River (refer Figure 4.23). No diversion of uphill runoff is required. *Stormwater management rating = minimal*.

GPF_ORA_03b

The proposed GPF_ORA_03b site is located near the top of a small vegetated hill, sloping to the west. An unnamed stream order 1 ephemeral watercourse lies to the north of the site and flows west into Nine Mine Creek. Minimal diversion of upstream runoff will be required due to the location of the proposed site (refer Figure 4.24). *Stormwater management rating = moderate plus erosion management*.

GPF_KIA_01a

GPF_KIA_01a is located on sparsely vegetated land, with a stream order 1 creek to the south-west that flows into Kogan Creek (refer Figure 4.25). Diversion may be required to manage overland flow from the adjacent upstream closed forest area. *Stormwater management rating = focused.*

WTF_GIL_01a and GPF_GIL_02

There are two unnamed creeks associated with WTF_GIL_01a and GPF_GIL_02 (refer Figure 4.26). A stream order 1 lies to the south-west, while a medium sized unnamed stream lies to the south-east. Both unnamed streams flow into Weir River. The sites are located on a small ridge that falls to the south. Stormwater runoff would likely be diverted to the south-east for both facilities. *Stormwater management rating = focused (WTF) and moderate (GPF)*.

GPF_WAA_03

Teatree Gully is an ephemeral watercourse to the south of the proposed GPF_WAA_03 site (refer Figure 4.27). The catchment is heavily vegetated, however, the site is located on open grazing land. Some diversion may be required to manage uphill runoff. *Stormwater management rating = moderate*.

WTF_GIL_01

There is an ephemeral stream order 1 watercourse to the west of the proposed WTF_GIL_01 site that flows into Cattle Creek (refer Figure 4.28). The land is reasonably flat and consists of open grazing land with scattered trees. This is an alternative site to WTF_GIL_01a. *Stormwater management rating = minimal*.

GPF_WAA_04

An unnamed ephemeral stream order 3 tributary of Jib Creek is located to the north east of the GPF_WAA_04 site. The land is open grazing with a low slope starting to the north of the site (refer Figure 4.29). Stormwater from the facility would be diverted into this tributary. *Stormwater management rating = moderate*.

GPF_ZIG_05

GPF_ZIG_05 is located on grazing land, while the catchment area is heavily vegetated. Two creeks are adjacent to the site, both flowing into Paddy Creek to the west. An unnamed stream order 1 is to the north, and an unnamed stream order 4 is to the south (refer Figure 4.30). Stormwater runoff would be directed into the southern tributary. Some diversion may be required to ensure runoff from the north-east does not flow through the facility. *Stormwater management rating = moderate*.

GPF_ZIG_06

GPF_ZIG_06 is located within a heavily vegetated catchment. Scrubby Creek is a stream order 1 ephemeral waterway flowing from the site to the north (refer Figure 4.31). The site is on relatively flat terrain, with minimal uphill diversion required for the site. *Stormwater management rating = minimal plus erosion management*.

4.4 Site constraints and opportunities

Most sites are unconstrained for development and associated stormwater management and can be constructed with basic treatment devices that are replicated at each site. However, some facilities

would require site-specific management techniques due to the location of the facility within the terrain. The installation of diversion drains at some sites is key to ensure runoff from upstream catchments does not enter the site.

The water treatment facilities have a smaller footprint than the gas processing facilities, and thus the scale of stormwater management would vary between the two generic facility designs. Vegetated swales and sedimentation basins will be provided for each site principally to capture sediment from runoff. The management of the devices needs to take into consideration the dispersive, silty clay nature of the soils in the region.

4.5 Catchment and watercourse descriptions at discharge locations

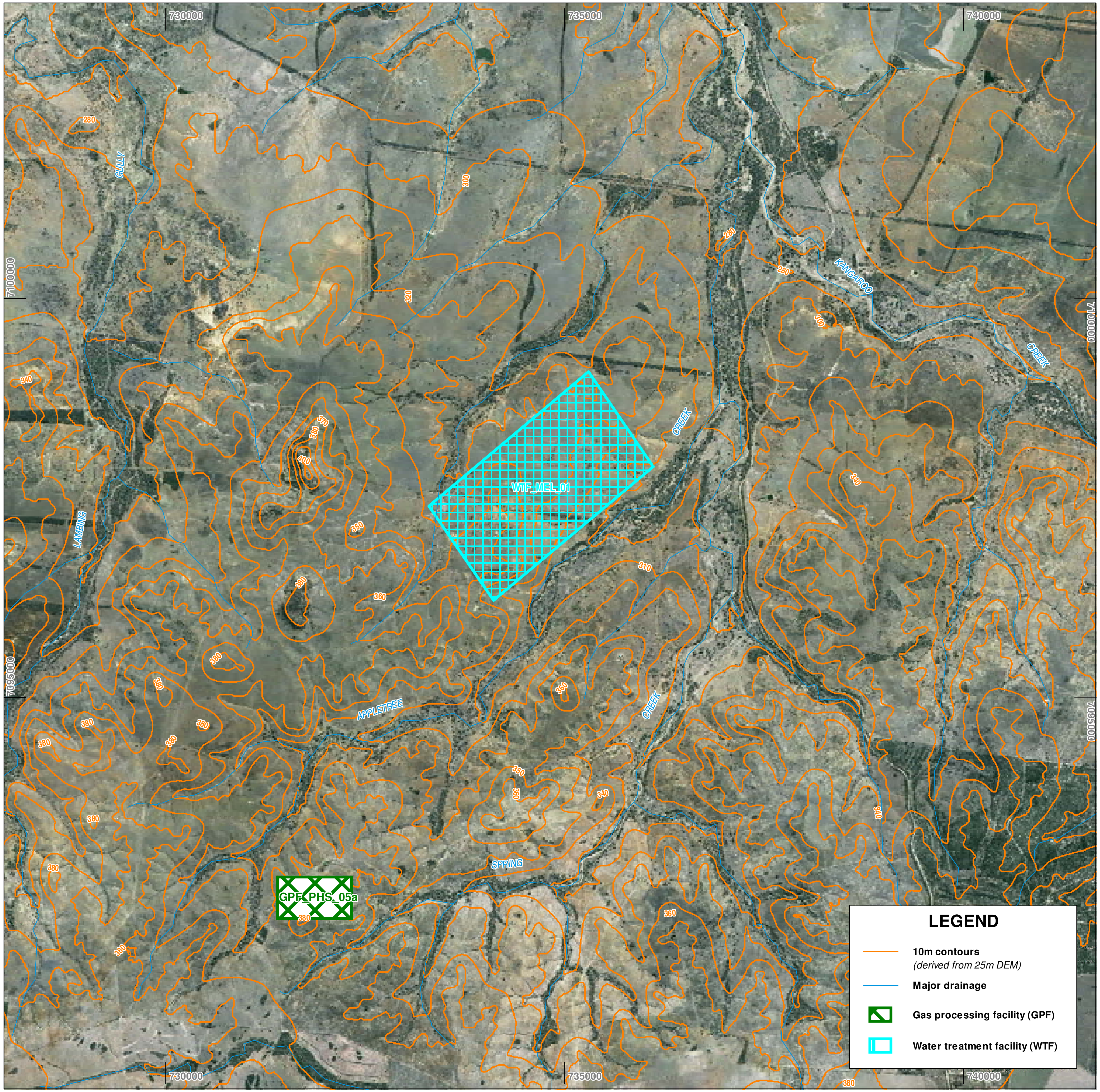
Table 4.3 provides a summary of the characteristics of the watercourses at which discharge is proposed.

Table 4.3 Discharge location description

Site	Catchment	Waterway	Primary Land Use	Topography	Channel Shape[1]	Bed Stability Rating[1]	Dominant Disturbance[1]	Overall Disturbance Rating[1]	Average Width[1]
WTF_MEL_01	Dawson River	Unnamed tributary of Kangaroo Creek (NW)	Grazing Cleared, sparse trees along creeks	Undulating-hills, 50-60m change in elevation	No information	No information	No information	No information	No information
WTF_RCK_01a	Condamine River	Yuleba Creek	Grazing Cleared, sparse trees along creeks Forestry in headwaters	Low-undulating with steeper headwater areas	U shape	Stable to moderate aggradation	Sediment-laden runoff	High to Very High	21-26 m
WTF_HCK_01	Condamine River	Tchanning Creek	Cleared grazing, forestry in upper reaches	Low-undulating	Flat U-Shape	Moderate aggradation	Sediment-laden runoff, Stock	Very High	50-56 m
WTF_BYM_01	Condamine River	Dulacca Creek	Cropping, grazing, cleared	Very low slope	No information	No information	No information	No information	No information
WTF_WOL_01	Dawson River	Woleebee Creek	Grazing, cleared	Low-undulating, hills in headwaters	Flat U-Shape	Severe aggradation	Sediment-laden runoff	Extreme	28 m
WTF_CON_01	Condamine River	Unnamed tributary of	Cleared grazing,	Very low slope	Two stage; Mulit-stage	Moderate aggradation	Sediment-laden runoff, Stock	Hight to Very High	60-66 m

Site	Catchment	Waterway	Primary Land Use	Topography	Channel Shape(1)	Bed Stability Rating(1)	Dominant Disturbance(1)	Overall Disturbance Rating(1)	Average Width(1)
		Condamine River	irrigated pasture, cropping						
WTF_GIL_01 or 01a	Border Rivers	Unnamed tributary of Weir River	Cleared grazing, forestry	Low undulating (no contours)	Two stage	Stable to moderate aggradation	Clearing vegetation	Very High	29-71 m

Source: Aquatic Ecology, Water Quality and Geomorphology Impact Assessment – Gas Fields (Hydrobiology 2009) (Volume 5, Attachment 20)



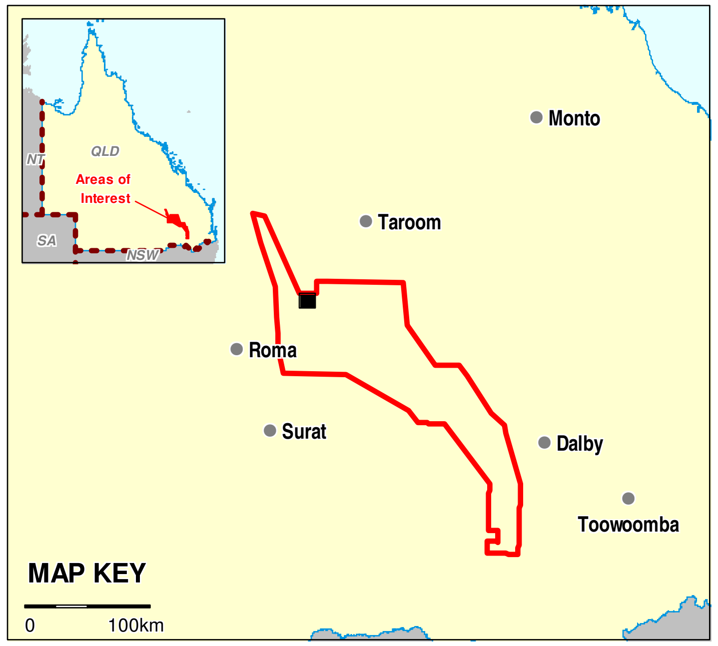


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Figure 4-3: Infrastructure Location - WTF_MEL01 and GPF_PHS_05a

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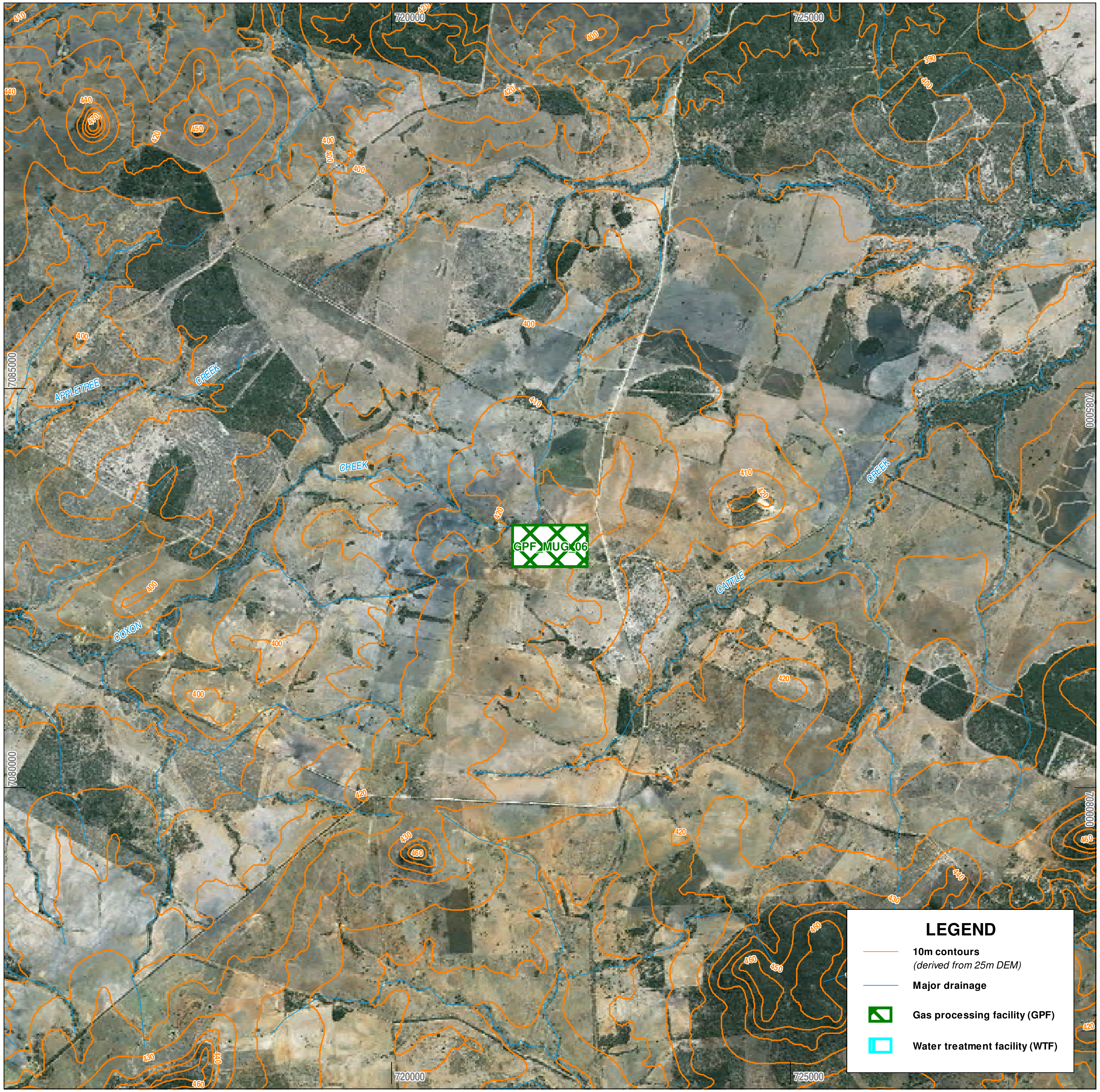

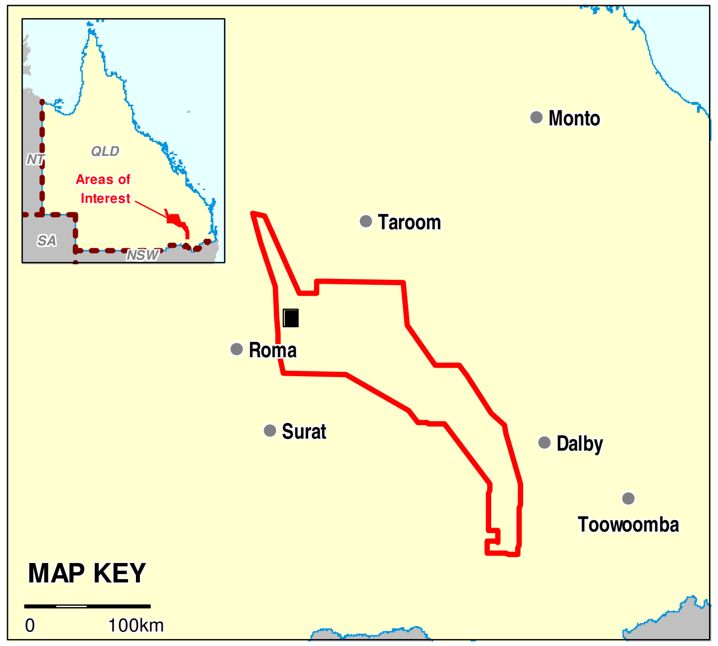

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Figure 4-4: Infrastructure Location - GPF_MUG_06

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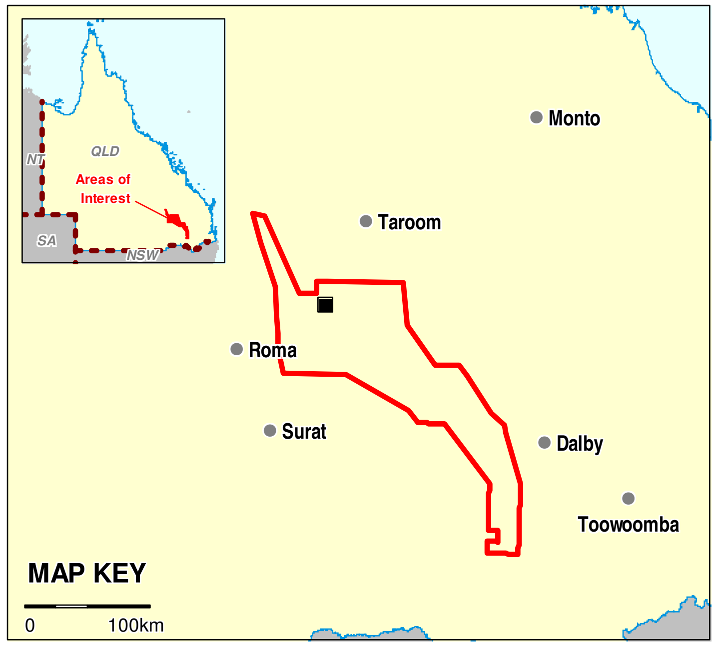

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Figure 4-5: Infrastructure Location - GPF_COM_03a

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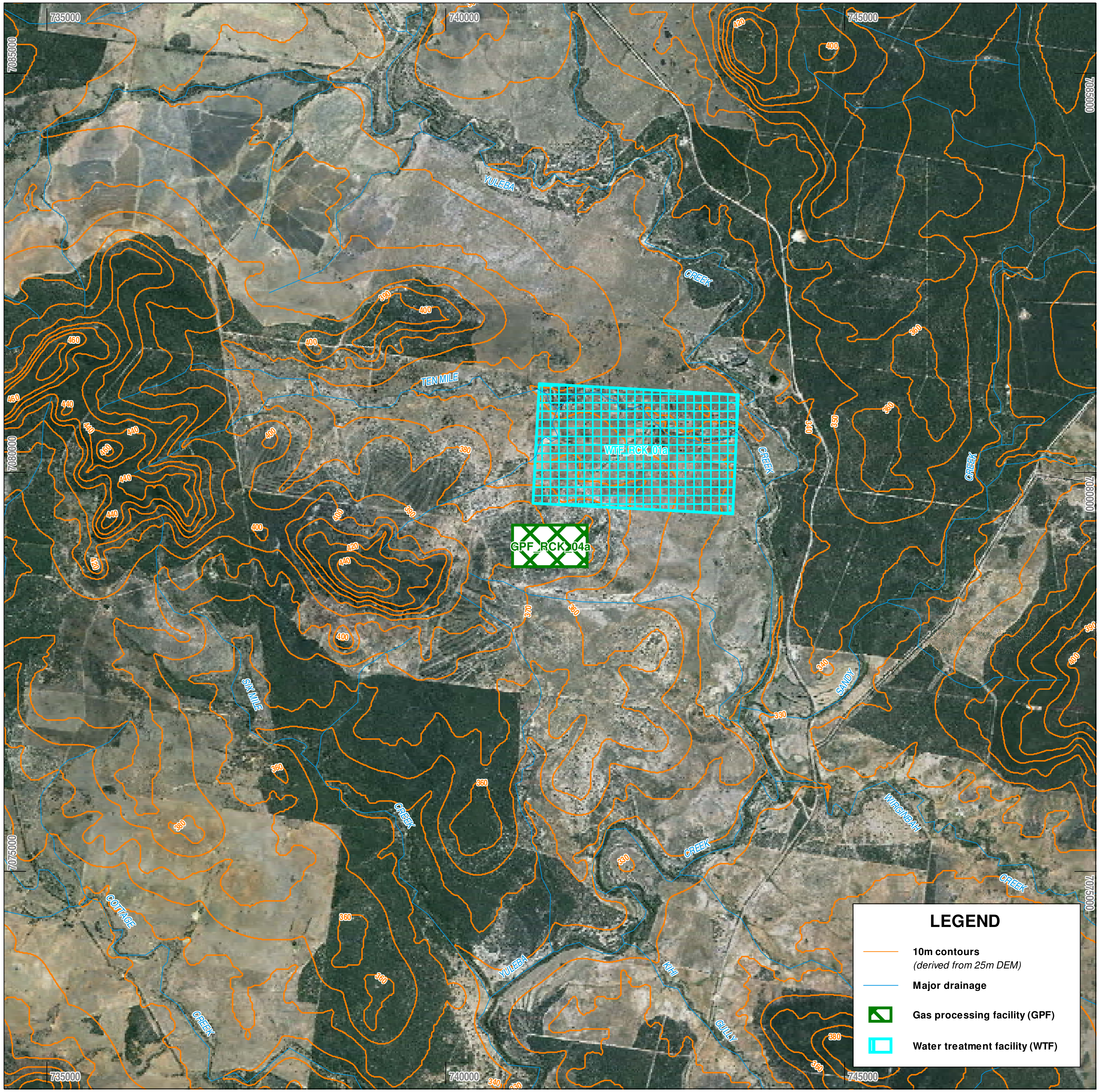

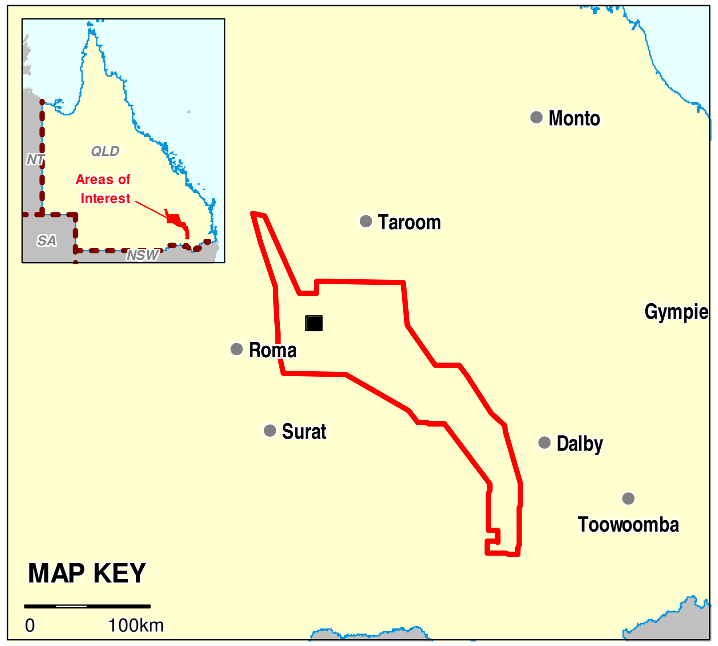

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Figure 4-6: Infrastructure Location - WTF_RCK_01a and GPF_RCK_04a

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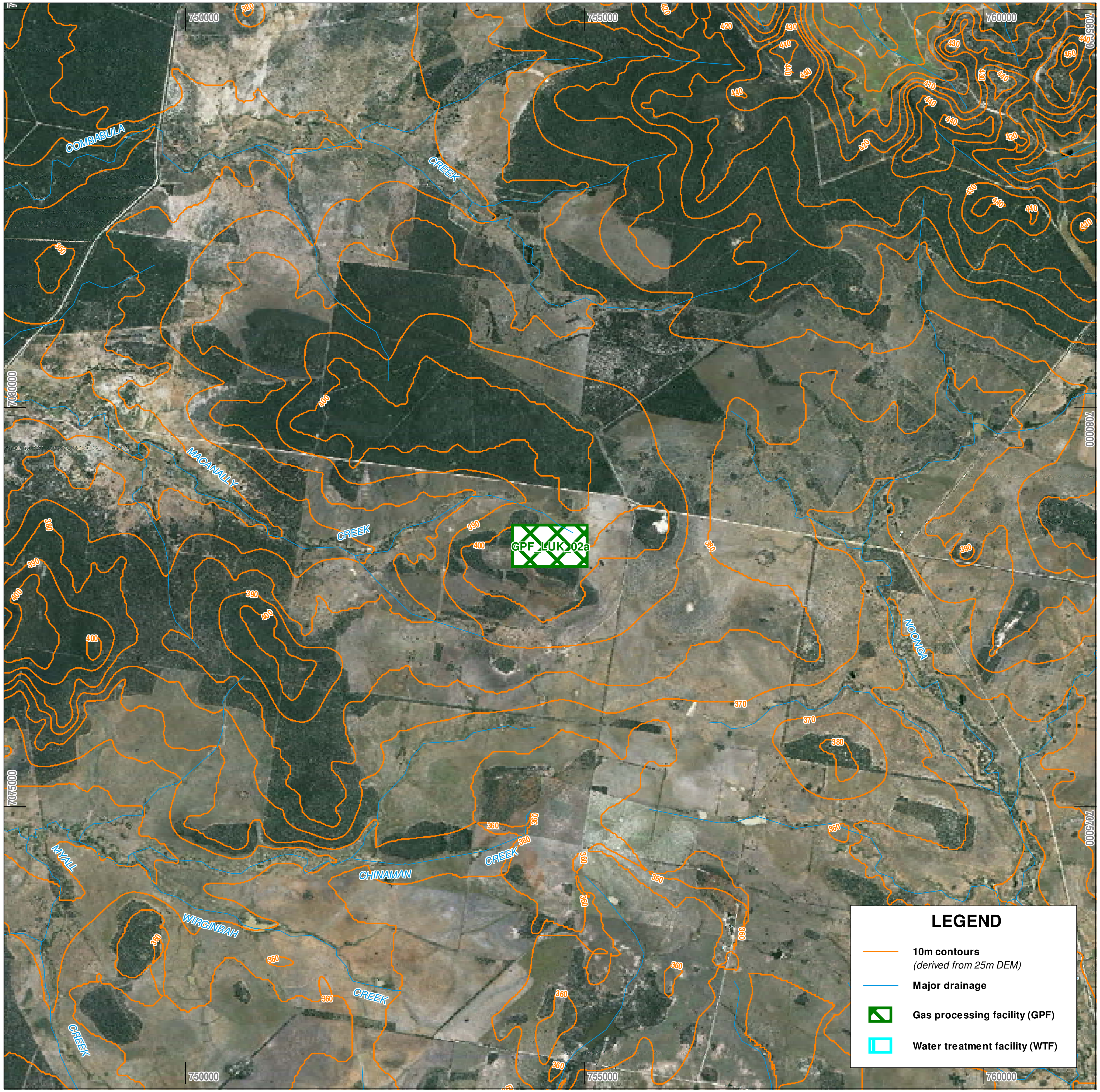

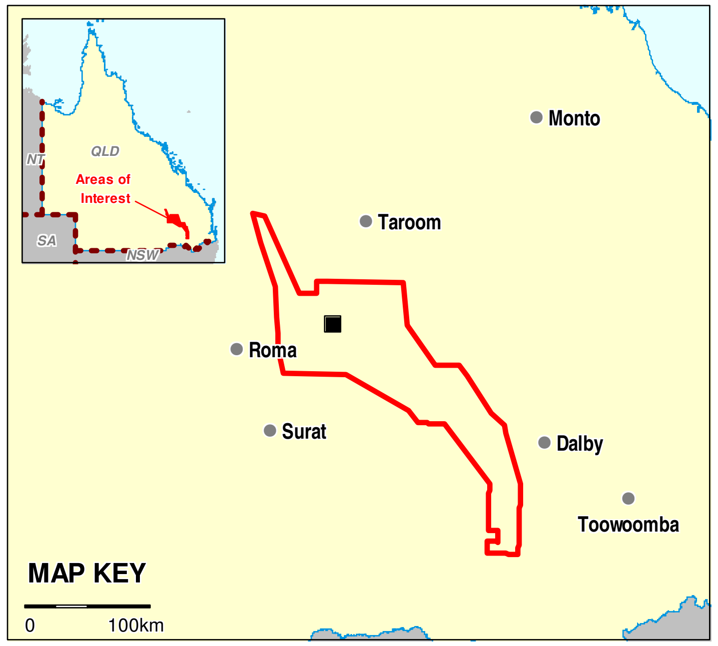

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Figure 4-7: Infrastructure Location - GPF_LUK_02a

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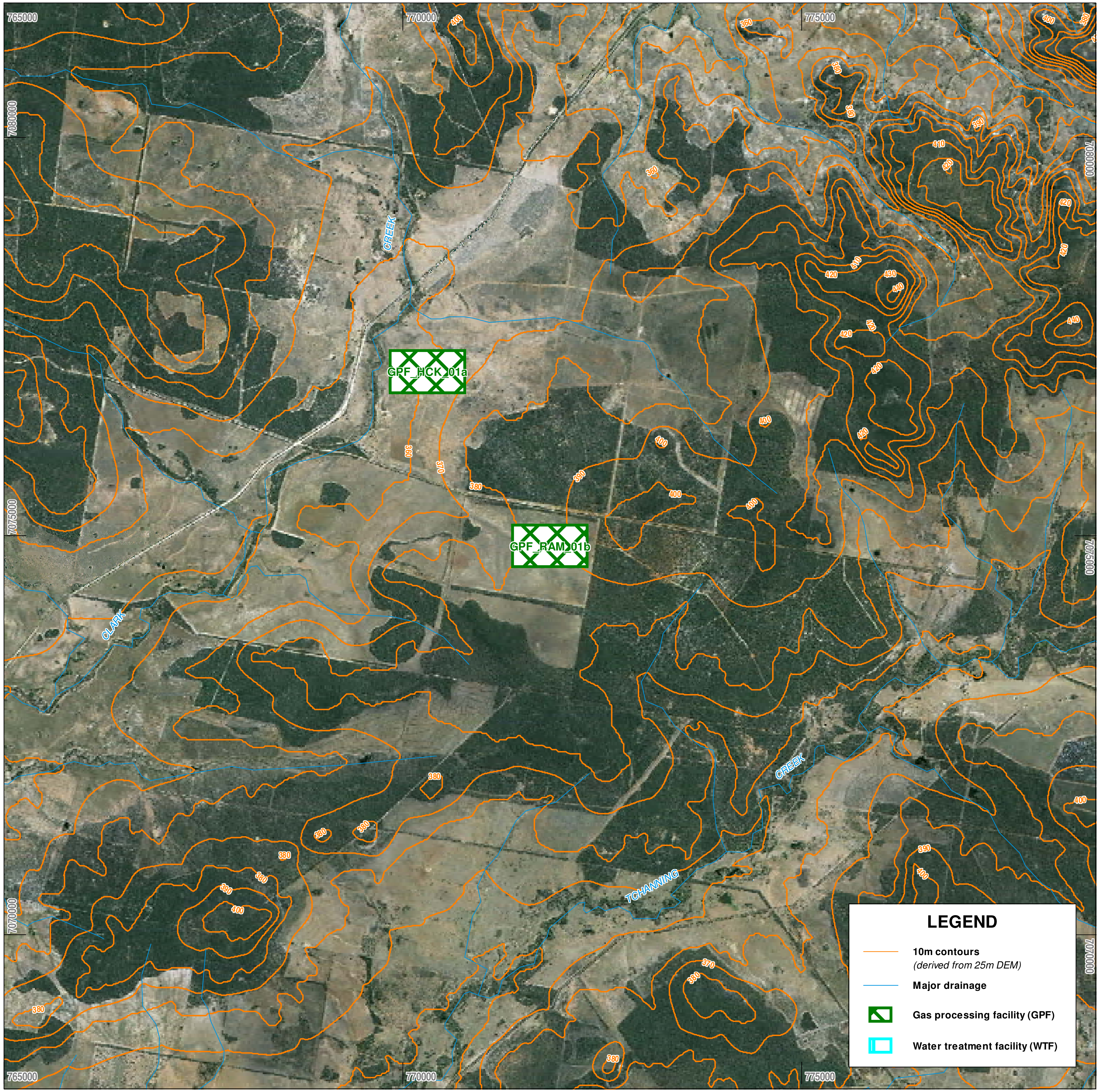

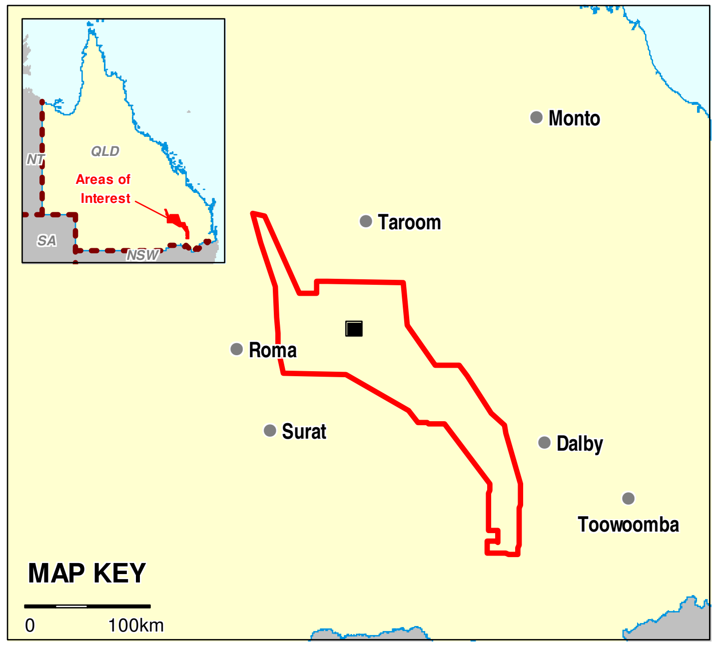

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Figure 4-8: Infrastructure Location - GPF_RAM_01b and GPF_HCK_01a

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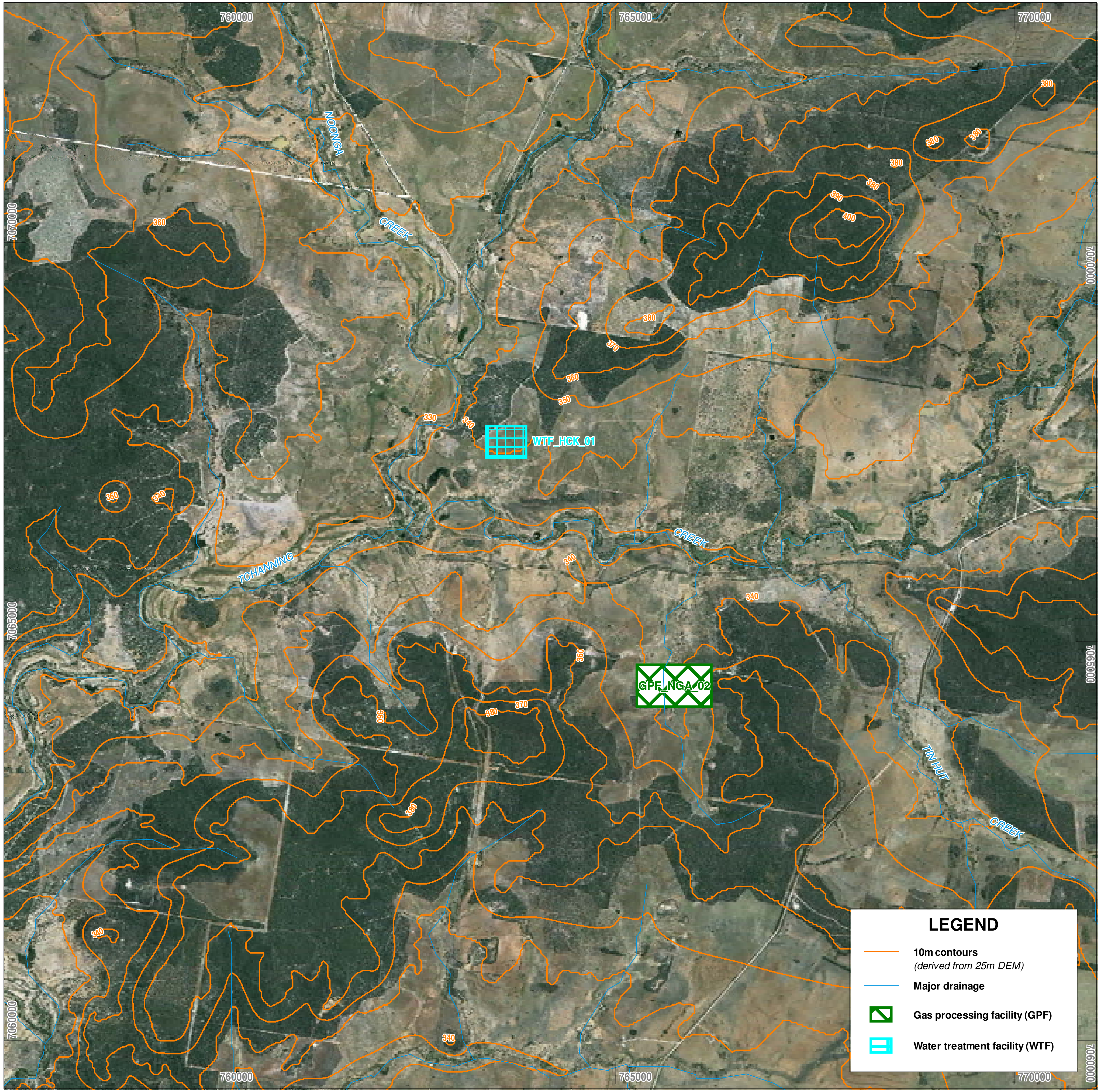

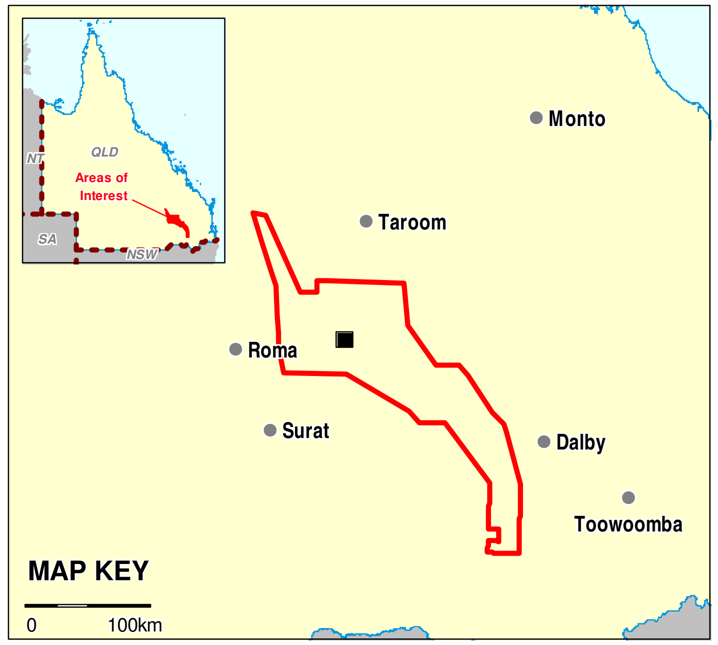

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Figure 4-9: Infrastructure Location - WTF_HCK_01 and GPF_NGA_02

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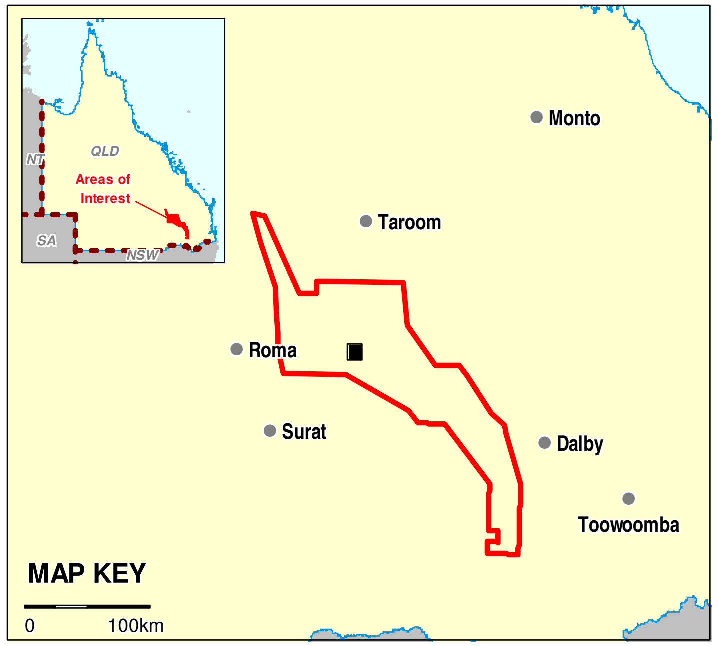

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Figure 4-10: Infrastructure Location - GPF_NGA_04

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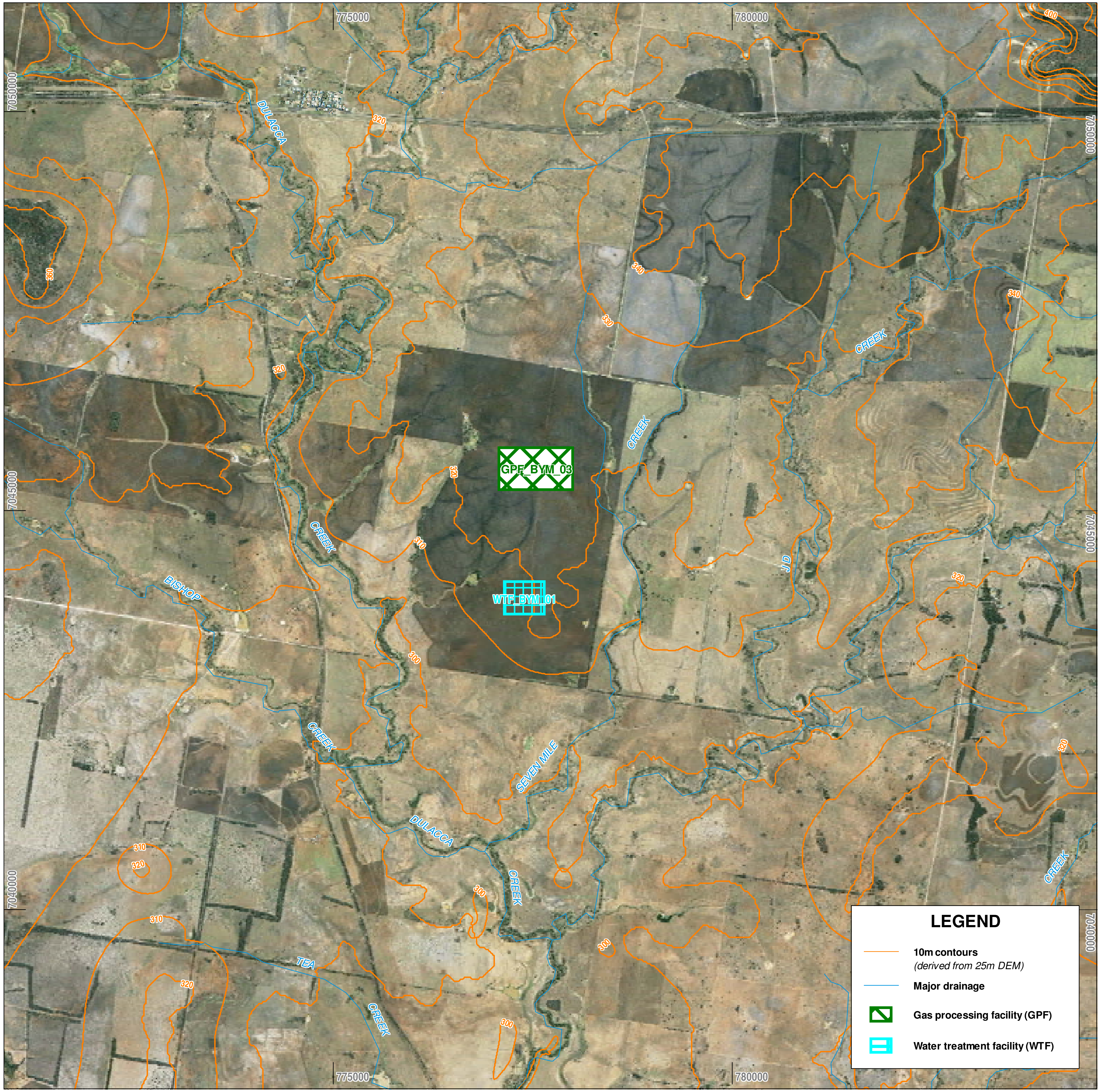

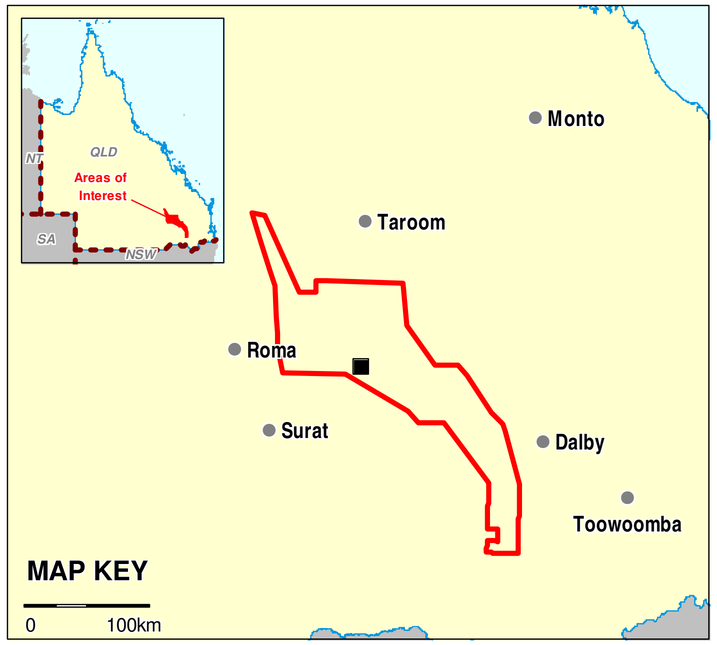

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Figure 4-11: Infrastructure Location - WTF_BYM_01 and GPF_BYM_03

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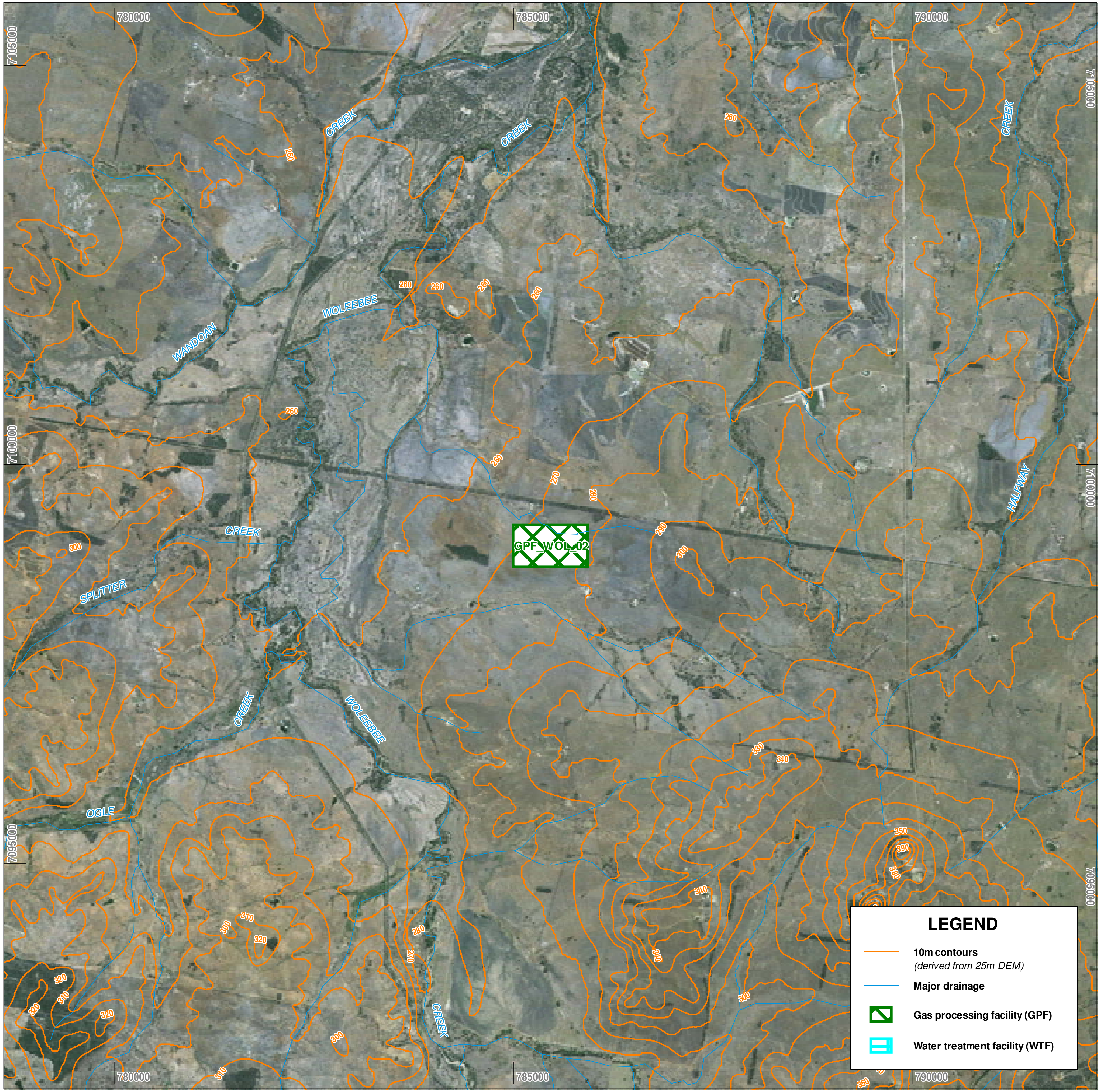

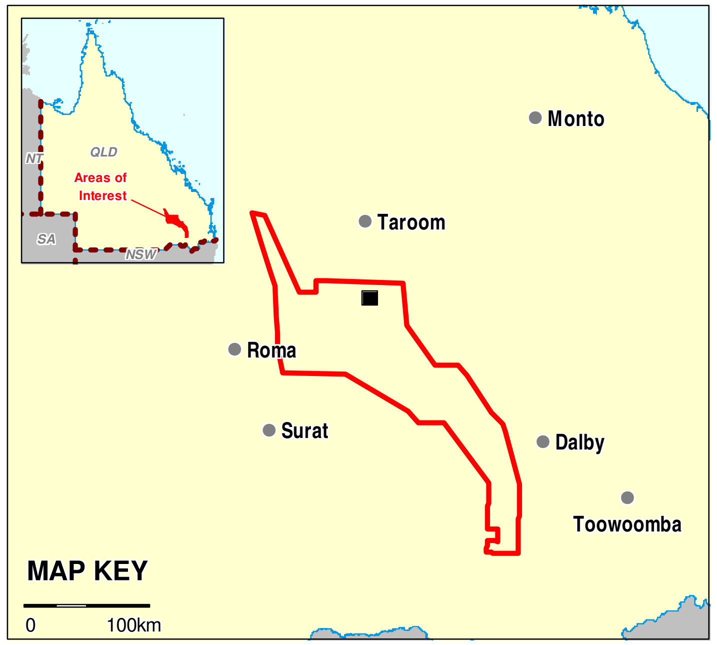

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Figure 4-12: Infrastructure Location - GPF_WOL_02

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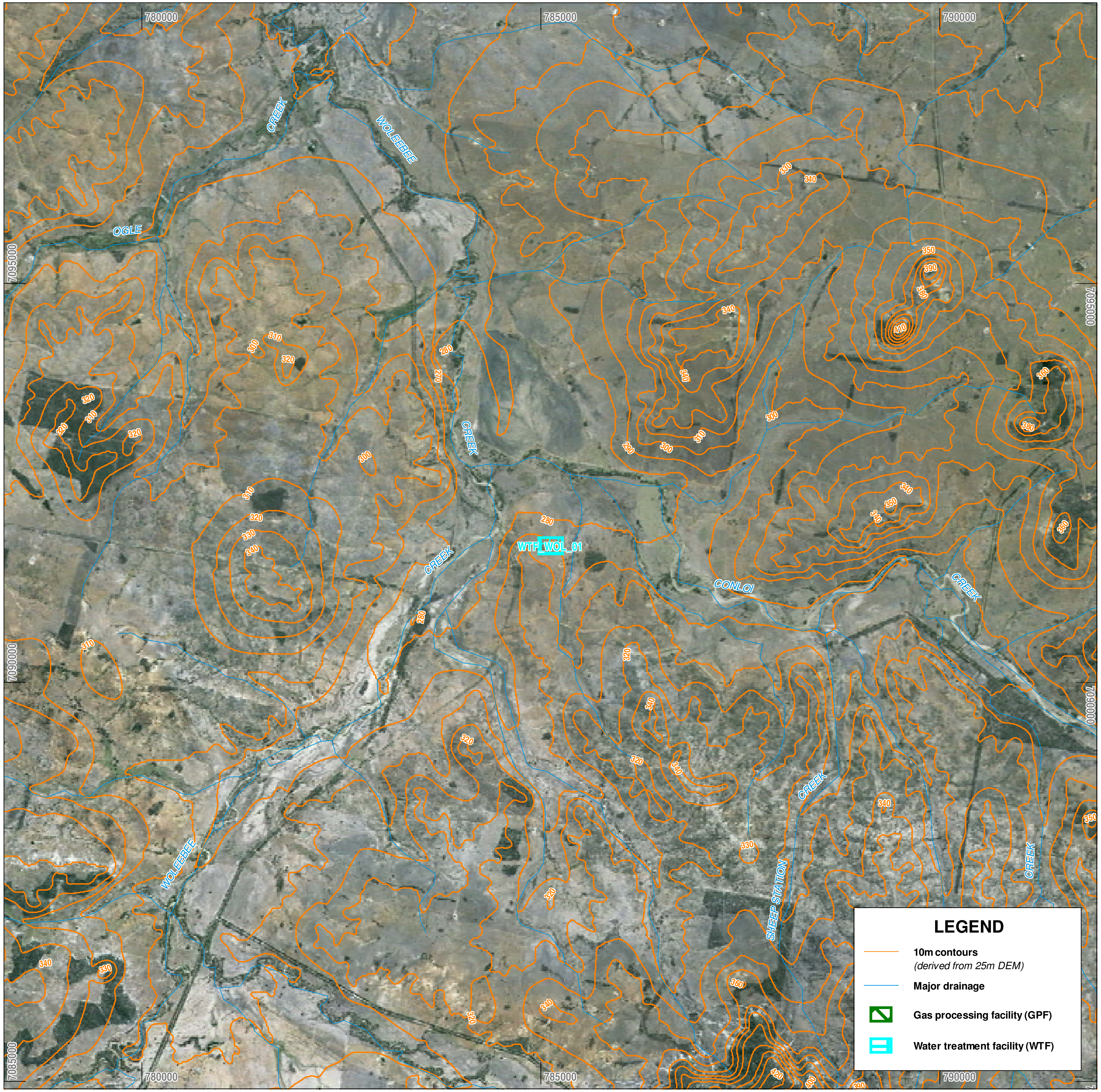

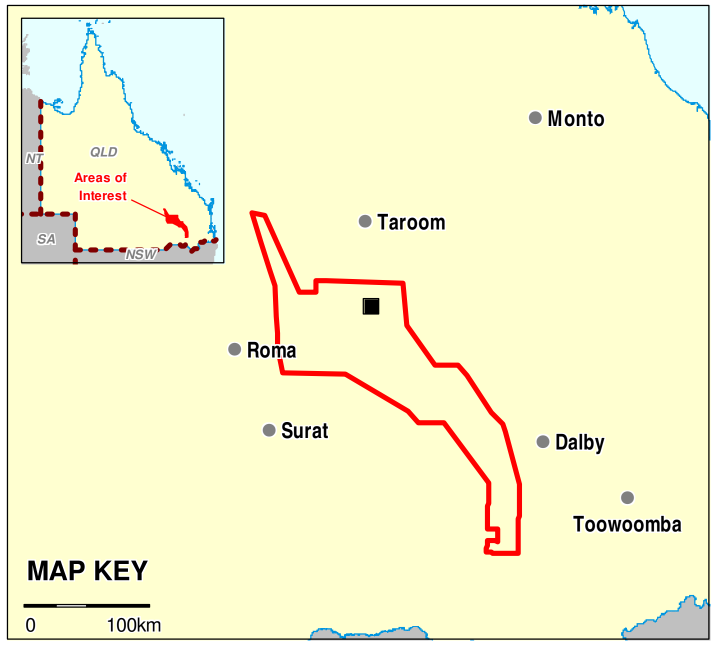

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Geocentric Datum of Australia 1994

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Figure 4-13: Infrastructure Location - WTF_WOL_01

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-0235 | Rev: 0

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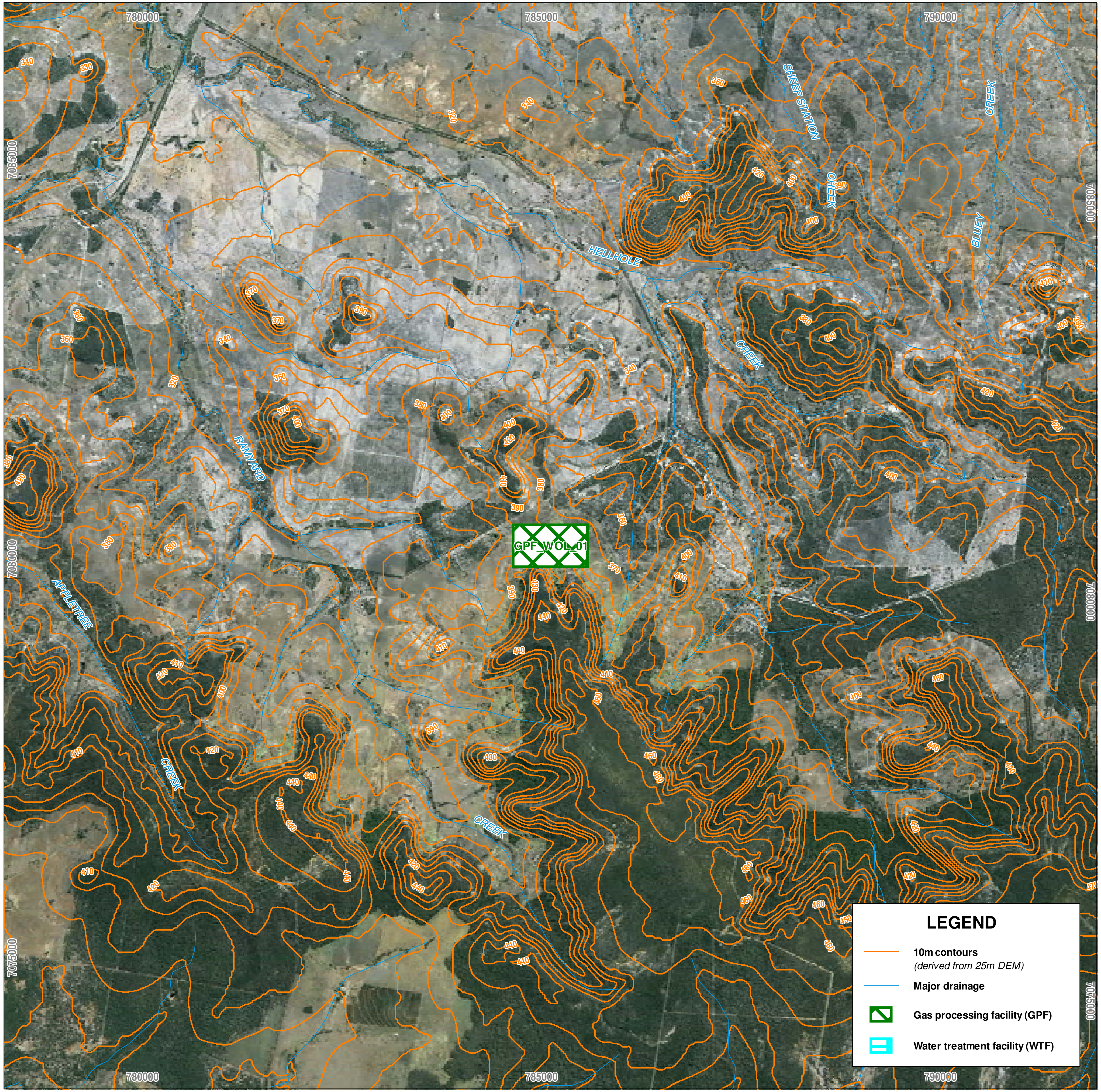

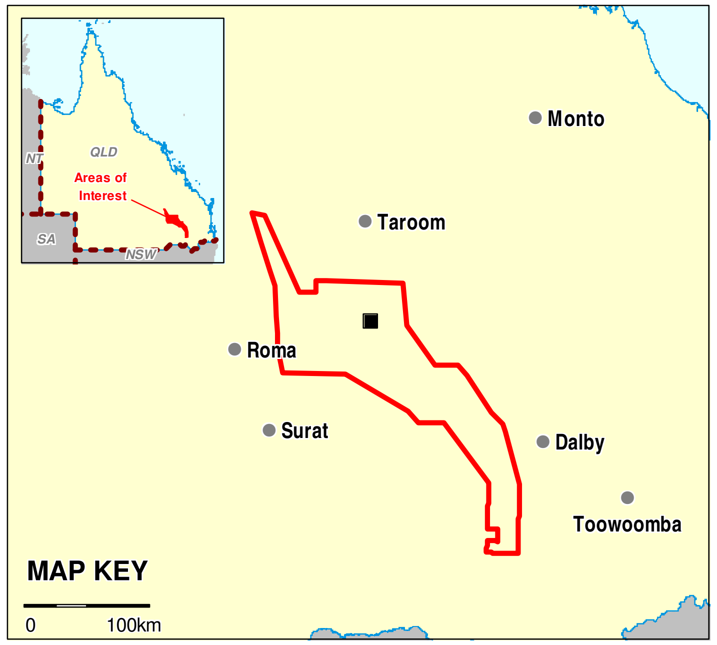

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Figure 4-14: Infrastructure Location - GPF_WOL_01

