Bellvista area (based on BMT WBM, 2011b), which was dynamically linked to the 12m grid domain using a 2D domain linking technique. Each square grid element contains information on ground topography (sampled from a DEM), surface resistance to flow (Manning's 'n' roughness coefficient) and initial water level.

The ground topography of the TUFLOW model was sampled from the DEM. This DEM has a 5m grid resolution and is based on an Airborne Laser Scanning (ALS) survey and cross section surveys of the channels of Bells Creek, Lamerough Canals and the canals of Pelican Waters. The topography of the Caloundra South area is presented in Figure 2-2.

Linear features that potentially influence the flow behaviour, such as natural gullies and creeks, levees and embankments, were incorporated into the model using 3D 'breaklines' to ensure that these were contained within the model grids and accurately represented in the model.

3.3.3 Hydraulic Structures

The Glasshouse Mountains Road and the Bruce Highway form significant obstructions to natural flow patterns and convey flood waters at numerous bridge and culvert crossings.

To accurately represent drainage through existing embankments, bridge and culvert crossings were incorporated as one-dimensional structures in the TUFLOW model and dynamically linked to the 2D model domain.

Surveys were conducted to determine the length, location and dimensions of the culvert crossings. These surveys were undertaken by RPS (formerly Conics Ltd). Invert levels and loss coefficients were based on information provided in the *Caloundra South Flood Study*.

Details on the hydraulic structures included in the TUFLOW model are provided in Appendix C. Their locations are shown in Figure 3-7.

The proposed road crossings in the master plan have been included in the model, and assumed to be flood immune. It is noted that not all crossings will be required to be flood immune; this approach was adopted as a conservative approach. Hydraulic structures through the crossings were included based on a preliminary design. These crossings will be assessed in more detail during the design stage of the assessment.

3.3.4 Hydraulic Roughness

Roughness coefficients represent the resistance to flood flows in channels and floodplains. In the hydraulic model, the adopted Manning's 'n' roughness coefficients were based on the *Caloundra South Flood Study*. The adopted Manning's 'n' roughness coefficients for the land uses within the proposed development are also listed in Table 3-5. The land use delineation of the Base Case model is based on aerial photography and presented in Figure 3-3.

Table 3-5 Adopted Manning's 'n' Roughness Coefficients in TUFLOW Model

Land Use	Manning's 'n' coefficients
Existing Land Uses	
Forested	0.120
Very high dense rural	0.100
Urban	0.090
Medium rural pasture	0.050
Permanently inundated areas	0.045
Sparse rural pasture	0.040
Proposed Land Uses	
Forested	0.120
Urban	0.100
Conservation and open space	0.080
Roads	0.020
Maintained parkland / playing fields	0.045

3.3.5 Boundary Conditions

The TUFLOW model includes various inflow boundaries and a downstream water level boundary. The boundary conditions used in the TUFLOW model are discussed below. The inflow boundaries applied in the TUFLOW model are shown in Figure 3-8.

3.3.5.1 Local Inflow Boundaries

RAFTS-XP local inflows from the subcatchments within the Bells Creek and Lamerough Creek catchments were specified as 'source-area' inflow boundaries in the TUFLOW model. These boundaries distribute inflows over the 2D domain, and are based on the sub catchments from the RAFTS-XP model.

3.3.5.2 Lamerough Creek Inflow Boundary

The hydraulic model does not include the upper catchment of Lamerough Creek. The upstream model extent on Lamerough Creek is in the vicinity of Bellvista. Therefore, it was necessary to derive flow hydrographs to simulate flow into Lamerough Creek at the upstream boundary.

A single 12m 2D domain version of the existing case hydraulic model that included the full extent of the Lamerough Creek catchment was run for all design runs. Flows at the location corresponding to the upstream boundary on Lamerough Creek in the multi-domain model were extracted from the single domain model results, and applied as upstream boundary conditions in the multi-domain model.

3.3.5.3 Duck Holes Creek Inflow Boundary

The inflows from the Duck Holes Creek catchment were applied as lateral inflow in the TUFLOW model. These inflows were based on the RAFTS-XP total inflows from this catchment.

3.3.5.4 Downstream Boundary Condition

At the downstream end of the TUFLOW model, a static water level boundary condition was specified. The water level specified represents the tide within Pumicestone Passage. The water levels used are discussed below.

Downstream Water Levels during Freshwater Flood Events

Based on SKM (2006), storm tide events and freshwater flood events have been considered to be statistically independent and it has been assumed that there is a minimal correlation between storm tide events and freshwater flood events. Consequently, for the modelling of freshwater flood events, ocean levels are not assumed to be affected by storm surge. For the modelling of freshwater flood events, a constant water level of 0.45mAHD was used as a downstream boundary condition. This water level represents a mean high water spring level at Golden Beach (from the Tide Tables and Boating Safety Guide (MSQ, 2007)).

Storm Tide Levels

The flood risk at the low lying land adjacent to Pumicestone Passage is dominated by inundation due to storm surge events. To assess the flood risk of these parts of the site, design storm levels at the proposed development site have been assessed. As discussed above, storm tide events have been assumed to be statistically independent of catchment flooding.

For this study, the storm tide levels from the *Caloundra City Storm Tide Study* (Connell Wagner, 2003) were adopted. The adopted design storm tide levels (without allowances for climate change impacts) are listed in Table 3-6. It is noted that the storm tide levels from the Caloundra City Storm Tide Study differ significantly from design storm tide levels reported in Moreton Bay Regional Council Storm Tide Hazard Study (Cardno, 2008) for a nearby location. Therefore, it is recommended to review the adopted storm tide levels.

Table 3-6 Adopted Present-Day Design Storm Tide Levels (Reference Year 2009)

Average Recurrence Interval (years)	Peak Storm Tide Levels
5	1.73mAHD
50	1.93mAHD
100	1.95mAHD

3.3.6 Model Verification

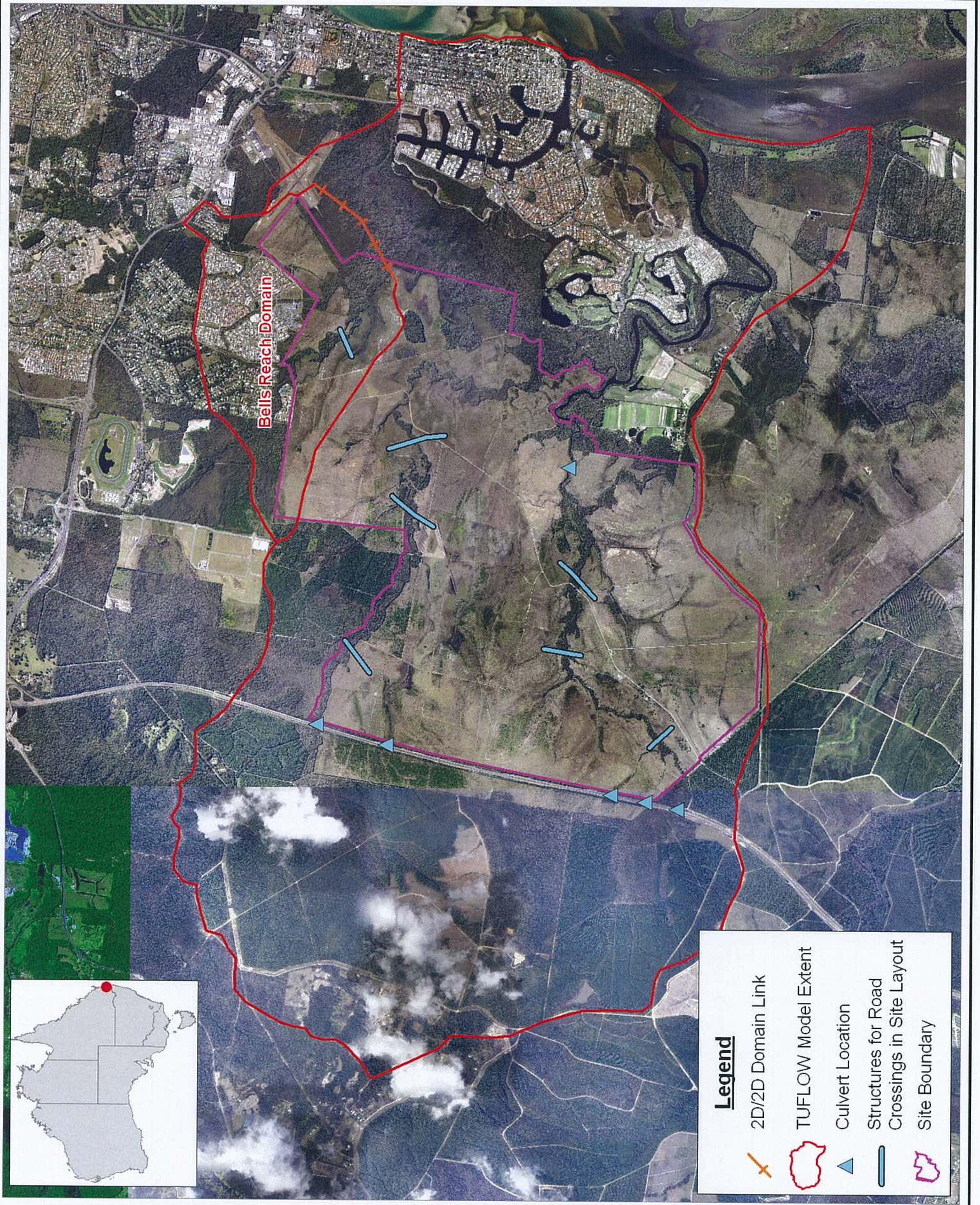
As part of the previous BMT WBM flood assessment of the Caloundra South development, the TUFLOW model was simulated using the same design flood events and assumptions as modelled in the *Caloundra South Flood Study*. The results of the Caloundra South TUFLOW model were compared with predictions of the *Caloundra South Flood Study* MIKE-FLOOD model.

