

Papua LNG Project – Downstream

Flood Assessment – Final Report

Tetra Tech Coffey Pty Ltd

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1 GLOSSARY OF TERMS

AHD	Australian Height Datum. 0m AHD approximately corresponds to mean sea level
AEP	Annual Exceedance Probability: The measure of the likelihood (expressed as a probability) of an event equalling or exceeding a given magnitude in any given year
Initial Loss	The rainfall lost to interception, infiltration, and depression storages prior to the commencement of surface runoff. Initial loss has a major influence on the size of the flood peak produced by a storm.
Continuing Loss	The average rainfall loss rate during the remainder of the storm following the initial loss (see initial loss definition).
Critical Duration	The event duration which causes the maximum peak discharge and flood stage derived from a design storm event.
Daylighting	When a slope is projected from a design elevation to the point where it intercepts an existing ground surface.
Storm surge	The meteorological component of the coastal water level variations associated with atmospheric pressure fluctuations and wind setup.
Storm tide	Coastal water level produced by the combination of astronomical and meteorological (storm surge) ocean water level forcing.
Tailwater Boundary	The water boundary located immediately downstream from the hydraulic structure being investigated. In this instance the ocean (i.e., Caution Bay) is the tailwater boundary.



2 INTRODUCTION

Water Technology was commissioned by Tetra Tech Coffey Pty Ltd (previously Coffey Environments Australia Pty Ltd) to update the existing flood study conducted by Water Technology in 2013 for the Papua LNG Project – Downstream (the Project).

The Project incorporates the expansion of the existing Papua New Guinea Liquefied Natural Gas Plant (LNG Plant) onshore and marine facilities in Caution Bay. The LNG Plant is located in Central Province, approximately 20 km north-west of Port Moresby, between the coastal settlements of Boera and Papa (see Figure 2-1). The current LNG Plant is operating on an approved Environmental Impact Statement (EIS) submitted to the Papua New Guinea government in 2009 (Coffey, 2009). The Project requires submission of a new EIS. This flood assessment will support the EIS and is intended to be submitted in conjunction with the EIS.

Water Technology has been asked to undertake the following scope of services:

- Characterise baseline flood conditions at the LNG Plant site and nearby sensitive receptors using existing data;
- Assess the movement of surface waters into and through the LNG Plant site (and surrounding area) to quantify hazards from flooding of the site, including consideration of high tides and storm surges;
- Provide early indication of potential exceedance of any relevant legislation, guidelines or standards (including Environment Permit criteria) to allow design modifications to be implemented; and
- Identify appropriate measures to avoid, minimise or mitigate adverse impacts from flooding and to manage residual impacts.



Figure 2-1 Location plan



3 BACKGROUND

ExxonMobil PNG Limited (EMPNG) is the operator of the PNG LNG Project, an integrated development that is commercialising the gas resources of Papua New Guinea. The Papua LNG Project is a gas development project for the Papua LNG Project Joint Venture participants, which are:

- TotalEnergies EP PNG Limited (TotalEnergies, an affiliate of TotalEnergies SA)
- ExxonMobil PNG Antelope Limited (EMPNGA) and ExxonMobil PNG Elk Limited
- Pac LNG Assets Limited, Pac LNG International Limited, Pac LNG Investments Limited, Pac LNG Overseas Limited and Pac LNG Holdings Limited (affiliates of Santos Limited)
- JX Nippon Oil and Gas Exploration Pty Ltd.

The Papua LNG Project will involve producing and processing the gas and condensate from the Elk and Antelope gas reserves in Petroleum Retention License 15 (PRL-15), located in Gulf Province, Papua New Guinea and piping them overland, then offshore in the Gulf of Papua, to the existing Papua New Guinea Liquefied Natural Gas Project (PNG LNG Project) producing and exporting facility (LNG Plant) located in Caution Bay (Central Province) approximately 20 km north-west of Port Moresby.

The existing PNG LNG Project commercialises gas reserves in the Hela, Western and Southern Highlands provinces of Papua New Guinea. The gas is transported to the LNG Plant via pipeline, where it is liquefied and exported as LNG by ship to international gas markets. The LNG Plant also currently supplies gas to the NiuPower Port Moresby Power Station and the Dirio Central Province Power Station (DCPPS) for the domestic electricity market.

The LNG Plant is located approximately 800 m from the coastline and approximately 20 km to the north-west of Port Moresby. Elevations range at the site from 0 m to 6 m Australian Height Datum (AHD). The catchment is bounded by steep slopes that increase to around 300 m AHD to the south and east of the project area.

The Papua LNG Project involves two components:

- The Papua LNG Project – Upstream (referred to as the Upstream component)
- The Papua LNG Project – Downstream (referred to as the Downstream component, hereafter referred to as ‘the Project’ and the component addressed in this document).

Execution of the Upstream component and the Project is delimited in Caution Bay, with the approximate boundary (i.e., the scope break) between the components located near the fence of the LNG Plant, south of the existing jetty.

A summary of the components of the Project is provided in Table 3.1.



Table 3-1 Project components at the LNG Plant

Component	Description
Additional LNG trains	New LNG processing trains and supporting systems including: • Inlet gas receiving and treatment • Dehydration/mercaptan removal • Mercury removal • Heavy-hydrocarbon separation (which contains benzene, toluene, ethylbenzene, and xylenes (BTEX)) • Liquefaction • Stabilisation • Refrigeration system • Fractionation system (naphtha).
Expansion of existing utilities	Existing PNG LNG shared utilities to be expanded include: • Hot oil system • Service and instrument air system • Nitrogen system • Freshwater system • Desalinated water system • Potable water system • Diesel oil storage and distribution system • Open and closed drain systems • Sanitary wastewater treatment systems
New utilities	Options for new separate utilities may include: • Integrated fuel gas system • Power generation system • Tempered water system • Highly demineralized water system for the Heat Recovery Steam Generators • Flare system (wet and dry gas) • Thermal oxidizer
Product storage	Product storage will include: • New condensate storage tanks with a total storage capacity of up to 110,000 m³.



Component	Description
New marine facilities	Marine facilities will include: • CALM buoy and associated exclusion zones • Pipeline end manifold (Upstream construction component, Project operations component) • Naphtha and condensate pipelines (Upstream construction component, Project operations component) • New feed pipelines including a gas pipeline and a condensate pipeline (Upstream construction component and Project operations component), with landfall and tie-in to the pipelines at the LNG Plant. The pipelines will be located south of the existing LNG Plant jetty. • Alternate shipping channel • Navigation and monitoring systems
Temporary facilities	Temporary construction facilities will be located within the PNG LNG Project's lease boundaries and will include laydown areas, offices, workshops and concrete batching plants.
Waste	Options being reviewed for waste facilities and management being assessed may include: • Temporary onsite storage and segregation of waste, with transport to and disposal at third party waste management facility • New engineered solid waste landfill within existing LNG Plant fence for disposal of non-restricted waste, with limited disposal of restricted waste by specialised service.
Logistics route	Most construction materials will be transported by truck from Motukea Port in Port Moresby to the LNG Plant site via the Lea Lea Road. Repairs or upgrades to the road, or relocation of overhead cables may be required to allow transport of large, heavy loads.
Workforce	Construction: Approximately 5,000 to 7,000 personnel at peak. Operation: Increase of approximately 50 operations and maintenance technicians with support staff anticipated to increase.
Accommodation	Construction: Up to 6,000 bed construction camps. Operation: Up to 500 beds at the existing operations camp.

The Project area is defined as all areas within the model domain (and their associated environmental and social values) that could be directly or indirectly impacted as a result of the construction and operation of project activities, facilities and infrastructure.

3.1 Previous Study

A flood study was completed in 2013 by Water Technology, which assessed the movement of surface water into and through the LNG Plant to quantify likely hazards from flooding of the site. To assess the flood hazards, hydrologic and hydrodynamic modelling was conducted for the 1% and 10% Annual Exceedance (AEP) storm events. Site flooding due to tidal inundation and flooding associated with high tidal water levels were also assessed. For greater detail on the findings from the previous study see Water Technology's 2013 Flood Assessment document.

Relevant information from the previous study was used in the current assessment.



3.2 Data Collation and Review

3.2.1 Proposed Plan Expansion

The proposed plant expansion footprint is outlined in yellow in Figure 3-1. The proposed development areas addressed in this study include: waste management facilities, roads, camp upgrade, condensate tanks, new LNG trains area, flare stack and retention ponds. Temporary construction, camp and laydown areas are not included in the flood assessment.

It is noted that an existing camp is located at the southeast corner of the plant: however, as upgrades are proposed within this facility, the footprint was included within the developed conditions scenario with existing conditions represented by the available topographic data. It is also noted that the laydown area located in the northwest corner of the plant is an existing feature and not part of the developed conditions. A raised footprint was included in the developed conditions model for this area; however, it is considered to have a minimal impact on the results, as shown in Section 5.2.1.

3.2.2 Topographic Data

Water Technology was provided with three topographic datasets to facilitate the hydraulic model development. These datasets were as follows:

- Light Detection and Ranging (LiDAR) with 1 m grid resolution captured in 2018;
- LiDAR with 0.1 m grid resolution captured in 2022; and
- 5 m contour dataset.

The extent of these datasets is shown in Figure 3-2. The more recent LiDAR dataset includes the entire extent of the 2018 LiDAR dataset. The 2022 LiDAR dataset is the most accurate and covers the entire LNG Plant area including areas of proposed development. The 5 m contour dataset was developed using 5 m contours during the 2013 flood study (Water Technology 2013) and covers the upper catchment of the project area only.

The 2022 LiDAR and the contour dataset were used to prepare a Digital Elevation Model (DEM) of the greater catchment area. The DEM forms the basis upon which all flood hazard related assessments have been completed as part of this study. Greater detail on the application of this data within the model is provided in Section 5.2.1.

3.2.3 Sea Level Data

The literature from the PNG LNG EIS (Esso Highlands Limited 2008 and GEMS 2008), was reviewed to gain an understanding of the ocean water level variation at the site. Data was collected for a period of 5 months in Caution Bay, commencing 20th December 2007. The data indicates that the maximum recorded tide was around 1.5 metres above mean sea level.

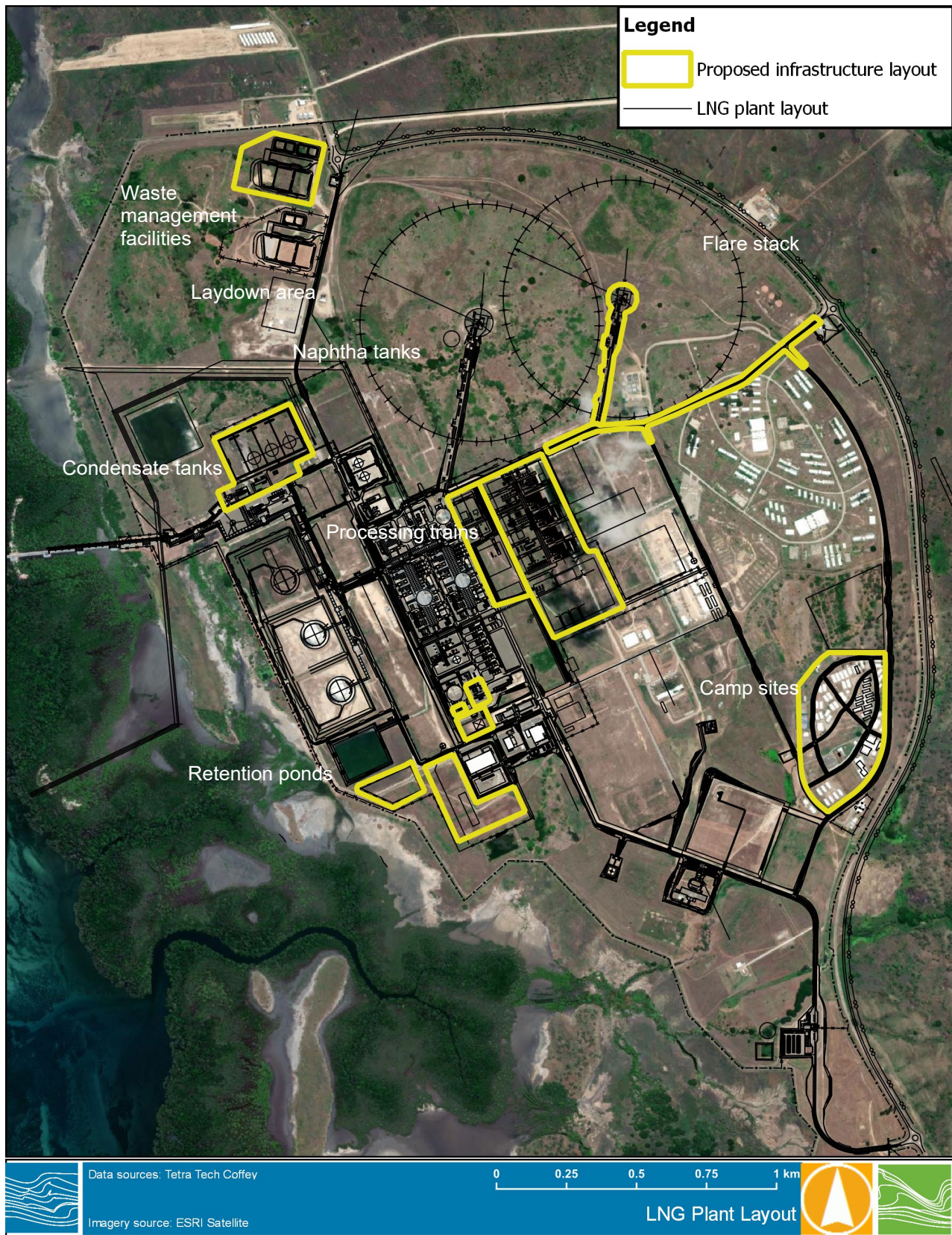
The effect of tropical cyclones was also assessed in the PNG LNG EIS. The maximum storm surge experienced due to Tropical Cycle Guba, which passed to the south of Caution Bay on 14 November 2007, was recorded as 0.1 m. A theoretical 990 hPa Tropical Cyclone was also modelled (GEMS, 2008) to estimate the maximum storm surge due to cyclones at the LNG Plant. The modelled maximum storm surge was 0.25 m.

The literature review conducted during this study returned no additional studies or data since the original flood study by Water Technology in 2013. Ideally, when determining storm surge levels, there was a suitable long-term dataset from which to calculate the appropriate values. In this case, there is a paucity of data, so conservative estimates of storm surge levels have been calculated (GEMS, 2008).



The joint probability of extreme flooding and storm surge could not be assessed due to lack of data and has not been done in the literature. However, it is expected that it would be very unlikely for a significant storm surge to coincide with both a spring tide and an extreme flood event. To overcome the lack of data, and ensure the assessment covers the full range of expected conditions, a conservative tailwater condition in the hydraulic modelling was applied (described further in Section 5.2.2):

- For the 10% AEP flood event, a value corresponding to the maximum recorded tide was considered appropriate: 1.5m above sea level.
- For the 1% AEP flood event, applying the sum of the maximum recorded tide and the modelled maximum storm surge - a total of 1.75m above sea level – was considered an appropriate conservative level.



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Figure 3-1 Existing LNG Plant layout and proposed infrastructure

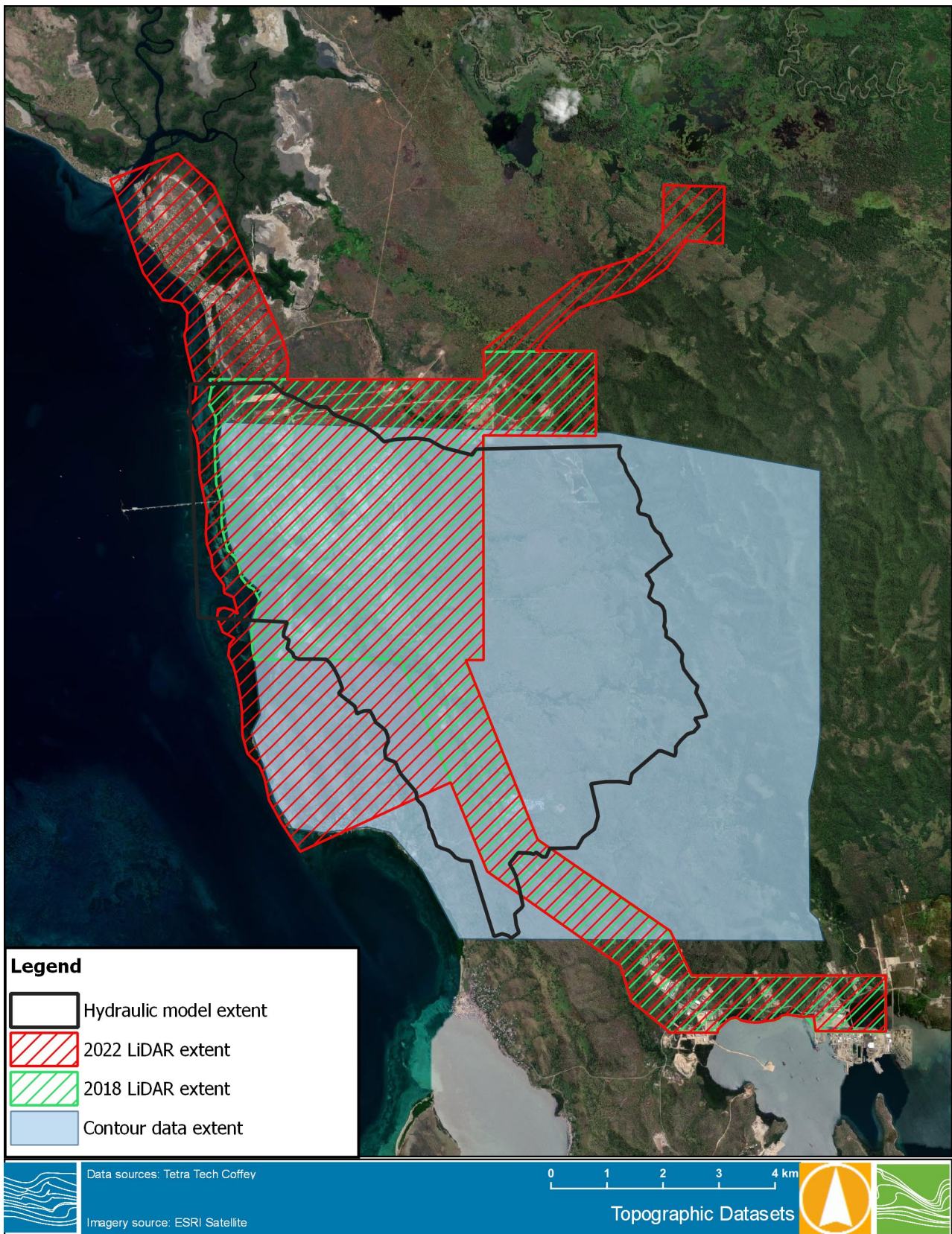


Figure 3-2 Topographic dataset extent



4 HYDROLOGY

The most up to date guidelines on rainfall estimation for Papua New Guinea and the Project area are the PNG Flood Estimation Manual (SMEC, 2018). This is a completely revised update to the previous edition of the Flood Estimation Manual published in 1990.

