NORTH TUNCURRY DEVELOPMENT PROJECT – MODELLING AND DESIGN OF FLOOD ALLEVIATION SCHEME FOR GROUNDWATER FLOODING

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1. Overview

The Proposed North Tuncurry Development Project overlies an unconfined sand dune aquifer. Evidence indicates that the site is subject to periodic groundwater flooding.

SMEC were engaged by UrbanGrowth NSW (Landcom) to conduct groundwater flood assessments, and to design groundwater flood alleviation measures for the site.

2. Background

The site lies immediately to the north of Tuncurry, about 3 km north of Forster on the NSW Mid North Coast – refer **Figure 1** for site location. The site lies within the Great Lakes Council local government area.

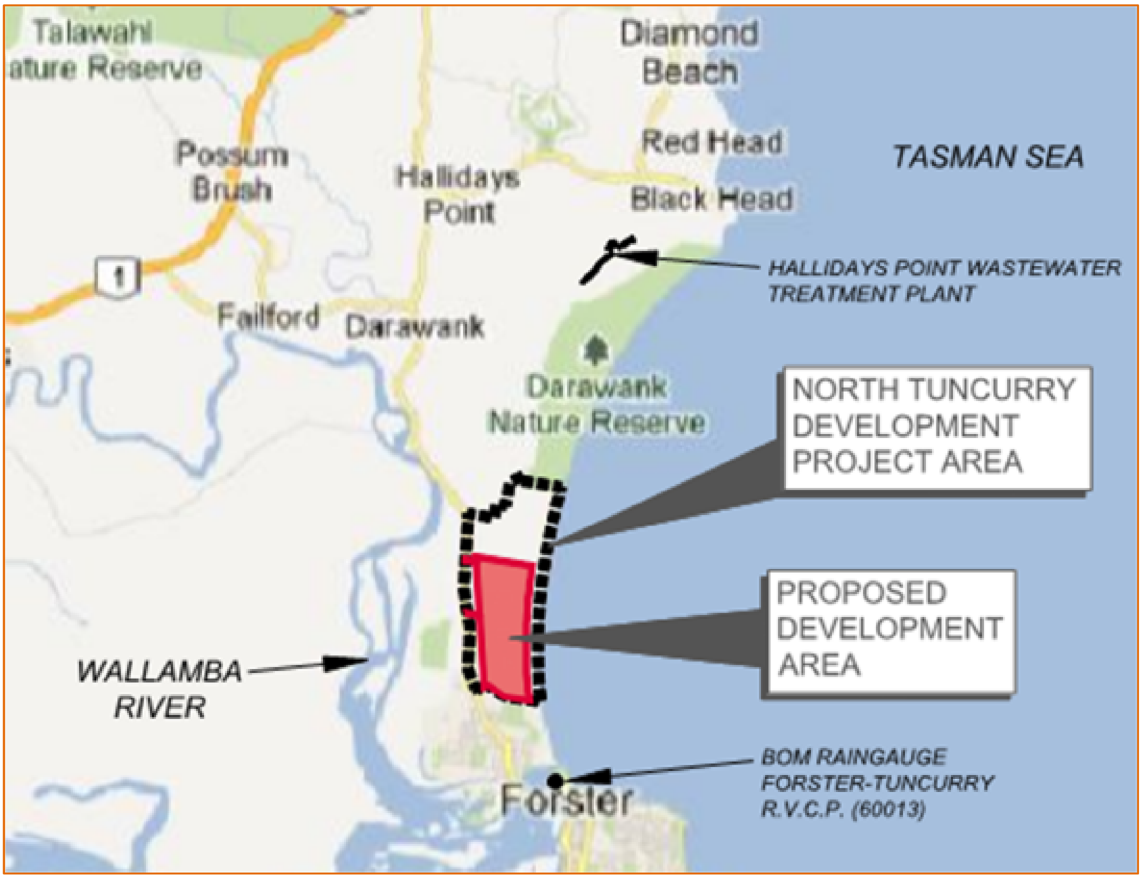

Figure 1 – Site Locality (Google Maps)

The site is located over an unconfined coastal aquifer. The topography is characterised by undulating aeolian sand dune systems. Infiltration rates are high and no distinct surface drainage paths have developed among the dunes which are shaped by the wind rather than water. Accordingly, all rainfall that falls over the site is either lost to evapotranspiration processes or drains vertically through the upper soil layer into the aquifer through a process referred to as recharge. Water leaves the aquifer though both evapotranspiration processes and lateral groundwater flow to the east (to the Pacific Ocean) and the west (the Wallamba River). The dynamics of these processes vary depending on the groundwater flow characteristics, prevailing rainfall and evaporation rates. **Figure 2** shows a photo of typical site topography.

Figure 2 – Typical Site Topography (SMEC)

The history and frequency of groundwater flood inundation on the site has not been well documented, as the site has largely been unoccupied. However, anecdotal information suggests that the site has been subject to significant historical flooding, in particular a large flood that occurred in the early 1960s (exact date unclear) that was recalled by some local residents and (the then) DLWC officers who were undertaking periodic inspections of the site. More recent flooding occurred in March 2013, but this was of a more minor nature. **Figure 3** shows a photo of typical site topography.

A method of assessing the groundwater flooding at the subject site was developed by SMEC.

Figure 3 – Photo showing recent groundwater inundation at the subject site

3. Groundwater Flood Study Approach

Groundwater flooding within the Project Area is a somewhat unique problem in that there is no natural surface drainage from the site (i.e. no creek or waterways). During periods of high rainfall, groundwater levels rise as a result of recharge. **Figure 4** outlines the key rainfall / recharge processes. The rate of rise and the ultimate peak groundwater level from a given rainfall event is governed by numerous factors, which include:

- Antecedent conditions such as soil moisture content and groundwater level at the beginning of the rainfall event.
- Rainfall durations and intensities.
- Recharge characteristics which govern the portion of rainfall that recharges into the underlying groundwater system.
- Hydraulic characteristics of the aquifer which govern the rate at which groundwater flows from the Project Area into either the Pacific Ocean to the east or the Wallamba River to the west.
- Evapotranspiration loss rates from the aquifer.
- Aquifer storage characteristics and surface storage characteristics which govern the rate of rise of the groundwater table.