Project No: 301001-00448	Figure: 00448-00-EN-DAL-0236	Rev: 0

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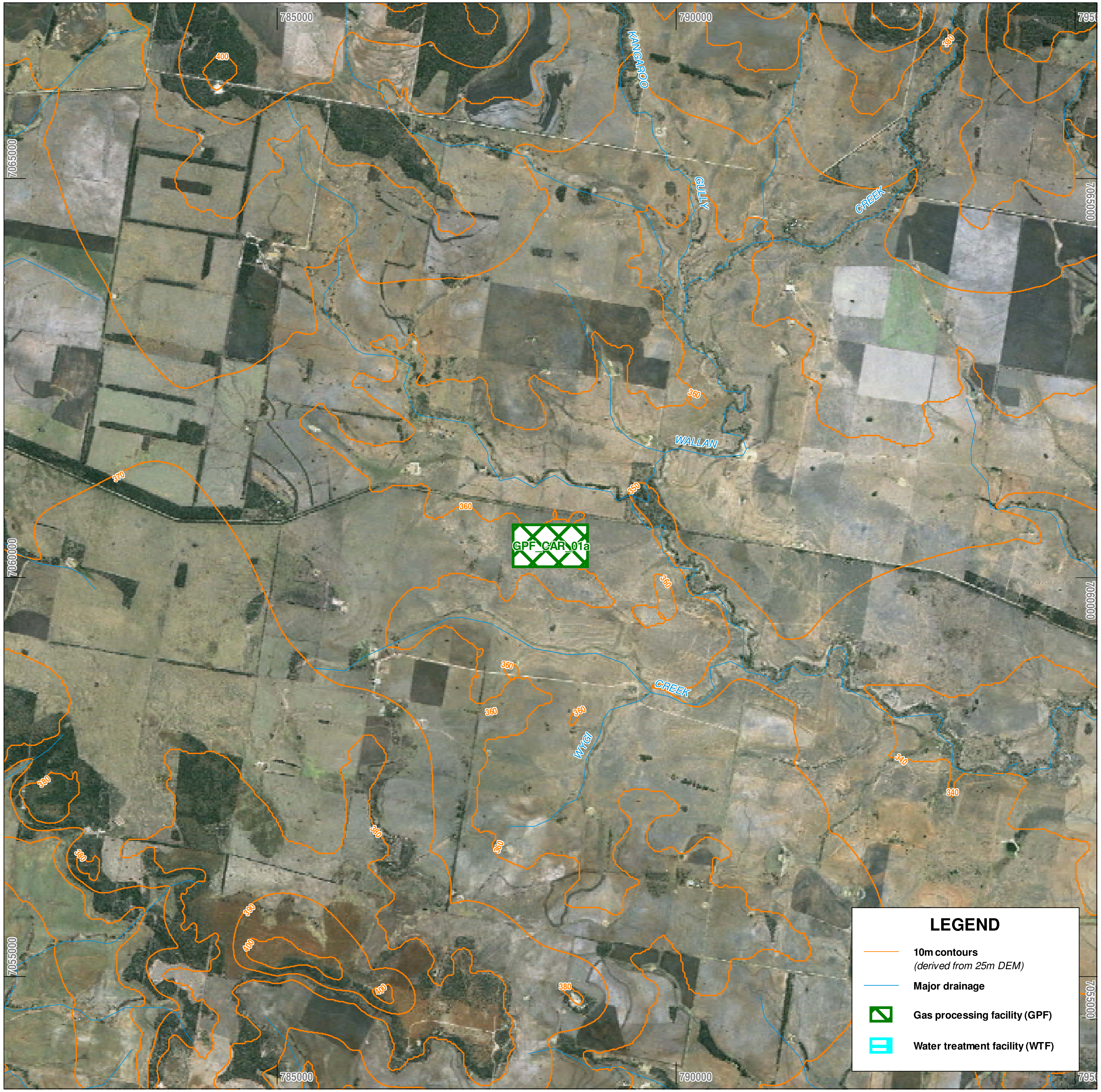

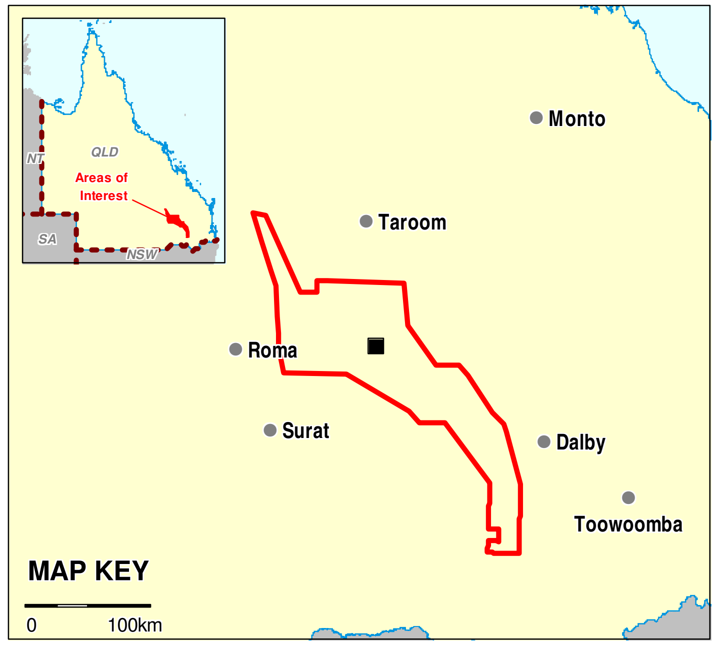

ALOS Imagery supplied by client 09/01/2009

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Figure 4-15: Infrastructure Location - GPF_CAR_01a

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-0237 | Rev: 0

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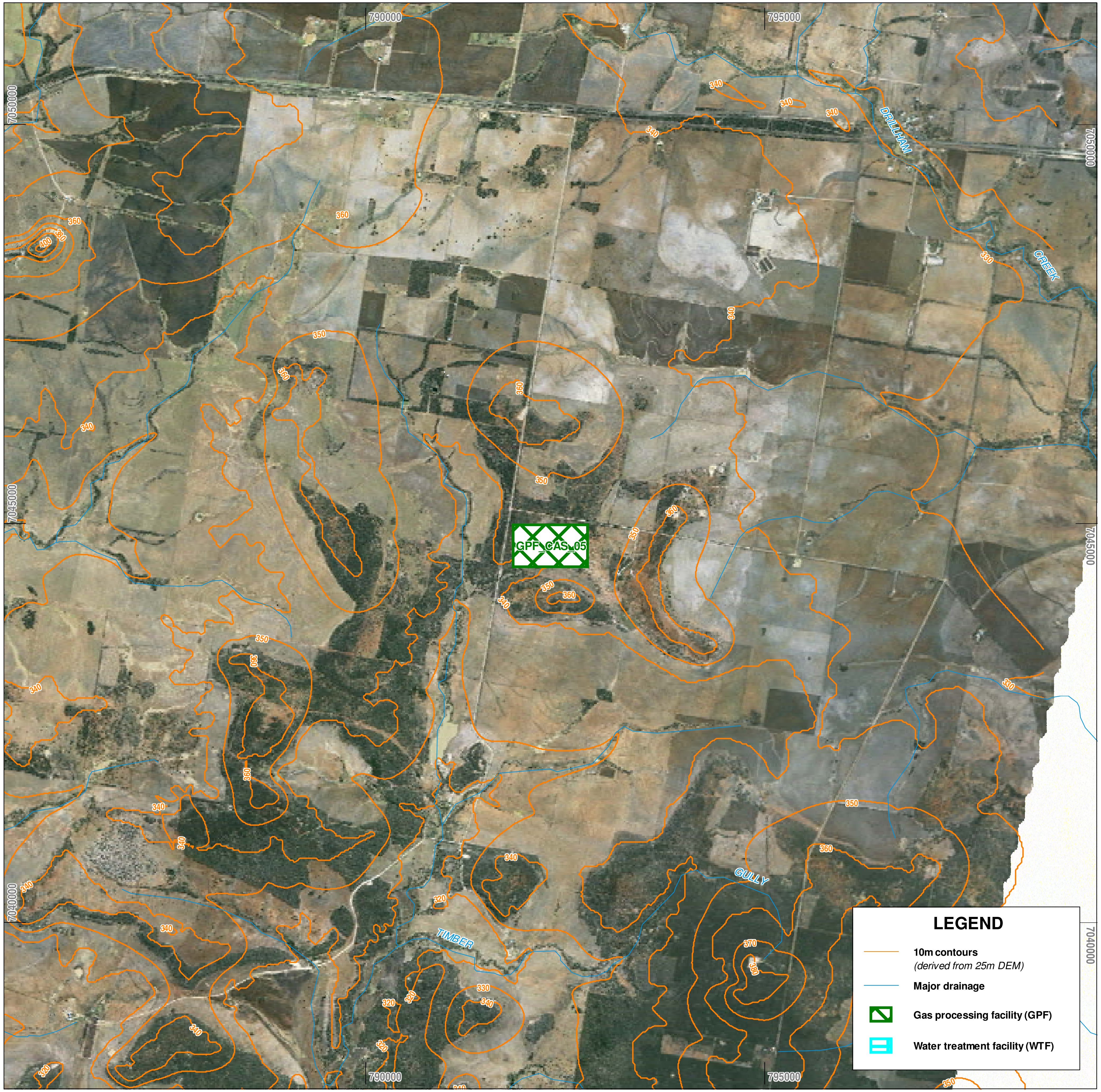

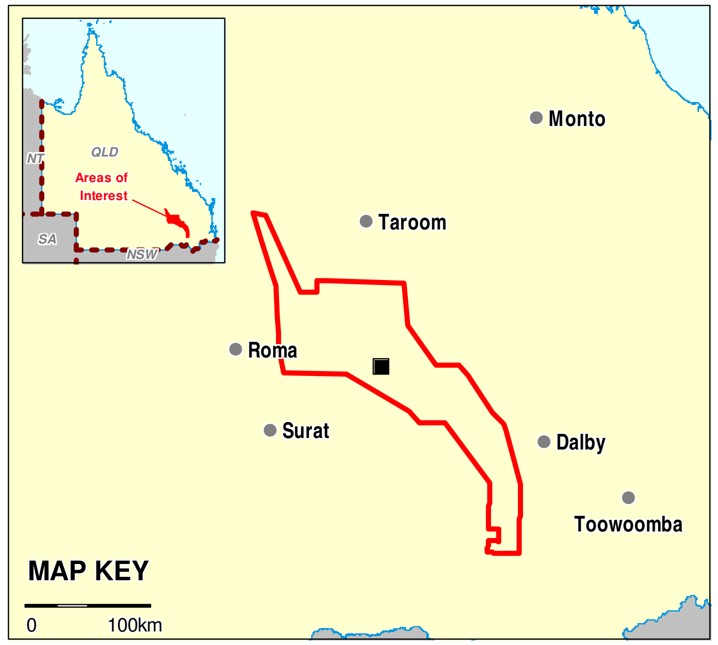

ALOS Imagery supplied by client 09/01/2009

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Map Grid of Australia Zone 55
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Figure 4-16: Infrastructure Location - GPF_CAS_05

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-0238 | Rev: 0

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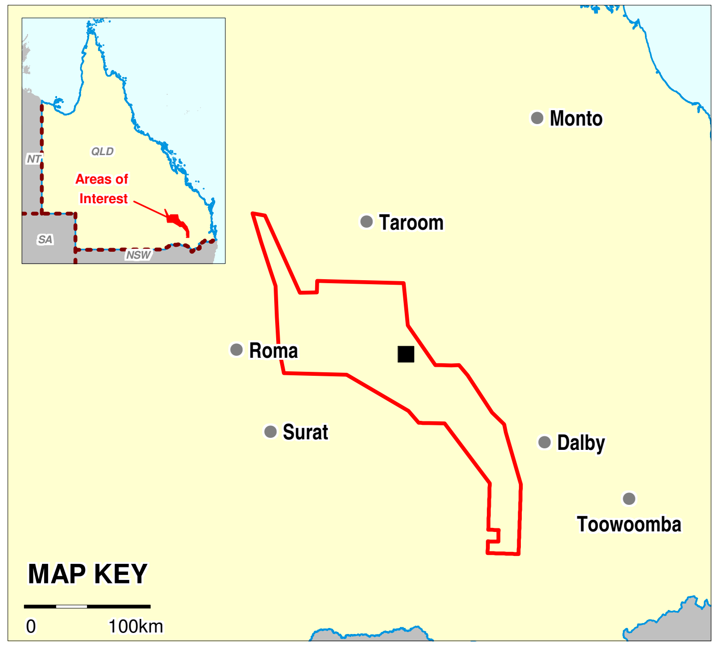

Aerial imagery captured June 2009 and suuplied by client.

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SCALE - 1:50,000 (at A3)

Map Grid of Australia Zone 55
Geocentric Datum of Australia 1994

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Figure 4-17: Infrastructure Location - GPF_DAL_01b

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-0239 | Rev: 1

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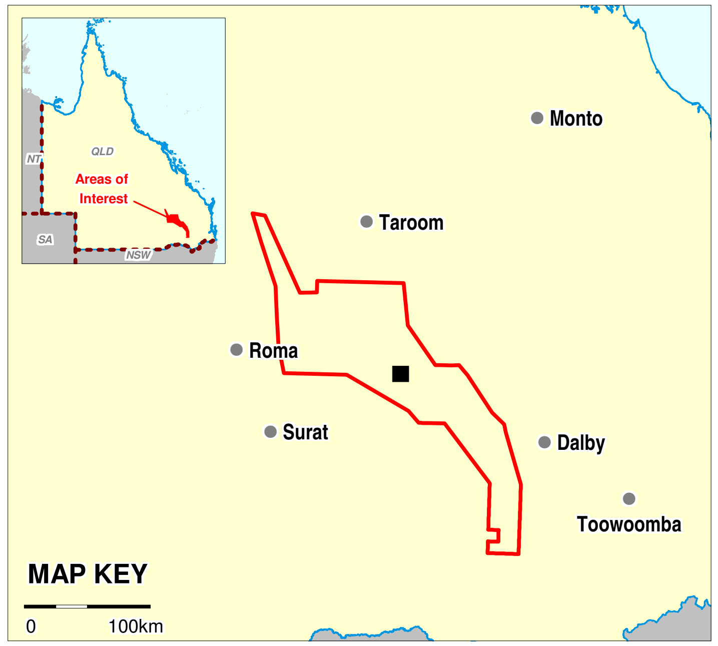

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Figure 4-18: Infrastructure Location - GPF_DAL_02

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-0240 | Rev: 1

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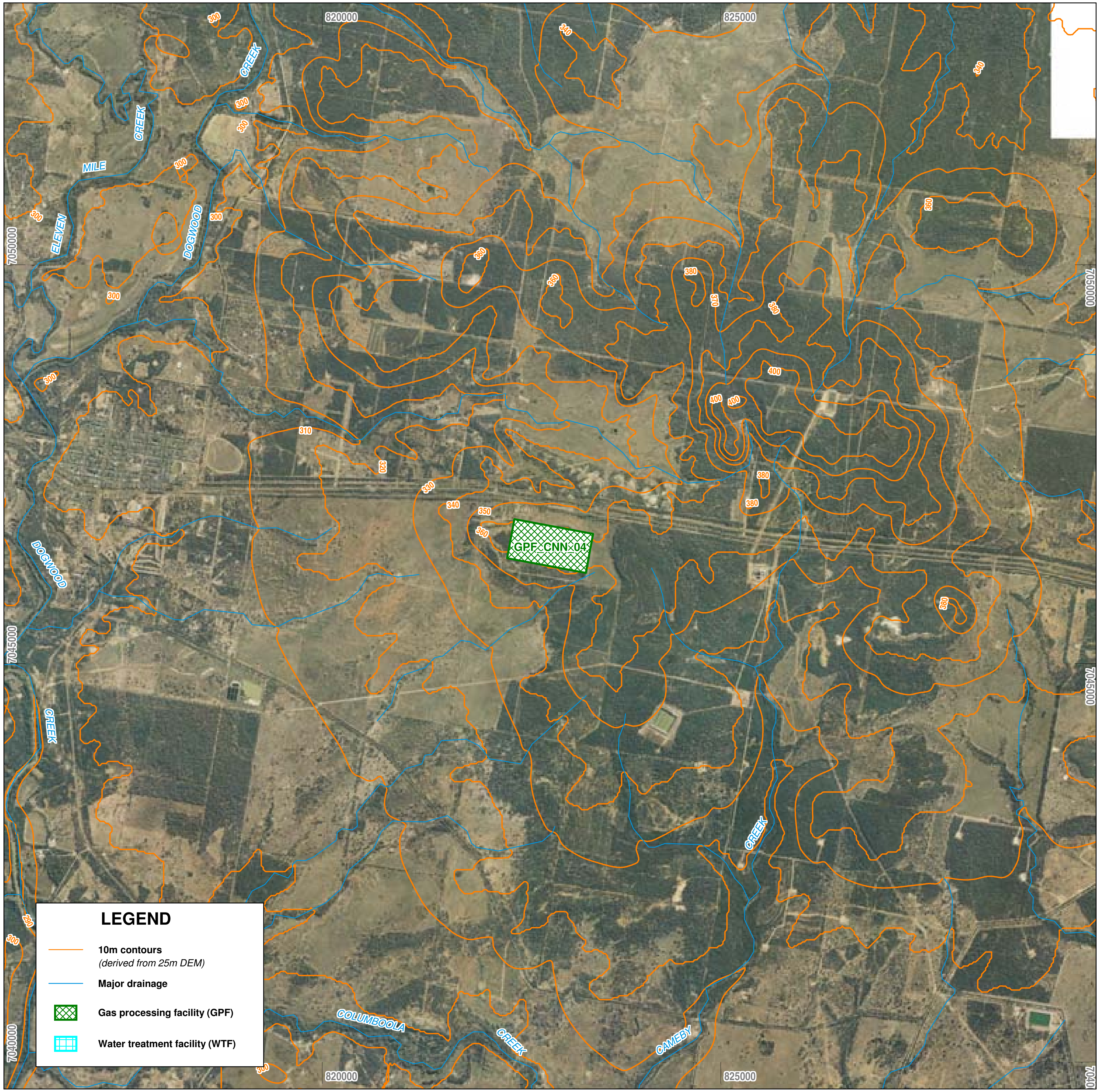

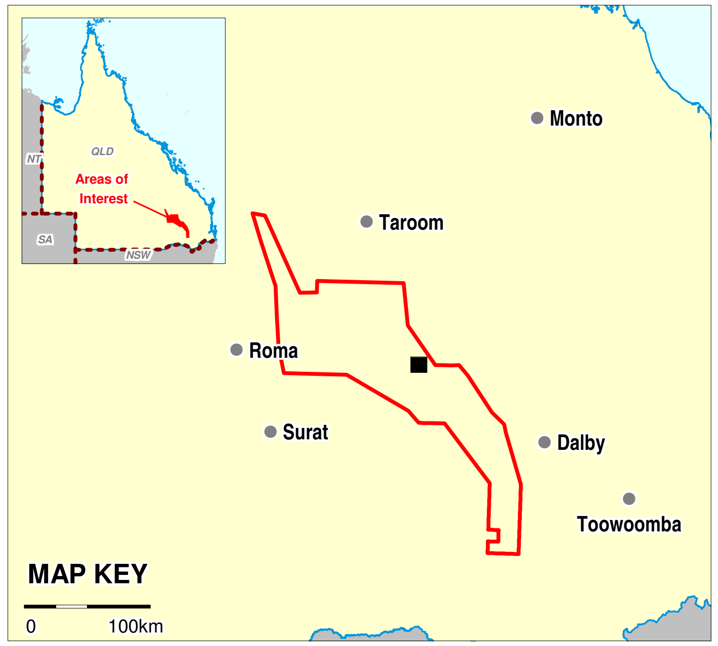

Aerial imagery captured June 2009 and suuplied by client.

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Map Grid of Australia Zone 55
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Figure 4-19: Infrastructure Location - GPF_CNN_04

Project No: 301001-00448	Figure: 00448-00-EN-DAL-0241	Rev: 1

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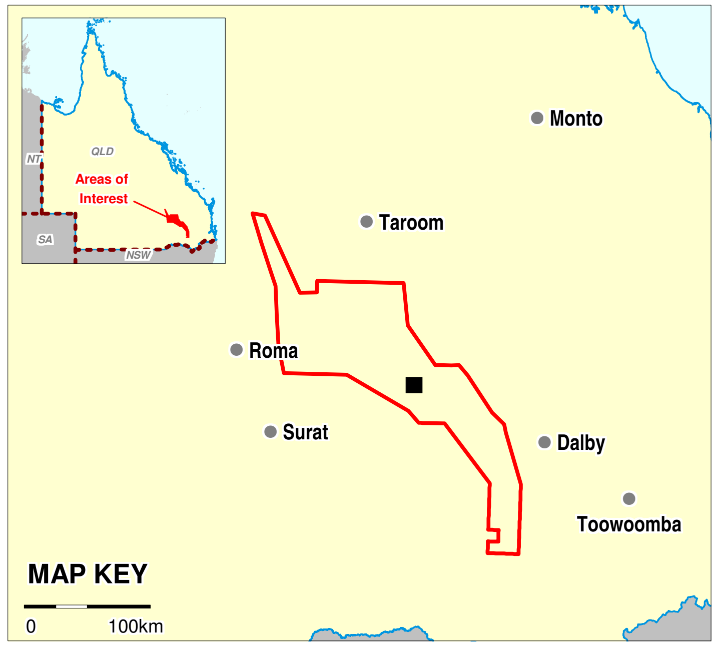

Aerial imagery captured June 2009 and suuplied by client.

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SCALE - 1:50,000 (at A3)

Map Grid of Australia Zone 55
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Figure 4.20: Infrastructure Location - WTF_CON_01 and GPF_CON_02b

Project No: 301001-00448	Figure: 00448-00-EN-DAL-0242	Rev: 1

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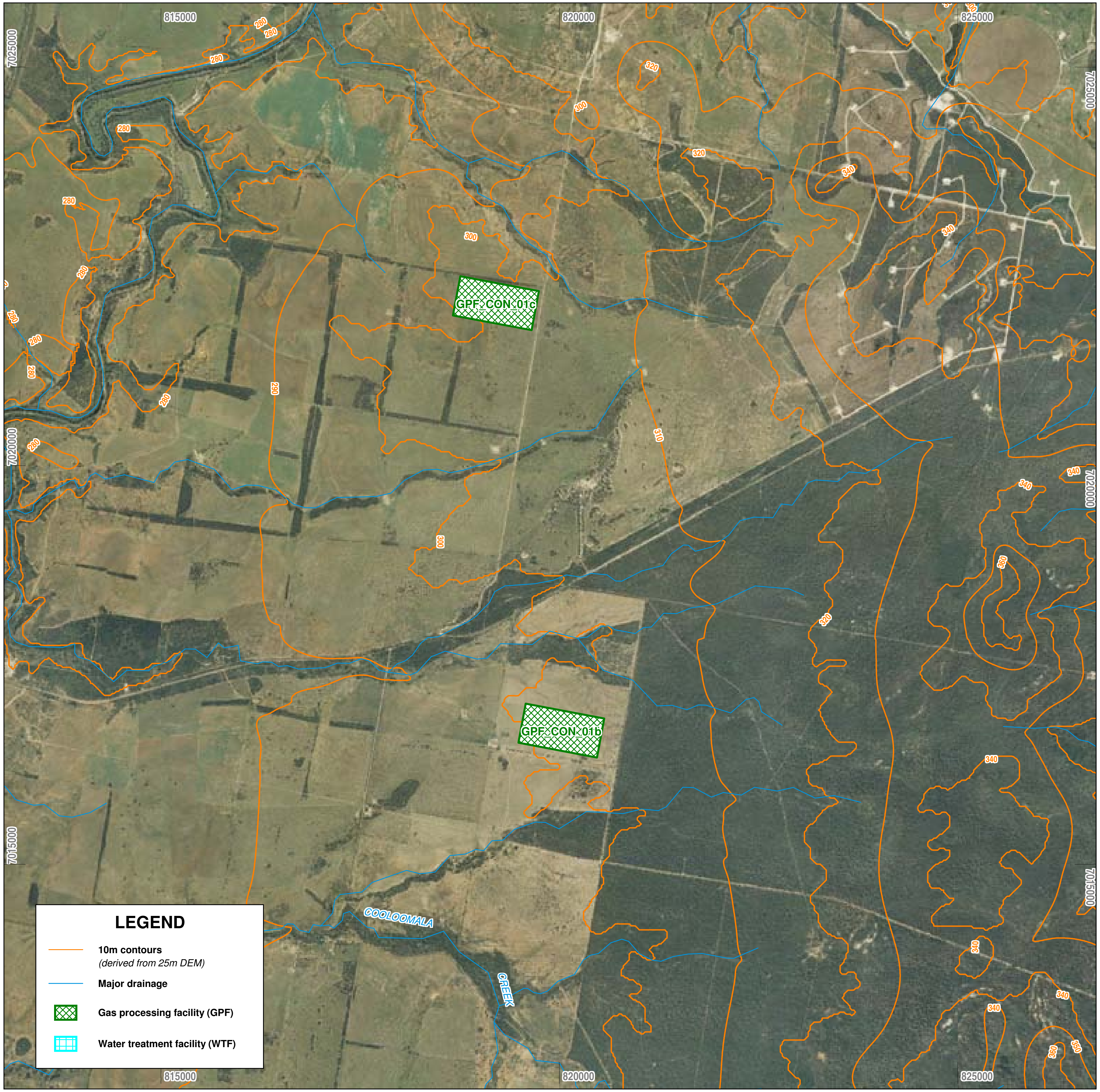

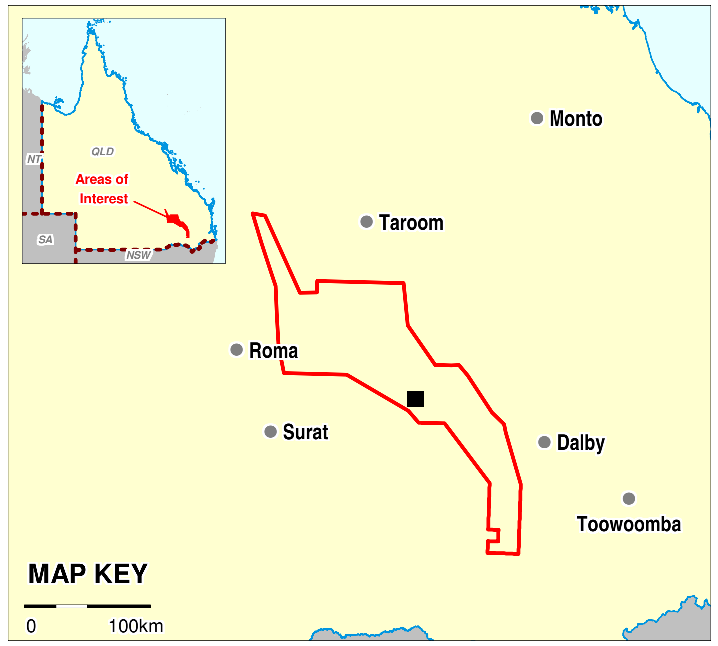

Aerial imagery captured June 2009 and suuplied by client.

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Map Grid of Australia Zone 55
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Figure 4-21: Infrastructure Location - GPF_CON_01b and GPF_CON_01c

Project No: 301001-00448	Figure: 00448-00-EN-DAL-0243	Rev: 1

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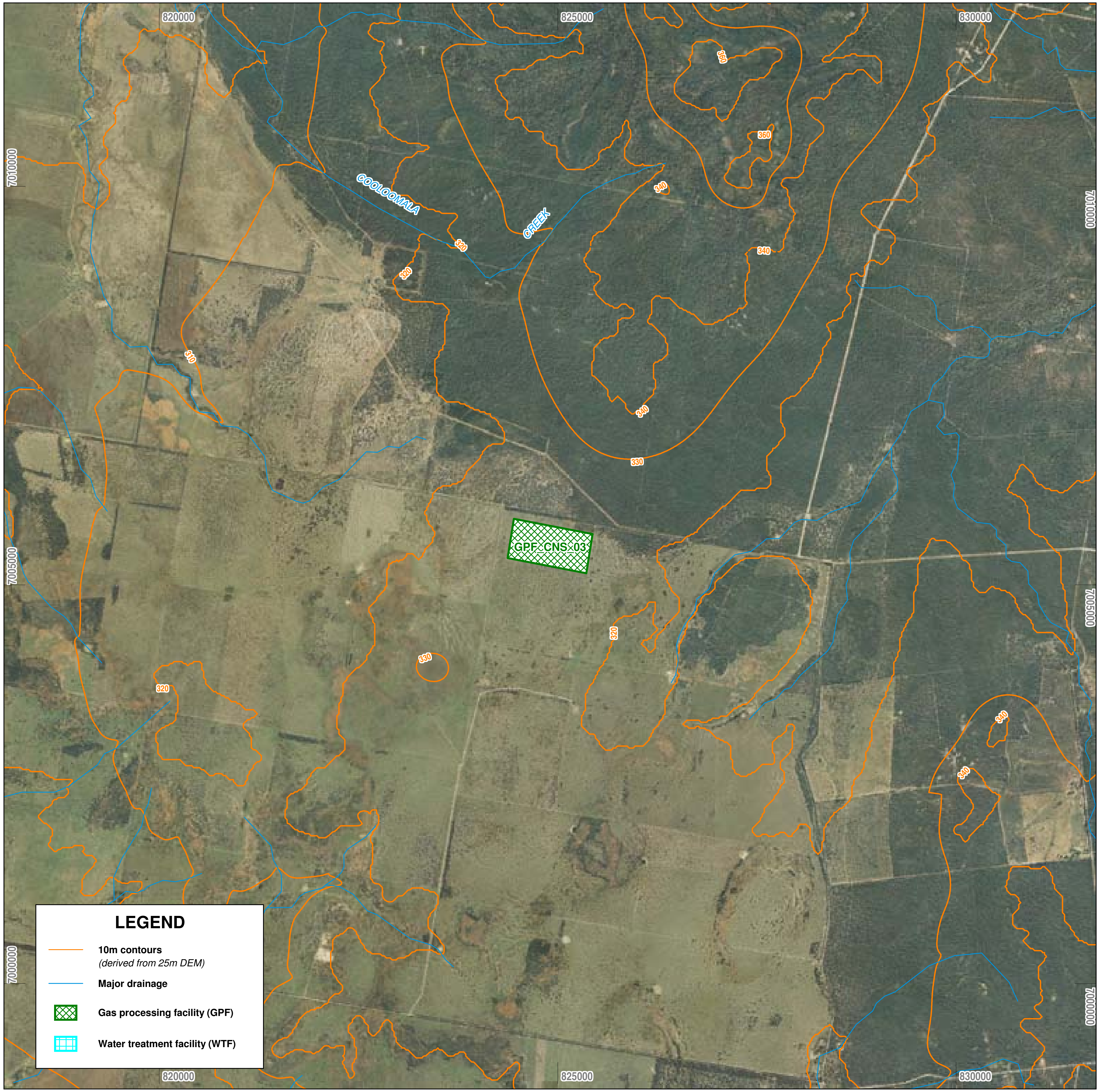

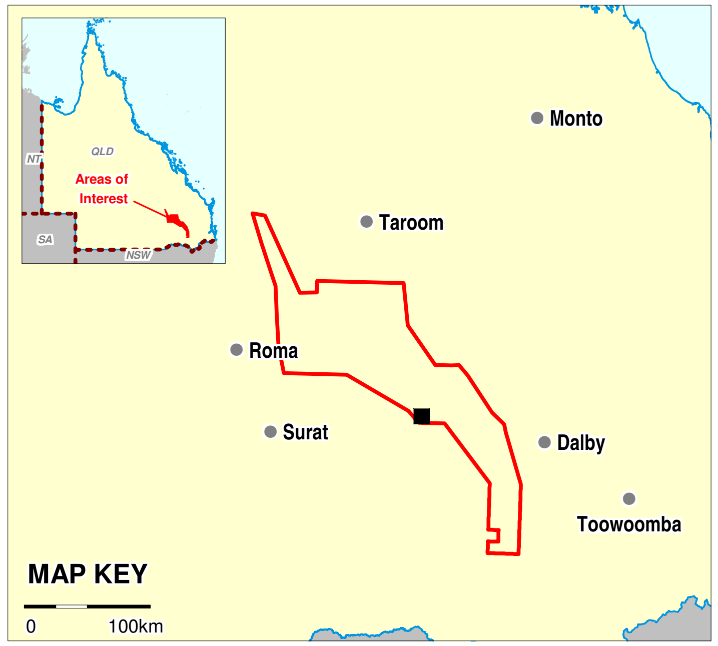

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Figure 4-22: Infrastructure Location - GPF_CNS_03

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-0244 | Rev: 1

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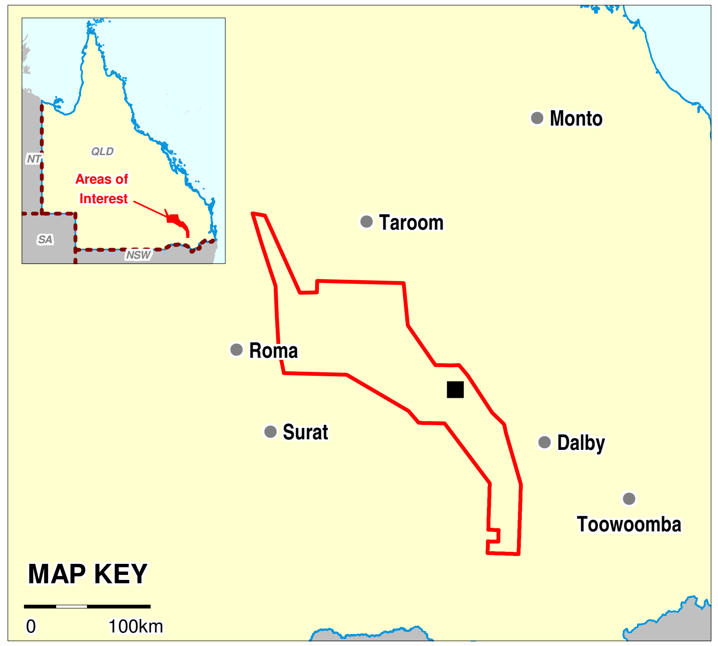

Aerial imagery captured June 2009 and suuplied by client.

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Map Grid of Australia Zone 55
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Figure 4-23: Infrastructure Location - GPF_OAN_04

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-0245 | 1

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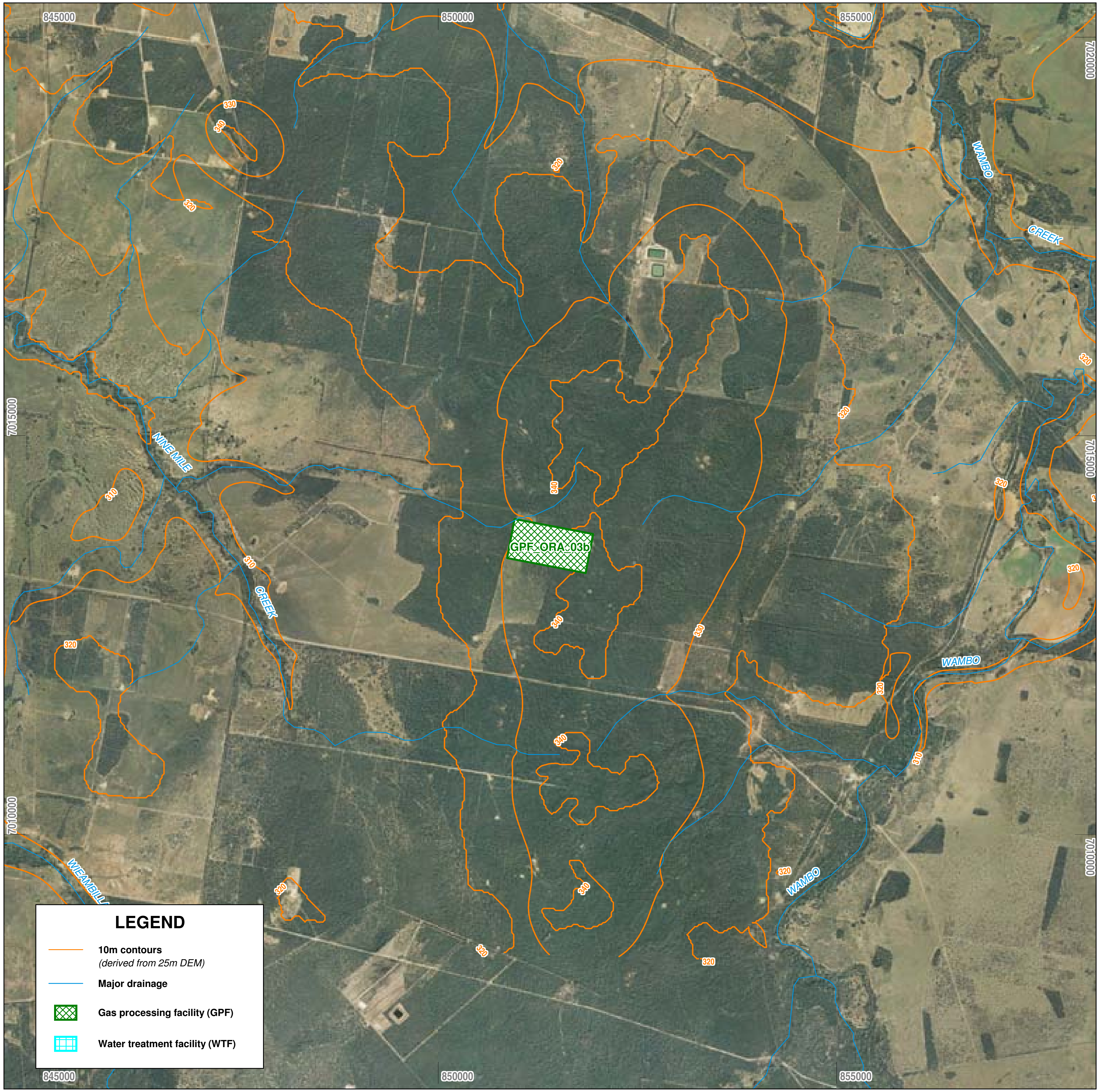

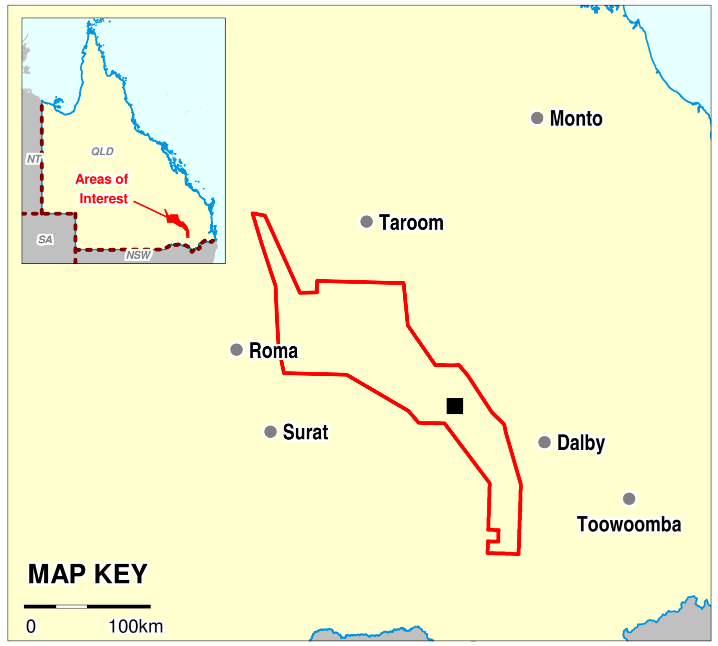

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Geocentric Datum of Australia 1994

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Figure 4-24: Infrastructure Location - GPF_ORA_03b

Project No: 301001-00448	Figure: 00448-00-EN-DAL-0246	Rev: 1

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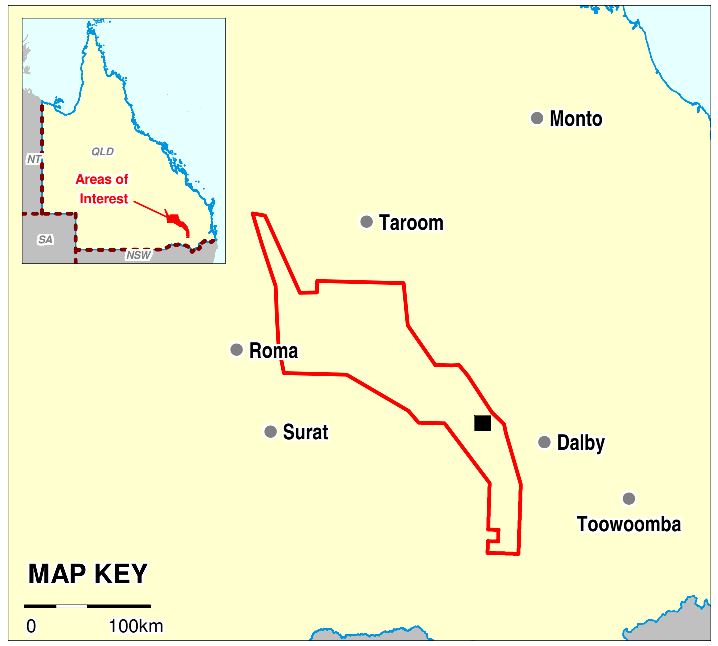

Aerial imagery captured June 2009 and suuplied by client.

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Map Grid of Australia Zone 55
Geocentric Datum of Australia 1994

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Figure 4-25: Infrastructure Location - GPF_KIA_01a

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-0247 | Rev: 1

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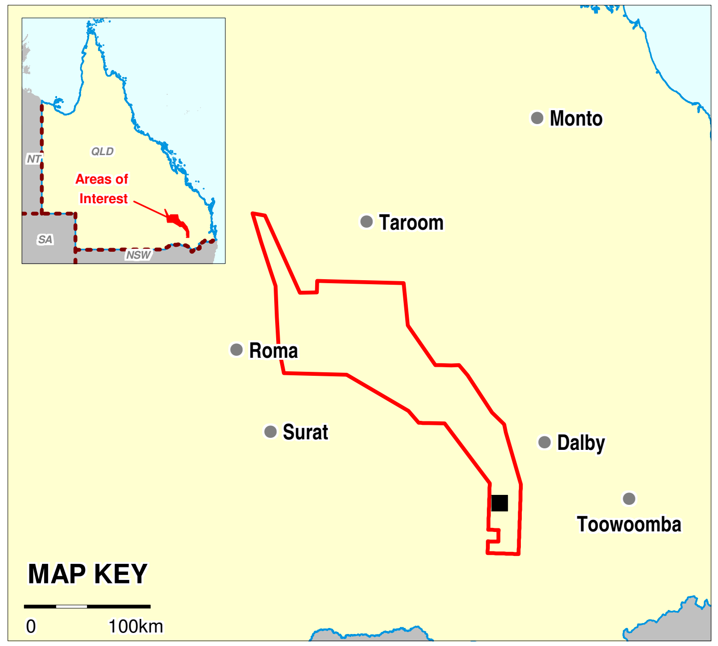

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Geocentric Datum of Australia 1994

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Figure 4-26: Infrastructure Location - WTF_GIL_01a and GPF_GIL_02

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-0248 | Rev: 1

Compiled by BRISBANE INFRASTRUCTURE GIS SECTION

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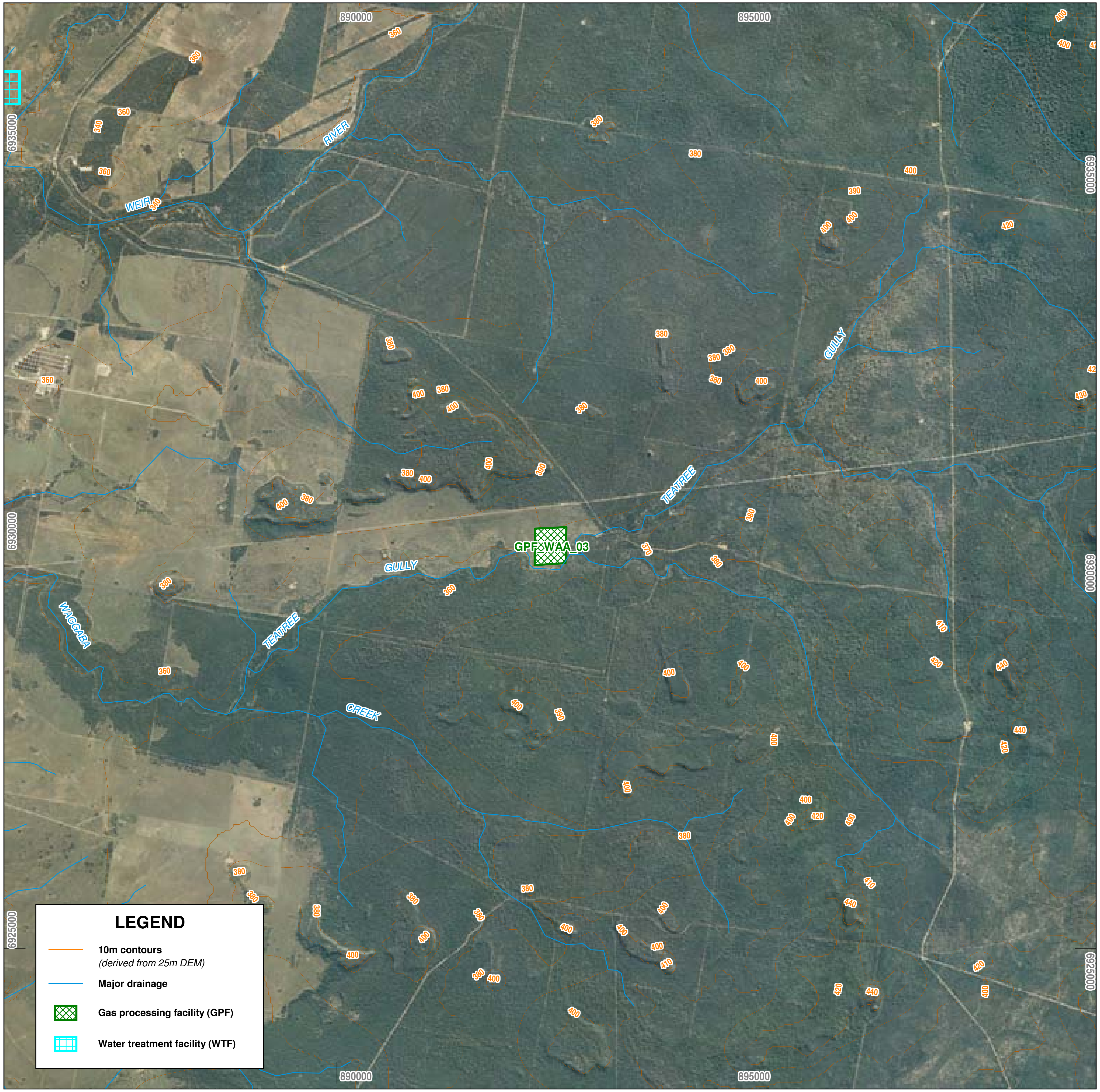
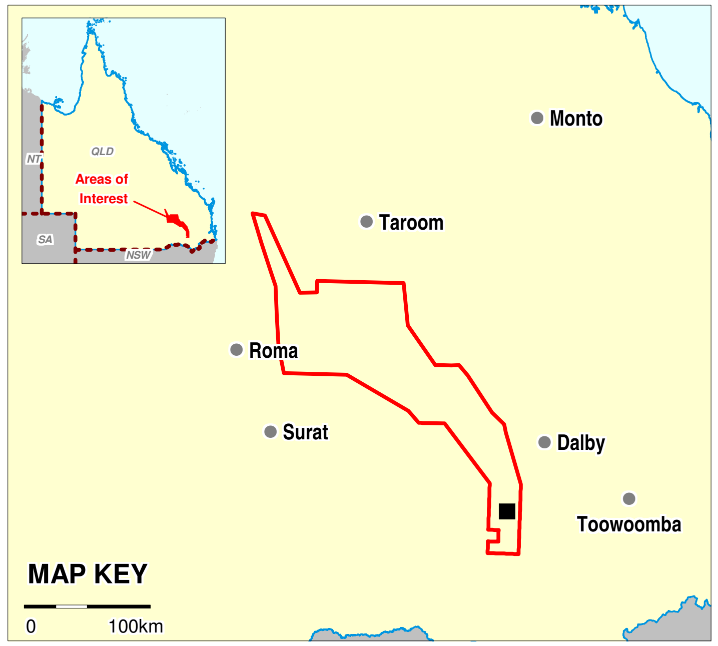


LEGEND

- 10m contours (derived from 25m DEM)
- Major drainage
- Gas processing facility (GPF)
- Water treatment facility (WTF)



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Geocentric Datum of Australia 1994

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Figure 4-27: Infrastructure Location - GPF_WAA_03

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-0249 | Rev: 1

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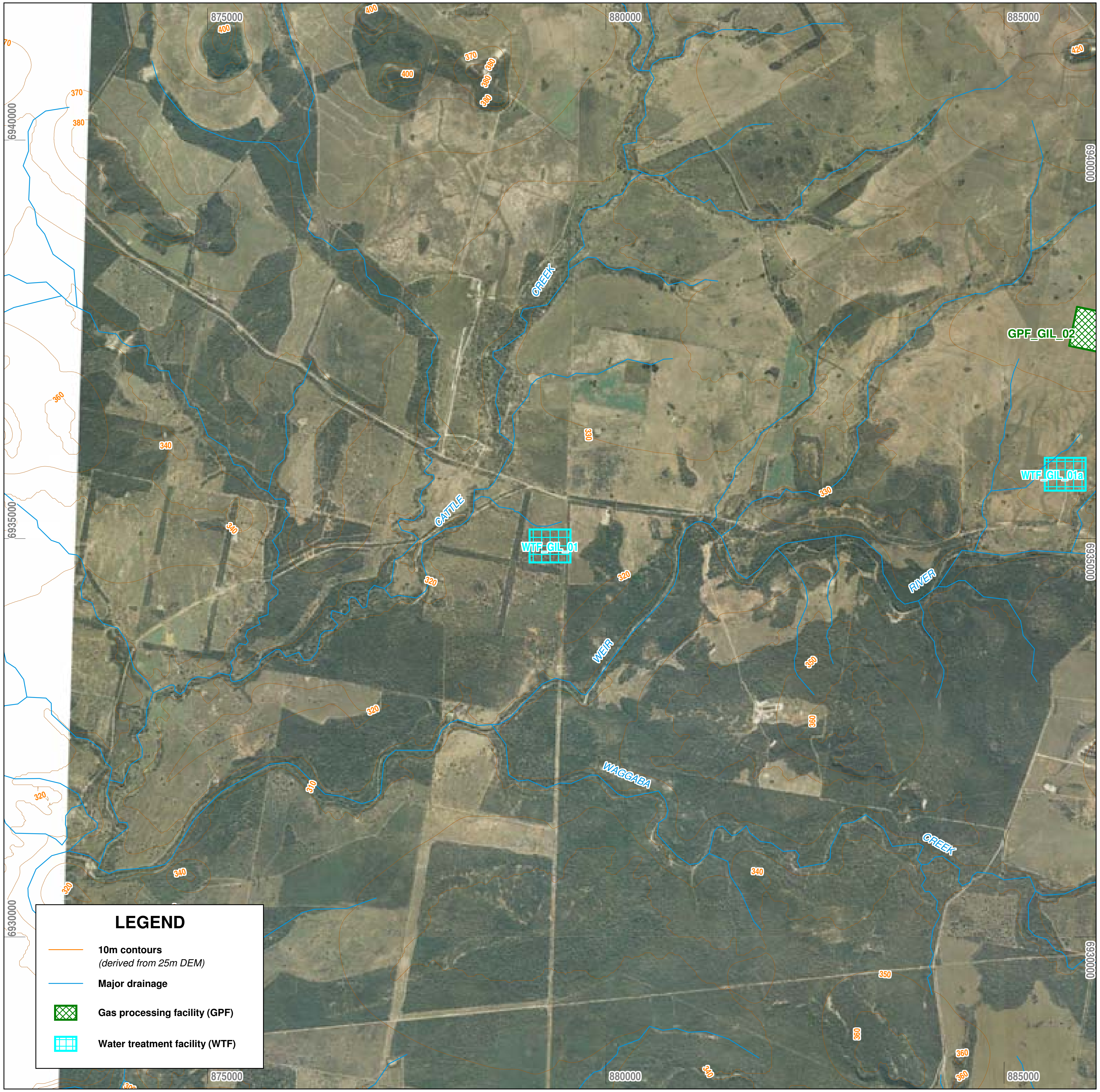

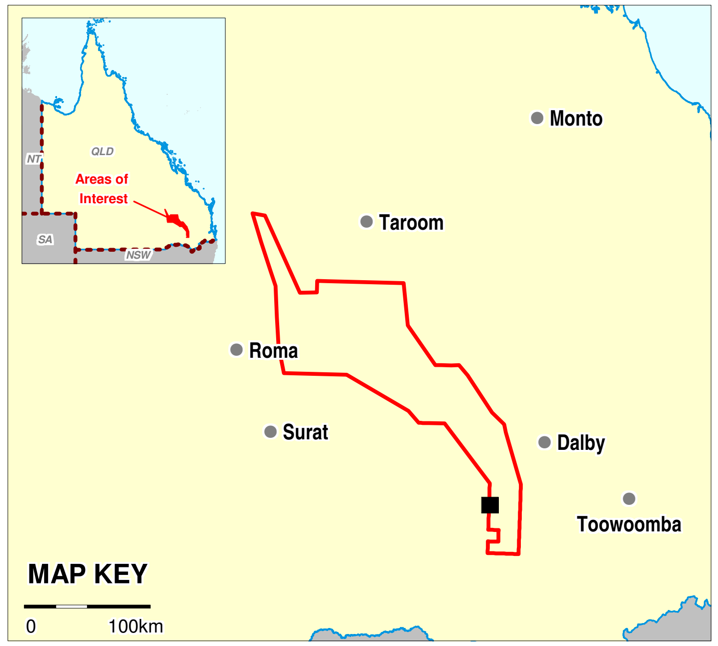

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Aerial imagery captured June 2009 and suuplied by client.

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SCALE - 1:50,000 (at A3)

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Figure 4-28: Infrastructure Location - WTF_GIL_01

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-0250 | Rev: 1

Compiled by BRISBANE INFRASTRUCTURE GIS SECTION

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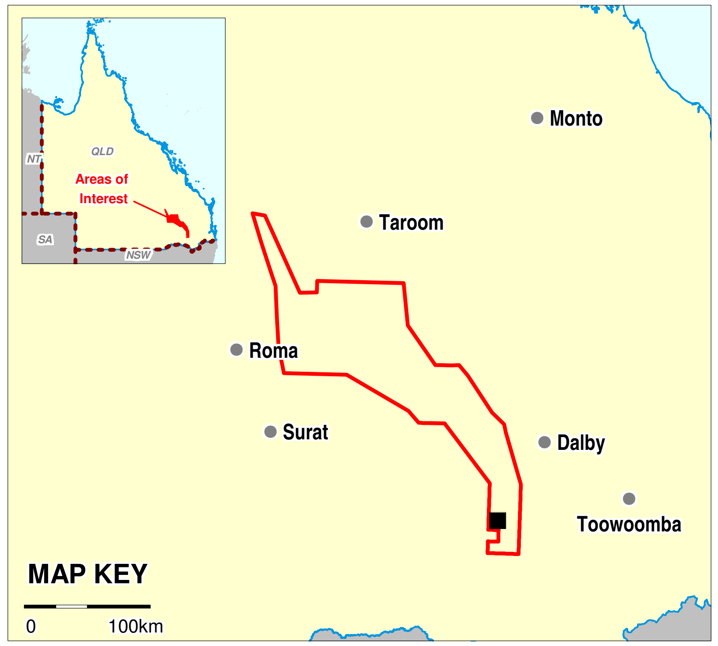

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Figure 4-29: Infrastructure Location - GPF_WAA_04

Project No: 301001-00448	Figure: 00448-00-EN-DAL-0251	Rev: 1

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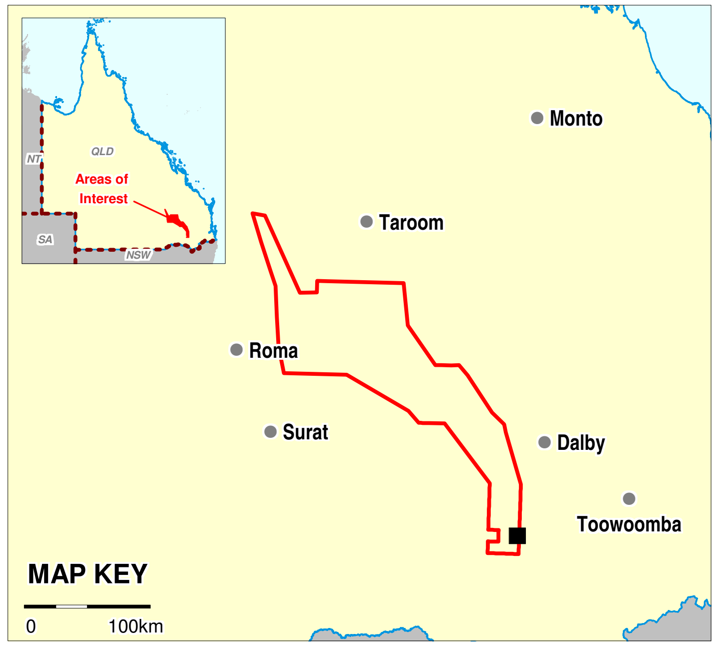
LEGEND

- 10m contours *(derived from 25m DEM)*
- Major drainage
- Gas processing facility (GPF)
- Water treatment facility (WTF)



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Map Grid of Australia Zone 55
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Figure 4-30: Infrastructure Location - GPF_ZIG_05

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-0252 | Rev: 1

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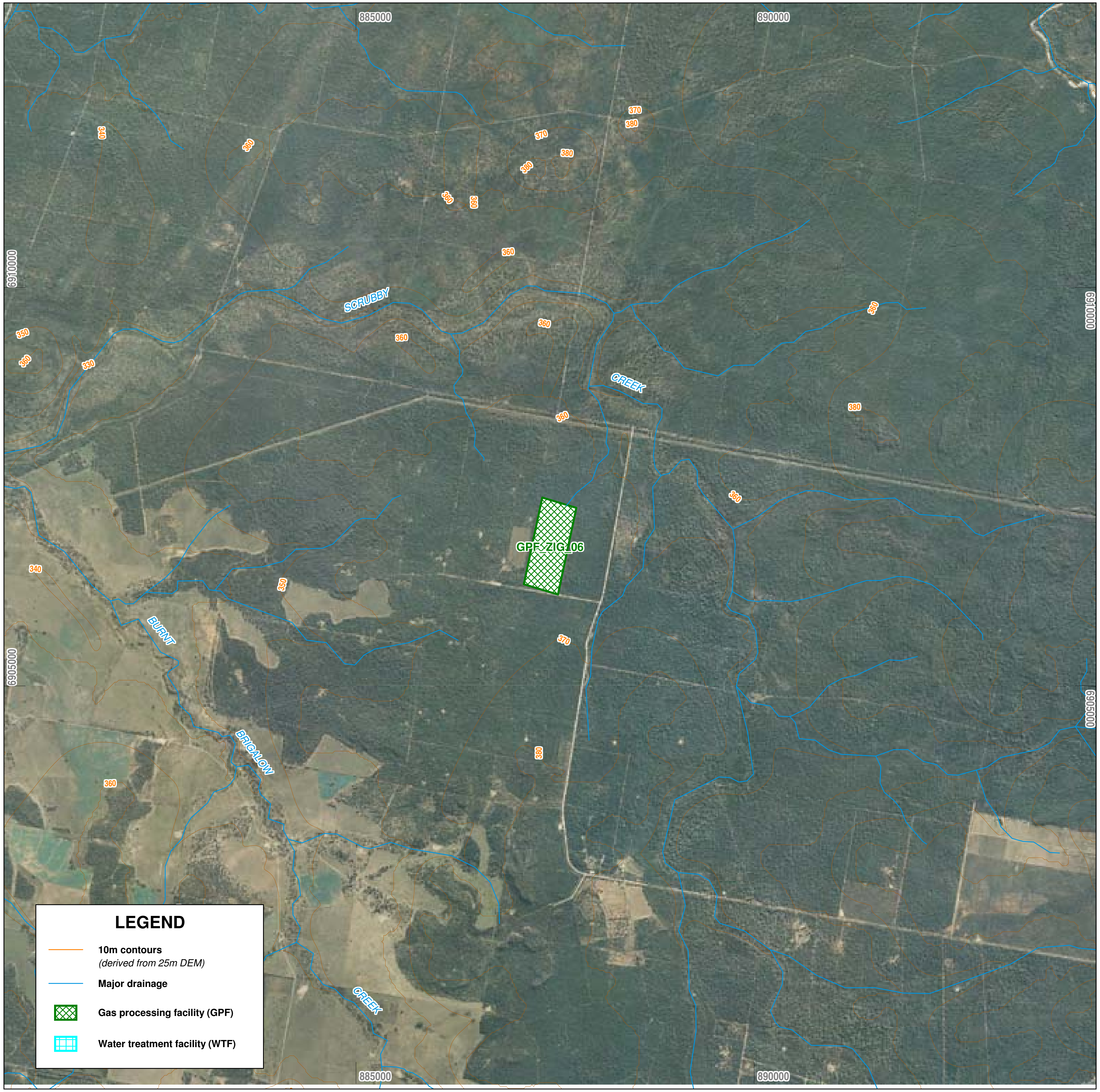

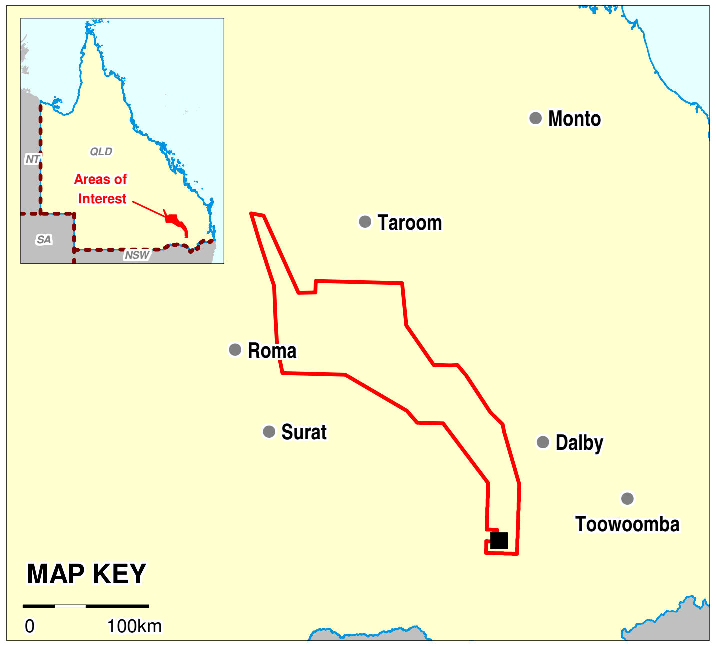

Aerial imagery captured June 2009 and suuplied by client.

