Conceptual Design of MacMasters Beach Seawall - Coastal Processes, Design Conditions and Design Concepts

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1 Introduction

1.1 Background

During a significant coastal storm (4-7 June, 2016) where large ocean swell combined with "King" tides, the southern end of MacMasters Beach was eroded. The width of foreshore fronting an outdoor dining area at the MacMasters Surf Life Saving Club (SLSC) building was significantly eroded. There are concerns that further storm activity might cause partial collapse of the outdoor dining area.

Central Coast Council has engaged Salients Pty Ltd (Salients) to prepare a conceptual design for a seawall to protect the area. Salients has, in turn, engaged the services of Pells Consulting to provide a geotechnical investigation.

The Coastal Zone Management Plan for Gosford's Open Coast Beaches (the CZMP) has identified that a rock revetment seawall would be an appropriate management option for this length of coastline. The present report examines coastal processes in and around the site, and derives appropriate design conditions for such a structure. Design options for the structure are considered in Chapter 5.

As part of the work presented here, the following have been completed:

- Key study team members have inspected the site;
- Previous assessment of the Beach was considered, primarily with reference to the existing *Coastal Processes and Hazard Definition Study* (Worley Parsons, 2014a);
- Historical aerial photographs of the Beach were acquired, georeferenced and orthorectified to enable examination of historical beach behaviour;
- Existing wave and water level data were acquired and reviewed; and
- A numerical wave model was developed to analyse design wave conditions at MacMasters Beach.

Along with the above items, the coastal processes assessment (Chapter 3) includes consideration of geotechnical information provided by Pells Consulting (appended to this report) and is intended to provide information suitable for incorporation into a subsequent review of environmental factors (REF). Furthermore, design conditions derived from our assessment of coastal processes are presented. The report is structured as follows:

- Chapter 2 outlines the locality and characteristics of the site;
- Chapter 3 presents the coastal processes assessment, incorporating the main elements of work outlined in the preceding set of dot points, and a summary of data relied upon;

- Chapter 4 presents design conditions suitable for the revetment; and
- Chapter 5 discusses the conceptual design.

2 Site Description

MacMasters Beach is located around 12.5km to the south-east of Gosford's central business district. Its location and an aerial photograph of the Beach from 2015 are presented in Figure 1. MacMasters Beach, also known as Copacabana -MacMasters Beach, sits within a 1.4km long south-east facing embayment between Tudibaring Head to the north and Second Point to the south. The intermittent Cockrone Lagoon discharges across the middle of the Beach.

The residential suburbs behind the Beach to the north and south of Cockrone Lagoon are Copacabana and MacMasters Beach respectively. Macmasters Beach was the earlier settlement with Copacabana being subdivided during the 1960's[1]. The Macmasters SLSC was formed in 1946, with the present building apparently constructed in the 1970's.

The area which is being studied closely as part of this assessment is presented in Figure 2. The conceptual design covers a length of foreshore stretching from the boat ramp at the northern end of the SLSC building to the stairs landward of the ocean pool to the south of the building. The foreshore length of concern is around 90m.

The contours on Figure 2 show that the foreshore fronts a level area upon which the SLSC building has been constructed. This levelled area narrows towards the south, where the immediate foreshore merges with the slopes of Second Point. Landward of the levelled area, the ground rises steeply. This topography strongly suggests that the level area upon which SLSC building has been constructed is artificial. To the north of the boat ramp, a stormwater line exists across the Beach, draining a depression immediately to the north of the SLSC building (i.e. at the end of Marine Parade).

In front of the SLSC building, and immediately behind the foreshore, there exist a variety of landscaping elements, including Norfolk Island pines, some of which are threatened by undermining if further erosion occurs. Furthermore, seating, paving and fencing exist between the SLSC building and the foreshore, comprising an outdoor dining area for patrons of the café which operates out of the SLSC building.

During the storms of 3 – 7 June, the foreshore fronting the dining area was eroded by large waves occurring at the same time as elevated tides. For some of this time the waves arrived from east and north-easterly directions, meaning that the southern end of MacMasters Beach was directly exposed to incoming waves. Figure 3 shows the state of the embankment during an inspection undertaken a few days after the storm. More photographs of the area described above are presented in Appendix A.