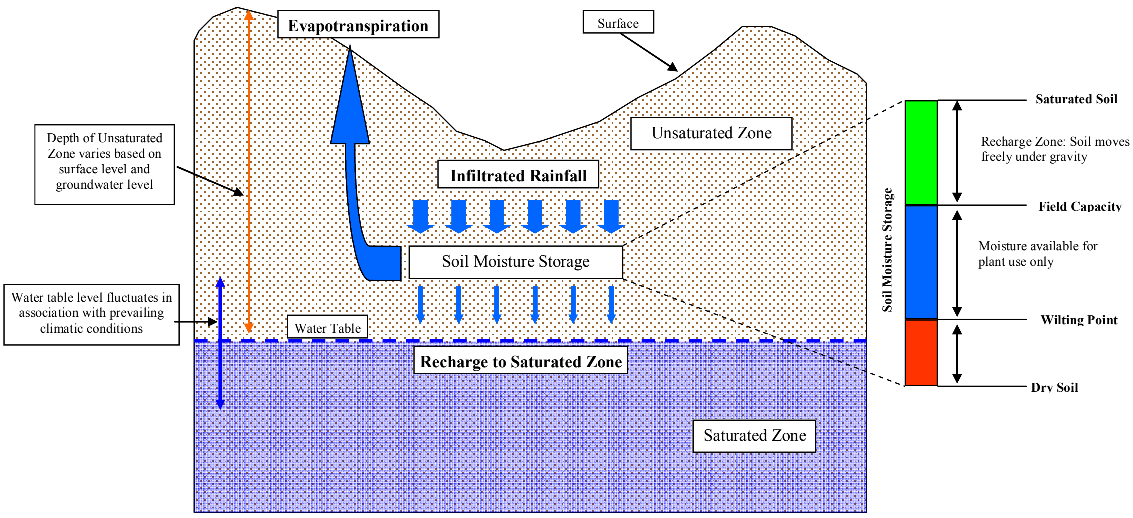

Figure 4 – Recharge Processes

A comprehensive assessment considering the abovementioned factors was undertaken. The assessment required the development of the following three separate models (developed for both existing and proposed development conditions):

1. A recharge model that was developed to estimate the site recharge characteristics (i.e. the portion of rainfall that recharges into the groundwater system) for a wide range of rainfall events.
2. An Empirical Groundwater Model that was developed (utilising the recharge model) to assess the likely groundwater conditions within the Project Area for a wide range of historic events – refer **Figure 5** for conceptual recharge model schematic. This model was used to:

   - Estimate the groundwater levels within the Project Area using a long-term climatic record.
   - Identify rainfall durations and intensities that are likely to result in groundwater flooding within the Project Area.
   - Identify a historic rainfall event that is likely to have produced the highest groundwater levels within the Project Area during the rainfall record, in order to define a design flood planning level for the development.

3. A detailed three–dimensional groundwater model was developed using the Visual MODFLOW SURFACT software package. This model was applied to estimate the peak groundwater levels within the Project Area for the design event that was identified using the Empirical Groundwater Model.

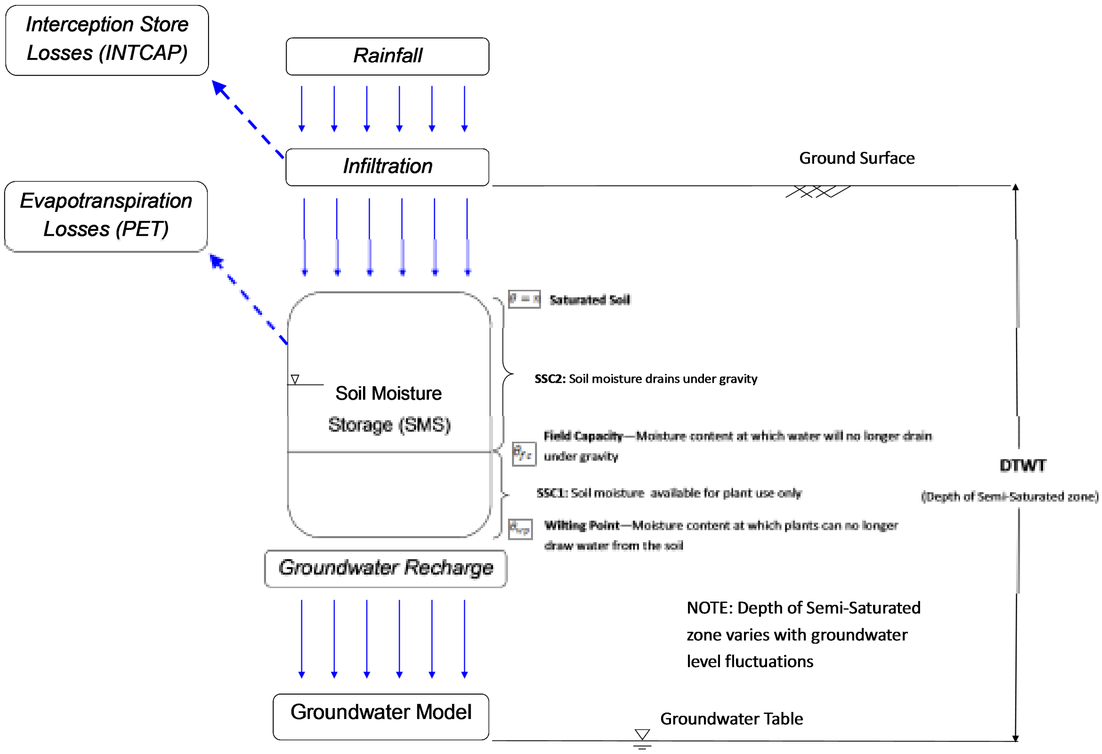

Figure 5 – Model 1 – SMEC Recharge Model

The Empirical Groundwater Model was established to estimate representative groundwater conditions within the development area over a 112 year simulation period (1900 to 2012). The model conceptualises the groundwater dynamics by applying a three dimensional "box" around the 260ha development area – refer **Figure 6**. For each model time step, the change in water volume stored within the "box" is calculated based on inflows (due to recharge) and outflows (due to evapotranspiration and lateral groundwater flows). A representative groundwater level is calculated based on the calculated volume of storage in the "box" and the groundwater / surface storage characteristics for the given groundwater level.