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Map Grid of Australia Zone 55
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Figure 4-31: Infrastructure Location - GPF_ZIG_06

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-0253 | Rev: 1

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5. Regional flooding investigation

5.1 Existing catchment description

There are a number of catchments and associated river systems which the gas fields study area intersects.

The Condamine River is a major tributary of the Darling River, located in the upper Murray-Darling catchment. Its boundaries to the east and north are formed by the Great Dividing Range, approximately 1,400m above sea level, near Toowoomba and Warwick. Its southern boundaries comprise the much lower Herries Range, which is approximately 800m above sea level. The western boundaries comprise the Dogwood Creek sub-catchment which flows into the Condamine River, where it becomes the Balonne River.

The Dawson River catchment is a sub-catchment of the Fitzroy Basin. It has a total area of about 50,800km^2 and is bordered by the Auburn, Calliope, Ulam and Dee Ranges to the east. The Great Dividing Range lies to the west and south, and the Lynd and Canarvon, Expedition and Bigge ranges to the north west (Telfer 1995). The south western headwaters of the Dawson River flow easterly through relatively narrow valleys until about the Nathan Gorge constriction. From there the channel alters direction, flowing north, with a gradual downstream broadening of the valley to wide alluvial plains.

The Border Rivers catchment is located on the Queensland – New South Wales (NSW) border and covers about 50 000 km^2. The south eastern headwaters border the Great Dividing Range in NSW, whereas the north west headwaters border the southern section of the Condamine River catchment near Millmerran.

5.2 Hydrologic model development

Eight separate XP-RAFTS hydrologic models were developed for use in this investigation. These models were used to predict peak flow rates within the gas fields for each design rainfall event investigated as part of this analysis. This includes the 10, 20, 100 and 500 year Average Recurrence Interval (ARI) design rainfall events.

Model input data, parameters and all assumptions for the hydrologic models created for this study are detailed below in Sections 5.2.2 to 5.2.7.

5.2.1 Tenement hydrologic models

Catchment size and model descriptions for each XP-RAFTS model developed for this study are summarised in Table 5.1.

Table 5.1 Hydrologic Model Details

Model name (Refer figure 3-1)	Total catchment area (km2)	No of sub-catchments	Remarks
EPP606N (Horse Creek & adjacent systems)	956	48	Model catchment covers part of tenement EPP606 flowing north into Fitzroy River
EPP606S (Yuleba Creek)	584	16	Model catchment covers part of tenement EPP606 flowing south into Condamine-Culgoa Rivers
EPP663 (Weir River & Western Creek)	1,498	31	Model catchment covers tenement EPP663
EPP692/EPP973 (Dogwood Creek & Dulacca Creek)	5,065	90	Model catchment covers tenement EPP692, EPP973 and Northern EPP702 (See note 1)
EPP972 (Tchanning Creek)	779	18	Model catchment covers tenement EPP972
PL209 (Woleebee Creek)	490	12	Model catchment covers tenement PL209
PL226 (Condamine River)	24,916	125	See note 2
SEP692 (Kogan Creek)	202	18	Model catchment covers tenement SEP692

Note 1 Dulacca Creek is a western side tributary of Dogwood Creek Catchment which joins Dogwood Creek at a location some 45 km downstream of Gil Weir.

Note 2 The set up of PL226 hydrologic model was based on the Condamine River XP-RAFTS model previously prepared for the "Flood Investigation for Talinga Coal Seam Gas Development" (WorleyParsons, 2008). The model represents the largest catchment within the study area and has undergone minor modification to enable prescriptive modelling of the gas fields study area. This includes increased sub catchment detail within discrete areas of the lower model, and extension of the model further downstream.

5.2.2 Rainfall data

The design rainfall Intensity-Frequency Duration (IFD) data for the 10, 20, 100 and 500 year ARI design storm events were derived based upon the procedures outlined in Book 2 of Australian Rainfall and Runoff (AR&R 2001). As the overall gas fields study area and associated tenements cover a significant area, a unique IFD dataset was created for use in each of the different hydrologic models covering the respective tenement/catchment areas.

IFD datasets created for this study are included in Appendix A. Table 5.2 summarises the different parameters used to create the IFD datasets, whilst Figure 5.1 shows the location of the different hydrologic models.

Table 5.2 IFD data for respective hydrologic models

Hydrologic model ID (Refer Figure 3-1)	2 yr ARI intensities (mm/hr)	50 yr ARI intensities (mm/hr)	Skewness and geographical factors
EPP606N (Horse Creek & adjacent systems)	$^{2}I_{1} = 40.72$	$^{50}I_{1} = 71.20$	Skewness G = 0.22 Geographical Factor $F_2 = 4.24$ $F_{50} = 16.61$
	$^{2}I_{12} = 5.78$	$^{50}I_{12} = 10.75$	
	$^{2}I_{72} = 1.44$	$^{50}I_{72} = 2.96$	
EPP606S (Yuleba Creek)	$^{2}I_{1} = 40.46$	$^{50}I_{1} = 70.74$	Skewness G = 0.23 Geographical Factor $F_2 = 4.24$ $F_{50} = 16.57$
	$^{2}I_{12} = 5.72$	$^{50}I_{12} = 10.68$	
	$^{2}I_{72} = 1.41$	$^{50}I_{72} = 2.95$	
EPP663 (Weir River & Western Creek)	$^{2}I_{1} = 34.72$	$^{50}I_{1} = 60.07$	Skewness G = 0.32 Geographical Factor $F_2 = 4.33$ $F_{50} = 16.65$
	$^{2}I_{12} = 5.28$	$^{50}I_{12} = 10.04$	
	$^{2}I_{72} = 1.40$	$^{50}I_{72} = 2.63$	
EPP692/EPP973 (Dogwood Creek & Dulacca Creek)	$^{2}I_{1} = 39.32$	$^{50}I_{1} = 67.41$	Skewness G = 0.26 Geographical Factor $F_2 = 4.28$ $F_{50} = 16.75$
	$^{2}I_{12} = 5.97$	$^{50}I_{12} = 10.50$	
	$^{2}I_{72} = 1.48$	$^{50}I_{72} = 2.99$	
EPP972 (Tchanning Creek)	$^{2}I_{1} = 40.12$	$^{50}I_{1} = 70.05$	Skewness G = 0.24 Geographical Factor $F_2 = 4.26$ $F_{50} = 16.64$
	$^{2}I_{12} = 5.84$	$^{50}I_{12} = 10.57$	
	$^{2}I_{72} = 1.48$	$^{50}I_{72} = 2.99$	
PL209 (Woleebee Creek)	$^{2}I_{1} = 40.30$	$^{50}I_{1} = 70.10$	Skewness G = 0.24 Geographical Factor $F_2 = 4.26$ $F_{50} = 16.71$
	$^{2}I_{12} = 5.91$	$^{50}I_{12} = 10.62$	
	$^{2}I_{72} = 1.50$	$^{50}I_{72} = 3.00$	
PL226 (Condamine River)	$^{2}I_{1} = 38.0$	$^{50}I_{1} = 65.0$	Skewness G = 0.27 Geographical Factor $F_2 = 4.29$ $F_{50} = 16.75$
	$^{2}I_{12} = 5.8$	$^{50}I_{12} = 10.4$	
	$^{2}I_{72} = 1.50$	$^{50}I_{72} = 2.93$	
SEP692 (Kogan Creek)	$^{2}I_{1} = 36.06$	$^{50}I_{1} = 61.31$	Skewness G = 0.29 Geographical Factor $F_2 = 4.31$ $F_{50} = 16.76$
	$^{2}I_{12} = 5.41$	$^{50}I_{12} = 10.19$	
	$^{2}I_{72} = 1.43$	$^{50}I_{72} = 2.82$	

Note: ARI is the Average Recurrence Interval in years of a design rainfall event (100 Year ARI = 0.01 Average Exceedance Probability).

5.2.3 Areal reduction factors

The derived rainfall intensities presented in Table 5.2 from AR&R Book 2 (2001) are applicable strictly to a point only. The relevant rainfall intensity cannot reasonably be maintained over the large catchment areas represented by each of the hydrologic models. As such, it is typical practice to adjust point rainfall through the use of areal reduction factors (ARF) according to the catchment size of each of the hydrologic models as presented in Table 5.1. These ARF factors typically reduce the flow in large catchments. The ARF values are derived from the Depth-Area Ratio Curves shown in Figure 1.6, AR&R Book 2 (2001) for a catchment size up to 1,500 km^2. The ARF values, shown in Table 5.3 are then applied to adjust the rainfall intensities for each ARI event and storm duration. However there was a different approach taken to the hydrologic models representing tenement EPP606 (EPP606N and EPP606S).

The project facilities proposed for tenement EPP606 are located in detached local sub catchments at the upper reaches of the respective Dawson and Balonne River sub-basins. Applying ARF to the catchment rainfall based on entire catchment areas would produce reduced flows and hence was not considered appropriate for the hydrologic investigation for EPP606. As such, the design point rainfall (without ARF) have been adopted for the hydrologic models representing the separate sub-basins in tenements EPP606. This ensures design flows predicted by the hydrologic models established for these separate catchments are representative.

Table 5.3 Areal reduction factors for catchments < 1,500km^2

Storm duration (hrs)	Hydrologic model					
	EPP 606N	EPP 606S	EPP 663	EPP 972	PL 209	SEP 692
6hr	n/a	n/a	0.84	0.87	0.89	0.93
9hr	n/a	n/a	0.86	0.89	0.91	0.95
12hr	n/a	n/a	0.87	0.90	0.92	0.96
18hr	n/a	n/a	0.89	0.91	0.93	0.96
24hr	n/a	n/a	0.91	0.93	0.94	0.96
36hr	n/a	n/a	0.92	0.94	0.95	0.97
48hr	n/a	n/a	0.93	0.94	0.95	0.97
72hr	n/a	n/a	0.93	0.94	0.95	0.97

Since the catchment sizes for the Dogwood Creek & Dulacca Creek (EPP692/EPP973) and Condamine River (PL226) hydrologic models are much greater than 1,500 km2, the Depth-Area Ratio Curves in Book 2, AR&R (2001 edition) are not suitable to be used to derive a representative ARF value set for the catchments. As such, the ARF values for these two catchments are derived from the formulae for calculating ARF for large to extreme events as detailed in page 58, AR&R Book 6 (2001). The formulae provided in Book 6 have no catchment area limitation.

In these cases, the calculated ARF values were then applied to adjust the rainfall intensities for each Average Recurrence Interval (ARI) and storm duration. The resulting modelled flows were compared with a flood frequency analysis in Section 5.4.1 for all design events. The flood frequency analysis was derived from the historic flood records obtained from the gauging stations in these two catchments. It is noticed that the formulae provided in Book 6 have a tendency to overestimate the ARF values for smaller events. As such, the calculated ARF values for these two catchments have been slightly adjusted so that a best fit of the modelling results to the recorded catchment flows could be achieved.

As the peak flows compared favourably with the flood frequency analysis results, the ARF values adopted for the XP-RAFTS models established for the Dogwood Creek and Condamine River Catchments are considered appropriate and representative. Table 5.4 presents the adopted ARF values for these two catchments.

Table 5.4 Areal reduction factors for catchments > 1500km^2

Storm duration (hrs)	Hydrologic model			
	EPP692/EPP973		PL229	
	Up to 100yr ARI for EPP692 and All Events for EPP973	500 yr ARI (EPP692 only)	Up to 100yr ARI	500 yr ARI
12hr	0.82	0.75	0.81	0.7
18hr	0.85	0.75	0.82	0.7
24hr	0.89	0.75	0.82	0.7
36hr	0.90	0.75	0.82	0.7
48hr	0.90	0.75	0.82	0.7
72hr	0.90	0.75	0.82	0.7

5.2.4 Design rainfall temporal patterns

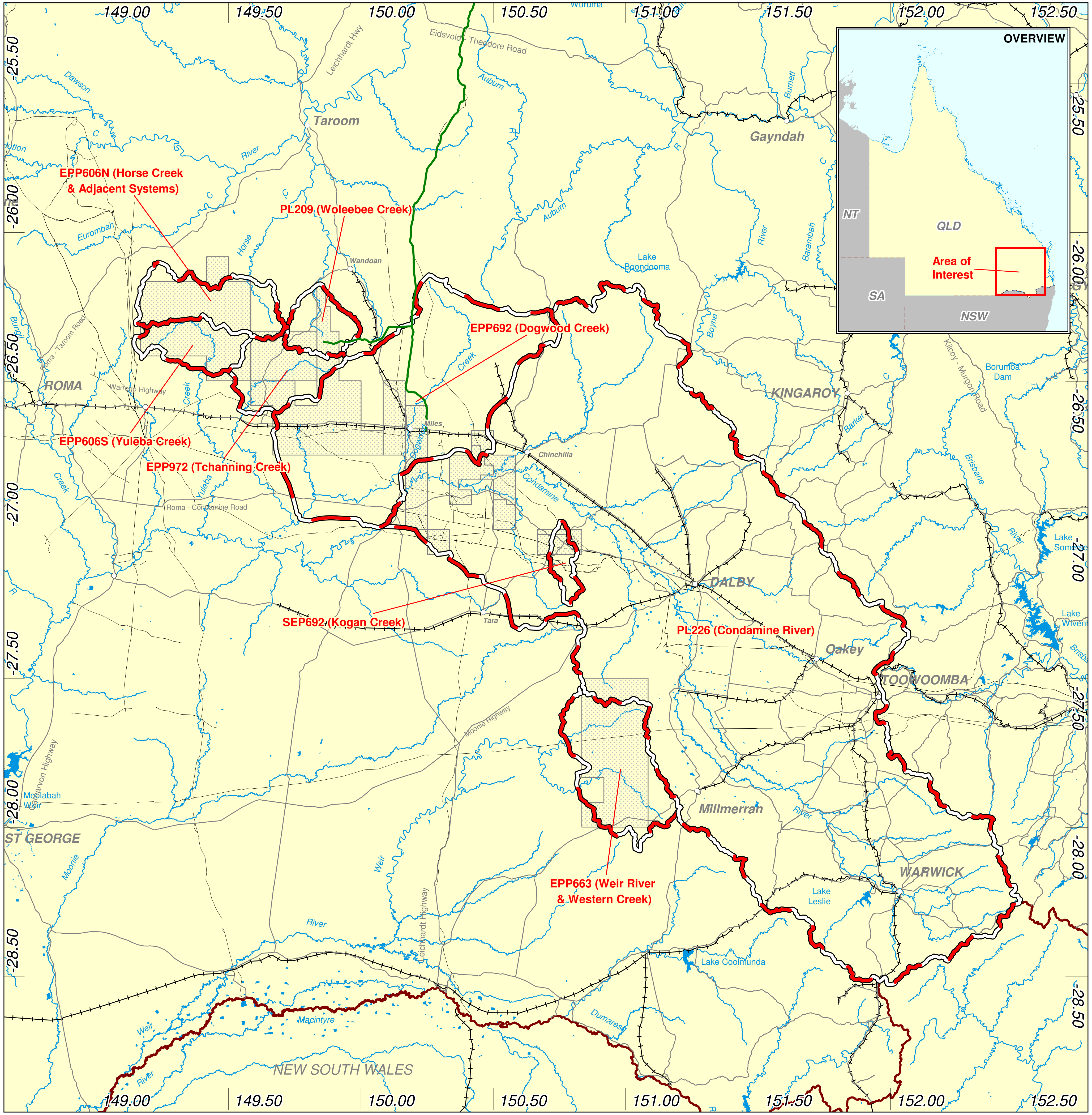
The design rainfall temporal patterns used for the respective contributing catchments to the tenement areas are presented in Table 5.5.

Table 5.5 Rainfall temporal pattern

Hydrologic model (Refer Figure 3-1)	River basin division	Temporal pattern
EPP606N (Horse Creek & Adjacent Systems)	North-East Coast Division (Fitzroy River)	Standard ARR Zone 3 Pattern
EPP606S	Murray-Darling Division	Standard ARR Zone 2 Pattern

Hydrologic model (Refer Figure 3-1)	River basin division	Temporal pattern
(Yuleba Creek)	(Condamine-Culgoa Rivers)	
EPP663 (Weir River & Western Creek)	Murray-Darling Division (Condamine-Culgoa Rivers)	Standard ARR Zone 2 Pattern
EPP692 (Dogwood Creek & Dulacca Creek)	Murray-Darling Division (Condamine-Culgoa Rivers)	Standard ARR Zone 2 Pattern
EPP972 (Tchanning Creek)	Murray-Darling Division (Condamine-Culgoa Rivers)	Standard ARR Zone 2 Pattern
PL209 (Woleebee Creek)	North-East Coast Division (Fitzroy River)	Standard ARR Zone 3 Pattern
PL226 (Condamine River)	Murray-Darling Division (Condamine-Culgoa Rivers)	Standard ARR Zone 2 Pattern
SEP692 (Kogan Creek)	Murray-Darling Division (Condamine-Culgoa Rivers)	Standard ARR Zone 2 Pattern

Source: Figure 2.2, AR&R Book 2 (2001) pp34-35.



LEGEND

- Town
- Existing railway
- State border
- Road
- Major watercourses
- Preferred pipeline alignment
- Catchment boundary
- Walloons gas fields development areas

0 50 100km

SCALE - 1:1,500,000 (at A3)

Latitude / Longitude

Geocentric Datum of Australia 1994

N E S W

Rev	Date	Revision Description	ORIG	CHK	ENG	APPD
0	22/11/2009	Issued for use	JM	DH		

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Figure 5.1 Hydrological Model Locations and Boundaries

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-2208 | Rev: 0

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5.2.5 XP-RAFTS hydrological models

The eight delineated catchments were analysed as separate models utilising the non-linear runoff routing program XP-RAFTS. XP-RAFTS uses a network model with concentrated sub-catchment storages. Hydrographs for design rainfall events were produced by routing rainfall through the storages and along channel links. This analysis involved division of the catchment into various sub-catchments, derivation of the various physical properties of the sub-catchment and assembly of the sub-catchments by nodal network. Sub-catchment hydrographs were added in sequence to the flow based on location within the nodal network. Model input data was based on 10m contours, vegetation information from QLUMP (1999), and stream profiles site surveys of vegetation roughness (where available).

5.2.6 Catchment delineation

The catchment and sub catchment definitions for the eight separate hydrologic models were delineated based on DERM 10m contour topographic data. This is discussed in more detail in Section 3.1.

The delineation of sub catchments was also based on consideration of the end use of the hydrographs (inflows into the hydraulic models) and the location of proposed infrastructure.

Appendix B provides the sub-catchment breakdowns for all of the hydrologic models developed as part of this study.

5.2.7 Hydrologic model parameters

Rainfall loss model

Rainfall loss on each sub-catchment in the hydrologic models was applied using an initial and continuing rainfall loss model. Design loss parameters for the XP-RAFTS model were based on guideline values as recommended within AR&R (2001) and Queensland Urban Drainage Manual (2nd Ed. 2007)(QUDM) recommendations.

A review of previous studies undertaken by WorleyParsons (tenement PL226 in the Condamine River catchment) was also completed. The relevant hydrologic model had been calibrated against stream flow gauging stations throughout the catchment. As the Condamine River catchment is located adjacent to and has similar soil type properties to the catchments contributing to the subject tenements in this investigation, it was deemed appropriate to adopt these values for all hydrologic models developed for this study. These values are within the recommended values of AR&R (2001) and QUDM. The adopted loss parameters applied to the XP-RAFTS model are summarised in Table 5.6.

Validation of the model for tenement EPP692 as discussed in more detail in Section 5.4.1 showed an increased continuing loss of 2.5 mm/hr was appropriate for this catchment. This value resulted in a good fit with the flood frequency analysis of gauged flows.

Table 5.6 Rainfall loss parameters (excluding Model EPP692)

Loss type	Pervious area	Impervious area
Initial loss (mm)	20	0
Continuing loss (mm/hr)	2	0

Sub-catchment storage parameter (ßx factor)

Model parameters for sub-catchment storage have been selected from recommended design values for vegetation types. The sub-catchment storage (ßx factor) is altered according to a scale typically from 0.5 (impervious surfaces PERN = 0.015) to 3 (forest PERN = 0.1). The ßx storage coefficient is typically used when calibrating a gauged catchment. During calibration of a gauged catchment the ßx parameter is modified to suit the storage of the sub-catchments to alter the shape, peak and timing of a hydrograph. As previously mentioned, the Condamine River model previously prepared by WorleyParsons has been calibrated to gauging stations. This calibration confirmed that the timing and shape of the modelled hydrographs with the aforementioned loss values closely matched the gauging station hydrographs. Consideration was also given to the end use of the sub catchment hydrographs, namely local hydrograph application to the TUFLOW hydraulic models, where catchment storages are often better represented. As such, it was not deemed necessary to adjust the ßx factor from the default value.

Given these considerations, a storage coefficient (ßx) factor of 1.0 was adopted for all hydrologic models as shown in this study.

Catchment land use, imperviousness and PERN values

The adopted percentage impervious and corresponding pervious area roughness parameters (PERN values) for land uses throughout the catchments are summarised in Table 5.7. Land use has been derived from review of orthophoto data, review of GeoScience Australia's Native Vegetation GIS layers, and the Queensland Land Use Mapping Project (QLUMP, 1999) datasets. All fraction impervious (percentage impervious) parameters have been based on recommendations in QUDM (2007). All impervious areas were assigned a PERN value of 0.015.

A summary of impervious and pervious areas as well as corresponding PERN values for each sub-catchment in the hydrologic models is presented in Table 5.7.

Table 5.7 Catchment land use parameters

Description	Impervious %	Pervious Mannings 'n'	Impervious Mannings 'n'
Native / thick vegetation	0	0.100	0.015
Cleared vegetation (farmland)	2	0.070	0.015
Town	45	0.025	0.015
Water body	100	0.015	0.015

Channel routing / roughness coefficients

Routing of flow within the catchments for each hydrologic model is performed by the Muskingum-Cunge method. This method is an extension of the Muskingum routing method, however incorporates

physical attributes that were previously not included in the Muskingum method such as channel roughness, shape and slope. This allows the deduction of suitable Muskingum routing parameters.

Existing channel roughness conditions in the main channel and flood plains were evaluated using orthophoto imagery and inspection of the site where possible. Manning's 'n' values used in the hydrology model were based on observed data and were varied to simulate roughness across the representative cross sections.

The range of channel roughness values used for various waterway conditions is summarised in Table 5.8.

Table 5.8 XP-RAFTS channel routing roughness

Flow profile	Roughness (Manning's 'n')	
	Lower bound	Upper bound
Low flow waterway	0.060	0.120
Farmland / riparian vegetation & native forest	0.080	0.120

5.3 Hydraulic model development

The overall study area covers a significant number of creeks and river systems, many of which have had limited or no previous numerical modelling based flooding investigations undertaken.

The purpose of this investigation is therefore to provide estimations of flooding behaviour for the major waterways flowing through each tenement area. WorleyParsons has constructed a series of nine TUFLOW one dimensional (1D)/two dimensional (2D) hydrodynamic flood models to facilitate a detailed representation of flood behaviour.

All details concerning model development, baseline data, assumptions and parameters are detailed in Sections 5.3.1 to 5.3.2 below.

5.3.1 Modelling software

Hydraulic analysis of the study area was undertaken using the coupled 1D and 2D finite difference model TUFLOW. The model can simulate unsteady hydrodynamic flow in two directions on a rectilinear grid as well as 1D unsteady hydrodynamic flow through waterway structures such as culverts. The model is based on a finite difference solution scheme able to compute both subcritical and supercritical flow regimes.

The 1D/2D TUFLOW model is suited to simulation of dynamic hydraulic behaviour of overland flow in rural areas. Based on this and TUFLOW's ability to couple hydraulic structures such as culverts and bridges at road crossings, the modelling system was considered the most appropriate investigative tool for the topographic characteristics of the tenement areas.

Major advantages of a combined 1D and 2D modelling approach over traditional 1D approaches include:

- Full topographic survey terrain models are used, rather than selected, discrete cross sections
- Flow patterns are dictated by the influence of topography and surface roughness conditions rather than by 'forced' flow paths, as used in quasi-two-dimensional networks
- Flow directions and paths can vary with stage and flow conditions

- Production of detailed output of flow patterns, flood rise and fall animations, and output suitable for direct GIS interfacing. This allows production of accurate depth of flooding, velocity and hazard maps as well as area of influence maps.

5.3.2 Hydraulic model construction and parameters

The 1D / 2D TUFLOW models constructed for the tenements consist of the following elements, each of which are described in more detail below.

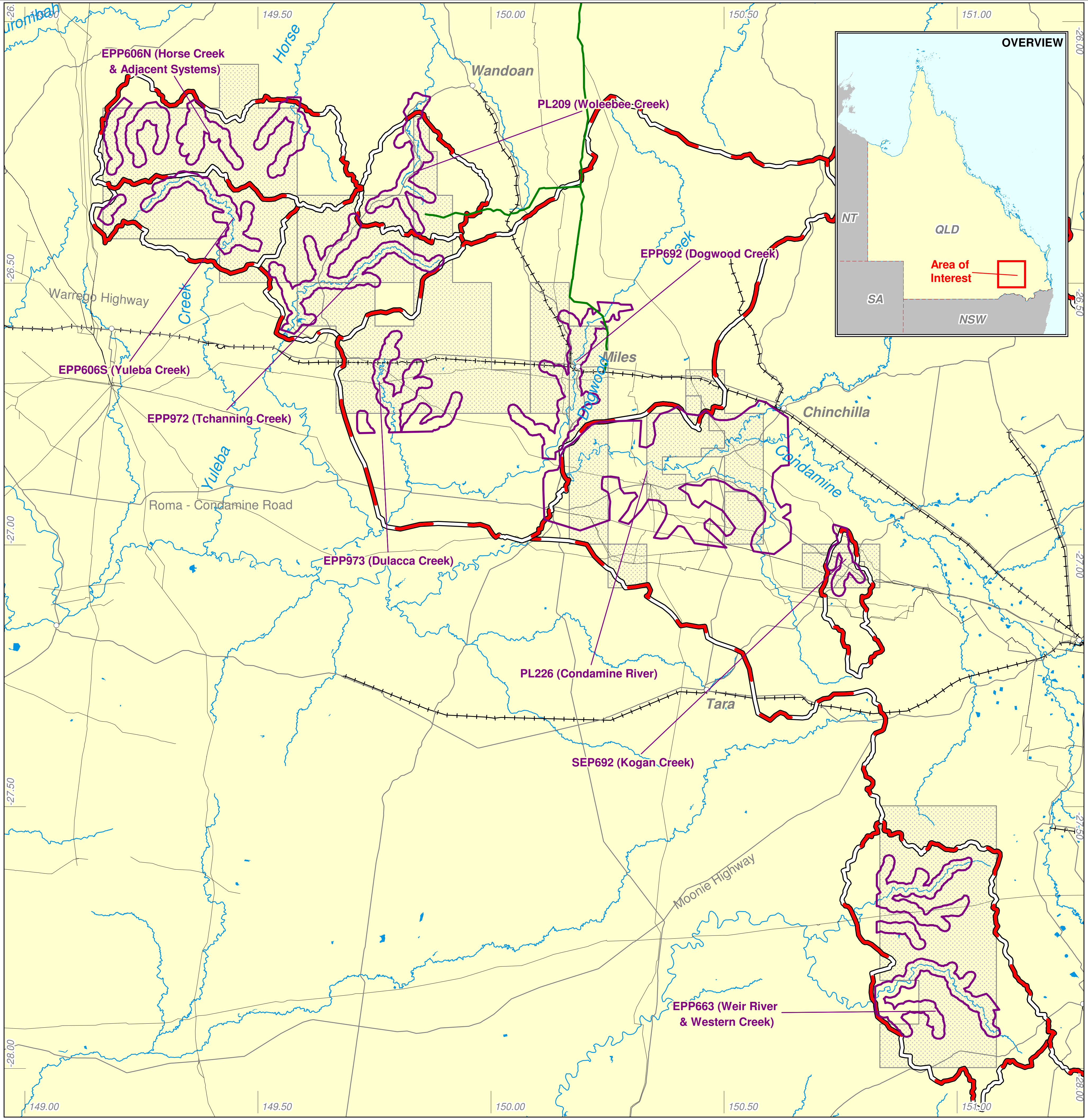
- A 2D curvilinear grid representing the topographic detail within the study area extracted directly from the Digital Elevation Model (DEM) constructed using photogrammetry data provided by Australia Pacific LNG
- Manning's roughness map covering the area of interest
- 1D elements within the 2D grid extent that represent hydraulic structures
- Downstream water level boundaries applied at the respective model outlets,
- Rainfall boundary conditions simulating the local sub-catchment inflow response within the 2D modelling area.

Table 5.9 summaries the adopted model grid for each of the nine hydraulic models.

Figure 5.2 illustrates each of the respective modelling areas, whilst more detailed individual model layout plans for each of the nine individual models are included in Appendix C.

Two dimensional topographic grid

The 2D model topography was derived using the DEM constructed from the photogrammetry data as supplied by AAMHatch. A balance between the number of computational points, level of modelling detail, density of topographic survey points and model run times were an important factor in terms of delivering suitably accurate outcomes in a timely and efficient manner. The decision on the appropriate grid spacing to adopt in the 2D model is a critical one and as such a significant amount of thought and testing has gone into the selection of an appropriate grid size for all models used in this study. Table 5.9 summarises the adopted model grid size for each of the nine hydraulic models.



LEGEND

- Town
- Existing railway
- State border
- Road
- Major watercourse
- Preferred pipeline alignment
- Catchment boundary
- Hydraulic model boundary
- Walloons gas fields development areas

0 25 50km

SCALE - 1:850,000 (at A3)
Latitude / Longitude
Geocentric Datum of Australia 1994

N W E S

Rev	Date	Revision Description	ORIG	CHK	ENG	APPD
0	22/11/2009	Issued for use	JM	DH		

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Figure 5.2 Hydraulic Model Boundary Overview

Project No: 301001-00448	Figure: 00448-00-EN-DAL-2209	Rev: 0

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Table 5.9 Hydraulic model grid size summary

Model ID (Refer Figure 4.1)	2D Domain Grid Size
EPP606N (Horse Creek & Adjacent Systems)	20m
EPP606S (Yuleba Creek)	15m
PL209 (Woleebee Creek)	20m
EPP972 (Tchanning Creek)	20m
EPP973 (Dulacca Creek)	15m
EPP692 (Dogwood Creek)	20m
PL226 (Condamine River)	30m
SEP692 (Kogan Creek)	20m
EPP663 (Weir River & Western Creek)	20m

The model grid sizes shown in Table 5.9 allow for sufficient detail to be achieved whilst maintaining practical model simulation times. The extent of hydraulic modelling and the DEM utilised for each model area is shown in Appendix C. The DEMs created from the photogrammetric data are used as the basis for the hydraulic models, and as such minor modifications have been carried out in some discrete areas to improve representation of some structures, road crossings and other drainage features.

The 2D hydraulic models, depending on the location of the modelling area, are based on a horizontal datum of either Map Grid of Australia 1994 (MGA94) Zone 55 or Zone56 and use Australian Height Datum (AHD) for elevation.

One dimensional hydraulic structure elements

In a full 2D modelling environment it is often not possible to accurately describe the hydraulic behaviour of structures such as culverts and bridges. This is due to the fact that grid cell sizes often exceed the dimensions of various structures in addition to the grid cells only representing bottom friction and consequently no roof friction or specific hydraulic structure losses. As a result, hydraulic structures are often more accurately modelled in a 1D modelling environment within the 2D domain, thus allowing prescriptive modelling of the exact characteristics of the various structures.

Within some of the 2D models, 1D model elements have been introduced to represent various floodplain structures. Where larger scale structures such as bridges over major waterways were present, these have typically been modelled by way of a layered 2D flow constriction as discussed below.

Insertion of 1D elements into the 2D domain has been adopted to represent some key hydraulic structures at major structure locations. A detailed summary is provided in Table 5.10. Appendix C provides graphical representation of the included 1D structure locations within the various TUFLOW models. As constructed drawings were typically not available for the hydraulic structures. Invert levels were interpolated by way of review of the surrounding topography and aerial photography.

Table 5.10 One dimensional hydraulic structure summary

Model ID	Location Description	Inlet Invert Level (m AHD)	Outlet Invert Level (m AHD)	Description
PL226	Glenolive Road/ Condamine River	272.1	272	7/3600 x 1600 RCBC
PL209	Jackson Wandoan Road/ Ramyard Creek	298.7	298.5	3/3650 x 2180 RCBC
EPP972	Jackson Wandoan Road/ Noonga Creek	327.0	326.8	6/2450 x 2200 RCBC
	Jackson Wandoan Road/ Tchanning Creek	321.0	320.8	9/1200 x 1200 RCBC

Two dimensional hydraulic structure elements

Due to the expansive nature of the study areas and the large watercourses and structures to be represented, in most cases bridges were represented in the 2D scheme.

Flow constrictions allow the constriction of flow across a 2D cell in a number of ways to represent large hydraulic structures such as bridges and banks of large scale box culverts. Typically approach has been adopted for large bridge structures through use of TUFLOW's layered flow constriction (2d_lfcsh) capability. Layered flow constrictions allow the attributes of the structure to be varied with water depth. This provides the opportunity to model in 2D the flow under and over a bridge deck, whilst specifying individual loss rates for each layer.

Table 5.11 specifies which structures in each hydraulic model have been descriptively modelled using this feature. Figure 5.3 to Figure 5.12 shows the structures modelled using the flow constriction.

Table 5.11 Two dimensional hydraulic structure summary

Model ID (Refer Appendix C)	Location	Structure description
EPP692 (Dogwood Creek)	Miles	Warrego Highway Bridge crossing (Refer Figure 5.3) Single carriageway concrete beam bridge
		Rail Bridge crossing (Refer Figure 5.4) Single rail wooden beam bridge
	Columboola Creek	Leichardt Highway bridge crossing of Columboola Creek (Refer Figure 5.5) Single carriageway concrete beam bridge
EPP973 (Dulacca Creek)	Dulacca	Warrego Highway Bridge crossing of Dulacca Creek (Refer Figure 5.6)

Model ID (Refer Appendix C)	Location	Structure description
		Single carriageway concrete beam bridge
		Rail Bridge crossing of Dulacca Creek (Refer Figure 5.7) Single rail wooden beam bridge
PL226 (Condamine River)	Condamine	Chinchilla Tara Road bridge crossing of Condamine Creek (Refer Figure 5.8) Single carriageway beam bridge
		Leichardt Highway bridge crossing of Condamine Creek (Refer Figure 5.9) Single carriageway beam bridge
PL209 (Woleebee Creek)	Woleebee Creek	Jackson Wandoan Road bridge crossing (Refer Figure 5.10) Single carriageway concrete beam bridge
		Jackson Wandoan Road bridge crossing (Refer Figure 5.11) Single carriageway wooden beam bridge
SEP692 (Kogan Creek)	Kogan	Dalby Kogan Road bridge crossing (Refer Figure 5.12) Single carriageway wooden beam bridge

Figure 5.3 Warrego Highway crossing of Dogwood Creek at Miles

Figure 5.4 Rail crossing of Dogwood Creek at Miles

Figure 5.5 Leichardt Highway bridge crossing of Columboola Creek

Figure 5.6 Warrego Highway crossing of Dulacca Creek at Dulacca

Figure 5.7 Rail crossing of Dulacca Creek at Dulacca

Figure 5.8 Chinchilla – Tara Road Bridge crossing of the Condamine River

Figure 5.9 Leichardt Highway Bridge crossing at Condamine River

Figure 5.10 Jackson – Wandoan Road bridge crossing of Woleebee Creek

Figure 5.11 Jackson Wandoan Road Bridge crossing at Woleebee Creek (oblique photograph unavailable)

Figure 5.12 Dalby – Kogan Road bridge crossing of Kogan Creek

Model boundary conditions

Inflow boundaries

The hydrographs represented at each model inflow location were extracted from the XP-RAFTS hydrological model over the full range of design flood events analysed (10, 20, 100 and 500 year ARI events). Hydrographs were applied to the 2D model domains by way of direct application of either the representative local or total catchment hydrograph to the delineated sub catchment area or inflow boundary for each model as presented in Appendix C.

Tailwater boundaries

As the majority of the waterway systems modelled as part of this investigation are classified as ephemeral, a normal depth boundary condition was adopted at each representative waterway downstream boundary. Boundaries were typically established at areas of no dry weather flow. TUFLOW automatically generates a stage discharge curve based on the boundary cross section topography, Mannings 'n' value at the boundary location, and a specified water surface slope (gradient). Numerous model iterations were undertaken to determine the most accurate slope parameters for each boundary location. It was found that due to the flat nature of the topography in the study areas, adopted water surface slopes generally ranged from 0.002m/m to 0.010m/m. Boundary condition locations in each hydraulic model are graphically represented in Appendix C.

2D model roughness

GIS roughness maps covering the nine (9) hydraulic modelling areas were created to define the hydraulic roughness spatially across the floodplains. The floodplain roughness maps applied to each of the hydraulic models are illustrated in Appendix D. Each grid cell is assigned a Manning's 'n' roughness value based upon land use defined on the map. The GIS layer of existing land use was generated using a combination of orthophoto imagery, site observations (including oblique photography) and previous experience in 2D hydraulic modelling applications. The Manning's "n" roughness parameters adopted in the model ranged from 0.015 for open water bodies through to 0.250 for Towns / District Centres. These values are typical of those adopted for floodplain roughness for studies of this nature. Table 5.12 documents roughness assigned to each land use.

Table 5.12 Adopted roughness parameters

Land use type	Manning's 'n' roughness
Water Body	0.015
Road Carriageway	0.025
Cleared Land/Agriculture	0.040
Cleared Land/Sporadic Vegetation	0.045
Light Vegetation	0.060
Bushland/Natural Environments	0.080
Buildings/Homestead	0.100
Township/District Centre	0.250

5.4 Calibration and validation

5.4.1 Hydrologic modelling

Validation of the eight hydrologic models has been undertaken to ensure the models are producing reliable and accurate flow predictions for each rainfall event throughout the respective catchments.

Depending on the availability of stream flow data, models were either validated against existing stream flow data using flood frequency analysis methodology, or by comparisons against the Rational Method, as outlined in the Queensland Urban Drainage Manual (2nd Ed, 2007). The following sections summarise the flow validation undertaken for each hydrologic model.

PL226 (Condamine River)

As discussed in Section 5.2, the XP-RAFTS model established for Tenement PL226 was updated from the Condamine River model developed for the Flood Investigation for Talinga Coal Seam Gas Development (WorleyParsons 2008). The Talinga model was calibrated to the 1988 historical rainfall event.

To validate flows predicted by the current updated model, the calculated flows for various design flood events were compared to the flood frequency analysis undertaken for the catchment using approximately 80 years of flood data recorded from 1921 to 2004 at Chinchilla Weir (Gauge Stations 422308B and 422308C). The peak annual flow data for the gauge stations (obtained from the DERM) was reviewed and extracted to undertake a flood frequency analysis using Log Pearson III (LP3) distribution to fit the available flood data using the following approaches:

- Using the entire 80 years data produces best fit to the lower end but a poor fit for larger events. This is attributed to the flow data being heavily weighted to smaller rainfall events
- Conducting a second LP3 analysis by removing the flows with a value less than 600m^3/s. This approach produces a better fit for the larger events.

Based on the outcomes of the above flood frequency analyses, the LP3 distribution results produced by the entire flow data have been adopted for flood events up to 20 year ARI while the LP3 distribution results produced by removing the flows lesser than 600m3/s have been adopted for flood events larger than 20 year ARI. The 10, 20, 100 and 500 year ARI peak flows calculated by the XP-RAFTS hydrological model upstream of the Chinchilla Weir (Node “NE_15”) were then compared to the outcomes from the flood frequency analysis. Table 5.13 presents the comparison of the peak flows. It can be seen that the flows calculated by XP-RAFTS compare favourably to the flood frequency analysis results.

Table 5.13 Comparison of flows at Chinchilla Weir (422308C)

ARI event	Flood frequency analysis (Log Pearson III distribution) (m3/s)	XP-RAFTS flow (m3/s)
10yr ARI	1,268	1,424
20yr ARI	1,694	1,664
100yr ARI	3,590	3,304
500yr ARI	3,950	3,951

To further validate the current XP-RAFTS model developed for Tenement PL226, the 100 year ARI design flows calculated by the model created for the Talinga Coal Seam Gas Development (WorleyParsons 2008) were compared with the flows calculated by the current model at the following locations:

- Node Sub_36 (Confluence of Charleys Creek and Condamine River)
- Node Sub_43 (original Talinga model outlet).

The calculated 100 year ARI design flows at the above two locations are presented in Table 5.14. It can been seen from the table that the flows calculated by the current model compare well with the original Talinga model and as such, the current model is considered to be representative. This model was therefore used to calculate inflow boundary conditions for the hydraulic model developed for the Condamine and the surrounding tenements.

Table 5.14 100 year ARI peak flow comparison – PL226 model

Location	Current model peak flow prediction (36hrs) (m3/s)	Talinga model peak flow prediction (24hrs) (m3/s)
Sub_36	4,396	4,386
Sub_43	4,716	4,540

EPP692 (Dogwood Creek)

Tenement EPP692 is within the Dogwood Creek catchment. Historical peak annual flows covering a period of 60 years from 1946 to 2005 were available for the catchment at Gil Weir gauge station (42220B). The peak annual flow data for this gauge station (obtained from the DERM) was reviewed and extracted to undertake a flood frequency analysis using Log Pearson III (LP3) distribution to fit the available flood data. The 10, 20, 100 and 500 year ARI peak flows calculated by XP-RAFTS at Gil Weir (Node "GIL_OUT") were then compared to the outcomes from the flood frequency analysis. Table 5.15 below presents the comparison of the flood flows.

Table 5.15 Comparison of flows at Gil Weir (422202B)

ARI event	Flood frequency analysis (Log Pearson III distribution) (m3/s)	XP-RAFTS flow (m3/s)
10yr ARI	410	454
20yr ARI	618	649
100yr ARI	983	1184
500yr ARI	1175	1413

As can be seen from the above table, the design flows predicted by the XP-RAFTS model at Gil Weir were found to be consistently higher than the expected flows from the flood frequency analysis and particularly with reference to the larger and extreme flood events. Increased flow predictions are associated with a number of possible factors including landuse changes within the catchment and flow

breakouts around the gauging station site through the floodplain. Since the model predicted flows are in the anticipated upper region of the flood frequency analysis results, the XP-RAFTS model established for EPP692 Dogwood Creek catchment was considered adequately representative for the catchment and has conservatively calculated design event inflow hydrographs for the hydraulic model developed for the EPP692 tenements.

EPP606N and PL209

Suitable flow gauges for validation within the catchments or close to the outlets of the two XP-RAFTS models in the Dawson-Fitzroy River Basin (EPP606N & PL209) were not available and consequently direct validation to gauge stations was not possible. Hence, a regional flood frequency analysis has been carried out in accordance with Book IV, Section 2 of AR&R (2001). Peak series data reports were acquired from the DERM for seven gauge stations. Flood frequency analyses were carried out for each of these peak series and various distribution types were plotted against the recorded data. The distribution that best fit each data set was selected. Figure 5.13 summarises the results of the 100 year ARI flood frequency flows versus catchment area along with a line of best fit. Flows at the outlets of the XP-RAFTS models were also plotted for comparison. As shown in Figure 5.13 modelled flows compare well to the gauge data in the Dawson River Basin.

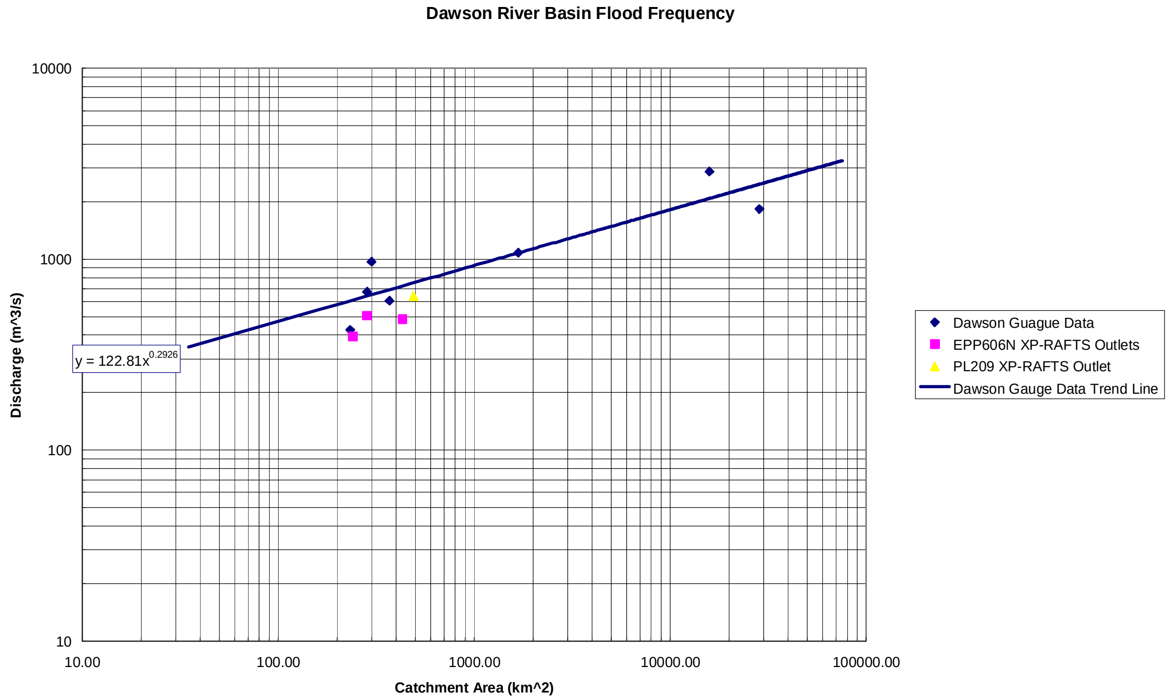


Figure 5.13 Dawson River Basin flood frequency

EPP606S, EPP972 and EPP973

As for the models in the Dawson River Basin, the three north-western models in the Condamine River Basin (EPP606S, EPP972 & EPP973) did not have suitable gauge stations within the model area to allow validation directly to gauged data. Consequently, a flood frequency analysis of seven gauge peak series data reports obtained from the DERM was carried out to determine a regional flood frequency relationship. The analysis has been carried out in accordance with Book IV, Section 2 of AR&R (2001). Flood frequency analyses were carried out for each of the peak series and various distribution types were plotted against the recorded data. The distribution that best fit each data set

was selected. Figure 5.14 summarises the results of the 100 year ARI flood frequency flows versus catchment area along with a line of best fit. Flows at the outlets of the XP-RAFTS models are included for comparison. As shown in Figure 5.14, modelled flows compare well to the gauge data in the Condamine River Basin.

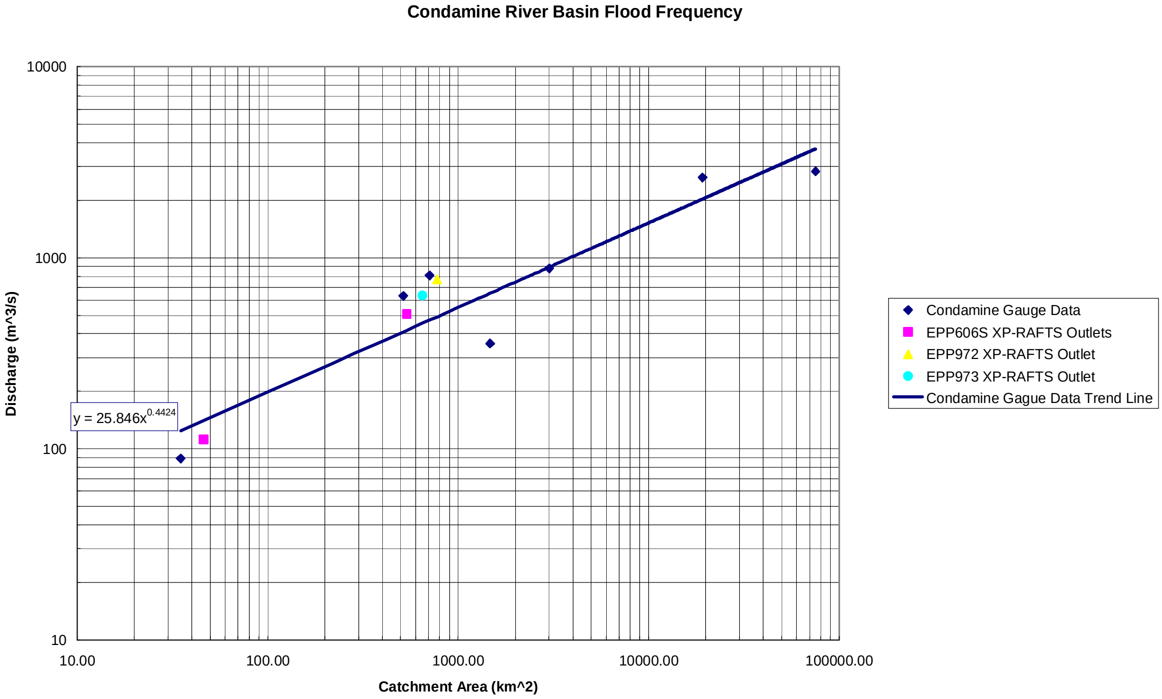


Figure 5.14 Condamine River Basin flood frequency

EPP663 (Weir River & Western Creek)

The catchments for the EPP663 model did not have any suitable flow gauges available from the DERM for a direct validation of the model. A regional flood frequency analysis has been carried out in accordance with Book IV, Section 2 of AR&R (2001). Five gauges within the Border Rivers basin were selected. As presented in Figure 5.15 the Border Rivers gauges do not show a strong correlation. Therefore, an additional six gauges from the adjacent Balonne Condamine Basin were selected. Although model EPP663 is in the Border Rivers Basin, it is closer geographically to many of the gauges in the Condamine River Basin and is expected to compare well to the flow-area relationship for the Condamine gauges. Peak series data reports were acquired from the DERM for each of the eleven selected gauge stations. A flood frequency analysis was conducted for each data set and the calculated distributions were plotted against the recorded data to select the most representative distribution type at each site. Figure 5.15 summarises the results of the 100 year ARI flood frequency flows versus catchment area. The gauges within the Condamine basin show a much stronger relationship than the Border Rivers gauges. A best fit relationship for all gauges was plotted along with the flows at the outlets of the XP-RAFTS model. As seen in Figure 5.15 the modelled flows compare well to the gauge data, particularly for the nearby Condamine gauges.

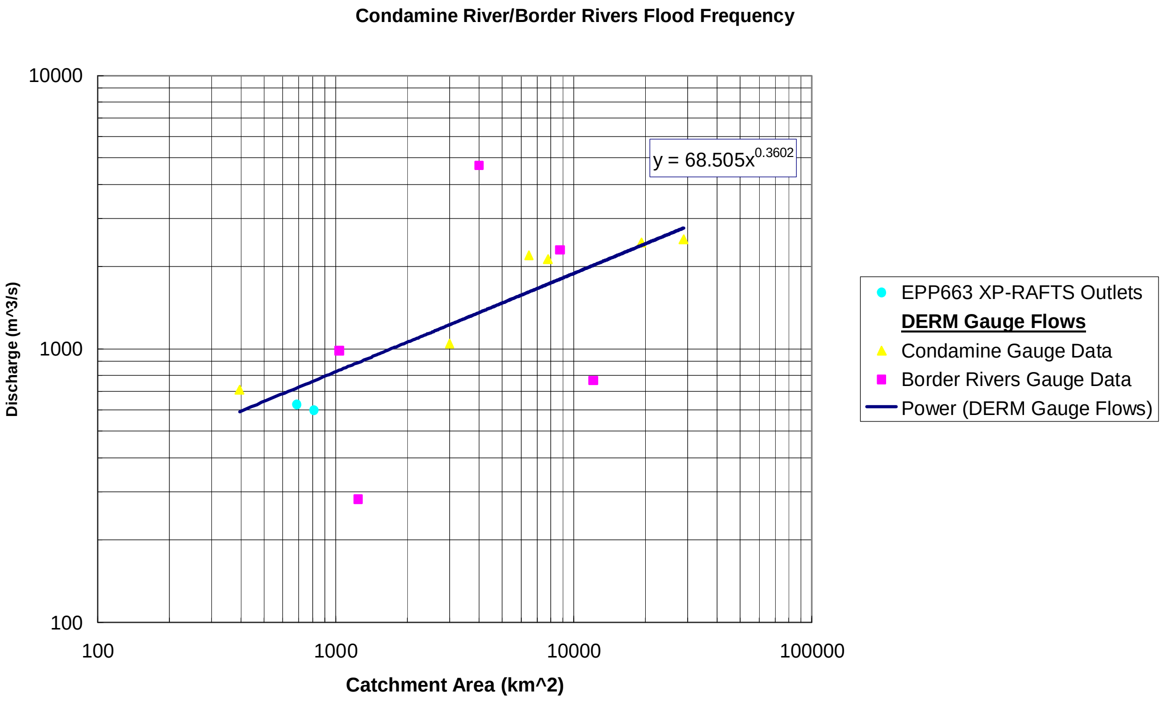


Figure 5.15 Condamine River/Border Rivers Basin flood frequency

Model Validation by Rational Method

As a result of lack of recorded stream flow data available, verification of the XP-RAFTS model for the SEP692 tenement catchment was undertaken by comparison to the Rational Method as described in the Queensland Urban Drainage Manual (2007). Rational Method calculations were also undertaken for EPP606N, EPP606S, EPP663, EPP972, EPP973 and PL209 prior to the Regional Flood Frequency Analyses presented above. These calculations provided further validation of the respective model results.

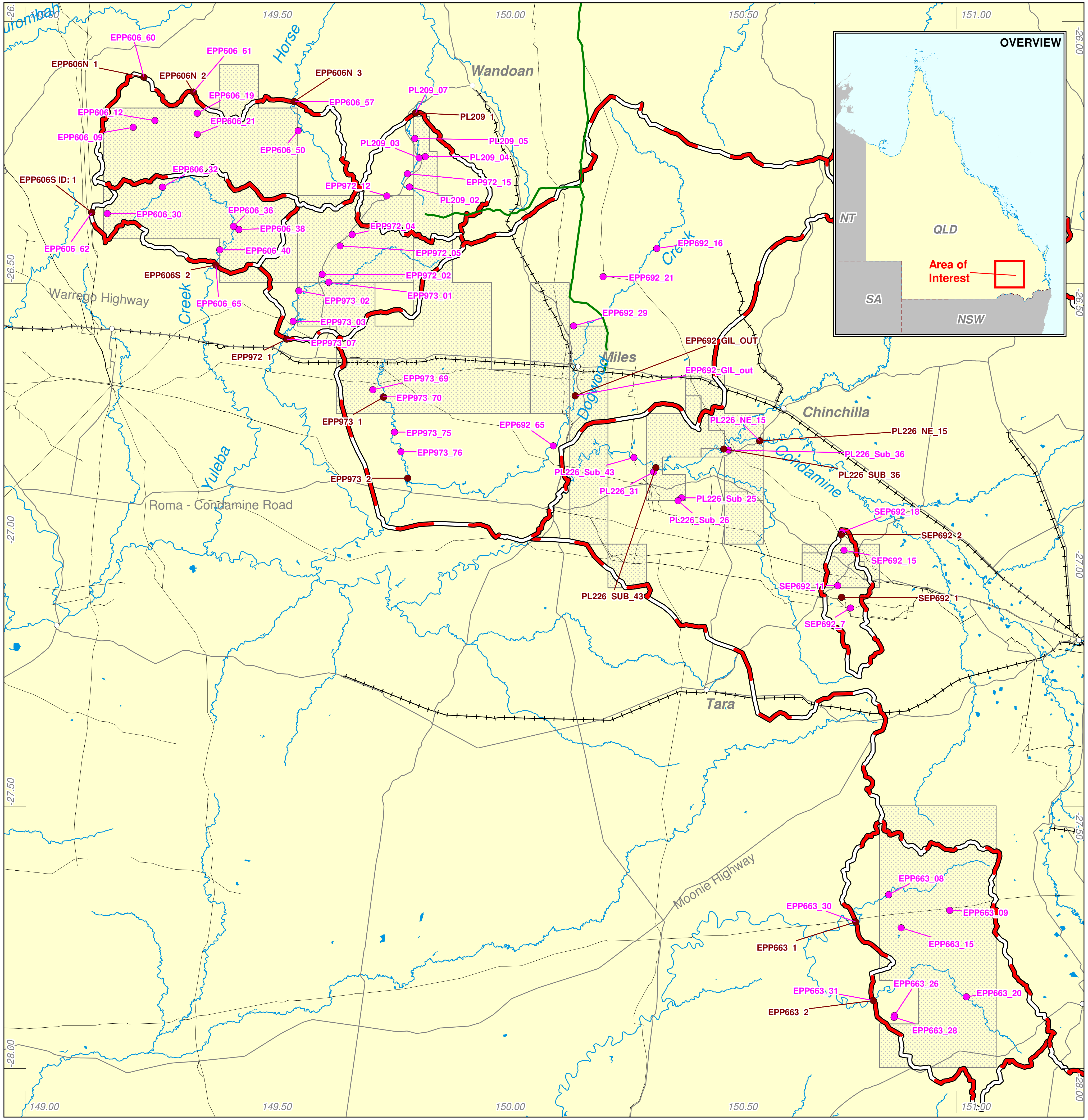
Due to the majority of the catchments being classified as undisturbed rural catchments, the Bransby-Williams equation was generally used for determining time of concentration in the Rational Method calculation. The coefficient of discharge (C_{10} value) for each of the catchments was obtained from Table 4.05.3 (b), QUDM 2007 based on various soil permeability characteristics and land usage throughout the catchments.

The comparison results for the 100 year ARI storm events at key locations for each of the catchments are presented in Table 5.16 to Table 5.22 and the locations of the comparison points are illustrated in Figure 5.16. The Rational Method results are generally within 10% of modelled flows, demonstrating a good correlation. Some of the rational calculations differ by slightly more than 10% from the modelled results. An examination of these catchments suggests that in these cases the difference may be due to the partial area effect as described in section 4.03.2 of QUDM (2007). The catchments with larger differences exhibit either elongated shapes (as for EPP606S_OUT2 and SEP_18) or a significant area of higher roughness in the upper reaches of the catchment (as for EPP606N_OUT01). This leads to a shorter time of concentration in these catchments than what would be expected by desktop analysis, and contributes to possible underestimation of peak flows by the Rational Method.

Overall, the Rational Method calculations closely aligned with the model results for areas where streamflow data or gauged data is not available. The analysis of predicted peak flows supported the

results from the regional flood frequency analysis undertaken for these models, and as such the models are considered to predict peak flows within the respective catchments to an acceptable level of accuracy.

The XP-RAFTS models developed for this study are therefore considered suitable for the creation of local and total inflow hydrographs to the nine TUFLOW hydraulic models.