The 2013 Water Technology flood study adopted the 1990 SMEC guidelines when developing the hydrological estimation for the project. The hydrological estimation approach recommended in the latest Flood Estimation Manual is similar to the one described in the 1990 guidelines but updated using longer records of rainfall as well as incorporating recommendations from the Australian Rainfall and Runoff 2016 (ARR 2016). In this study, the 2018 Flood Estimation Manual was adopted for developing the hydrology estimation.

Similar to the 2013 Water Technology flood study, the runoff estimates produced from the hydraulic modelling using design rainfall and losses documented in Section 4.1 were compared to Regional Flood Frequency Method (RFFM) estimates discussed in Section 5.3. The purpose of this was to validate the peak flow estimates from the hydraulic modelling to ensure inputs to the hydraulic model were appropriate.

4.1 Design Rainfall

Design rainfall depths were developed using the methodology described in Chapter 3 of the PNG Flood Estimation Manual (SMEC, 2018).

The Port Moresby weather office rainfall station (55006) in the Central Province was chosen to represent the study area as it is located approximately 20 km away and had 70 years of continuously recorded daily rainfall data. The Port Moresby rainfall station (55055) used in the 2013 Water Technology flood study was not included in the latest Flood Estimation Manual and was therefore replaced for this update.

Design rainfall depths were obtained for the 1% and 10% Annual Exceedance Probability (AEP) events. Longer duration (24-hour, 48 hour and 72 hour) storm events used the rainfall station tabulated data in the PNG Flood Estimation Manual (SMEC, 2018). Shorter durations were calculated using scaling factors from the PNG Flood Estimation Manual (SMEC, 2018). All design rainfall depths were adjusted using areal reduction factors as recommended by the PNG Flood Estimation Manual (SMEC, 2018).

The design rainfall depths adopted in this study are summarised Table 4-1.

As well as design rainfall depth, temporal patterns are required for each design event (i.e., 1% and 10% AEP). The temporal patterns adopted for the modelling were extracted from the PNG Flood Estimation Manual (SMEC, 2018) are tabulated in Appendix A for reference.



Table 4-1 Design rainfall depth

Duration (hour)	Design Rainfall Depth (mm)	
	1% AEP	10% AEP
0.25	45	27
0.50	74	47
1	109	70
2	146	95
3	169	111
6	206	131
9	229	145
12	248	155
24	309	194

4.2 Design Losses

The 1% and 10% AEP design losses were adopted from Chapter 10 of the PNG Flood Estimation Manual (SMEC, 2018) and are tabulated in Table 4-2. These losses were also adopted for the 2013 study (Water Technology, 2013). These losses were initially used as a guide and were later verified during the peak flow calibration process discussed in greater detail within Section 5.3.

Table 4-2 Adopted initial loss and continuing loss values (Water Technology, 2013)

AEP	Initial Loss (mm)	Continuing loss (mm/hr)
1%	30	4
10%	30	4



5 HYDRAULIC MODELLING

5.1 Modelling Approach

The modelling approach applied for this project included a Rain-on-Grid (RoG) approach. No historic flood information, such as aerial flood photography or surveyed flood marks, or flood modelling was available for verification of model results.

The RoG (or direct rainfall) approach has been used extensively for flood mapping of urban and rural areas. The TUFLOW hydraulic modelling package is the most widely used in Australia, with no limitations on model size or complexity. The High Performance Computing (HPC) version of TUFLOW is used in this study as it allows for the use of high-end graphics cards to significantly reduce model run times, without sacrificing model accuracy.

The rainfall on grid methodology for a comprehensive flood hazard assessment is illustrated as a flow chart in Figure 5-1. This methodology can provide highly detailed flood hazard mapping for the full range of possible applications.

5.2 Model Setup

TUFLOW is a hydrodynamic computational model with 1D and 2D capabilities. The latest TUFLOW HPC (Heavily Parallelised Compute) is a new 2D solver with an explicit solution of the full 2D Shallow Water Equations. The HPC scheme conserves volume and momentum. TUFLOW HPC in combination with a Graphic Processing Unit (GPU) hardware enables high accuracy and high-speed solution of complex 1D and 2D hydraulics. Also, the 2D solver is dynamically linked to the 1D solver.

The existing TUFLOW model developed during the 2013 study was updated to incorporate the GPU solver and the most recent proposed development layout. The details of the model setup are documented in the sections below.

5.2.1 Model Domain and Topography

The model domain of this study covered the existing LNG Plant footprint including the proposed development area. The model extent includes the upper catchment of the LNG Plant to ensure that all runoff upstream of the LNG Plant is appropriately accounted for. The extent of the model domain is shown in Figure 5-2.

As discussed in Section 3.2.2, a DEM was generated from two datasets, 1 m LiDAR and 5 m contours, with the LiDAR dataset having a higher resolution than the 5 m contour data. It should be noted that using data with varying grid resolutions results in 'banding' across the terrain which has the potential to alter flow patterns across the area of interest. To alleviate this 'banding' effect, the area where the two datasets intersect were interpolated to ensure a smooth transition. It is important to note that in this instance the banding is situated within the upper catchment and not within the project area so has not had a significant impact on the modelled flood levels and water movement at the project area. Additionally, since the previous study (Water Technology, 2013) the resolution of the DEM within the project area has increased from 5 m to 1 m which has improved the accuracy of modelled flood levels in this area. Using higher resolution LiDAR data would further improve the accuracy of the modelling within the upper catchment, however at present this is not considered necessary given that the study area is covered by high resolution topography data.

Additional to the topographic datasets discussed above, existing infrastructure at the LNG Plant was assumed to be adequately bundled and was represented in the model by raising the developed areas well above the modelled water levels.

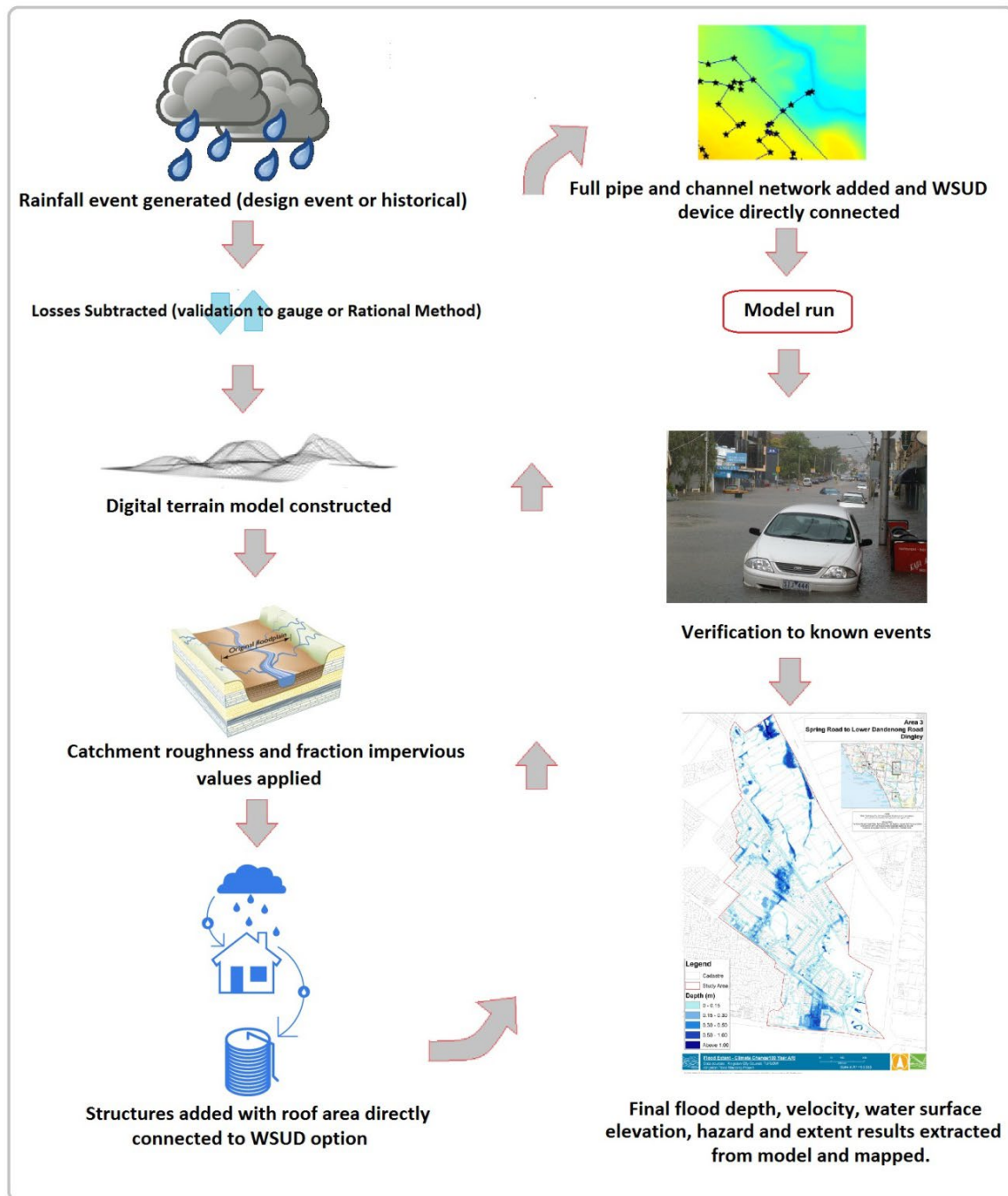
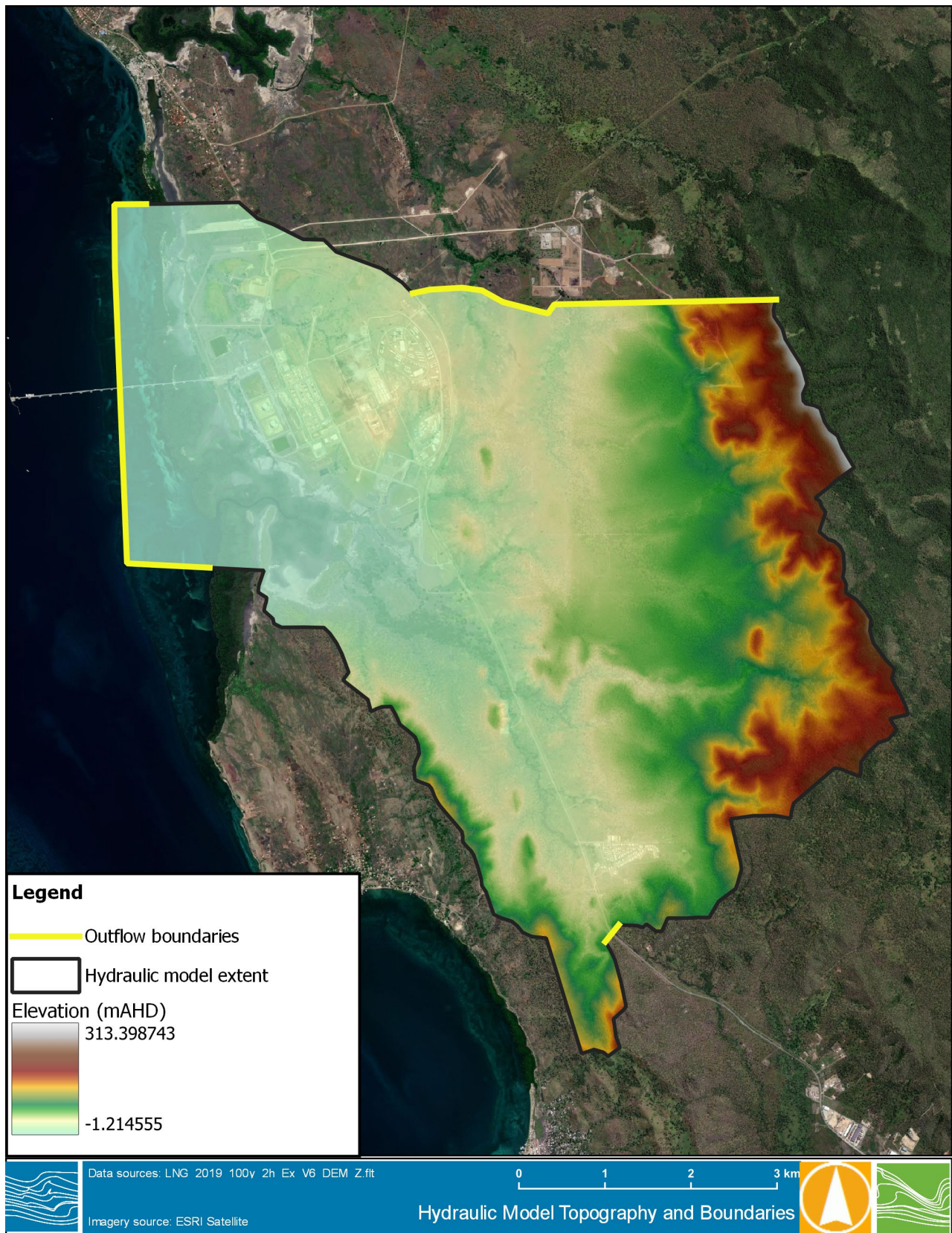


Figure 5-1 Rainfall on grid modelling approach

The DEM formed the basis for the development of the hydraulic model for both the existing and proposed conditions for this study. In the post development scenario, the proposed infrastructure footprint was represented in the DEM by artificially raising the ground elevation to a height above the predicted water levels, as was done for the existing scenario modelling. This was to simulate flood waters backing up behind the proposed infrastructure which is what will take place once the infrastructure is constructed. The proposed scenario also included the raised footprints for the existing infrastructure to assess the total impact of the LNG Plant upon completion of the proposed construction. As mentioned in Section 3.2.1, the existing laydown area located in the northwest corner of the plant was modelled as part of the developed scenario, despite being an existing feature. Given it is away from identified flow paths, the change in model results is negligible.



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Figure 5-2 TUFLOW model extent and boundaries



5.2.2 Model Boundary Conditions

Two types of outflow boundaries were applied to the hydraulic model; a tailwater boundary and a normal slope boundary. The tailwater boundary had a fixed water level elevation at the model boundary. In this instance, the tailwater boundary was used to simulate the tidal level within Caution Bay. The normal slope boundary was assigned to allow water to flow out of the model and not back up at the model boundary.

There were two normal slope boundaries used in the model. One on the northern side of the model extent and a second one on the southern extent of the model (see Figure 5-2). The normal slope boundaries were configured to account for discharges out of the model domain. The normal slopes used in this model were consistent with the those used in Water Technology's 2013 flood study.

On the western side of the model, a tailwater boundary was used to apply a fixed tidal level. The tidal levels adopted are as follows:

- 10% AEP – 1.5 m
 - This corresponds to the highest recorded tidal level in Caution Bay during the data collection period (5 months between December 2007 and April 2008).
- 1% AEP – 1.75 m
 - This is a conservative level that corresponds to the highest recorded tidal level in Caution Bay during the data collection period (5 months between December 2007 and April 2008), plus the theoretical modelled storm surge due to cyclone of 0.25 m.

5.2.3 Roughness

The hydraulic roughness represents the different surface conditions across the catchment and is defined using a Manning's 'n' value. The roughness of the project area was classified based on aerial imagery. The model domain was broken down into four categories of land use and their respective adopted Manning's 'n' values. These are shown in Table 5-1.

Table 5-1 Adopted Manning's 'n' values

Land Use	Manning's 'n'
Undeveloped	0.08
Cleared land with no buildings	0.03
Developed land with buildings	0.20
Roads	0.025

5.2.4 Model Assumptions and Limitations

Various assumptions were made in the process of developing the TUFLOW model. They are summarised as follows:

- A number of the existing culverts under the access and site roads (within the model domain) were identified using a combination of aerial imagery and topographic data. However, details relating to the culverts dimensions was not available. The culvert locations are shown in Figure 5-4. In the absence of culvert dimensions, they have been represented in the model by artificially lowering the road to allow runoff (from the upper catchment) to drain freely across the road, thereby preventing runoff backing up behind the roads. This was deemed a conservative approach and appropriate at this stage of the design process and EIS stage;



- The height of the proposed infrastructure was not available at the time of this study due to its preliminary nature. In the absence of this information the proposed development height was represented in the model by raising the proposed footprint by approximately 5 m for structures and 1 m for roads. Results from this modelling may be used to set a suitable infrastructure height;
- The design of the 3D surface (i.e., road grades) within the proposed infrastructure areas was not available at the time of this study. In the absence of this information the proposed development surface was not modelled in detail. Instead, the entire extent of each infrastructure area was raised as described above. Therefore, surface water mitigation within the infrastructure areas was not conducted in this study; and
- As appropriate bunding was assumed for both existing and proposed infrastructure, no rainfall was applied to the bunded areas containing existing or proposed infrastructure.

5.3 Model Verification

To verify the peak flood estimates developed using the methods discussed above (see Section 4 and 5) the Regional Flood Frequency Method (RFFM) was selected as the appropriate method which is in accordance with the PNG Flood Estimation Manual (SMEC, 2018). The method described in the updated Flood Estimation Manual is simpler than the method described in the 1990 manual but is considered more accurate as it is based on more years of data. The estimates from the RFFM were then compared to the peak flow estimates generated from the updated hydraulic modelling (see Section 5).

In the previous flood study of the project area (Water Technology, 2013) RFFM was applied to two upper catchments labelled East and South, which are shown in Figure 5-3. It was deemed appropriate to verify the two catchments located furthest upstream in the catchment, as RFFM applied to the other catchments shown in Figure 5-3 would include the runoff from these upper catchments as well.

The RFFM, as outlined in the PNG Flood Estimation Manual (SMEC, 2018), only requires the physical catchment parameter of area.

5.3.1 Accuracy of the RFFM

The 1990 PNG Flood Estimation Guide has assessed standard errors associated with the application of the RFFM. The recently updated PNG Flood Estimation Manual (SMEC, 2018) does not provide any such recommendations. Although the standard errors presented in the 1990 guidelines are not entirely applicable to the method used in this study, these errors are presented here for some guidance in the absence of other values. The 1990 guidelines standard error approach was applied to the East and West catchments to quantify the accuracy of the RFFM estimates. The standard errors associated with the application of the RFFM (SMEC, 1990) are shown in Table 5-2.

Table 5-2 Standard errors for RFFM

Event	Standard Error (%)
10% AEP	+100%, -40%
1% AEP	+150%, -50%

The standard error can exceed the bounds listed in Table 5-2 under the following circumstances:

- Area, P2 and slope of catchment are outside the limits found in Figure 17 to 19 in the PNG Flood Estimation Manual (SMEC, 1990);
- The occurrence of karst land is greater than 10%;
- The occurrence of swamp or flood prone land is greater than 5%; and

- The value of the catchment shape index is greater than 2 (refer to Figure 15 of the PNG Flood Estimation Manual, SMEC 1990).