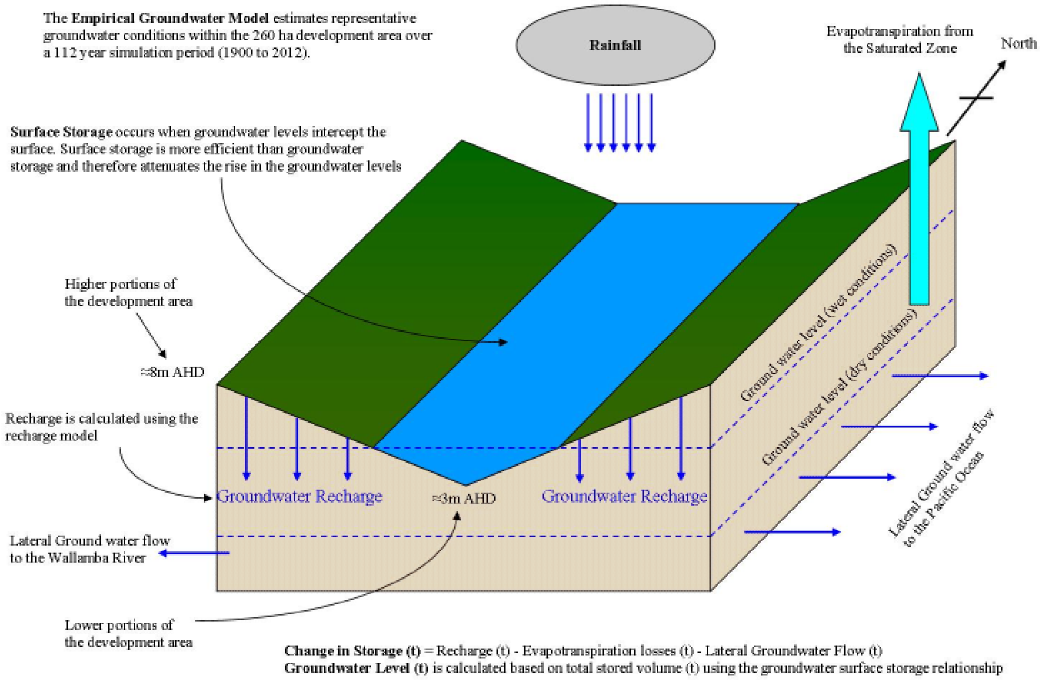

Figure 6 – SMEC Model 2 - Empirical Model to determine flooding characteristics for long rainfall record (1900 to 2012)

The results of this Empirical Groundwater modelling are outlined below in **Figure 7**. This modelling indicated that the worst case groundwater flooding occurred in 1963, when over 1500mm of rainfall occurred over a 3 month period, and the modelling predicted a peak groundwater level of RL 5.0m, AHD would be reached on the site during the 1963 event. This is typically some 2 – 2.5m above the lower part of the subject site.

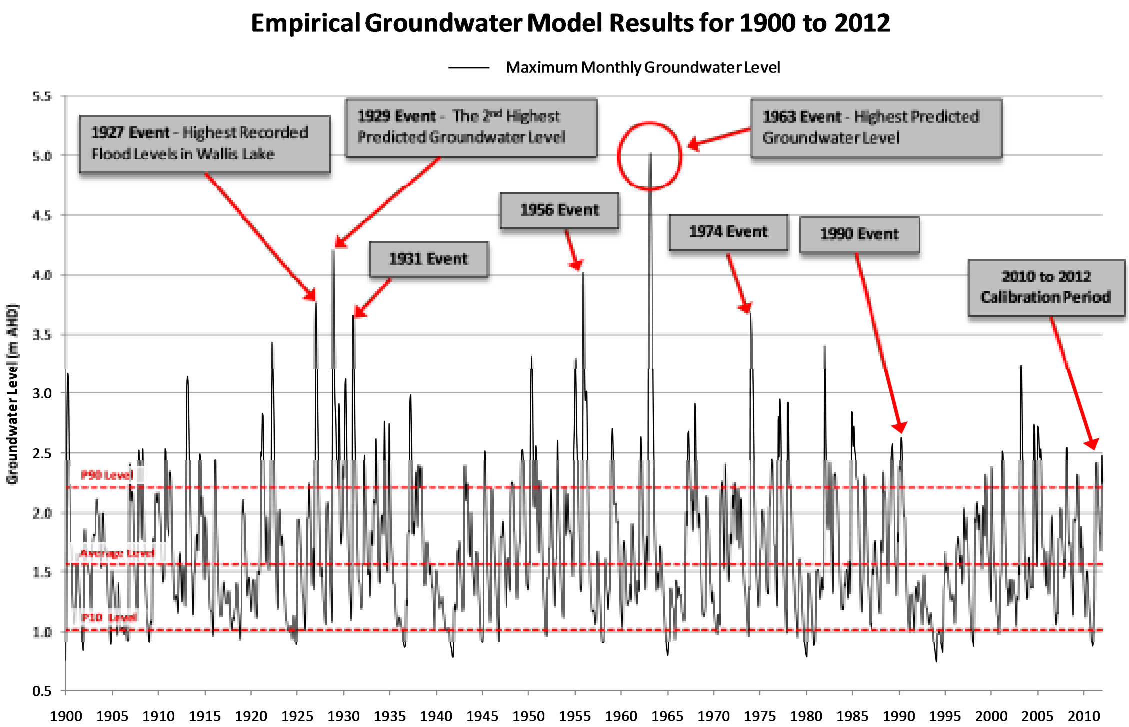

Figure 7 – SMEC Model 2 – Empirical Model Results showing peak 1963 flood event (record length 1900 to 2012)

The rainfall recharge conditions derived from the Empirical modelling were used in the MODFLOW Surfact model to determine the design groundwater surface profile across the subject site.