LEGEND

- Town
- Validation point
- Reporting point
- Existing railway
- State border
- Road
- Major watercourse
- Preferred pipeline alignment
- Catchment boundary
- Walloons gas fields development areas

0 25 50km

SCALE - 1:850,000 (at A3)
Latitude / Longitude
Geocentric Datum of Australia 1994

N W E S

Rev	Date	Revision Description	ORIG	CHK	ENG	APPD
0	15/12/2009	Issued for use	JM	DH		

WorleyParsons
resources & energy

AUSTRALIA PACIFIC LNG PTY LIMITED

AUSTRALIA PACIFIC LNG PROJECT EIS
Figure 5.16 XP-RAFTS/Rational Method Flow Comparison Locations

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-2210 | Rev: 0

Compiled by BRISBANE INFRASTRUCTURE GIS SECTION

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Table 5.16 XP-RAFTS and Rational Method results - 100 Year ARI Event for EPP606N catchment

ID	XP-RAFTS Node	Area (Km2)	C10 value	Rational Method (m3/s)	XP-RAFTS (m3/s)	Percentage Difference (%)
1	EPP606N_OUT01	284	0.40	453	506	11.7
2	EPP606N_OUT02	241	0.40	415	393	-5.2
3	EPP606N_OUT03	431	0.32	527	484	-8.1

Table 5.17 XP-RAFTS and Rational Method results - 100 Year ARI Event for EPP606S catchment

ID	XP-RAFTS Node	Area (Km2)	C10 value	Rational Method (m3/s)	XP-RAFTS (m3/s)	Percentage Difference (%)
1	EPP606S_OUT1	46	0.36	124	112	-9.9
2	EPP606S_OUT2	537	0.32	451	508	12.5

Table 5.18 XP-RAFTS and Rational Method results - 100 Year ARI Event for EPP663 catchment

ID	XP-RAFTS Node	Area (Km2)	C10 value	Rational Method (m3/s)	XP-RAFTS (m3/s)	Percentage Difference (%)
1	EPP663_OUT1	687	0.32	589	627	6.4
2	EPP663_OUT2	811	0.32	584	598	2.5

Table 5.19 XP-RAFTS and Rational Method results - 100 Year ARI Event for EPP972 catchment

ID	XP-RAFTS Node	Area (Km2)	C10 value	Rational Method (m3/s)	XP-RAFTS (m3/s)	Percentage Difference (%)
1	EPP972_OUT1	779	0.32	761	770	1.2

Table 5.20 XP-RAFTS and Rational Method results - 100 Year ARI Event for PL209 catchment

ID	XP-RAFTS Node	Area (Km2)	C10 value	Rational Method (m3/s)	XP-RAFTS (m3/s)	Percentage Difference (%)
1	PL209_OUT01	490	0.40	700	642	-8.3

Table 5.21 XP-RAFTS and Rational Method results - 100 Year ARI Event for SEP692 catchment

ID	XP-RAFTS Node	Area (Km2)	C10 value	Rational Method (m3/s)	XP-RAFTS (m3/s)	Percentage Difference (%)
1	SEP692_10	105	0.40	246	224	-10%
2	SEP692_18	202	0.40	322	363	12%

Table 5.22 XP-RAFTS and Rational Method results - 100 Year ARI Event for EPP973 catchment

ID	XP-RAFTS Node	Area (Km2)	C10 value	Rational Method (m3/s)	XP-RAFTS (m3/s)	Percentage Difference (%)
1	EPP973_70	218	0.32	219	228	4%
2	EPP973_77	650	0.32	612	634	4%

5.4.2 Hydraulic modelling

Flood frequency analysis

As only a limited number of river height stations are available in the study areas, there was limited scope for definitive validation of all model results to historical flood levels. The river height stations that are available in the study areas are summarised in Table 5.23.

Table 5.23 River gauging stations within the study areas

Model ID (Refer Figure 4.1)	Owner	Details	Location
EPP692	Department of Environment and Resource Management	Station ID: 422202B Commissioned: 1949	Dogwood Creek – Gil Weir
	Bureau of Meteorology	Flood Warning River Height Station: 042049 Commissioned: 1974 (Earlier commissioning from 1945 – 1950 as a gauging station)	Dogwood Creek – Miles
PL226	Department of Environment and Resource Management	Station ID: 422308C Commissioned: 1955	Condamine River – Chinchilla Weir
	Bureau of Meteorology	Flood Warning River Height Station: 042048 Commissioned: 1922	Condamine River – Crawford Bridge, Condamine

To validate flood levels predicted by the hydraulic model, flood frequency analyses have been carried out using the annual peak gauge height data obtained from the above river height stations. Various frequency distribution methods including Log Pearson 3 (LP3), Log Normal and Generalised Pareto distributions have been used to achieve a best fit to the recorded data for each of the gauged stations. This best fit flood level distribution was then compared to the modelled 10, 20, 100 and 500 year ARI water surface levels at the corresponding locations to validate the hydraulic model. A comparison of the flood levels for Tenement EPP692 (Dogwood Creek) and Tenement PL226 (Condamine River) are summarised below.

Tenement EPP692 (Dogwood Creek)

Validation of model EPP692 (Dogwood Creek) was undertaken by comparison of model results to gauging station data for the Gil Weir gauging station (422202B) operated by DERM, located on Dogwood Creek at Gil Weir, approximately 6km south of the township of Miles. Table 5.24 presents comparison results at Gil Weir. There was insufficient data relating to the station at Miles to allow a flood frequency analysis to be undertaken.

Table 5.24 Model validation results – Model EPP692 (Dogwood Creek) at Gil Weir

Dogwood Creek – Gil Weir Station ID: 422202B				
Rainfall event	Flood Frequency Analysis (Generalised Pareto Method)			TUFLOW predicted peak flood level (m AHD)
	Peak flood level (Average) (m AHD)	Peak flood level (Lower bound) (m AHD)	Peak flood level (Upper bound) (m AHD)	
10 year ARI	295.86	295.04	296.67	294.60
20 year ARI	296.65	295.93	267.57	295.22
100 year ARI	297.54	296.89	298.93	296.09
500 year ARI	297.86	297.19	299.63	296.34

Validation of model results for the EPP692 (Dogwood Creek) hydraulic model was undertaken by comparison of model results to gauging station data for Gil Weir, approximately 6km south of the township of Miles. Data for the Gil Weir gauging station is somewhat limited and includes 206 gauged events covering a period of 45 years. Comparison was also undertaken to the gauging/flood warning station at Miles, also on Dogwood Creek, where limited data was available for peak flood level comparison purposes. The amount of data for the Miles gauging station was not considered sufficient to undertake an accurate flood frequency analysis (5 years of recorded gauge data only).

Review of flooding histories within the Dogwood Creek catchment reveals the largest recorded flooding event occurred in 1956, where isohyets show rainfalls of up to 13 inches fell in the upper Dogwood Creek catchment in the 72hours to 9am on the 22nd January. It is noted in the report 'Record Floods in South East Queensland January 1956' (BOM 1956) regarding this flood event that some rainfall records were incomplete due to many gauges overtopping or being washed away. Nonetheless, according to IFD data for the catchment, this represents a design rainfall event greater than a 1 in 500 year ARI event (approximately 10 inches over a 72 hour period). These record rainfalls resulted in record peak flood levels of approximately 300.76mAHD and 297.18mAHD at the Miles and Gil Weir gauges respectively.

Review of peak flood levels for the 500 year ARI modelled event at Miles shows results to be below the peak event recorded in 1956. The TUFLOW model predicted flood levels of approximately 299.64mAHD at the Miles gauging site, some 1.12m lower that the peak recorded level 300.76mAHD at this location. Given rainfall records indicate the 1956 event was greater than a 500 year ARI design rainfall event, these results would appear acceptable. Results from rainfall intensity sensitivity analysis (30% increase in rainfall intensity for 100 year ARI) shows that the intensities adopted for this scenario are near to those recorded in the 1956 event (assuming a catchment averaged 12 inches of rainfall over the Dogwood Creek catchment over a 72 hour period). The modelled peak flood level at Miles for this sensitivity analysis was predicted to be 300.41mAHD, whilst the peak level at Gil Weir was shown to be 296.88mAHD. These are comparable to the recorded 1956 event flood levels of 300.76mAHD and 297.18mAHD at the respective locations.

Model results included predicted peak flood levels for all design rainfall events that were lower than those predicted by the flood frequency analysis using the Generalised Pareto method at Gil Weir.

Average differences of approximately 1.3m between predicted peak flood levels and the average peak flood level predicted by the Generalised Pareto Method for all ARI events were recorded. Peak flows within the TUFLOW model were shown to be comparable to those from the XP-RAFTS hydrologic model.

Skewness of data records at the Gil Weir gauging station may result in the over-prediction of flood heights for the larger rainfall events investigated as part of this study. The record 1956 flood event peak flood level, which rainfall records indicate to be greater than a 500 year ARI event for the 72 hour storm duration (the critical duration for the catchment), is predicted by the Generalised Pareto Method to be less than a 50 year ARI event according to the flood frequency analysis. This would suggest the recorded dataset for this station may be skewed, making statistical interpolation of higher order rainfall event flood heights difficult.

Given the aforementioned considerations, it is concluded that the small data sample available for the Gil Weir gauging station and resultant analysis by the Generalised Pareto Method would appear to over predict peak flood levels at this location. Comparison of model results for the rainfall intensity sensitivity analysis (30% increase in rainfall intensity) for the 100 year ARI event which were shown to be similar rainfall intensities to those recorded in the 1956 72hr record flood event for the catchment were shown to be favourable, with only minor differences at the two gauging stations (approximately 300mm difference at both locations). These differences are considered acceptable given the changes to land uses within the catchment, floodplain roughness differences as a result of land clearing, and other topographic and structural changes (e.g. raising of Gil Weir in 1994) that have occurred in the catchment and watercourse since this event. Baseline topographic data accuracies (±1m for measured points[3] and ±1.25m for derived points[4] on clear ground) may also be impacting upon modelled peak flood levels. These values represent standard error (68% confidence level or 1 sigma), in meters.

As a result, it is considered that the TUFLOW model constructed for the EPP692 tenement (Dogwood Creek) is predicting flood behaviour in an acceptable fashion and has been adopted for use in this study.

Tenement PL226 (Condamine River at Chinchilla Weir and Condamine gauge stations)

Validation of model PL226 (Condamine River) was undertaken by comparing model results to data from two river gauging stations:

- Chinchilla Weir Gauge Stations (422308 B&C) operated by Department of Environment and Resource Management located on Condamine River at Chinchilla Weir. Frequency distribution method used is Log Pearson III (LP3); and
- Condamine Flood River Height Warning Station (042048) operated by Bureau of Meteorology located on Condamine River at Crawford Bridge at Condamine. Frequency distribution method used is Generalised Pareto.

Method of frequency distribution chosen in the analysis is based on the distribution that produces the best fit to the recorded data. Table 5.25 and Table 5.26 present the flood level comparison results at Chinchilla Weir gauge station and Condamine River Height station at Crawford Bridge.

It can be seen that the modelled results compared favourably with the flood levels estimated from the flood frequency analysis for all studied events across the two gauging stations. The largest difference

[3] "Measured Points" are those observed directly.

[4] "Derived Points" are those interpolated from a digital elevation model (DEM).

difference was found occurred in the 20 year ARI flood level at Condamine gauge. Model results predicted peak flood levels for the 20 year ARI event to be approximately 400mm lower than the flood level estimated by the flood frequency analysis at Condamine gauge. It is noted that about 20 years of the annual flood level records are missing within the 80 year record period of the Condamine gauging station and the missing data may result in an over-prediction of flood height for the 20 year ARI event.

To ascertain if the TUFLOW model constructed is able to predict acceptable flood behaviour for tenement PL226 for all design flood events, a comparison of the 20 year ARI XP-RAFTS and TUFLOW flows at the tailwater boundary has been conducted. The flow from the TUFLOW model at this location is approximately 10% less than the flow calculated by the XP-RAFTS hydrologic model. This difference is considered acceptable, as the TUFLOW model accounts for channel storage and flow restriction caused by the structures along the waterways; whilst XP-RAFTS routes the flows right through to the bottom of the catchment resulting higher flows from XP-RAFTS than TUFLOW. The lower flows predicted from TUFLOW is consistent across all ARI flood events assessed in this analysis.

Therefore, based on these verifications, it is considered that the TUFLOW model constructed for the PL226 tenement (Condamine River) is adequately predicting flood behaviour and has been adopted for use in this study.

Table 5.25 Model validation results – Chinchilla Weir

Condamine River – Chinchilla Weir	Station ID: 422308C	
Rainfall event	**Flood frequency analysis (Log Pearson III distribution) expected peak flood level (m AHD)**	**TUFLOW predicted peak flood level (m AHD)**
10 year ARI	298.51	298.59
20 year ARI	298.96	298.80
100 year ARI	299.69	299.94
500 year ARI	300.21	300.35

Table 5.26 Model validation results – Condamine Crawford bridge

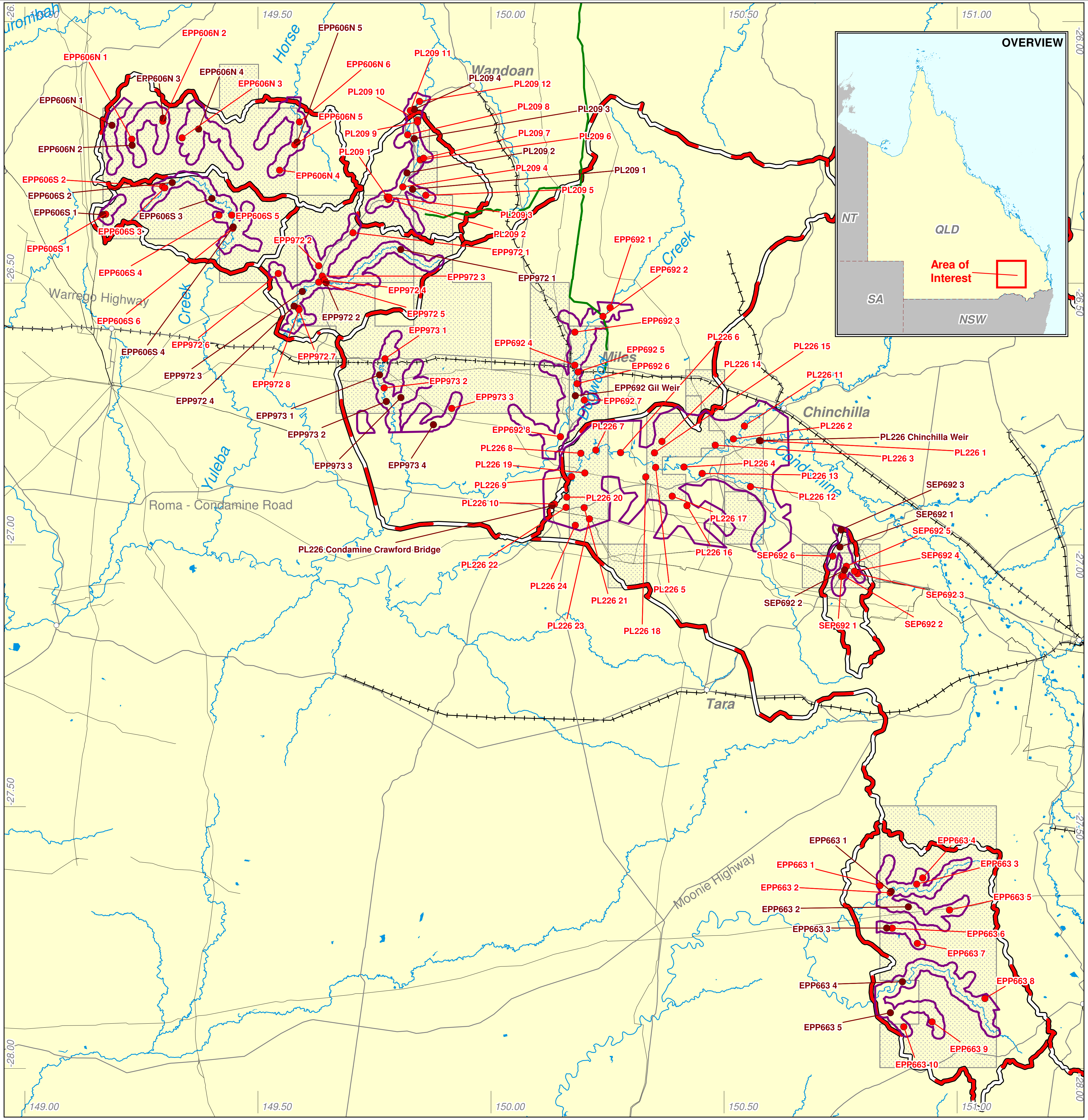
	Condamine River – Condamine	Station ID: 042048
Rainfall event	**Flood frequency analysis (Log Pearson III distribution) expected peak flood level (m AHD)**	**TUFLOW predicted peak flood level (m AHD)**
10 year ARI	280.08	280.05
20 year ARI	281.34	280.96
100 year ARI	283.12	283.24
500 year ARI	283.98	283.66

Manning's equation validation

Given the limited opportunities for model validation through comparison with historical flood levels, alternative methods were required for those models with no river height gauging stations or previous flood study information available.

As a result, it was determined that model results at discrete cross sections selected within each model would be analysed by utilising the Mannings equation for open channel flow. Representative cross sections were extracted from the relevant model topography, and applicable parameters such as surface slope, and Mannings 'n' roughness were applied. Peak water surface levels were extracted at each cross section location from the TUFLOW model results. This information allowed the calculation of a peak flow for each representative cross section by way of the Mannings equation. This predicted peak flow value was then compared to peak flows extracted from the TUFLOW model results to ensure the flood levels predicted by each model were representative of the peak flow conveyed through each section. Validation locations in each model are shown in Appendix C. Table 5.27 summarises the model validation results.

The results of Models EPP692 and PL226 have already been compared to river height gauge information at the available gauging stations shown in Table 5.23. As models EPP692 and PL226 were subject to expansive flooding with high flow rates and relatively flat water surface gradients, use of the Mannings equation to validate results was not considered appropriate for these models, and hence were not included in this analysis.



LEGEND

- Town
- Validation point
- Reporting point
- Existing railway
- State border
- Road
- Major watercourse
- Preferred pipeline alignment
- Catchment boundary
- Hydraulic model boundary
- Walloons gas fields development areas

0 25 50km

SCALE - 1:850,000 (at A3)
Latitude / Longitude
Geocentric Datum of Australia 1994

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Rev	Date	Revision Description	ORIG	CHK	ENG	APPD
0	24/11/2009	Issued for use	JM	DH		

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Figure 5.17 Hydraulic Model Validation & Flood Level Comparison Point Locations

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-2211 | Rev: 0

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Table 5.27 Model validation results

Model ID	Validation location	Mannings equation predicted peak flow (m3/s)	TUFLOW 100 year ARI recorded peak flow (m3/s)	Difference (%)
EPP606N (Horse Creek and surrounding tributaries)	1	185	167	-10
	2	104	113	9
	3	103	109	6
	4	170	184	8
	5	264	242	-9
EPP606S (Yuleba Creek)	1	91	88	-3
	2	140	149	6
	3	200	219	9
	4	264	280	6
PL209 (Woleebee Creek)	1	112	120	7
	2	411	377	-8
	3	529	581	10
	4	715	707	-1
EPP972 (Tchanning Creek)	1	88	85	-4
	2	292	326	11
	3	604	625	3
	4	737	698	-5
EPP973 (Dulacca Creek)	1	178	187	5
	2	218	234	7
	3	129	134	4
	4	88	90	3
SEP692 (Kogan Creek)	1	29	32	11
	2	278	293	5
	3	419	387	-7

Model ID	Validation location	Mannings equation predicted peak flow (m3/s)	TUFLOW 100 year ARI recorded peak flow (m3/s)	Difference (%)
EPP663 (Weir River and Western Creek)	1	188	205	9
	2	106	113	7
	3	123	116	-5
	4	297	332	12
	5	233	230	-1

Generally the flood levels and corresponding peak flows obtained in the seven TUFLOW models with no stream gauging data available compare favourably with results obtained from use of the Mannings equation.

Notable differences (+/- 10%) were typically only evident in areas with large flow rates and where water surface gradients were extremely flat. These characteristics create heightened sensitivity in the Mannings equation predictions, where slight changes in adopted water surface slope or Mannings values can skew results considerably. As such, Mannings equation predictions for areas of high flow and flat water surface slope are considered to be indicative only given the high sensitivity to input parameters.

The Mannings equation validation analysis results demonstrate that the remaining models adequately estimate flooding behaviour and that it is appropriate to adopt the model results for this investigation.

5.5 Design event modelling

5.5.1 Hydrologic modelling

Design rainfall event modelling was undertaken for the 10, 20, 100 and 500 year ARI events. As detailed in Section 3 this was based on various IFD datasets corresponding to the catchment location and based on land uses delineated from orthophoto imagery.

Storms with rainfall durations from 60 minutes to 72 hours were simulated for the four ARI events for each of the catchments to determine critical storm durations at key locations. The critical duration represents the storm duration that results in the largest peak flow from the modelled catchment, and hence typically results in the peak flood level. This can vary throughout the catchment based on location and topographic The modelling results for each of the catchments are summarised below.

Model EPP606N (Horse Creek & adjacent systems)

The EPP606N study area lies in the extreme upper reaches of the Dawson River basin. An analysis of peak flow results at locations near proposed facilities showed minimal variation between critical and non-critical duration flows. For this reason, the 540 minute (9 hour) storm duration was adopted as the predominant critical duration within the hydraulic modelling area at key locations. Table 5.28 summarises the peak flows and durations at various locations throughout the model.

Table 5.28 Peak flow summary for EPP606N catchment

XP-RAFTS node*	Q10		Q20		Q100		Q500	
	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical Duration
EPP606_09	70	360 min	93	360 min	152	360 min	211	360 min
EPP606_12	7	180 min	9	180 min	15	180 min	21	180 min
EPP606_60	286	360 min	380	360 min	627	360 min	915	360 min
EPP606_19	24	360 min	31	360 min	50	360 min	72	360 min
EPP606_21	31	540 min	41	540 min	66	360 min	94	360 min
EPP606_61	222	360 min	296	720 min	490	540 min	723	540 min
EPP606_50	123	1080 min	165	1080 min	274	540 min	407	540 min
EPP606_57	250	1080 min	341	1080 min	582	1080 min	870	1080 min

* XP_RAFTS node location shown in Appendix B

Model EPP606S (Yuleba Creek)

The critical duration for the EPP606S model was found to be 1080 minutes (18 hours). Flow from sub-catchments with a peak duration other than 1080minutes were compared to the 1080 minute event peak flow. Typically these flows were shown to be less than 5% different than the 1080 minute event. As such the 1080 minute (18 hour) event was adopted to provide inflows into the hydraulic model for this tenement. Table 5.29 presents predicted peak flows and corresponding critical durations at various locations within the EPP606S model.

Table 5.29 Peak flow summary for EPP606S catchment

XP-RAFTS node*	Q10		Q20		Q100		Q500	
	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration
EPP606_30	18	1080 min	24	1080 min	40	1080 min	59	1080 min
EPP606_62	60	1080 min	79	1080 min	127	1080 min	188	4320 min
EPP606_38	48	1080 min	67	1440 min	116	1440 min	180	1440 min

XP-RAFTS node*	Q10		Q20		Q100		Q500	
	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration
EPP606_32	29	60 min	35	60 min	53	1440 min	79	1440 min
EPP606_36	113	1080 min	154	1440 min	271	1440 min	430	4320 min
EPP606_40	188	1440 min	268	2160 min	470	2160 min	742	2160 min
EPP606_65	225	1080 min	321	2160 min	573	2160 min	904	2160 min

* XP_RAFTS node location shown in Appendix B

Model PL209 (Woleebee Creek)

The PL209 model has a critical duration of 1080 minutes (18 hours). For the 100 year ARI event only three sub-catchments have a peak flow at durations other than 1080 minutes and these flows differ from the 1080 minutes flow by less than 1 m^3/s each. Table 5.30 presents predicted peak flows and corresponding critical durations at various locations within the model.

Table 5.30 Peak flow summary for PL209 catchment

XP-RAFTS node*	Q10		Q20		Q100		Q500	
	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration
EPP972_12	47	1080 min	60	1080 min	97	540 min	142	540 min
PL209_02	55	1080 min	73	1080 min	122	1080 min	175	1080 min
EPP972_15	173	1080 min	227	1080 min	366	1080 min	530	1080 min
PL209_03	189	1080 min	249	1080 min	400	1080 min	582	720 min
PL209_04	56	60 min	75	1440 min	127	1440 min	194	1080 min
PL209_05	245	1080 min	326	1080 min	537	1080 min	786	1080 min
PL209_07	290	1080 min	390	1080 min	642	1080 min	930	1080 min

* XP_RAFTS node location shown in Appendix B

Model EPP972 (Tchanning Creek)

The critical duration for the EPP972 model was found to be 4320 minutes (72 hours). An additional duration of 5760 minute (96 hours) was simulated for the EPP972 model to ensure that the critical

duration was not greater than 4320 minutes. The peak flow for all sub-catchments is within 4 m^3/s of the 4320 minute event flow for the 100 year ARI event. As such, this duration has been adopted for inflows into the hydraulic model. Table 5.31 presents predicted peak flows and corresponding critical durations at various locations in the hydrologic model.

Table 5.31 Peak flow summary for EPP972 catchment

XP-RAFTS node*	Q10		Q20		Q100		Q500	
	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration
EPP972_02	47	4320 min	68	4320 min	113	2160 min	178	4320 min
EPP972_04	42	4320 min	58	4320 min	92	1080 min	145	4320 min
EPP972_05	61	4320 min	84	4320 min	135	4320 min	211	4320 min
EPP973_01	127	2160 min	183	4320 min	313	2160 min	498	4320 min
EPP973_02	255	4320 min	365	4320 min	611	2160 min	973	4320 min
EPP973_03	287	4320 min	412	4320 min	692	4320 min	1118	4320 min
EPP973_07	316	4320 min	454	4320 min	770	4320 min	1244	4320 min

* XP_RAFTS node location shown in Appendix B

Model EPP973 (Dulacca Creek)

Critical durations of the EPP973 model vary across sub-catchments. For the purposes of undertaking a single TUFLOW hydraulic model simulation for each design event, analysis of the peak flow distribution patterns across the sub catchments was undertaken. It was determined that the 2160 minute (36 hour) event was the most appropriate single critical duration to best represent peak flooding at key locations within the respective hydraulic model. Peak flows from sub-catchments with a critical duration other than 2160 minutes are not significantly larger than the 2160 minute flow (average less than 3% up to 100yr ARI and 6% for the 500yr ARI). Table 5.32 presents predicted peak flows and corresponding critical durations at various locations within EPP973.

Table 5.32 Peak flow summary for EPP973 catchment

XP-RAFTS Node*	Q10		Q20		Q100		Q500	
	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration
EPP973_69	14	2160 min	20	4320 min	33	4320 min	54	4320 min
EPP973_70	92	2160 min	130	2160 min	228	4320 min	375	4320 min

XP-RAFTS Node*	Q10		Q20		Q100		Q500	
	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration
EPP973_75	176	2160 min	250	4320 min	435	4320 min	713	4320 min
EPP973_76	238	2160 min	339	4320 min	597	4320 min	990	4320 min

* XP_RAFTS node location shown in Appendix B

Model EPP692 (Dogwood Creek)

Similar to EPP973, critical durations of EPP692 model vary across sub-catchments. Selecting a single critical duration has been carried out for this catchment to provide inflows into the hydraulic model for this tenement. By analysing the peak flow distribution patterns across the sub catchments, it is found the 2160 minute (36 hour) critical duration is an appropriate single critical duration to be adopted for EPP692. Peak flows from sub-catchments with a critical duration other than 2160 minutes are not significantly larger than the 2160 minute flow (average less than 3% for all ARI events). Table 5.33 presents predicted peak flows and corresponding critical duration at various locations within the model.

Table 5.33 Peak flow summary for EPP692 catchment

XP-RAFTS Node*	Q10		Q20		Q100		Q500	
	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration
EPP692_16	213	2160 min	309	2160 min	578	4320 min	695	4320 min
EPP692_21	90	2160 min	133	2160 min	245	2160 min	290	4320 min
EPP692_29	149	2160 min	215	2160 min	390	4320 min	471	4320 min
GIL_out	454	2160 min	649	2160 min	1184	2880 min	1413	2160 min
EPP692_65	558	2160 min	790	2160 min	1424	2880 min	1719	4320 min

* XP_RAFTS node location shown in Appendix B

Model PL226 (Condamine River)

Regional riverine flow on Condamine River and the floodplain throughout tenement PL226 are dominated by the flows from the upper Condamine River discharging through Chinchilla Weir (RAFTS model node PL226_NE_15) into the study site. Chinchilla Weir has a total catchment area of some 25,000km^2. Critical durations for the regional riverine flow scenario varied from 720 minutes (12 hours) to 4320 minutes (72 hours) for different ARI events. Table 5.34 presents predicted peak regional flows and corresponding critical durations at various locations within the Condamine River model.

Table 5.34 Peak flow summary for PL226 catchment

XP-RAFTS Node*	Q10		Q20		Q100		Q500	
	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration
NE_15	1424	2160 min	1664	2160 min	3304	2160 min	3951	4320 min
Sub_36	1966	2160 min	2170	720 min	4396	2160 min	5269	4320 min
Sub_25	162	1080 min	229	1440 min	397	1080 min	459	1080 min
Sub_26	77	1080 min	106	1080 min	179	1080 min	207	1080 min
Sub_31	58	4320 min	80	4320 min	127	1080 min	147	4320 min
Sub_43	2116	1080 min	2337	720 min	4716	2160 min	5664	4320 min

* XP_RAFTS node location shown in Appendix B

As discussed previously in Section 5.2.3, the rainfall applied to the PL226 hydrologic model for regional flood has been reduced by an Aerial Reduction Factor (ARF) according to the overall catchment area. However, the local tributaries traversing the tenement are creeks with smaller catchment sizes. Applying the same ARF reduced catchment rainfall to the local tributary catchments is likely to underestimate the local tributary flows. Moreover, it is unlikely that coincident rainfall would occur over the local tributaries and the entire catchment of Condamine River. As such, an ARF has not been applied to the catchment rainfall for predicting local tributary flows within tenement PL226. Critical durations for local tributaries within PL226 are 2160 minutes (36 hours) for Charleys and Wambo Creeks, and 1080 minutes (18 hours) for other tributaries across tenement PL226.

To assess the impacts of regional riverine flows on the Condamine River flood plain and local flows on the tributaries of tenement PL226, the following rainfall durations presented in Table 5.35 have been adopted for the hydrologic model to provide inflows into the hydraulic model.

Table 5.35 Adopted critical durations for hydrologic model PL226

ARI event	Critical duration		
	Regional flood	Local flood (Charleys and Wambo Creeks)	Local flood (Other Tributaries)
500 yr ARI	4320 min	2160 min	1080 min
100 yr ARI	2160 min	2160 min	1080 min
20 yr ARI	2160 min	2160 min	1080 min
10 yr ARI	2160 min	2160 min	1080 min

5.5.2 Model SEP692 (Kogan Creek)

The SEP692 model has a critical duration of 1080min (18 hour). Only a few sub-catchments had a critical duration other than 1080min and the peak flows for these sub-catchments did not differ significantly (average less than 3%) from the flows estimated for the 1080min storm duration. As such the 1080min (18 hour) event was adopted to provide inflows into the hydraulic model for this tenement. Table 5.36 presents predicted peak flows and corresponding critical durations at different locations within the SEP692 model.

Table 5.36 Peak Flow Summary for SEP692 Catchment

XP-RAFTS Node*	Q10		Q20		Q100		Q500	
	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration
SEP692_7	72	4320 min	129	1080 min	159	1080 min	242	4320 min
SEP692_11	108	1080 min	202	1080 min	251	1080 min	374	1080 min
SEP692_15	144	1080 min	272	1080 min	336	1080 min	509	1080 min
SEP692_18	153	4320 min	291	1080 min	362	1080 min	550	1080 min

* XP_RAFTS node location shown in Appendix B

5.5.3 Model EPP663 (Weir River and Western Creek)

The EPP663 model has a critical duration of 1440min (24 hour). Only a few sub-catchments had a critical duration other than 1440min and the peak flows for these sub-catchments did not differ significantly (<5%) from the flows estimated for the 1440min event. Table 5.37 presents predicted peak regional flows and corresponding critical durations at different locations within the EPP663 model.

Table 5.37 Peak flow summary for EPP663 catchment

XP-RAFTS Node*	Q10		Q20		Q100		Q500	
	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration
EPP663_08	73	2160 min	105	1440 min	192	1440 min	313	1440 min
EPP663_09	17	2160 min	24	2160 min	47	1440 min	78	1440 min
EPP663_15	33	1440 min	47	1440 min	84	1440 min	132	1080 min
EPP663_30	238	2160 min	343	1440 min	627	1440 min	1010	1440 min

XP-RAFTS Node*	Q10		Q20		Q100		Q500	
	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration	Peak flow (m3/s)	Critical duration
EPP663_20	69	2160 min	99	2160 min	184	1440 min	295	1440 min
EPP663_26	63	2160 min	90	2160 min	168	1440 min	275	1440 min
EPP663_28	31	2160 min	43	1440 min	78	1440 min	124	1080 min
EPP663_31	221	2160 min	320	2160 min	598	1440 min	977	1440 min

* XP_RAFTS node location shown in Appendix B

5.5.4 Hydraulic modelling – existing case

Hydraulic modelling of all gas field areas was undertaken using nine representative 1D/2D TUFLOW models with the parameters as outlined in Section 5.3.

Modelling was performed for the 10, 20, 100 and 500 year ARI design rainfall events, with either local catchment or total catchment hydrographs developed as part of the hydrologic modelling works applied to the hydraulic model where appropriate.

Where infrastructure such as gas processing facilities are proposed adjacent to a regional watercourse, flood levels have been tabulated to provide indicative flood levels for these locations. For a discussion of flood behaviour within the respective waterway systems, refer to the sections below. Refer to Appendix C for locations of the flood level reporting points in each of the following hydraulic models.

As water levels are calculated at cell centre and cell sides for all cells within the 2D model, it is not practical to tabulate flood levels for all computation points throughout the model. Flood levels for the 2D scheme are commonly best presented using flood surface and extent maps created in a GIS environment. Flood inundation plans have been prepared from model results based upon the photogrammetry topographic data provided by AAMHatch for the purposes of this study and are presented in Appendix E. As such, the flood levels provided in the following tables and in any digital data provided are inherently reliant upon the accuracy of the baseline topographic data provided for use as part of this investigation.

Model EPP606N (Horse Creek & adjacent systems)

Horse Creek and the adjacent northern draining minor watercourses of tenement EPP606 all are within the Dawson River sub-basin. These systems flow predominately through rural areas, with number of minor road crossings of the various watercourses within the modelling area. The modelling area is located within the upper reaches of the respective catchments. Therefore flood extents are typically well defined due to the nature of the surrounding topography.

Most roads throughout the modelling area are shown to be inundated for all rainfall events modelled. These include Yuleba – Taroom Road at Horse Creek (Main and West branch), Four Mile Gully crossings, the Clifford Road crossing of Kangaroo Creek, the Pine Hills Road and Glenarden Road crossings of Barton Creek, and the Glenarden Road crossings of Barton and Sugarloaf Creeks.

These crossings are typically of a natural or causeway design, and likely to be subject to frequent inundation.

Results from the hydraulic modelling indicate most proposed infrastructure locations would be well outside the existing case 100 year ARI flood extent. However, the proposed location of the water treatment facility 'WTF_MEL_01' would intersect the existing case 100 year ARI flood extent of one of the upper tributaries of Kangaroo Creek. The proposed location of infrastructure WTS_COM_04 would also intersect the existing case 100 year ARI flood extent in one of the minor tributaries of Horse Creek (Main Branch). Table 5.38 summarises predicted peak flood levels at areas adjacent to proposed major infrastructure locations within the northern waterways of Tenement EPP606.

Table 5.38 Existing case peak flood levels – model EPP606N

	Existing case peak flood levels (m AHD)			
Reporting point location*	**10 year ARI (m AHD)**	**20 year ARI (m AHD)**	**100 year ARI (m AHD)**	**500 year ARI (m AHD)**
1	304.19	304.28	304.44	304.60
2	293.88	293.92	294.00	294.08
3	326.99	327.00	327.02	327.04
4	315.75	315.77	315.82	315.86
5	287.98	288.22	288.60	288.98
6	272.09	272.30	272.74	273.13

* Reporting locations shown in Appendix C

Model EPP606S (Yuleba Creek)

Yuleba Creek is the other major watercourse within the EPP606 tenement area, and lies within the Balonne River sub-basin. This watercourse initially flows east before turning in a southerly direction to the tenement boundary, where it continues on in a southerly direction to eventually reach the Balonne River. Yuleba Creek flows through predominately rural areas, and is crossed by typically only minor/local roads within the study area.

Cattle Creek Road at its northern most extent crosses and runs parallel to Yuleba Creek. These crossings are predicted to be inundated for all rainfall events modelled. Additionally, areas of Cattle Creek Road which run parallel to the watercourse are shown to be inundated during flow breakouts from the main watercourse for all rainfall events. Within the lower extent of the modelling area, Yuleba – Taroom Road and Seaside Roads are predicted to be inundated during all rainfall events. Yuleba – Taroom Road is inundated by flows from Sandy and Wirginbah Creeks, whilst Seaside Road is inundated at its crossing of the Yuleba Creek main channel and floodplain areas.

Most proposed infrastructure locations would be located well outside the 100 year ARI flood extent. However a small portion of the corner of infrastructure site WTS_PHS_07 would be located within the 100 year ARI flood extent of an adjacent tributary. The conceptual location of WTF_RCK_01a would be situated across the flow path of Ten Mile Creek in its proposed location. It is estimated that during the 100 year ARI event approximately 28m^3/s flows down this section of Ten Mile Creek, with velocities typically in the order of 1.5m/s and depths ranging up to approximately 1.5m. Table 5.39

summarises predicted peak flood levels at areas adjacent to proposed major infrastructure within the Yuleba Creek catchment areas of Tenement EPP606.

Table 5.39 Existing case peak flood levels – model EPP606S

	Existing case peak flood levels (m AHD)			
Reporting point location*	10 year ARI (m AHD)	20 year ARI (m AHD)	100 year ARI (m AHD)	500 year ARI (m AHD)
1	372.19	372.27	372.43	372.61
2	367.52	367.72	368.12	368.43
3	367.81	367.92	368.19	368.45
4	349.39	349.49	349.68	349.83
5	334.63	334.91	335.47	335.99
6	330.96	331.37	331.97	332.38

* Reporting locations shown in Appendix C

Model PL209 (Woleebee Creek)

Woleebee Creek is a tributary of Juandah Creek, which is situated in the Dawson River sub-basin. Woleebee Creek flows in a northerly direction through predominantly cleared rural land. The flood extent is well defined in the upper reaches of the catchment and broadens out into an expansive flood plain at the outlet of the model.

The Jackson Wandoan Road runs adjacent to the Woleebee Creek main channel through the study area with several crossings of the creek and associated flood plain. Large stretches of this road are predicted to be inundated for all rainfall events modelled, particularly in the northern parts of the study area where the road alignment is located in the flood plain. In the upper reaches of the catchment, Jackson Wandoan Road crosses well defined channels where inundated sections are considerably shorter in length.

Results from the hydraulic model indicate that all proposed infrastructure sites are outside of the 100 year ARI flood extent. It is noted that the proposed location of WTS_RAM_01 would be within 20m of the 100 year ARI flood extent. Due to the local topography and well defined channel, the edge of the proposed site is outside the flood extent for all storm events modelled.

Table 5.40 summarises predicted peak flood levels at areas adjacent to proposed major infrastructure within the Woleebee Creek catchment.

Table 5.40 Existing case peak flood levels – model PL209

	Existing case peak flood levels (m AHD)			
Reporting point location*	10 year ARI (m AHD)	20 year ARI (m AHD)	100 year ARI (m AHD)	500 year ARI (m AHD)
1	308.87	309.03	309.27	309.43
2	311.68	311.70	311.98	312.14
3	307.73	307.84	308.08	308.31

	Existing case peak flood levels (m AHD)			
Reporting point location*	10 year ARI (m AHD)	20 year ARI (m AHD)	100 year ARI (m AHD)	500 year ARI (m AHD)
4	293.16	293.67	294.36	294.89
5	316.18	316.23	316.39	316.55
6	276.20	276.32	276.54	276.75
7	278.58	278.58	278.58	278.58
8	265.00	265.03	265.11	265.20
9	257.67	257.76	258.00	258.20
10	257.59	257.71	257.94	258.12
11	254.77	254.83	254.97	255.14
12	250.35	250.36	250.41	250.58

* Reporting locations shown in Appendix C

Model EPP972 (Tchanning Creek)

Tchanning Creek is a southerly flowing watercourse that forms part of the Dogwood Creek and Balonne River catchments. The study area is in the upper reaches of the Tchanning Creek catchment with a well defined flood extent. The catchment is predominantly agricultural land with some patches of native vegetation.

The Jackson Wandoan Road runs north-south through the study area along the tributary named Clark Creek. The road is inundated at the crossing of Clark Creek near the top of the catchment, at Noonga Creek near the junction with Clark Creek and at Tchanning Creek.

Most of the proposed infrastructure sites within the Tchanning Creek study area lie outside of the 100 year ARI flood extent as predicted by the hydraulic model. However, modelling results suggest the edge of proposed infrastructure GPF_HCK_01a would be within 20 meters of the 100 year ARI flood extent and would slightly intersect the 500 year ARI event flood extent.

Table 5.41 summarises predicted flood levels at areas adjacent to proposed major infrastructure within the Tchanning Creek study area.

Table 5.41 Existing case peak flood levels – model EPP972

	Existing case peak flood levels (m AHD)			
Reporting point location*	10 year ARI (m AHD)	20 year ARI (m AHD)	100 year ARI (m AHD)	500 year ARI (m AHD)
1	355.58	355.71	355.93	356.16
2	334.20	334.34	334.60	334.86
3	328.97	329.17	329.48	329.85
4	327.89	328.17	328.53	328.91

	Existing case peak flood levels (m AHD)			
Reporting point location*	**10 year ARI (m AHD)**	**20 year ARI (m AHD)**	**100 year ARI (m AHD)**	**500 year ARI (m AHD)**
5	326.69	326.97	327.44	327.97
6	336.41	336.51	336.68	336.92
7	N/A	N/A	314.78	315.38
8	313.76	314.24	314.78	315.32

*Reporting locations shown in Appendix C

Model EPP973 (Dulacca Creek)

Dulacca Creek is a major watercourse within the Balonne River sub-basin, which flows in a southerly direction through Tenement EPP973. The system flows through predominately rural areas, with the only major infrastructure crossing the creek system being the Warrego Highway and adjacent Rail Bridge at the township of Dulacca. The adjacent Range Creek and Moraby Creek minor waterway systems are also included in the study area, and similarly predominately flow through rural environments. These systems join Dulacca Creek a small distance downstream of the tenement boundary.

Modelling results for Dulacca Creek show that flooding is typically well defined and contained within the main watercourse in the upper reaches of all systems, with more expansive flooding evident in the lower reaches of Dulacca Creek, where the topography flattens and flow paths become less defined.

Road crossings of the major waterways shows most minor road crossings are inundated during all rainfall events modelled. These include Harrisons Road crossing of Byrne Creek and adjacent tributary, Dulacca South Road crossing of Bishop and Tea Creeks and Butlers Extens Road crossing of Dulacca Creek, both crossings of Maher Creek, Range Creek, and the crossing of an unnamed tributary higher in the catchment (adjacent to J D Creek). Olivers – Walkers Road also is shown to be inundated by Range Creek for all rainfall events modelled. These are typically causeway crossings, and hence subject to frequent inundation. However, the Warrego Highway which traverses the upper modelling area, is predicted to be flood free for all the rainfall events modelled as part of this investigation.

Results indicate that the proposed locations of all major infrastructure within the tenement would be well outside the predicted 100 year ARI regional flood extent. Table 5.42 summarises predicted peak flood levels at areas adjacent to proposed major infrastructure within the Dulacca, Range and Moraby Creek systems of Tenement EPP973.

Table 5.42 Existing case peak flood levels – model EPP973

	Existing case peak flood levels (m AHD)			
Reporting point location*	**10 year ARI (m AHD)**	**20 year ARI (m AHD)**	**100 year ARI (m AHD)**	**500 year ARI (m AHD)**
1	310.55	310.85	311.37	311.95
2	297.40	297.71	298.13	298.44

	Existing case peak flood levels (m AHD)			
Reporting point location*	10 year ARI (m AHD)	20 year ARI (m AHD)	100 year ARI (m AHD)	500 year ARI (m AHD)
3	308.59	308.70	308.95	309.18

* Reporting locations shown in Appendix C

Model EPP692 (Dogwood Creek)

Dogwood Creek is a major watercourse within the Balonne River sub-basin, which flows in a southerly direction through Tenement EPP692 past the town of Miles, before eventually continuing to the Balonne River. Significant flows are evident in the watercourse during large rainfall events, which lead to some large expansive areas of inundation within the floodplain.

Roads within the model area are predicted to be subject to differing degrees of inundation. Local unsealed roads are typically inundated at crossings during all ARI events modelled due to the nature of the crossings (natural crossings). Examples include Myall Park Road and Pelham Road in the northern areas of the model, and Butlers, Billabong and Yellowstone Roads in the southern modelling area.

The Warrego Highway crossing of Dogwood Creek at Miles is predicted to be inundated for all rainfall events modelled as part of this investigation. Inundation varies from approximately 150mm to 2.5m during the 10 and 100 year ARI events respectively. Similarly, the Leichardt Highway crossing of Columboola Creek is predicted to be inundated for events greater than and including the 20 year ARI, whilst further north near Miles, model results predict the Leichardt Highway to be inundated for all events modelled at a discrete location approximately 1200m south of the town. This area is adjacent to the Miles golf course and is a marked low point in the surrounding topography. Flows in this area originate from a breakout from the main channel in Dogwood Creek.

All proposed infrastructure site locations within the EPP692 tenement area are positioned higher in the catchment, and are predicted to be outside of the regional flood extents for all the design rainfall events modelled as part of this investigation. Table 5.43 summarises predicted peak flood levels at locations adjacent to proposed major infrastructure within the Dogwood Creek catchment area of Tenement EPP692.

Table 5.43 Existing case peak flood levels – model EPP692

	Existing case peak flood levels (m AHD)			
Reporting point location*	10 year ARI (m AHD)	20 year ARI (m AHD)	100 year ARI (m AHD)	500 year ARI (m AHD)
1	304.01	304.75	306.01	306.40
2	303.60	304.31	305.50	305.88
3	302.30	302.42	302.69	302.76
4	297.13	298.09	299.39	299.79
5	296.90	297.60	298.79	299.17
6	295.64	296.67	297.91	298.28

	Existing case peak flood levels (m AHD)			
Reporting point location*	10 year ARI (m AHD)	20 year ARI (m AHD)	100 year ARI (m AHD)	500 year ARI (m AHD)
7	294.72	295.02	295.42	295.59
8	288.86	289.50	290.46	290.76

* Reporting locations shown in Appendix C

Model PL226 (Condamine River)

Tenement PL226 is situated in the wide alluvial floodplain of the Condamine River catchment, and is bounded by Warrego Highway to the north, Chinchilla Tara Road to the east, Kogan Condamine Road to the south and the Leichhardt Highway to the west.

A number of major local tributaries traverse the study site and discharge into Condamine River. These tributaries include;

- Charleys Creek, which flows in a south-westerly direction and joins the Condamine River approximately 4 km downstream of Chinchilla Weir
- Wambo Creek, which flows in a north-westerly direction and joins the Condamine River some 12 km downstream of the Charleys Creek confluence
- Bogrambilla Creek, which discharges in a southerly direction and joins the Condamine River in the centre of the model area
- Wieambilla Creek, which discharges in a north-westerly direction into the Condamine River about 1 km upstream of the confluence of Bogrambilla Creek.

There are also several smaller un-named tributaries that traverse the PL226 tenement contributing flows into the Condamine River.

The flood plain along the Condamine River is dominated by the regional riverine flow from the upper Condamine River, whereas the headwaters of the tributaries across the tenement are dominated by local catchment flows. As discussed in Section 5.2.3, regional riverine flows were calculated by the catchment rainfall factored by an Areal Reduction Factor (ARF) according to the overall size of the catchment to predict regional riverine flows for the TUFLOW model to determine regional flooding across the tenement.

However, the local tributaries traversing across the tenement are waterways with a smaller catchment size and it is unlikely that coincident rainfall would occur over the local tributaries and the entire catchment of Condamine River. As such, point catchment rainfall without an ARF has been used to predict local tributary flows for the TUFLOW model to determine local flooding across the tenement.

To facilitate a conservative modelling approach, ensuring peak flood levels are determined throughout the study area at all locations, the TUFLOW model has been run separately to determine flooding behaviour and flood extents throughout tenement PL226 for the 10, 20, 100 and 500 year ARI events for both local and regional flood scenarios. The model results for local and regional floods for each event analysed have been combined to provide a maximum extent of flooding with respect to both local and regional rainfall events. The predicted peak flood levels at key locations within the Tenement PL226 are presented in Table 5.44.

The model results indicate that significant flows occur in the Condamine River for all four design events, which lead to some large expansive areas of inundation within the floodplain. The model also predicts that flooding is typically well defined in the upper reaches of all tributaries within the tenement from the local storm event, with more expansive flooding evident in the lower reaches, where the flood is dominated by the large regional flows of the Condamine River catchment.

Most road crossings of the major waterways are predicted to be inundated during all rainfall events modelled. These include Kogan-Condamine Road crossing of Wieambilla Creek and adjacent tributary, Kogan-Condamine Road crossing of Wambo Creek and the Chinchilla-Tara Road Bridge crossing of the Condamine River downstream of Chinchilla Weir.

The model predicts that significant road inundation of the Leichardt Highway and Kogan-Condamine Road in the vicinity of the township of Condamine. Road inundation at this location is due to flow breakouts from the Condamine River. Leichardt Highway and Kogan-Condamine Road are expected access routes to not only infrastructure in the PL226 tenement but also other tenement and infrastructure areas to the west and south.

Most of the proposed infrastructure sites within the tenement PL226 study area are located outside of the 100 year ARI flood extent as predicted by the hydraulic model. However, the edge of the proposed GPF_CON_02b site would slightly cross the headwaters of an un-named ephemeral stream. The ephemeral stream would also traverse through the central part of the proposed WTF_CON_01 site some 1200m downstream of the GPF_CON_02b pad assuming their current conceptual locations. Infrastructure WTS_TAL_00 located on the eastern embankment of Borgrambilla Creek near the floodplain of Condamine River would be completely totally inundated during the 100 year ARI flood. Further detailed discussions on these infrastructure sites are included in Section 5.7.7.

Table 5.44 Existing case peak flood levels – model PL226

	Existing case peak flood levels (m AHD)			
Reporting point location*	10 year ARI (m AHD)	20 year ARI (m AHD)	100 year ARI (m AHD)	500 year ARI (m AHD)
1	298.58	298.8	299.95	300.36
2	296.57	296.78	298.49	299.02
3	295.24	295.56	297.54	298.06
4	293.2	293.39	294.54	294.91
5	291.43	291.67	293.26	293.73
6	288.83	289.09	290.41	290.76
7	286.08	286.52	288.45	288.97
8	285.14	285.65	287.63	288.13
9	284.03	284.67	286.72	287.2
10	280.82	281.62	283.82	284.23
11	297.09	297.63	298.88	299.55

	Existing case peak flood levels (m AHD)			
Reporting point location*	**10 year ARI (m AHD)**	**20 year ARI (m AHD)**	**100 year ARI (m AHD)**	**500 year ARI (m AHD)**
12	300.46	300.87	301.48	302.02
13	293.43	294.04	294.91	295.72
14	298.4	298.49	298.65	298.84
15	290.89	290.95	292.4	292.87
16	295.07	295.37	295.92	296.34
17	292.01	292.31	293.33	293.81
18	291.44	291.69	293.33	293.81
18	284.69	285.25	287.33	287.84
20	282.12	282.76	284.59	284.97
21	294.91	294.95	295.08	295.2
22	282.12	282.73	284.29	284.63
23	297.2	297.28	297.44	297.6
24	288.05	288.16	288.35	288.51

* Reporting locations shown in Appendix C

Model SEP692 (Kogan Creek)

Kogan Creek is a northerly flowing river which joins the Condamine River 15 km north of the town of Kogan. The catchment of Kogan Creek is made up of some cleared agricultural land amongst large patches of natural vegetation. The hydraulic modelling area is in the lower reaches of Kogan Creek, upstream of the junction with the Condamine River.

The Dalby Kogan Road/Kogan Condamine Road runs from south east to north west across the study area. The road is inundated at the crossings of various tributaries either side of Kogan, and at the crossing of Kogan Creek at Kogan. The Kogan Tara Road south of Kogan is predicted to experience some inundation during the larger rainfall events modelled as part of this investigation.

Results from the hydraulic model indicate that all proposed infrastructure site locations are outside of the 100 year ARI flood extent.

Table 5.45 summarises predicted peak flood levels at areas adjacent to proposed major infrastructure within tenement SEP692.

Table 5.45 Existing case peak flood levels – model SEP692

	Existing case peak flood levels (m AHD)			
Reporting point location*	10 year ARI (m AHD)	20 year ARI (m AHD)	100 year ARI (m AHD)	500 year ARI (m AHD)
1	332.39	332.55	332.76	332.93
2	331.28	331.41	331.64	331.88
3	339.17	339.20	339.25	339.30
4	335.41	335.50	335.64	335.78
5	328.67	328.87	329.23	329.55
6	335.77	335.81	335.91	335.99

* Reporting locations shown in Appendix C

Model EPP663 (Weir River and Western Creek)

The two main watercourses within Tenement EPP663 are the Weir River and Western Creek. Both of these systems are located within the Weir River sub-basin, flow in a westerly direction, and are classified as major waterways. Western Creek joins the Weir River some 40km downstream of the tenement boundary.

Flood modelling results for Weir River and Western Creek suggest flows for all events modelled are well defined, with only a few areas of shallow expansive flooding, typically in the lower modelling areas.

All road crossings within the modelling area are shown to be inundated during all rainfall events modelled as part of this investigation. Typically the roads are of an unsealed nature with natural crossings of the creek systems. Examples include Bull Creek Road in the southern modelling area and both Weir River and Cecil Plains Roads in the northern modelling area.

Within the Weir River catchment of the tenement, most proposed infrastructure is shown to be located outside the 100 year ARI event flood extent. However, GPF_WAA_03 within the upper Tea Tree Gully system (a minor tributary of the Weir River), would intersect the 100 year ARI flood extent in its current conceptual location.

All proposed infrastructure locations within the Western Creek catchment and contributing tributaries are all shown to be well outside the 100 year ARI flood extent. Table 5.46 summarises peak flood levels at locations adjacent to proposed infrastructure.

Table 5.46 Existing case peak flood levels – model EPP663

	Existing case peak flood levels (m AHD)			
Reporting point location*	10 year ARI (m AHD)	20 year ARI (m AHD)	100 year ARI (m AHD)	500 year ARI (m AHD)
1	321.93	322.05	322.31	322.58
2	318.74	319.35	320.33	321.17
3	333.79	333.90	334.10	334.28
4	339.08	339.24	339.56	339.84
5	357.31	357.49	357.82	358.12
6	315.42	315.73	316.37	316.91
7	336.73	336.80	336.92	337.04
8	376.43	376.44	376.46	376.49
9	357.33	357.37	357.46	357.53
10	334.75	334.85	335.14	335.46

* Reporting locations shown in Appendix C

5.6 Sensitivity analysis

A number of sensitivity analyses have been undertaken on all of the 1D/2D TUFLOW hydraulic models developed as part of this investigation.

These sensitivity analyses were aimed at determining the possible effects that changes to model parameters (Mannings 'n' roughness values) as well as increased rainfall intensity as a result of climate change would have on catchment responses and predicted flood levels.

Sections 5.6.1 and 5.6.2 detail the sensitivity analysis undertaken as part of this investigation.

5.6.1 Rainfall sensitivity analysis (climate change)

A sensitivity analysis of the nine 1D/2D TUFLOW models was undertaken to determine the impact that an increase in extreme event rainfall intensity for the 100 year ARI event would have of predicted peak flood levels and overall catchment responses.

Review of available literature for the assessment of the impact of climate change on rainfall intensities showed a range of recommended values for rainfall intensity increases. Guidelines published by the Australian Governments Department of the Environment and Heritage (Australian Greenhouse Office) titled 'Climate change scenarios for initial assessment of risk in accordance with risk management guidance' (CSIRO, 2006) were deemed the most applicable for the purposes of this assessment.

These guidelines state various rainfall values for different climates across Australia. The 'South East Queensland' zone was deemed representative of the study area, and the recommended values were reviewed. The guidelines state that *'Increases in extreme weather events are likely to lead to increased flash flooding'* with suggested changes to rainfall intensity for the 20 year ARI event varying

from no change in the 'low global warming' scenario to a 30% increase in the 'high global warming scenario'. These changes were based on predictions for the year 2040.

Review of proposed increases in rainfall intensities for the catchments revealed that a 30% increase in the 20 year ARI rainfall intensity resulted in similar intensities to the 100 year ARI rainfall events (and therefore results in similar flood levels and extents). To facilitate review of impacts on the required design event for most infrastructure associated with the project (100 year ARI), an increase of 30% intensity for the 100 year ARI rainfall event was adopted for the purposes of this investigation. This intensity was shown to be similar to the 500 year ARI design event rainfall intensity for some catchments.

Increases in rainfall intensity of 30% for the 100 year ARI event were simulated in the XP-RAFTS models, with the resultant local and total catchment hydrographs applied to the nine 1D/2D TUFLOW hydraulic models.

The following sections summarise the impacts on regional flood levels only as a result of the rainfall sensitivity analysis on the performance of each hydraulic model.

Model EPP606N (Horse Creek & adjacent tributaries)

Model results suggest peak flood levels for the 100 year ARI event as a result of a 30% increase in rainfall intensity would be some 50mm – 300mm higher than those predicted for the existing case 100 year ARI event.

Increases are shown throughout the modelling area, and peak flood levels are comparable to, if not slightly higher in some locations than the 500 year ARI existing case predicted flood levels and extents.

However, as most infrastructure pads would be located well outside the existing case 500 year ARI flood extents within the model, impacts on proposed infrastructure as a result of increased rainfall intensity for those areas within the northern facing aspects of tenement EPP606 are considered negligible.

Model EPP606S (Yuleba Creek)

Review of model results suggests increases in peak flood levels for the 100 year ARI event as a result of the 30% increase in rainfall intensity varies from approximately 50mm in the higher catchment areas to up to 600mm in discrete regions of the lower Yuleba Creek modelling area. Similarly to the northern facing watercourses, peak flood levels are comparable to the 500 year ARI existing case peak flood levels and extents in most locations.

Again, as most infrastructure would be located outside of the 500 year ARI existing case flood extent (except for those previously mentioned in Section 0) the increased rainfall intensity is unlikely to impact on most proposed infrastructure site locations from a regional flooding perspective.

Model PL209 (Woleebee Creek)

Model results for the 30% increased rainfall intensity scenario in the Woleebee Creek catchment suggest that peak flood levels would increase by up to 300mm or less over most of the study area. The upper reaches of the tributaries show an increase generally in the order of 50mm to 200mm. A number of discrete sections of the main channel have increases in peak flood levels ranging from 300 to 500mm while only a few isolated patches throughout the model have an increase greater than 500mm. The 500 year ARI existing case peak flood levels and flood extent are very similar to the

increased rainfall scenario peak flood levels and extent. The climate change model levels are generally within 30mm of the 500 year ARI existing case levels.

All proposed infrastructure sites would be outside of both the 500 year ARI existing extent and the 30% increased rainfall extent. Inundation of roads is also comparable. Hence, impacts on proposed infrastructure due to possible regional flooding impacts of climate change in the Woleebee Creek catchment are considered negligible.

Model EPP972 (Tchanning Creek)

Peak flood level predictions for the 30% increased rainfall intensity scenario vary across the modelled area with increases of up to approximately 750mm evident in some areas, but with most of the model showing increases of less than 500mm over the existing case 100 year ARI event. The largest increases typically occurred at junctions and in the lower reaches of the study area. The upper reaches of the study area showed increases of between 50mm to 200mm. The 30% increased rainfall scenario results are very similar in peak flood level to the 500 year ARI existing case results but are slightly lower over the model area. The 30% increased rainfall peak flood levels are up to 200mm lower than the 500 year ARI levels.

Most proposed infrastructure is outside of the flood extent for both the 100 and 500 year ARI events and remains well outside for the 30% increased rainfall scenario. However, at its current proposed location, ‘GPF_HCK_01a’ would be very close to the 100 year ARI flood extent and slightly inundated for the 500 year ARI event. A level of inundation similar to the 500 year ARI event occurs for the 30% increased rainfall scenario.

Model EPP973 (Dulacca Creek)

Increases in predicted peak flood levels of between 150mm and 600mm are predicted for the 100 year ARI peak flood level as a result of a 30% increase in rainfall intensity within the EPP973 modelling area.

These levels are comparable to the predicted 500 year ARI existing case flood levels in the entire modelling area.

As most proposed infrastructure would be located well outside the 500 year ARI existing case flood extent, increases in flood levels and extents are not considered to impact on proposed infrastructure sites in the modelling area.

Model EPP692 (Dogwood Creek)

Increasing rainfall intensity by 30% for the 100 year ARI event is shown to have a noticeable impact on flood levels within the EPP692 modelling area. Increases of up to 1250mm over the existing case 100 year ARI rainfall event in the upper modelling area are shown to occur, whilst increases are less marked within the smaller tributaries from the west, with increases in peak flood levels predicted to be around 300mm.

The significant increases in predicted peak flood levels due to the 30% increase in rainfall intensity for the 100 year ARI event are considered to be a result of the large contributing Dogwood Creek catchment (slightly less than 5,000km^2). Large catchments of this nature tend to show non-linear relationships to rainfall intensity, and hence 30% increases in rainfall intensity for the 100 year ARI design rainfall event represent similar intensities to extreme rainfall events much greater than the 500 year ARI design rainfall event. Results therefore predicting significant increases to flood levels are likely to be representative of catchment and waterway response to larger, extreme rainfall events.

Results for this analysis are shown to be comparable to the 1956 record flood event, where rainfall isohyets indicate similar intensities for the 72 hour storm event to those adopted in this sensitivity analysis.

Nonetheless, as all proposed infrastructure pads within tenement EPP692 would be located at elevation and well outside the regional flood extents for all events modelled as part of this investigation, the possibility of a 30% increase in rainfall intensity would not appear to impact on infrastructure site locations with respect to regional flooding.