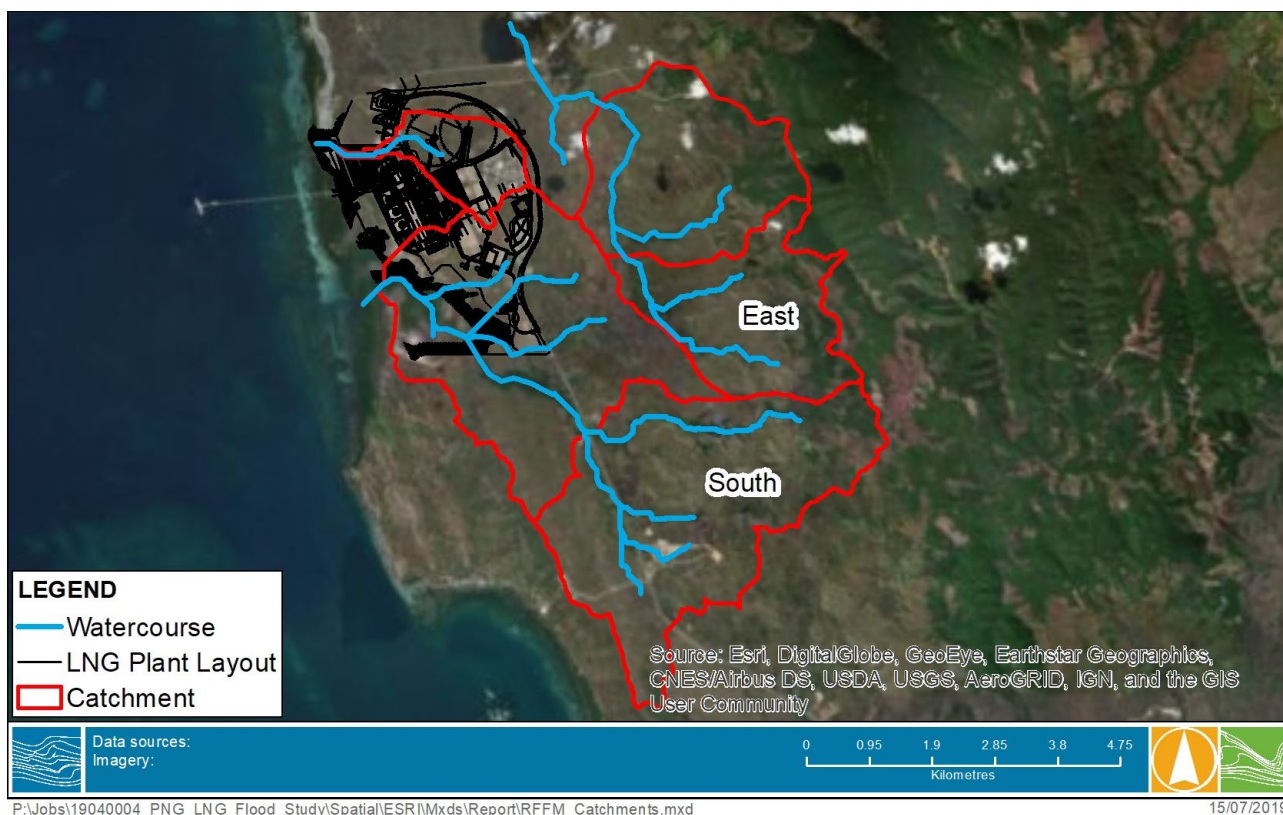


Figure 5-3 Catchments used in RFFM calculations

The RFFM estimates for the catchments analysed could be greater than the standard error because the catchments have slope that is outside the limits found in Figure 17 to 19 in the 1990 PNG Flood Estimation Manual and it must be kept in mind these errors apply to a different RFFM method. The large variance in the accuracy of the regional flood method indicates the need for a conservative hydraulic modelling approach and parameter selection.

Table 5-3 presents the peak flow estimates for East and South catchments (Figure 5-3) obtained using the RFFM as well as from peak flows generated from the TUFLOW modelling discussed in Section 5.2. A comparison has been made between the peak flow estimates using these two methods. In general, the peak discharges estimated using the TUFLOW method were higher than the RFFM. The runoff from the larger southern catchment exceeded the standard errors for both AEP events whereas the runoff from the eastern catchment remained within the standard errors. In the case of the southern catchment this exceedance suggests that the peak flow estimates are conservative.

Table 5-3 Comparison of peak flow between RFFM and hydraulic model estimates

Catchment	10% AEP Peak Flow (m ³ /s)			1% AEP Peak Flow (m ³ /s)		
	RFFM	TUFLOW	Difference	RFFM	TUFLOW	Difference
South	64	139	+117%	107	289	+170%
East	42	83	+98%	71	165	+132%



5.4 Model Scenarios and Events

Two modelling scenarios were developed for this project to ascertain the varying drainage conditions and impacts associated with the proposed infrastructure footprint. A description of the two varying modelling scenarios is provided below:

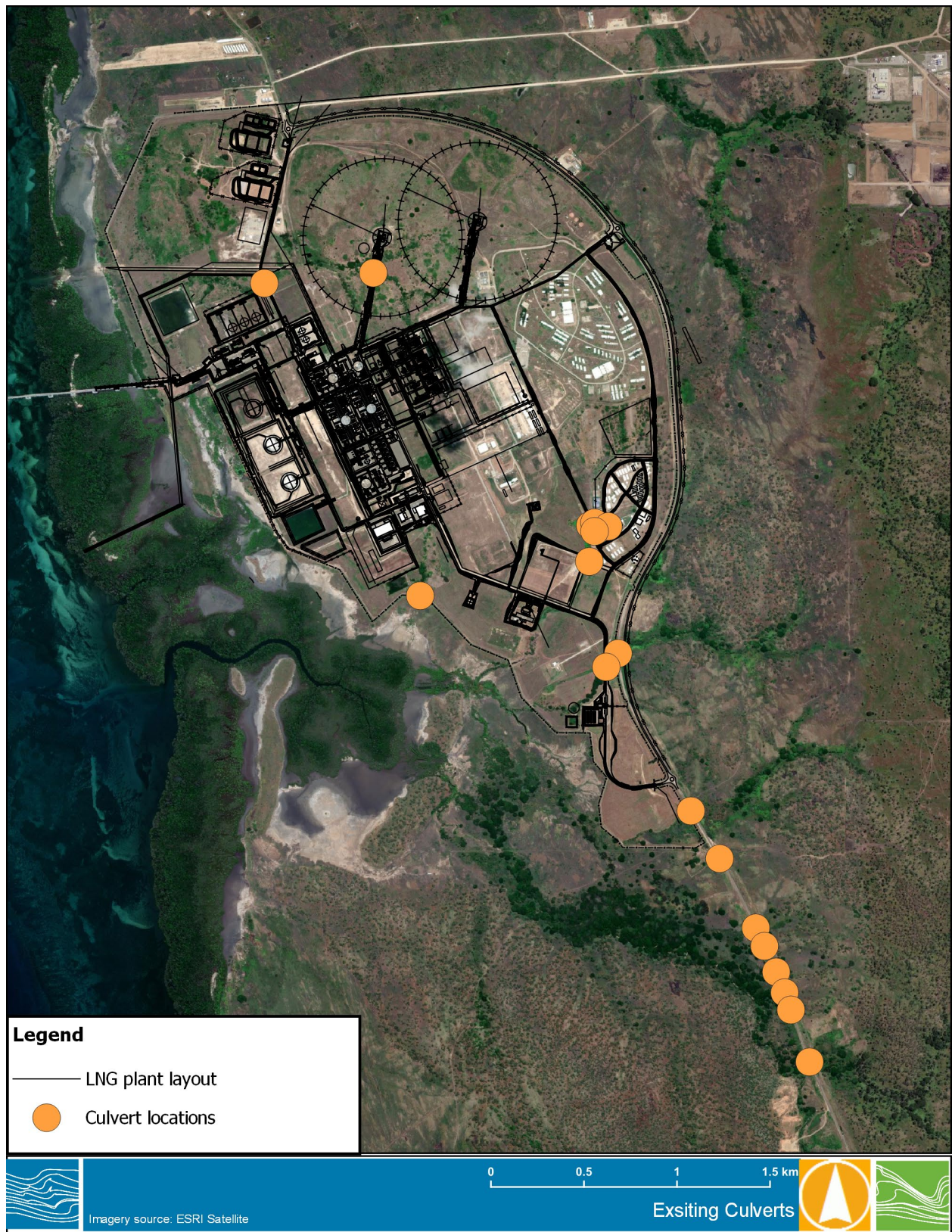
- 1) **Existing** – 1% and 10% AEP storms under varying durations (15 minute to 24 hour) were applied to the DEM with current footprint raised to a level 5 m above existing ground level (this was built into the DEM) and roads lowered where existing culverts are located; and
- 2) **Proposed** – 1% and 10% AEP storms under varying durations (15 minute to 24 hour) were applied to the DEM with the proposed footprint raised to a level 5 m above existing ground level.

5.4.1 Climate Change

Australian Rainfall and Runoff 2016 (ARR 2016) provides guidance on estimating design rainfall under climate change conditions. Global Climate Models (GCM) project the rise in average global temperature based on Representative Concentration Pathways (RCPs). RCPs represent greenhouse gas trajectories in the future. There are four (4) pathways defined in ARR 2016, those being RCP8.5, RCP6, RCP4.5 and RCP2.6, with each pathway representing the estimated greenhouse gas emissions up to the year 2100. The four (4) RCPs represent a range of emission scenarios, with RCP8.5 having the least mitigation and RCP2.6 having the strongest mitigation efforts (IPCC, 2014). ARR 2016 recommends using RCP8.5 and RCP4.5 for flood related impact assessment.

The PNG Flood Estimation Manual (SMEC, 2018) provides recommendations for percentage change in rainfall under various climate change scenarios. These were used to develop a rainfall projection for the year 2050 under the change scenario of RCP4.5 and for the year 2090 under the RCP8.5. The Flood Estimation Manual recommends using the medium (RCP6) scenario, however for this study the projected change for RCP4.5 at 2050 was identical to RCP6 at 2050. RCP8.5 was adopted for the year 2090 for a conservative approach. The projected change in rainfall suggests an annual increase in rainfall of 5% for RCP4.5 at 2050 and of 16% for RCP8.5 at 2090. This increase was applied to the existing rainfall patterns for both the 1% and 10% AEP rainfall events whilst also applying the predicted sea level rise for the year 2050 assuming RCP4.5 and 2090 assuming RCP8.5; see below.

The prediction of sea level rise and increase in temperature at the Project were obtained from the Intergovernmental Panel on Climate Change (IPCC, 2014) report which indicates a 0.5 m sea level rise to be a conservative estimate for RCP4.5 at 2050 and 0.8 m to be a conservative estimate for RCP8.5 at 2090.



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Figure 5-4 Existing culverts within the project area



6 RESULTS AND DISCUSSION

6.1 Critical Duration

To identify the catchment's critical duration for both the 1% and 10% AEP (i.e., the duration resulting in maximum flood depth), a total of nine storm durations were analysed for both AEP events. The storm durations analysed were 15 minutes, 30 minutes, 1 hour, 2 hours, 3 hours, 6 hours, 9 hours, 12 hours and 24 hours. Figure 6-1 to Figure 6-4 illustrate the comparisons between the maximum estimated flood depths for each of the nine durations analysed. Note that all result grids are filtered to remove depths below 20 mm.

Selection of the critical duration is by visual inspection. Based on this visual assessment of the 1% AEP event, the upper catchment is dominated by the 2 hour event whilst low-lying areas close to the ocean are dominated by the 6-hour storm (see Figure 6-1 and Figure 6-2). Closer the LNG Plant, the critical duration upstream of key infrastructure locations is 2 hours. Based on the existing conditions result and the location of proposed developments, the 2-hour duration was identified as critical and used for the remaining 1% AEP modelling presented in this report. Upon further analysis, the maximum depth at key locations differed insignificantly between the two durations.

Similarly, the 2-hour duration was identified as critical for the 10% AEP event, see Figure 6-3 and Figure 6-4. Based on this assessment, the hydraulic modelling adopted the 2-hour storm for both the 1% and 10% AEP events.

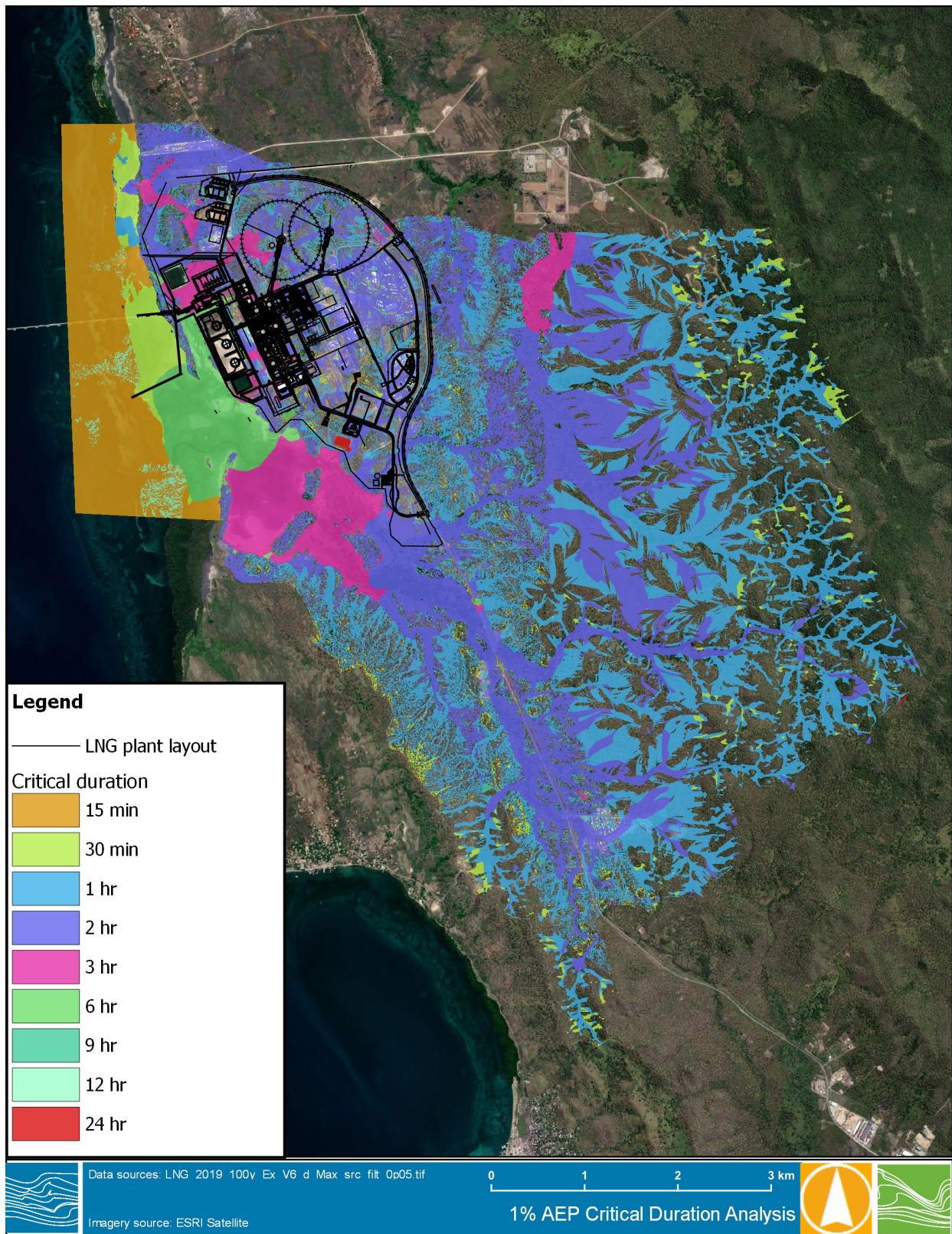
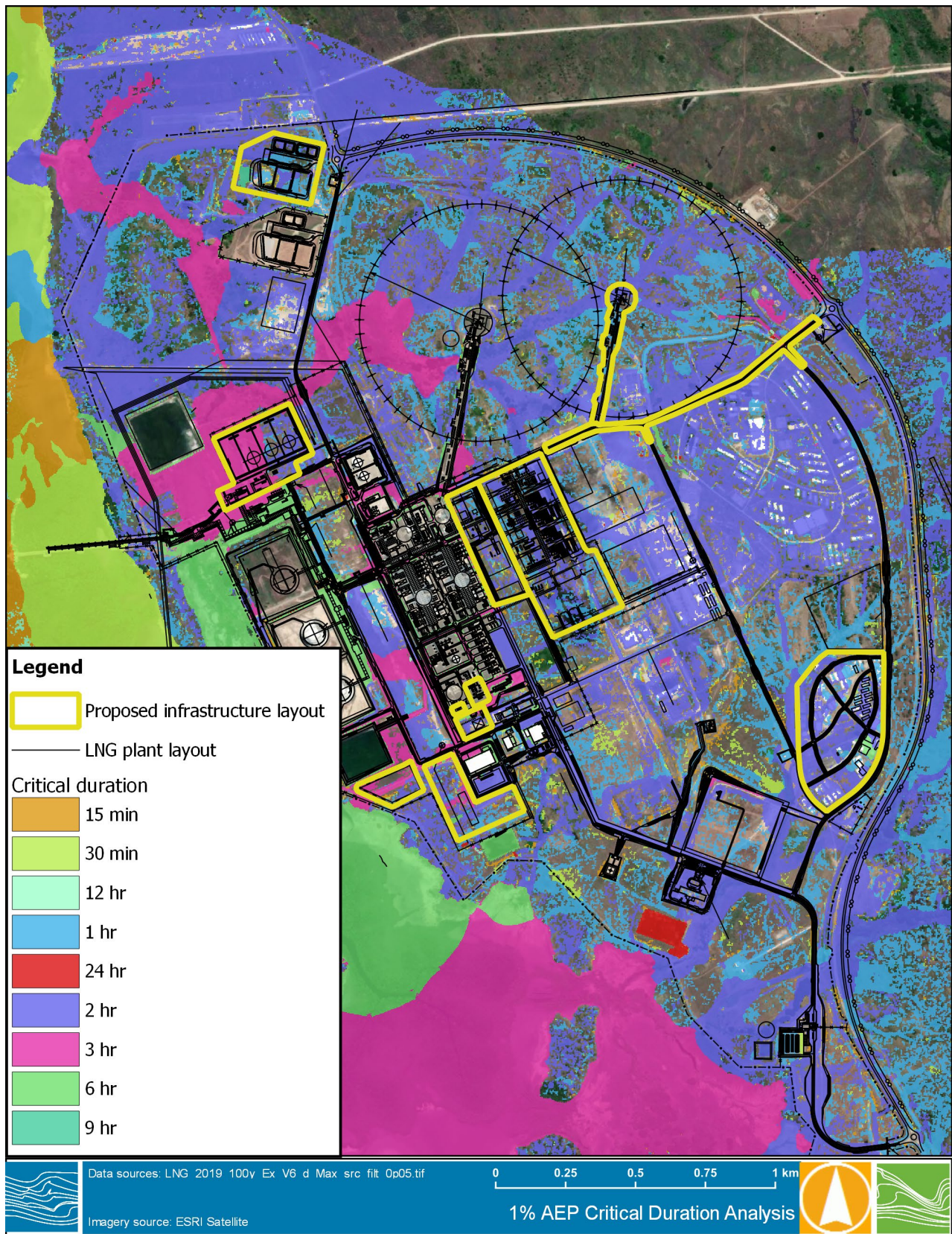