The results of this assessment are outlined below in **Figure 8** which shows a plan indicating maximum depth of ponding on the subject site. This modelling indicated that the worst case groundwater flooding occurred in 1963, when over 1500mm of rainfall occurred over a 3 month period, and the modelling predicted a peak groundwater level of RL 5.0m, AHD would be reached on the site during the 1963 event. The majority of surface ponding would be less than 1m deep, however, ponding depths in excess of 2m are predicted in some localised areas where surface levels are below RL 3 m AHD. Importantly, existing surface water ponding attenuates the rise of groundwater levels as surface water storage is more efficient than groundwater storage. Hence, if the ponded areas were filled, groundwater levels would rise further to compensate for the loss in surface storage.

Comparison of the peak groundwater levels for the existing climate conditions and climate change scenario, indicate that a 0.91m rise in sea level would result in an increase in the peak groundwater levels in the eastern portion of the development area. With reference to **Figure 9**, the groundwater levels as affected by sea level rise are approximately 0.5 m higher at the eastern boundary of the development area. There is negligible (less than 0.1m) difference between the peak levels 400m inside the eastern development boundary. These results demonstrate that higher ocean levels will not significantly increase groundwater flood levels within the development area.

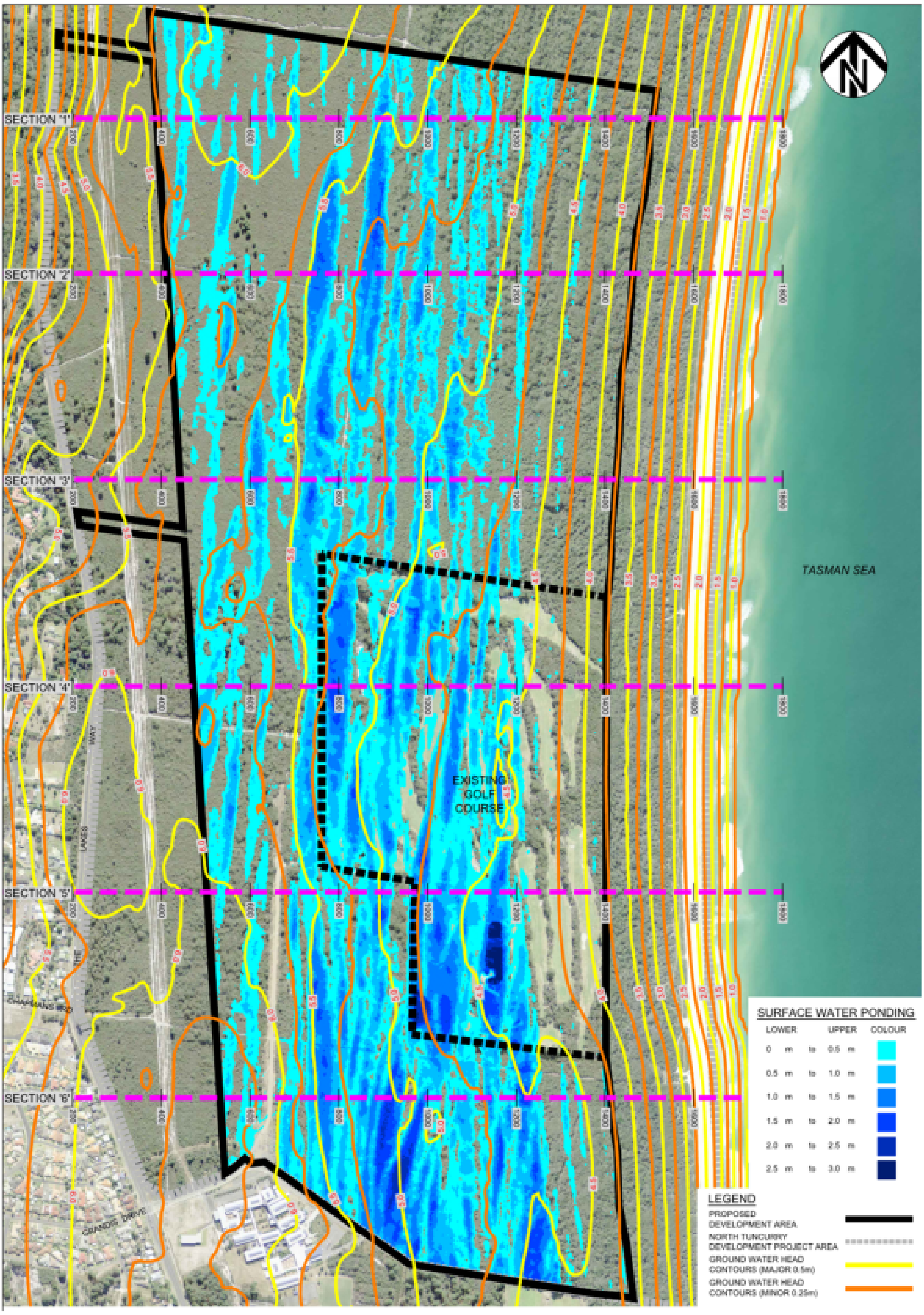

Figure 8 – Predicted Flood Depths across the site for the 1963 Design Flood Event (no sea level rise)

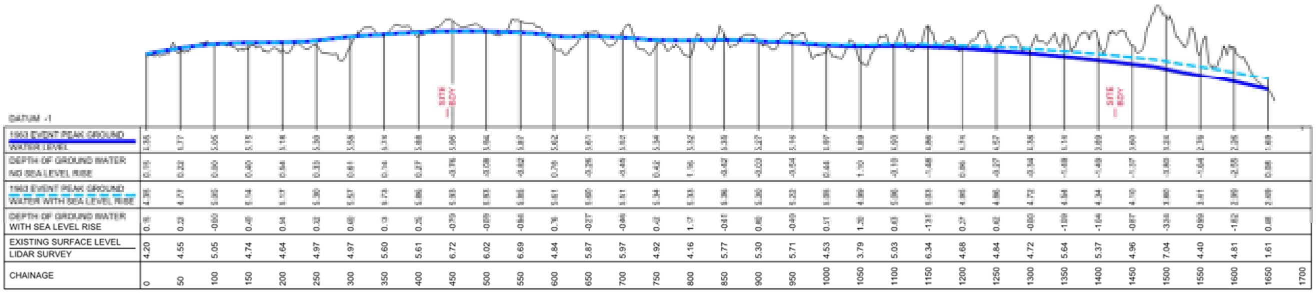

Figure 9 – Predicted Flood Profiles across the site for the 1963 Design Flood Event (with and without sea level rise)

A Flood Frequency Analysis (FFA) was undertaken on the predicted flood events, to determine the relative Average Recurrence Interval (ARI) of the storm events – refer **Figure 10**. The FFA indicates that the 1963 event is estimated to be close to approximately a 190 year ARI event. Due to the level of uncertainty involved in this assessment, the 1963 event peak flood level has been adopted as the Flood Planning Level for the project.