Model PL226 (Condamine River)

Modelling of the increased rainfall intensity for the 100 year ARI event over the PL226 modelling area predicts increases in peak flood levels of between 1000mm to over 1500mm along the Condamine River and its flood plain. Unlike other models with a catchment area typically within 1,000 km^2, the increased flood levels in PL226 are much larger than the predicted 500 year ARI rainfall event. This is due to the non-linearity behaviour of rainfall intensity for the extremely large catchment (25,000 km^2) of the Condamine River system. The EPP692 Dogwood Creek model with a total catchment area of some 5,000 km^2 possesses similar non-linearity of rainfall behaviour (refer to Section 5.2).

Although the predicted flood extent of the 100 year ARI event with a 30% increase in rainfall intensity is significantly larger than the 500 year ARI event, the majority of proposed infrastructure would be located well outside the existing case 500 year ARI flood extent, therefore the possible impacts associated with increased regional flood levels as a result of climate change effects would impact on the proposed infrastructure already identified in Section 5.5.4.

Model SEP692 (Kogan Creek)

According to model results, a 30% increase in rainfall intensity in the Kogan Creek catchment is predicted to result in increases in peak flood levels of 300mm or less over most of the modelling area. Some areas within the downstream limits of the model show discrete areas of larger increases of between 300 to 500mm. The 100 year ARI with 30% increased rainfall scenario results are very similar to the 500 year ARI existing case results across the modelling area. All proposed infrastructure would be well outside the 30% increased rainfall scenario flood extent, and as such a 30% increase in rainfall intensity in the 100 year ARI event is not predicted to impact on proposed infrastructure locations in the SEP692 modelling area.

Model EPP663 (Weir River and Western Creek)

Modelling of the increased rainfall intensity for the 100 year ARI event over the EPP663 modelling area predicts increases in peak flood levels of between 20mm to over 1000mm in some areas.

These increased flood levels are comparable to those predicted for the 500 year ARI existing case rainfall event. Similarly to other modelling areas, the majority of proposed infrastructure would be located well outside the existing case 500 year ARI flood extent, therefore the possible impacts associated with increased flood levels as a result of climate change effects are limited to the proposed infrastructure identified in Section 5.5.4.

Summary of rainfall intensity sensitivity analysis results

Each of the nine TUFLOW hydraulic models showed similar representative increases in flood levels as a result of a 30% increase in rainfall intensity for the 100 year ARI event.

All models except EPP692 and PL226 showed similar flood levels to the predicted existing case 500 year ARI event flood levels. Flood levels predicted in EPP692 and PL226 models are well above the 500 year ARI event flood levels. This is attributed to non-linearity behaviour of rainfall intensity due to extremely large catchment areas.

As most proposed infrastructure within the separate tenement areas are located outside the 500 year ARI flood extent, the possible increase in rainfall intensity and regional flooding as a result of predicted climate change impacts is not considered to cause any significant impacts to proposed infrastructure locations. The majority of the proposed infrastructure site locations shown to be within existing case flood extents would not be affected by rainfall increases if infrastructure items were located in areas clear of existing case 500 year ARI flood extents in most models. Similarly, modification to pad locations may also be utilised as a mitigation measure, as discussed in Section 5.7.

5.6.2 Mannings ‘n’ sensitivity analysis

Investigation was undertaken into the sensitivity of the nine 1D/2D TUFLOW models to an increase in Mannings ‘n’ roughness values of 20% as shown in Table 5.47.

Table 5.47 Adopted sensitivity analysis roughness parameters

Land use type	Normal Manning’s ‘n’ roughness	Adopted sensitivity Manning’s ‘n’ roughness
Water body	0.015	0.018
Road carriageway	0.025	0.030
Cleared land / agriculture	0.040	0.048
Cleared land / sporadic vegetation	0.045	0.054
Light vegetation	0.060	0.072
Bushland / natural environments	0.080	0.096
Buildings / homestead	0.100	0.120
Township / district centre	0.250	0.300

Depending on catchment topography and sub catchment sequencing, increased roughness values may cause routed hydrograph peaks to be slightly delayed, and thus may coincide with other hydrograph peaks causing increases in flood levels, or cause previously combined peaks to separate, therefore lowering flood levels. Typically in watercourse specific regional floodplain studies of this nature, an increase in Mannings ‘n’ roughness values will slow flows within floodplain areas, causing typically increased peak flood levels and extents.

Modelling results suggest increases of 20% on the adopted Manning’s ‘n’ roughness for the watercourses investigated as part of this analysis caused some minor variations in the peak flood levels throughout the different waterway systems.

The following sections discuss each model’s sensitivity to the increased Mannings ‘n’ values.

Model EPP606N (Horse Creek & adjacent tributaries)

Increased Mannings ‘n’ values within the EPP606N hydraulic model typically results in increases across the modelling area of between 20mm and 150mm for the 100 year ARI event. Most areas

besides Horse Creek (Main Branch) are shown to have increased peak flood levels of between 50mm and 100mm. Horse Creek (main branch) typically has increases of between 100mm and 150mm.

Flood extents remain largely unaffected as a result of the minor increases in flood levels, and no additional proposed infrastructure locations to those identified in Section 5.5.4 would be affected as a result of the minor increases in flood levels.

Model EPP606S (Yuleba Creek)

Review of modelling results suggest that increases in flood levels as a result of a 20% increase in adopted Mannings 'n' values are generally between 20mm and 150mm. These increases vary across the study area.

Flood extents as a result of the increased water level remain largely unaffected, with only one area near the northern most point of the system showing an additional minor flow path as a result of the increased water level. This area is not near any proposed infrastructure.

Model PL209 (Woleebee Creek)

Modelling results for an increase in adopted Mannings 'n' values show a minor impact on peak flood levels and extents with an increase of up to 200mm over most of the study area. The increases are fairly uniform throughout the waterway.

Flood extents are largely unchanged from the 100 year ARI existing case and the proposed infrastructure would remain outside of the peak flood extents.

Model EPP972 (Tchanning Creek)

A review of model results for the Tchanning Creek model show that increases in flood levels due to a 20% increase in adopted Mannings 'n' values are predominantly less than 200mm with most areas showing an increase ranging from 40mm to 150mm.

The flood extent for the increased Mannings 'n' scenario is very similar to the 100 year ARI existing case flood extent with some widening in localised areas. All proposed infrastructure within the study area would remain outside of the flood extent although the edge of 'GPF_HCK_01a' would be in very close proximity to the predicted existing case 100 year ARI extent as well as the increased Mannings 'n' flood extent.

Model EPP973 (Dulacca Creek)

Modelling results suggest a 20% increase in Mannings 'n' values across the EPP973 hydraulic model generally results in increases in peak water surface levels of 50mm-150mm throughout the modelling area. Discrete areas with greater increases are shown near the downstream boundaries and are considered to be the result of the boundary condition type (normal depth) and its inherent dependence on the Mannings 'n' values. Flood extents remain largely unchanged, with only very small discrete regions of additional flood prone land as a result of the predicted increase in flood levels. No additional impact on proposed infrastructure is predicted as a result of increased floodplain roughness.

Model EPP692 (Dogwood Creek)

Increasing Mannings 'n' values in the EPP692 (Dogwood Creek) model resulted typically in increases in predicted peak flood levels throughout the system. Higher increases of up to 350mm were evident in the upper limits of the model, where high flow rates are concentrated in areas of higher roughness values (n=0.080). Within the lower regions of the model, where cleared farmland style landuse (and

associated low Mannings 'n' values) is evident, increases are in the order of 100-150mm. No additional impact on proposed infrastructure is predicted as a result of increased floodplain roughness as infrastructure areas are located well outside the 100 year ARI flood extent.

Model PL226 (Condamine River)

Increasing the Manning's 'n' values by in the PL226 (Condamine River) regional flood model by 20% resulted in a difference in water surface levels of up to 300mm within the flood plains of the Condamine River. For the tributaries, increases of water surface levels are predicted to be typically within the order of 100mm. Modelling results revealed that a 20% increase in manning's n values has only minor impacts on flood extents throughout the PL226 modelling area. No additional infrastructure was predicted to be inundated as a result on the increased model floodplain roughness.

Model SEP692 (Kogan Creek)

Modelling results for the 20% increase in Mannings 'n' scenario in the Kogan Creek catchment show an increase in peak flood levels of 50 to 150mm over the study area. Some larger increases are evident near the downstream boundary of the model and are considered to be a result of the normal depth boundary used. These areas are well downstream of the Origin tenement boundary and proposed infrastructure. There is very little change to the flood extent and all proposed infrastructure remains well outside the flood extent.

Model EPP663 (Weir River and Western Creek)

Review of model results suggests that a 20% increase in adopted Mannings 'n' values across the floodplain results in increases in predicted flood levels across the modelling area of up to approximately 250mm.

The increases are typically in areas of already high Mannings 'n' values (n=0.080). One area in the extreme lower reaches of Weir River modelling area shows significant increases as a result of the increased Mannings 'n' values. This would appear to be a direct result of the model boundary type at this location (normal depth) and its inherent sensitivity to Mannings 'n' values. No additional infrastructure would be impacted as a result of increased floodplain roughness.

Summary of Mannings 'n' sensitivity analysis results

Each of the nine TUFLOW hydraulic models showed varying sensitivities to a 20% increase in Mannings 'n' values.

Those areas with well defined flow paths and already high roughness values or high flows experience the most marked increase in predicted flood levels, whereas areas of expansive flooding with lower velocities and flows did not appear to be as heavily impacted.

Based on the results of the sensitivity analysis and previous experience in floodplain studies of this nature, it is considered the Mannings 'n' roughness values adopted as shown in Table 5.12 are representative of floodplain roughness and have been adopted for the purposes of this investigation.

No additional infrastructure sites are predicted to be inundated as a result of increased Mannings 'n' floodplain roughness values.

5.7 Base case & flood mitigation assessment

Hydraulic modelling for the base case for the PL226 modelling area was undertaken using the 1D/2D TUFLOW models as used in the existing case analysis, with incorporation of proposed infrastructure where appropriate.

Modification of the PL226 model to facilitate modelling of the base case was undertaken to represent proposed infrastructure sites that were located within the regional flood extents and were considered to be capable of causing changes to flood behaviour in surrounding areas.

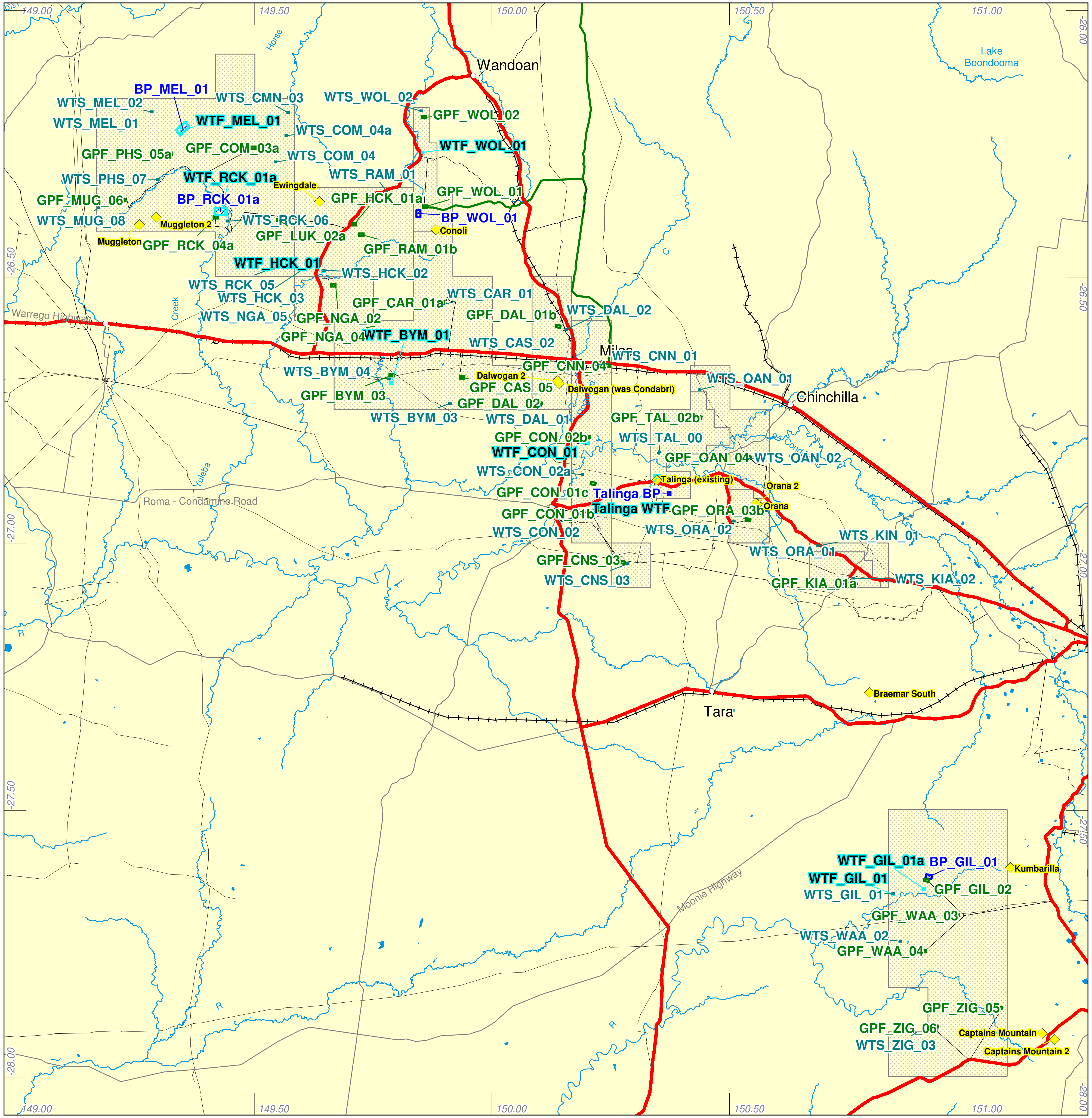
Most hydraulic modelling areas representing the other tenements were not simulated for the base case due to:

- Proposed infrastructure locations were shown to be well outside the flood extents for all rainfall events modelled, meaning no impacts on regional flood behaviour would result
- There was only minor/partial intrusion into flood extents from proposed infrastructure sites where it was noted slight re-alignment/re-positioning of the proposed location would eliminate any impact on flood behaviour.

Access roads to the various infrastructure locations throughout the tenement areas were typically along existing road alignments. Given the extensive nature of inundation and large flow rates at most crossings in the existing case for the larger rainfall events, it is not considered necessary or feasible for flood immunity to be improved as part of the project. Road inundation is not expected to cause major operational constraints for the project. However, in case Australia Pacific LNG should decide in the future that selected crossings need to be upgraded; investigation into crossing augmentation has been undertaken in accordance with the Queensland Urban Drainage Manual (2007). This includes 50 year ARI and 10 year ARI immunity for major and minor road crossings respectively. All of the road crossing augmentation options listed take into consideration impacts on flood behaviour on surrounding areas. Again, it is noted that crossing upgrades are not proposed by Australia Pacific LNG at this stage, and these investigations are for discussion purposes only.

Modelling for the base case in the PL226 tenement to determine the impacts of proposed infrastructure pad locations within the existing flood extents was performed for the 100 year ARI rainfall event only. Detailed flood afflux mapping for the base case is presented in Appendix F for the PL226 TUFLOW hydraulic model.

Figure 5.18 shows the location of the proposed infrastructure investigated as part of this regional flooding analysis.



LEGEND

- Town
- Communication tower
- Existing railway
- Road
- Major watercourse
- Preferred pipeline alignment
- Major access road
- Minor access road
- Water treatment facility
- Water transfer station
- Brine pond
- Gas processing facility
- Walloons gas fields development areas

0 25 50km

SCALE - 1:850,000 (at A3)

Latitude / Longitude
Geocentric Datum of Australia 1994

N W E S

Rev	Date	Revision Description	ORIG	CHK	ENG	APPD
0	24/11/2009	Issued for use	JM	DH		

WorleyParsons
resources & energy

AUSTRALIA PACIFIC LNG PTY LIMITED

AUSTRALIA PACIFIC LNG PROJECT EIS
Figure 5.18 Locality Plan of Proposed Infrastructure

Project No: 301001-00448	Figure: 00448-00-EN-DAL-2212	Rev: 0

Compiled by BRISBANE INFRASTRUCTURE GIS SECTION

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5.7.1 Model EPP606N (Horse Creek & adjacent tributaries)

Most of the proposed infrastructure sites within the northern facing catchments of tenement EPP606 are shown to be flood free for all the rainfall events modelled as part of this investigation. However there are portions of a limited number of proposed site locations that are shown to slightly intersect the 100 year ARI flood extent.

The western corner of WTF_MEL_01 is shown to intersect the flood extent of a minor tributary of Kangaroo Creek high in the catchment. Flows at this location are in the order of $15m^3/s$ during the 100 year ARI event. If this portion of the site was used for infrastructure placement, flows would pond behind the fill until flowing around the northern edge and rejoining the existing tributary alignment. It is therefore recommended that no infrastructure items or fill be placed within this area of the proposed site. Alternatively, slight re-alignment of the proposed site location could also be undertaken. These options would leave the tributary in its natural state and negate the need for mitigation works. Review of the proposed site shows this could be achieved without causing changes to the natural flow regimes of the adjacent Appletree Creek. Similarly, the location of WTS_COM_04 places it within the 100 year ARI flood extent. The flooding at this location arises from local catchment flow only (less than $5m^3/s$), and again slight re-alignment of the site location (in a south west direction) would enable the minor tributary and flood behaviour at this location to remain unchanged, negating the need for scour protection or channel works.

Access roads within the EPP606N model extents are predicted to be susceptible to varying degrees of inundation. Potters Flat Road crosses a number of minor tributaries in the upper catchment before crossing Horse Creek (Main Branch) approximately 2km upstream from WTS_COM_04a. Flows at this point are approximately $100m^3/s$ during the 10 year ARI event. Given the peak flow rate and flood behaviour at this location, it is expected that either a bridge structure or series of appropriately sized culverts would enable 10 year ARI immunity at this location. Further downstream, Potters Flat Road crosses Horse Creek (West Branch) and Four Mile Gully at their confluence with the Main Branch. Inundation of the roadway at these locations during the 10 year ARI event ranges from shallow sheet flow (<100mm) in some floodplain areas up to approximately 1m in the main channel of the West Branch and Four Mile Gully. Peak flows of approximately $85m^3/s$ are predicted in the West Branch during the 10 year ARI event. It is envisaged a combination of road raising and appropriately sized cross road drainage in the form of a series of banks of culverts incorporated at appropriate low points in the floodplain topography would be required at the West Branch of Horse Creek to facilitate 10 year ARI immunity. A similar approach in Four Mile Gully, where flood extents are less expansive and are typically contained in the main channel, should also provide 10 year ARI immunity.

Clifford – Yuleba Road is shown to be inundated by flows from Kangaroo Creek during all the events modelled. Flows of approximately $95m^3/s$ during the 10 year ARI event are predicted to occur, with flooding depths at the crossing of up to 0.5m. Whilst the flooding at this point is expansive (800m wide during the 10 year ARI event), much of the flow is shallow, and it is envisaged a combination of improved channel definition, crossing upgrade and road raising at this location would enable 10 year ARI immunity.

Glenarden Road crosses Barton Creek as it heads in a westerly direction from the intersection with Wykola Wallumbilla Road. This would be the likely access route to WTS_MEL_01. Topography at this crossing is not well defined, and flows are shown to be expansive with a flood width of 220m for the 10 year ARI event. Flood waters at the Barton Creek crossing are approximately 0.6m deep with a peak flow of approximately $50m^3/s$. Road raising and incorporation of an appropriately sized culvert structure would facilitate a 10 year ARI immunity of Glenarden Road.

Any impacts on peak flood levels from the upgrading of the creek crossings for access roads within EPP606N modelling area are likely to remain localised and well within the Australia Pacific LNG lease holding. Review of available data reveals it is unlikely any nearby residential buildings would be impacted from crossing augmentation within the EPP606N modelling area, however further modelling of any proposed crossing augmentation designs would be undertaken to confirm no impacts on surrounding residences or infrastructure prior to those augmentation works proceeding.

5.7.2 Model EPP606S (Yuleba Creek)

The current proposed site location of the alternate site WTF_RCK_01a lies over a section of Ten Mile Creek. If infrastructure items and associated fill were located in this portion of the site, it would be expected that flows (peak flow of 25m^3/s in the 100 year ARI event) in Ten Mile Creek would back up behind the fill, before flowing around the north western corner and down along the northern edge of the site before joining Yuleba Creek. Whilst flows in Ten Mile Creek are minor, the current location of the site essentially blocks the existing alignment of the creek, and as such, it is envisaged that adjustment to the site location (southward movement) could be undertaken at this location to nullify any possible impacts and eliminate the need for channel diversion or scour protection works adjacent to the infrastructure sites. Alternatively, infrastructure items could be placed on areas of higher elevation within the designated infrastructure site (banks of Ten Mile Creek) outside of the 100 year ARI flood extent to maintain natural flow regimes within the creek.

Similarly, the proposed location of WTS_PHS_07 is shown to slightly intersect the existing case 100 year ARI flood extents of Yuleba Creek. Again, slight adjustment of the positioning of the site at this location or restricting fill and infrastructure within this portion would facilitate an elimination of any localised impacts on flood behaviour.

Review of the proposed (existing) access roads within the EPP606S modelling area shows a number of crossings of the Yuleba Creek system. The alignment of Cattle Creek Road (access to GPF_MUG_06) shows the road crossing Yuleba Creek twice, approximately 1500m upstream and 2500m downstream of WTS_PHS_07. Flows in these locations are in the order of 35m^3/s and 65m^3/s respectively during the 10 year ARI event. Given the nature of flood behaviour at these locations, it is envisaged that adequately sized culverts at the main channel crossings in combination with road raising would be required to facilitate adequate crossing immunity. It is also noted the existing alignment of Cattle Creek Road along the northern extent of Yuleba Creek also shows significant areas of roadway inundation where the alignment runs parallel to the creek. Flow breakouts across the road are shown to occur even during the 10 year ARI event. Road raising along this section would be required if 10 year ARI immunity of this access road was found to be required in the future for project operation purposes.

Similarly, significant inundation of crossings in the southern regions of the EPP606 tenement is evident near WTS_RCK_06. Yuleba – Taroom Road crosses two significant tributaries of Yuleba Creek (Wirginbah Creek and Sandy Creek), whilst Seaside Road crosses Yuleba Creek a short distance downstream of the aforementioned confluences. Much of this area is shown to be inundated during all of the rainfall events modelled during this investigation. Flood depths within the Yuleba Creek floodplain along Seaside Road are in the order of 200mm, with much deeper depths in the main channel of around 3.5m, whilst peak flows crossing Seaside Road are in the order of 170m^3/s during the 10 year ARI event. Peak flows of approximately 60m^3/s are shown to flow across Yuleba – Taroom Road from Wirginbah Creek, with flood depths of typically 1.5m during the 10 Year ARI event. Similarly, flows crossing Yuleba – Taroom Road from Sandy Creek are in the order of 50m^3/s, with flood depths of up to 300mm in the floodplain, and 1.5m in the main channel during the 10 year ARI event.

It is envisaged that road raising through the aforementioned floodplain areas and either bridge construction or significant culvert augmentation over the main waterway channels would be required to achieve 10 year ARI flood immunity at these locations.

Any impacts on peak flood levels from the upgrading of the creek crossings for access roads within EPP606S modelling area would likely remain within the Australia Pacific LNG lease holding. Review of available data reveals raising Cattle Creek Road through low lying floodplain areas to achieve 10 year ARI immunity may impact on one residential building upstream. Similarly, one residence to the north of Seaside Road may be affected by the augmentation of the Seaside Road crossing of Yuleba Creek. Further modelling of any proposed crossing augmentation designs would be undertaken to confirm no impacts on surrounding residences or infrastructure prior to those augmentation works proceeding.

5.7.3 Model PL209 (Woleebee Creek)

All proposed infrastructure site locations in the PL209 model are outside of the predicted flood extents for all rainfall events. The location of WTS_RAM_01 is shown to be close to the predicted 100 year ARI flood extent, however is predicted to remain flood free.

Jackson Wandoan Road runs adjacent to the main channel of Woleebee Creek and crosses both Woleebee Creek and its tributaries several times. In the upper reaches of the catchment the road crosses a couple of small unnamed tributaries with flows in the order of 20 to 40m^3/s. Achieving flood immunity for these crossings will be a matter of determining adequate cross road drainage capacities and road embankment elevations at these locations. Jackson Wandoan Road crosses another unnamed tributary approximately 2km north of the Ramyard Creek crossing with a 10 year ARI flow of 65m^3/s. Model results show that there are peak flood depths over the road of approximately 1.5m. Flood immunity for the 10 year ARI event will likely require a combination of adequate road raising and appropriately sized cross road drainage.

The crossing of Ramyard Creek is also in the upper reaches of the catchment and consists of three (3) 3.65m x 2.18m RCBC (as measured on the site inspection). The 10 year ARI flow in Ramyard Creek of 35m^3/s is predicted to be within the current capacity of the culverts, with no inundation of the road crossing predicted.

Jackson Wandoan Road also crosses the main branch of Woleebee Creek a number of times in the lower reaches of the model area. One crossing is just to the west of WTF_WOL_01. An existing bridge crosses the main channel of Woleebee Creek at this point, but the flat nature of the topography results in an expansive flood width of approximately 750m at this crossing. A 10 year ARI flow of 200m^3/s crosses the road along this length with peak flood depths over the road ranging from less than 400mm to as much as 1.3m. Providing 10 year ARI flood immunity at this location would appear difficult due to the magnitude of flow and flat topography at this location. Immunity would likely require significant channel and possibly bridge augmentation, road raising through the floodplain areas with a number of suitably sized culvert banks to provide cross road drainage at the low points of the flood plain.

The Ogle Creek crossing is another expansive flood extent with depths usually less than 400mm. The 10 year ARI flow at this crossing is 43m^3/s with a flood width of approximately 600m. The provision of 10 year ARI flood immunity would likely require a raised road embankment with appropriately sized culvert banks at respective low points in the flood plain to provide adequate cross road drainage.

In the lowest reaches of the study area Jackson Wandoan Road winds through the flood plain with an extended length of inundation in the 10 year ARI event. A flow of approximately 320m^3/s crosses the

road along this length with depths of generally less than 300mm over the flood plain. Providing flood immunity for the 10 year ARI event across this flood plain will require a raised road embankment and a series of culvert banks to provide cross road drainage at respective low points along the flood plain. The flow in the main channel also inundates the road at the bridge crossing. Some channel works may be enough to provide flood immunity at this crossing. Otherwise, the level of the bridge and road approaches will need to be raised.

The last flow crossing of Jackson Wandoan Road in the study area is of another unnamed tributary. Results of the 10 year ARI event show that this crossing is inundated. As for other crossings, flood immunity will require a combination of appropriately sized culverts and possible raising of the road level.

Access roads for proposed facilities WTS_WOL_02 and GPF_WOL_02 have not been specified and an examination of aerial photography shows no major existing road access. There are some unnamed unsealed roads to the west of Woleebee Creek that may provide suitable access however the crossings of these roads have not been modelled. Access could also be provided from Jackson Wandoan Road, which is 3km to the west. This options would require another crossing of the Woleebee Creek flood plain.

A proposed access road for GPF_WOL_01 runs from Giligulgul Road near WTF_WOL_01 south to the facility. This road does not appear on recent aerial photography. The road crosses the model area and the flood extent at Hellhole Creek. The 10 year ARI event results show a flow of approximately $50m^3/s$ at this crossing. The proposed crossing point is in a flat region with an expansive flood extent compared to nearby sections of the channel which are better defined. Realigning the proposed road further to the east will reduce the inundated length and could take advantage of an existing unsealed road. This would allow a reduction in the works required at the crossing to achieve 10 year ARI immunity.

Any impacts on peak flood levels from the upgrading of the creek crossings for access roads within PL209 modelling area are likely to remain localised and well within the Australia Pacific LNG lease holding. Review of available data reveals it is unlikely any nearby residential buildings would be impacted from crossing augmentation within the modelling area. Further modelling of any proposed crossing augmentation designs would be undertaken to confirm no impacts on surrounding residences or infrastructure prior to those augmentation works proceeding.

5.7.4 Model EPP972 (Tchanning Creek)

Most proposed infrastructure sites within the EPP972 model area is well outside the regional flood extents for all ARI events modelled. As noted in earlier sections of this report the proposed location of GPF_HCK_01a is located on the edge of Clark Creek and is just outside the 100 year ARI flood extent, but slightly within the 500 year ARI flood extent. For a greater degree of flood immunity it is recommended that this site is moved slightly to the east, away from Clark Creek, or infrastructure is not places within the portion of the site subject to inundation. This will reduce potential impacts on local flooding or the need for mitigation works.

Jackson Wandoan Road runs through the centre of the EPP972 model and extends north into the model of PL209. The road crosses the main waterway, Tchanning Creek, a tributary called Noonga Creek and runs beside Clark Creek with several minor creek crossings before crossing Clark Creek at the northern end of the model area.

The crossing of Tchanning Creek is just downstream of the junction with Clark Creek and Noonga Creek and has a 10 year ARI flow of $240m^3/s$. The existing crossing is of a causeway design and

consists of nine (9) 1.2m x 1.2m RCBC. These culverts were shown to be typically blocked during site inspection and are not enough to provide flood immunity and the 10 year ARI model results show flood depths of 3m or more over the road. The Tchanning Creek channel is well defined in the vicinity of this road crossing. Providing 10 year ARI flood immunity in this case will require raising the level of the road and potentially increasing the capacity of the cross road drainage.

The Noonga Creek crossing of Jackson Wandoan Road currently consists of six (6) 2.45m x 2.2m RCBC and the road is inundated for the 10 year ARI existing case. The flow at this crossing is $52m^3/s$ with approximately $22m^3/s$ flowing over the road at depths of 1 to 2m. As for the Tchanning Creek crossing, flood immunity will require raising the level of the road and increasing the capacity of the cross road drainage.

North of Noonga Creek, Jackson Wandoan Road runs along the western side of Clark Creek. At some locations the road passes along the edge of the flood extent with some shallow inundation. These model results may be due to the grid size used in the model not capturing the existing road embankment in sufficient detail. In regions where the road does not have a sufficient embankment height, flood immunity could be achieved by raising the road level.

At the northern end of the model area Jackson Wandoan Road crosses Clark Creek. Model results for the 10 year ARI event show that at this point there is a flow of $37m^3/s$ in Clark Creek. Due to the flat topography in the vicinity of the crossing flow passes over the flood plain and flows beside the road for a period before crossing the road to the south of the main channel crossing. The flood depths along the flood plain are generally less than 100mm at the road. Depths at the crossing of the main channel are up to 700mm. Providing flood immunity for the 10 year ARI event may require some channel works upstream to confine the flow to the main channel as well as slightly raising the road and providing appropriately sized cross road drainage in the main channel in the form of a bank of culverts. If channel work is unable to confine the flow to the main channel then appropriate cross road drainage at low points of the flood plain would also be necessary. It is noted that there is a farm house approximately 600m upstream of the road crossing which is predicted by model results to be inundated in the 10 year ARI event by approximately 100mm. Any improvements to flood immunity at this crossing will need further modelling to be undertaken to confirm no impacts on this residence prior to those augmentation works proceeding.

It is noted Jackson Wandoan Road crosses many minor tributaries within the tenement that were not included in the analysis due to the regional nature of the hydraulic model. These crossings were typically of a causeway nature. As such, further analysis of these catchments and crossings would need to be undertaken to ensure adequate road immunity at these locations if road immunity was required.

Any impacts on peak flood levels from the upgrading of the creek crossings for access roads within EPP972 modelling area are likely to remain localised and well within the Australia Pacific LNG lease holding. Review of available data reveals a select number of residential houses may be impacted should crossing augmentation occur. These are typically near GPF_HCK_01a in the upper catchment area. Again, further modelling of any proposed crossing augmentation designs would be undertaken to confirm no impacts on surrounding residences or infrastructure prior to those augmentation works proceeding.

5.7.5 Model EPP973 (Dulacca Creek)

All proposed infrastructure within the model boundaries of EPP973 are predicted to remain well clear of regional flood extents for all the ARI events modelled as part of this investigation.

GPF_BYM_03, WTF_BYM_01 and WTS_BYM_04 all are located at elevation and are some distance from the 100 year ARI flood extent. Similarly, WTS_BYM_03, whilst located within 150m of the predicted 100 year ARI flood extent, has approximately 10m of freeboard to the 100 year ARI flood level at that location.

Most proposed access roads within the modelling area are shown to be above the regional flood extent, with crossings of minor tributaries only (outside this regional flood model extent). The Warrego Highway crossing of Dulacca Creek at Dulacca is predicted to be immune for all events up to the 500 year ARI.

5.7.6 Model EPP692 (Dogwood Creek)

All proposed infrastructure site locations within the EPP692 area are at elevation and located well outside the regional flood extents of Dogwood Creek for all rainfall events modelled.

However, significant inundation of the Leichardt and Warrego Highways is predicted to occur. These major road corridors are expected access routes to not only infrastructure in the EPP692 tenement but also other tenement and infrastructure areas to the west and south.

Flooding of the Warrego Highway occurs for all rainfall events modelled at the crossing of Dogwood Creek at Miles. Flows in the order of 400m^3/s with the over road flood depths of approximately 150mm are predicted during the 10 year ARI event. Whilst this inundation depth is considered trafficable under Queensland Urban Drainage Manual (QUDM) guidelines, the hazards associated with a crossing of this nature during periods of flooding would likely render the crossing impassable. Inundation of the crossing increases to some 2.5m during the 100 year ARI event. Australia Pacific LNG may require an increase level of road immunity at this location due the access the crossing provides to facilities to the west. As such, significant infrastructure works would be required at this location to facilitate a crossing immunity of at least a 10 year ARI event. This would include bridge augmentation / replacement and associated earthworks within the vicinity of the bridge to improve flow regimes through the structure.

Similarly, the Leichardt Highway approximately 1200m to the south of Miles is shown to be inundated for most rainfall events modelled as part of this investigation. However, review of topographic and peak water surface levels reveal the road to be only slightly inundated at a few discrete locations during the 10 year ARI event. This may be due to the raised road embankment not being accurately represented in the topographic data and hence the hydraulic model. Without detailed road centreline survey, it is difficult to accurately determine the level of road immunity at this location. Works to improve immunity at this location would be limited to road raising and possible local drainage augmentation. There would not likely be any impacts as a result of minor road raising at this location.

The Leichardt Highway crossing of Columboola Creek is predicted to already have 10 year ARI immunity.

5.7.7 Model PL226 (Condamine River)

The majority of proposed infrastructure within the PL226 tenement other than GPF_CON_02b, WTF_CON_01 and WTS_TAL_00 are predicted to be flood free for all rainfall events up to the 500 year ARI, and therefore will have no impact on flood behaviour. This includes infrastructure items COMS TOWER ORANA, COMS TOWER ORANA2, COMS TOWER TALINGA (existing), GPF_CNS_03, GPF_CON_01b, GPF_CON_01c, GPF_OAN_04, GPF_ORA_03b, GPF_TAL_02b, TALINGA_BP, TALINGA_WTF, WTS_CON_02, WTS_CON_02b, WTS_CNS_03, WTS_OAN_01, WTS_OAN_02, WTS_ORA_01, and WTS_ORA_02.

GPF_CON_02b

The south-western corner of the proposed GPF_CON_02b site location is shown to cross the headwaters of an un-named minor ephemeral stream flowing south from the site into Condamine River approximately 3.5km away. Flooding at this location is from local catchment flow only. A slight re-alignment of the site location to the east by some 200m would enable this minor stream and flood behaviour at this location to remain its existing state for all storm durations without the need for channel works.

WTF_CON_01

The proposed WTF_CON_01 site lies on the same un-named minor ephemeral stream approximately 1200m downstream of GPF_CON_02b. Flooding at this location is from local catchment flow only. The site is located between two ridges and an ephemeral stream flows through the centre of the site, effectively bisecting the site into two parts. Options for this site include;

- Locating infrastructure on areas of higher elevation, outside of the 100 year ARI flood extents
- Relocation of the proposed infrastructure site either side of the stream (approximately 500m movement)

These options allow for the stream to remain in its natural undisturbed state.

WTS_TAL_00

WTS_TAL_00 is located on the eastern embankment of Borgrambilla Creek, some 4km upstream of the confluence of Borgrambilla Creek and Condamine River . The site is subject to regional flooding in extreme storm events. The flood study undertaken by WorleyParsons in August 2008 entitled 'Flood Investigation for Talinga Coal Seam Gas Development' shows that the site is marginally outside the 100 year ARI flood extent. Topographic information used in the flood study was the Airborne Laser Scan (ALS) data supplied by Origin Energy supplemented with a 25m DEM obtained from the Department of Environment and Resource Management.

However, the current flood model predicts that the WTS_TAL_00 site location will be inundated during the 100 year ARI event and above. The inundation of the pad location is attributed to different topographic data being used in the modelling. The current model utilised the Australia Pacific LNG supplied photogrammetric data recently collected by AAMHatch. A comparison of Digital Elevation Models (DEM) used in the current and previous studies revealed that the ground elevations of the flood plain at the vicinity of the WTS_TAL_00 based on the current photogrammetric data are generally 500mm to 1000mm lower than the previous study. Significant differences in surface elevations between the photogrammetric data collected for this study and the 25m DEM obtained from DERM are also evident immediately downstream of the site. These differences in surface elevations used in the model all contribute to the difference in model results.

To simulate possible impacts on regional flood levels on surrounding areas resulting from the construction of the WTS_TAL_00 pad, the pad location was filled above the 100 year ARI flood level in the hydraulic model and was simulated with the 100 year ARI storm event for both regional and local flood scenarios. Modelling results revealed that there is negligible afflux across tenement PL226 during the 100 year ARI event for both regional and local flood scenarios as a result of the filling of the pad. As such, modelling results predict the filling of the WTS_TAL_00 pad location will have negligible impact on peak flood levels within the Condamine River system and its flood plain.

Access roads preliminary flood assessment

Access to both GPF_CON_02b and WTF_CON_01 is by McLennans Road. The road crosses the previous discussed un-named ephemeral stream with a 10 year ARI flow of some 18m^3/s. Model results show that there are peak flood depths over the road of 0.3m or more for a 10 year ARI event. it is likely that achieving flood immunity for the 10 year ARI event would require the implementation of a combination of road raising and culvert augmentation.

Access to GPF_CNS_03 is by Elerslea Lane East. The road crosses an un-named ephemeral stream with a 10 year ARI flow of some 45m^3/s. Model results show that there are peak flood depths over the road of 1.5m or more for a 10 year ARI event. Flood immunity for the 10 year ARI event will likely require a combination of road raising and significant cross road drainage augmentation.

Access to GPF_CON_01b is by Kogan-Condamine Road. Modelling results predict that Kogan-Condamine Road and the GPF_CON_01b access road are flooded in various locations by the breakout of the adjacent un-named stream running parallel to Kogan-Condamine Road. Flows in the order of 26m^3/s and over road flood depths in the order of 400mm are predicted during the 10 year ARI event along the road. Infrastructure works would be required at various sections of the road to provide at least a 10 year ARI event flood immunity of the road. This would include road raising and possible local drainage augmentation.

Access to GPF_ORA_03b is by Chinchilla-Tara Road. The road intersects Wieambilla Creek at the top reach of the Creek. Modelling predicts that the Chinchilla-Tara Road and the GPF_ORA_03b access road crossing the creek are flooded in all design rainfall events analysed. Flows in the order of 16m^3/s with flood depths in the order of 400 – 500 mm are predicted during the 10 year ARI event along the road. Significant infrastructure works would be required at various sections of the road to provide at least a 10 year ARI event flood immunity of the Chinchilla-Tara Road and access road. This would include road raising and possible local drainage augmentation.

Access to GPF_OAN_04 is by Avenue Road. From the modelling results, road crossing of Avenue Road at Wambo Creek is flooded for all design rainfall events analysed. The predicted 10 year ARI peak flow for Wambo Creek at this location is 250m^3/s and the peak depth within the main channel is 3.5m. Significant infrastructure works would be required at this section of the road to provide at least a 10 year ARI event road crossing immunity. This would include road raising and possible bridge construction.

Although no road augmentations are proposed by Australia Pacific LNG at this stage, it is not envisaged augmentation of the majority of the aforementioned crossings would impact on residential properties.

However, the proposed infrastructure location of WTF_CON_01 immediately upstream of the McLellens Road crossing may be impacted if crossing augmentation was to occur. Similarly, there may be impacts on residential properties should crossings around the township of Condamine occur. This includes the Leichardt Highway and Condamine-Kogan Road crossings of the Condamine River. As a result, modelling of any proposed upgrades to these crossings would be undertaken to confirm no impacts on surrounding residences or infrastructure should these augmentation works proceed.

5.7.8 Model SEP692 (Kogan Creek)

All proposed infrastructure within the SEP692 model area is predicted to be flood free for all rainfall events modelled. Existing roads have been selected to provide access to proposed facilities. These roads are inundated at several locations by Kogan Creek and its tributaries.

Dalby Kogan Road crosses an unnamed tributary approximately 3 km southeast of the town of Kogan. The peak flow for the 10 year ARI event at this location is approximately 17m^3/s with flow depths along

the road alignment of typically less than 250mm. The length of inundation along the road is approximately 450m. Photography suggests that there are currently some small culverts in place at the road crossing. A combination of road raising and appropriately sized cross road drainage would be required to facilitate 10 year ARI immunity at this location.

The main branch of Kogan Creek passes the town of Kogan by flowing along the eastern side of the town before flowing under a bridge and continuing north. The main road through Kogan which crosses the bridge changes its name from Dalby Kogan Road in the east to Kogan Condamine Road in the west. In the 10 year ARI event there is broad inundation of the carriageway approximately 600m wide at this location whilst in the 100 year ARI and 500 year ARI events, the width of inundation of the road at this location is even more expansive at approximately 800m. For the 10 year ARI event, a total flow of approximately 130m^3/s crosses the road in the floodplain. Flood depths in the main channel are approximately 2m while depths over the road in the low lying floodplain areas are up to 1.7m.

The major constraint to achieving 10 year ARI flood immunity of the Dalby Kogan/Kogan Condamine Road is to ensure a non worsening of flood levels in the town upstream and downstream of the crossing. Any solution is likely to include a combination of road level raising and channel works along the main channel and the breakout path through town as well as appropriately sized cross road drainage. It may be found that road immunity of the state controlled road at this location is not achievable without impacts on flood levels within the Kogan township.

Kogan Condamine Road crosses another unnamed tributary of Kogan Creek approximately 4km northwest of the town of Kogan. For the 10 year ARI event, the flow across the road is approximately 11m^3/s with a depth of up to 300mm. Flood immunity for the 10 year ARI event could be achieved with a combination of road raising and cross road drainage augmentation.

Tara Kogan Road exits Kogan to the south and runs along the eastern side of Kogan Creek. This road is the likely access road for WTS_KIA_02 and GPF_KIA_01a. Model results for the 10 year ARI event show that the section of access road is dry but the flood extent is very close to the road at the crossing of a small tributary just upstream of its junction with Kogan Creek. For larger rainfall events, the road is inundated at this crossing.

5.7.9 Model EPP663 (Weir River and Western Creek)

The majority of proposed infrastructure within the EPP663 tenement is predicted to be flood free for all rainfall events up to the 500 year ARI, and therefore will have no impact on regional flood behaviour. This includes infrastructure items BP_GIL_01, BP_GIL_01, BP_GIL_01, WTS_GIL_01, WTF_GIL_01, WTS_WAA_02, GPF_WAA_04, GPF_ZIG_05, GPF_ZIG_06 and WTS_ZIG_03.

However, the location of GPF_WAA_03 is shown to intersect the flood extent of Tea Tree Gully for all ARI events modelled. Review of aerial photography and surrounding topography suggests the location of this infrastructure item could be moved in a northerly direction approximately 250m. This would align the edge of the facility with the adjacent access road and remove the southern boundary of the pad from the flood extents for all design rainfall events modelled in this investigation. This would eliminate any impacts on the natural flow regimes of Tea Tree Gully and eliminate the need for stream diversion or scour protection works. Alternatively it is recommended not to place infrastructure in the portion of the site shown to be inundated by the 100 year ARI flood extent.

Access roads to the various infrastructure pad locations are typically aligned along existing access tracks within the tenement. The proposed access road to GPF_GIL_02 and WTF_GIL_01a crosses the Weir River in the upper reaches of the catchment. Flows at this location are shown to be

approximately $45m^3/s$ with a depth of 2m during the 10 year ARI event. Flow widths are only in the order of 90m, and it is envisaged an adequately sized culvert structure and road embankment at this location would provide 10 year ARI immunity.

Similarly, crossings of an adjacent tributary to the Weir River are also proposed to allow access to WTF_GIL_01a and GPF_GIL_02. These crossings have a predicted flow rate of approximately $15m^3/s$ and $17m^3/s$. Flood extents are well defined within the main channel, and installation of appropriately sized culvert crossings at these locations would facilitate 10 year ARI immunity.

The proposed crossing of Tea Tree Gully and adjacent tributary near GPF_WAA_03 are predicted to have peak flows of approximately $12m^3/s$ and $5m^3/s$ respectively during the 10 year ARI event. 10 year ARI immunity of these crossings would be easily achieved with appropriately sized culvert crossings. As the access track to GPF_WAA_04 continues south, minor tributaries are crossed which are outside the regional flood modelling extent. It is envisaged flows at these crossings would be minor and 10 year ARI immunity would be achievable.

Finally, access to GPF_ZIG_05 crosses Paddy Creek and an adjacent tributary. Flows at these crossing locations during the 10 year ARI event are $45m^3/s$ and $5m^3/s$ respectively. Flows in Paddy Creek are well defined and up to 2.5m deep during the 10 year ARI event, whilst flows in the adjacent tributary are in the order of 0.5m deep. It is envisaged appropriately sized culvert structures at these locations would facilitate 10 year ARI road immunity.

Any impacts on peak flood levels from the upgrading of the creek crossings for access roads within EPP663 modelling area would remain well within the Australia Pacific LNG lease holding and are not considered likely to significantly impact on surrounding residents.

5.7.10 Cumulative impacts

Investigation into impacts on regional flooding behaviour has typically centred on any impacts localised around proposed infrastructure.

It is reasonable to assume that other similar projects that may occur near the Australia Pacific LNG project will adopt a similar position with respect to minimising impacts on flooding. That is, it is likely major infrastructure will also be located outside of the regional flood extents. Furthermore, without conceptual layouts for other projects, it is not possible to definitively predict cumulative impacts on flooding. Therefore, cumulative impacts from other developments have not been assessed as part of this study.

6. Site based stormwater management plans

6.1 Proposed base case facility information

There are two types of facilities associated with the gas field element of the Australia Pacific LNG Project. These two types are; Gas Processing Facilities and the Water Treatment Facilities. With the exception of the Talinga gas field, locations are subject to refinement during the design phase of the project. The locations of each site are detailed in Section 4.3. The generic gas processing and water treatment facilities are described below.

6.1.1 Water treatment facility

There are six new water treatment facilities proposed throughout the study area. These are of a generic design and will be modified as necessary to suit each site. The facilities comprise a Reverse Osmosis plant and a power plant (refer Figure 6.1). The footprint for the facilities is approximately 200m x 200m. Any stormwater management devices would be located within this footprint. The management of the brine ponds is not included within this stormwater management plan.

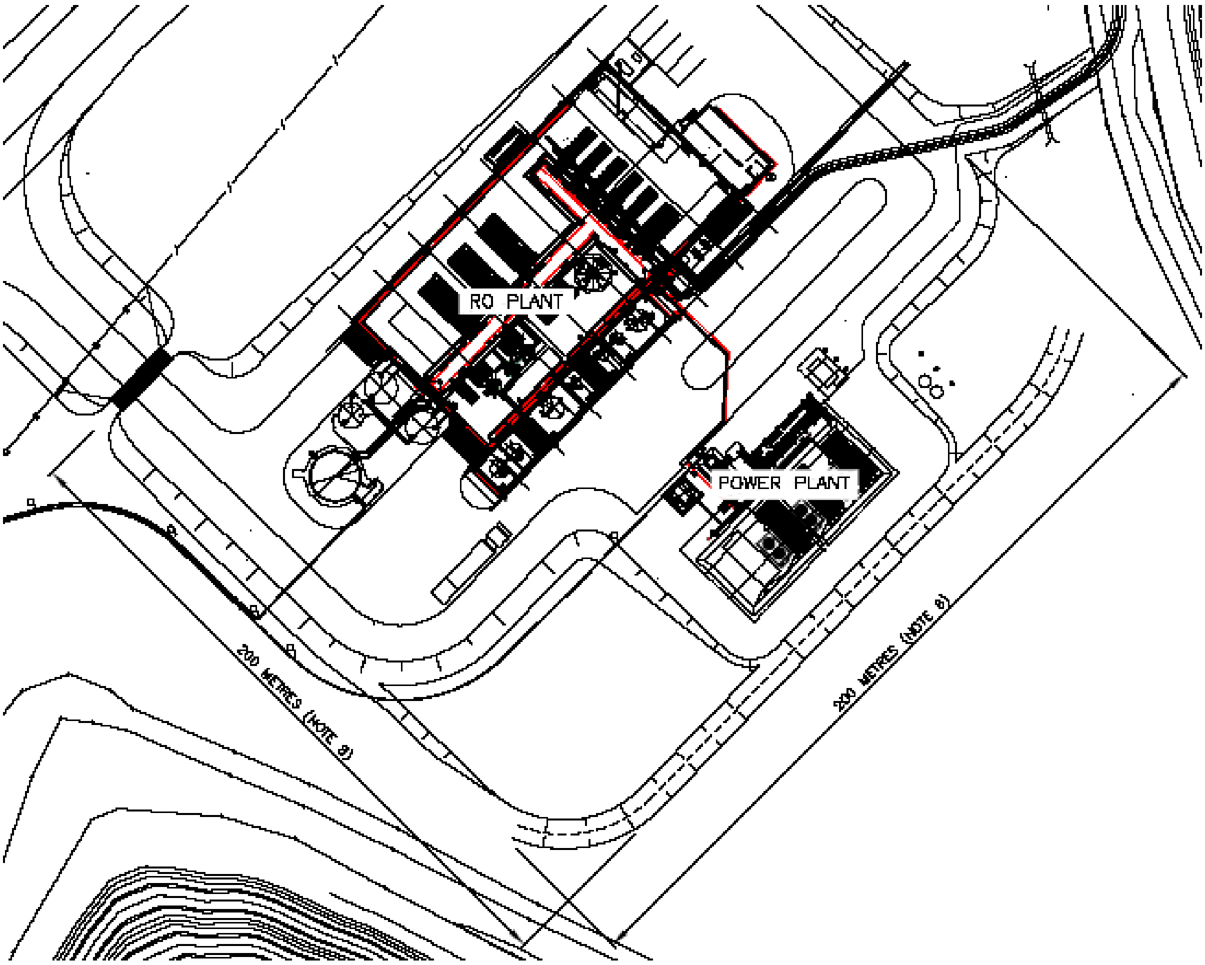


Figure 6.1 Water treatment facility concept layout (Source: Australia Pacific LNG)

6.1.2 Gas processing facility

Twenty three gas processing facilities are to be located throughout the gas fields excluding the Talinga area. They are of a generic design, being relatively uniform from site to site, although individually orientated to suit the site and associated infrastructure locations.

Figure 6.2 shows the generic site plan for the gas processing facilities. The footprint for these facilities is 1000m x 500m, comprising of the compressors, chemical shed, flare tower, work sheds and administration building. Any stormwater management devices would form part of this layout.

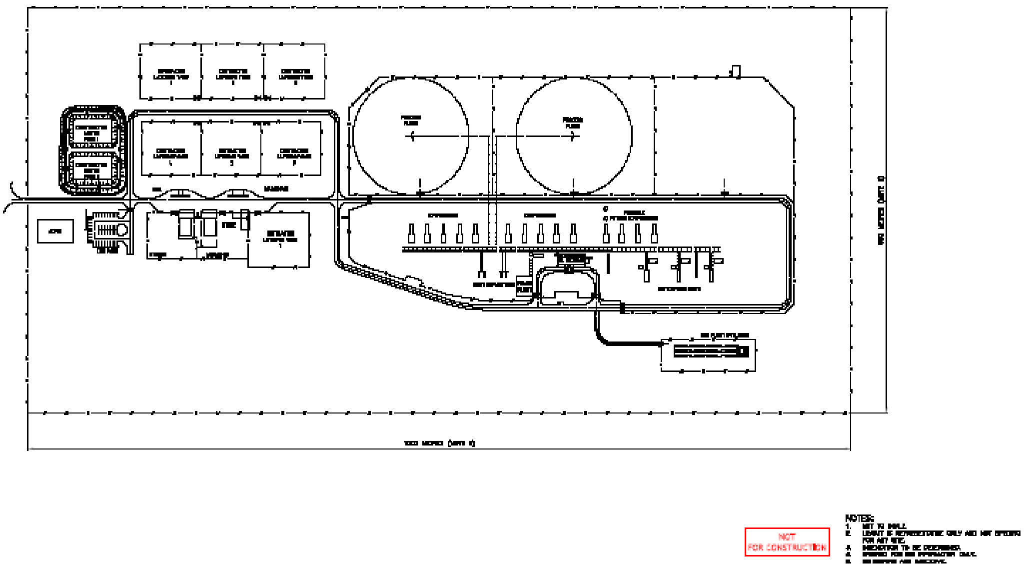


Figure 6.2 Gas processing facility concept layout (Source: Australia Pacific LNG)

6.2 Water quality

6.2.1 Potential pollutants

Due to the nature of the facility and the surrounding environment the likely stormwater contaminants generated from the various facilities include hydrocarbons and sediment. The impacts these pollutants may have on receiving waterways can include smothering of plants, reduction in light penetration and photosynthesis processes, and suffocation or poisoning of aquatic fauna. Nutrients are not expected to makeup a significant portion of pollution runoff from the facilities, however, they can be attached to sediments and therefore may be removed through the targeted removal of sediment.

The water treatment facilities also pose the risk of chemicals being released to the environment through accidental spillage at loading and storage areas. These areas will be bunded to contain spills of this nature, and will drain to on-site waste treatment areas. Therefore, these contaminants have not been included within the assessment of stormwater runoff.

6.2.2 Pollutant removal

In assessing the proposed generic layouts of the gas processing and water treatment facilities, and the expected pollutants generated from each facility, the following treatment train has been compiled:

- Swales – collecting water from the facility, the vegetated swales direct runoff towards the sedimentation basin assisting in the removal of particulate pollutants
- Sediment Basin – sized using the Brisbane City Council Guidelines (BCC 2001). The BCC Guidelines, while developed by Brisbane City Council, can be used to calculate sediment basin sizes throughout Queensland using the rainfall intensity and sediment dispersion characteristics of the subject area. Each facility will have a sediment basin capable of holding stormwater for the 1 year ARI, 24hr duration event. The basin will capture, treat and release all runoff from the facility. Sediments, with any pollutants attached, such as nutrients, will be captured within the basin, to be removed at a regular interval and disposed of at a suitable location. Additionally, residual hydrocarbons will be exposed to sunlight whilst in the sediment basin. Sunlight and micro-organisms break down hydrocarbon chains, removing the potential harm on the aquatic environment. The design of the sediment basins is outlined in Section 6.2.4.

6.2.3 Discharge targets

Table 6.1 lists the recommended targets for discharge from each stormwater basin. These targets have been based on Healthy Waterways WSUD Guidelines for South East Queensland (HW 2006). The Healthy Waterways WSUD Guidelines were chosen as no comparative guidelines exist for the region for load reduction. The Queensland Water Quality Guidelines refer to ANZECC/ARMCANZ 2000 *Australian and New Zealand Guidelines for Fresh and Marine Water Quality* for the study region in regards to concentrations of pollutants in the receiving waterways, however, limited information is available for load reduction targets within ANZECC/ARMCANZ (2000).

Table 6.1 Discharge water quality targets

Parameter	Target
Suspended solids	80% load reduction
Hydrocarbons	No visible sheen

Due to the nature of the facilities, it is not expected that nutrient generation from the site will increase significantly. Any nutrients exported from the site that are bound to the sediments will be removed from the system when the sediment basins are cleaned.

6.2.4 Sediment basin preliminary design

The soils at the various sites generally exhibit dispersive characteristics. Consequently, the conceptual design for sediment basins has been based on the Brisbane City Council's *Sediment Basin Design, Construction and Maintenance Guidelines* (2001), for Type D basins. The following parameters were adopted to size the sediment basins for the gas processing and water treatment facilities:

Settling Volume = 100 x ${}^{1yr}I_{24hr}$ x VRC x CA

Where: ${}^{1yr}I_{24hr}$ = 1 year ARI 24hr duration rainfall intensity at Wandoan = 2.75mm/hr

VRC = Volumetric Runoff Coefficient = 0.75

CA = Catchment Area = 4ha (water treatment facility) and 50ha (gas plant facility)

Wandoan has been selected as the reference rainfall gauge for all sites as it is the nearest rainfall gauge to the gas fields and is considered to be representative of the average rainfall characteristics in the region.

The required sediment basin settling volume at the gas processing facilities is 10,313.5m^3, while the water treatment facilities would require a settling volume of 412.5m^3. The BCC guidelines recommend a sediment storage volume of 50% of the settling volume be provided in addition to the settling volume yielding total volumes of 15,470m^3 and 620m^3 at the gas processing and water treatment facilities, respectively. The BCC guidelines recommend a minimum settling zone depth of 0.6m, a minimum length to width ratio of 3 and internal batter slopes of 1 in 4. The dimensions for the sediment basins based on the BCC guidelines are set out in Table 6.2.

Table 6.2 Sediment basin dimensions

Facility	Gas processing facility	Water treatment facility
Top length (m)	230.00	47.00
Top width (m)	80.00	19.00
Base length (m)	222.00	38.00
Base width (m)	72.00	10.00
Settling zone depth (m)	0.60	0.60
Sediment storage depth (m)	0.32	0.45
Spillway length (m)	15.00	7.50
Spillway depth (m)	0.60	0.30

The top length and width dimensions listed in Table 6.2 are measured at the level of the overflow spillways. The lengths and depths of the spillways have been calculated to discharge the 100 years ARI peak flow without freeboard.

The settled water is to be decanted from the basins two days after the runoff event. It may be necessary to add a flocculating agent, such as gypsum, to accelerate sedimentation should the sediment captured by the basin not settle sufficiently within the two days detention time.

The volumes of the settling zones plus the temporary detention storage above the spillway levels are greater than the detention storage volumes required to reduce the post-development peak flows to existing values. Therefore, peak flows discharged from the facilities are predicted to be less than existing as listed in Table 6.3, and thus not result in increased scour or erosion downstream.

Table 6.3 Peak flow mitigation through sediment basins

ARI (yrs)	Peak flow (m3/s)			
	Gas plant		Water treatment	
	Exist	Post-dev	Exist	Post-dev
100	1.523	1.519	9.754	8.106
50	1.365	1.347	8.643	6.824
20	1.212	1.155	7.185	5.433
10	1.047	0.983	6.074	4.319
5	0.922	0.849	5.241	3.477
2	0.718	0.644	3.991	2.179
1	0.518	0.432	2.741	0.605

6.2.5 Stormwater quality modelling

The performance of the proposed stormwater quality infrastructure has been estimated using the Model for Urban Stormwater Improvement Conceptualisation (MUSIC). MUSIC was developed by the Co-operative Research Centre for Catchment Hydrology (CRCCH) and simulates the hydrologic and water quality performance of stormwater systems at a range of temporal and spatial scales suitable for catchment areas from 1ha up to 100km^2 using timesteps of 6 minutes up to 1 day. The modelling is normally undertaken on a continuous simulation basis in order to simulate cumulative pollutant loadings and treatment.

MUSIC comprises a conceptual rainfall-runoff model that is coupled with a pollutant model to generate runoff and pollutant loads. The removal of pollutants through treatment devices is simulated using a range of device-dependent conceptual models.

The MUSIC modelling was undertaken using SILO Data Drill rainfall and evaporation data for Wandoan for the period 1989-98 as Wandoan is considered to be representative of conditions at the proposed facility sites. The period of 1989-98 is considered to be representative of average 10-year rainfall. The mean annual rainfall for the simulation period is 649mm and is similar to the long-term average rainfall at Wandoan of 619mm for the period of record from 1955 to 2008.

The estimation of stormwater runoff and quality for existing conditions was undertaken using parameters recommended in BCC guidelines (BCC, 2003) for industrial developments. The complete listing of parameters is included in Appendix L. Runoff and quality for existing conditions on the sites was estimated using BCC parameters for rural-residential landuse. The BCC Guidelines were used as there are no comparative industrial or rural-residential calculations for the study region. The MUSIC model inputs are summarised in Table 6.4.

Table 6.4 MUSIC model parameters

Parameter	Post-development	Existing
Impervious area (%)	50	0
TSS50 (log10mg/L) (storm / base)	1.92 / 0.78	2.26 / 0.53
TP50 (log10mg/L) (storm / base)	-0.59 / -1.11	-0.56 / -1.54
TN50 (log10mg/L) (storm / base)	0.25 / 0.14	0.32 / -0.52

The estimated stormwater runoff quality determined using MUSIC is summarised in Table 6.5.

Table 6.5 Stormwater runoff quality

Parameter	Pre-development	Post development			Comparison
	Existing	Generated	Discharged	%Reduction	(Existing-discharged)
Water treatment facility					
Runoff (ML/yr)	31.9	164	139	15.2	reduction
TSS (T/yr)	5.3	22.8	2.8	87.7	reduction
Gas processing facility					
Runoff (ML/yr)	2.6	13.1	12.1	7.6	increase
TSS (T/yr)	0.24	1.78	0.24	86.5	no change

The MUSIC modelling predicts that the swales and sediment basins will remove more than 85% of the suspended solids from the runoff from the proposed facilities yielding sediment export loads that are significantly lower than those for existing conditions for the GPFs and no change from existing conditions for the WTFs. Therefore, it is considered that the proposed facilities will not increase significantly the sediment loads in the natural drainage lines, and will not impact on the environmental values of the area.