Overall, the results showed a relatively high level of consistency, with differences in modelled peak flood levels on-site generally within 0.1m for all design events modelled. This is considered to be a good comparison. However, there were some areas where model results deviated.

The differences in flood extent were likely to be related to the different approach adopted for applying the RAFTS-XP subcatchment inflows. In the MIKE-FLOOD model, inflow boundaries were applied as source points at the centre point of the subcatchment, whilst in the TULFOW model the inflows were distributed over wetted areas of the subcatchment. The adopted approach in the TUFLOW model is considered to be more appropriate.

Differences in predicted peak flood levels upstream of the Bruce Highway were likely to be related to differences in culvert structures in the TUFLOW and MIKE-FLOOD model. (Culverts in the TUFLOW model were based on structure survey.) In addition, the RAFTS-XP subcatchment at the highway, BCS5, was divided into two subcatchments in the refined BMT WBM RAFTS-XP model.

The *Caloundra South Flood Study* MIKE-FLOOD model was not calibrated to historical flood events due to the absence of reliable data. Consequently, this analysis simply demonstrates that the previous TUFLOW model was predicting similar results to the uncalibrated MIKE-FLOOD model. The current TUFLOW model that has been used in this study is a refined and updated version of the previous TUFLOW model, with changes to the underlying land use classification within the catchment. Therefore, it would be inappropriate to compare the results of the updated TUFLOW model to the MIKE-FLOOD model.



Title:
Model Extent and Hydraulic Structures in TUFLOW Model

Figure: 3-7

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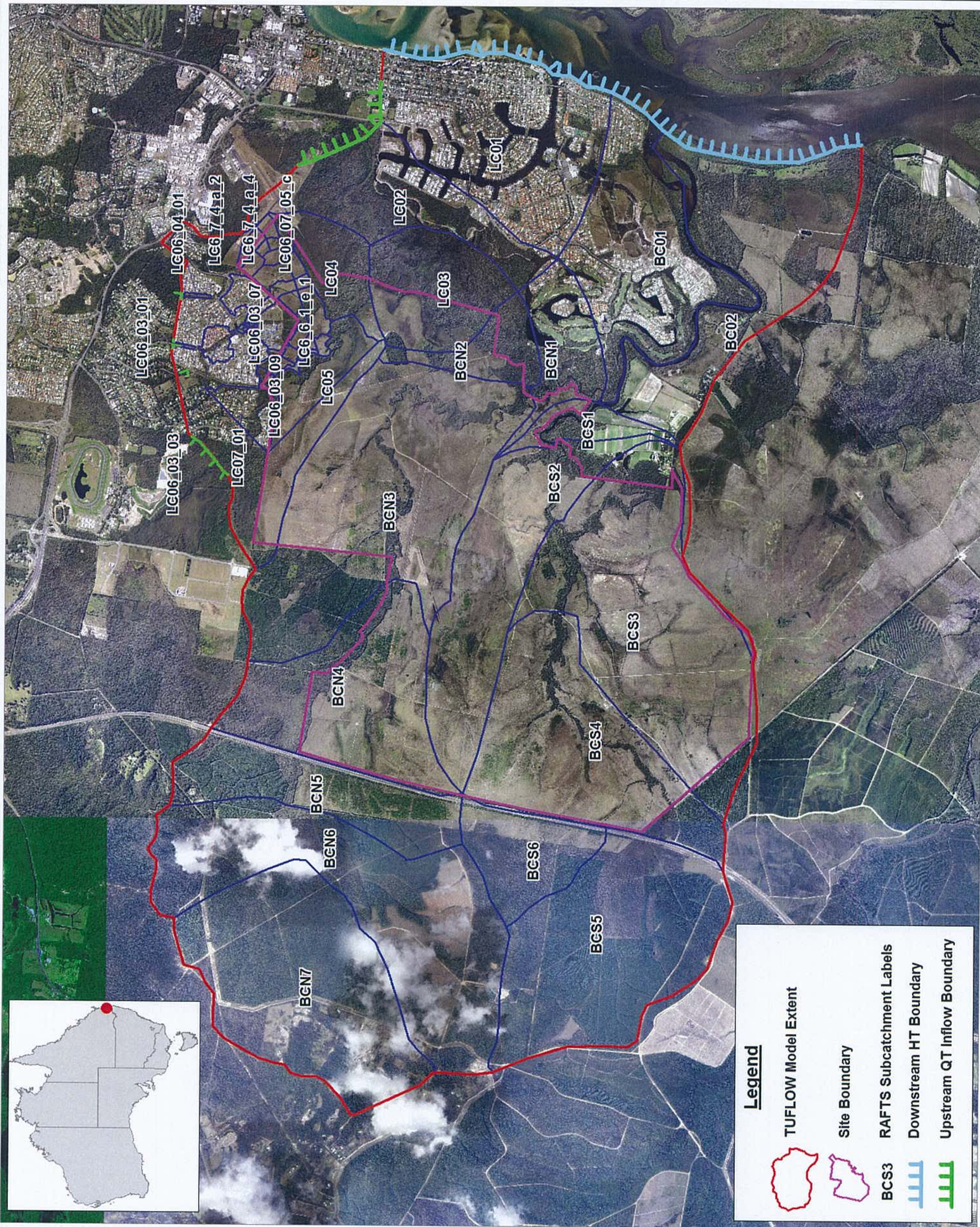

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Boundary Conditions for the TUFLOW Model

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3.4 Flooding Assessment

3.4.1 Composition of the Design Flood Event

The design flood results are comprised of three separate run scenarios:

1 6 hour storm duration rainfall event with a mean high water spring tide level at the downstream boundary;

2 12 hour storm duration rainfall event with a mean high water spring tide level at the downstream boundary; and

3 Ocean storm surge flooding with no rainfall event.

These events form the worst case flood conditions across the catchment, and the results presented in this report are an amalgamation of these three scenarios.

3.4.2 Description of Base Case Conditions

For the purposes of this flood assessment, it was assumed that the existing catchment conditions are as per the aerial photography from approximately May 2011. This is significantly different to the SKM study, for which the now cleared site was mainly forested. The Bells Reach development was not included in the base case.

3.4.3 Results of Base Case Scenarios

The flood model for the Base Case (i.e. pre-development situation) was simulated for the 5, 50 and 100 year ARI design flood events. The resulting peak flood levels and extents of flooding are presented in Figure 3-10 to Figure 3-12 and peak flow velocities in Figure 3-13 to Figure 3-15. The peak flood depths for the combined 5, 50 and 100 year ARI events are presented in Figure 3-16 to Figure 3-18.

Regarding the existing flooding behaviour of the site and surroundings, the following is noted:

- The general flooding pattern for larger flood events is characterised by diffuse overland flows with relatively wide and shallow inundated areas. The inundated areas are generally centred around Bells Creek North, Bells Creek South and Lamerough Creek.
- The area bounded by the proposed Caloundra South development footprint, Pelican Waters and the Aerodrome is characterised by extensive flooding that stores large volumes of flood water. Flood depths in this area are generally 1m to 2m, and reach up to approximately 4m during the 100 year ARI event. Flood water is mostly very slow flowing in this area with flow velocities generally less than about 0.1m/s during most events.
- The maximum inundation depth for the existing site within the development footprint is generally low and less than 0.75m at most locations during the 100 year ARI event. An exception is the area located directly west of the proposed town centre, where there is an overland flow path during major events.

3.4.3.1 Modelling of the Developed Case

The base case model was modified to represent the features of the proposed development in a 'developed case' configuration. Numerous iterations were required to derive a scheme that had minimal adverse impacts on the flood risk outside the Caloundra South site boundaries. The general principles in developing the flood risk management strategies were as follows:

- A minimum 100 year ARI flood immunity for all urban areas of the development; and
- Prevent adverse impacts on flood risk of properties outside the Caloundra South site boundary.