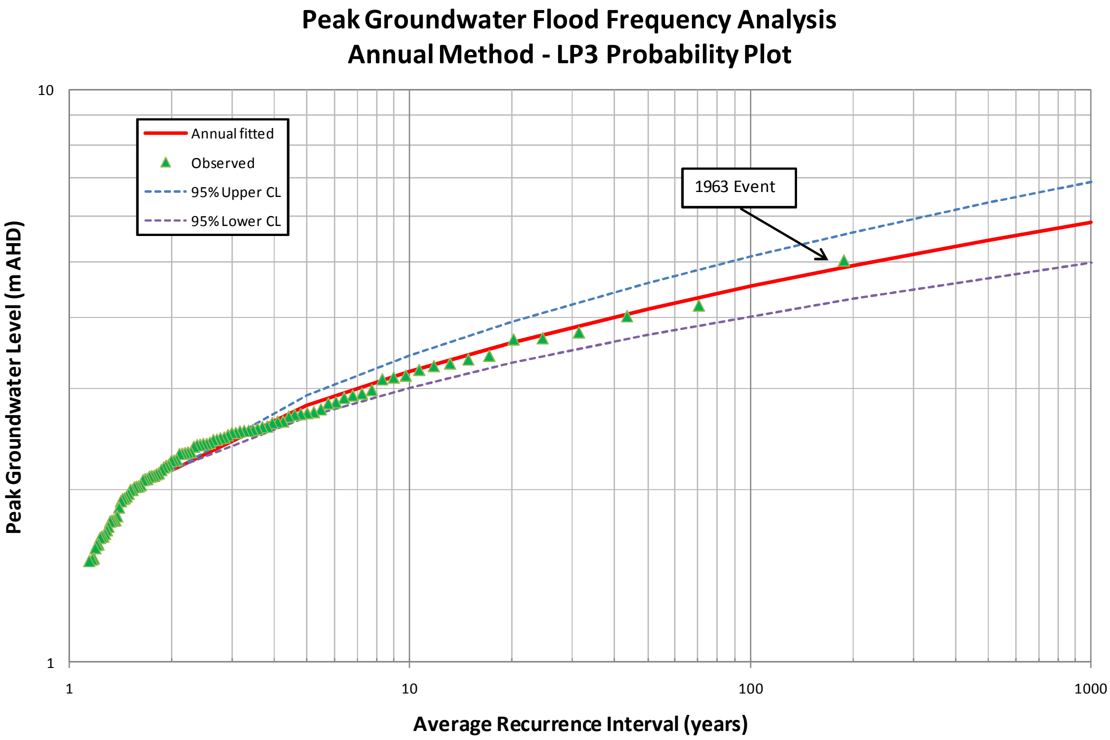

Figure 10 – FFA applied to predicted peaks (1900 to 2012) showing 1963 event close to 200 yr ARI

4. Floodplain Management Objectives

Once the flooding problem was defined by the flood study, floodplain management objectives were developed by SMEC to determine appropriate controls on development and to define the objectives of the proposed floodplain management measures to be designed for the site – refer **Table 1** below. Note these objectives are still in preliminary form, and are yet to be adopted by Great Lakes Council.

Table 1 – Proposed Floodplain Management Objectives to be applied to the development

Floodplain Management Objective	Rationale
Finished Development Surfaces – finished surface levels are required to be at least **1m** above the predicted peak groundwater levels (as defined by the 1963 flood event)	This will provide sufficient freeboard to properties, but also protect road bases and other buried infrastructure and will also ensure that the functionality of the stormwater systems will be maintained under peak groundwater conditions.
Surface Storages – sufficient surface storage volume is required to safely retain 100% of runoff from impervious surfaces during a 100 year ARI, 72 hour duration storm event.	During shorter duration and more intense rainfall events, proposed infiltration systems will become overwhelmed and significant runoff volumes will occur from impervious surfaces. This runoff must be stored safely on-site prior to discharge or infiltration to the groundwater (or discharge to the ocean via pumping). Surface storages will also provide some resilience to more extreme rainfall events.
Nuisance Flooding – normal application of minor / major stormwater management principles for surface drainage systems through-out the development area. Water quality treatment to be incorporated within the minor drainage system to meet Great Lakes Council requirements	The introduction of impervious surfaces such as roof and road areas will introduce a component of overland flow that does not currently exist at the site. Normal road drainage measures will be required to deal with these more intense short duration rainfall events.
Flood Evacuation during extreme rainfall events – designs should be robust such that there is sufficient time to evacuate residents from low lying areas during extreme rainfall events (e.g. up to and including a PMF flood event).	Due to the lack of existing overland flow paths, events more extreme than the design event will cause ponding above ground level, that will not flow away from the site. While difficult to design for flood free conditions, evacuation of residents should be possible without injury or loss of life.
Limitation of effects on regional / local groundwater regime – Council and NSW Office of Water (NOW) are keen to ensure that the proposed flood mitigation measures do not detrimentally affect the local / regional groundwater resource	Simply drawing down the local groundwater regime using pumping could affect the groundwater resource and could affect local groundwater Dependent Ecosystems (GDEs)

The criteria to have finished surface levels at least 1m above the predicted peak 1963 groundwater levels has been adopted due to the level of uncertainty associated with the adopted flood study assessment. Note also that adoption of the 1963 flood event also allows for a degree of conservatism, as this event has been demonstrated to exceed a 100 year ARI event.