The impervious areas within the facilities will increase the volume of runoff discharged. However, the impact of the predicted increased volume of runoff is expected to dissipate rapidly with increasing distance from the facilities as volume of runoff from the facilities represents very minor portions of the accumulated catchment runoff of the creek systems.

While MUSIC conceptualises the pollutant reduction of sediments, nutrients and flow, it does not calculate hydrocarbon removal. Hydrocarbons, such as oils and fuels, naturally break down through exposure to sunlight and some species of bacteria. It is expected that no to minimal hydrocarbons will form part of the stormwater pollution, given that containment bunds will be established surrounding any storage or unloading areas within the facilities. However, should any hydrocarbons enter the stormwater from the site, the sedimentation basin provides exposure to sunlight where breakdown of molecules will occur. Visual inspections of the sediment basins for hydrocarbon sheens should be made prior to stormwater release.

6.3 Risk assessment

Each water treatment and gas processing facility impose changes to the landscape and land use at each site, and will introduce new avenues for potential contamination of the surrounding environment that would not otherwise exist for the area. As such, these potential risks need to be identified and methods for mitigation incorporated into the development. Table 6.6 lists identified risks, potential impacts and mitigation techniques for the water treatment and gas processing facilities.

Table 6.6 Risks, impacts and mitigation techniques

Risk	Impact	Mitigation
Increase in erosion and/or sediments entering receiving waterways	Sedimentation, smothering/loss of habitat	Stormwater capture and treatment, removal of sediments from sedimentation basins in accordance with guidelines/standards, eg. Institute of Engineers Australia – Erosions and Sediment Control Guidelines (1996)
Contaminants (chemicals or hydrocarbons) from facility operations entering waterways	Reduction in aquatic flora and/or fauna from chemical or hydrocarbon toxicity	Bunding around chemical storage/loading areas, with drainage to go to trade waste treatment
Increase in runoff from impervious areas	Scour increase, terrestrial erosion, smothering of aquatic flora and fauna, loss of aquatic habitat	Detain water for slow release Divert up-slope runoff around facility, to avoid cross-contamination of clean stormwater

6.4 Construction issues

Appropriate erosion and sediment control measures should be in place during the construction phase as well as during normal operations for each gas field infrastructure.

Erosion and sediment control measures may include, but are not limited to the following:

- Sediment fences
- Limited excavation and earth movement during windy conditions
- Runoff from exposed areas directed to sediment ponds
- Keeping the area of exposed soil to a minimum as necessary for construction and operational purposes
- Early stage rehabilitation with grasses and mulches
- Keeping exposed soil moist to reduce wind drift.

Further information regarding erosion and sediment control is reported in Volume 5, Attachment 6.

The sediment basins are to be constructed during the site preparation stage for each facility and will receive and treat runoff during the construction and operational phases of each facility. The sediment basins have been designed to remove more than 85% of the suspended solids from the runoff from the proposed facilities yielding sediment export loads that are significantly lower than those for existing conditions. The overflow spillways have been designed to pass the 100 years ARI design storm runoff from the facilities without overtopping of the embankments.

The drainage swales to collect and convey runoff from each site to the sediment basins will be designed to convey runoff for the 2 years ARI design storm. Runoff in greater storm events will flow as shallow overland flow, mimicking natural runoff behaviour and minimising erosion.

6.5 Maintenance of structures schedule

Maintenance of the stormwater management structures will take place to ensure that water quality and quantity leaving the facility does not become contaminated or uncontrolled. Table 6.7 outlines the recommended maintenance schedule.

Table 6.7 Maintenance schedule

Structure	Potential issues	maintenance	Monitoring frequency
Drains and swales	Subsidence, erosion, weeds, litter, sediment build-up	Remove litter and weeds Repair subsidence / erosion areas and reinforce Remove built-up sediment	Weekly and after significant storm events, as part of general site inspections
Sediment basin	Structural damage, erosion or leaks	Repair at first indication, if extensive structural repair is required lower water level within the ponds prior to repair works	Weekly and after significant storm events, as part of general site inspections
Erosion and sediment control devices	Erosion, sediment build up, and sedimentation	Remove litter Remove sediment build up	Weekly and after significant storm events during operation

Structure	Potential issues	maintenance	Monitoring frequency
		Dismantle damaged sediment fences, and reinstate	

6.6 Stormwater management

6.6.1 Operation and construction phases

The following information is provided to identify controls and procedures as part of an overall Environmental Management Plan in order to minimise impacts on water quality in receiving waters.

6.6.2 Potential impacts

Addition of sediments, litter, nutrients, hydrocarbons and other pollutants to stormwater runoff may decrease environmental values of downstream receiving environments. These impacts may be minimised by implementation of stormwater management measures outlined in Section 6.6.4.

6.6.3 Objectives

The key objectives for water quality are:

- To ensure that on-site operations are carried out by such practicable means necessary to minimise the contact of incidental rainfall and stormwater runoff with wastes or other contaminants
- To ensure stormwater does not adversely impact upon the aesthetic or environmental values of receiving waters.

Secondary objectives are:

- To minimise the stormwater ingress onto the site
- To minimise the discharge of contaminated stormwater into the receiving environment
- To restrict soil erosion and mobilisation of sediments and contaminants off-site.

6.6.4 Stormwater management actions

- Access to the facility shall be limited to authorised vehicles through a controlled access point
- Transport loads shall be covered to prevent entry of pollutants to the stormwater system
- Any spillage of wastes, contaminants or other materials shall be cleaned up as quickly as practicable using procedures that prevent contaminants or material being transferred to the stormwater drainage system
- Chemical storage and handling areas shall be bunded and shall have drainage lines separate from the stormwater drainage, to reduce the likelihood of chemical contamination of stormwater
- Erosion and sediment controls shall be designed using the Engineers Australia, Queensland Division, “Soil Erosion and Sediment Control Engineering Guidelines for Queensland Construction Sites (Sections A5 – A6)” (IEA 1996)

- The stormwater system for the site shall be inspected regularly to identify any failures and, if necessary, repairs shall be undertaken
- Trapped sediment shall be removed from drainage control and stormwater system and relocated to a stabilised stockpile, either onsite or offsite, or taken to an appropriate land fill area.

6.6.5 Monitoring and maintenance frequency/schedule

Monitoring and maintenance of stormwater management structures will take place, as outlined in Table 6.7 of Section 6.5, to ensure that water quality and quantity leaving the facility does not become contaminated or uncontrolled.

The monitoring and recording of the performance of the drainage control devices, together with details of the rainfall and stormwater discharge will be undertaken following any rainfall event exceeding 25mm in 24hrs.

6.6.6 Responsibility

- The Contractor will be responsible for monitoring the performance of all stormwater management structures during construction of the facility
- The Contractor will be responsible for reporting any failures of stormwater management structures to the Project Manager or other designate of Australia Pacific LNG
- Australia Pacific LNG will be responsible for stormwater management structures inspections
- Australia Pacific LNG Site Manager will be responsible for monitoring, reporting and corrective action regarding stormwater management structures throughout the operation period.

6.6.7 Reporting

- Stormwater management structures will be maintained, with a record kept of failures and repairs for each device
- The results of the inspection of stormwater management structures are to be reported to the Senior Manager for the facility on a regular basis
- The results of the inspection of stormwater discharge following rain events will be communicated as soon as practicable to the Senior Manager
- A register will be maintained of the reports and will be kept available for inspection by relevant regulatory agencies.

6.6.8 Corrective actions

- Any failures in the stormwater management devices during major rainfall events will be immediately repaired to prevent uncontrolled discharge, erosion or scour
- Any failures after significant rain events shall necessitate a review of the design and/or replacement of stormwater management devices. The change in devices will be the responsibility of the Australia Pacific LNG Facility Manager or their delegate

- In the event of a failure in the stormwater system, appropriate remedial work to restore any disturbed areas will be the responsibility of the Australia Pacific LNG Facility Manager or their delegate
- In the event of a failure of stormwater management devices, a review will be conducted to assess the efficiency of the EMP and identify strategies to improve stormwater management.

7. Hydraulic stream flow impact assessment

7.1 Purpose

The process by which CSG is extracted from the coal seam requires the depressurisation of the seam via the extraction of water. This water is known as associated water and is defined by the EIS Terms of Reference as "'*Underground water taken by a petroleum tenure holder from a gas well*". This water is typically of varying quality but generally is unsuitable for direct use for potable, livestock or irrigation.

The management of CSG associated water is a major issue for the Australia Pacific LNG project. The Queensland Government (Queensland Government, 2009) has identified that the volumes of associated water that are expected to be generated by the CSG industry pose a significant challenge for the industry. However, there are significant opportunities for beneficial use of CSG associated water that include water supply for new biodiesel fuel crops and the supplementation of existing irrigation water allocations and/or environmental flows via aquifer injection or discharges to watercourses. The Adaptive Associated Water Management Plan (Volume 5, Attachment 12) for the project includes, as a preferred short to medium term option, the disposal of treated associated water through discharges to watercourses across the study area.

The purpose of the assessment in this Section 7 is to assess the potential impacts that discharge of associated water may have on the hydraulic characteristics of the watercourses proposed as discharge locations. In addition, the existing flow regimes are characterised at each location and recommendations as to potential discharge regimes developed.

7.2 Project context

7.2.1 Associated water management strategy

The finalisation of management approaches to associated water is challenging due to the complex interactions of a number of influencing factors. These factors include:

- An evolving legislative and policy regime
- Variable water quality and volumes across different locations and difficulties in estimating volumes in advance of production
- Supply contract negotiation uncertainties in relation to potential users of associated water
- The need for flexibility in terms of the development sequencing and timeframes for individual gas well locations due to difficulties in estimating gas volumes in advance of development of particular sites.

Taking into consideration the above complexity, Australia Pacific LNG has adopted an Adaptive Associated Water Management Plan. One option for the management of associated water is discharge to water courses.

The Adaptive Associated Water Management Plan currently involves the discharge of associated water, following treatment via reverse osmosis, to a watercourse located in close proximity to the proposed water treatment facilities (WTF) or the discharge of associated water following aggregation into the Condamine River at one of two central discharge locations. One of the proposed locations for discharge of aggregated flows into the Condamine River is from the existing Origin operated Talinga

Water Treatment Facility, the second is within a 4.5 km reach of the river adjacent to the proposed water treatment facility known as WTF_CON_01.

Discharges from the Origin operated Talinga Water Treatment Facility have been investigated previously (EECO Consulting 2008; EECO Consulting, 2009) and application is currently being made for a 35 ML/d discharge to the Condamine River, which will accommodate the approved Talinga 90TJ/d development. This EIS study assesses the maximum discharge that the Condamine River at this discharge point can accommodate. A summary of previous study findings is provided below and the previous investigations should be referred to for further detail.

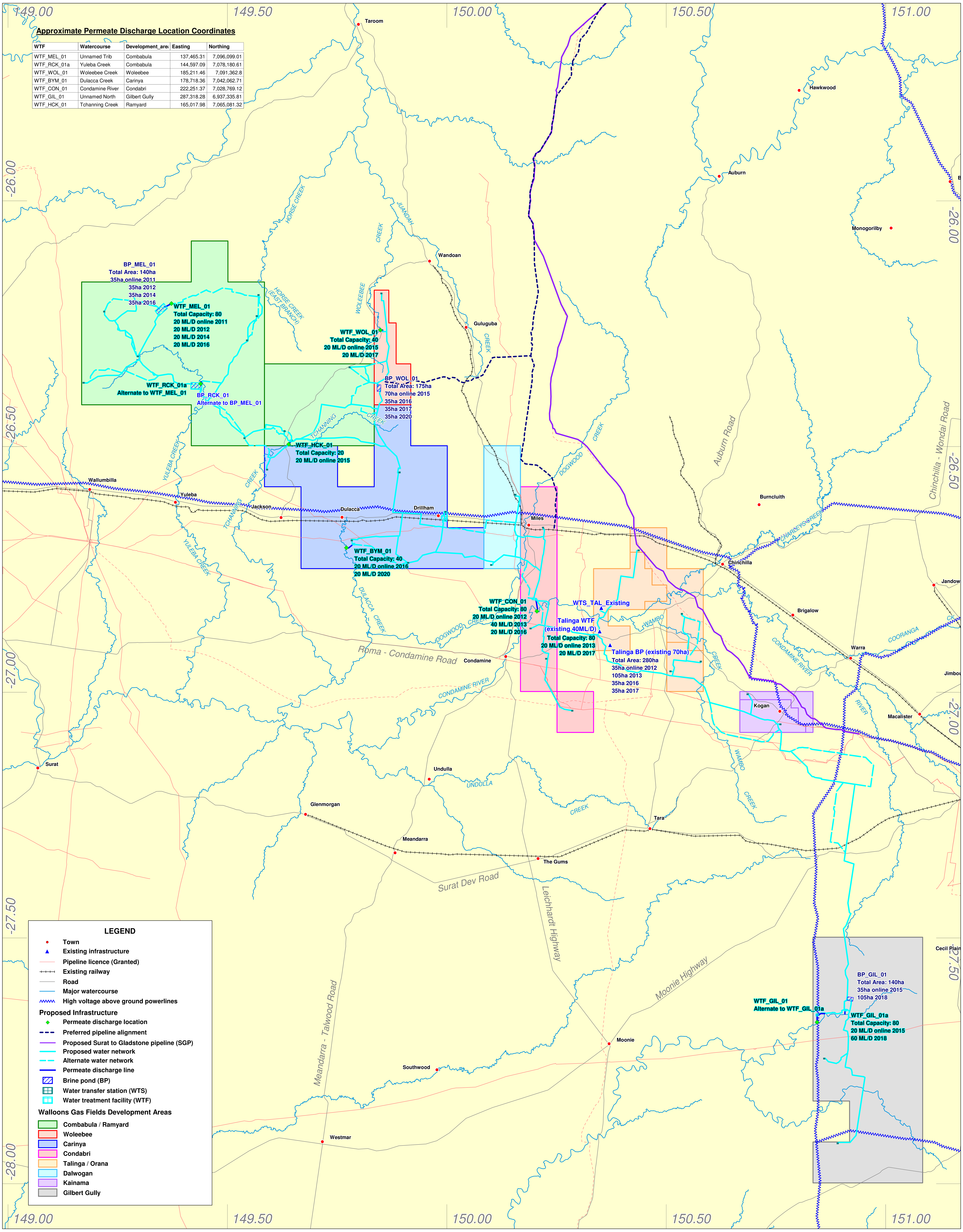
At this stage exact discharge locations have not been selected, however approximate discharge locations are provided in Table 7.1 and shown in Figure 7.1. Two alternative discharge scenarios are currently proposed; either discharge from a location in the vicinity of each of the proposed WTF or aggregation of flows and discharge to the Condamine River.

Table 7.1 Proposed discharge locations

WTF	Development area	Watercourse proposed for discharge
WTF_MEL_01	Combabula	Unnamed Tributary of Kangaroo Creek
WTF_RCK_01a	Combabula	Yuleba Creek
WTF_WOL_01	Wooleebee	Woleebee Creek
WTF_BYM_01	Carinya	Dulacca Creek
WTF_CON_01	Condabri	Condamine River
WTF_GIL_01	Gilbert Gully	Unnamed North Tributary of Weir River
WTF_HCK_01	Ramyard	Tchanning Creek
Combined Flows	All	Condamine River

Approximate Permeate Discharge Location Coordinates

WTF	Watercourse	Development_area	Easting	Northing
WTF_MEL_01	Unnamed Trib	Combabula	137,465.31	7,096,099.01
WTF_RCK_01a	Yuleba Creek	Combabula	144,597.09	7,078,180.61
WTF_WOL_01	Woleebee Creek	Woleebee	185,211.46	7,091,362.8
WTF_BYM_01	Dulacca Creek	Carinya	178,718.36	7,042,062.71
WTF_CON_01	Condamine River	Condabri	222,251.37	7,028,769.12
WTF_GIL_01	Unnamed North	Gilbert Gully	287,318.28	6,937,335.81
WTF_HCK_01	Tchanning Creek	Ramyard	165,017.98	7,065,081.32



SGP pipeline route digitised from Initial Advice Statement dated December 2008

0 12.5 25km

N W E S

Latitude - Longitude
Geocentric Datum of Australia 1994

Rev	Date	Revision Description	ORIG	CHK	ENG	APPD
0	15/12/2009	Issued for use	JM	DH		

WorleyParsons
resources & energy

AUSTRALIA PACIFIC LNG

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Figure 7.1 - Proposed Discharge Locations

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-2228 | Rev: 0

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7.2.2 Associated water production profile

The water profile developed for the EIS investigation indicates a maximum associated water production peak at around 170 ML/day. This is predicted to occur within the first 20 years.

Figure 7.2 below provide a summary of the water profile for the Walloons development, by area. Greater detail is provided in the Adaptive Associated Water Management Plan (Volume 5, Attachment 12)

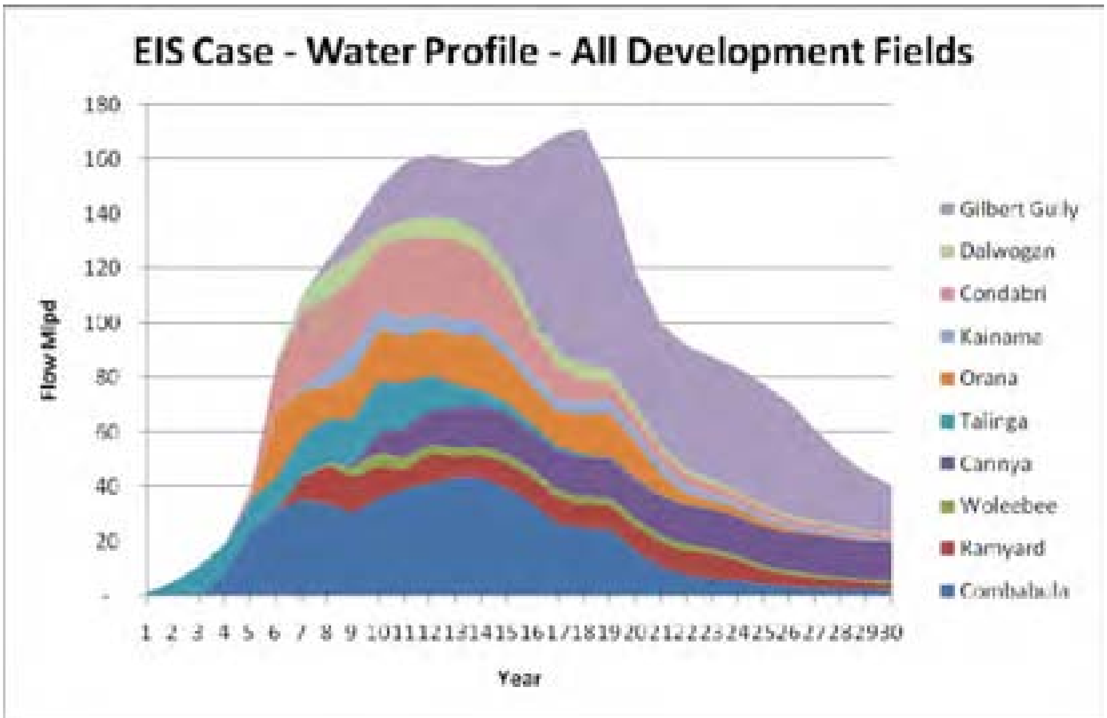


Figure 7.2 Predicted CSG Water Profile for the Walloons development

7.3 Scope of works for hydraulic stream flow assessment

The objectives of this section have been developed to inform the preparation of the Associated Water Management Plan for the Project and to meet the requirements of the Terms of Reference (particularly section 3.4 of the TOR) for the EIS. The objectives of the assessment have been to:

- Characterise the existing flow regimes in the watercourses identified as potential discharge locations
- Assess possible impacts to the existing flow regimes due to the discharge of treated associated water
- Determine the hydraulic characteristics of the watercourses and to assess the capacity of the systems to receive additional flows
- Make a qualitative assessment of influence and/or impacts of discharges on downstream users including the environment
- Provide recommendations as to possible discharges regimes for each watercourse.

As the associated water is planned to be treated by Reverse Osmosis (RO), salinity modelling that may be required with untreated CSG water has not be considered necessary as part of this assessment. Disposal of brine produced from the RO process is discussed in the Adaptive Associated Water Management Plan Volume 5, Attachment 12.

Geomorphology, water quality and aquatic ecological aspects of the proposed discharge regime are addressed separately in the technical report entitled "Aquatic ecology, water quality and geomorphology impact assessment – gas fields (Hydrobiology, 2009)" (Volume 5, Attachment 20).

7.4 Investigation limitations, assumptions and data

7.4.1 Climatic

The primary source of climatic data used within this study was obtained from the former DNRW SILO website. This service provides access to interpolated climatic data sets throughout Australia based on all available monitoring records from government sources (Jeffery et al., 2001, and www.longpaddock.qld.gov.au/silo/).

Records were requested for the locations noted in Table 7.2. These locations represent the discharge points within each catchment. These locations are also shown in Figure 7.1.

Table 7.2 SILO rainfall data locations sourced for hydraulic impact assessment

Location	Easting	Northing
Combabula/Ramyard	149.35	-26.20
Combabula/Ramyard (alt)	149.45	-26.35
Carinya	149.65	-26.50
Woleebee	149.85	-26.25
Condabri	149.80	-26.70
Gilbert Gully	150.85	-27.65

Rainfall records were also collected for two monitoring locations as shown in Table 7.3 and Figure 7.1.

Table 7.3 Rainfall record locations sourced for hydraulic impact assessment

Location	Gauge ref	Data period
Yuleba Forestry Station	043044	1995 to 2009
Wandoan	035041	1995 to 2009

A comparison of the SILO sourced rainfall data and the recorded rainfall data at Yuleba Forestry Station and Wandoan Station is provided in Figure 7.3. It can be seen that the rainfall series sourced through SILO generally has lower values than those recorded at both Yuleba Forestry Station and Wandoan. Therefore, when used for the purposes of this study, that is, to produce flow estimates at the discharge locations, the SILO data will produce slightly conservative estimates of flow regimes at the discharge locations.

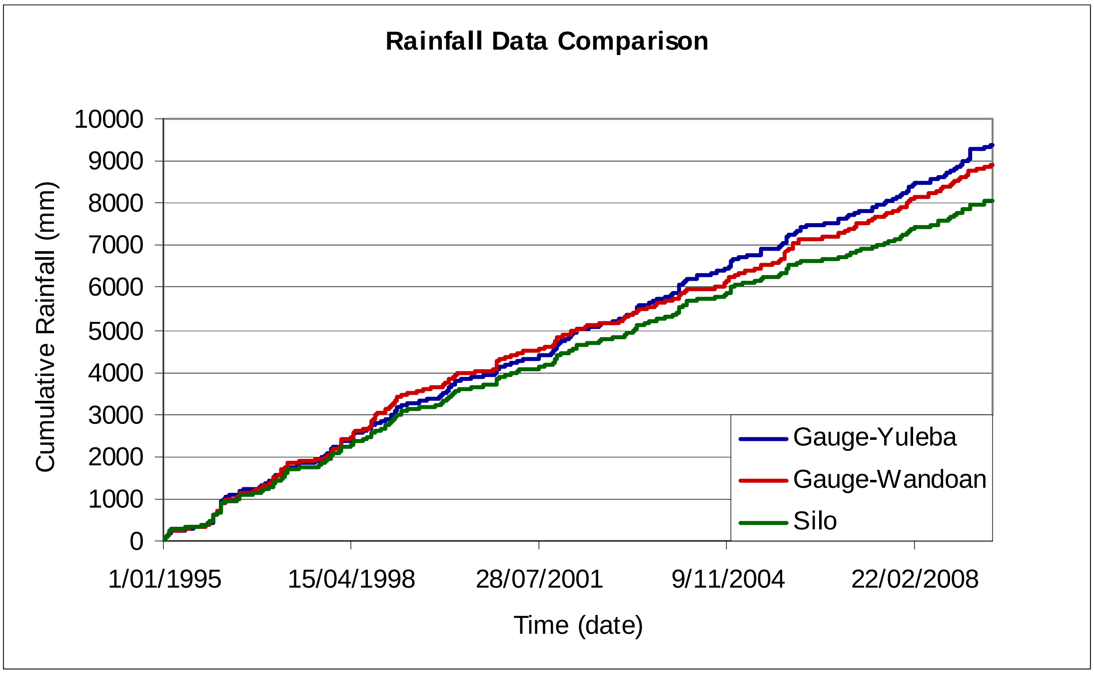


Figure 7.3 Rainfall data comparison

SILO, through the process of interpolation, provides the most suitable data set for use at locations any significant distance from a rainfall gauging station. There is likely to be localised variation in the spatial rainfall patterns however without an extensive monitoring network these spatial variations cannot be predicted with any degree of certainty.

Where rainfall gauge data, available for use in the SILO interpolation process, is widely spaced and of limited quality there is a risk that the SILO data may not accurately reflect the actual rainfall observed within the area. In this case, analysis of recorded streamflow data may indicate flow in watercourses were no rainfall is indicated in the SILO data set or vise versa. This is challenging for the calibration of a rainfall runoff model but can usually be partly overcome with focused interpretation of calibration results.

On the other hand, where a SILO data set is extracted within an area that has a relatively dense rainfall gauge network then the SILO interpolation process is likely to average the effects of any localised rainfall spikes and provide a better average representation of the rainfall that occurred within a catchment area.

There is likely to be significantly less spatial variation in evaporation rates however, a similar argument holds as provided above for the use of evaporation data from the SILO database in preference to that of recorded evaporation data. In addition, recorded evaporation data is generally less readily available than rainfall data and therefore long term averaging of data is necessary to produce a data set of equal length as the rainfall records available.

Therefore, SILO climatic data was selected for use in the prediction of runoff and streamflow at all proposed discharge locations where gauged streamflow data was not available.

The evaporation data used in this investigation was similarly obtained from the DNRW SILO dataset at the same locations as stated in Table 7.2.

7.4.2 Streamflow records

Streamflow records for gauges located in the vicinity of the proposed discharge locations were sourced through DERM and are listed in Table 7.4.

Table 7.4 Streamflow records

Location	Gauge ref	AMTD (km)	Catchment area (km2)	Data availability
Balonne River @ St. George	422201E	227.2	75,370	1971 to 2009
Balonne River @ Surat	422220A	405.9	47,251	2004 to 2009
Balonne River @ Weribone	422213A	357.8	51,540	1969 to 2009
Condamine River @ Bedarra	422344A	659.0	24,344	2007 to 2009
Condamine River @ Chinchilla	422308C	696.7	19,190	1955 to 2009
Condamine River @ Cotswold	422325A	537.5	28,930	1966 to 2009
Yuleba Creek @ Forestry Station	422219A	34.4	1,475	1972 to 2009
Weir River @ Gunn Bridge	416204A	237.9	4,423	1999 to 2009

Generally, the streamflow series were identified to have significant periods of questionable data quality and/or data gaps. This is generally due to equipment failure and is unavoidable. Periods identified as questionable were taken into account in the interpretation of results based on this data.

7.4.3 Survey data

Photogrammetric data as detailed in Section 3.1 was used in the creation of local hydraulic models for use in the discharge assessment.

7.4.4 Field investigations

A series of field investigations were undertaken as part of the scope of works for this hydraulic impact assessment. In addition, field observations and ground truthing information has been sourced from "Aquatic ecology, water quality and geomorphology impact assessment – gas fields (Hydrobiology, 2009)" (Volume 5, Attachment 20). This information was used in the construction of hydraulic models and to inform conclusions drawn from modelling results.

Due to land access issues during the field program the following discharge locations were not visited:

- WTF_MEL_01 – Unnamed tributary of Kangaroo Creek
- WTF_BYM_01 – Dulacca Creek.

7.4.5 Previous investigations

The following investigations have been carried out on behalf of Origin Energy and contain information relevant to this assessment:

- Preliminary Discharge Assessment, Walloons Coal Seam Gas Development, EECO Consulting et al, January 2008

- Talinga Development Project, Reverse Osmosis Permeate Discharge Assessment, EECO Consulting, January 2009.

These reports detail analysis undertaken for the existing Origin operated Talinga development area and a proposed discharge of treated associated water of 35 ML/d to the Condamine River. The key findings from these investigations are as follow:

- An initial field program to identify potential discharge locations determined that the most preferable discharge locations would be the Chinchilla Weir ponded area and, in closer proximity to the Talinga development area, a discharge location adjacent to the confluence with Wieambilla Creek. This discharge location is approximately 37 km downstream of the Chinchilla Weir
- The later study concluded that the Condamine River and the two tributaries (Sandy and Wieambilla Creeks) had adequate capacity to accept 35 ML/d of discharge, however it was recommended that hydraulically and based on sediment transport considerations the Condamine River discharge location was preferred over the Sandy Creek locations
- In order to minimise the likelihood of any negative impact associated with the discharge it was recommended that an Environmental Monitoring Plan be drafted including controlled inspections and surveys at key cross section locations to identify bed and bank stability issues, monitoring of water quality, water levels and flow, and a “Before/After Control/Impact (BACI) type experimental monitoring plan to assess the biological response to the discharge.” (EECO, 2009).

As part of the Associated Water Management Strategy discharges from the Talinga discharge location may be increased. As these previous investigations address the characterisation of flow regimes and hydraulic capacity of this discharge location no further assessment is undertaken as part of this study.

7.4.6 Limitations

The volumes of associated water estimated prior to production are inherently unreliable, with high levels of uncertainty regarding both the:

- Rate of produced water released from wells, to provide optimum gas production
- Size of the gas reservoir, resulting in changes in the staging of field development.

The quantity, timing and location of water demand is similarly anticipated to fluctuate, both in terms of the life of the project and seasonal conditions. The Adaptive Associated Water Management Plan therefore needs to be adaptive to reflect changing supply and demand conditions, whilst remaining sustainable including social, economic and environmental considerations.

This investigation was undertaken to support the ongoing development of the Adaptive Associated Water Management Plan and is based on information provided in the Adaptive Associated Water Management Plan at the time or writing.

The results of this investigation are considered sufficient to identify the likely impacts of the proposal to discharge associated waters to the selected watercourses and provide an initial assessment of the likely significance of those impacts. The intent of this investigation is to provide information to aid in the development of the associated water management strategy and to provide recommendations on further areas of investigation, necessary to enable detailed design work.

The analysis conducted within this investigation were constrained by the following factors:

- Level of project information – The conceptual level of development of the Associated Water Management Strategy and the level of information that was available regarding the rate and timing of water production
- Available survey detail – hydraulic models were constructed using survey based on photogrammetry and not detailed ground survey. Further discussion is provided in Section 3.1.
- Land access constraints – field reconnaissance was limited by issues encountered with land access during the field program. Some proposed discharge locations were not visited during this investigation and thus parameters such as watercourse roughness and vegetation density, were inferred from previous investigations, literature and aerial photography.

This investigation does not include any aspects associated with the quality of the discharges or the potential impacts of discharges on the water quality or aquatic ecology of the watercourses. For discussion of these aspects refer to the technical report entitled “Aquatic ecology, water quality and geomorphology impact assessment – gas fields (Hydrobiology, 2009)” (Volume 5, Attachment 20)

7.5 Analysis of flow characteristics

An analysis of flow characteristics was undertaken for each of the proposed discharge locations. This analysis consisted of two parts:

- Flow Exceedance Analysis – undertaken to determine the range of daily flows likely to occur in the watercourse and the statistical likelihood of these flows occurring in a given period
- Seasonal Flow Variation – undertaken to determine the flow pattern likely to occur in the watercourse over a year.

7.5.1 Regional perspective

The CSIRO as part of the Murray-Darling Basin Sustainable Yields Project completed a thorough investigation into the water availability in the Condamine-Balonne Region. The findings of this investigation are reported in the Water Availability in the Condamine-Balonne report released in June 2008.

The CSIRO study included extensive rainfall-runoff modelling over the region and have reported that “the annual rainfall and modelled runoff averaged over the Condamine-Balonne region are 514 mm and 19 mm respectively. Most of the rainfall occurs in the summer half of the year and runoff is highest in summer and early autumn. The region covers about 12.8 percent of the MDB and contributes about 8.5 percent of the total runoff in the MDB.” (CSIRO, 2008)

In the areas identified as possible discharge locations for the project the mean rainfall is between 500mm and 800mm and the mean modelled runoff is between 40mm and 10 mm (CSIRO, 2008). The variation in mean rainfall and runoff is shown in Figure 7.4, taken from the CSIRO report.

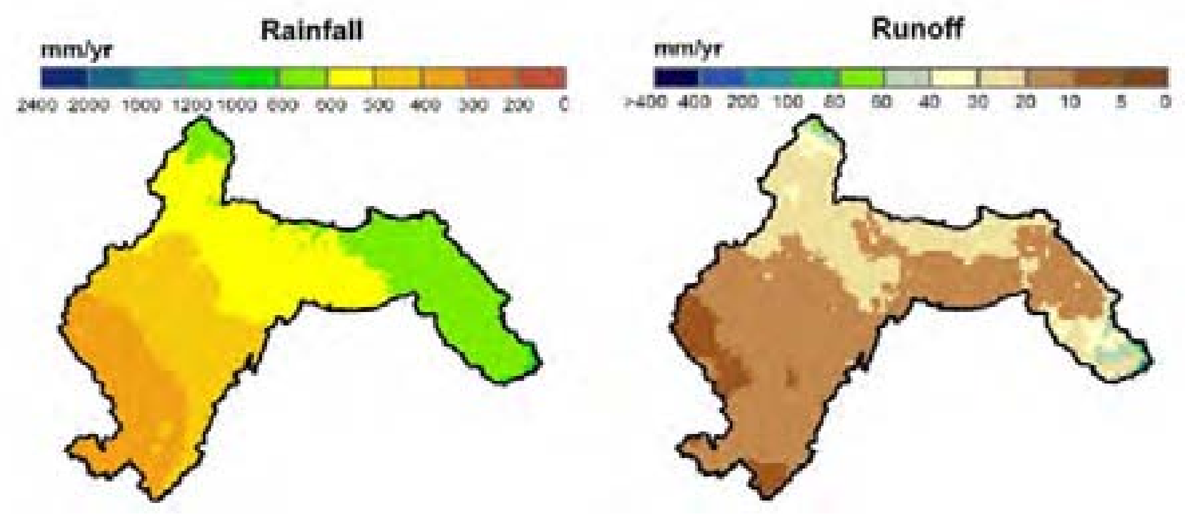


Figure 7.4 Condamine Balonne spatial distribution of mean annual rainfall and modelled runoff averaged over 1895-2006

(Source: CSIRO, 2008, Water availability in the Condamine-Balonne, p. 35)

It is also concluded in the CSIRO report that the rainfall and runoff over the ten-year period 1997 to 2006 are not statistically different to the 1895 to 1996 average values (CSIRO, 2008, p. 31).

Climate change projections are noted in the CSIRO report as indicating that the future runoff in the region is likely to decrease rather than increase, with the best median estimates showing a 9 percent reduction in the average annual runoff at 2030.

7.5.2 Flow exceedance analysis methodology

The study locality covers an area stretching from west of Millmerran through to east of Roma and Injune. As part of this study, discharge locations have been considered within the following watercourses:

- Condamine River
- Yuleba Creek
- Unnamed Creek (tributary of Kangaroo Creek)
- Tchanning Creek
- Woleebee Creek
- Dulacca Creek
- Unnamed Creek North (tributary of Weir River).

Historical streamflow monitoring records are limited to sites located along the Condamine River, one site located on Yuleba Creek and one site within Weir River at Gunns Bridge. Thus the majority of the proposed discharge sites are located in ungauged catchment. As such two distinct methodologies were adopted for the flow exceedance analysis.

a) For the proposed Condamine River discharges (a gauged catchment) a statistical analysis of the historic stream records at the DERM stream gauges upstream and downstream of the proposed discharge locations (422308C Condamine River at Chinchilla and 422325A Condamine River at

Cotswold, respectively) was undertaken. This analysis produced a likely range of flow exceedance curves from which generalised conclusions of flow pattern were drawn

b) For all proposed tributary discharge locations (ungauged catchments) a synthetic stream flow record was produced via use of a rainfall-runoff model calibrated to historic streamflow records. For all locations, other than the Gilbert Gully (Weir River) discharge location, the gauge records on Yuleba Creek at Forestry (422219A) where used as the basis of the calibration. For the Gilbert Gully (Weir River) location a calibration was undertaken using the Weir River at Gunns Bridge (417205A) streamflow records. This synthetic streamflow record was used to produce daily flow exceedance curves for the discharge locations.

7.5.3 Condamine River

The proposed discharge location within the Condamine River is located between the DERM streamflow gauge at Chinchilla (422308C) and the gauge at Cotswold (422325A), upstream and downstream, respectively. The Condamine River system is a highly modified and controlled system and thus the flow regime observed within the river is likely to have changed significantly over time. In order to highlight the potential changes in the observed flow regimes in this reach of the Condamine River the historic streamflow data was analysed over a series of defined time periods. From this analysis general conclusions have been drawn as to the observed trends in flow conditions.

The first significant potential impact on streamflow conditions in this reach of the Condamine was hypothesed to be the construction of the Chinchilla Weir. The historic streamflow data set was nominally divided at 1973, to coincide with the approximate timing of the construction of the Chinchilla Weir and the resulting flow exceedance curves are shown in Figure 7.5. The flow statistics produced based on analysis of historic stream flow records are provided in Table 7.5.

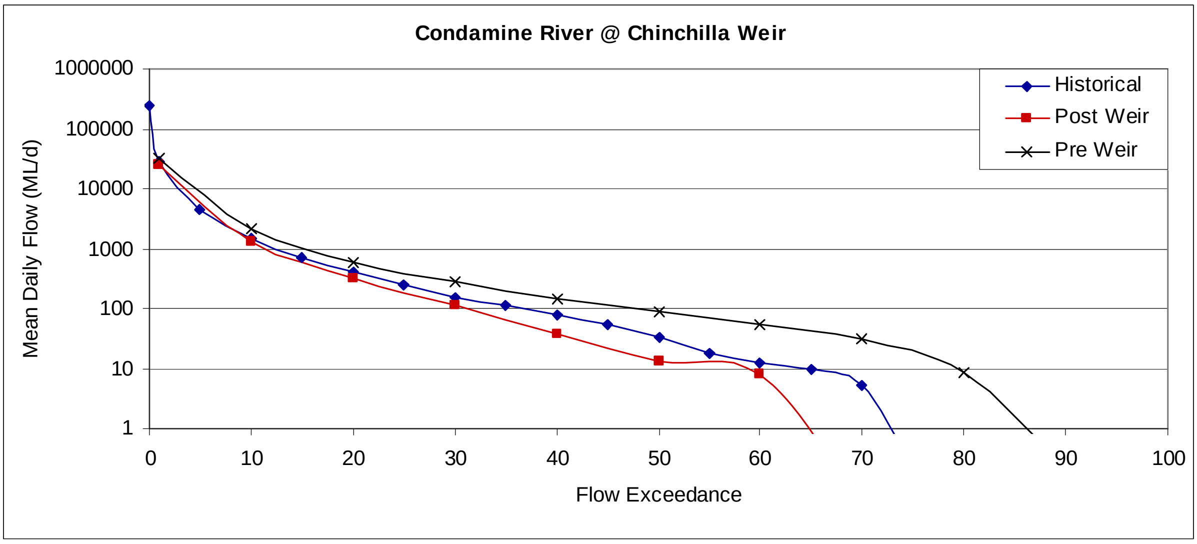


Figure 7.5 Flow Exceedance Curve, Condamine River Pre and Post Chinchilla Weir Construction

Table 7.5 Flow exceedance statistics, Condamine River pre and post Chinchilla Weir

Gauge	Data Available	Long Term Historic			Pre-weir (to 1972)			Post Weir (from 1973 onwards)		
		% time there is no flow	Median Flow (ML/d)	Low Flow* (ML/d)	% time there is no flow	Median Flow (ML/d)	Low Flow* (ML/d)	% time there is no flow	Median Flow (ML/d)	Low Flow* (ML/d)
Condamine River @ Chinchilla (upstream)	1966-present	20%	33	16.50	5%	88	44	25%	13	6.5
Condamine River @ Cotswold+	1955 – present	30%	35.4	17.7	10%	62.5	31.25	35%	29.6	14.8

* Low Flow as defined by the Condamine-Balonne ROP is half of the median daily flow within a simulation period
\+ Significant periods of missing data in historic records

The streamflow series were then divided further into periods of less than 10 years to determine if the streamflow patterns were observed to be changing over time. The resulting flow exceedance curves are provided in Figure 7.6.

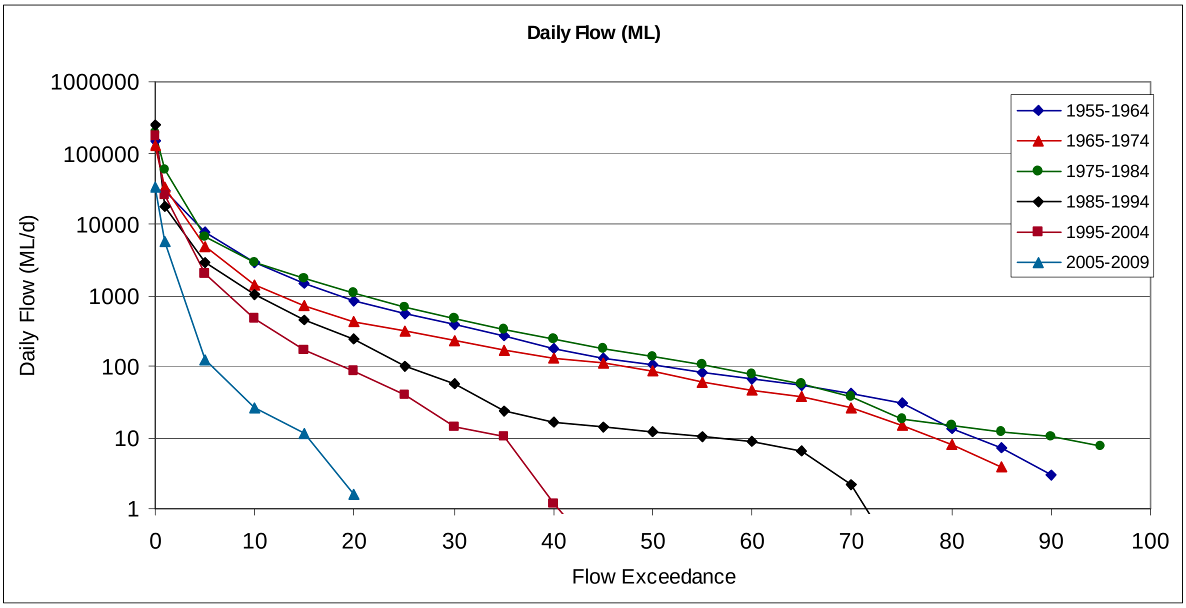


Figure 7.6 Flow exceedance curve, Condamine River

Results and conclusions

From the statistical analysis of stream flow data for the two Condamine River gauges the following general conclusions have been drawn:

- There is a significant increase in the percentage of time that no flow occurs within this reach of the Condamine River pre and post construction of the Chinchilla Weir

- Generally, the percentage of time that no flow occurs in this reach of the Condamine River has increased over time. The exception is the period between approximately 1975 and 1984 where a high proportion of the record is dominated by small flows of less than 100 ML/d
- There is no flow in this reach of the Condamine River for up to 75% of the period from 2005 to present
- There is no flow in this reach of the Condamine River for up to 45% of the period from 1995 to 2005
- Generally, there is a higher frequency of no flow observed at the gauge at Cotswold than at the Chinchilla gauge
- Analysis of the pre and post weir streamflow data shows that the median (50th percentile) flow has reduced from approximately 88 ML/d to 13.4 ML/d
- The median (50th percentile) flows have showed a decreasing trend since the early 1980's
- Similar trends are observed for high flows as noted for median flows.

Despite the findings of the CSIRO (2008), noted above in Section 7.5.1, that there was no statistical difference in the rainfall and runoff depths for the period 1997 to 2006 compared to a long term simulation (1895-2006), an observable trend in declining flows and increasing periods of no flow have been observed in the streamflow data analysed at Chinchilla. It is likely that the changes in observed flow patterns in this reach of the Condamine River may be attributed to factors such as:

- Increased control structures such as weirs
- Increased interception of overland flows
- Increased extractions of water from the river
- Increased extraction of groundwater resulting in alteration to the baseflow contribution.

7.5.4 Tributaries

In order to determine the likely flow patterns at each of the proposed discharge locations in ungauged catchments outside of the main reach of the Condamine River, a synthetic daily flow series was produced through the use of a rainfall runoff model. The construction, calibration and resulting flow exceedance analysis is provided below.

Rainfall runoff modelling

In developing streamflow series for each of the discharge locations in ungauged catchments outside of the main reach of the Condamine River the Australian Water Balance Model (AWBM) was utilised. This model is widely used within Australia and particularly Queensland as a tool to predict rainfall-runoff relationships of catchments.

AWBM is a catchment water balance model that can relate runoff to rainfall with daily or hourly data. The model uses 3 surface stores to simulate partial areas of runoff. The model calculates the moisture balance of each partial area at either daily or hourly time step. When the capacity of the store is exceeded runoff is generated. When runoff occurs from any store, part of the runoff becomes recharge of the baseflow store if there is baseflow in the streamflow. The surface runoff can be routed through a store if required to simulate the delay of surface runoff reaching the outlet of a medium to large catchment.

In this application, a single catchment model was developed to represent the contributing catchment to either the calibration stream gauge or the discharge locations. The adoption of a single catchment effectively 'lumps' the catchment characteristics into a single overall average catchment response. That is, if the contributing catchment consists of areas of grazing and forest the model assumes that the values used to represent the surface store capacities are 'average' values for grazing and forest areas within the contributing catchment.

While it is noted that each of the contributing catchments to the calibration gauge locations (Yuleba Creek at Forestry and Weir River at Gunns Bridge) and to the discharge locations will consist of significantly differing land use, topography and level of catchment development (overland flow interception, groundwater interactions), it was assumed that the catchment characteristics at the discharge locations were adequately represented by the 'average' characteristics of the gauged catchments. This assumption limits the accuracy of the runoff predictions, however, for the purpose of estimating the flow patterns at the discharge locations this approach is considered adequate.

Model calibration

Yuleba Creek is the only tributary in the northern half of the study area where gauge data is available. Weir River @ Gunns Bridge is the only gauge located in the southern half of the study area. Therefore, an AWBM was developed and calibrated to the Yuleba Creek at Forestry catchment in order to determine the representative AWBM input parameters for the northern discharge locations. These input parameters were then applied to the other ungauged catchments in which discharge is to be assessed in the northern half of the study area (namely, WTF_MEL_01, WTF_RCK_01, WTF_WOL_01, WTF_BYM_01 and WTF_HCK_01). Similarly an AWBM model was constructed and calibrated to the Weir River gauge data for transfer to the catchment at the WTF_GIL_01 discharge location.

The catchment area which contributes to the Yuleba Creek at Forestry gauging station is approximately 1475km^2. This gauge is located significantly downstream of the proposed discharge location on Yuleba Creek.

In calibrating the AWBM, SILO rainfall and evaporation data was obtained near to the gauging station.

An auto calibration tool for AWBM has been developed by eWater and the CRC for Catchment Hydrology and is available as part of the Rainfall-Runoff Library (RRL) through the CRC for catchment hydrology website (http://www.toolkit.net.au/rrl). The auto calibration tool was used to provide an initial calibration assessment of the AWBM parameters. However, it was determined that appropriate parameters were not achieved using the auto calibration model. This is a common outcome where the data available for calibration is of questionable consistency and quality.

Stream gauge data was available from the Yuleba Forestry Station for the period 1973 to 2009. Upon review of the data, there were a number of gaps in the data set, particularly in the first half of the data set. Therefore, a manual calibration was undertaken to determine appropriate AWBM parameters for use within this study. For calibration purposes the period from 1995 to 2007 was selected due to the higher quality of this period of data.

The Weir River at Gunn Bridge stream gauge data showed similar deficiencies in data and thus a manual calibration was undertaken for this location also. For calibration purposes the period of 2000 to 2009 was selected at the Weir River location.

A range of parameters were trialled within the AWBM and a comparison of the 'fit' of modelled streamflow to the gauged streamflow data made. Two tools were used to determine the success of the calibration exercise; a comparison of the cumulative total of runoff from the model compared to the

gauge data and a comparison of the daily modelled and recorded runoff values using a Nash-Sutcliffe[5] statistic. In addition, detailed review of the modelled output was undertaken on an event basis compared to the gauge data series.

A summary of the adopted calibrated AWBM parameters are outlined in Table 7.6.

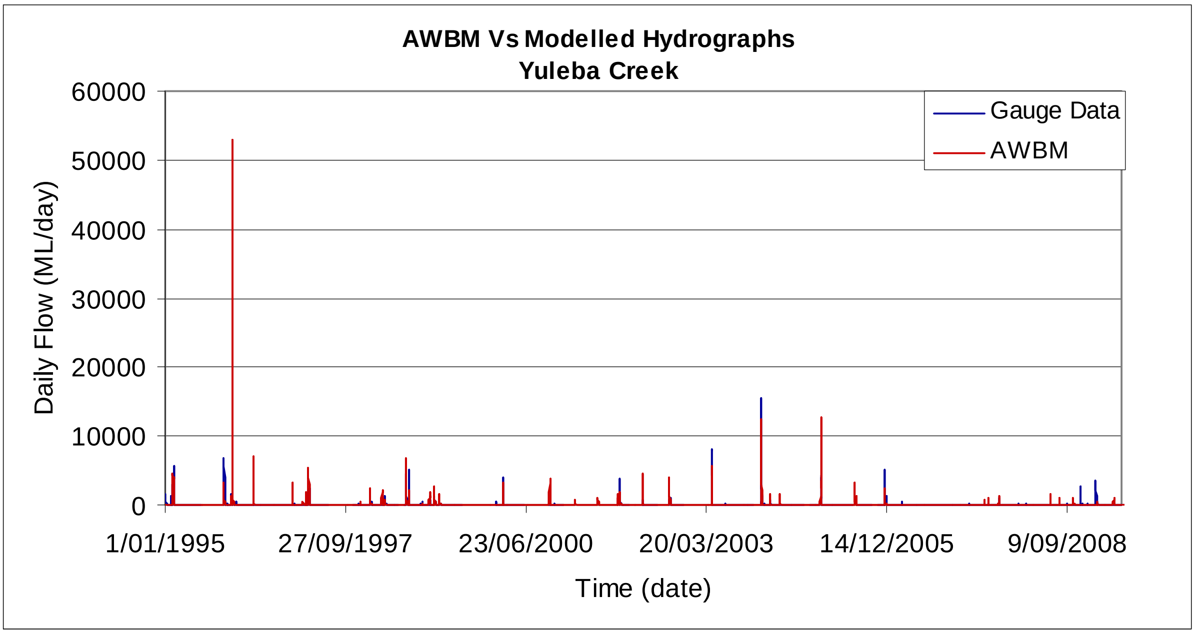
Table 7.6 Summary of AWBM calibration parameters

AWBM Parameter	Yuleba Creek Calibration Results	Weir River Calibration Results
A1	0.134	0.134
A2	0.433	0.433
C1	30	45
C2	70	89
C3	160	160
Ks	0.55	0.55
BFI	0.05	0.05
Kb	0.8	0.8

Overall the results from the AWBM were comparable to the observed stream gauge for the Yuleba Creek station and the Weir River gauge. A comparison of the hydrographs produced from AWBM versus gauge observation data is highlighted in Figure 7.7.

5 Nash-sutcliffe equation

R^2 =1 - sumproduct((observed discharges Qo - modelled discharges Qm)^2) / sumproduct((observed discharges Qo – average of observed discharges Qoavg)^2)



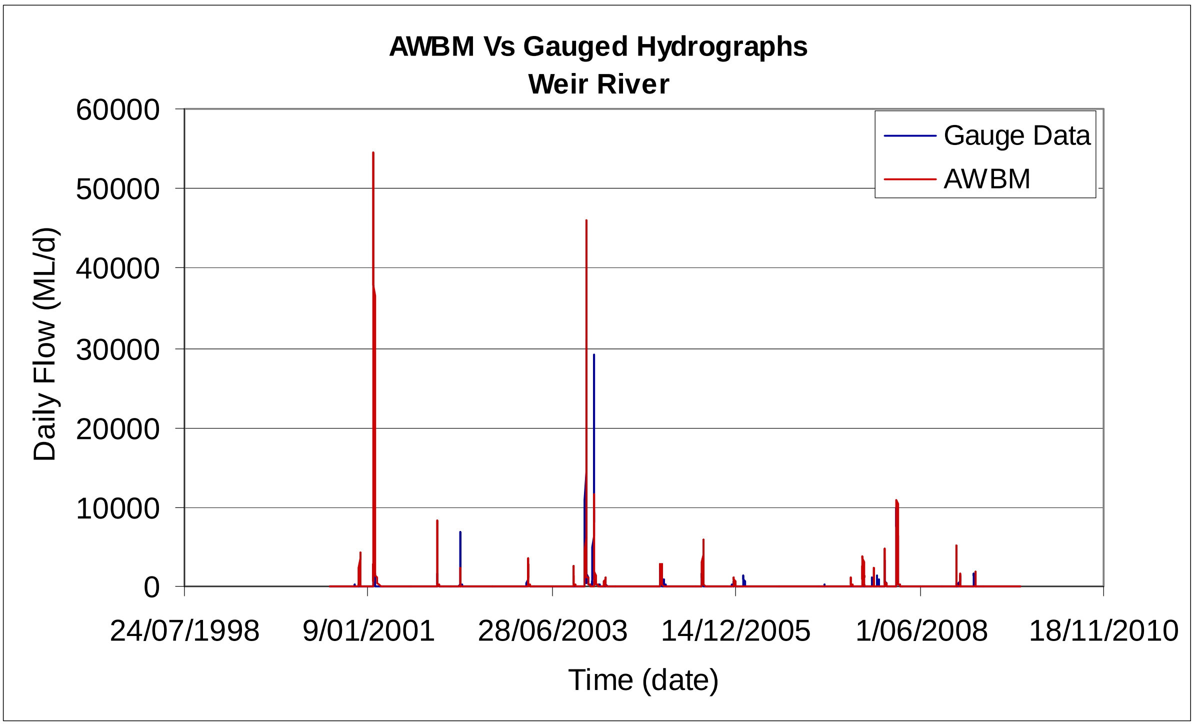
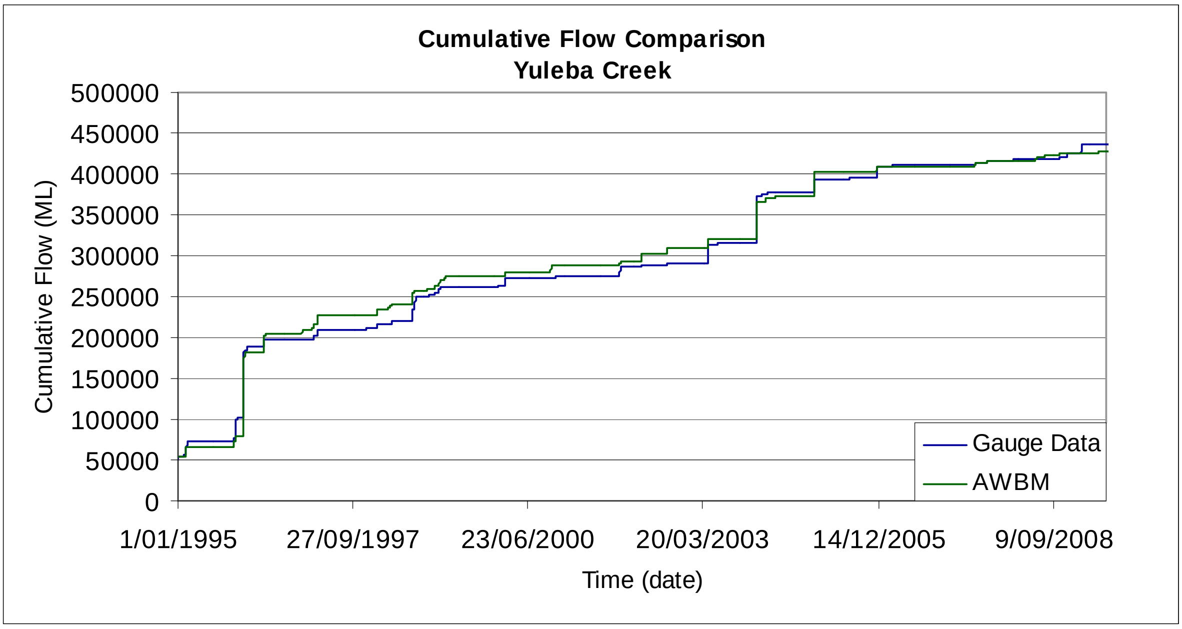


Figure 7.7 AWBM and observed gauge hydrographs (Yuleba Creek and Weir River)

Graphs showing the comparison of the cumulative total modelled runoff compared to the gauge data are shown in Figure 7.8 for the Yuleba Creek and Weir River calibration. These graphs provide a comparison of the cumulative volumes of runoff produced within the AWBM compared with that observed at the stream gauge. It can be seen that, in both cases, the modelled runoff volume correlates well with the observed data. It is noted that the gaps in the gauge data were removed from the model data for the purposes of producing this graph.



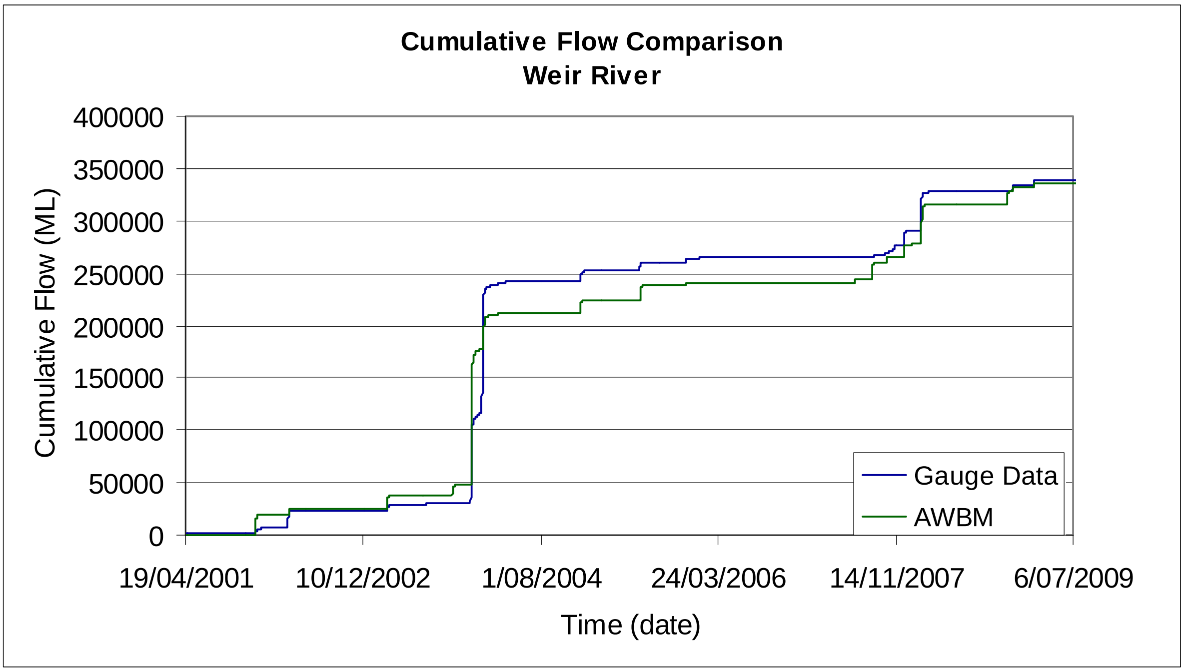


Figure 7.8 Cumulative flow graph comparing modelled and observed data

The Nash-Sutcliffe calibration plot for the Yuleba Creek calibration is provided in Figure 7.9. The outcomes of the calibration show a reasonable level of calibration based on this statistic. A calibration which achieves a Nash-Sutcliffe statistic of greater than 0.75 is said to be well calibrated and in this case a statistic of 0.65 was achieved. A lower Nash-Sutcliffe calibration statistic produced from daily data can be a result of a mismatch in the timing of the streamflows compared to the rainfall data and localised spatial variations in rainfall that cannot be accounted for at this level of model sophistication.

Due to confidence levels related to the streamflow gauge data at Weir River a Nash-Sutcliffe statistic as not produced for the Weir River calibration.

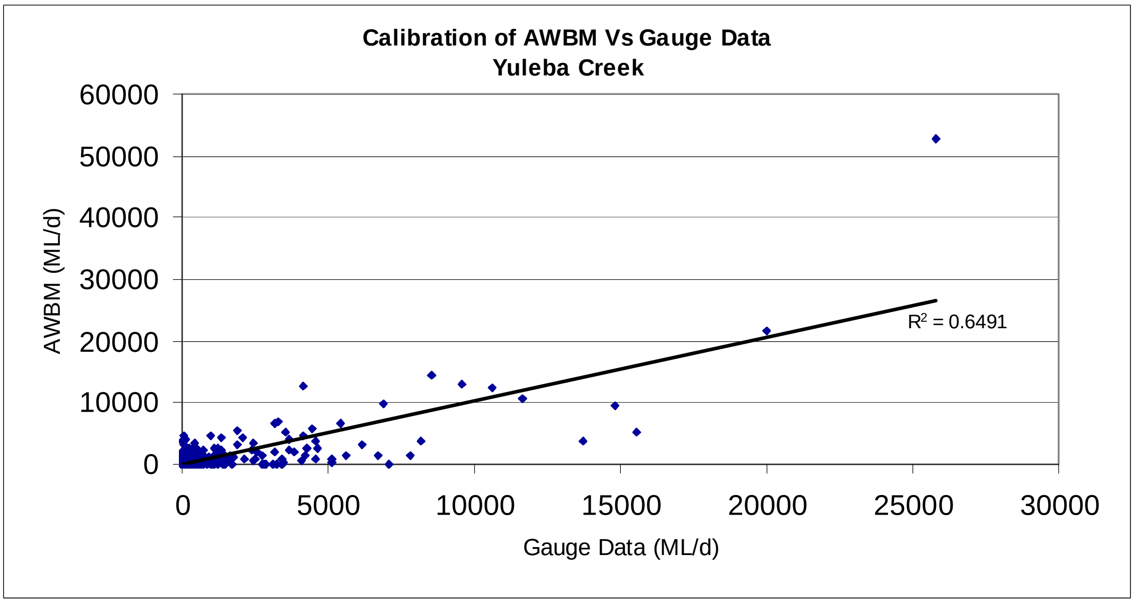


Figure 7.9 Nash-Sutcliffe statistic – Yuleba Creek AWBM calibration

Flow exceedance curves

The AWBM parameters calibrated for the Yuleba Creek catchment, were applied to all other catchments in which discharge to a creek was being considered except at the Gilbert Gully location (WTF_GIL01). The AWBM parameters calibrated for the Weir River catchment were used to produce a flow series for the Gilbert Gully location. SILO data was obtained for each discharge location catchment and utilised within an individual AWBM for the discharge locations. A summary of the contributing catchment areas for each discharge location is provided in Table 7.7.