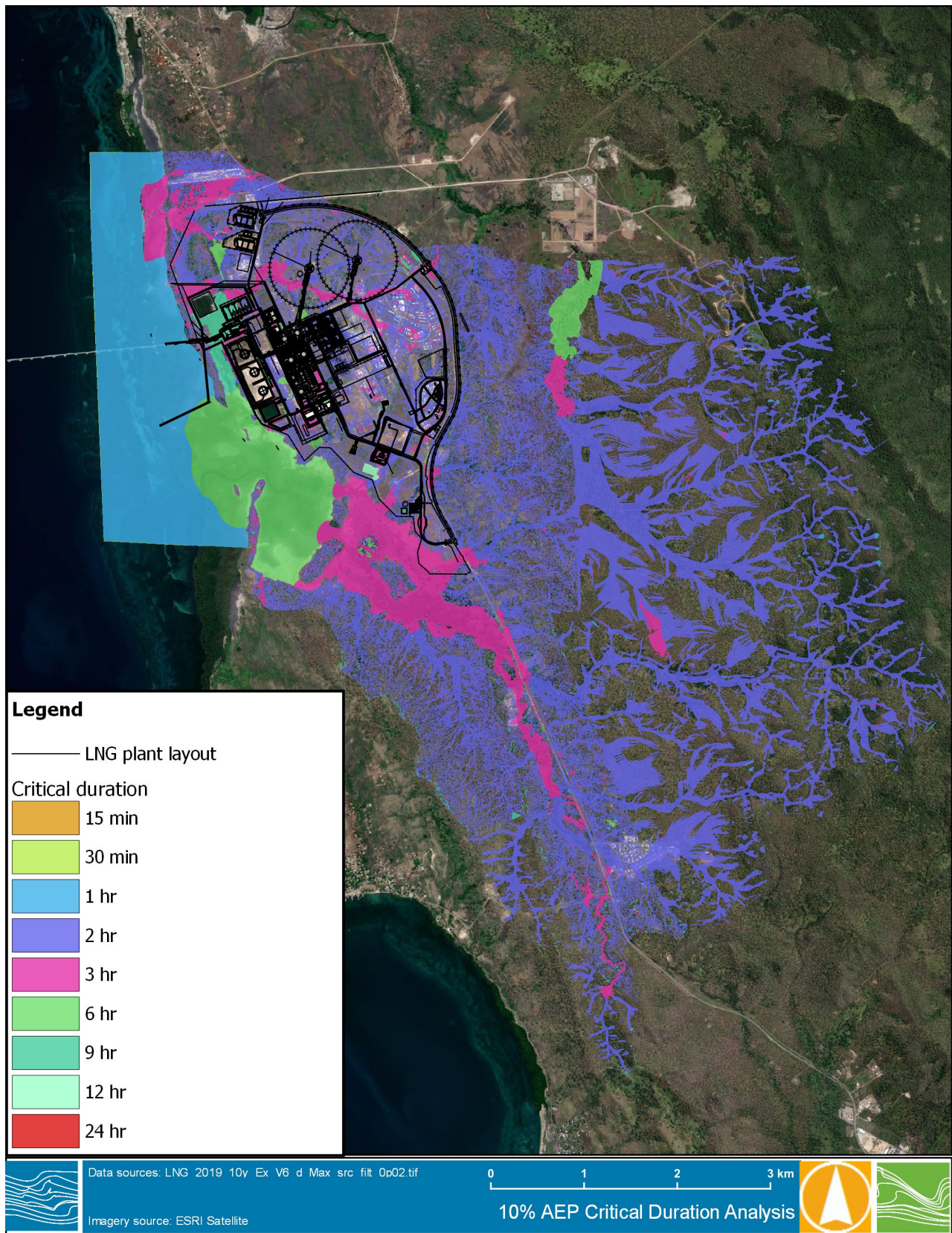
Figure 6-1 Catchment Scale – 1% AEP critical duration analysis



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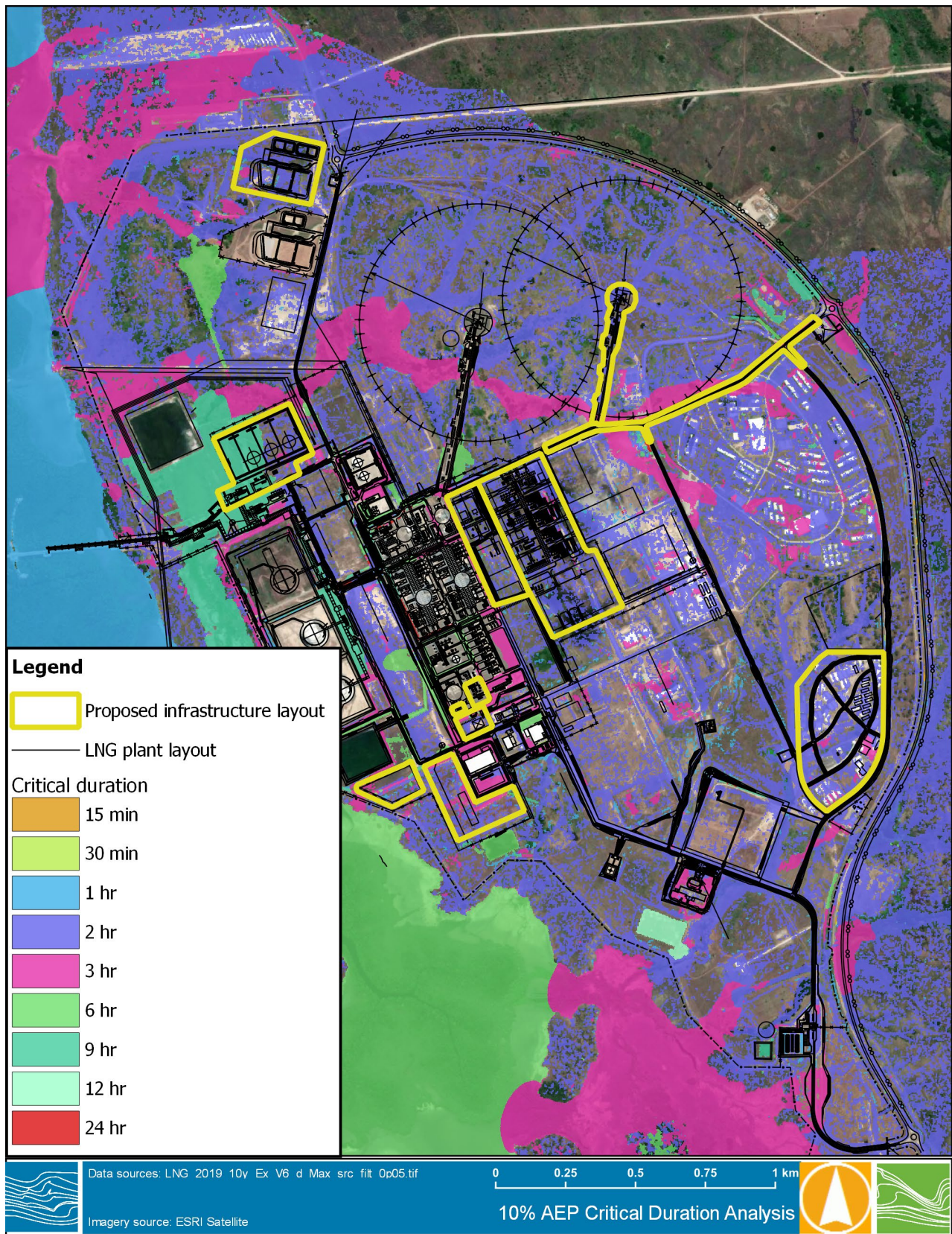
Figure 6-2 LNG Plant – 1% AEP critical duration analysis



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Figure 6-3 Catchment Scale – 10% AEP critical duration analysis



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Figure 6-4 LNG Plant – 10% AEP critical duration analysis



6.2 Comparison of Modelling Scenarios – Existing vs Proposed

TUFLOW modelling results for the existing and proposed scenarios are shown in Figure 6-5 to Figure 6-12 for both the 1% and 10% AEP flood events. Each figure shows flood depth and velocities to highlight zones of flood hazard in relation to the LNG Plant infrastructure existing and proposed footprint. These maps can be used to aid in the protection of roads, plant infrastructure, processing areas at the site during significant rainfall events. Flood depths below 50 mm were filtered out.

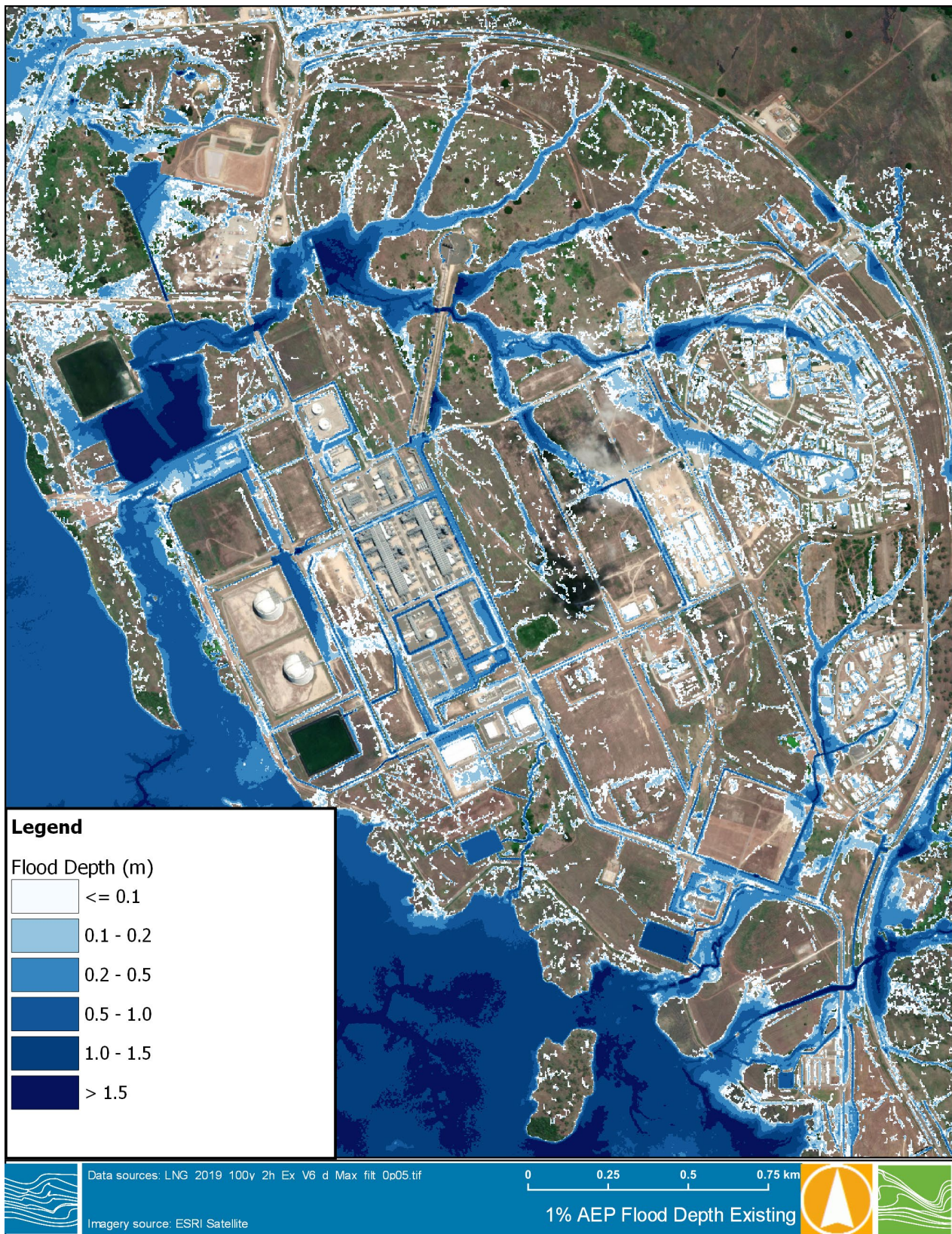
Hydraulic simulations were undertaken for the 1% and 10% AEP design flood magnitudes to establish the patterns of flood behaviour upstream of the plant infrastructure (trains, tanks, camp, road crossings) under both conditions; existing and proposed (i.e. expansion of LNG Plant). This demonstrated that under existing conditions (see Figure 6-5 to Figure 6-8), runoff on the eastern margins of the site is generally either confined to the drainage channels, runs along the existing roads or spreads laterally at the base of existing infrastructure. In the north western margins of the project area, surface water runoff slows and backs up in the vicinity of the condensate tanks. In this area the maximum flood depths extracted from the modelling are 2.3 m for both the 1% and 10% AEP events. Flow velocities generally mimic the topography with the higher velocities noted in the more defined parts of catchments near the existing flare; however, velocities are still generally less than 0.5 m/s across the site. Flow velocities are almost negligible (<0.1 m/s) in the far western margins of the site close to the coast which was expected given negligible topographic gradient.

Under the proposed developed scenario surface water modelling results suggest that runoff behaves in a similar fashion to the existing scenario for both the 1% and 10% AEP events, with the key differences being an increase in water levels on both sides of the access road running east-west past the two flare sites, increased water levels downstream of the proposed tanks and some local ponding upstream of proposed infrastructure. At the access road, water backs-up and spreads laterally at the base of the upstream side of the road, as well as in between the road and the proposed flare site. Flood depths in all other areas of the project area generally mimic the existing conditions scenario.

Flood hazard areas are highlighted in Figure 6-13 and Figure 6-14, showing ponding of water at the proposed processing trains and tanks, along the access road, at the flare stack and at the camp. Flood hazard areas were identified based on observed widespread and/or deep ponding behind roads and infrastructure. These are discussed further in relation to flood hazard in Section 7.

When considering flow velocities in the proposed post-development scenario, velocities generally remain the same as in the existing scenario, with minimal variations due to pooling of water, causing it to slow down.

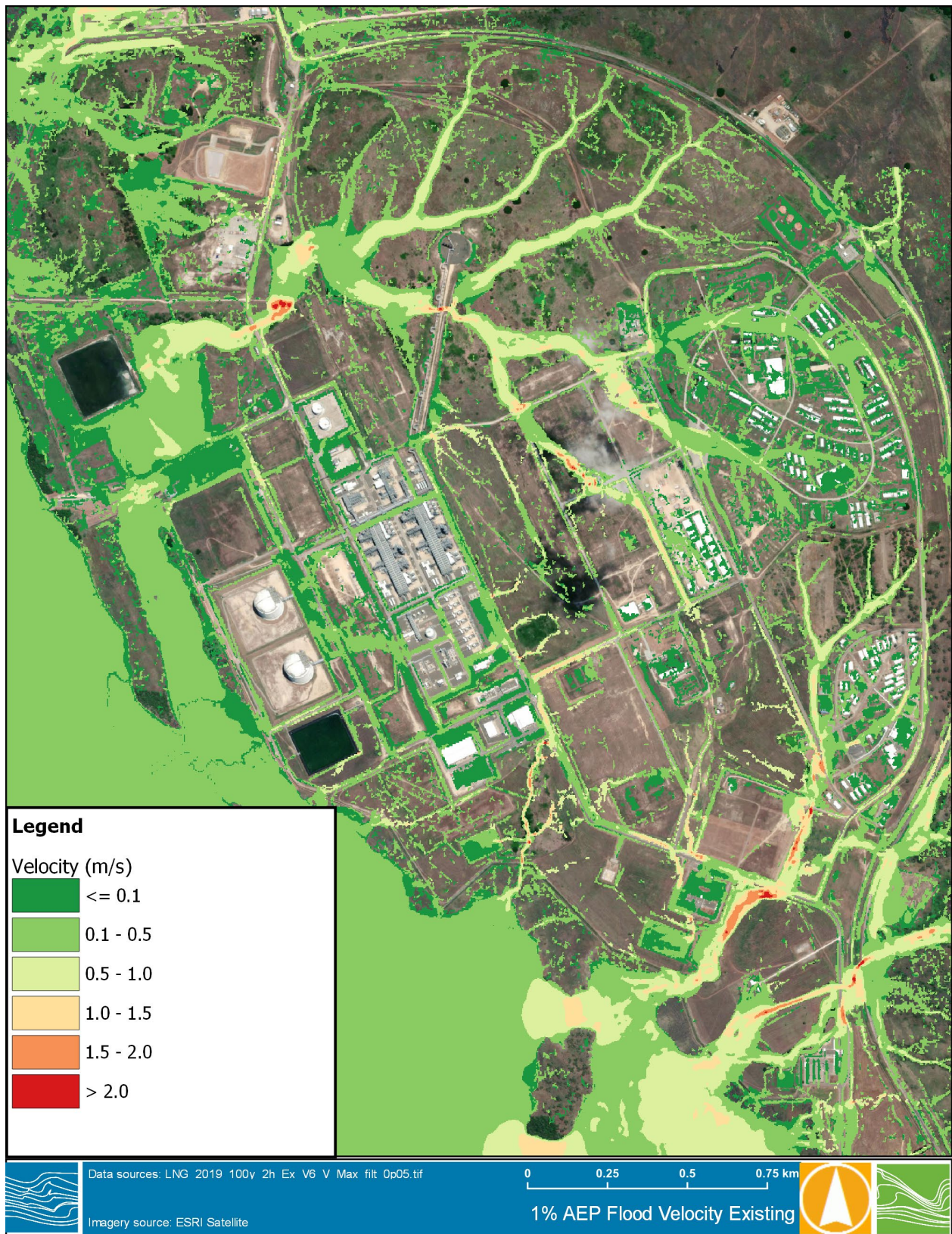
In summary, modelling suggests during both the 1% and 10% AEP events the proposed infrastructure is likely to have little impact on the existing flow regime away from the site, and minor impact within the site.