5. Option Assessment for Flood Mitigation Strategy

Broadly, there are three broad options that can be applied (either separately or in conjunction) to manage the groundwater constraints within the Project Area, and to achieve the above floodplain management objectives:

1. Site Filling

Key issues include:

- -ve cost (high fill volumes would be prohibitively expensive);
- -ve loss of surface storage (can increase groundwater levels); and
- -ve loss of surface storage removes resilience of the system during more extreme rainfall events.

2. Increased Surface Storage

Key issues include:

- -ve loss of development area (indirect cost)
- +ve local groundwater effects (i.e. effectiveness in reducing GWLs in development areas)

3. De-watering (pump or gravity drainage)

Key issues include:

- -ve discharge of water (aesthetic and coastal processes issues)
- Pump triggers (governs the frequency of pumping)
- +ve local / regional groundwater effects (i.e. effectiveness in reducing GWLs in development areas)
- cost (can be minimised by limiting pump size, frequency of pumping and discharge pipe / rising main length)

The preferred solution was a combination of Options 2 and 3, with strategically placed open voids that provide an effective means of managing surface stormwater as well as groundwater when compared to conventional methods for dewatering, such as the use of numerous spaced boreholes. During normal rainfall events, voids provide open storage to enable stormwater to be intercepted and soak into the groundwater system, but during extreme events when the water table rises significantly, the voids can provide storage to intercept surface runoff and reduce groundwater flooding and also serve as a means to dewater the aquifer via pumped drawdown. The concept of the open voids (or drains) combined with pumping is illustrated in **Figure 11**.

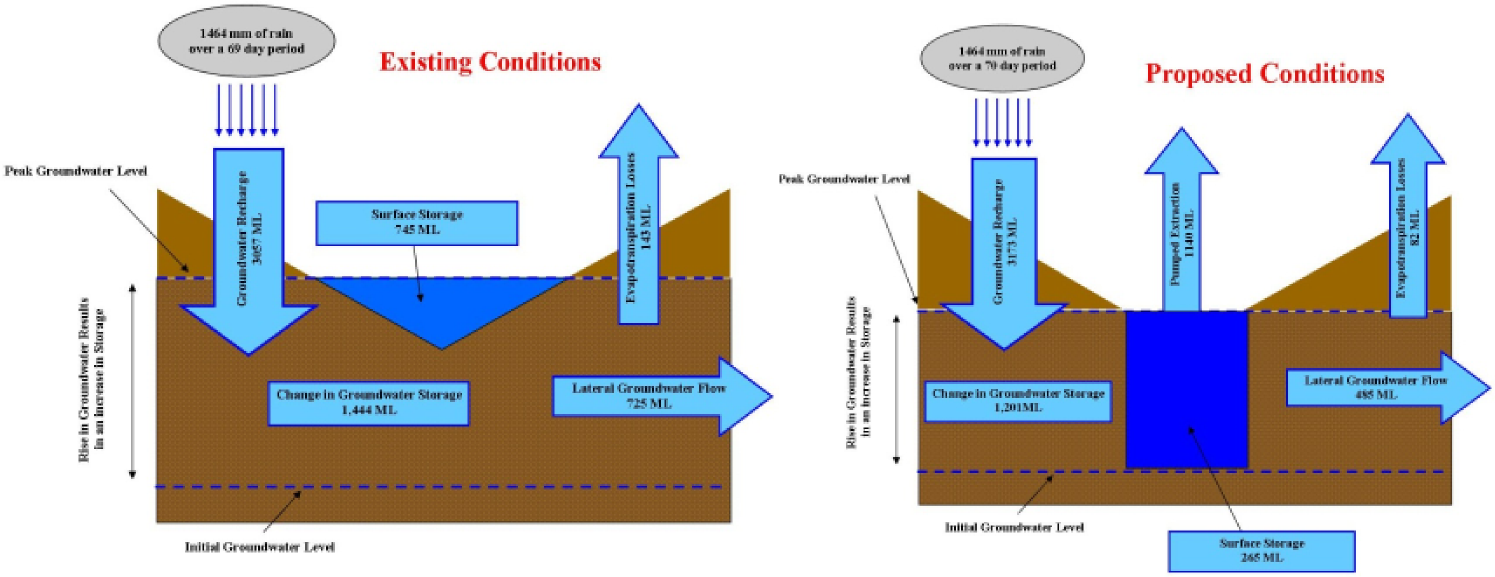

Figure 11 – Proposed Solution – open voids (drains) with pumping to manage groundwater flooding

Once the preferred concept was derived, optimisation of the design was carried out using the Empirical Groundwater modelling, and cost modelling. Variables to be optimised included the percentage of the total development area set aside for surface storages, the volume of fill required, the pump out rate from the voids, and the pump cut in level. **Figure 12** illustrates a typical optimisation to determine the appropriate area of development, against surface storage size and pumping requirements.