Table 7.7 Contributing catchment areas at each discharge location

WTF	Development area	Watercourse proposed for discharge	Contributing catchment (km^2)	Mean annual rainfall (mm)	Mean annual runoff (mm)
WTF_MEL_01	Conbabula	Unnamed Tributary of Kangaroo Creek	75	583	42
WTF_RCK_01a	Combabula	Yuleba Creek	525	583	41
WTF_WOL_01	Wooleebee	Woleebee Creek	489	603	50
WTF_BYM_01	Carinya	Dulacca Creek	383	586	41
WTF_GIL_01	Gilbert Gully	Unnamed North Tributary of Weir River	219	605	41
WTF_HCK_01	Ramyard	Tchanning Creek	778	582	26

Table 7.7 also provides a summary of the mean annual rainfall and modelled runoff for each catchment. The CSIRO (2008) determined that based on their rainfall-runoff modelling across the Condamine-Balonne catchment the average annual runoff was 19 mm. Within the mid Condamine area, in the vicinity of the discharge location catchment, the CSIRO report indicates that for areas with

mean rainfall totals of between 500 mm and 800 mm the mean modelled runoff is in the range of 20 mm to 40 mm (CSIRO, 2008). It can be seen that the average runoff totals produced during the AWBM modelling exercise align closely with that estimated in the CSIRO (2008) report.

Flow exceedance curves produced for each location are provided in Appendix G. Table 7.8 provides a summary of key flow statistics for the discharge locations excluding the main reach of the Condamine River (addressed in Section 7.5.3).

Table 7.8 Flow exceedance statistics – tributary discharge locations

Watercourse	WTF	Long Term Simulated		
		% time there is no flow	% time flow exceeds 10ML/d	% time flow exceeds 100ML/d
Yuleba Creek	WTF_RCK_1a	65%	10%	5%
Unnamed	WTF_MEL_01	70%	6%	1.5%
Tchanning Creek	WTF_HCK_01	65%	6%	5.5%
Woleebee Creek	WTF_WOL_01	55%	11%	5.5%
Dulacca Creek	WTF_BYM_01	65%	9.5%	4.5%
Unnamed North (Gilbert Gully)	WTF_GIL_01	67%	5.5%	2%

* Statistical analysis of the data indicates that the Low Flow and the Median Flow for all locations is equal to 0 ML/d. Low Flow as defined by the Condamine-Balonne ROP is half of the median daily flow within a simulation period

Results and conclusions

From the statistical analysis of simulated stream flow data for the tributary discharge locations the following general conclusions are drawn:

- It is confirmed that each of the tributaries to the Condamine River in the study area are ephemeral in nature with no flow conditions occurring between 55% and 70% of the time
- Any of the proposed discharge locations that are pursued as a viable option for discharge of the CSG associated water from the project will require a detailed hydrological analysis for the specific site location. The current results are provided for the preliminary selection of a feasible discharge location and will also be used to support subsequent detailed design, permit application and licensing.

7.5.5 Seasonal flow variations

The flow exceedance analysis provided above indicates that all discharge locations exhibit flow conditions that are intermittent or ephemeral in nature, that is, that they exist under no or low flow conditions for a proportion of the year. An analysis of the seasonal variation in flow provides an indication of the timing of these low or no flow periods and is shown in Figure 7.10.

As the rainfall and thus runoff patterns throughout the year are likely to be relatively consistent over the study area an analysis of the seasonal variation in flows has been conducted for the Condamine River only. It is proposed that seasonal flow patterns at all other proposed discharge locations will be similar and any conclusions drawn for the Condamine River flows will hold true for the other locations.

Based on the historical streamflow records available for the gauge on the Condamine River at Chinchilla (422308C) the majority of flow within the river occurs during the Summer and Autumn period with an average of 45% of the flow occurring in the summer season (December to February) and 30% of the average annual flows occurring in autumn (March to June). The remaining 25% of the average annual flows occur in the winter and spring seasons (July to November).

These observations of the seasonal nature of flow within the study area are supported by the CSIRO (2008) report as outlined in Section 7.5.1 above.

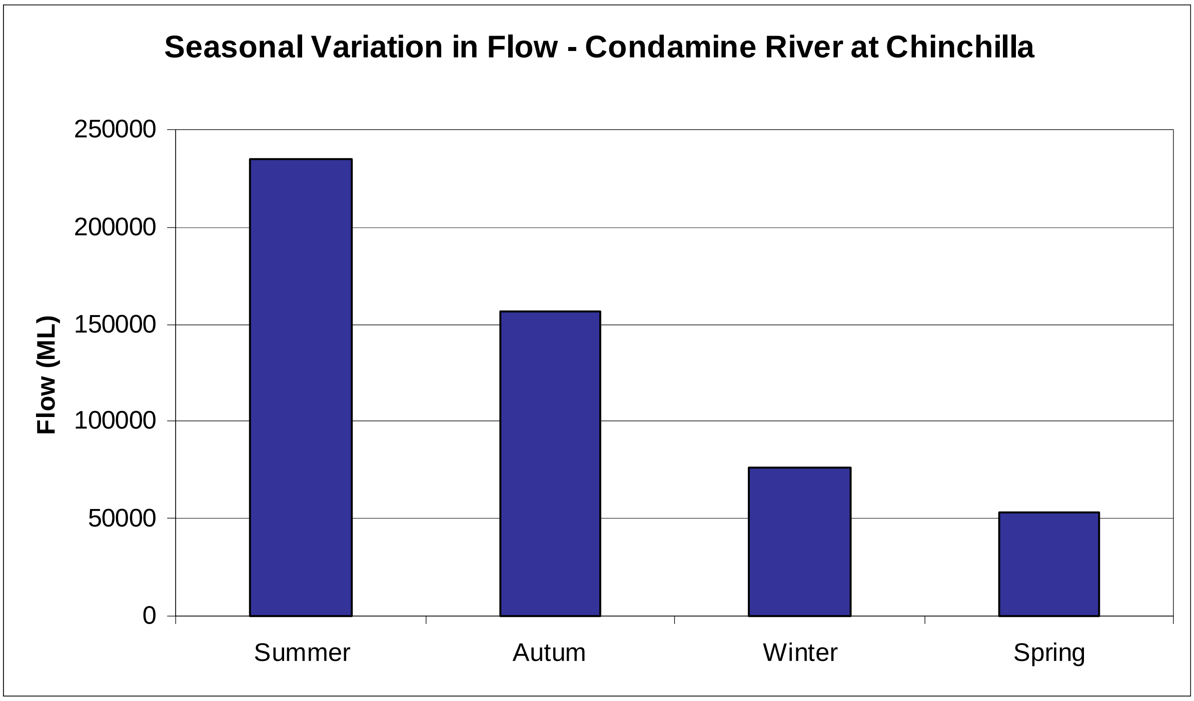


Figure 7.10 Seasonal variation in flow – Condamine River at Chinchilla

7.5.6 Climate change

No specific climate change scenarios have been modelled as part of this investigation. However, as noted in Section 7.5.1 above the CSIRO as part of the Water Availability in the Condamine-Balonne report undertook an analysis of the impact of climate change conditions on the hydrology of the region. This report concludes that

> *"Rainfall-runoff modelling with climate change projections from global climate models indicates that future runoff in the region is more likely to decrease than increase. Sixty percent of the modelling results show a decrease in mean annual runoff and 40 percent show an increase in mean annual runoff. The median estimate is a 9 percent reduction in mean annual runoff by ~2030 relative to ~1990. However, there is considerable uncertainty in the modelling results with the extreme estimated ranging from -20 percent to +26 percent. These extreme estimates come from the high global warming scenario, and for comparison the range from the low global warming scenario is -7 to +7 percent change in mean annual runoff. The main sources of uncertainty are in the global warming projections and the global climate modelling of local rainfall response to the global warming. The uncertainty in the rainfall-runoff modelling of climate change impact on runoff is small compared to the climate change projections." (CSIRO, 2008)*

From this information it is concluded that while flow patterns, both volume and seasonal variability, in the region may be altered as a result of climate change the uncertainty around the climate change and global warming scenarios makes it difficult to predict the level and even direction of impact.

7.6 Hydraulic stream capacity assessment

In order to assess the hydraulic capacity of the watercourses at the proposed discharge locations a set of preliminary hydraulic models were constructed. The intent of this modelling exercise was to provide a preliminary characterisation of the hydraulic properties over the reach of the watercourse downstream of the proposed discharge locations in order to determine the suitability of this location to discharge and likely impacts of a proposed discharge in this location. This analysis was limited by a number of factors as outlined below and should not be used as a basis for the design of discharge infrastructure. Due to the local scale of the analysis the hydraulic models constructed and discussed in Section 5 were not suitable for use in this component of the study.

7.6.1 Model construction

For each of the proposed discharge locations a 1 dimensional hydraulic model was constructed for approximately 2 kilometres downstream of the proposed discharge location. The exception was the proposed discharge location on the Condamine River where a 4.5 km reach was identified for the release point. In this case the hydraulic model was extended to include the 4.5 km reach. Cross-sections were taken every 20m.

The hydraulic model was constructed using the photogrammetry survey sourced for the regional flood modelling tasks. It was found that whilst the general cross-section profile of the watercourses appeared acceptable this survey lacked detail sufficient to accurately represent the channel profile consistently along the defined model reaches. It was observed that inverts at the selected cross sections rose and fell by up to 3m from one section to the next. Upon reviewing the data contained within the long section, it was seen that there were number of consistent low points along the long section. Using these low points an average slope of the watercourse was determined. The cross-sections were adjusted so that the invert was consistent with the determined average slope of the watercourse.

The key parameters used in each of the hydraulic models and a brief description of the watercourse is provided in Table 7.9.

Table 7.9 Summary of hydraulic inputs

Creek Discharge	Mannings	Ave Slope (%)	Cross-Section Description
Yuleba	0.05	0.13	High and steep in-banks. Trees and low shrubs on banks. Bed is largely free of vegetation, with dead logs and branches
Unnamed	0.03	0.37	Shallow in-bank. Some trees and shrubs on embankment. Short and sparse vegetation on sandy bed.
Woleebee*	0.045	0.08	Flat U-shaped channel. Average width 28 m.
Dulacca*	0.045	0.12	Information only available from survey
Tchanning	0.045	0.08	Moderately deep and slight grade within in-banks.

Creek Discharge	Mannings	Ave Slope (%)	Cross-Section Description
			Grassed banks with intermittent spacing of small trees. Bed is well defined and void of vegetation
Weir River	0.03	0.10	Shallow in-bank. Grassed banks with intermittent spacing of medium sized trees. Bed is well defined, void of vegetation and quite sandy
Condamine River	0.03	0.02	Deep in-banks with moderate grade on banks. Grassed banks with intermittent spacing of large trees

* Sites not visited during field trips due to access constraints. Observations drawn from survey information and aerial photography

A sensitivity analysis of the adopted Mannings roughness was undertaken for each hydraulic model. It was found that while there were variations in the key results the sensitivity of the models was not significant and does not impact conclusions drawn from the model results.

7.6.2 Scenarios

The hydraulic models were run in steady state over a range of flow conditions based on the flow exceedance curves presented in Section 7.5.4 above. A nominal discharge was added to the existing flows to determine the impact of a proposed discharge into the watercourses may cause. Additional discharges were added up to either the maximum possible daily capacity of each of the WTF or, at the Condamine River, up to a maximum possible aggregated daily flow condition projected during the project.

The following flow scenarios were analysed:

- No catchment flow (discharge only)
- Catchment flow between median flow and 95th percentile flow
- Catchment flow between median flow and 95th percentile flow plus additional discharge up to the possible maximum at each proposed location
- Bank full.

Table 7.10 details the flow rates that were used to represent additional discharges and the flow determined to represent bank full conditions for each watercourse. It can be seen that the additional discharges would represent a very small proportion (less than 10%) of the bankfull flow conditions.

Table 7.10 Maximum of range of discharges and identified bankfull condition

Watercourse	Maximum analysed additional discharge (m3/s)	Bank full (m3/s)
Yuleba Creek	0.93	40
Unnamed Tributary of Kangaroo Creek	0.93	10
Tchanning Creek	0.23	50
Woleebee Creek	0.46	20
Dulacca Creek	0.46	20
Unnamed Tributary of Weir River	0.69	80
Condamine River	2.72	800

7.6.3 Hydraulic characteristics

The hydraulic models were used to extract depth, velocities, stream power and stream stress at each cross section for a range of flow conditions. The extracted results are presented as averaged over the model reach. While in each model there were localised variations in the modelled results it was found that the average was relatively representative of the reach results and not overly skewed by outliers.

Tables highlighting the various results obtained for velocity, stream power and stream stress are provided in Appendix H.

A study undertaken for streams in the Bowen Basin (central Queensland) (Fisher Stewart 2002) determined typical stream hydraulic parameters within waterways of the area with the objective of developing a guideline for the design and rehabilitation of watercourse diversions in the Bowen Basin. Whilst the hydraulic characteristics of streams are dependant of geographical location this material provides guidance on ranges of hydraulic characteristics that will likely produce or maintain a stable stream profile.

Stream power is a measure of the excess energy available in a stream to do work in and on the channel. It is calculated using a product of channel slope and discharge. If the stream power is too high the channel will typically erode and conversely a low stream power will generally indicate aggradation.

Stream velocity is the speed at which water flows in a stream and is calculated as the product of the discharge and the cross sectional area of the stream.

Shear stress is defined as the force exerted on the channel bed and banks by the action of the flowing water. It is also a function of channel slope and discharge.

The Bowen Basin study (Fisher Stewart, 2002) provides the following typical ranges for hydraulic parameters of incised (bankfull flows greater than a 5 year ARI flow) waterways:

- Stream power between 20 W/m^2 and 80 W/m^2 but typically less than 60 W/m^2 in a 2 year ARI event and between 50 W/m^2 and 220 W/m2 in a 50 year ARI event
- Stream velocity typically between 1 m/s and 1.5 m/s in a 2 year ARI event and between 1.5 m/s and 2.5 m/s in a 50 year ARI event
- Shear Stress typically less than 40 N/m^2 in a 2 year ARI event and less than 100 N/m^2 in a 50 year ARI event.

In considering the hydraulic characteristics of a waterway it is useful to note that high flows which tend to equate to a bank full event (or a close to or exceedance of a bank full event) are much more likely to cause geomorphic changes that may include bank erosion, sedimentation and associated ecological impacts

It is also noted that the introduction of regular flows into an ephemeral watercourse during natural periods of no flow is associated with a high risk of geomorphic changes that may include bank erosion, sedimentation and associated ecological impacts. Further discussion of associated ecological impacts is provided in Volume 5, Attachment 20.

7.6.4 Results and conclusions

Condamine River

A summary of the findings of the hydraulic analysis for the Condamine River is provided below.

- Velocities within this reach were found to be relatively low reaching up to 1.7 m/s under bankfull conditions
- Stream power was found to be very low (less than 7 W/m^2) up to a flow equivalent to the 90th percentile long term historic flow exceedance
- From the 90th percentile flows to bank full conditions (800 m^3/s or 69120 ML/d) stream powers were greater than 7 W/m^2 but generally less than 35 W/m^2
- Bank full flow conditions generally correlated with flows above the 99 percentile exceedance flow, that is, flow which occurred less than 1 % of the simulation period
- The hydraulic model results for stream power support the geomorphic observations from the site visit, that this reach is generally undergoing aggradation
- Stream stress was generally low, up to an average of 2.6 N/m^2 at a 90th percentile flow and an average of 15 N/m^2 at bank full
- The upper limit of the modelled range of additional discharge of 235 ML/d into no flow conditions produced a modelled depth of approximately 0.7 m
- Under flow conditions up to a 90th percentile flow depths were generally less than 2 meters
- Bankfull flow was approximately 8.6 meters deep throughout the 4.5 km reach of the Condamine River
- Based on a flood frequency analysis the 1 in 2 year ARI flow was found to be approximately 226 m^3/s and the 1 in 50 year ARI flow approximately 3320 m^3/s. Therefore the 1 in 2 year ARI flow is significantly less than bank full conditions.

Based on the findings of the hydraulic analysis the following conclusions are drawn:

- An additional discharge, of any magnitude under no to low flow conditions, results in depth, velocity, stream power and shear stress values that are relatively low and that would not be expected to have a significant hydraulic impact on this reach of the Condamine River
- Additional discharges up to approximately 235 ML/d, to this reach of the Condamine River, between low flow conditions and up to bank full conditions are not likely to result in significant hydraulic impacts. Percentage changes in key parameters decrease as flows within the creek without the discharge increase towards bank full conditions. Absolute stream power, shear stresses and velocities resulting with the additional discharges remain below acceptable values and within the range likely to be experienced in this waterway reach under natural conditions
- It is noted that, depending on the proposed rate of discharge, significant engineering controls may need to be constructed to ensure localised control of velocity and scour and erosion at the discharge location. The presented results have been developed on the basis that the discharge infrastructure is able to be designed to ensure that flows into the creek section are controlled to minimise impacts of high localised velocities and the direction of discharge optimised to ensure protection of local bed and banks.

Local tributaries

A summary of the findings of the hydraulic analysis is provided below.

- Bank full flow conditions generally correlated with flows above the 99 percentile exceedance flow, ie flow which occurred less than 1 % of the simulation period

- Modelled velocities in all tributaries (Table 7.11) were found to be relatively low (less than 1 m/s) up to bank full conditions with the exception of Yuleba Creek which showed average velocities at greater than 1 m/s at bank full conditions and up to 1.4 m/s at some locations during bank full flow conditions
- Stream power in all modelled tributaries up to approximately a 95th percentile historic flow conditions were less than 7 W/m^2. Under flow conditions from the 95th percentile to bank full conditions stream powers were generally within the range of 7 W/m^2 to 35 W/m^2 with the exception of Yuleba Creek which was observed to have modelled stream powers above 35 W/m^2 at bank full conditions
- Average stream stresses were relatively low to bank full conditions in all modelled tributaries except Yuleba Creek. Bank full stream stresses in Yuleba Creek were higher than other modelled tributaries but still within a low to medium value range.

Table 7.11 Summary of hydraulic results for the tributaries

Watercourse	WTF	Approximate bank full flow (m3/s)	Bank full average depth (m)	Bank full average velocities (m/s)	Bank full average stream power (W/m2)	Bank Full average stream stress (N/m2)
Yuleba Creek	WTF_RCK_1a	40	1.70	1.14	38.7	32.6
Unnamed	WTF_MEL_01	10	0.60	1.00	15.1	13.6
Tchanning Creek	WTF_HCK_01	50	2.60	0.92	13.9	14.9
Woleebee Creek	WTF_WOL_01	20	1.90	0.61	6.0	9.4
Dulacca Creek	WTF_BYM_01	20	2.00	0.81	10.8	12.9
Unnamed North (Gilbert Gully)	WTF_GIL_01	80	3.22	0.74	8.9	11.2

Based on the findings of the hydraulic analysis the following conclusions are drawn:

- The addition of discharges up to the maximum proposed daily capacity of the local water treatment facility to each of the tributaries under no to low flow conditions was determined to have a moderate to significant hydraulic impact
- The significance of the hydraulic impacts of discharge into the local tributaries under low flow conditions compared to the Condamine River discharge location are driven primarily by the proportion of time that the tributaries are predicted to exist under no to low flow conditions. Generally the tributaries displayed modelled flow depths of less than 100 mm for up to the 85th percentile flow exceedance. The 85th percentile exceedance flows are predicted to be between 0.23 ML/d and 2.6 ML/d, thus 85% of the time flows of less than these values are expected. Therefore, while the hydraulic capacity of the tributaries can accept the discharges proposed, with the potential change in sediment transfer being relatively minor on a daily basis, the fact that these minor changes in sediment transfer would exist for up to 85% more often results in the potential for significantly greater levels of sediment transfer than under natural flow patterns. It is for this reason that the potential impacts of discharge to these ephemeral tributaries during

low or no flow are classified as being more significant than constant discharges to the Condamine River

- The addition of discharges to the tributaries during natural flow conditions up to bank full conditions are not likely to cause significant hydraulic impacts
- It is noted that, depending on the proposed rate of discharge, significant engineering controls may need to be constructed to ensure localised control of velocity and scour and erosion at the discharge location. The presented results have been developed on the basis that the discharge infrastructure is able to be designed to ensure that flows into the creek section are controlled to minimise impacts of high localised velocities and the direction of discharge optimised to ensure protection of local bed and banks.

7.7 Potential downstream influence of discharge

Given the location of the majority of Project infrastructure and the high level of water demand relative to supply in the vicinity it is likely that the influence of any discharges would occur within the Condamine Balonne River system and therefore occur in areas relating to the Condamine Balonne Water Resource Plan (WRP) and Resource Operations Plan (ROP).

However, any decision to discharge to the local watercourses at WTF_MEL_01 or WTF_WOL_01 would involve the discharges flowing into the Dawson River catchment within the Fitzroy Basin water resource planning area. Similarly, if discharge was proposed to the Weir River tributary from WTF_GIL_01 (or 01a),the action would fall under the Border Rivers water resource planning area.

The discharge locations that fall within the Fitzroy Basin and Border Rivers catchments are located in the upper reaches of the relevant river systems. Therefore, it is unlikely that discharges in these reaches, of the proposed magnitudes, could be adequately assessed within the planning instruments and associated modelling tools currently available under these plans.

7.7.1 Planning and legislative constraints

Any proposal to undertake discharge or withdrawals to/from the Condamine River or its tributaries must take into account the provisions of the Water Resource (Condamine and Balonne) Plan 2004 and the Condamine Balonne Resource Operations Plan 2008. Part 4 of the WRP provides acceptable measures as to the Performance Indicators and Objectives that are applicable to these systems.

There are two types of performance indicators defined by the Plan; Environmental Flow Objectives (EFOs) and Water Allocation Security Objectives (WASOs).

The EFO’s are assessed at a number of locations along the Condamine River, described in Schedule 2 of the plan. The performance indicators (flow statistics) at EFO nodes may be affected by changes in upstream development and water use.

The relevant performance indicators measured at these nodes are defined in the plan, and the definitions are summarised following.

a) Low Flow: the total number of days in the simulation period in which the daily flow is not more than half the pre-development median daily flow

b) Summer Flow: the average number of days in summer (1st December to last day in February) that the flow is greater than the median pre-development flow

c) Beneficial Flooding Flow: The median of the wet season 90-day flows for the years in the simulation period. The wet season 90-day flow, for a year, means the total flow in the continuous 90 day period with the highest total of daily flows

d) 1 in 2 year flood: the daily flow that has a 50% probability of being reached at least once a year.

Section 12 of the plan describes the maximum changes to the above performance indicators that are allowable under the plan in relation to a decision about a change to water allocation rules.

At nodes F, H, and I (nodes within the IQQM model relevant to the proposed discharge locations), the performance indicators are required to be:

a) not less than the lesser of the following—

   i) 66% of the indicator for the pre-development flow pattern

   ii) the indicator immediately before the decision is made

b) not more than the greater of the following—

   i) 133% of the indicator for the pre-development flow pattern

   ii) the indicator immediately before the decision is made.

WASOs are measures of the security of supply to groups of water users. The WASO performance indicators are:

a) Annual volume probability. This is defined as:

   i) For a water allocation group for taking unsupplemented water—the percentage of years in the simulation period in which the volume of water that may be taken by the group is at least the total of the nominal volumes for the group

   ii) For a water allocation group for taking supplemented water—the average annual volume of water that may be taken by the group in the simulation period as a percentage of the total of the nominal volumes for the group.

b) 45% annual volume probability. This is defined as:

   i) For a water allocation group, means the percentage of years in the simulation period in which the volume of water that may be taken by the group is at least 45% of the total of the nominal volumes for the group.

Any changes to the water allocation rules should not result in any reductions to the WASO performance indicators which applied just prior to the change.

In order to assess the compliance of proposed discharge regimes with the requirements of the Condamine-Balonne WRP and ROP request was made to access the IQQM model developed by DERM for the mid Condamine River. Description of the model and the analysis undertaken is provided in Volume 5, Attachment 12.

As most of the proposed discharge locations are located off the main branch of the Condamine River and as such are not included in the mid Condamine IQQM model flow duration curves developed for each of the discharge locations have been plotted with the 66% and 133% band widths to provide an understanding as to how this requirement may relate to each of the proposed discharge locations. These figures are provided in Appendix G.

7.7.2 River system modelling

The CSIRO completed a comprehensive study of the water availability and water use within the Condamine-Balonne river system as part of the Water availability in the Condamine-Balonne report (2008). This study utilised the IQQM model constructed and used in the development of the Condamine Balonne ROP. The CSIRO river system modelling provided the following relevant key outcomes:

- The current average surface water availability for the Condamine-Balonne is1305 GL/year
- The current relative level of use for the entire region is extremely high, with 53 percent of the average available water is diverted for use. In the Condamine-Balonne, 55 percent of the average available surface water is diverted for use. This is compared with other Queensland catchments where generally only 10 to 40 percent of available water is diverted for use
- The high level of use in the Condamine-Balonne has significantly reduced end-of-system flows in the Condamine-Balonne system
- In the Queensland part of the Condamine Balonne medium security and town water supplies are both highly utilised: 97 percent of available Queensland medium security is used and 89 percent of the available Queensland town water supply water is used
- Utilisation of Queensland un-supplemented access, Queensland floodplain harvesting and New South Wales unregulated access is lower at 34, 56 and 62 percent, respectively. This likely reflects a low level of water availability for these users
- Once current groundwater extraction levels in the Condamine-Balonne reach equilibrium with the river, that is, if there is no change in current groundwater extraction and the groundwater levels stabilise, the total streamflow impact will be a 10 GL/year loss to groundwater loss. This will have minor impact on streamflow overall and on end-of-system flows
- Under the best estimate 2030 climate, average surface water availability in the Condamine-Balonne would be reduced by 8 percent. Average surface water diversions would be reduced in the Condamine-Balonne system by 5 percent. As a result, the total end-of-system flow would fall by 12 percent
- In the Condamine-Balonne system, under the wet extreme 2030 climate, average surface water availability would increase by 17 percent, diversions would increase by 7 percent and total end-of-system flows would increase by 21 percent
- In the Condamine-Balonne system, under the dry extreme 2030 climate, average surface water availability for the entire region would decrease by 27 percent, diversions would decrease by 17 percent and total end-of-system flows would decrease by 35 percent
- Future farm dam developments, as documented in the CSIRO investigation, would cause a 4 GL/year reduction in inflows on average. The overall impacts of future development on streamflow and water use would be negligible.

Given the level of utilisation of un-supplemented access to flows within Queensland is as low as 34 percent (CSIRO, 2008), based on the findings of the CSIRO report, there is significant potential for existing un-supplemented users to access additional flows within the reach of the Condamine directly downstream of the proposed release point. However, access to additional discharges within the river would be constrained by the current un-supplemented water allocation licence conditions containing flow condition triggers. Table 7.12 provides a summary of the total un-supplemented water allocations between the proposed discharge location in the Condamine River and the upstream extent of the St

George WSS and the attached river flow condition triggers. If water could be released such that these flow conditions are met then these un-supplemented users may be able to access the additional discharge.

Table 7.12 Condamine-Balonne River un-supplemented water allocations downstream of Condamine River discharge locations with flow conditions (CBU-04 to CBU-09)

Flow Condition (ML/d)	Total Nominal Allocation (ML)
-	2334
367	1385
605	348
1210	4660
1728	5620
1730	5340
3024	280
Total	**19967**

If, on average, the utilisation of the un-supplemented water allocations is 34 percent then, with the correct discharge regime, on average up to 66 percent or 13,178 ML/year may be accessed within the reach of the Condamine River between the discharge locations and the upstream extent of the St George WSS.

Significant volumes of water are lost from the Condamine-Balonne river system in transmission losses and surface-groundwater interactions. As such a portion of discharges to the river system will be lost from the system via these mechanisms.

As part of the CSIRO (2008) study a water balance was conducted for each reach of the river system between gauging station for the period between 1990 and 2006 comparing modelled and accounted mass balances. In the reach between the Condamine River at Chinchilla gauge and the Condamine River at Cotswold gauge the water balance indicated there were between 24 GL/year (modelled) and 54 GL/year (accounted) of unspecified or unattributed losses from the system. In the reach between Cotswold and the Balonne River at Weribone the unspecified and unattributed losses averaged 51 GL/year and 155 GL/year respectively and from Weribone to St George the unspecified and unattributed losses averaged 36 GL/year and 120 GL/year respectively.

Within the Condamine Balonne ROP, an allowance is made in the calculation of allocations to medium and high priority users within the Chinchilla Weir WSS for transmission losses of 50% of the release volume.

Surface-groundwater interactions have an impact on the losses observed within a river system. Groundwater extraction can have a significant influence on the volumes of water that may be lost from a river system. The CSIRO (2008) study categorised reaches of the Condamine-Balonne system as ‘losing’ or ‘gaining’ reaches with respect to surface-groundwater connectivity. The CSIRO classified the reach from Chinchilla to Cotswold as a losing reach, Cotswold to Weribone as a gaining reach and Weribone to St George and neither a gaining nor a losing reach. From this information it is unclear as to whether there is likely to be significant net losses of additional discharge to groundwater in this reach of the Condamine-Balonne. The CSIRO (2008) also concluded that while there was a modelled

10 GL/year loss to groundwater within the region there would be a minor impact on streamflow overall and on end-of-system flows.

From this information on transmission losses and groundwater-surface water interactions, it is unclear as to the likely proportion of additional discharges to the system that may be lost due to these factors. The IQQM model developed within the ROP framework incorporates the best estimates of losses from the system. Assessment of additional discharges within this model provides the best indication of the likely volumes of discharges lost from the system.

Due to the complex triggers and operating rules associated with the un-supplemented water allocations within this reach of the Condamine-Balonne system the ability of these users to access water is best assessed using the IQQM model developed for the system. DERM has provided access to this model and the results of the analysis are provided in Volume 5, Attachment 24.

Preliminary IQQM modelling has been undertaken to determine a range of discharge regimes that will comply with the conditions of the Condamine-Balonne ROP and minimise the potential impact on flow regimes. The simplest discharge regime modelled includes a constant discharge at either of the two Condamine River discharge locations. It was determined that constant daily discharges of up to 65 ML/d (23,725 ML/year) could be released from the downstream Condamine River discharge location and comply with the requirements of the ROP for Environmental Flow Objectives and Water Allocation Security Objectives, which are designed to ensure acceptable environmental and water security outcomes. The preliminary IQQM modelling indicates that the release of water within the acceptable parameters of the ROP at either of the proposed discharge locations on the Condamine River will improve the reliability of supply to stock and domestic users, Condamine Town Water Supply and some supplemented irrigation users from the Chinchilla Water Supply Scheme. These users will benefit from releases.

The IQQM modelling also notes that improvements to the supply for users from the St George Water Supply Scheme may also result but this has not been quantified at this stage. The IQQM model supplied for use by the DERM extends only to the upstream extent of the ponded area of Beardmore Dam. An IQQM model was not available beyond this location.

The CSIRO (2008) provided an assessment of the impact that water resource development has had on the nationally important wetlands on the Balonne River floodplain. It concluded that the average period between flood events to these wetlands had increased by about 5 months and had reduced the average flood volume by about five months (or nearly 22 percent) (CSIRO, 2008, p.ii). The study notes that "the number of years in which Back and Clear lakes (part of the Narran Lake Nature Reserve Ramsar site) provide optimal waterbird feeding habitat has been reduced by over 60 percent and the number of years in which Narran Lakes provides optimal waterbird feeding habitat has been reduced by more than 50 percent. Overall, water resource development has greatly reduced the contribution the Narran Lakes system is able to make to the status of the lakes' waterbird population."(CSIRO, 2008, p.ii)

This assessment would indicate that the addition of discharges of appropriate timing and quality may improve the hydrologic condition of the Narran Lakes system.

7.7.3 Surface water users

Surface water users are relatively well documented within the Condamine Balonne system as part of the water resource planning framework. Within the Condamine Balonne region there are:

- Four water supply schemes – the Upper Condamine, Chinchilla Weir, Maranoa River and St George

- Four water management areas – Upper Condamine, Condamine-Balonne, Lower Balonne and Tributaries
- Water licences issued for harvesting overland flow in the Lower Balonne Water Management Area.

The discharge location associated with the current Talinga WTF discharges at the downstream extent of the Chinchilla Weir Water Supply scheme. The proposed discharge location on the Condamine River is located just downstream of the Chinchilla Weir Water Supply scheme within an unregulated reach of the Condamine River.

CSIRO in the Water availability in the Condamine-Balonne report (2008) summarise the water users within the catchment. Table 7.13 is reproduced from the CSIRO (2008) report.

Table 7.13 Summary of surface water sharing arrangements within the Condamine-Balonne region in Queensland (Source: CSIRO, 2008, Water availability in the Condamine-Balonne)

Water products	Priority of access	Allocated entitlement		
		Regulated Water Supply Schemes[1]	Unregulated Water Management Areas[2]	Nebine Water Resource Plan
			ML	
Total licensed (long-term) extraction limit		123,394	444,578	2039
Annual volumetric extraction limit		123,394	not specified	3209
Supplemented access	high priority	7,552	21,522[8]	0
	risk [7]	8,245		
	medium priority	107,597		
Domestic and stock			0[3]	0[3]
Unallocated			0	1000
Unsupplemented access	low priority		1,597,574	2039
Water harvesting of overland flow[4]			868,375	
Substitution of groundwater[5]			not specified	
Environmental provisions			[6]	[6]

[1] Includes Upper Condamine Water Sharing Scheme, Chinchilla Weir Water Supply Scheme, Maranoa River Water Supply Scheme and the St George Water Supply Scheme.
[2] Includes Upper Condamine Water Management Area, Condamine-Balonne Water Management Area, Tributaries Water Management Area and Lower Balonne Water Management Area.
[3] Domestic and Stock allocations have been converted to nominal allocations.
[4] Lower Balonne Water Management Area. This limit is included in the unsupplemented access storage limit.
[5] Upper Condamine Water Management Area. The volume is not specified as it is tied to groundwater use which is not part of the Condamine and Balonne draft Resource Operations Plan, but is constrained by existing infrastructure
[6] Environmental provisions are taken into consideration when setting the conditions of extraction on the entitlement to ensure there is a volume of water available for the environment.
[7] Risk Class A and B as defined in the Condamine and Balonne Resource Operations Plan
[8] The volume of Cooby Dam and Warra Weir is included as an allocation has not yet been assigned to demands from these storages
Source: Queensland Government (2003, 2004).

Within the water supply schemes, supplemented water licences are issued with a priority level from medium to high. Annual allocations for medium priority users remain at zero unless the annual allocation for high priority users reaches 100 percent. The total licensed volume for these allocations is 123.4 GL. (CSIRO, 2008)

Un-supplemented water licences are issued within the four water management areas and access to water is dependant on river flows. In effect access to water outside of the water supply schemes is opportunistic. The total nominal licensed volume for these areas is 444.58 GL and the maximum annual volumetric limit is 1597.6 GL (CSIRO, 2008). Water can be accessed over multiple years resulting in the difference between the nominal and the volumetric limits.

Irrigation in the area has expanded rapidly since the 1960s due to the construction of the public water storages including the Jack Taylor Weir at St George (1959), the Chinchilla Weir (early 1970s), Leslie Dam (early 1980s) and Beardmore Dam (1980s). Cotton accounted for 63 percent of the irrigated area

in 2000 and other irrigated crops include winter cereal crops, pastures for fodder and a small area of horticulture. (CSIRO, 2008)

As the proposed downstream discharge to the Condamine River is located downstream of the downstream extent of the Chinchilla Weir Water Supply scheme area water users upstream of this location (including within the Chinchilla Weir Water Supply Scheme (WSS) and Upper Condamine Water Sharing Scheme, and the Upper Condamine Water Management Area) are not likely to be influenced by discharges. Based on the water allocation tables contained within Attachment 10 of the Condamine and Balonne Resource Operations Plan (attached as Appendix I) the un-supplemented water allocations downstream of the proposed discharge location (assumed to be the downstream extent of the Chinchilla Weir WSS) and upstream of the St George WSS are summarised in Table 7.14. Also included in Table 7.14 are un-supplemented water allocations on Yuleba and Tchanning Creek. These users may be impacted by releases from WTF_RCK_01 and WTF_HCK01, respectively.

Inspection of the IQQM model developed by DERM for the mid Condamine River as part of the water resource planning activities indicates that irrigators (in model zone CBU-04) located downstream of the upstream Condamine River discharge location (Talinga WTF) but still within the Chinchilla Weir WSS have a combined water allocation of 726 ML/annum.

Preliminary IQQM modelling indicates that the supply to this group of irrigators will be improved with discharge to the river at the Talinga WTF discharge location. With a constant continuing discharge of 50 ML/day at Talinga, the IQQM model results indicate (for the simulation period 1922 to 1995) that the mean annual diversions to this group will increase from 606 Ml/yr to 783 ML/yr (an increase of 29%). The reliability of supply to this group (as measured by the Supplemented Annual Volume Probability - see section 7.7.1 and Volume 5, Attachment 24 for definition) will increase from 83.5% to 107.5%.

Users with stock and domestic licences along the river downstream of Chinchilla Weir to Weribone will also have improved supplies. The modelling indicates diversions to this group will increase by approximately 40%.

Further details of the IQQM model are provided in Volume 5, Attachment 24.

Table 7.14 Un-supplemented water allocations downstream of the proposed downstream Condamine River discharge location

Zone	Description	AMTD (km)	Flow Condition (ML/d)	Nominal Limit (ML)	Volumetric Limit (ML)
CBU-04	Condamine River – from downstream limits of the Chinchilla Weir Water Supply Scheme downstream to Cotswold gauging station	643.7 – 537.5	Total	6793	13707
			-	1505	1613
			605	348	348
			1210	4660	11435
			3024	280	311
CBU-05	Condamine River – from Cotswold gauging station downstream to the confluence of Dogwood Creek	537.5 – 507.8	Total	1509	2004
			-	4	4
			1728	1505	2000
CBU-06	Balonne River – from the confluence of Dogwood Creek downstream to the confluence of Yuleba Creek	507.8 – 466.8	Total	1638	2928
			-	253	253
			367	1385	2675
CBU-07	Balonne River – from the confluence of Yuleba Creek downstream to Surat Weir	466.8 – 405.2	Total	4531	13976
			-	416	416
			1728	4115	13560
CBU-08	Balonne River – from Surat Weir downstream to the confluence of Cogoon River	405.2 – 359.1	Total	96	96
			-	96	96
CBU-09	Balonne River – from the confluence of Cogoon River downstream to the upstream extent of the St George Water Supply Scheme.	359.1 – 305.0	Total	5400	18710
			-	60	80

Zone	Description	AMTD (km)	Flow Condition (ML/d)	Nominal Limit (ML)	Volumetric Limit (ML)
			1730	5340	18630
Total in Condamine – Balonne Downstream of Chinchilla WSS, Upstream of St George WSS				19967	51421
Yuleba			Total	503	558
			-	78	78
			605	425	480
Tchanning			86	1640	6415

7.7.4 Results and conclusions

This analysis indicates that if additional discharge to the Condamine River were released in a way that enabled un-supplemented water users access to the additional water and up to a level which complied with the environmental flow objectives of the ROP then it would appear that up to 65 ML/day (23,725 ML/year) could be discharged to the Condamine-Balonne system with a resulting benefit to downstream users.

If flows did reach Beardmore Dam (the controlling structure within the St George WSS) then flows would be controlled under the provisions of the supply scheme. Flows would then be released as per the conditions of the ROP.

Based on the fact that the hydrological regime within the Narran Lakes has been significantly altered by existing water resource development within the Condamine Balonne catchment the addition of discharges of appropriate timing and quality may improve the hydrologic condition of the Narran Lakes system and result in a positive benefit to the system.

7.8 Recommended discharge regimes

The analysis of the existing flow regimes in the waterways proposed to receive discharges and the preliminary hydraulic assessment provide the initial basis for the development of acceptable release regimes. There are a number of additional factors that will be considered in the future development and refinement of discharge regimes or have been assessed in other sections of the EIS. These include:

- Potential impacts on the aquatic and riparian ecosystems (Volume 5, Attachment 20)
- The potential impact of the duration of discharges to the watercourses, that is, whether the scheme is proposed over a short term (3 years) or long term (30 years or project life)
- Potential impacts on downstream users (including increased availability of water, potential impacts on water pricing)
- Planning and legislative constraints
- Storage and discharge infrastructure requirements.

The potential impacts on aquatic ecology and water quality are addressed in Volume 5, Attachment 20.

The potential impacts on downstream users from a water availability perspective were addressed in Section 7.7. Potential impacts on water pricing are beyond the scope of this investigation.

7.8.1 Impacts of proposed discharge timeframes

The current associated water management strategy proposes that initially, the primary disposal option will be discharge to watercourses. There will also be some opportunity for water to be distributed for irrigation. Following this initial period it is proposed that increased volumes of produced water be directed to alternative beneficial use and disposal options, reducing the dependence on discharge to watercourses.

If a constant discharge regime is adopted (that is without mimicking no flow periods within the year), to either the Condamine River or the local tributaries, then the potential impacts of this discharge would be considered, on a relative basis, less if the regime was proposed only over the initial period of the water management strategy than if it was proposed to continue over the life of the project.

7.8.2 Impacts on low flow regimes

The assessment of aquatic ecology completed as part of this EIS (Volume 5, Attachment 20) identified changes to the low flow regime as a potential impact to aquatic ecosystems. In order to assess the likely extent of impact to low flow patterns as a result of proposed discharge regimes the IQQM model supplied by DERM was used to produce flow duration curves at locations downstream of the proposed Condamine River discharge locations. Comparison of the resulting flow duration curve with a range of discharge scenarios was compared to the predevelopment model case developed by DERM against which water resource proposals are assessed.

It is noted that the predevelopment case is an artificial model scenario which is constructed by modifying the calibrated IQQM model to remove all water resources development (including infrastructure and extractions) and running the model over the simulation period with historic climatic inputs. Therefore, the flow duration curves produced for the predevelopment case will not compare to those produced from historic streamflow records and presented in Section 7.5.3. Flow duration curves developed based on streamflow data include the impacts of water resource development over time.

In order to minimise the potential impact of discharge to aquatic ecosystems discharge regimes should be developed which mimic the natural flow pattern. Flow duration curves provide a measure as to the extent to which a discharge regime follows the natural flow variability. If the flow duration curve produced with the discharges incorporated follows the shape of the predevelopment curve, particularly in the lower range of flows, then it is likely that the natural variability in the low flow regime is mimicked.

Volume 5, Attachment 24 provides a full description of the IQQM model and results.

7.8.3 Engineering constraints

A discharge proposed at any of the identified locations will require the following infrastructure. Potential constraints on the rate and timing of discharges based on infrastructure requirements are also identified below.

Off stream storages

Any proposed discharge regime other than a discharge to a local or adjacent watercourse at an equivalent rate to the production of treated water will require storage infrastructure. The size of this

storage will depend of the adopted discharge regime and will need to be sized through the construction of a water balance model.

It may be possible to use/construct a storage that will be used for an alternative purpose in the future, such as brine storage, as part of the water management strategy and negate the need to construct a purpose built treated water balance storage. Alternatively, existing irrigation infrastructure may be used for storage and transfer of water for beneficial reuse. This will be addressed as part of the on-going development of the associated water management strategy.

Cost benefit assessment of the storage requirements is likely to constrain or influence the adopted discharge regime.

Discharge infrastructure

The following infrastructure will be required at any proposed discharge location:

- Discharge pipeline and associated pumps and valves (from the treatment facility to the discharge location)
- Bank and bed stabilisation and protection works (such as geotextiles, gabion and/or rock protection)
- Flow monitoring instrumentation.

Depending on the rate and velocity of discharge and the local watercourse conditions the design of the discharge infrastructure may include:

- A spill-over weir arrangement – to distribute flow and reduce velocity
- A stilling basin – to minimise velocity
- A distributed discharge or number of outlets along a selected reach – to distribute flow and reduce velocity.

In stream protection works

An assessment of the downstream stream conditions will be required at a proposed discharge location in order to identify the need for in stream protection works downstream of the discharge location. A detailed survey of the watercourse in the vicinity of the proposed discharge location should be undertaken and as part of an on-going monitoring program visual inspection and cross sectional survey should be undertaken to assess the need for any future in-stream protection works.

Stream protection works may include revegetation, gabion or rock protection and channel shaping or modification works. Appropriate approvals and permits would need to be obtained where necessary.

7.8.4 Cumulative impacts of discharge

There are a number of existing and proposed CSG operations within the region which have to potential to adopt stream discharge as an associated water management option. The Origin operated Talinga WTF proposes to discharge up to 35 ML/day to the Condamine River. A discharge regime for this project may be adopted which incorporates additional discharge at this location as well as at the Condamine discharge location downstream,

The IQQM model developed by DERM for the Condamine River provides a tool for the assessment of potential cumulative impacts of discharge to the Condamine River system. Preliminary IQQM modelling (Volume 5, Attachment 24) indicates that a discharge regime which increases the discharge

rate from the Talinga WTF location to a total of 50 ML/day constant discharge or a combined discharge regime from both Condamine River discharge locations (35 ML/d from Talinga and 30 ML/d downstream) will meet the environmental flow objectives of the ROP.

7.8.5 Possible discharge regimes

Four possible discharge regime options are presented below. These have been developed based on the characteristics of the existing environment and in consideration of the factors identified above. These options are listed in order of decreasing potential impacts on the environment based on the factors discussed above.

- OPTION 1 - Constant discharge
- OPTION 2 - Constant discharge with a period of no flow
- OPTION 3 - Discharge factored based on average seasonal flows
- OPTION 4 - Discharge triggered by watercourse flows.

Each of these options is presented below and advantages and disadvantages provided. The level of impact is assessed based on the risk assessment methodology outlined in Volume 1, Chapter 4 of the EIS and provided in Appendix J.

Where appropriate discharge regimes have been modelled within the IQQM river system model developed by the DERM as part of the Condamine Balonne ROP. Access to the IQQM model was provided for use in this study and a complete description of the model modification and results is provided in Volume 5, Attachment 24.

Option 1 - Constant discharge

Description

- Constant discharge at one or several discharge locations
- Preliminary IQQM modelling indicates that up to 65 ML/day discharged from the downstream Condamine River discharge location or up to 50 ML/d discharged from the Talinga WTP discharge location would comply with the Environmental Flow Objectives of the ROP
- However, inspection of the flow duration curves from the IQQM model indicate there is a significant alteration in the low flow regime due to the constant discharge
- Discharge to watercourses other than the Condamine River could not be modelled in IQQM at this time. Analysis of the flow exceedance curves presented in Section 7.5.4 and Appendix G indicates that a constant discharge to any of the tributary discharge locations would alter the low flow pattern for at a minimum between 55% and 70% of the time. This percentage of time the flow pattern is altered would increase as the rate of discharge is increased.

Advantages

- Easy to manage from an operational perspective
- Minimal infrastructure required.

Disadvantages

- Alteration of natural, intermittent or ephemeral nature of watercourses (both Condamine River and tributary discharge locations)

- Potential impact on aquatic and riparian ecosystems due to alteration of low flow regime.

Mitigation

- Short term (3 years) duration of discharges would reduce the potential impacts on aquatic and riparian ecosystems
- Implementation of alternative disposal/beneficial uses to reduce the volume of storage/discharge required
- Limit discharge volumes to that which comply with the environmental flow objectives of the ROP.

Impact

- Unmitigated this regime is assessed as a severe risk
- Mitigated this regime is assessed as a high risk.

Option 2 - Constant discharge with a period of no flow

Description

- Constant discharge with a no discharge period to coincide with historic seasonal periods of low flow volumes.
- Length and timing of proposed no discharge periods are provided in Table 7.15.

Table 7.15 Proposed periods of no discharge

Watercourse	WTF	% time there is no flow	Proposed period of no discharge
Condamine River	WTF_CON_01 (and aggregated flows)	20% (long term gauge records) 5% (pre development conditions from IQQM model)	August to October
Yuleba Creek	WTF_RCK_1a	65%	March to October
Unnamed	WTF_MEL_01	70%	March to October
Tchanning Creek	WTF_HCK_01	65%	March to October
Woleebee Creek	WTF_WOL_01	55%	March to October
Dulacca Creek	WTF_BYM_01	65%	March to October
Unnamed North (Gilbert Gully)	WTF_GIL_01	67%	March to October

- This discharge regime was modelled in the IQQM model for the Condamine River. Results of the modelling indicate that a release of 50 ML/day from the Talinga discharge location or a 65 ML/day release from the downstream Condamine River discharge location with a period of 3 months of no release will comply with the environmental flow objectives contained within the ROP

- Analysis of the flow duration curves from the IQQM model at various locations downstream of the discharge location indicate that while the flow duration curve produced including the additional discharge does not mimic the exact predevelopment flow pattern there is an observed period of no flow which correlates with that observed in the predevelopment case. There is still some alteration of the low flow regime as compared to the predevelopment case
- Discharge to watercourses other than the Condamine River could not be modelled in IQQM at this time. Analysis of the flow exceedance curves presented in Section 7.5.4 and Appendix G indicates that if a discharge was ceased for between 55% and 70% of the time then the periods of low flow and ephemeral nature of the systems could be maintained.

Advantages

- Relatively easy to manage from an operational perspective
- Maintains a period of natural flow conditions during low flow periods of the year.

Disadvantages

- Requires infrastructure and/or alternative disposal or reuse options to manage produced water during no discharge periods
- Requires higher discharge rates during discharge periods to manage equivalent volumes of produced water (only required if alternative disposal or beneficial use alterative are not implemented)
- Potential impact on aquatic and riparian ecosystems if flow regime does not mimic natural conditions. This potential is significantly reduced as compared to Option1.

Mitigation

- Short term (3 years) duration of discharges would reduce the potential impacts on aquatic and riparian ecosystems
- Implementation of alternative disposal/beneficial uses to reduce the volume of storage/discharge required
- Limit discharge volumes to that which comply with the environmental flow objectives of the ROP.

Impact

- Unmitigated this regime is assessed as a high risk
- Mitigated this regime is assessed as a medium risk.

Option 3 - Discharge factored based on average seasonal flows

Description

- Monthly or seasonal discharge volumes factored based on the historical seasonal flow pattern
- Discharge patterns for the Condamine River based on the average flow volumes recorded at Chinchilla gauge is provided in Table 7.16.

Table 7.16 Seasonal discharge pattern based on annual average volume

	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
Condamine River	14%	21%	7%	10%	13%	7%	5%	3%	2%	1%	7%	11%

- This discharge regime was not modelled in the IQQM model for the Condamine River at this stage. However, analysis of the model results for Option 2 above indicate that there may be opportunity to increase discharge rates during wet or medium to high flow periods to better mimic the predevelopment flow duration curve.
- Release rates would be developed to comply with the environmental flow objectives contained within the ROP and to produce a flow regime which mimics the predevelopment case
- Change in release rate could be triggered by a flow threshold within the River or on a maximum period of release conditions
- Additional hydrologic modelling and streamflow monitoring would be required in order to develop a similar discharge regime for tributary locations.

Advantages

- Attempts to mimic natural flow variations based on long term historic flow patterns
- Easier to manage from an operational perspective than a flow triggered regime.

Disadvantages

- Requires significant infrastructure and operational management procedures to manage discharge volumes
- Potential impact on aquatic and riparian ecosystems if flow regime does not mimic natural conditions. This potential is significantly reduced as compared to Option1 and Option 2.

Mitigation

- Short term (3 years) duration of discharges would reduce the potential impacts on aquatic and riparian ecosystems
- Implementation of alternative disposal/beneficial uses to reduce the volume of storage/discharge required
- Limit discharge volumes to that which comply with the environmental flow objectives of the ROP.

Impact

- Unmitigated this regime is assessed as a medium risk
- Mitigated this regime is assessed as a low risk.

Option 4 - Discharge triggered by watercourse flows

Description

- Discharge triggered by flows within the watercourse and volumes controlled by adherence to ROP
- No release for up to 30% of the water year based on stream flows falling below a minimum flow in the river at the model node just upstream of Talinga. This minimum flow will be slightly more than the maximum release flow required to supply irrigators with scheme allocation within the regulated section of the Chinchilla Weir
- This discharge regime was modelled in the IQQM model for the Condamine River. Preliminary modelling indicates that up to 100 ML/day could be released from the Talinga discharge location if releases ceased if river flow falls below 6 ML/day. However, no release conditions were capped at a maximum of 30% of the water year, that is, if flows were below 6 ML/day for greater than 30% of the water year discharge could occur once no release conditions had occurred for 30% of the water year
- Analysis of the resulting flow duration curve for this scenario indicates that the discharge scenario generally mimics the predevelopment flow regime. See Volume 5, Attachment 24 for details of IQQM model results
- Additional hydrologic modelling and streamflow monitoring would be required in order to develop a similar discharge regime for tributary locations.

Advantages

- Mimics natural flow variations based on long term historic flow patterns
- Meets the requirements of the WRP and ROP.

Disadvantages

- Requires significant infrastructure and operational management procedures to manage discharge volumes.

Mitigation

- Short term (3 years) duration of discharges would reduce the potential impacts on aquatic and riparian ecosystems
- Implementation of alternative disposal/beneficial uses to reduce the volume of storage/discharge required
- Limit discharge volumes to that which comply with the environmental flow objectives of the ROP.

Impact

- Unmitigated this regime is assessed as a medium risk
- Mitigated this regime is assessed as a low risk.

7.9 Recommendations

Based on the outcomes of this investigation the following is recommended:

- Discharge to the Condamine River is preferred with discharges to the other proposed locations limited to opportunistic releases during flow periods
- Additional hydrologic modelling and streamflow monitoring would be required in order to develop a similar discharge regime for tributary locations
- A discharge regime based on Option 3, a discharge factored on averaged seasonal flows or Option 4, discharge triggered by watercourse flows, is to be adopted
- Discharge volumes to be limited to 50ML/day constant release with a 3 month period of no release from August to October from the Talinga discharge location or an alternative release arrangement which complies with the ROP to achieve a medium impact
- Discharge volumes to be limited to 100 ML/day discharge from the Talinga discharge location with a no flow condition for up to 30% of the water year triggered by a flow of less than 6 ML/day within the Condamine River or an alternative release arrangement which complies with the ROP and produces a flow duration pattern which mimics the predevelopment flow regime to achieve a low impact
- Design of discharge infrastructure be undertaken such that localised velocity and scour is minimised and appropriate mixing of discharge is achieved
- An ongoing program of monitoring be developed which includes regular inspection of discharge locations and cross section survey as well as monitoring of aquatic and riparian ecosystems
- Investigation and implementation of alternative disposal/beneficial uses be undertaken to reduce the volume of storage/discharge required.

8. Failure impact assessment – water storage facilities

8.1 Legislation and guidelines

Dam safety in Queensland is currently controlled under the *Water Act 2000, and the Water Supply (Safety and Reliability) Act 2008.* Previously, this was regulated by the *Water Resources Act 1989*, where safety requirements were incorporated into the conditions of waterworks licences issued under that act.

The *Water Supply (Safety and Reliability) Act 2008* defines the way in which dams are defined and regulated. The legislation puts the onus on particular dam owners to asses the impacts of dam failure by way of completing a 'dam failure impact assessment' if the dam meets criteria requiring assessment. The result of this assessment determines if the dam is referrable under the act. If the dam is classified as referrable, the chief executive of the Department of Natural Resources and Mines (DNRM) (now within the Department of Environment and Resource Management [DERM]) is responsible for the regulation of the referable dams in Queensland .

If a dam contains hazardous waste it is not a referrable dam, and will be governed by the provisions of the *Environmental Protection Act 1994* instead of the *Water Supply Act.*

The former EPA (now also within DERM) has released two information sheets relating to the definition and management of hazardous dams for assessment:

- Determining Dams Containing Hazardous Waste
- Managing Dams Containing Hazardous Waste.

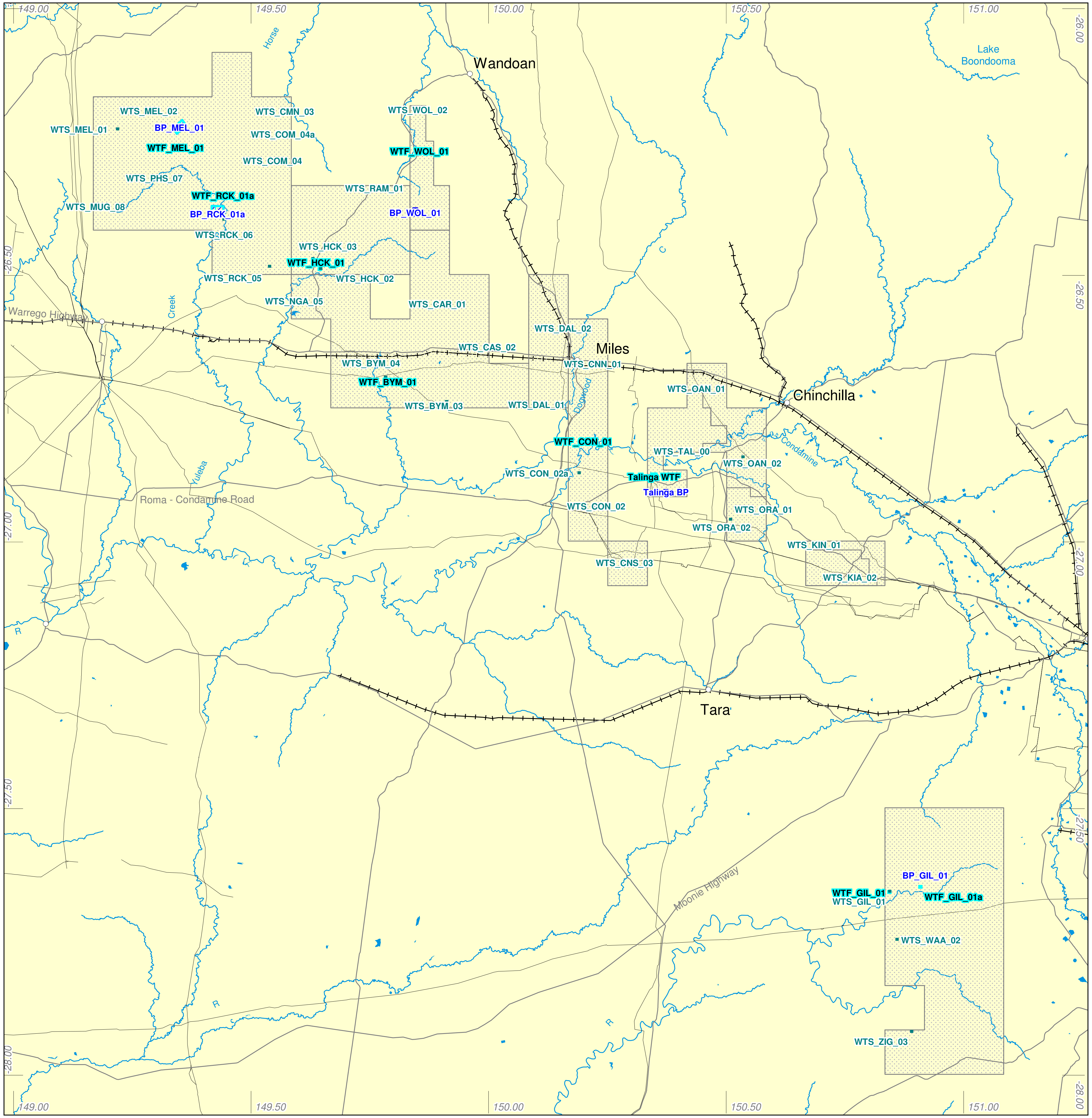
These guidelines are to be superseded (still in draft) by:

- Manual for Assessing Hazard Categories and Hydraulic Performance of Dams.

The Manual for Assessing Hazard Categories and Hydraulic Performance of Dams is currently in draft format and is still pending approval for release as a final document. Therefore, this study will consider the contents of this draft manual, and the two approved information sheets.

Under the *Water Act 2000* the Chief Executive of DERM can require a dam failure impact assessment be carried out on any dam. The Terms of Reference for the Project requires dam assessments be carried out for all project storages that meet the required criteria. The process for dam assessment is illustrated in Figure 8.2.

The water transfer station, water treatment facility and brine pond storages as shown in Figure 8.1 have been assessed against the process defined in Figure 8.2. Treated associated water storages are not proposed in the current project scope.