Without mitigation measures, the proposed development has the potential to affect flooding in the following ways:

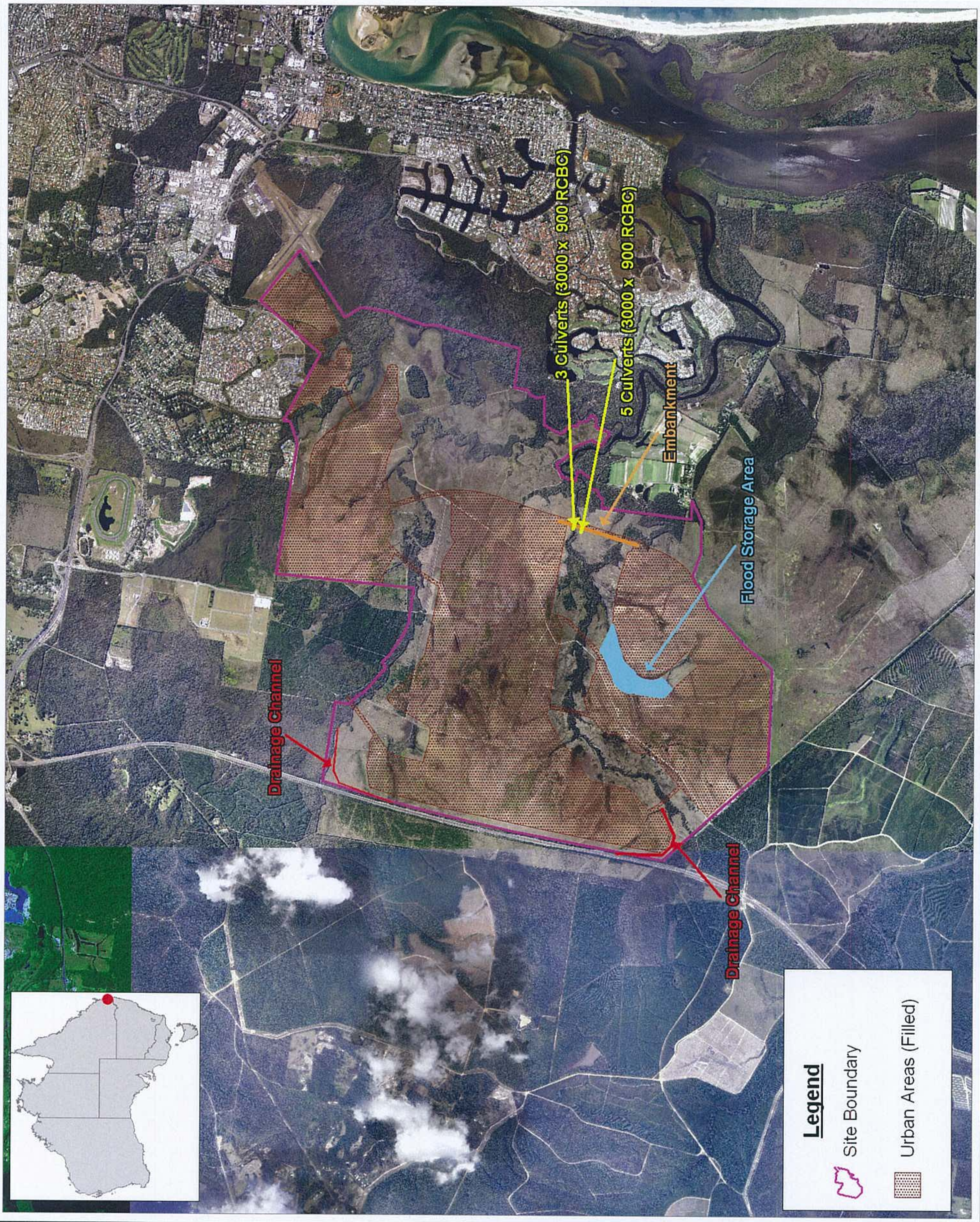
- The urban areas of the proposed development have the tendency to cause a quicker and more intense flood water runoff response during storm events. This generally leads to earlier and greater peak creek inflows.
- In areas where fill is proposed to provide relevant flood immunity, existing flow paths could be obstructed. This may impact on the hydraulic conveyance of the creek systems. In addition, fill may reduce the floodwater storage capacity of a floodplain. A reduction of floodwater storage capacity could result in increased flooding in areas downstream of the development.

In order to mitigate flood impacts of the Caloundra South development, flood mitigation works will be required. The key features of the developed case are presented in Figure 3-9 and include the following general elements:

- All urban areas (residential, industrial/commercial, including internal roads) have been filled above the 100 year ARI design flood level;
- Manning's 'n' roughness coefficients have been applied in accordance with the proposed land use changes;
- A road embankment across the Bells Creek South floodplain with a crest level above the 100 year ARI design flood level and two sets of box culverts through the embankment. One set of culverts consisting of three 3000 x 900 RCBCs is located within the creek channel and one set of culverts consisting of five 3000 x 900 RCBCs is located on the floodplain;
- An area of 26.3ha on the Bells Creek South floodplain was lowered to provide additional floodwater storage capacity. Proposed ground elevations in this detention area vary between 2mAHD and 3mAHD (an average lowering of about 1.7m);
- An open drainage channel along the eastern edge of the Bruce Highway between Bruce Highway culvert crossing BH36_037 and Bells Creek North, draining to the north; and
- An open drainage channel along the eastern edge of the Bruce Highway between Bruce Highway culvert crossing BH_BCS6 and Bells Creek South, draining to the south.

Results of Developed CaseThe results of the developed case flood modelling are illustrated in Figure 3-19 to Figure 3-30. Flood risk within the proposed development footprint has been mitigated by filling on the floodplain. A key feature is the detention basin on Bells Creek South. The detention storage area is inundated to approximate flood depths of 1m and 2m for the 5 year ARI and 100 year

ARI flood events respectively in the vicinity of the embankment. A discussion on the differences in flood levels between the base and developed cases is contained below.



Title:

Key Features of the Proposed Development Scheme

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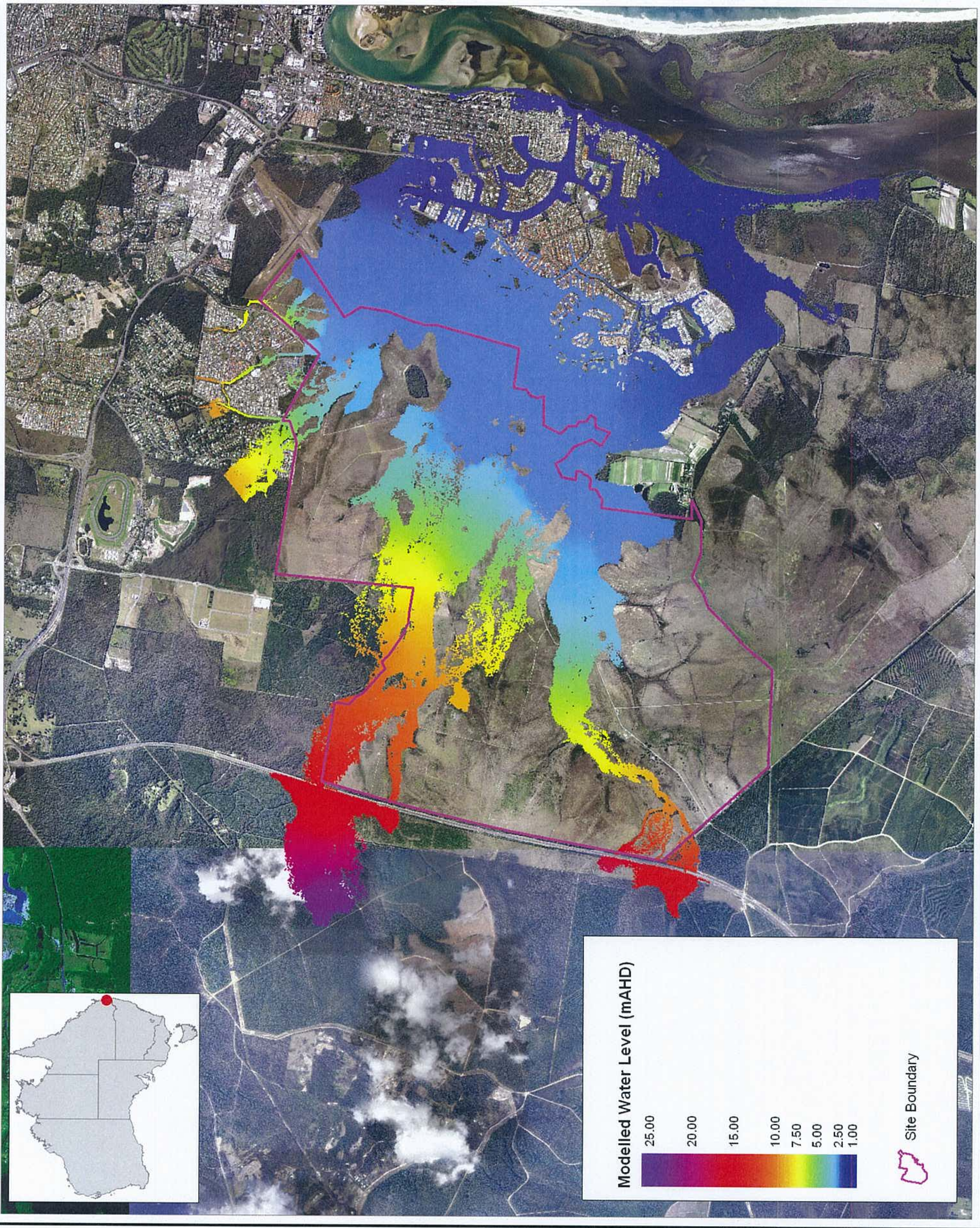