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Figure 6-5 1% AEP existing condition flood depth



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Figure 6-6 1% AEP existing condition velocity

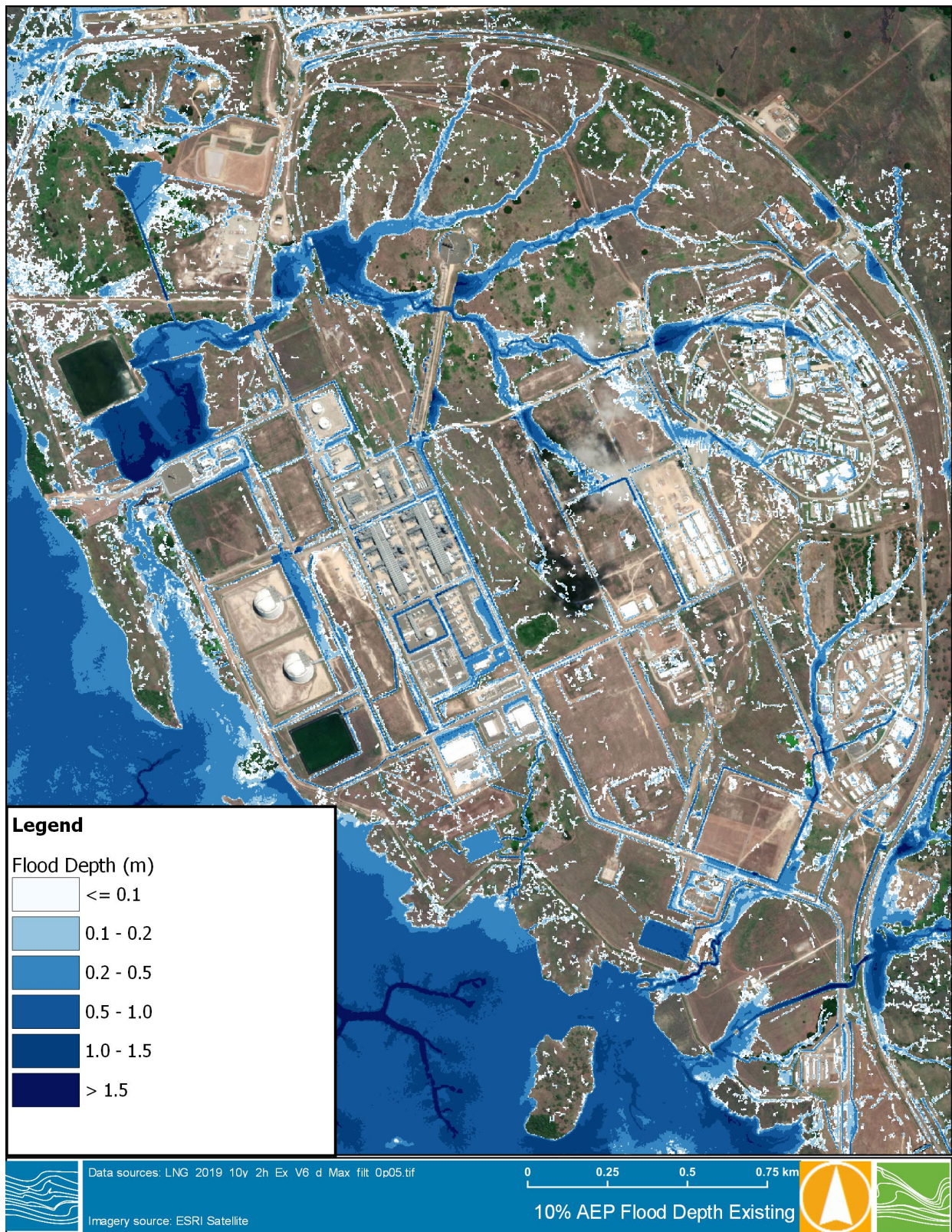


Figure 6-7 10% AEP existing condition flood depth

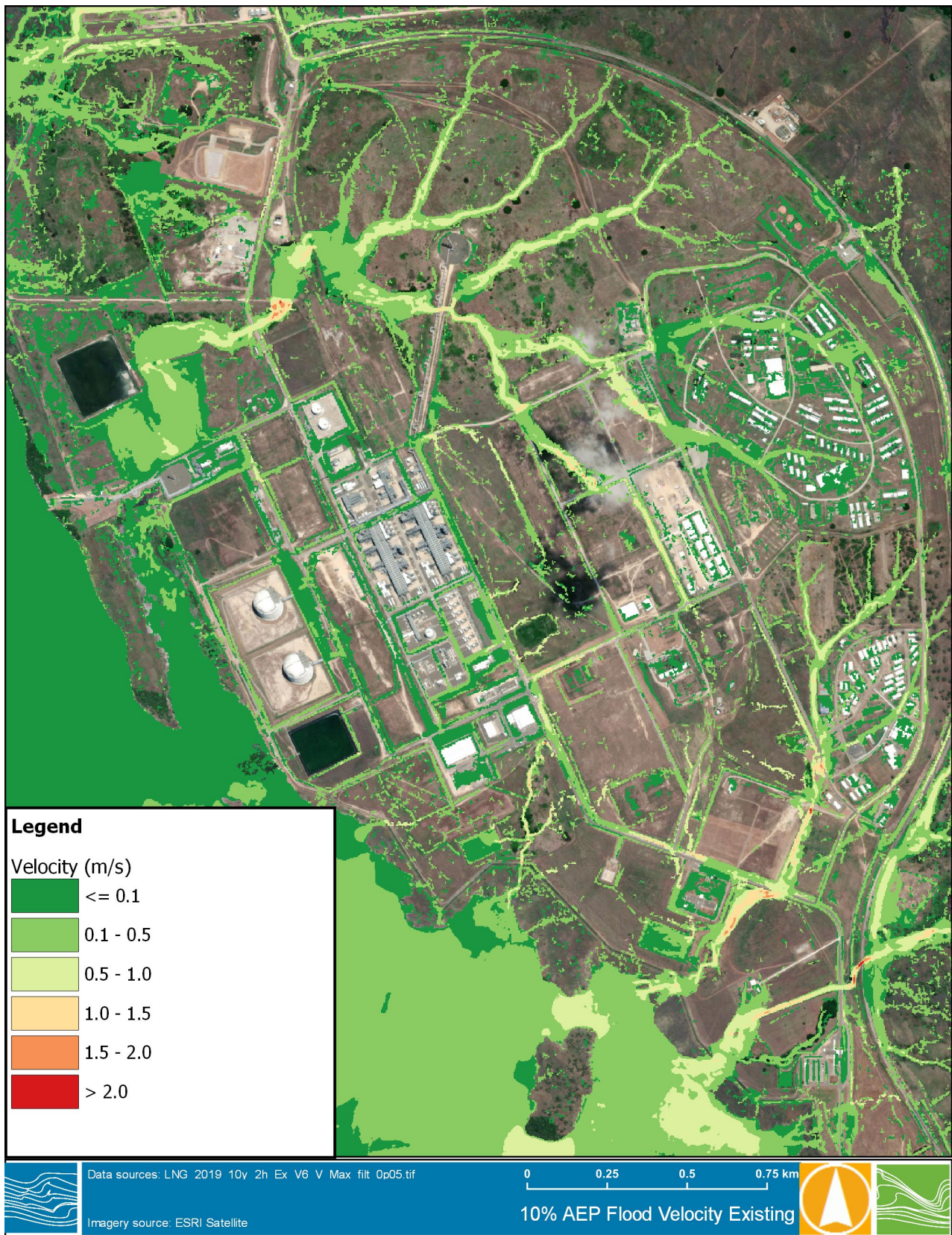
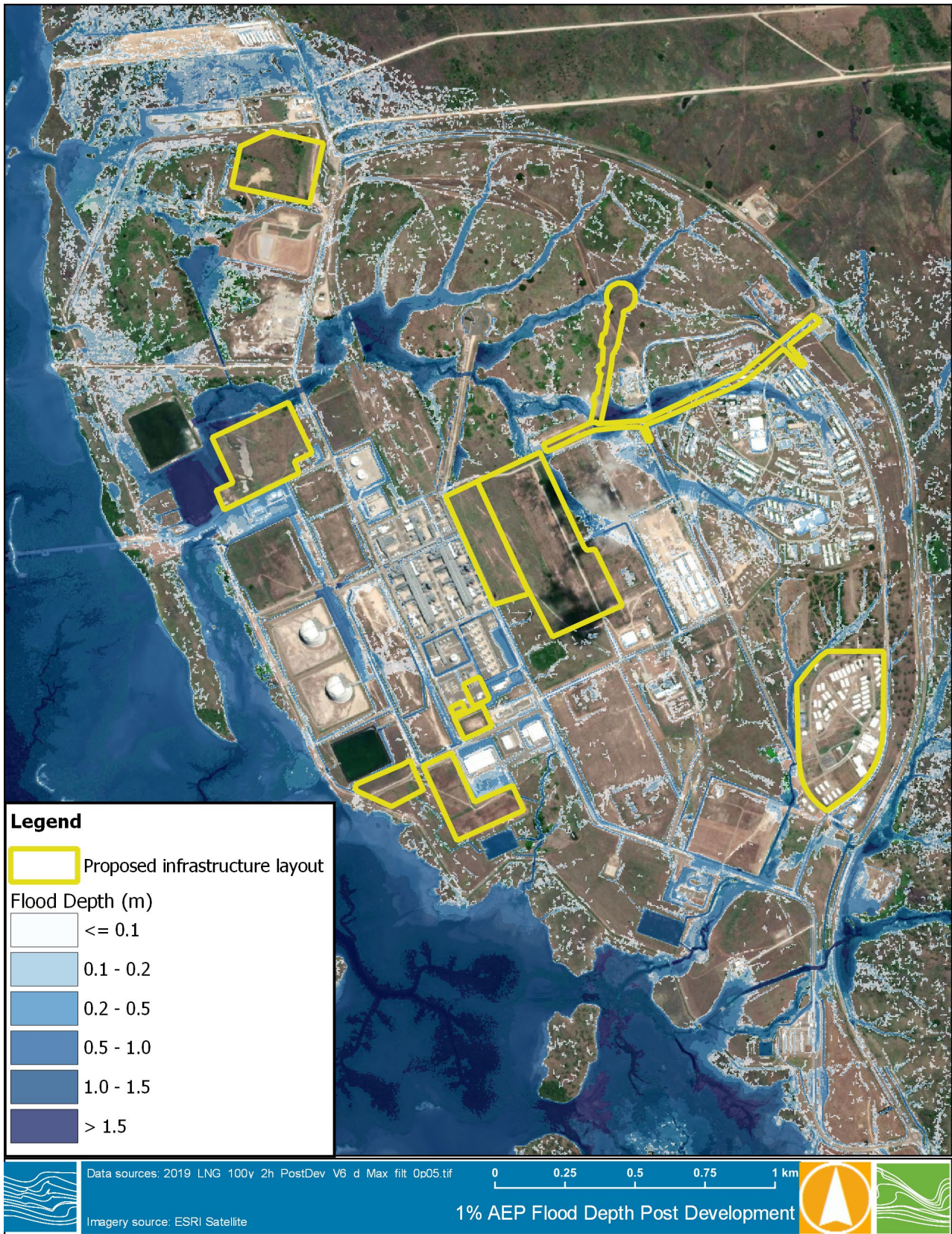


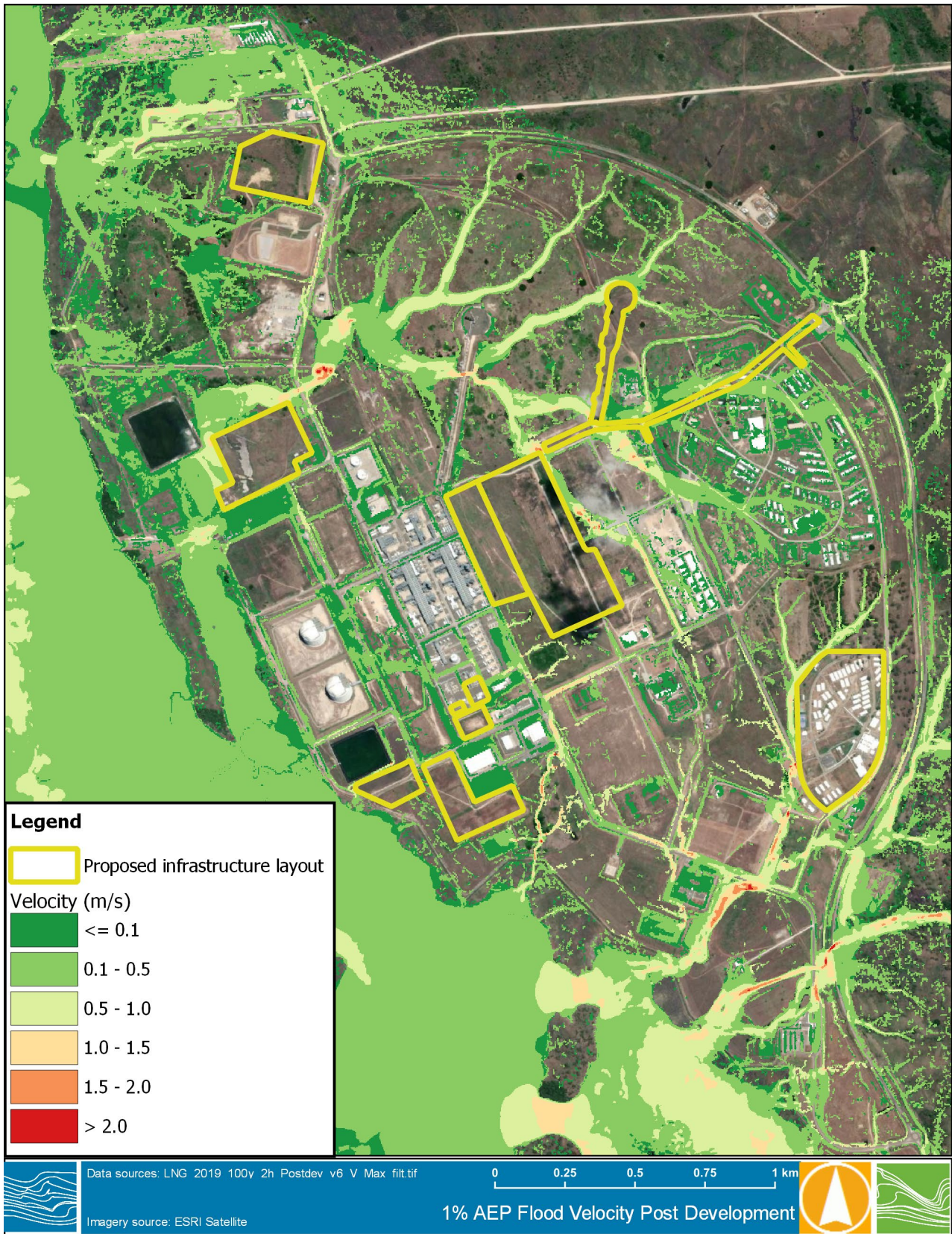
Figure 6-8 10% AEP existing condition velocity



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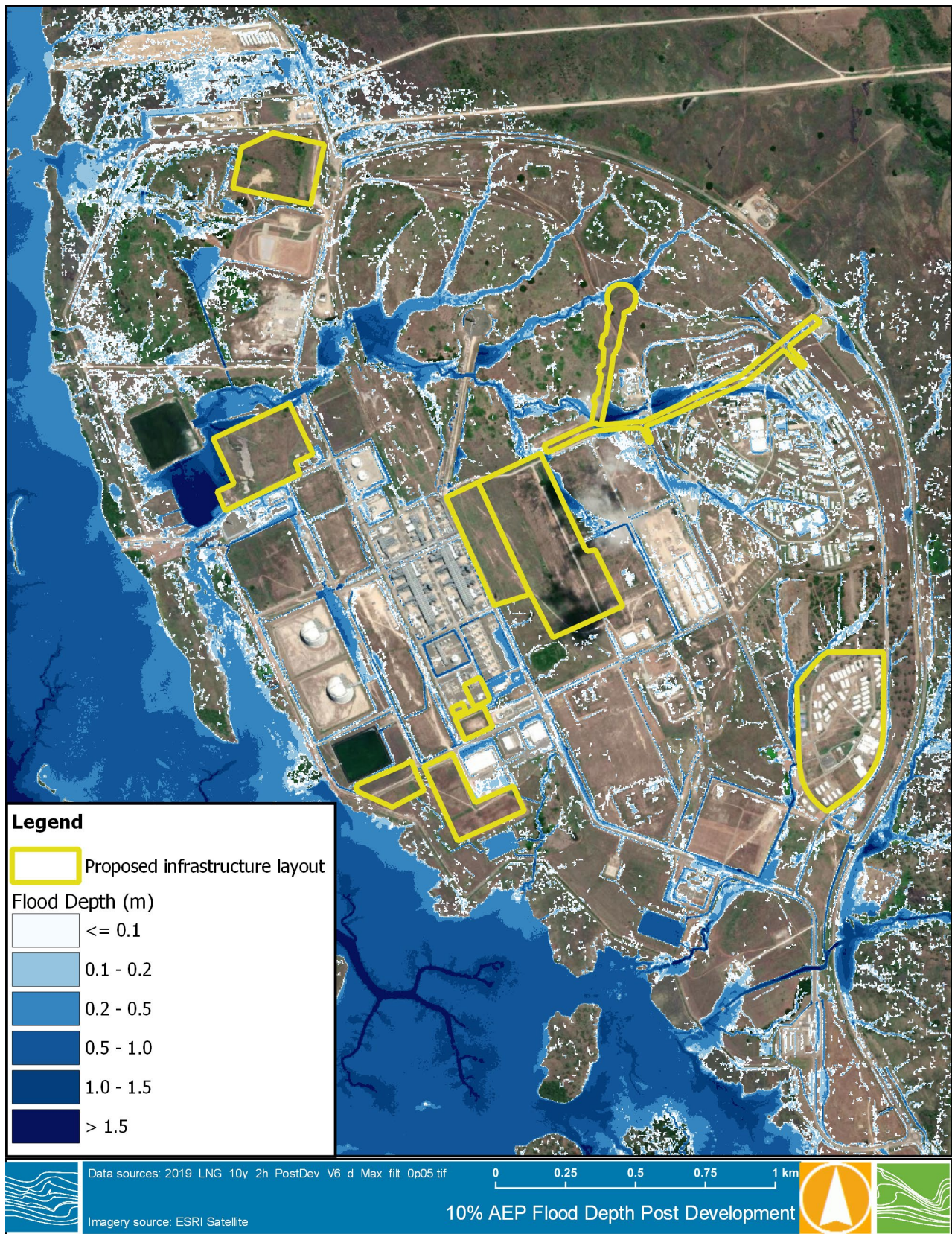
Figure 6-9 1% AEP post development condition flood depth