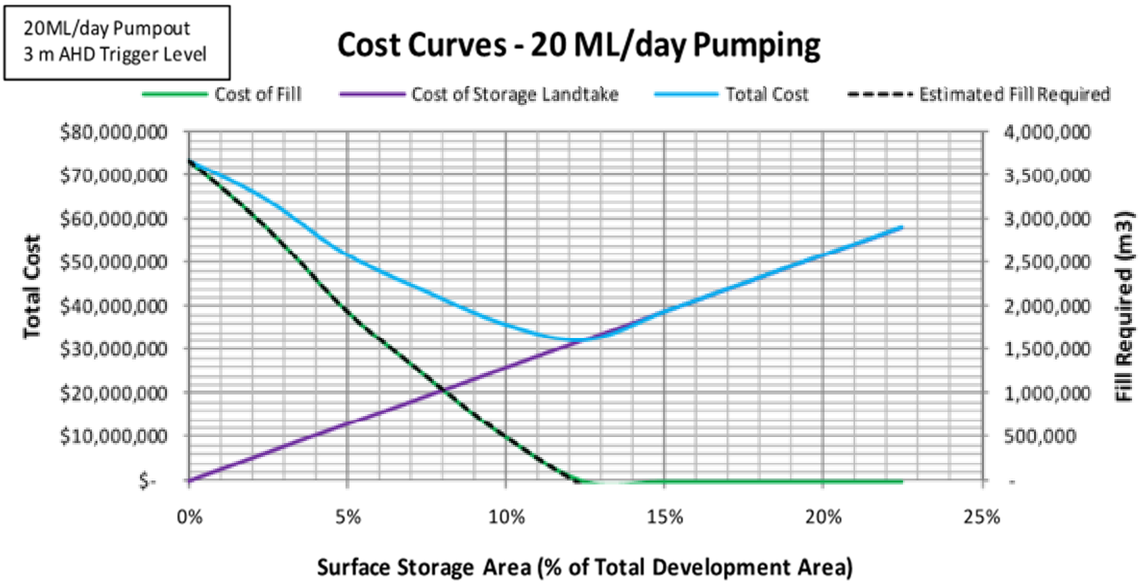

Figure 12 – Floodplain Management Option Comparison

A key parameter is the rate that water could drain towards the open voids. Empirical modelling of the voids was carried out to determine the draw down required to ensure that the peak 1963 groundwater level remained within the Floodplain Management Guidelines derived above (i.e. at least 1m below the finished ground surface) – refer **Figure 13**.

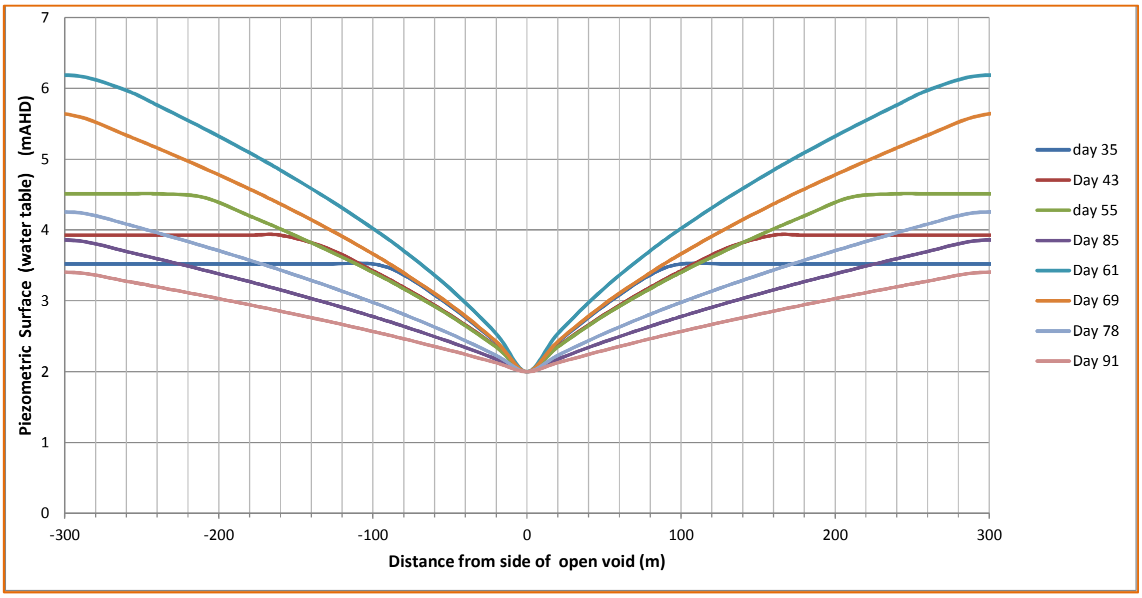

Figure 13 – Simulated depression in water table due to drainage to an open void – 1963 event

6. Concept Design of Flood Mitigation Strategy

The preferred option was developed into a concept design that included a continuous open drain running through the middle of the development, that forms a large "C" shape, and that is drained via pumped extraction at the north-eastern and south-eastern corners – refer **Figure 14**.

It should be noted that this concept is yet to be developed further, most likely into a series of open ponds or dry basins interconnected by smaller open channel, or even culverts. Masterplan design on the preferred development layout including the preferred drainage system will be commencing immediately and will be completed over the next few months.