Title:

Q100 Peak Flood Levels For Base Case

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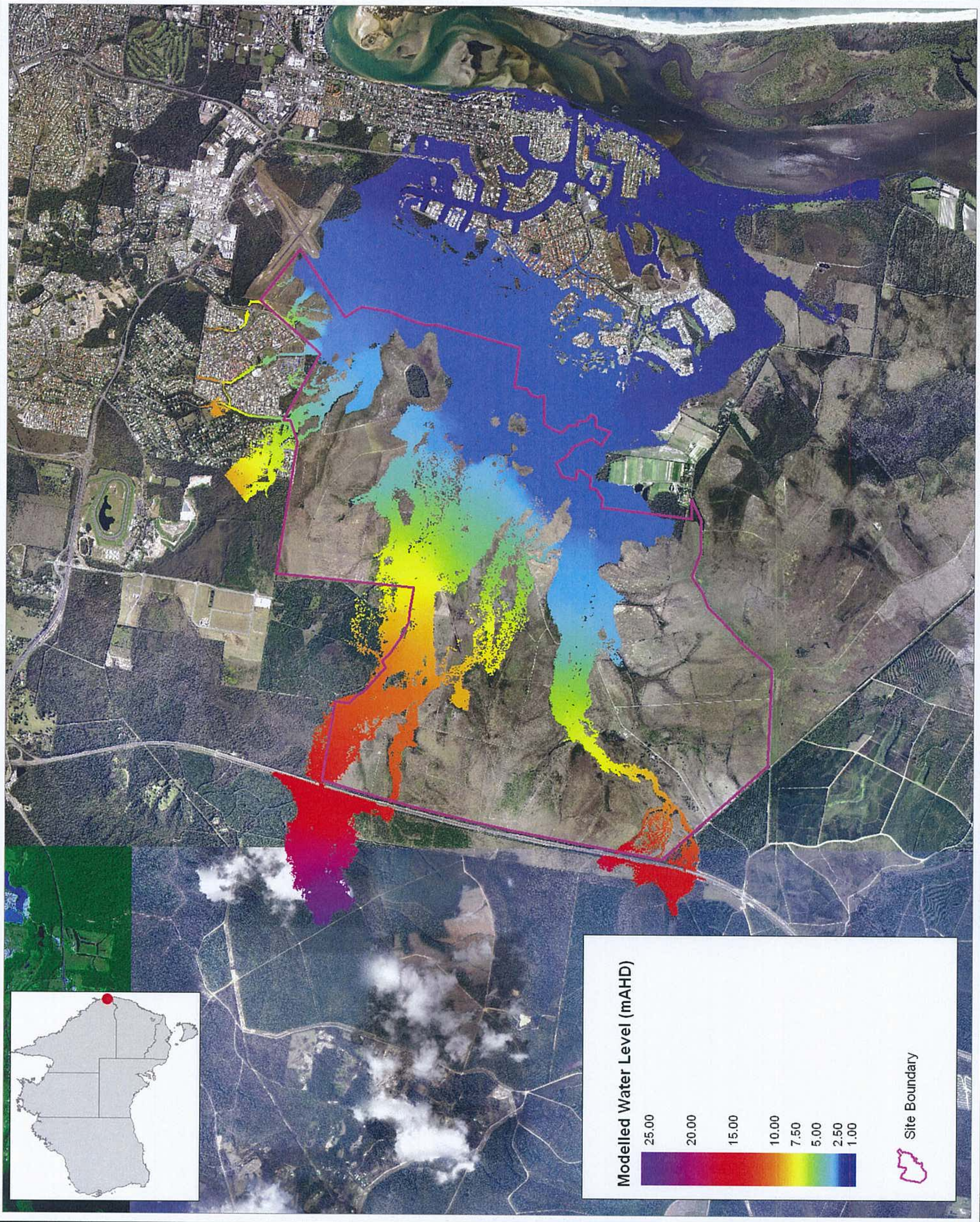

Title:

Q050 Peak Flood Levels For Base Case

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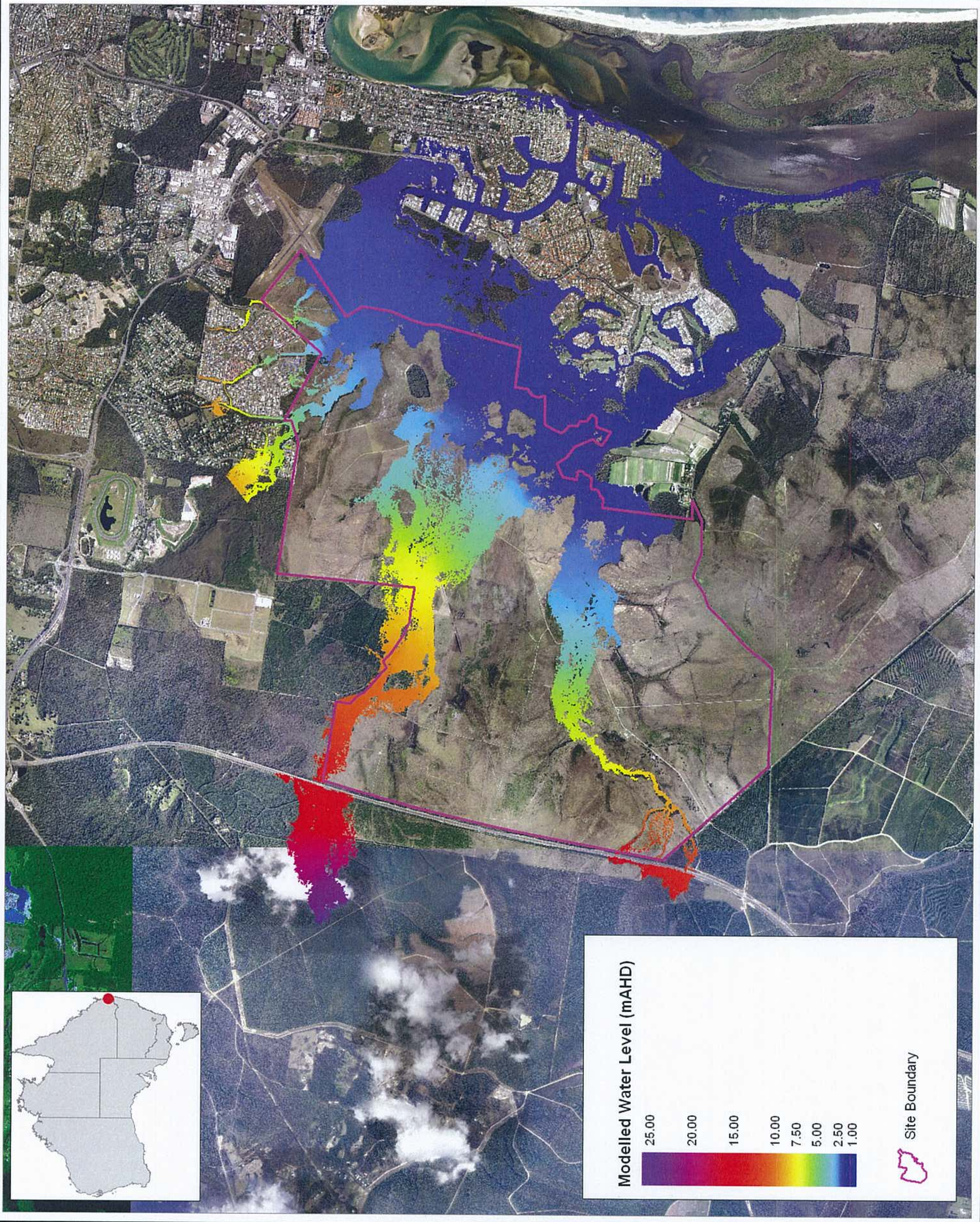