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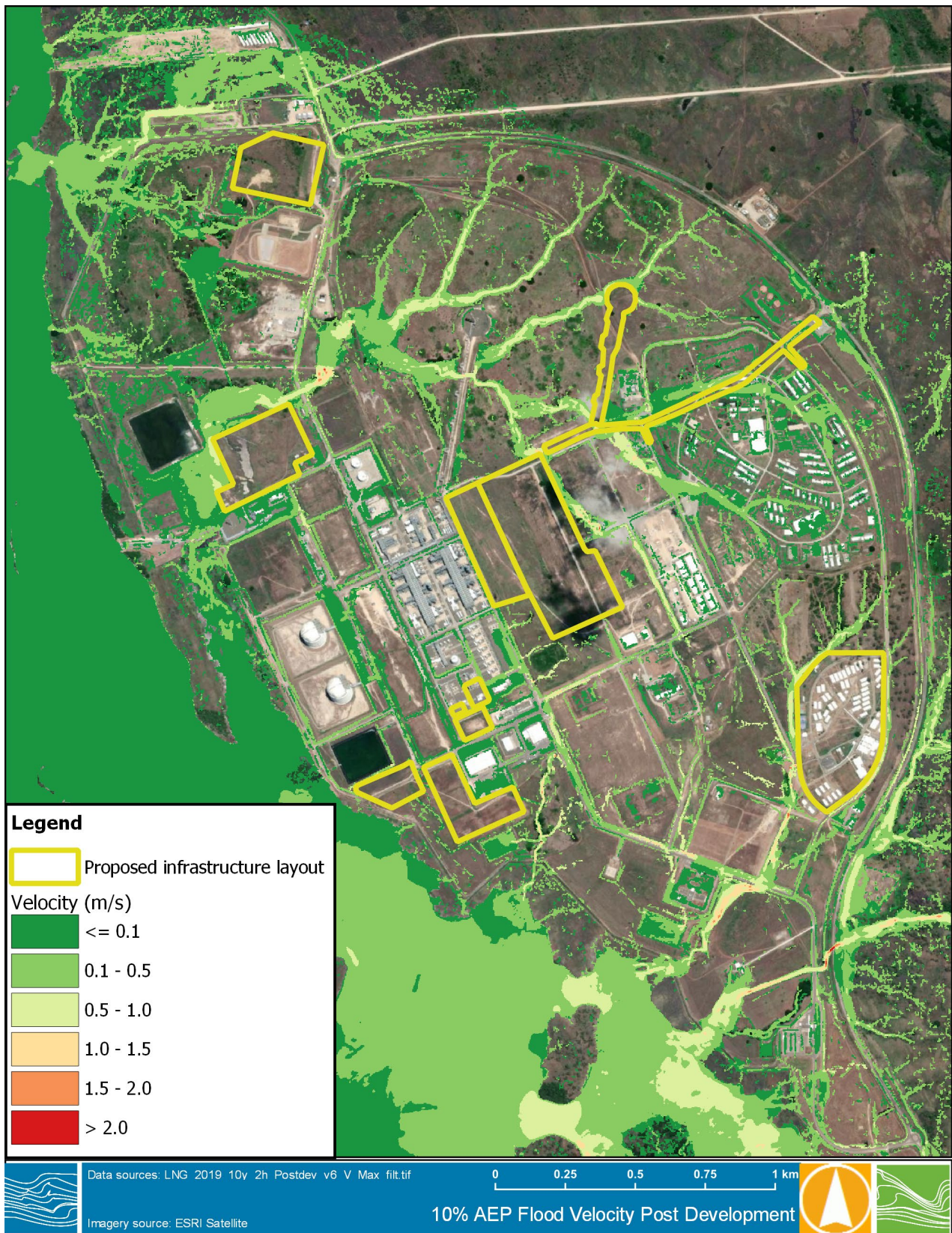
Figure 6-10 1% AEP post development condition velocity



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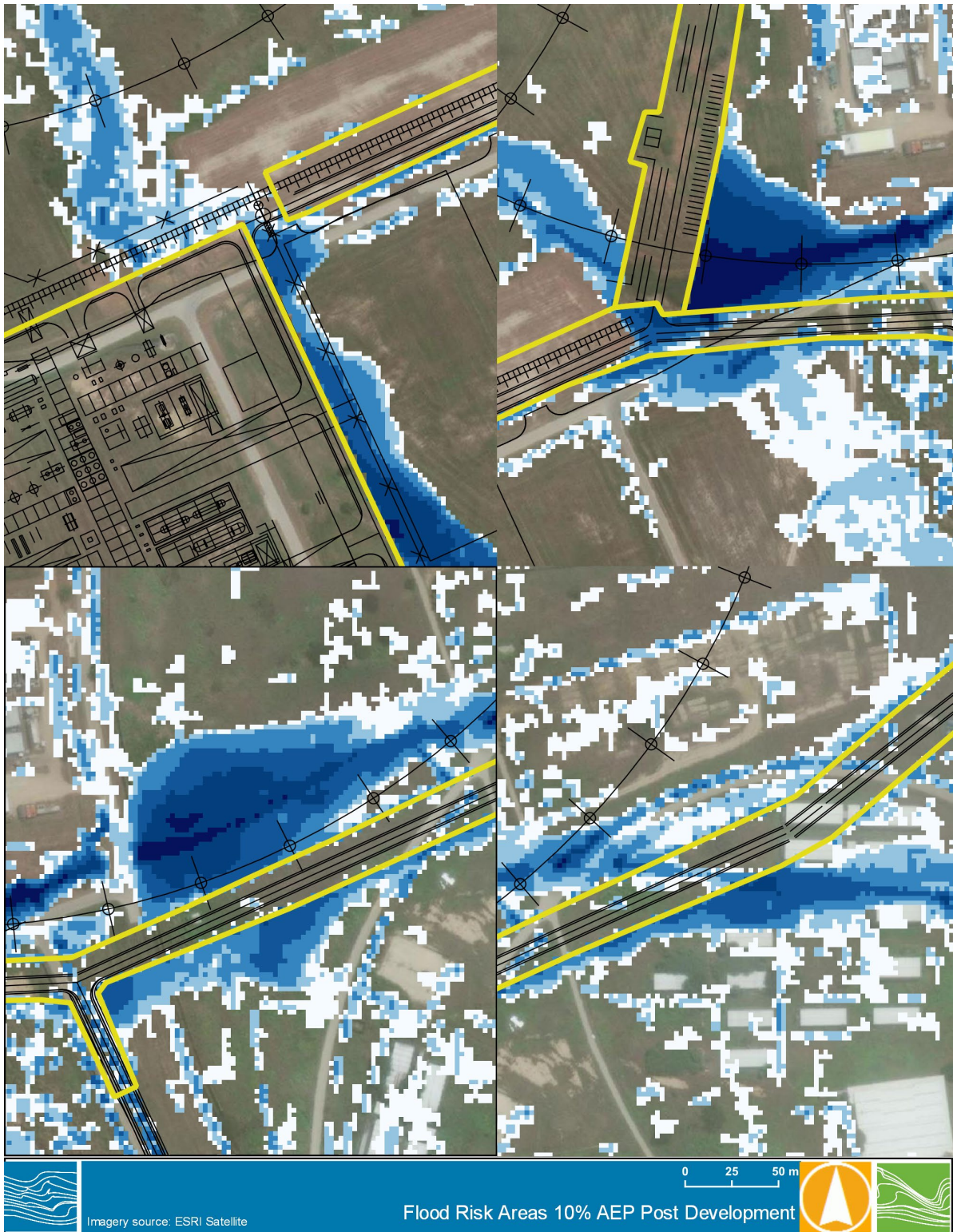
Figure 6-11 10% AEP post development condition flood depth



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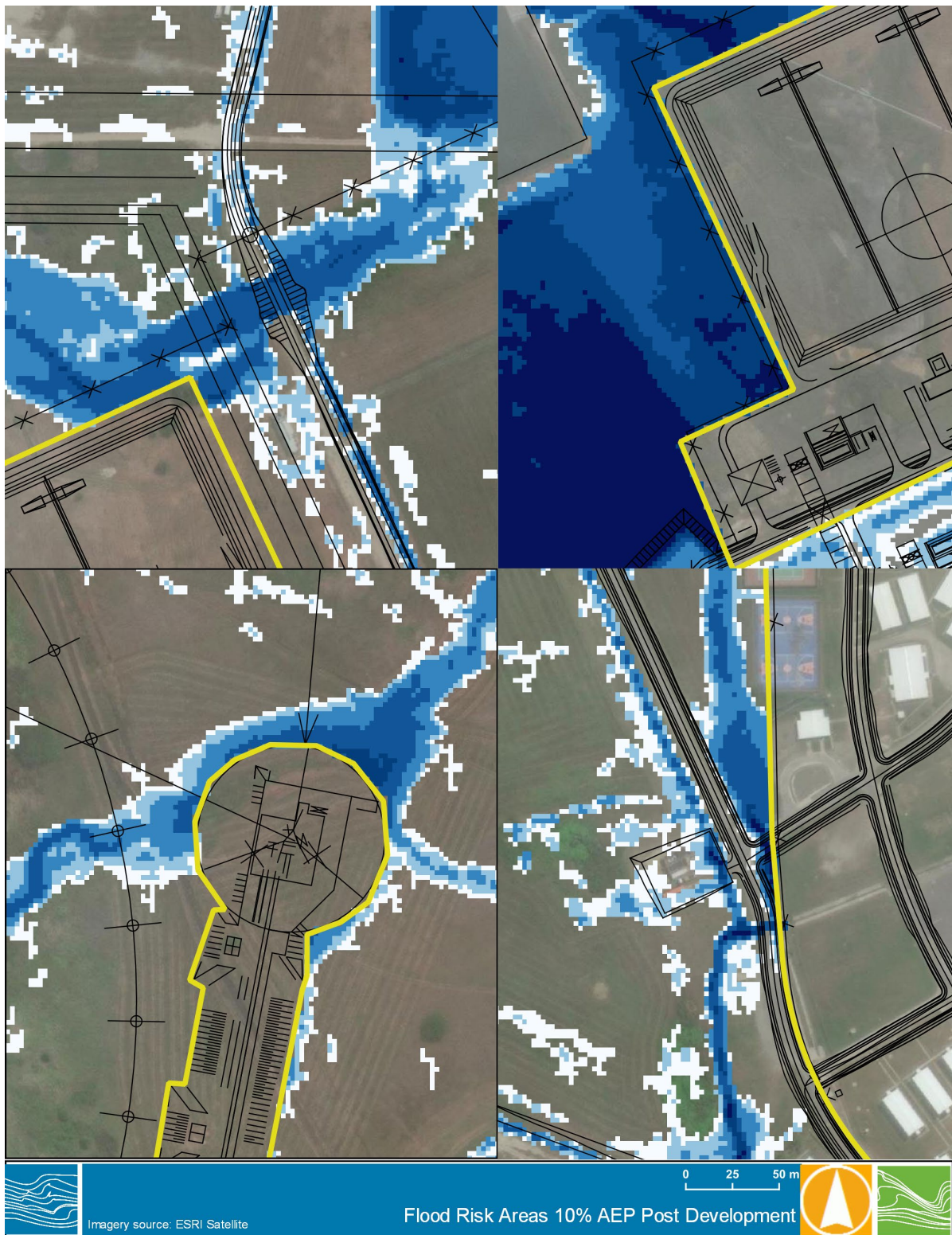
Figure 6-12 10% AEP post development condition velocity



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Figure 6-13 Flood risk areas: Top left – NE corner of processing trains, Top right – along access road, Bottom left – along access road, Bottom right – along access road



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Figure 6-14 Flood risk areas: Top left – NE corner of tanks, Top right – downstream of tanks, Bottom Left – at flare stack, Bottom right – at camp



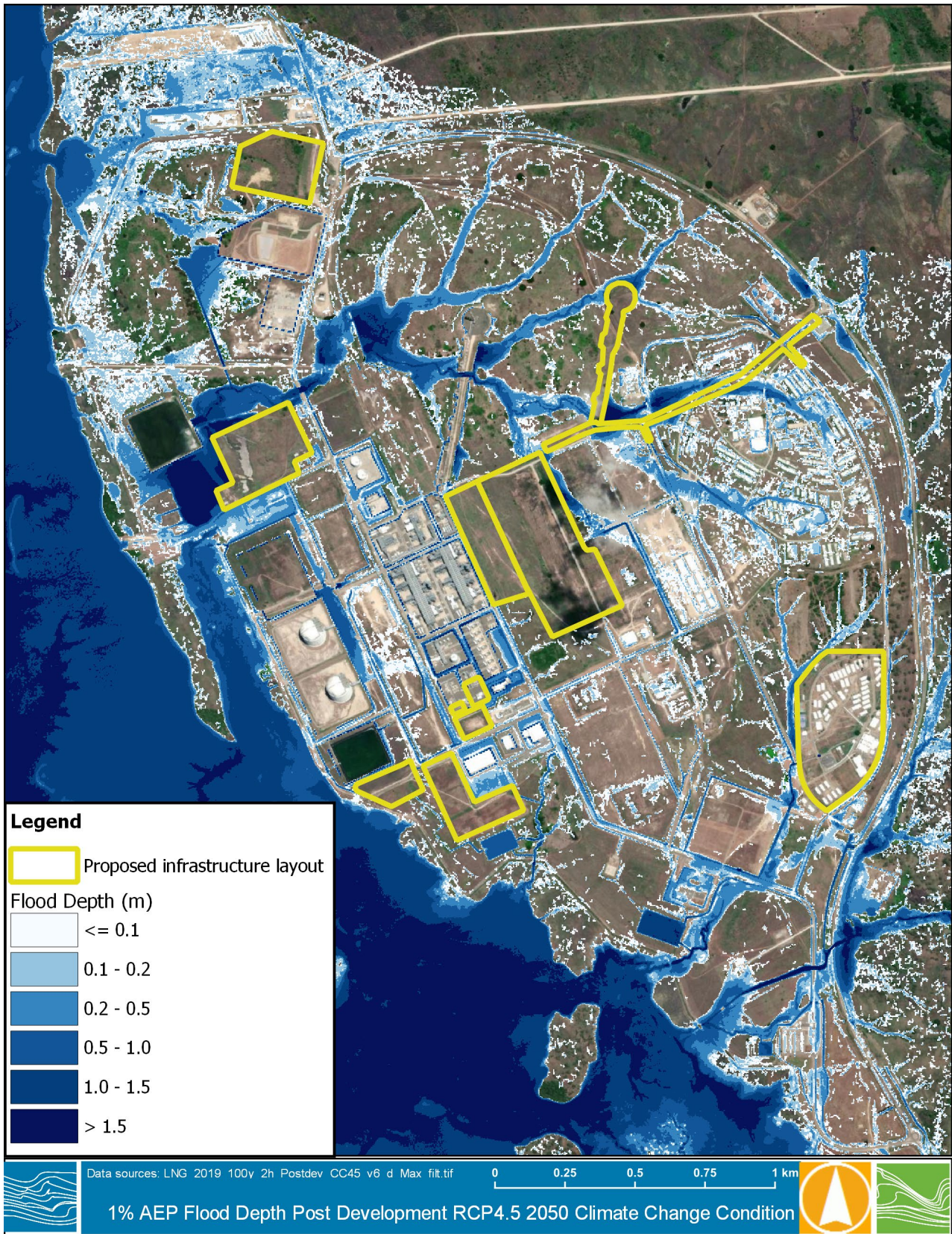
6.3 Climate Change Conditions

Modelling was conducted for the 1% and 10% AEP storm events for the 2050 and 2090 climate change scenarios; see Section 5.4.1 for greater detail. The results of the assessment are presented in Figure 6-15 to Figure 6-18 to quantify flood characteristics under the future 2050 and 2090 climate change scenarios.

When rainfall intensities were increased by 5% and the sea level was raised by 0.5 m as recommended for RCP4.5, the hydraulic model results under both flood events suggested flood depths increased with up to 0.5 m in the coastline part of the model (as prescribed by the boundary condition) but the flood extent did not increase here. Flood depths increased slightly within the overland flow paths intersecting the plant area, in most areas up to 0.03 m with increases up to 0.1 m in localised ponding areas. All changes to flood extents and depths were considered minimal and no additional flood hazard areas were identified.

When the rainfall intensities were increased by 16% and a sea level was raised by 0.8 m as recommended for RCP8.5, the hydraulic modelling results under both flood events suggested flood depths increased with up to 0.8 m in the coastline part of the model (as prescribed by the boundary condition) but the flood extent did not increase significantly here. Flood depths within the plant area increased within existing flow paths, in most areas up to 0.1 m with increases up to 0.25 m in localised ponding areas. However, the overall flood behaviour remained unchanged, with no further areas inundated outside of already existing flow paths.

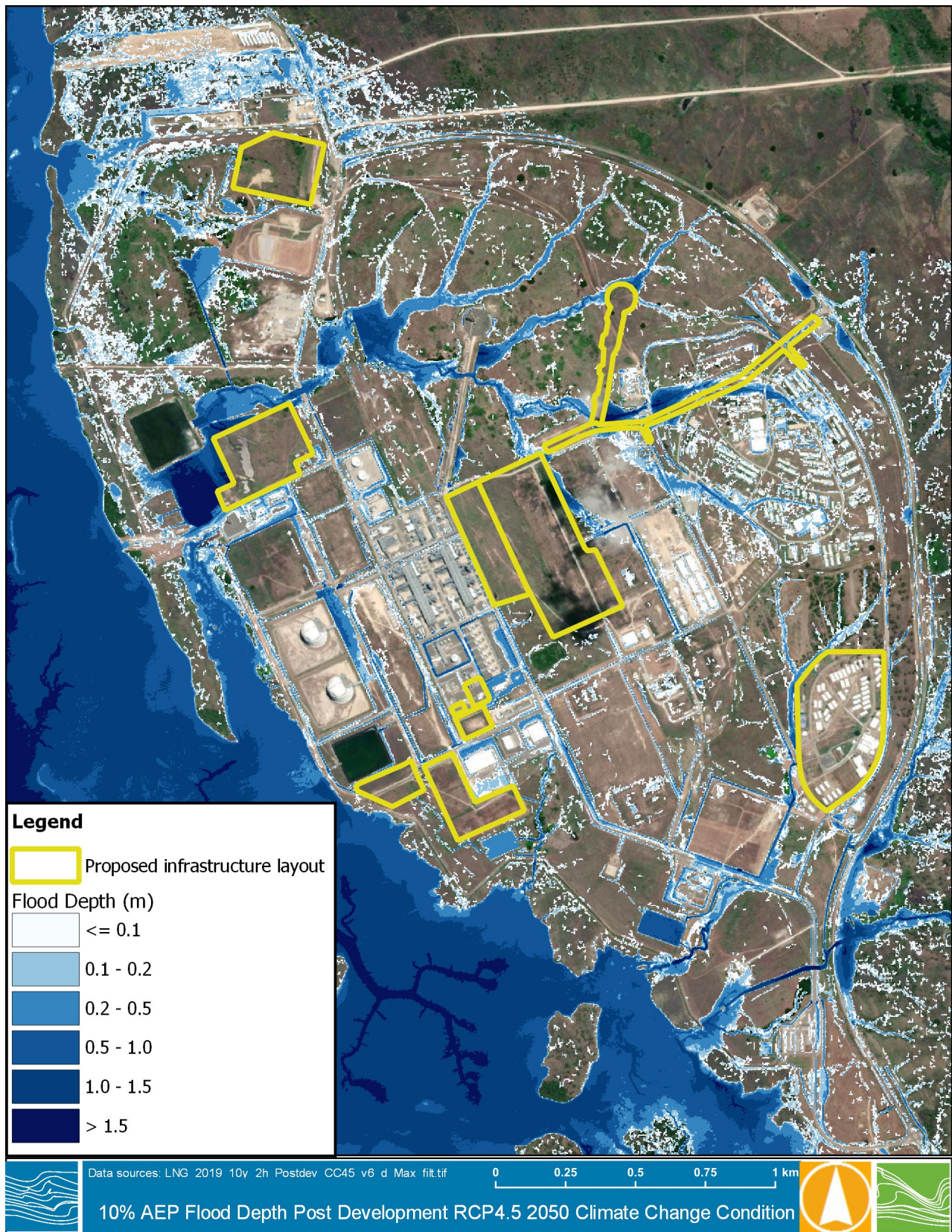
The increased flood depths could imply an increase in flood hazard in localised areas; however, as flood behaviour remained unchanged, with minor increases to extent and limited depth increased, no additional flood hazard areas were identified.