LEGEND

- Town
- Existing railway
- Road
- Major watercourse
- Water treatment facility
- Water transfer station
- Brine pond
- Walloons gas fields development areas

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Rev	Date	Revision Description	ORIG	CHK	ENG	APPD
0	09/12/2009	Issued for use	SF	DH		

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Figure 8.1 Water Storage Locality Plan

Project No: 301001-00448 | Figure: 00448-00-EN-DAL-2227 | Rev: 0

Compiled by BRISBANE INFRASTRUCTURE GIS SECTION

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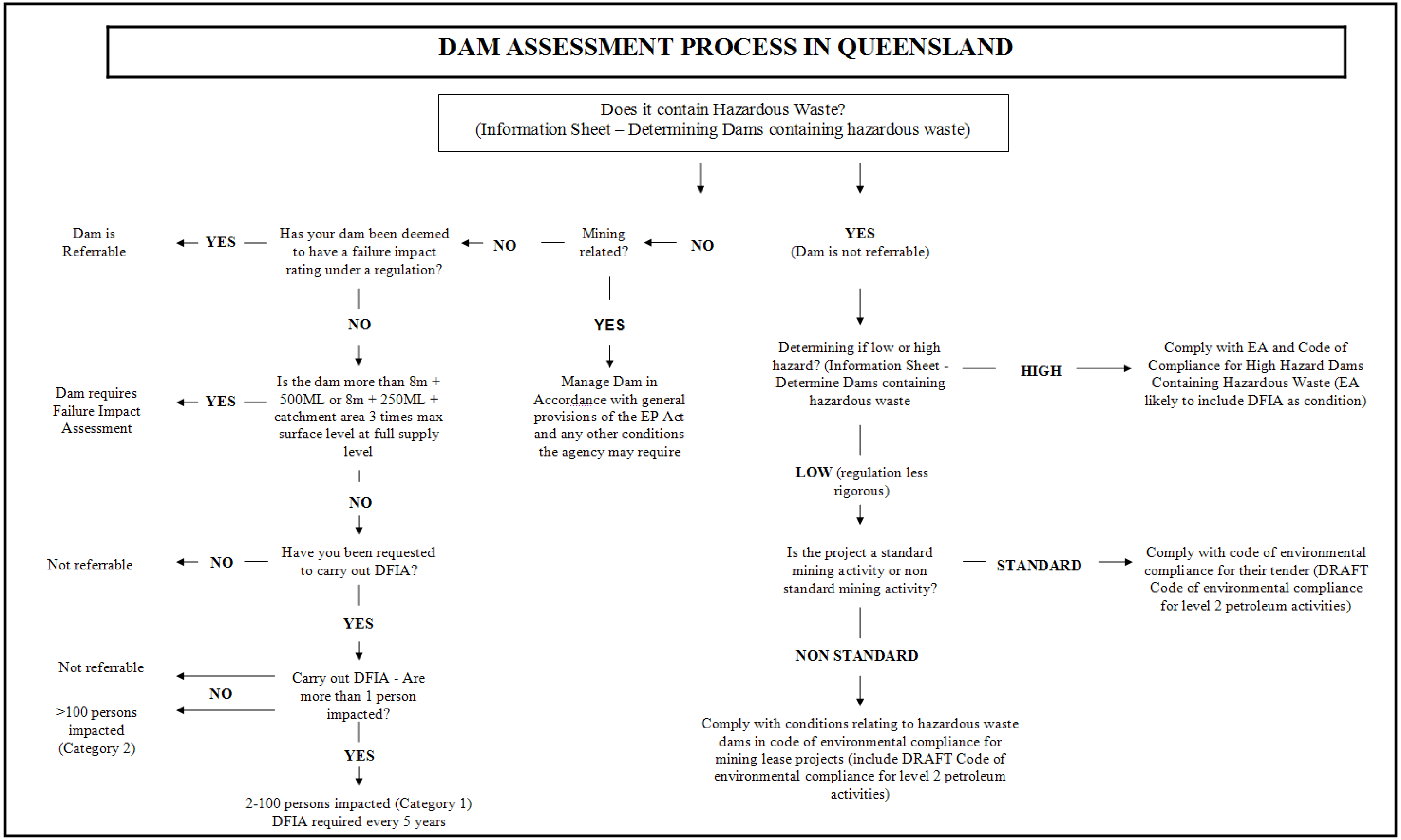

Figure 8.2 Dam Assessment Process in Queensland

8.1.1 Determining dams containing hazardous waste

The information sheet for 'Determining Dams Containing Hazardous Waste' (EPA 2002) outlines the steps to determine if a dam contains hazardous waste associated with a mining and mineral processing activity. Two questions need to be answered to determine if a dam is classified as hazardous or contains hazardous waste:

Does the dam contain a hazardous waste?

A dam is considered to contain a hazardous waste if the dam's contents exceed the criteria outlined in Table 8.1.

Table 8.1 Hazardous waste criteria

Parameter	Liquor	Total solids
Arsenic	1.0 mg/L	500 mg/kg
Boron	5.0 mg/L	15 000 mg/kg
Cadmium	10 µg/L	100 mg/kg
Cobalt	1.0 mg/L	500 mg/kg
Copper	1.0 mg/L	5 000 mg/kg
Mercury	2 µg/L	75 mg/kg
Nickel	1.0 mg/L	3 000 mg/kg
Zinc	20 mg/L	35 000 mg/kg
Chloride	2 500 mg/L	-
Sulphate	1 000 mg/L	-
Fluoride	2.0 mg/L	-
Cyanide	10 mg/L	2 500 mg/kg
pH	Between 4 and 8	Net acid generation of pH<4

Source: Determining Dams Containing Hazardous Waste, EPA (2002).

Based on the information provided by Australia Pacific LNG (Origin, Aug, 2009) on the potential water quality of the brine, permeate and associated water, most water quality values are likely to be within the acceptable ranges. One parameter that falls outside the acceptable guideline range is pH.

It is likely that the pH will be within the range of 9.5-10.2 for the brine ponds and should remain fairly stable as the total dissolved solids (TDS) increase in the brine ponds. Similarly, the associated water is likely to have an average pH of 9.3 (WorleyParsons, 2009). Therefore the water transfer station, water treatment facility and brine pond storages are classified as containing a hazardous waste based on the pH range.

The permeate pH is predicted to be within the range of 6.5-8.5, with a typical value of 7.7 (WorleyParsons, 2009). Therefore, permeate storage ponds (if required) would not be expected to be defined as hazardous, however based on the higher range values have potential to fall into the hazardous category.

How to determine a high hazard dam?

The hazard assessment criteria are used to determine if a proposed dam is a high hazard dam. A high hazard dam is one which poses a significant risk to the environment and is regulated accordingly. From the guideline, a dam is high hazard only if it contains hazardous waste and at least one of the following occur:

a) In the event of dam failure or overflow, the dam's content would have one of the more of the following actions:

    i) Flow to a sensitive or commercial place

    ii) Flow to a riverine area containing permanent water

    iii) Contaminate a water supply for human consumption

    iv) Contaminate a water supply for stock.

b) The dam is located within a:

    i) Declared catchment or sub artesian area

    ii) Watercourse and the dam's surface area exceeds one hectare.

c) The dam has a surface area greater than two hectares.

Several of the above parameters are likely to be triggered by each of the proposed storages associated with the project. In particular, all of the proposed water transfer station, water treatment facility and brine pond storages are proposed to have surface areas of greater than two hectares and are therefore classified as high hazard dams containing hazardous waste.

8.1.2 Managing dams containing hazardous waste

The information sheet for Managing Dams Containing Hazardous Waste provides details of the former EPAs system for managing dams that contain hazardous waste that are directly associated with mining and mineral processing activities. The information sheet provides details on the requirements for lodging an environmental authority (EA) for a mining and mineral processing activity.

For high hazard dams containing hazardous waste the applicant is required to comply with the Environmental Authority Streamlined conditions for high hazard dams containing high hazard waste, and the Code of Compliance for High Hazard Dams Containing High Hazard Waste (EPA, 2002).

8.1.3 Manual for assessing hazard categories and hydraulic performance of dams

This manual supersedes the two information sheets discussed in Sections 8.1.1 and 8.1.2, namely Determining Dams Containing Hazardous Waste and Managing Dams Containing Hazardous Waste. However, at this stage the document is still in draft and has not been released as final.

From this manual, hazard category is defined by the assessment of hazard potential. If a dam should be regulated, it is assessed by the hazard category and is based on height of man made embankment and the hazard category based on contaminant concentrations and minimum volume.

Hazard category – based on the assessment of hazard potential (failure event scenarios)

Two possible failure scenarios are considered, failure to contain and dam break. The potential environmental harm resulting from each scenario is assessed resulting in a high, significant or low, hazard category. Where dams have a capacity that is greater than 250ML, dam failure impact must be assessed to classify hazard in accordance with the manual. The project storages will have capacities ranging from approximately 170ML to 8300ML. Table 8.2 and Table 8.3 show the failure to contain and dam break scenario assessment criteria.

Table 8.2 Failure to contain

Environmental harm	**Failure to contain hazard category**		
Categories of harm	**High**	**Significant**	**Low**
General environmental	Location such that harm to a significant environment value is likely or serious environmental harm is possible. Such a value might include the presence of protected or endangered flora or fauna.	The environmental value is lesser significance and harm is possible but not likely, or material environmental harm is possible	No environmental values of significant, or only trivial environmental harm is possible
Loss or harm to humans	Consumption of contaminated waters by humans with consequent loss or harm is likely.	Consumption of contaminated waters by humans with consequent loss or harm is possible.	No contamination of waters used for human consumption expected.
Loss of stock	Consumption of contaminated waters by stock with consequent loss or harm is likely.	Consumption of contaminated waters by stock with consequent loss or harm is possible.	Contaminated water not available to stock or no harm expected from consumption.
General economic loss	Serious harm to communities, industrial, commercial or agricultural facilities, important utilities, or other water resources in the failure path.	Material harm to industry, secondary roads, minor railways, public utilities or other water resources in the failure path.	Trivial harm to environmental values such as environmental nuisance arising from minor spills.

Source: 2.4 Manual for Assessing Hazard Categories and Hydraulic Performance of Dams, EPA (draft)

Table 8.3 Dam Break Scenario

Environmental harm	Failure to contain hazard category		
Categories of harm	**High**	**Significant**	**Low**
General environmental	Location such that harm to a significant environmental value is likely or serious environmental harm is possible. Such a value might include the presence of protected or endangered flora or fauna.	The environmental value is of lesser significance and harm is possible but not likely, or material environmental harm is possible.	No environmental value of significance or only trivial environmental harm is possible.
Loss or harm to humans	Location such that people are routinely present in the failure path and if present, loss or harm is likely. Consumption of contaminated waters by humans with consequent, loss or harm is likely.	Location such that people are routinely present in the failure path and if present, loss or harm is possible. Consumption of contaminated waters by humans with consequent, loss or harm is possible.	Location such that people are not routinely present in failure path. No contamination of waters used for human consumption expected.
Loss of stock	Location of stock such that loss of stock is likely. Consumption of contaminated waters by stock with consequent loss or harm is likely.	Location of stock such that loss of stock is possible. Consumption of contaminated waters by stock with consequent loss or harm is possible.	Stock not in path of dam break flood. Contaminated water not available to stock or no harm expected from consumption.
General economic loss	Serious harm to communities, industrial, commercial or agricultural facilities, important utilities, or other water resources in the failure path.	Material harm to industry, secondary roads, minor railways, public utilities or other water resources in the failure path.	Trivial harm to environmental values such as environmental nuisance arising from minor spills.

Source: 2.4 Manual for Assessing Hazard Categories and Hydraulic Performance of Dams, EPA (draft)

Hazard categories as defined by the dam failure impact assessment are shown in Table 8.8 to Table 8.15.

Hazard category – based on height of man-made embankment

A dam is a regulated dam if it incorporates a man-made embankment and the height of that embankment is greater than 8m. Proposed storage heights for the water treatment facility and brine pond storages are 4m and 3m respectively. As a result, subject to confirmation following the

completion of detailed design work, no project storage is expected to be a regulated dam on the basis of height.

Hazard category – based on contaminant concentrations and minimum volumes

While the dams are assessed against failure to contain and dam break scenarios as summarised in Table 8.2 and Table 8.3, a dam is considered regulated if it satisfies either of the following:

- The dam contains contaminants at greater concentrations than shown in Table 8.1 from the 'Guideline for Determining Dams Containing Hazardous Waste'
- A dam has a crest volume greater than 2.5ML.

In the Manual for Assessing Hazard Categories and Hydraulic Performance of Dams, the following contaminants (shown in Table 8.1) have been added or adjusted from Table 1 Acceptance Criteria in the information sheet for Determine Dams Containing Hazardous Waste, (EPA, 2002) for defining if a dam contains a hazardous waste.

Table 8.4 Contaminant concentrations

Contaminant	Liquor	Total solids
Lead	0.5 mg/L	1,500 mg/kg
Selenium	50 µg/L	150 mg/kg
pH	5 to 9 inclusive	-
Salinity (conductivity)	4000 µs/cm	-

Salinity has been added as a contaminant. The salinity of the brine ponds is predicted to be greater than 45,000 µs/cm (WorleyParsons, 2009) and hence the brine ponds would be triggered as hazardous, based on both the salinity and pH trigger values.

The water transfer station and water treatment facility storages are predicted to comprise of an average salinity of approximately 4000mg/L TDS. The electrical conductivity for the groundwater within the Walloons coal measure is between 2000-10,000 µs/cm, and can be higher than 30,000 µs/cm in some areas (PB, 2004). Therefore, these storages have potential to be classified as containing a hazardous waste based on these new guidelines.

The salinity of the permeate is predicted to be approximately 130 µs/cm. Therefore, the permeate storage ponds (should they be included in the Project) would not be classified as hazardous based on the inclusion of conductivity in the new manual.

The manual shows an adjusted acceptable range for pH from between 4 and 8 to between 5 and 9. It is likely that the water transfer station, water treatment facility and brine pond storages will fall outside of this range. However, under the new guidelines the permeate pH would fall within the acceptable range.

Therefore based on the revised manual, the water transfer station, water treatment facility and brine pond storages will be regulated dams. A regulated dam must comply with EPA conditions including certified annual inspections and reports.

Hydraulic performance criteria for regulated dams

The hydraulic performance criteria for regulated dams are reliant upon hazard ratings determined by the failure impact assessments carried out for each water treatment facility storage and brine pond. The failure impact assessment is summarised in Sections 8.2 and 8.3.

An overview of the resultant hydraulic performance criteria for the water treatment facility and brine pond storages assessed in the failure impact assessments is provided in Section 8.3.11.

8.2 Failure impact assessment

Failure of water storage structures can result from a number of causes, many of which are inherently dependant on the environment in which the relevant water storage is located. Similarly, the intended use and design of the storage facility may also influence the avenues in which dam failure can occur. Typically for the storages investigated as part of this study, failure may result from (but is not limited to) the following prominent causes;

- Overtopping (and associated erosion)
- Seepage (either through the dam or along internal conduits)
- Slope embankment slides
- Equipment malfunction
- Structural damage
- Foundation failure
- Sabotage
- Seismic activity. (DNRM 2002)

These causes of dam failure may result in an uncontrolled release of the contents of the dam into the surrounding environment. Typically a dam is considered to have failed when there is a partial or total physical collapse of the dam (including slumping, partial erosion due to overtopping) or when a section of the dam has been removed due to foundation weakness. (DNRM 2002) Similarly, equipment malfunction may also result in an uncontrolled release of the dam contents.

For the purposes of this assessment, dam failure statistics and impacts have been based on hypothetical dam embankment breach. The parameters and methodology used to determine the breach characteristics and associated flows are discussed in Section 8.2.4.

8.2.1 General storage information

Given the numerous tenement areas and associated facilities within the Australia Pacific LNG project area, a number of different water transfer station, water treatment facility and brine pond storages have been proposed across the tenement areas. Table 8.5 summarises the proposed storage volumes and physical characteristics of each of the storage facilities. Failure impact assessments have not been undertaken for water transfer stations because their storage volumes do not trigger the need for a failure impact assessment. A brief discussion of the water transfer stations is included in Section 8.3.9.

Table 8.5 General storage details

Storage name	Approximate location (Lat/Long MGA 94)	Approximate capacity (ML)	Purpose
WTF_MEL_01	149.349254, -26.221573	800#	Water treatment facility
BP_MEL_01	149.349254, -26.221573	4100*	Brine pond
WTF_RCK_01a (alternate to WTF_MEL_01)	149.429316, -26.376904	800#	Water treatment facility
BP_RCK_01 (alternate to BP_MEL_01)	149.429316, -26.376904	4100*	Brine pond
WTF_WOL_01	149.854757, -26.266031	400#	Water treatment facility
BP_WOL_01	149.845379, -26.381051	5200*	Brine pond
WTF_HCK_01	149.644659, -26.488601	200#	Water treatment facility
WTF_BYM_01	149.787911, -26.698711	400#	Water treatment facility
WTF_CON_01	150.199356, -26.810285	800#	Water treatment facility
Talinga WTF (existing facility upgrade)	-	800#	Water treatment facility
Talinga BP (existing facility upgrade)	-	8300*	Brine pond
BP_GIL_01	150.919456, -27.624158	4100*	Brine pond
WTF_GIL_01a	150.909077, -27.646833	800#	Water treatment facility
WTF_GIL_01 (alternate to WTF_GIL_01a)	150.84394, -27.656772	800#	Water treatment facility

* denotes volumes calculated from proposed surface areas and design depths (refer Section 8.2.2)
# denotes 10-day storage volume

8.2.2 Assessment methodology

Three analytical techniques are provided by the 'Guidelines for Failure Impact Assessment of Water Dams' (DNRM 2002). These include:

- Two Dimensional Flow Analysis
- Simplified Assessment,
- Comprehensive Assessment.

These guidelines state that a simplified assessment may be justified where there is little doubt as to the population at risk from dam failure. A review of areas downstream of the proposed water storage locations reveals only a limited number of inhabited residential buildings, with many predicted to be well clear of the predicted flow path of the hypothetical dam failure. As a result, and given the conceptual nature of the design of the water storages at this stage of the project, it has been

concluded that a simplified approach coupled with a two dimensional hydraulic flow analysis is appropriate in relation to the water treatment facility and brine pond storages.

Given that a series of regional two dimensional hydraulic models have been developed as part of a separate flood study investigation, it was deemed appropriate to combine the simplified assessment and the two dimensional flow analysis analytical techniques for this assessment to facilitate a more detailed analysis of flow behaviour resulting from the hypothetical dam breach.

Selected aspects of the comprehensive assessment have also been included in the methodology, where considered necessary, to facilitate the determination of dam breach hydrographs for application into the TUFLOW hydraulic models.

8.2.3 Derivation of dam volumes and surface areas

Due to the conceptual nature of the dam designs for both the water treatment facility and brine pond storages, limited information with respect to design characteristics is available.

WorleyParsons has however undertaken additional analysis on the proposed dams to derive approximate storage volumes for brine ponds (based on proposed surface areas) and approximate surface areas for the water treatment facility storages (based on provided volumes).

This involved the adoption of the following assumptions to facilitate these calculations:

- All ponds have 3H:1V batter slope
- All ponds have a flat base
- All ponds are rectangular
- All ponds have a length to width ratio of 2:1

With the aforementioned assumptions, volumes and surface areas of brine ponds and water treatment facility storages respectively were able to be determined by way of the end area method.

At the request of Australia Pacific LNG, a 10 day total storage volume for the water treatment facility storages was adopted. For example, water treatment facilities with a capacity of 80ML/day have a total volume for this analysis of 800ML. Table 8.5 and Table 8.6 summarise the total storage volumes for each proposed storage facility.

8.2.4 Determining dam failure flow rates and hydrographs

In order to accurately determine the possible impacts on downstream environments from a dam failure, derivation of breach characteristics and resultant peak flow rates are required for inclusion in the various two-dimensional (2D) hydraulic models.

A methodology involving a combination of all three assessment techniques has been adopted to facilitate the creation of inflow hydrographs for the 2D hydraulic models.

Peak flow rates were determined using the simplified assessment technique as outlined in the 'Guidelines for Failure Impact Assessment of Water Dams' (DNRM, 2002). Assessment was also undertaken using some aspects of the comprehensive assessment, namely to determine the breach characteristics and time to breach (peak flow).

Dam failure peak flow rates – simplified assessment

Peak flow rates that could be expected from failure of the various water storages are provided in Table 8.6. These are based on the simplified assessment as defined in the DNRM guidelines. This is considered to be a conservative approach. Flows are calculated based on the total water volume of the dam and also the maximum depth of water in the dam. The empirical discharge relationship used to determine the peak flow rate as detailed in the 'Guidelines for Failure Impact Assessment of Water Dams' (DNRM, 2002) is based on the failure of a typical homogeneous earth fill embankment.

Hydrograph Derivation – 2D Modelling Analysis

In order to apply the predicted peak flows to the TUFLOW hydraulic models in a representative manner, hydrographs were required to simulate an embankment breach. Methodologies as described in Section 4.7.5 of the 'Guidelines for Failure Impact Assessment of Water Dams' (DNRM, 2002) were adopted to enable the breach development time to be determined and hence the time to the predicted peak flow.

This requires the calculation of a breach formation factor (BFF). This value is determined as a function of the proposed total storage volume of the facility and height difference between headwater and tailwater levels (typically the design depth in this investigation). Once the BFF is determined, a nominal volume of material removed in the breach as a function of the breach size can be calculated.

This volume is then used to determine the breach development time. The breach development time is a function of the material removed during the embankment breach. Table 8.6 summarises the key parameters used to determine the predicted peak flow rates and breach development times for each storage facility within the tenement areas.

Table 8.6 Dam failure peak discharge & hydrograph derivation parameters

Dam name	Proposed capacity (ML)	Dam depth (m)	Peak discharge (m3/s)	Breach formation factor (ML.m)	Volume of material removed (m3)	Breach development time (mins)
WTF_MEL_01	800#	4	581	3200	1705	34
BP_MEL_01	4100*	3	2025	12398	5983	51
WTF_RCK_01a (alternate to WTF_MEL_01)	800#	4	581	3200	1705	34
BP_RCK_01 (alternate to BP_MEL_01)	4100*	3	2025	12398	5983	51
WTF_WOL_01	400#	4	343	1600	897	28
BP_WOL_01	5200*	3	2403	15524	7370	55
WTF_HCK_01	200#	4	203	800	472	23
WTF_BYM_01	400#	4	343	1600	897	28
WTF_CON_01	800#	4	581	3200	1705	34

Dam name	Proposed capacity (ML)	Dam depth (m)	Peak discharge (m3/s)	Breach formation factor (ML.m)	Volume of material removed (m3)	Breach development time (mins)
Talinga WTF (existing facility upgrade)	800#	4	581	3200	1705	34
Talinga BP (existing facility upgrade)	8300*	3	3442	24914	11426	63
BP_GIL_01	4100*	3	2025	12398	5983	51
WTF_GIL_01a	800#	4	581	3200	1705	34
WTF_GIL_01 (alternate to WTF_GIL_01a)	800#	4	581	3200	1705	34

* denotes volumes calculated from cumulative proposed surface areas and design depths (refer Section 8.2.2)
# denotes 10-day storage volume

Two dimensional model parameters

Six of the eight 1D/2D TUFLOW hydraulic models developed as part of the regional flooding investigation of the tenement areas and as described in the 'Gas Field Flooding Investigation' report have been modified for use in this investigation.

Modification involved the removal of design rainfall inputs, derivation of inflow locations representing hypothetical dam breaches within the model, and in some cases modification to modelling areas. The following sections summarise the model parameters adopted as part of the sunny day failure analysis undertaken as park of this study.

Inflow boundaries

The hydrographs representing the hypothetical dam breaches for each water treatment facility and brine pond storage were applied to the 2D model domains by way of direct application of the failure hydrograph to the appropriate delineated location of the pond as supplied by Australia Pacific LNG.

Tailwater boundaries

As the majority of the waterway systems modelled as part of this investigation are classified as ephemeral, normal depth boundary conditions were adopted at each representative waterway downstream boundary. It was found that due to the flat nature of the topography in the study areas, adopted water surface slopes generally ranged from 0.002m/m to 0.010m/m.

2D model roughness

GIS roughness maps covering the hydraulic modelling areas were created to define the hydraulic roughness spatially across the floodplains. The GIS layer of existing land use was generated using a combination of orthophoto imagery, site observations (including oblique photography) and previous experience in 2D hydraulic modelling applications. The Manning's "n" roughness parameters adopted in the model ranged from 0.015 for open water bodies through to 0.250 for Towns / District Centres.

These values are typical of those adopted for floodplain roughness for studies of this nature. Table 8.7 documents roughness assigned to each land use.

Table 8.7 Adopted roughness parameters

Land use type	Manning's 'n' roughness
Water body	0.015
Road carriageway	0.025
Cleared land / agriculture	0.040
Cleared land / sporadic vegetation	0.045
Light vegetation	0.060
Bushland / natural environments	0.080
Buildings / homestead	0.100
Township / district centre	0.250

Density of water

No changes to the default 'density of water' parameter within the TUFLOW hydraulic models were undertaken for the dam failure simulations.

Whilst the high salt content of the brine ponds would result in increased water density, there are also expected to be higher temperatures within the ponds of on average 40-50°C. This acts to reduce the higher density characteristics of the brine pond contents back to near normal density values. As a result, no change to the default TUFLOW parameter was undertaken.

8.3 Failure impact assessment results

Results for each failure impact assessment are summarised below. Typically flows from the simulated dam breaches are limited to existing areas of regional flooding as determined in the Section 5.

Where water levels or flood extents are compared to regional flooding as a result of rainfall events, these levels have been obtained from the aforementioned report.

Sections 8.3.1 to 8.3.8 summarise the impacts of the respective simulated dam failures in each modelling area. Inundation maps resulting from hypothetical dam failure scenarios are presented in Appendix K.

8.3.1 WTF_MEL_01 and BP_MEL_01

Review of modelling results shows that the extent of inundation as a result of the simulated dam breaches for both WTF_MEL_01 and BP_MEL_01 are limited to areas immediately downstream of the dam locations, and the main watercourse flowing to the modelling boundary. These watercourses include Kangaroo Creek and an upstream contributing tributary.

Breach simulation for WTF_MEL_01 shows that once flows reach the main watercourse, flood levels and extents are shown to be similar to those expected for the 500 year ARI design rainfall flooding event. Peak flow depths ranging from approximately 1.5m to 3.5m within the main creek channel are predicted, with a gradual narrowing of the flood extent into the main creek channel as flows travel

further downstream. By the time flows reach the model boundary, inundation is predicted to be below what is expected for a 10 year ARI rainfall event.

Simulation results for the BP_MEL_01 brine pond shows more significant inundation as a result of the simulated failure, with peak flow depths in the main channel of between approximately 2.5m and 4.5m. Flood levels as a result of the dam breach immediately downstream of the simulated failure are shown to be approximately 1.3m higher than a 500 year ARI rainfall event. This equates to approximately a 60m wider flood extent immediately downstream of the dam location than the 500 year ARI event. Flood levels and inundation extents at the model boundary for the BP_MEL_01 simulation are shown to be more comparable to the 500 year ARI rainfall flooding event, with flood levels some 400mm above the predicted 500 year ARI event.

Review of inundation extents on aerial photography for the study area revealed no residential buildings impacted as a result of the failure of either WTF_MEL_01 or BP_MEL_01.

Upon review of the available information and model results, the dam break hazard category for both WTF_MEL_01 and BP_MEL_01 are summarised in Table 8.8.

Table 8.8 Dam break hazard category – WTF_MEL_01 and BP_MEL_01

Categories of harm	WTF_MEL_01 harm rating	BP_MEL_01 harm rating
General environmental	Significant	Significant
Loss or harm to humans	Low	Low
Loss of stock	Significant	Significant
General economic loss	Low	Low

8.3.2 WTF_RCK_01a and BP_RCK_01a

WTF_RCK_01a and BP_RCK_01a are alternate site locations for WTF_MEL_01 and BP_MEL_01. Modelling results predicted that the extent of inundation as a result of the simulated dam breaches for both WTF_RCK_01 and BP_MEL_01 are limited to areas immediately downstream of the pond locations, and the main watercourse flowing to the modelling boundary. These watercourses include Yuleba Creek and an upstream contributing tributary.

Breach simulation for the WTF_RCK_01a storage facility showed that flood levels and extents were predicted to be similar to the 100 year ARI design rainfall flooding event immediately downstream of the storage location. This resulted in peak flow depths ranging from approximately 3.5m in Yuleba Creek immediately downstream of the dam to 1.0m within the main creek channel at the model outlet. Flows dispersed into the floodplain quickly, and levels were shown to fall to below the modelled 10 year ARI design rainfall event flood height approximately 2km downstream of the breach location.

Simulation results for the BP_RCK_01a brine pond showed significant inundation as a result of the simulated failure, with peak flow depths in the main channel of approximately 5.5m downstream of the failure. Flood levels downstream of the failure were shown to range from approximately 1m above the predicted 500 year ARI flood event level immediately downstream of the dam breach to approximately 0.2m below the predicted 100 year ARI flood level at the model boundary. Flood levels and extents for the dam failure and 500 year ARI rainfall event were shown to be comparable approximately 4km downstream of the dam breach. Increased areas of inundation were evident in the localised areas around the dam failure, and an approximately 30m wider flood extent than the 500 year ARI flood extent within Yuleba Creek was evident a small distance downstream of the dam location. This extent

was reduced as flows travelled further downstream and predicted flood levels reduced as flows dispersed in the floodplain.

Review of inundation extents on aerial photography for the study area revealed no residential buildings impacted as a result of the failure WTF_RCK_01a, although flow extents do come within close proximity to one residence. The higher inundation levels resulting from the simulated failure of BP_RCK_01a showed flood extents almost inundating the same residence adjacent to Yuleba Creek. Close inspection of model results shows the extent of flooding to be within extremely close proximity to the residence, yet the residence is considered to still remain inundation free.

The dam break hazard categories for both WTF_RCK_01a and BP_RCK_01a based on modelling results and review of available data are summarised in Table 8.9.

Table 8.9 Dam break hazard category – WTF_RCK_01a and BP_RCK_01a

Categories of harm	WTF_RCK_01a harm rating	BP_RCK_01a harm rating
General environmental	Significant	Significant
Loss or harm to humans	Significant	Significant
Loss of stock	Significant	Significant
General economic loss	Low	Significant

8.3.3 WTF_WOL_01 & BP_WOL_01

Modelling results for both WTF_WOL_01 and BP_WOL_01 show that the extent of inundation due to a simulated dam breach is limited to the areas immediately downstream of the proposed dam sites and along the main watercourse channel including Woleebee Creek.

Simulation results for WTF_WOL_01 show that flood extents immediately adjacent to the pond are broad and extend outside of the 500 year ARI existing case flood extent. In the unnamed tributary of Woleebee Creek immediately downstream of the dam, the peak water depths are 100mm to 200mm higher than the 500 year ARI event flood depths. By the time flows reach Woleebee Creek, flow depths are reduced and are predicted to be approximately 100mm lower than the 10 year ARI event in the main channel. The simulated dam breach flow disperses quickly within Woleebee Creek and ceases to cause any significant over bank flow before reaching the model boundary.

The site of BP_WOL_01 is on a ridge in the upper reaches of the catchment beside Ramyard Creek. The flood extent due to the hypothetical breach of this brine pond is well defined due to the nature of the topography as it flows north along Ramyard Creek. Flood depths along Ramyard Creek are typically between 2 and 5m, depending on flow regimes and local topographic features. At Ramyard Creek the peak flood height due to a breach of BP_WOL_01 is generally 2 to 3m higher than the 500 year ARI rainfall event flood height and the flood extent is approximately 250m broader near Jackson Wandoan Road. The peak flood height remains about 1m higher than the 500 year ARI rainfall event flood depths with a wider flood extent within Woleebee Creek. By the time flows in Woleebee Creek reach the Jackson Wandoan Road crossing near WTF_WOL_01 flood depths due to a breach of the brine pond are similar in magnitude to the 500 year ARI flood depths. At the downstream boundary of the model the flood depths from the pond breach are less than the 10 year ARI flood depths and are maintained within the well defined channels that cross the model boundary.

A review of inundation extents on aerial photography shows no residential buildings within the flood extent of the WTF_WOL_01 breach. However, there is a farm house in very close proximity to the

pond and an inspection of model results shows the broad flood extent adjacent to the pond to be within extremely close proximity to the residence. Based on model predictions the residence remains inundation free. In the case of the simulated breach of BP_WOL_01 no residential buildings are within the flood extent as determined from aerial photography. Again, several residences along Woleebee Creek are predicted to be within extremely close proximity to the increased flood extent due to a breach of the brine pond, yet they are still considered to be inundation free.

Based on review of the available information, the dam break hazard category for both WTF_WOL_01 and BP_WOL_01 are summarised in Table 8.10.

Table 8.10 Dam break hazard category – WTF_WOL_01 and BP_WOL_01

Categories of harm	WTF_WOL_01 harm rating	BP_WOL_01 harm rating
General environmental	Significant	Significant
Loss or harm to humans	Significant	Significant
Loss of stock	Significant	Significant
General economic loss	Low	Significant

8.3.4 WTF_HCK_01

An analysis of results for the breach scenario of WTF_HCK_01 reveals that flood inundation occurs immediately downstream of the proposed pond before entering the main waterway of Tchanning Creek. Once within the major waterway, flood waters are retained within the well defined channel of Tchanning Creek and extend less than 4km downstream before tapering out to a negligible flow well before reaching the downstream model boundary. Where the flow first enters Tchanning Creek flood depths are up to 3m but generally 0.5 to 1m lower than the 10 year ARI flood depths. Flood depths due to the breach remain less than the 10 year ARI depths over the downstream reach of the flood extent.

An examination of aerial photography in comparison to the breach flood extent shows no residential buildings to be inundated or within close proximity of the flood waters.

The dam break hazard category for WTF_HCK_01 is summarised in Table 8.11.

Table 8.11 Dam break hazard category – WTF_HCK_01

Categories of harm	WTF_HCK_01 harm rating
General environmental	Significant
Loss or harm to humans	Low
Loss of stock	Significant
General economic loss	Low

8.3.5 WTF_BYM_01

Failure simulation for WTF_BYM_01 predicted minimal impact as a result of dam failure. Besides the area immediately downstream of the facility, flood levels as a result of the breach remained well below the 10 year ARI design rainfall flood levels within the Dulacca Creek system downstream of the pond

location. Flood depths ranged from 3.5m in the Dulacca Creek main channel where the flows first enter the watercourse to 1.4m at the model boundary.

Review of failure simulation results and aerial photography show that no residential buildings are predicted to be impacted as a result of a possible dam failure at this location.

Upon review of the available information, the dam break hazard category for WTF_BYM_01 is summarised in Table 8.12.

Table 8.12 Dam break hazard category – WTF_BYM_01

Categories of harm	WTF_BYM_01 harm rating
General environmental	Significant
Loss or harm to humans	Low
Loss of stock	Significant
General economic loss	Low

8.3.6 WTF_CON_01

WTF_CON_01 lies on an un-named minor ephemeral stream which flows south into the Condamine River approximately 2,500m downstream of the facility. The site is located between two ridges and the ephemeral stream flowing through the central part of the site. Breach simulation for the facility predicted the flood extents to be relatively expansive with a maximum width predicted to be 600m, wider than the 500 year ARI existing rainfall flood event. The maximum depth of the breach is about 1m higher than the 500 year ARI event flood height at the stream immediately downstream of the facility. Flows disperse into the Condamine River floodplain quickly, and levels are shown to fall to below the modelled 10 year ARI design rainfall event flood once reaching the main waterway of Condamine River.

A review of inundation extents on aerial photography shows there is a residential building situated marginally inside a corner of the flood extent of the WTF_CON_01 breach, some 700m south of the infrastructure location. Inundation depths at this location are predicted to be shallow (approximately 100mm depth), with low velocities (less than 0.5m/s). It is noted this residential dwelling is owned by Australia Pacific LNG. As the depth of inundation is less than 300mm at this location, there is not considered to be a ‘population at risk’ as outlined in the DNRM ‘Guidelines for Failure Impact of Water Dams’ (April 2002).

Upon review of the available information, the dam break hazard category for WTF_CON_01 is summarised in Table 8.13.

Table 8.13 Dam break hazard category – WTF_CON_01

Categories of harm	WTF_CON_01 harm rating
General environmental	Significant
Loss or harm to humans	Significant
Loss of stock	Significant
General economic loss	Significant

8.3.7 Talinga WTF & Talinga BP

Talinga_WTF is located between two ridges and lies in a local gully draining in a northerly direction to the lower reach of the Wieambilla Creek, a local tributary of the Condamine River within tenement PL226. The 500 year ARI existing case flood extent of Wieambilla Creek is some 500m downstream of the facility.

Breach simulation for the facility predicts that the flood extents due to a breach are expansive with a maximum width of 800m. The maximum depth of the breach at the local gully immediately downstream of the facility is about 1m higher than the 500 year ARI event flood height. Flows disperse into the Wieambilla Creek floodplain quickly and levels are shown to fall to below the modelled 10 year ARI design rainfall event flood height within the main waterway of Wieambilla Creek approximately 2,000m downstream of the breach location.

Review of failure simulation results and aerial photography show that no residential buildings are predicted to be impacted as a result of a possible dam failure at the Talinga_WTF.

The site of Talinga_BP is located on a ridge adjacent to an ephemeral stream flowing northeast into the middle portion of the Wieambilla Creek. The flood extent due to a breach of this brine pond fans out along the ephemeral stream in a triangular shape into the main waterway of Wieambilla Creek with a maximum width of 2,000m.

The maximum depth of the breach immediately downstream of the brine pond is about 5m higher than the 500 year ARI event flood height at the ephemeral stream. Once reaching Wieambilla Creek, flows disperse into the main waterway with peak flood levels dropping to about 300mm below the 100 year ARI flood height. Flood waters drop down rapidly to below the 10 year ARI event flood within a short distance in the main waterway of Wieambilla Creek.

Review of failure simulation results shows that the brine pond has minimal impact to the Wieambilla Creek and the adjacent floodplain as a result of dam failure and aerial photography shows that no residential buildings are predicted to be impacted as a result of a possible dam failure at the Talinga brine pond.

Upon review of the available information, the dam break hazard category for both Talinga_WTF and Talinga_BP is summarised in Table 8.14.

Table 8.14 Dam break hazard category – Talinga_WTF and Talinga_BP

Categories of harm	Talinga_WTF harm rating	Talinga_BP harm rating
General environmental	Significant	Significant
Loss or harm to humans	Low	Significant
Loss of stock	Significant	Significant
General economic loss	Low	Low

8.3.8 BP_GIL_01, WTF_GIL_01 & WTF_GIL_01a

Breach simulation results for the brine pond BP_GIL_01 shows significant inundation of the downstream areas adjacent to the proposed dam location. Significant flows in the adjacent unnamed tributary are some 4m higher than those predicted in the 500 year ARI design rainfall flood event in some areas immediately downstream if the simulated breach. These substantial flows remain above the predicted 500 year ARI flood level until approximately 2km upstream of the model boundary,

where the dispersion of the flows from the brine pond fall below the 500 year ARI regional flood level. Flood depths remain substantial within the main watercourse channels, and can range up to 7m in depth in some locations.

Whilst flood depths and predicted flood levels are shown to be significant with respect to the modelled design rainfall flooding events, review of modelling results and aerial photography predict that no residential buildings would be inundated as a result of a possible dam failure of BP_GIL_01a.

The alternate location of WTF_GIL_01 is located near the hydraulic model boundary, and as such information is limited to the modelling area. However, simulation results of a dam failure of WTF_GIL_01 showed expansive areas of flow of approximately 200mm depth, with discrete regions of deeper flow of approximately 1.2m within the more defined topographic features adjacent to the pond location. Flood levels at the boundary of the model as a result of the dam failure are shown to be approximately 500mm higher than the predicted 500 year ARI regional flood level.

Although the location of the pond places it near the downstream boundary of the model and inundation information is limited, review of aerial photography reveals the nearest residential building downstream of the model boundary to be some 5km away. It is predicted that due to the volume of water stored at this location that flood levels as a result of a dam breach would disperse considerably by the time the flows reach these residences, and inundation, whilst possible, is unlikely. Houses located slightly upstream in the adjacent Cattle Creek watercourse are shown to remain unaffected.

WTF_GIL_01a is located near the proposed BP_GIL_01 brine pond higher in the Weir River catchment. Results predict that whilst expansive areas of flooding are shown immediately downstream of the dam location, once flows enter the main Weir River watercourse, peak water surface levels are shown to be similar to the 100 year ARI regional flood level. As flows travel downstream and dissipate further, flood levels reduce significantly, and at the point flows reach the model boundary, peak water surface levels are some 1m below the predicted 10 year ARI regional flood level. Review of modelling results reveals no residential buildings are predicted to be inundated as a result of the simulated dam failure.

Upon review of the available information, the dam break hazard category for WTF_GIL_01, WTF_GIL_01a and BP_GIL_01 are summarised in Table 8.15.

Table 8.15 Dam break hazard category – WTF_GIL_01, WTF_GIL_01a, and BP_GIL_01

Categories of harm	WTF_GIL_01 harm rating	WTF_GIL_01a harm rating	BP_GIL_01 harm rating
General environmental	Significant	Significant	Significant
Loss or harm to humans	Significant	Significant	Significant
Loss of stock	Significant	Significant	Significant
General economic loss	Low	Significant	Significant

It is noted the location of BP_GIL_01 with respect to WTF_GIL_01a allows for the possibility of sequential failure possibility at this location. This is discussed in more detail in Section 8.3.10.

8.3.9 Water transfer stations (WTS)

Water transfer stations are proposed throughout the gas fields study area. Due to the proposed storage volumes of approximately 170ML, failure impact assessments have not been undertaken for these facilities. It is noted these facilities have been predicted to have a significant hazard rating due

to the storage water constituents. Refer to Section 8.1 for details regarding the classification of hazardous dams.

As orthophoto and topographic data was available for the areas where the water transfer stations were located, desktop review has been undertaken of the proposed locations. Typically the water transfer stations were found to be well clear of any residential buildings.

8.3.10 Cumulative impacts

Cumulative impacts as a result of storage failures have been considered throughout the gas fields study area.

Sequential failure simulations were not considered necessary, subject to future design refinements, as typically water storages throughout the gas fields were situated in discrete locations where sequential failure of dams could not occur.

However, the conceptual locations of some infrastructure items indicate some possibility of sequential failure of storages, dependant on the final location of the storage within the proposed designated infrastructure site. These are discussed in the following sections.

WTF_HCK_01 and WTS_HCK_02

The locations of these two storages are shown to slightly overlap, and it is recommended that Australia Pacific LNG consider the impacts of possible sequential failure when the final locations of the storages are confirmed. That is, locating the storages so that the failure of one water storage cannot impact on the other by way of flood wave effects or embankment destabilisation.

BP_GIL_01 and WTF_GIL_01a

The conceptual location of infrastructure item BP_GIL_01 is located immediately upstream of WTF_GIL_01a. Modelling results show failure of BP_GIL_01 may impact the proposed location of WTF_GIL_01a through flood wave effects. This would depend on the location of the actual storage within the designated area. Should the storage be located within the predicted failure extents of BP_GIL_01, this increases the possibility of sequential failure of the storages, and it is recommended that Australia Pacific LNG either locate the storage outside of the predicted inundation area resulting from the failure of BP_GIL_01, or use the proposed WTF_GIL_01 site.

BP_MEL_01 and WTF_MEL_01

Both BP_MEL_01 and WTF_MEL_01 as well as their respective alternate sites of BP_RCK_01a WTF_RCK_01a have overlapping conceptual infrastructure locations.

Sequential failure probability for these sites is dependant on the location of the actual storages within the delineated infrastructure areas. It is envisaged Australia Pacific LNG would locate the respective storages within the infrastructure area in a way which eliminates the possibility of sequential failure.

8.3.11 Hydraulic performance criteria for regulated dams

This section refers to the requirements for design, calculation of the design storage allowance (DSA) and mandatory reporting level (MRL).

As all storages had a significant hazard rating due to the composition of the associated or brine water, hydraulic performance criteria are the same for all storages in the gas fields. The hydraulic performance criteria for all proposed water storages in the gas fields is presented in Table 8.16. This

includes the required event probability for design storage allowance, mandatory reporting level, and spillway/levee rating.

Table 8.16 Design criteria for gas field storages (project life – 30 years)

Performance Criteria	Design Requirement
Design storage allowance (event probability)	10 year ARI
Mandatory reporting level (event probability	100 year ARI
Spillway/levee rating (event probability	5000 year ARI

It is noted that the assessment that has been carried out is preliminary, as final locations and sizes of dams are still being finalised.

Following finalisation of the sizes and locations of the storages, calculations for DSA, MRL, and spillway and levee ratings should be carried out. It is likely that these will be a requirement of the Environmental Authority (EA) for the dams.

Based on this preliminary assessment of the proposed pond infrastructure, Table 8.16 indicates the design requirements for each storage facility. These criteria should be applied in the design stage to provide adequate design immunity to the infrastructure. Once the location and size are finalised, these requirements will need to be reviewed.

9. Conclusions and recommendations

9.1 Regional scale flooding investigation

The flood analysis results as determined using the XP-RAFTS and TUFLOW models have been successful in quantifying flood behaviour within the major watercourses. The study has demonstrated that in the existing case, significant flows within many of the waterway systems leads to both expansive flooding across the lower floodplain areas, as well as significant flood depths and velocities where the watercourses are more defined by the surrounding topography. Inundation of existing local road crossings was also found to occur. These crossings were typically of a causeway or natural style, and as such subject to frequent inundation.

Review of the location of proposed infrastructure sites within the gas fields study area revealed that the majority were located outside the regional flood extents for all design rainfall events modelled as part of this investigation. However, a select number of proposed locations were shown to lie partially or completely within the 100 year ARI flood extent. These are summarised in Table 9.1.

Table 9.1 Infrastructure locations within modelled flood extents

Infrastructure type	Infrastructure name	Rainfall event in which regional flood inundation occurs*
Water treatment facility	WTF_MEL_01	10-year ARI
Brine pond	BP_MEL_01	10-year ARI
Water transfer station	WTS_COM_04	10-year ARI
Water treatment facility	WTF_RCK_01a	10-year ARI
Brine pond	BP_RCK_01a	10-year ARI
Water transfer station	WTS_PHS_07	10-year ARI
Gas processing plant	GPF_HCK_01a	500-year ARI
Water treatment facility	WTF_CON_01	10-year ARI
Gas processing plant	GPF_CON_02b	10-year ARI
Water transfer station	WTS_TAL_00	10-year ARI
Gas processing plant	GPF_WAA_03	10-year ARI

The following options were identified where sites were located within the existing case flood extent:

- Locate infrastructure items/fill outside of the 100 year ARI flood extent but within the designated site
- Move infrastructure site clear of 100 year ARI flood extent

These options are the most favourable to eliminate any impacts on regional flood behaviour, and also serve to leave the watercourse in its natural state.

Access roads to the various proposed infrastructure locations were shown to suffer from varying degrees of inundation. Most local roads were affected by frequent inundation (all rainfall events

modelled as part of this investigation) due to the existing nature of their crossings or low elevation with respect to the surrounding topography or predicted peak design flood level. While a level of road immunity is not currently proposed by Australia Pacific LNG, 10 year ARI road immunity for most of these crossings is considered achievable through the use of appropriately designed cross road drainage features (culverts or bridges) in combination with road raising through floodplain areas. Table 9.2 summarises the major (state controlled) roads that are inundated in the gas fields study area.

Table 9.2 Major roads inundated in the study area

Road name	Location	Rainfall event in which regional flooding overtopping occurs*	Depth of Inundation (m)	Period of inundation (hours)
Warrego Highway	Dogwood Creek, Miles (EPP692)	10-year ARI	0.2	>23
Leichardt Highway	South of Miles (EPP692)	10-year ARI	0.1	11
Jackson-Wandoan Road	North of Jackson (EPP972)	10-year ARI	5.9	>37
Jackson-Wandoan Road	South of Wandoan (PL209)	10-year ARI	2.2	>32
Kogan-Condamine Road	Kogan (SEP692)	10-year ARI	1.4	>19
Dalby-Kogan Road	Kogan (SEP692)	10-year ARI	1.0	>24
Tara-Kogan Road	Kogan (SEP692)	10-year ARI	0.2	4
Warra-Kogan Road	Kogan (SEP692)	10-year ARI	0.7	>21
Chinchilla-Tara Road	South of Chinchilla	10-year ARI	1.4	>59
Kogan-Condamine Road	East of Condamine (multiple crossings)	10-year ARI	5.3	>108

Under the current base case scenario (no improved road immunity required), it is not considered that any residential houses or public road access is affected as a result of the proposed infrastructure. Should Australia Pacific LNG require increased road immunity at selected crossings, it is envisaged some localised changes to flood levels may occur. The impacts from any proposed crossing upgrades would be qualified through hydraulic modelling when road locations, designs and immunity requirements are finalised to ensure impacts are minimised and mitigation of any changes to flood behaviour is achievable.

Detailed GIS mapping tasks have been undertaken to fully illustrate flooding behaviour in the various watercourses within the tenement areas. These maps have included detailed flood depth mapping for the 100 year ARI event.

The outcomes from this study will provide important information to assist Australia Pacific LNG in managing flood risks in the modelling areas. This flood information can be used in emergency planning and response, flood warning and a range of other emergency management procedures.

Table 9.3 shows the risks and potential impacts of the facilities as a result of flooding.

Table 9.3 Flooding risks and possible impacts

Potential risk	Possible causes	Possible consequence
Damage to property Risk of injury or death	Placement of fill/cut in the floodplain Incorrect sizing or construction type of structures Other changes to flow regimes from development, such as creek diversions	Increase in inundation extents or velocities
Damage to environment	Changes to natural flow regimes as a result of construction	Erosion of watercourses and increased turbidity of waterways

9.2 Stormwater management plans

A desktop assessment of the proposed location of facilities identified potential stormwater management requirements for each site. Several sites were identified as requiring diversion of uphill runoff, through the use of interception swales or trenches. Eight sites were also identified as potentially being located within a dedicated flow path or gully, which will require either relocation of the facility or diversion of the flow path.

Swales and sediment basins were selected for their site adaptation for the onsite stormwater treatment devices. The basins were sized using standard design guidelines to hold stormwater runoff up to the 1 year Average Recurrence Interval (ARI), 24hr duration event and then modelled using MUSIC for treatment capacity. It was predicted that over 85% of sediment exported from the facilities will be captured and removed by the treatment devices.

The discharge of stormwater from each facility will be directed towards the nearby watercourses. Nine sites have been identified as potentially requiring scour protection to reduce the risk of erosion downstream of the facility. The stormwater management devices and locations of scour protection need to be further investigated and designed for each site during detailed design.

The outcomes of this study provide important information to assist Australia Pacific LNG in planning and managing stormwater infrastructure for the water treatment and gas processing facilities.

The risks and potential impacts of the facilities in relation to stormwater management are shown in Table 9.4

Table 9.4 Stormwater risks and possible impacts

Potential risk	Possible cause(s)	Possible consequence
Damage to watercourses, or loss of habitat	Increased runoff scouring sediment from site	Sedimentation of watercourses
Chemicals or hydrocarbons entering watercourses	Spills at the facility	Degradation of aquatic habitat and water quality
Increase runoff from the site into receiving watercourses	Increased impervious areas	Scour/erosion within the watercourses, loss of aquatic habitat
Contamination of runoff from upstream	Runoff from upstream catchment entering the facility area and mixing with contaminated stormwater	Increased volumes of contaminated stormwater to treat and mitigate

9.3 Hydraulic stream flow impact assessment

The hydraulic stream flow impact assessment concludes that the watercourses at all proposed discharge locations are non-perennial in nature with no flow periods extending between 5% and 70% of the simulated time. The Condamine River at Chinchilla shows a significant trend in decreasing flow and increasing probabilities of no flow conditions from the 1950s to present. The statistical analysis of flows from mid 1950s to the construction of the Chinchilla Weir indicate that the flow pattern consisted of no flow conditions 5% of the time, whereas the long term statistical analysis indicates that no flow conditions occurred 20% of the recorded period. Simulated long term flow series analysed for the other discharge locations indicated that these tributaries are likely to exist under no flow conditions for between 55% and 70% of the time.

The watercourses were also shown to be dominated by summer and autumn flows with 45% and 30% of flows occurring in the summer and autumn months respectively.

The watercourses were estimated to exhibit low velocity, stream power and stream stress at all discharge locations under flow conditions up to bank full. It is concluded that at all proposed locations the watercourses had the hydraulic capacity to accept the addition of permeate discharges. If discharge occurred under normal flow conditions, up to bank full, there was unlikely to be a significant alteration of the geomorphic characteristics of the watercourse. Discharge should be avoided during periods of low or no flow conditions in order to reduce the total sediment transport impacts.

A significant number of water users are identified along the reach of the Condamine River between the proposed discharge location and the upstream extent of the St George Water Supply Scheme. The Condamine-Balonne system also exhibits a relatively low utilisation of unsupplemented water supply allocations indicating that additional discharges may, if released under appropriate conditions, act to increase the water availability to these users. Preliminary IQQM modelling supports this assertion.

Based on the outcomes of this investigation the following is recommended:

- Discharge to the Condamine River is preferred with discharges to the other proposed locations limited to opportunistic releases during flow periods
- Additional hydrologic modelling and streamflow monitoring would be required in order to develop a similar discharge regime for tributary locations
- A discharge regime based on Option 3, a discharge factored on averaged seasonal flows or Option 4, discharge triggered by watercourse flows, is to be adopted
- Discharge volumes to be limited to 50ML/day constant release with a 3 month period of no release from August to October from the Talinga discharge location or an alternative release arrangement which complies with the ROP to achieve a medium impact
- Discharge volumes to be limited to 100 ML/day discharge from the Talinga discharge location with a no flow condition for up to 30% of the water year triggered by a flow of less than 6 ML/day within the Condamine River or an alternative release arrangement which complies with the ROP and produces a flow duration pattern which mimics the predevelopment flow regime to achieve a low impact
- Design of discharge infrastructure be undertaken such that localised velocity and scour is minimised and appropriate mixing of discharge is achieved
- An ongoing program of monitoring be developed which includes regular inspection of discharge locations and cross section survey as well as monitoring of aquatic and riparian ecosystems. (Refer to Volume 5, Attachment 20 for details of monitoring for aquatic ecosystems)

- Investigation and implementation of alternative disposal/beneficial uses be undertaken to reduce the volume of storage/discharge required

The potential impacts resulting from discharges of treated associated water on stream flow and downstream users are summaries in Table 9.5.

Table 9.5 Stream flow risks and possible impacts

Potential risk	Possible cause(s)	Possible consequence
Damage to Environment	Changes to natural flow regimes	Erosion of watercourses and increased turbidity of waterways Changes to aquatic ecology
Damage to Environment	Discharge beyond the hydraulic capacity of waterways	Hydraulic and geomorphic changes to waterways
Impact to downstream water users (including the environment)	Addition of flows to the system	Increase in water availability to downstream users Improved hydrologic condition of Narran Lakes

9.4 Dam failure impact assessment

This study has investigated the hazard classification of each storage facility as recognised by the former Environmental Protection Agency (EPA) and Department of Environment and Resource Management (DERM) literature, as well as undertaking dam failure impact assessments for each proposed water treatment facility and brine pond storage facility.

It has been determined that due to the physical and chemical properties of the associated and brine water that these facilities are classified as high hazard dams and are therefore designated as 'regulated' under the former EPA and DERM guidelines. This includes high pH levels in the water transfer station and water treatment facility storages, as well as salinity levels within the brine ponds.

The simulated impacts of theoretical dry weather dam breaches of the various water treatment facility and brine pond storages within the gas fields study area were also investigated using six modified TUFLOW hydraulic models developed as part of the regional flooding analysis undertaken in the 'Gas Field Flooding Investigation' by WorleyParsons for Australia Pacific LNG. Results typically showed that due to the proposed locations of many of the storage facilities that there was limited scope for loss or harm to human life. It is noted however that a select number of dam breach simulations were shown to have flood extents that came within close proximity too or slightly inundated residential buildings. Only one property was predicted to be inundated as a result of dam failure, and flows were extremely shallow (approximately 100mm depth) with low velocities. This property is owned by Australia Pacific LNG. According to DERM guidelines, no population was considered at risk. Consideration was also given to areas of predicted inundation as a result of a hypothetical dam breach where frequent human interaction occurs. Where there was scope for frequent human interaction, these dams were also given a significant hazard rating with respect to possible harm to human life. Possible impacts of dam failures on built infrastructure were also considered (general economic loss), however due to the locations of the dams, scope for economic loss as a result of a hypothetical dam breach was limited.

The most hazardous impact from hypothetical dam failures of the proposed the water transfer station, water treatment facility and brine pond storages is environmental harm. This is a result of the chemical and physical properties of the brine and associated water. As described in the Aquatic Ecology, Water Quality and Geomorphology Impact Assessment – Gas Fields, (Hydrobiology 2009), 'Substantial detrimental impacts to local biota could occur if this contaminated water was released to local watercourses, through direct toxicity. Indirect, long-term impacts may result from internal loading and cycling of metals/metalloids and nutrients'.

This investigation has resulted in the definitive determination of hazard ratings for each of the proposed storage facilities. Due to the composition of the waters in all of the proposed storages or due to the physical characteristics of the ponds themselves, all the the water transfer station, water treatment facility and brine pond storages associated with the gas field element of the Project are classified as high hazard dams and are classified as regulated under the EPA and DERM guidelines. This requires Australia Pacific LNG to comply with the Environmental Authority Streamlined conditions for high hazard dams containing high hazard waste, and the Code of Compliance for High Hazard Dams Containing High Hazard Waste (EPA, 2002). Based on the hazard ratings determined from this analysis, performance criteria have also been determined for certain design components of the storages.

The potential impacts resulting from dam failure are identified in Table 9.6.

Table 9.6 Dam failure risks and possible impacts

Potential risk	Possible cause(s)	Possible consequence
Damage to property Risk of injury or death	Placement of fill/cut in the floodplain Incorrect sizing or construction type of structures Other changes to flow regimes from development (ie. creek diversions)	Increase in inundation extents or velocities from dam failure
Damage to environment	Changes to natural flow regimes as a result of construction	Erosion of watercourses and increased turbidity of waterways from dam failure

9.5 Assessment outcomes

A summary of the environmental values, sustainability principles, potential impacts and mitigation measures is shown in Table 9.7. The risk level following application of mitigation measures is also given.

Table 9.7 Summary of environmental values, sustainability principles, potential impacts and mitigation measures

Environmental values	Sustainability principles	Potential impact	Possible causes	Mitigation measures	Residual risk level
Life, health and wellbeing of people. Diversity of ecological processes and associated ecosystems. Land use capability, having regard to economic considerations.	Minimise adverse environmental impacts. Enhance the benefits associated with our activities, products or services. Maintain biodiversity values, and enhance these where the opportunity exists.	Scour/erosion within the watercourses and floodplains. Loss of aquatic habitat. Damage to infrastructure. Damage to riparian areas from filling for infrastructure. Damage to crops, injury to stock from dam failure. Sedimentation of watercourses. Degradation of aquatic habitat and water quality.	Placement of fill/cut in the floodplain. Incorrect sizing or construction type of structures. Changes to flow regimes from development (i.e. creek diversions). Changes to natural flow regimes as a result of construction (temporary). Increased runoff scouring sediment from site. Spills at facilities. Increased impervious areas. Runoff from upstream catchment entering the facility area and mixing with contaminated	Design sediment basin to capture runoff and release appropriately. Avoid locating infrastructure within watercourses and flood extents. For extraction from waterways, locate suction pumps to avoid significant vegetation and minimise disturbance to vegetation and locate above the watercourse bed to minimise erosion. Include fish screen on intake. Ensure that sediment and erosion control devices are implemented according to Queensland Guidelines for Sediment and Erosion Control. Minimise removal of riparian vegetation. Include swales and sediment basins at each facility to capture and remove sediment export. Include bund storage and chemical loading areas. Use design criteria (mandatory reporting level, design storage allowance and spillway design levels) for all storage structures. Ensure the capacity of hydrotest holding ponds and other	Low

Environmental values	Sustainability principles	Potential impact	Possible causes	Mitigation measures	Residual risk level
			stormwater.	storage ponds provides sufficient freeboard to allow for additional rainwater, and prevent uncontrolled release. Ensure discharge of hydrotest or trench water is in compliance with all regulatory and landholder requirements and shall not cause environmental harm. Ensure adequate monitoring of hydrotest water and receiving water quality is undertaken to determine impacts of release of the hydro test water to the receiving environment.	
Management of finite resources	Identifying, assessing, managing, monitoring and reviewing risks to our people, our property, the environment and the communities affected by our activities.	Sedimentation of watercourses. Degradation of aquatic habitat and water quality. Increased volumes of contaminated stormwater to treat and mitigate. Changes to natural flow regime. Sedimentation of watercourses.	Uncontrolled release. Pond Leakage. Discharge of water. Sourcing of Water from waterways. Increased runoff scouring sediment from site. Spills at the facility. Increased impervious areas. Runoff from upstream	Include swales and sediment basins at each facility to capture and remove sediment export. Include bund storage and chemical loading areas. Minimise chemical treatment required for water uses or select chemical additives that are least harmful to the environment. Where biocide and oxygen scavenger is used, it should be of a type that will neutralise, biodegrade, and not bio-accumulate in the soil. Reuse hydrotest water where possible. Divert runoff from upstream around facility.	Low

Environmental values	Sustainability principles	Potential impact	Possible causes	Mitigation measures	Residual risk level
		Degradation of aquatic habitat and water quality. Contamination of stock water. Increased water availability to downstream users (including the environment).	catchment entering the facility area and mixing with contaminated stormwater. Placement of fill/cut in the floodplain. Incorrect sizing or construction type of structures. Changes to flow regimes from development (i.e. creek diversions). Changes to natural flow regimes as a result of construction (temporary).	Minimise discharges during periods of no or low flow. Implement appropriate stream protection at discharge locations. Carry out ongoing monitoring of downstream reach. Continue investigation and implementation of alternative beneficial use and disposal options. Ensure adequate monitoring of hydrotest water and receiving water quality is undertaken to determine impacts of release of the hydro test water to the receiving environment. Locate infrastructure items clear of flood extents. Implement appropriate stream protection during construction works. Implement appropriate design (including factor of safety, immunity and site selection). Adopt appropriate release regime to ensure maximum volumes of water available at correct times for use.	

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