Title:

Q005 Peak Flood Levels For Base Case

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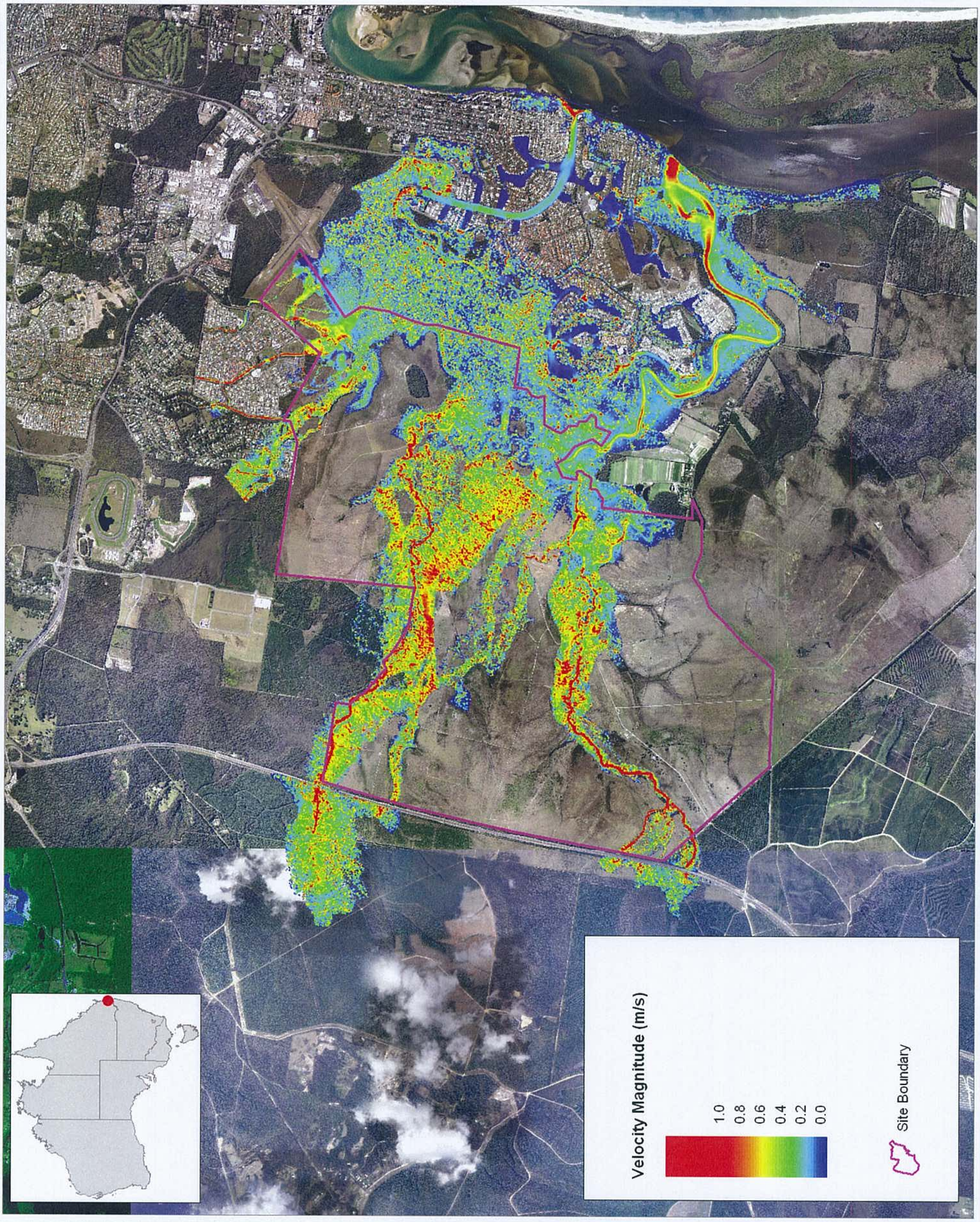

Title:

Q100 Peak Flow Velocities for Base Case

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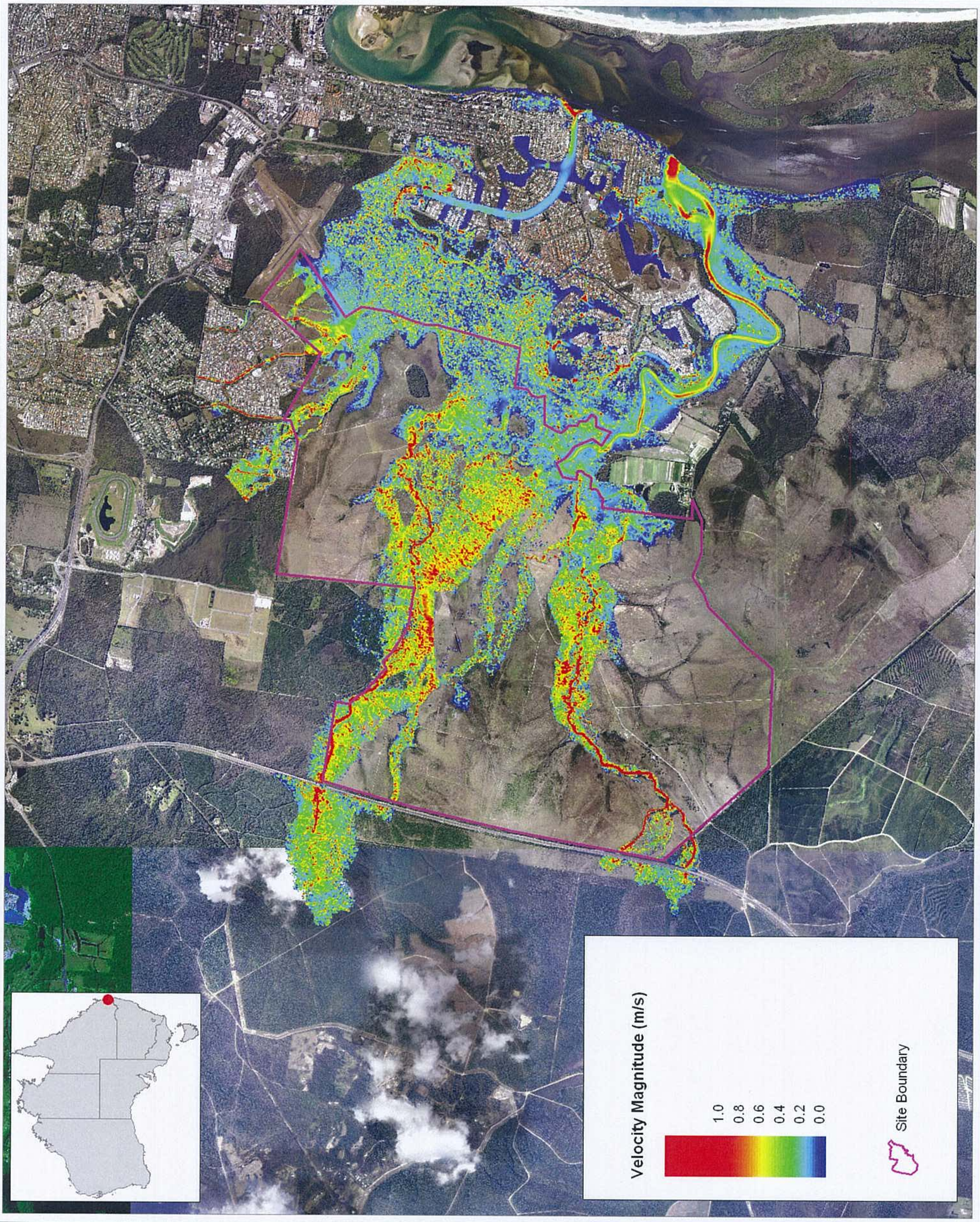

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Q050 Peak Flow Velocities for Base Case