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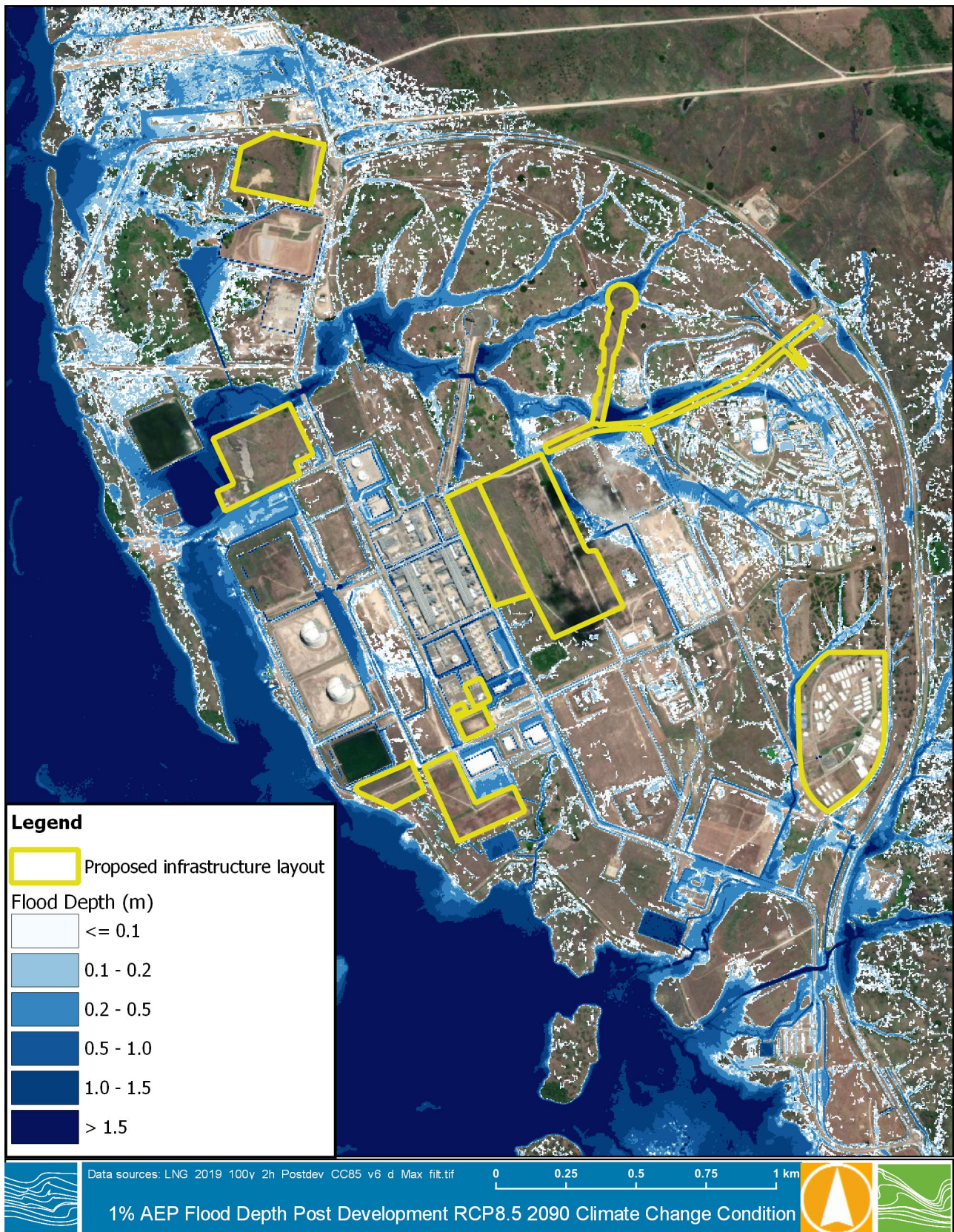
Figure 6-15 1% AEP post development RCP4.5 2050 climate change condition flood depths



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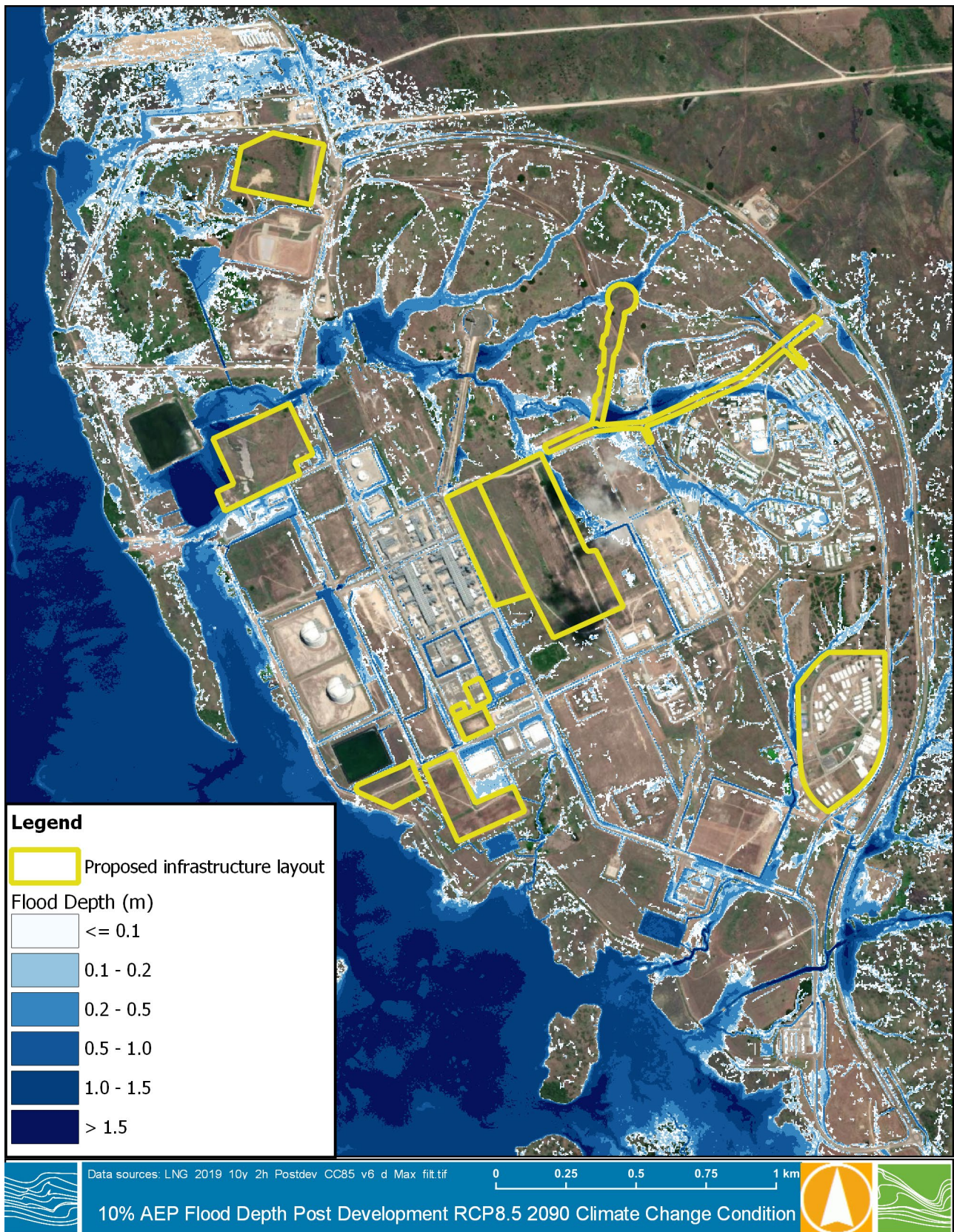
Figure 6-16 10% AEP Post Development RCP4.5 2050 climate change condition flood depths



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Figure 6-17 1% AEP Post Development RCP8.5 2090 climate change condition flood depths



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Figure 6-18 10% AEP Post Development RCP8.5 2090 climate change condition flood depths

7 FLOOD HAZARD ASSESSMENT

In the absence of available flood hazard categorisation guidelines for Papua New Guinea, this study applied the ARR 2016 flood hazard categories to the modelled flood depths and velocities. According to the ARR 2016 guidelines, flood depth, flood velocity and the product of depth and velocity are used to quantify and classify flood hazard. In the previous flood hazard assessment for this site (Water Technology, 2013) the flood hazard assessment considered people only, whilst vehicles and buildings were completed using separate flood hazard curves. Since Water Technology's 2013 study the ARR has been updated and now recommends the use of a combined set of hazard vulnerability curves which considers both people, buildings and vehicles for a strategic floodplain management assessment. This combined and more up-to-date flood hazard methodology has been adopted here. Figure 7-1 illustrates ARR's 2016 flood hazard categories in relation to depth, velocity and depth by velocity product. The specific flood hazard criteria are also provided in tabular form within Table 7-1.

A flood hazard assessment based on the criteria presented in Figure 7-1 was conducted for the 1% AEP event, for the proposed development scenario, not including climate change factors. Based on the small changes in depth and velocity on site observed during the climate change modelling, it was considered appropriate to conduct the flood hazard assessment based on the current day rainfall conditions. The results of this assessment are presented in Figure 7-2 and are discussed below.

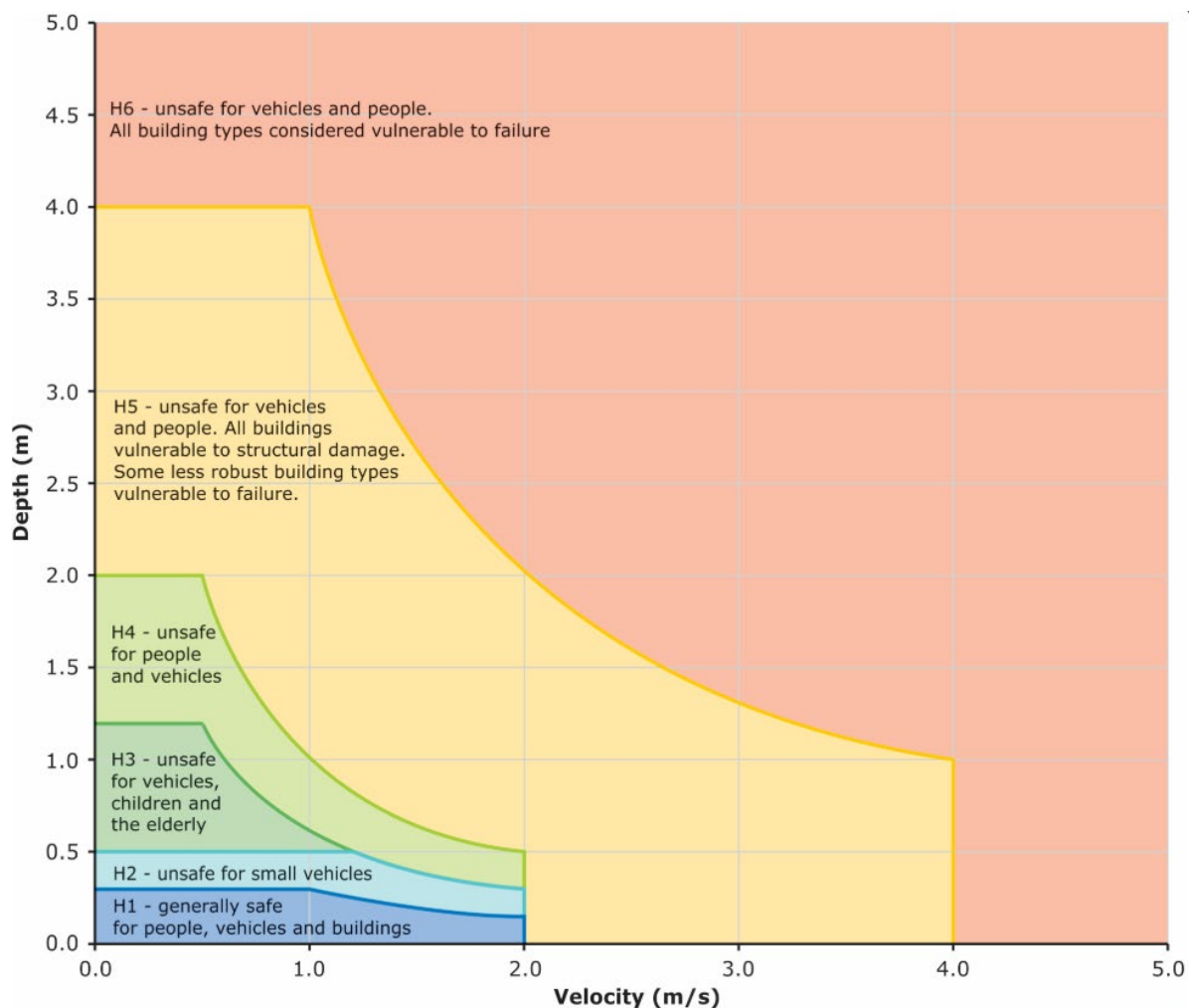


Figure 7-1 Combined flood hazard curves (Smith et al., 2004)



Table 7-1 Combined flood hazard curves vulnerability classification limits (Smith, et al., 2004)

Hazard Vulnerability Classification	Classification Limit (D and V in combination)	Limiting Still Water Depth (D)	Limiting Velocity (V)
H1	$D \cdot V \leq 0.3$	0.3	2.0
H2	$D \cdot V \leq 0.6$	0.5	2.0
H3	$D \cdot V \leq 0.6$	1.2	2.0
H4	$D \cdot V \leq 1.0$	2.0	2.0
H5	$D \cdot V \leq 4.0$	4.0	4.0
H6	$D \cdot V > 4.0$	-	-

D = depth. V = velocity

7.1 Analysis of Flood Hazard Mapping

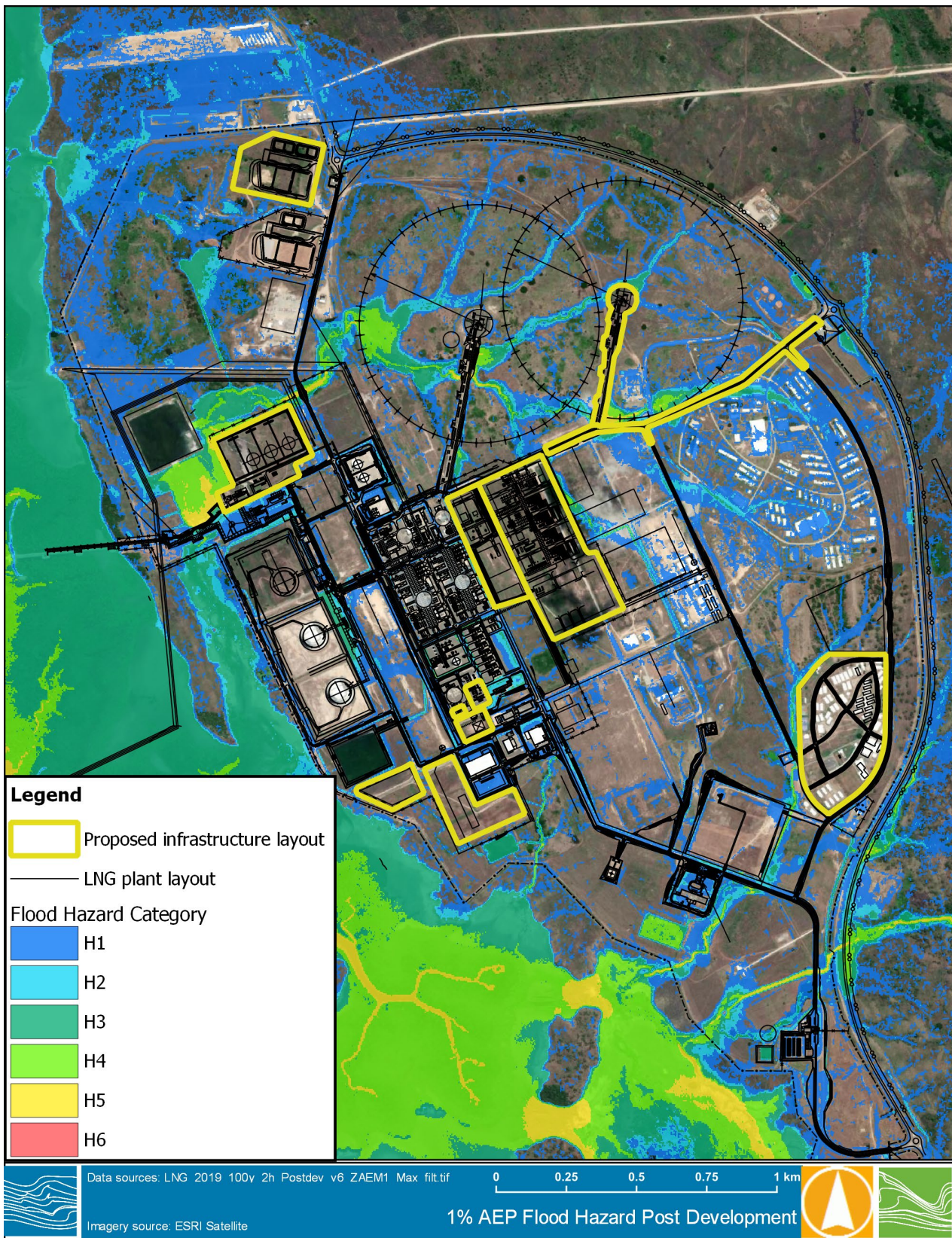
Analysis of the 1% AEP flood hazard mapping with the post development conditions analysis in Section 6.2 has highlighted areas within the LNG Plant site that have varying levels of Flood Hazard. These areas are mainly adjacent to roads and infrastructure where flooding could be unsafe for vehicles and pedestrians, potentially causing accidents.

The five key areas were identified as potentially hazardous during the 1% AEP event are presented in Table 7-2 and their locations are shown in Figure 7-3 to Figure 7-5.

The flood hazard assessment largely suggests there are some localised areas across the site that are prone to flooding, which may cause accidents during the 1% AEP event if people are present. Of importance are the high traffic areas such as Zone 5 near the camp, Zone 1 and 2 in close proximity to the condensate tanks and Zone 4 adjacent to the proposed flare stack. The importance of these areas relates to the movement of people and vehicles and as such these areas should be managed (to reduce the flood hazard) by adopting appropriate mitigation measures.