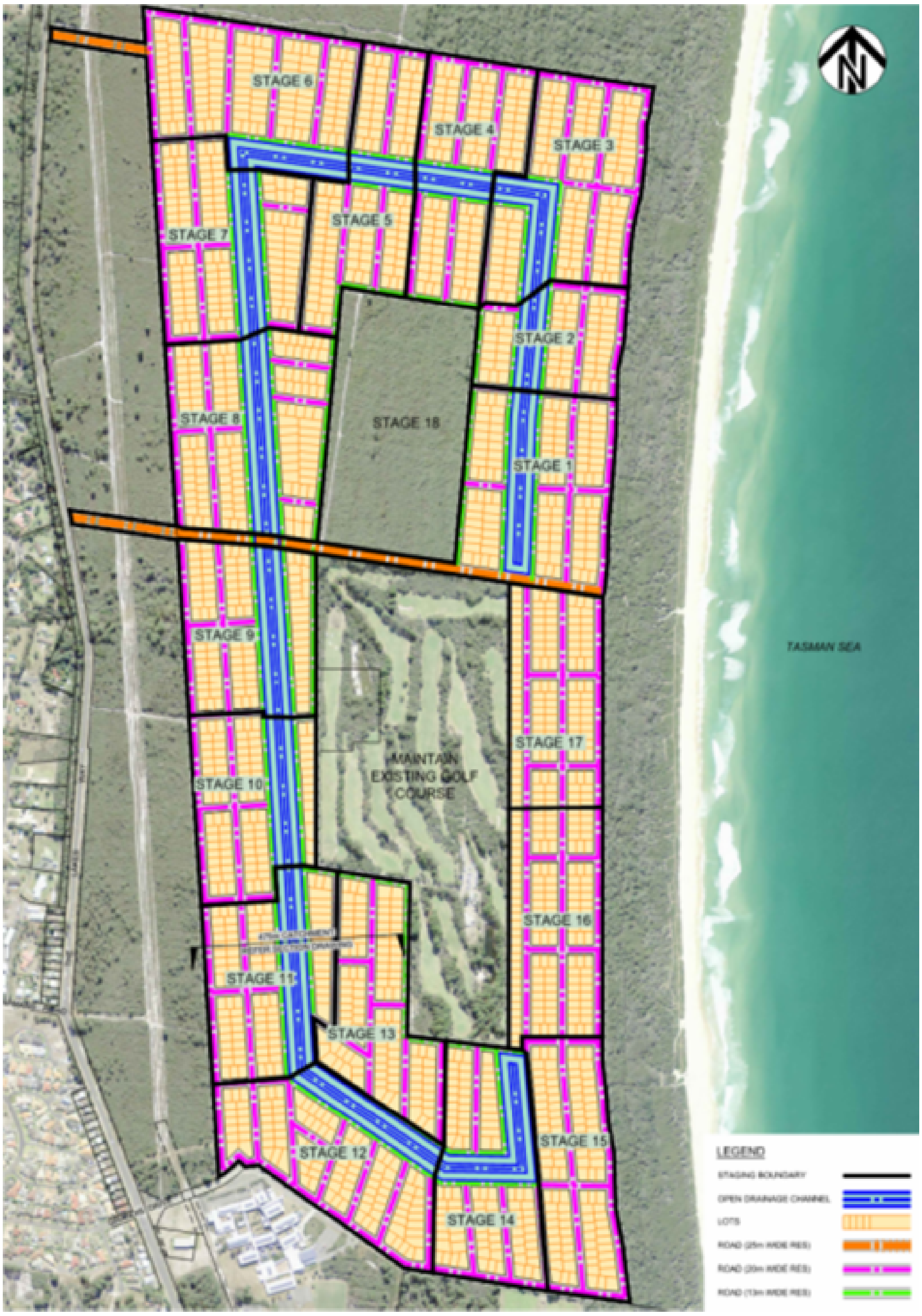

Figure 14 – Proposed Open Drain around site

The extracted water is intended to be exfiltrated via a series of large exfiltration beds within the back-dune area, such that excess water reaches the ocean via the groundwater system which is considered preferable to having visible pipes and outlets at the beach – refer **Figure** 15.

Pumping would be controlled by the rise in water level in the open drain, such that only under certain high groundwater levels, would pumped discharge to the ocean occur.

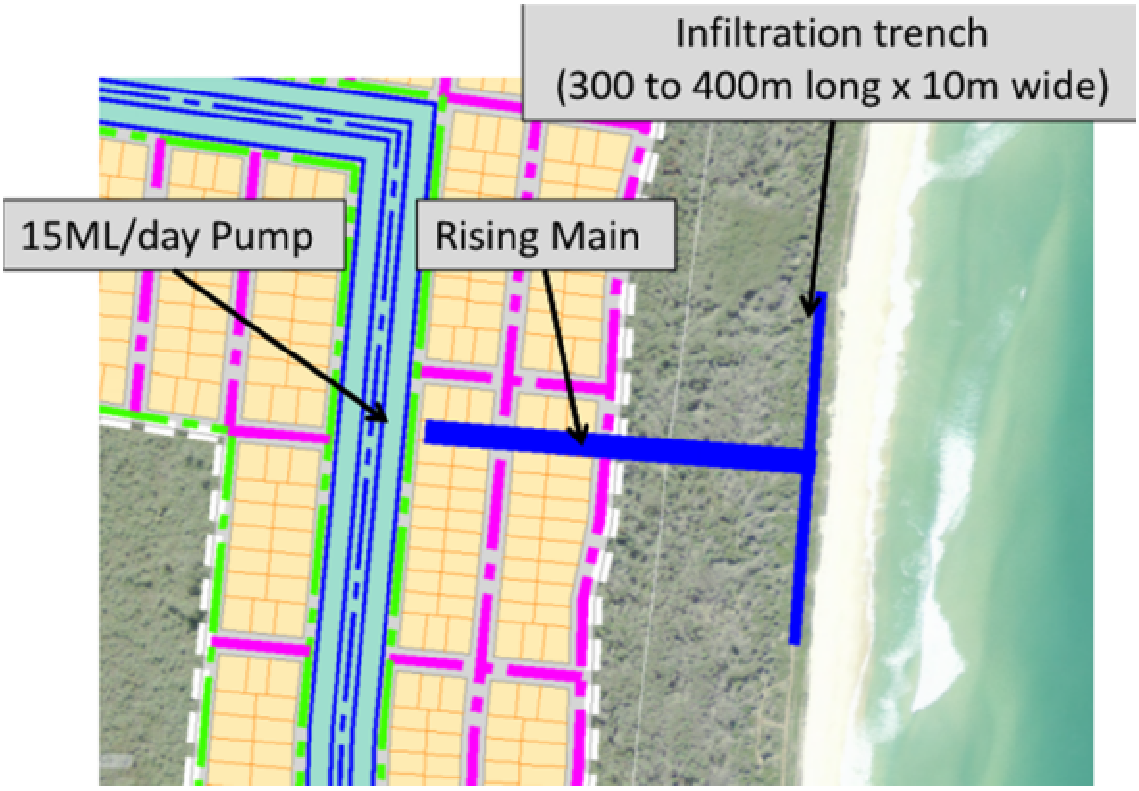

Figure 15 – Pumped outlet rising main and exfiltration bed at beach-dune

7. Conclusions

The groundwater monitoring network, field testing and calibrated MODFLOW Surfact model as used in combination with a custom built model of rainfall recharge to groundwater has provided a basis for highlighting the extent of possible flooding due to groundwater rise during extreme rainfall events.

The significance of having confirmed this prior to final planning is that appropriate measures can now be included in the planning of the North Tuncurry Development Project to mitigate and minimise the risk of groundwater flooding.

Also relevant is the relative insensitivity of the predicted flood levels to rising sea levels, implying that sea level rise should not be a significant risk to the project provided that appropriate mitigating measures for groundwater flooding are included in the integrated water management plan for the development.

Monitoring networks established for the project should be maintained with a view to being able to quantify the effects any future rainfall extremes and also serve to enable model verification as the project evolves over time. The network will also provide information for water quality management.