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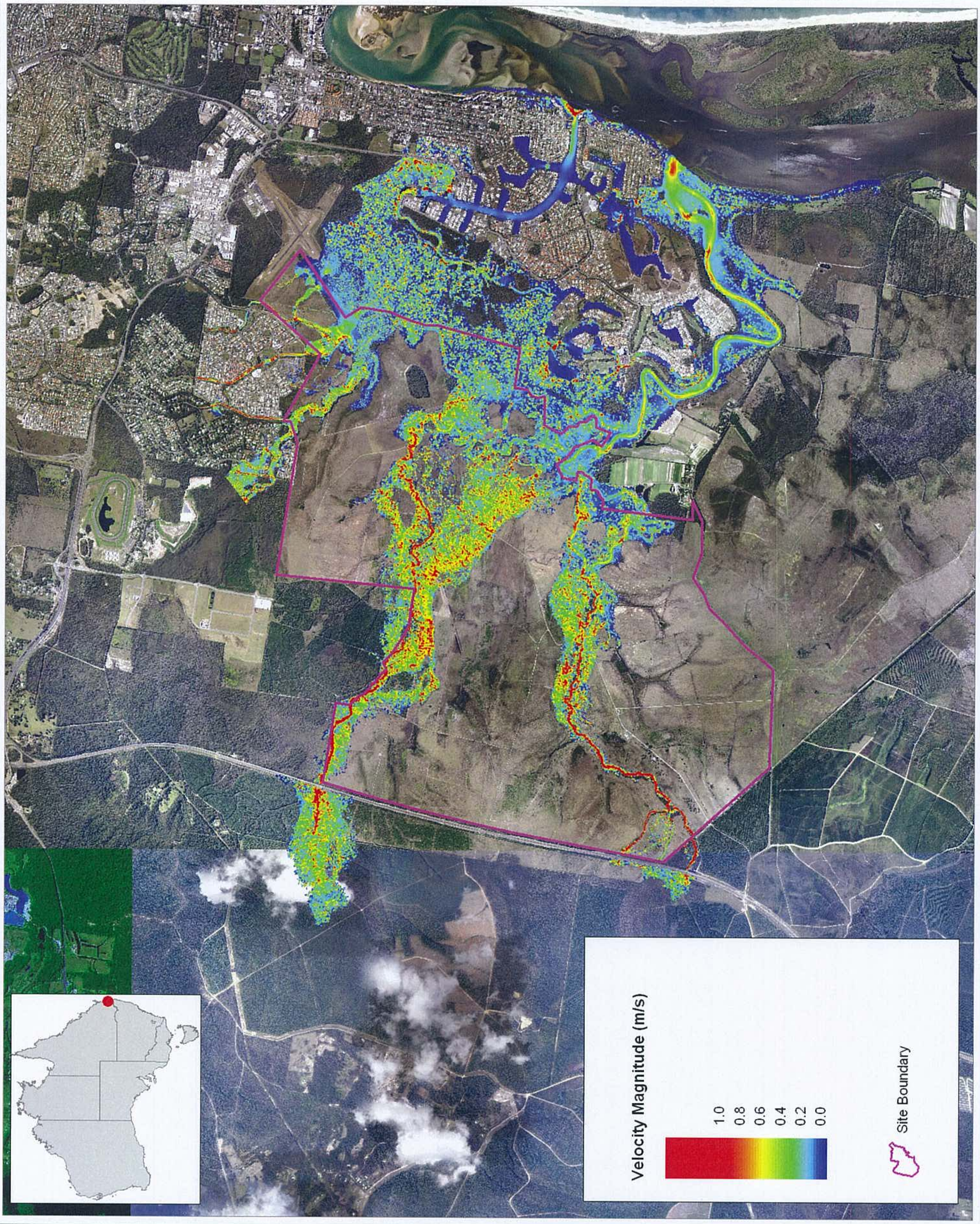

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Q005 Peak Flow Velocities for Base Case

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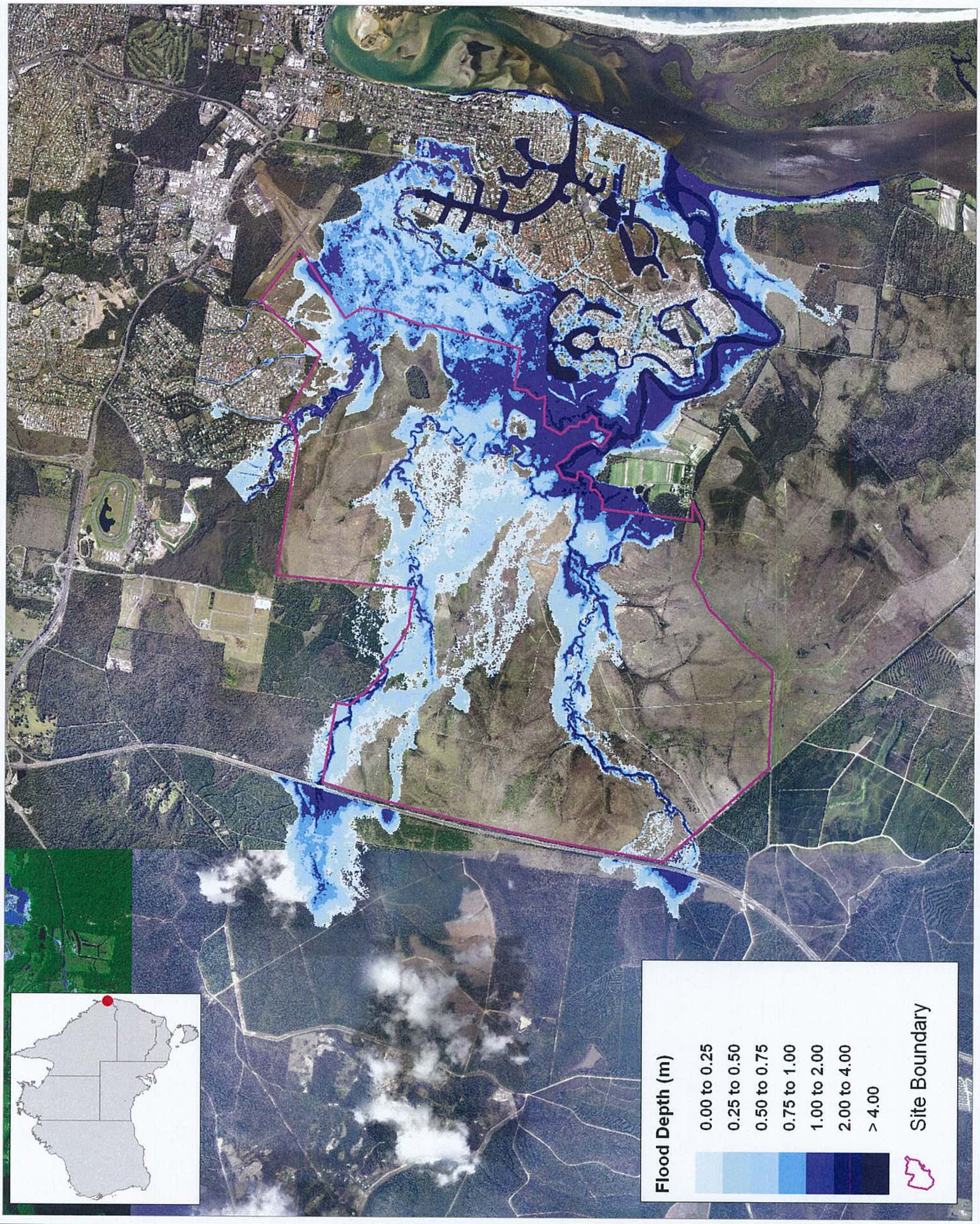

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Q100 Peak Flood Depth For Base Case