Table 7-2 1% AEP event - key flood hazard areas

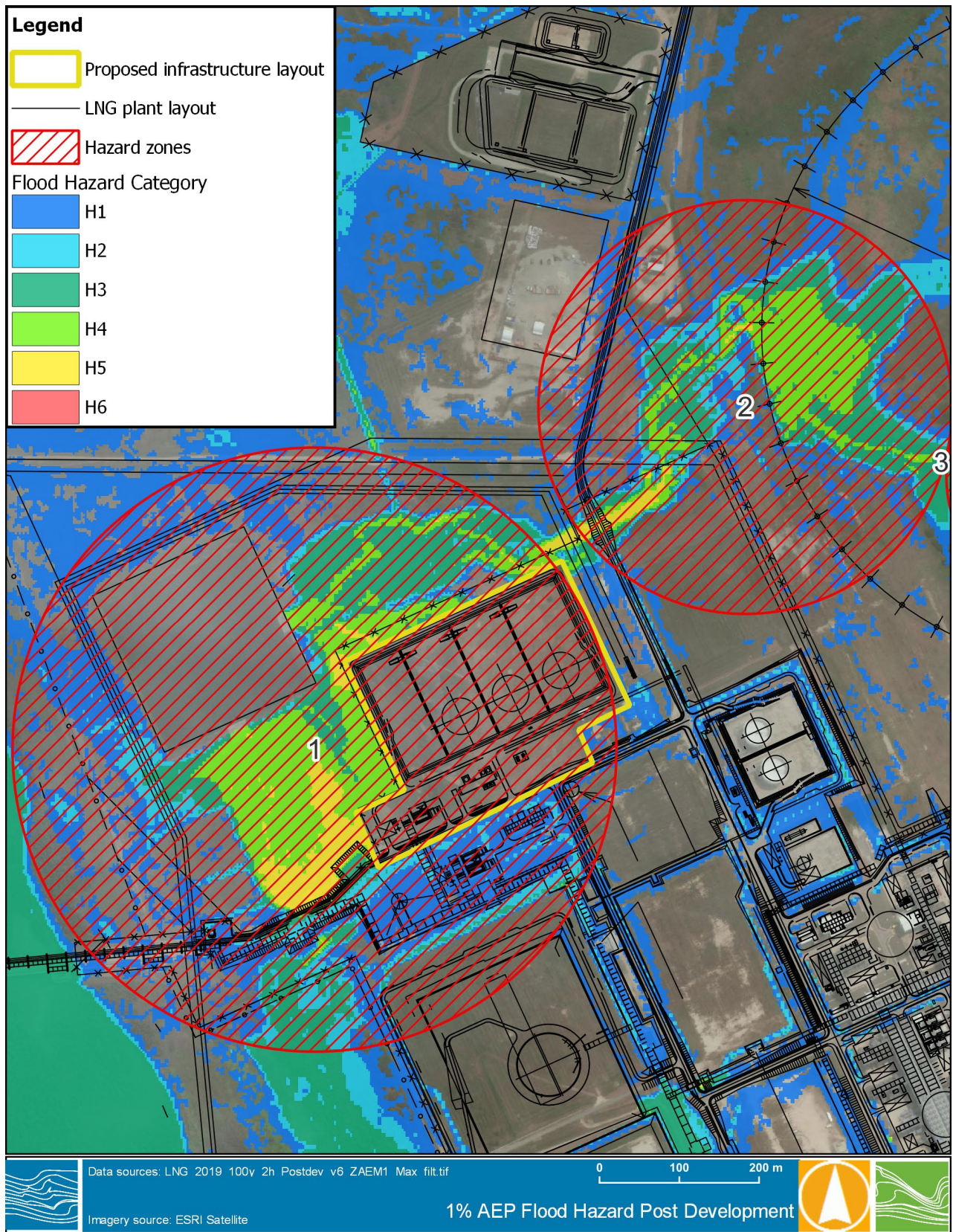
Flood Hazard Zone ID	Area Description	Average Flood Hazard Ranking for Area	Maximum Flood Hazard Ranking
1	Immediately north and west of condensate tanks	H4 - Unsafe for people and vehicles.	H5 - Unsafe for vehicles and people. All buildings vulnerable to structural damage.
2	North of Naphtha tanks	H4 - Unsafe for people and vehicles.	H5 - Unsafe for vehicles and people. All buildings vulnerable to structural damage.
3	Existing flare	H4 - Unsafe for people and vehicles.	H5 - Unsafe for vehicles and people. All buildings vulnerable to structural damage.
4	South of proposed flare	H4 - Unsafe for people and vehicles.	H5 - Unsafe for vehicles and people. All buildings vulnerable to structural damage.
5	Western margin of camp	H3 - Unsafe for vehicles, children and elderly.	H6 - Unsafe for vehicles and people. All building types considered vulnerable to failure.



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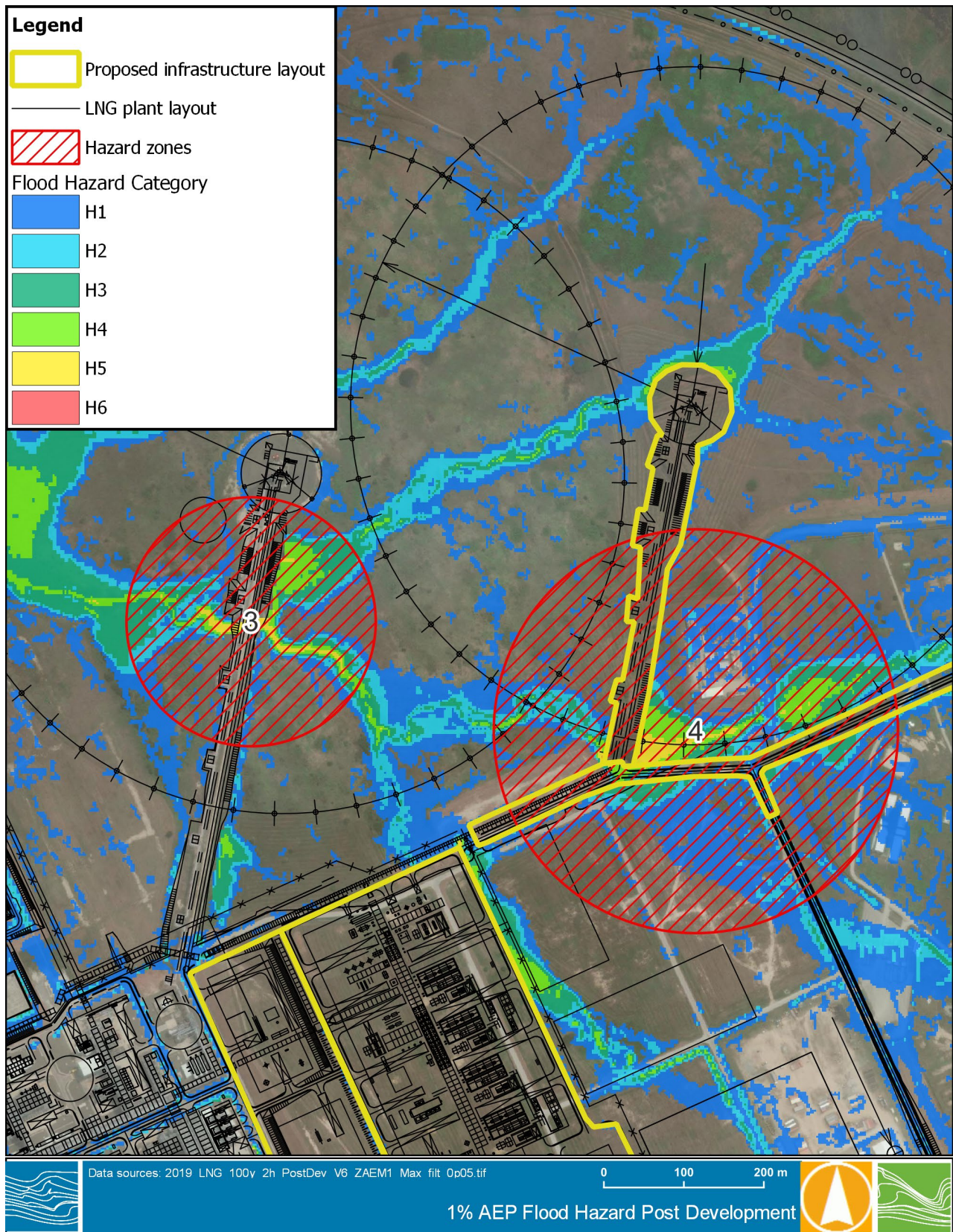
Figure 7-2 1% AEP Post development scenario hazard mapping



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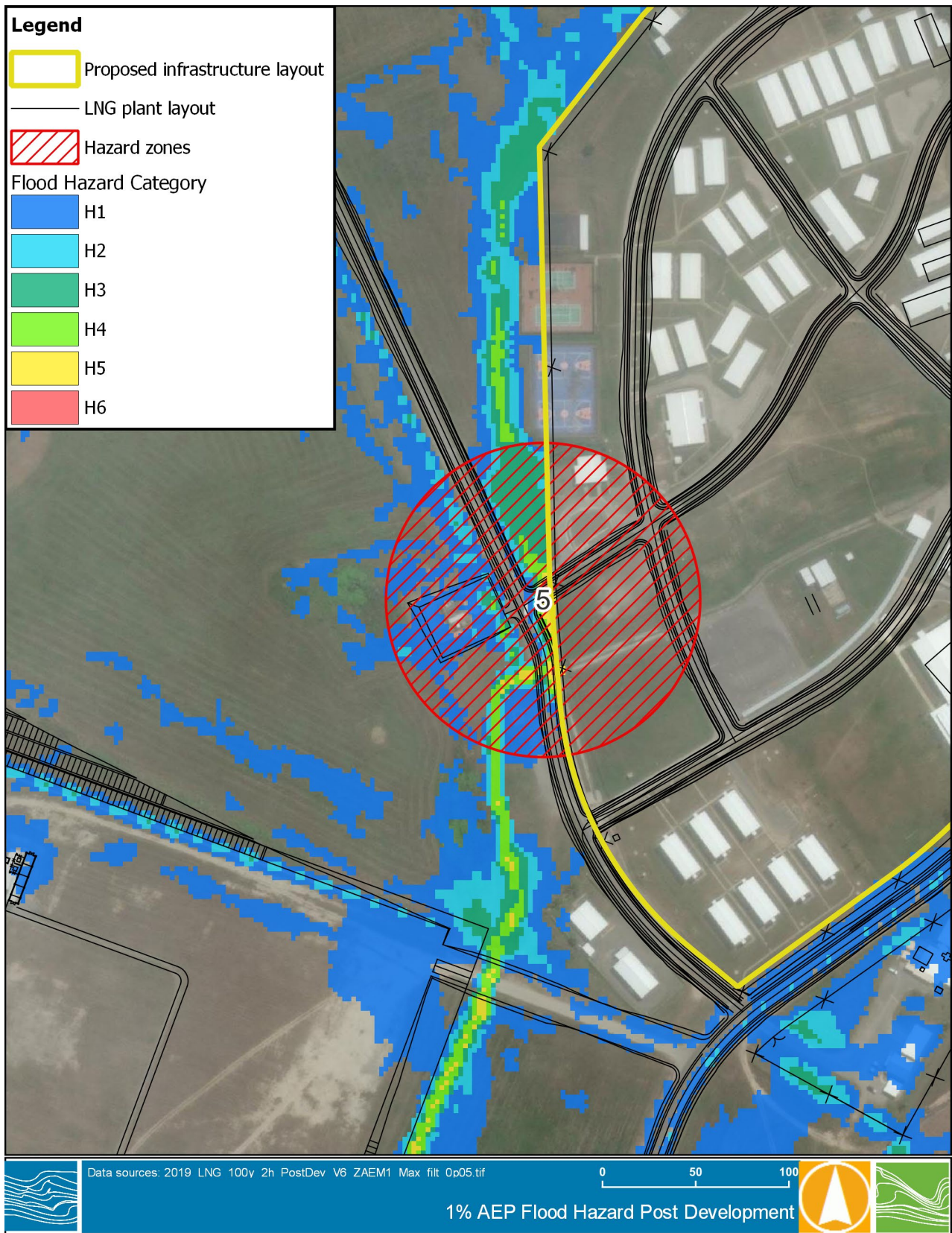
Figure 7-3 1% AEP post development- Flood hazard zones 1 & 2



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Figure 7-4 1% AEP post development - Flood hazard zones 3 & 4



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Figure 7-5 1% AEP post development- Flood hazard zones 5



8 CONCLUSIONS

Water Technology conducted a flood assessment to understand flood hazard for the Papua LNG Project expansion at the LNG Plant site. The flood assessment showed:

- Modelling suggests during both the 1% and 10% AEP event the proposed infrastructure is likely to have little impact on the existing flow regime. Proposed infrastructure located within the existing flood extents include the condensate tanks and proposed roads and mitigation is required to prevent flooding of these assets.
- The flood modelling under the proposed scenario indicates that flow velocities in vicinity of the proposed site infrastructure are negligible (<2 m/s) and as such erosion protection is not required.
- The flood hazard assessment identified five areas that are prone to hazardous flooding, based on flood depths and velocities. These five areas have been demarcated and a specific flood management strategy should be developed for these areas. These five areas are:
 - Immediately north and west of condensate tanks
 - West of the existing flare stack/north of Naphtha tanks
 - At the existing flare stack
 - At the proposed new flare stack
 - At the western margin of the camp
- When the rainfall intensities were increased and the sea level was raised during climate change conditions, both flood events and both climate change scenarios suggested flood depths increase on the western margins of the Project (adjacent to the coastline) with slight increases to flood extent. Flood depths also increase within the infrastructure area within existing flow paths and areas of ponding. However, all changes to flood extents and depths were minor and did not alter the impact assessment.



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APPENDIX A TEMPORAL RAINFALL PATTERNS





Table 9-1 Temporal patterns for rainfall durations up to 2 hours

Time (hr)	1% AEP rainfall (mm)				10% AEP rainfall (mm)			
	0.25 hr	0.5 hr	1 hr	2 hr	0.25 hr	0.5 hr	1 hr	2 hr
0.00	0	0	0	0	0	0	0	0
0.08	14.4	9.7	5.0	4.5	8.6	6.2	3.2	2.9
0.17	22.5	12.1	8.0	3.9	13.5	7.7	5.1	2.6
0.25	8.1	17.5	8.8	5.0	4.9	11.1	5.7	3.2
0.33	0	14.7	9.5	6.0	0	9.3	6.1	3.9
0.42	0	11.2	11.8	3.4	0	7.1	7.6	2.2
0.50	0	8.8	15.0	5.0	0	5.6	9.7	3.2
0.58	0	0	12.6	4.2	0	0	8.1	2.8
0.67	0	0	10.5	3.8	0	0	6.7	2.5
0.75	0	0	9.8	8.0	0	0	6.3	5.2
0.83	0	0	8.0	8.8	0	0	5.1	5.7
0.92	0	0	6.0	10.5	0	0	3.9	6.8
1.00	0	0	4.0	13.9	0	0	2.6	9.0
1.08	0	0	0	11.7	0	0	0	7.6
1.17	0	0	0	9.5	0	0	0	6.2
1.25	0	0	0	8.0	0	0	0	5.2
1.33	0	0	0	4.7	0	0	0	3.0
1.42	0	0	0	6.3	0	0	0	4.1
1.50	0	0	0	3.8	0	0	0	2.5
1.58	0	0	0	3.4	0	0	0	2.2
1.67	0	0	0	5.3	0	0	0	3.4
1.75	0	0	0	4.2	0	0	0	2.8
1.83	0	0	0	4.4	0	0	0	2.9
1.92	0	0	0	5.5	0	0	0	3.6
2.00	0	0	0	2.3	0	0	0	1.5

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Table 9-2 Temporal patterns for rainfall durations 3 hours and 6 hours

Time (hr)	1% AEP rainfall (mm)		10% AEP rainfall (mm)	
	3	6	3	6
0	0	0	0	0
0.25	10.0	3.3	6.5	2.1
0.5	7.8	3.3	5.1	2.1
0.75	12.5	6.6	8.2	4.2
1	14.0	6.6	9.2	4.2
1.25	16.2	5.4	10.7	3.4
1.5	28.2	8.4	18.5	5.4
1.75	23.2	4.7	15.2	3.0
2	14.0	9.7	9.2	6.2
2.25	15.5	5.6	10.2	3.5
2.5	10.8	11.9	7.1	7.6
2.75	11.2	16.1	7.3	10.2
3	5.6	25.5	3.7	16.2
3.25	0	18.1	0	11.5
3.5	0	13.8	0	8.8
3.75	0	9.7	0	6.2
4	0	10.9	0	6.9
4.25	0	7.0	0	4.5
4.5	0	5.4	0	3.4
4.75	0	7.8	0	5.0
5	0	10.3	0	6.6
5.25	0	4.9	0	3.1
5.5	0	4.3	0	2.8
5.75	0	4.3	0	2.8
6	0	2.3	0	1.4

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Table 9-3 Temporal patterns for rainfall durations 9 hours and 12 hours

Time (hr)	1% AEP rainfall (mm)		10% AEP rainfall (mm)	
	9	12	9	12
0	0	0	0	0
0.5	4.6	6.2	2.9	3.9
1	10.3	7.7	6.5	4.8
1.5	16.9	10.7	10.7	6.7
2	5.5	9.4	3.5	5.9
2.5	6.4	10.2	4.1	6.4
3	9.4	8.2	5.9	5.1
3.5	8.7	4.7	5.5	2.9
4	22.0	6.7	13.9	4.2
4.5	41.0	10.9	26.0	6.8
5	24.0	13.4	15.2	8.4
5.5	14.7	16.4	9.3	10.2
6	9.4	34.2	5.9	21.4
6.5	7.6	25.0	4.8	15.7
7	8.7	12.9	5.5	8.1
7.5	12.4	12.2	7.8	7.6
8	11.2	4.5	7.1	2.8
8.5	10.8	5.2	6.8	3.3
9	5.5	8.2	3.5	5.1
9.5	0	3.7	0	2.3
10	0	11.7	0	7.3
10.5	0	6.4	0	4.0
11	0	8.7	0	5.4
11.5	0	7.7	0	4.8
12	0	3.2	0	2.0



Table 9-4 Temporal patterns for rainfall duration 24 hours

Time (hr)	1% AEP rainfall (mm)	10% AEP rainfall (mm)
0	0	0
1	10.2	6.4
2	18.2	11.4
3	10.5	6.6
4	3.4	2.1
5	8.0	5.0
6	13.6	8.5
7	6.2	3.9
8	6.8	4.3
9	12.1	7.6
10	12.1	7.6
11	25.6	16.1
12	44.5	27.9
13	20.7	13.0
14	19.5	12.2
15	15.1	9.5
16	17.9	11.3
17	10.2	6.4
18	4.6	2.9
19	7.4	4.7
20	8.3	5.2
21	11.1	7.0
22	6.2	3.9
23	12.1	7.6
24	4.6	2.9

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