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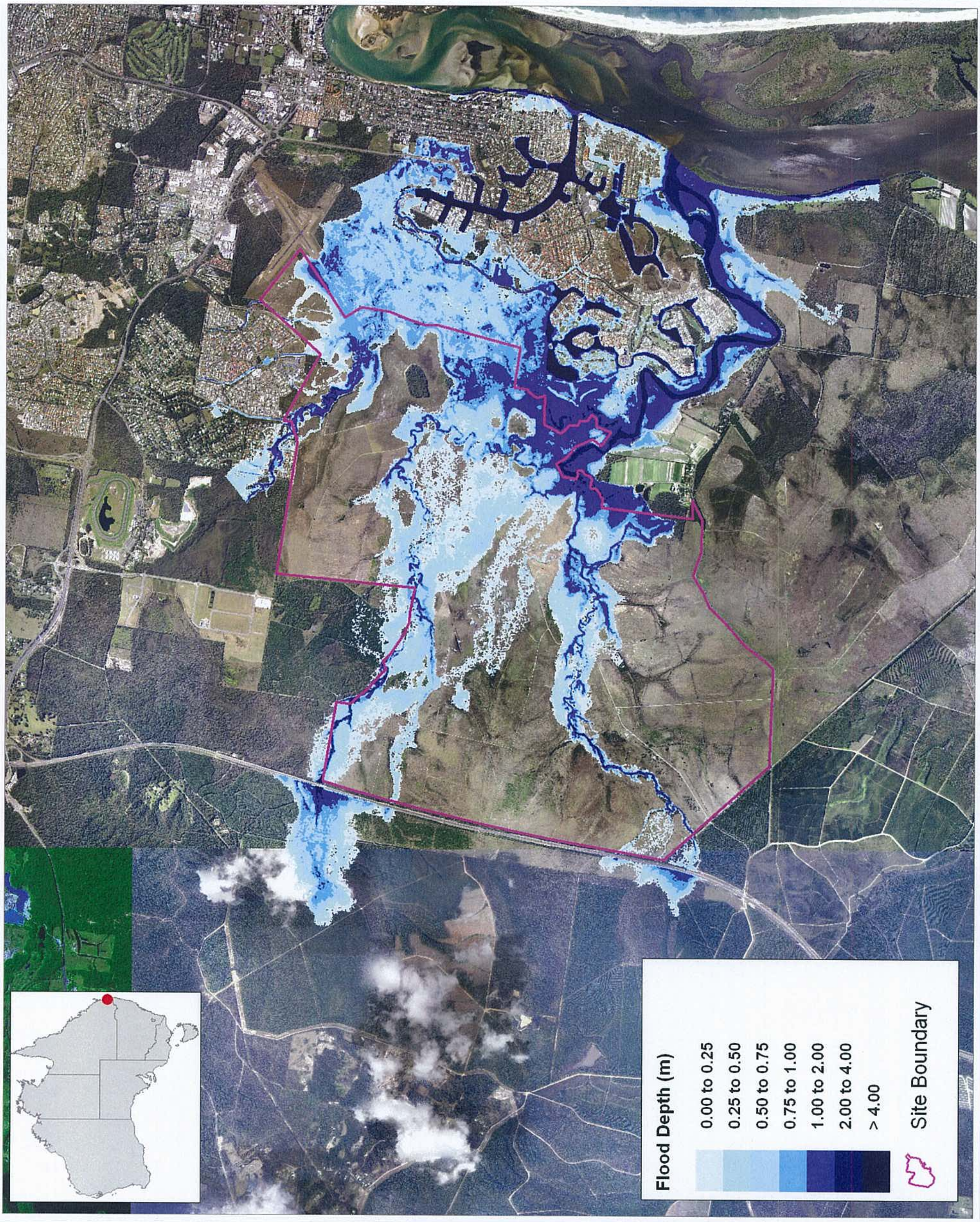

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Q050 Peak Flood Depth For Base Case

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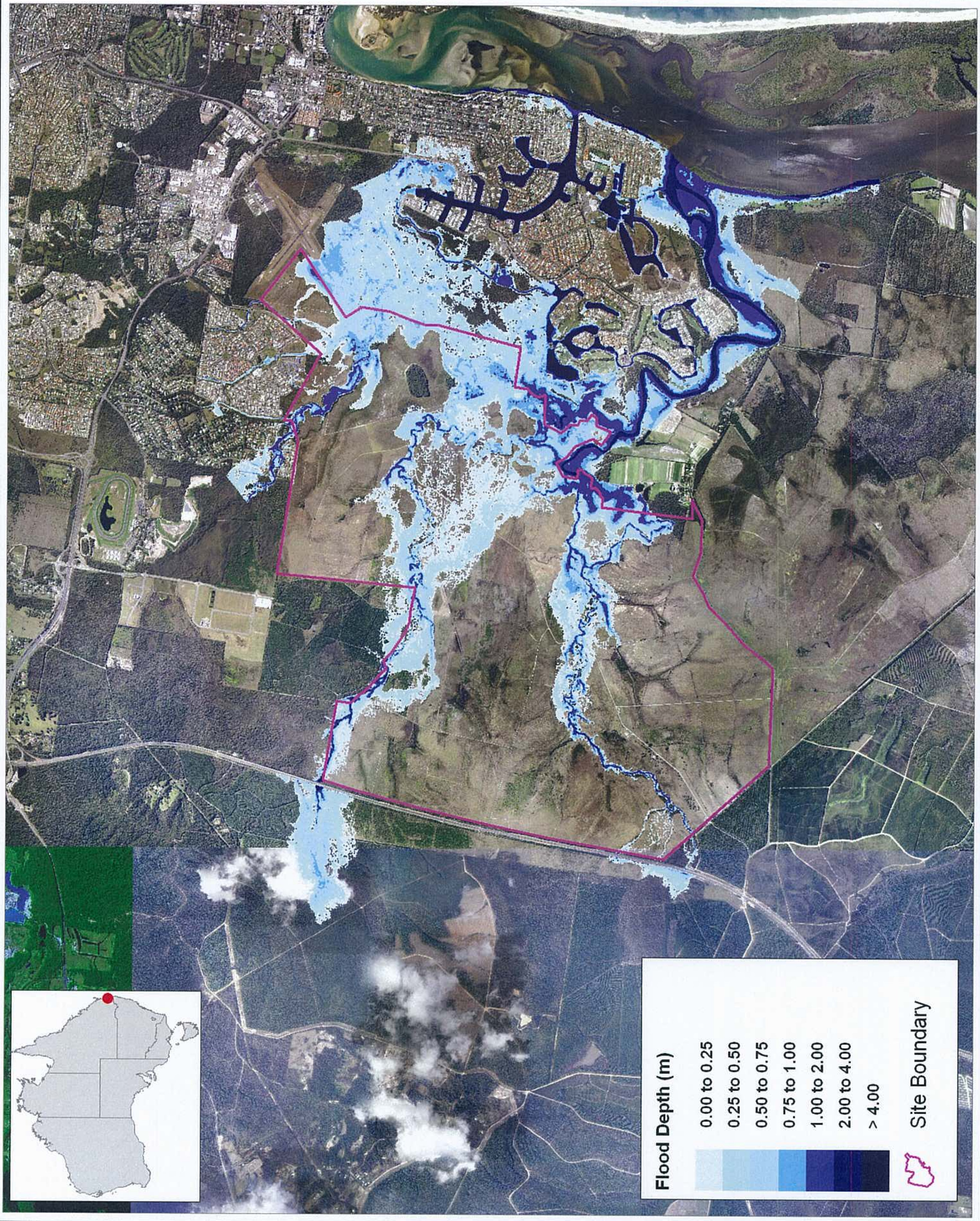

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Q005 Peak Flood Depth For Base Case

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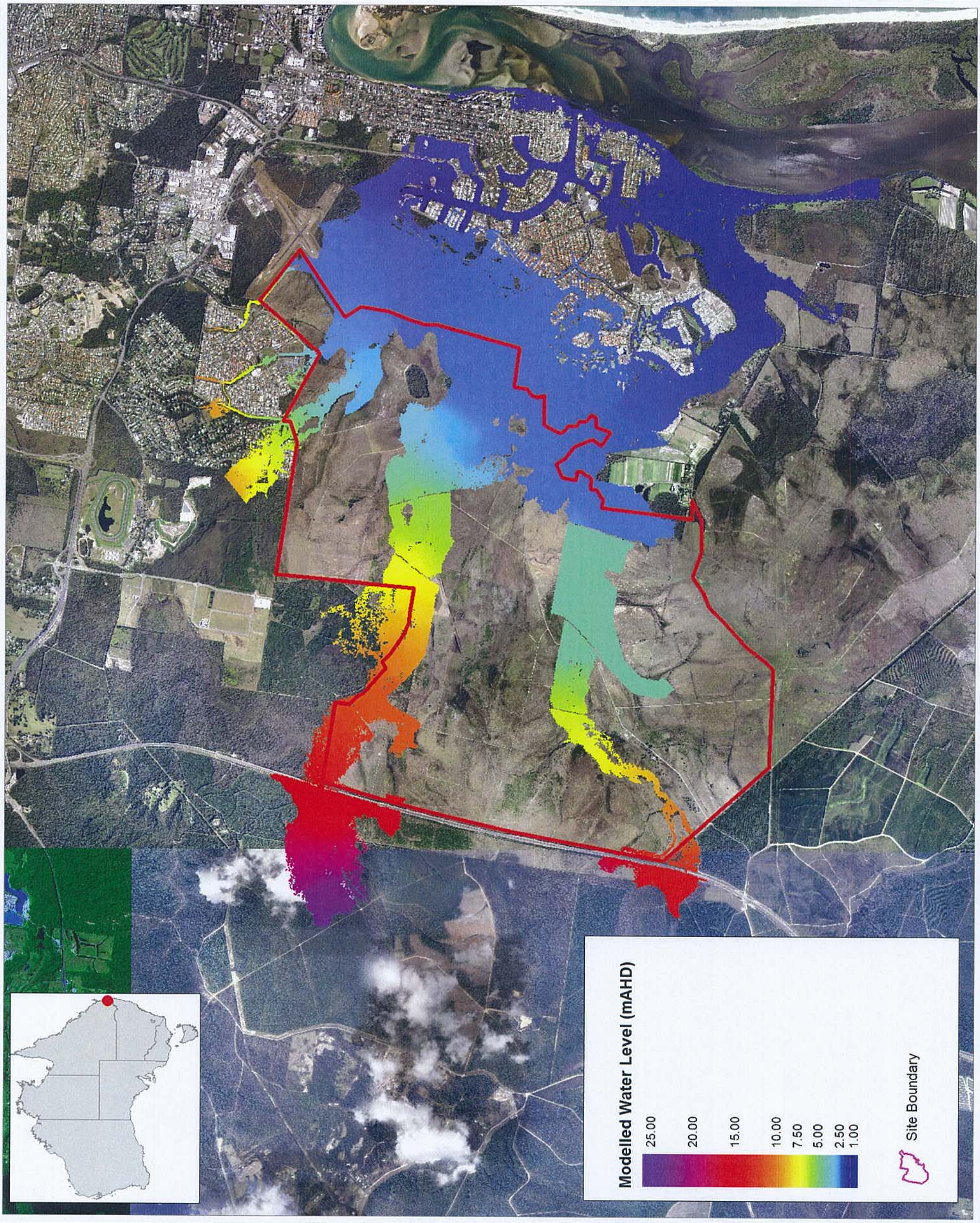

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Q100 Peak Flood Levels For Developed Case

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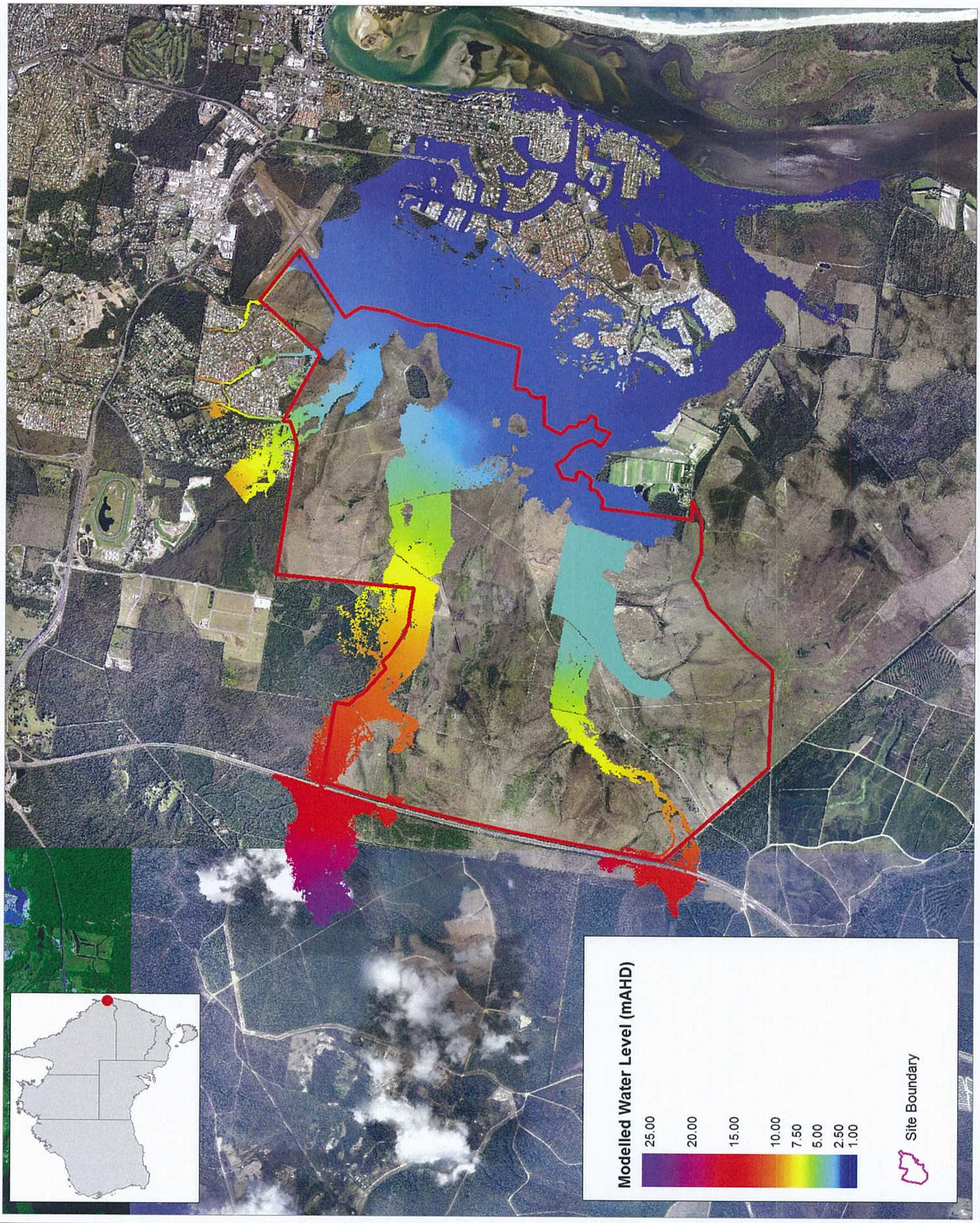

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Q050 Peak Flood Levels For Developed Case

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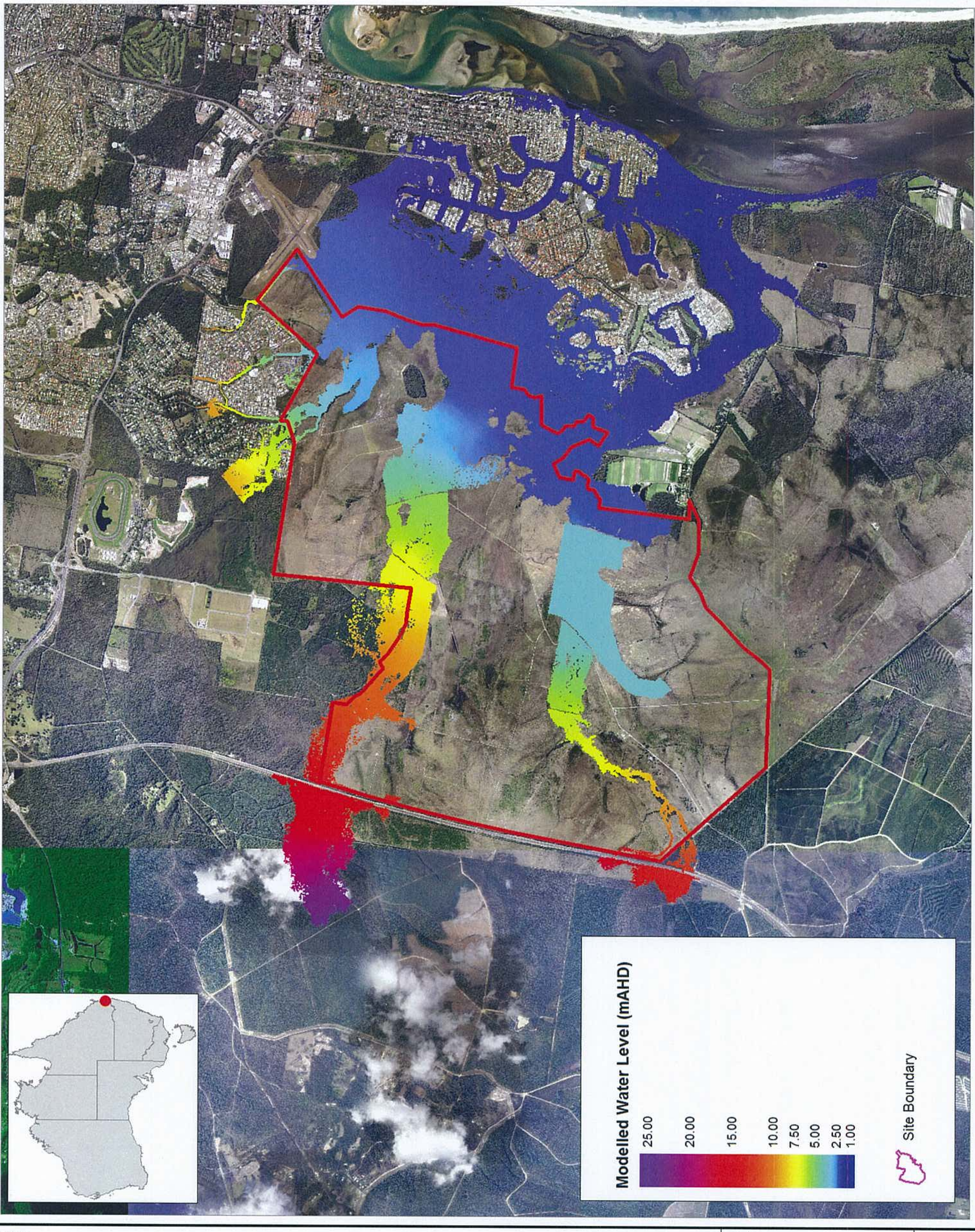

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Q020 Peak Flood Levels For Developed Case

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