[1] https://beachsafe.org.au/beach/nsw/gosford/macmasters-beach/macmasters. Accessed 14 October, 2016

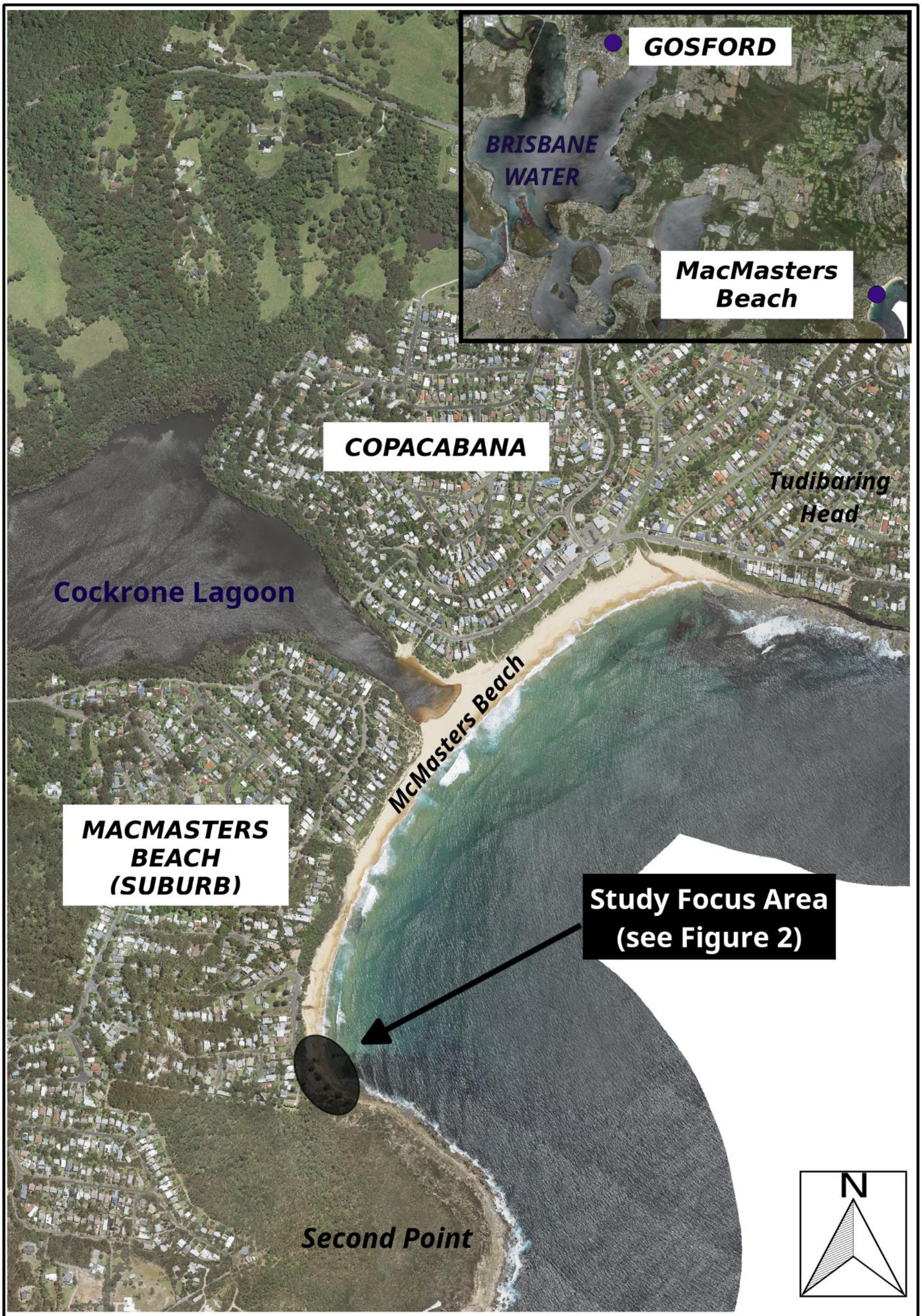

Figure 1: MacMasters Beach Locality

Conceptual Design of MacMasters Beach Seawall	REV A	DRAWN DJW	CHECK DJW	Salients

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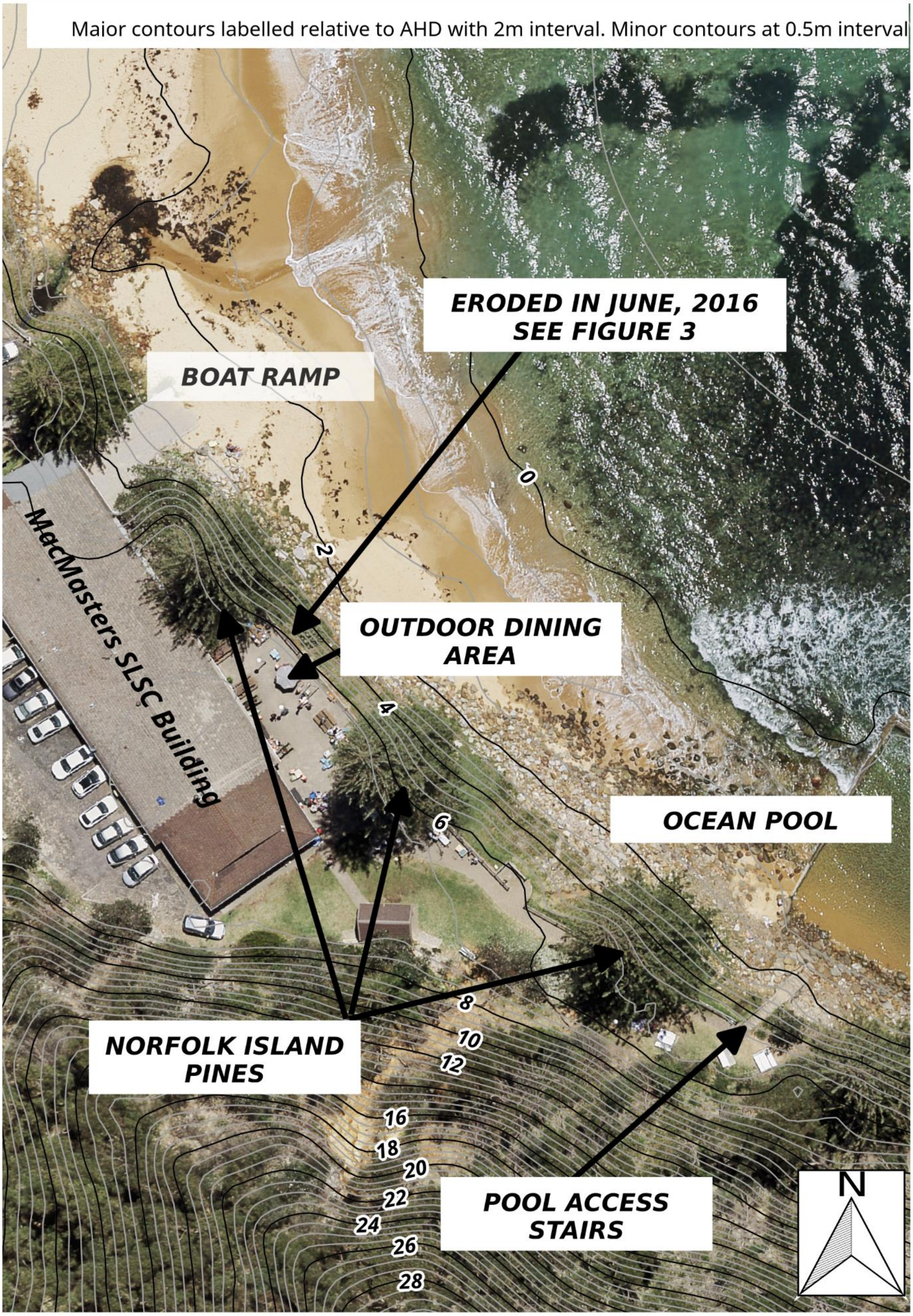

Figure 2: Southern MacMasters Beach

Conceptual Design of MacMasters Beach Seawall

REV	DRAWN	CHECK
A	DJW	DJW

GIS File: /Projects/P00016_MacMastersRevetmentConceptDesign/Outgoing/Figures/Figure_02_A.qgs

Salients

Figure 3 Eroded Foreshore, June 2016

During the storm, waves impacted directly against the base of the embankment. SLSC members have reported that the width of a grassed verge adjacent to the front edge of the dining area (i.e. seaward of the glass barrier visible in Figure 3) had receded by more than a metre during and immediately following the storm. SLSC members were concerned that further collapse was imminent and eventually placed bags filled with sand in front of the embankment. These can be seen in Appendix A (Figure A2).

Examination of the embankments soil stratigraphy reveals surface topsoil, underlain by loosely placed (i.e. uncompacted) sand which differs from the quartzose sand present on the beach face to the north of the site. The sand fill overlies what appears to be residual soil resulting from weathering of the underlying bedrock. Considering overall topography of the site, and the prevalence of smoothed Hawkesbury sandstone boulders across the foreshore (mixed with some building rubble), it was expected that bedrock could be encountered close to the base of the eroding embankment.

The embankment at this location remains vulnerable, particularly if it is exposed to a further storm sufficiently severe to enable waves to attack the loosely placed, imported sand fill layer. Collapse of the embankment and parts of the outdoor dining area could foreseeably occur if the situation is not addressed. The sand bags placed by the SLSC do not provide a suitable level of protection against coastal erosion.

3 Coastal Processes

3.1 Previous Assessments

A general description of conditions at Macmasters – Copacabana Beach is provided on the Surf Life Saving Australia *Beachsafe* website[2]. Therein, it is noted that the Beach faces towards the east-southeast, receiving average waves of 1.5m at the northern (Copacabana) end, decreasing to an average of around 1.0m at the MacMasters end.

A single bar is generally present and attached to the Beach, but cut by between 6-8 rips. These rips decrease in strength to the south (MacMasters end) and are prone to infilling. During high wave conditions, a strong rip can form against the southern headland, commonly occurring during north easterly waves that are more prevalent during summer months.

The most recent coastal processes and hazard definition study (Worley Parsons, 2014a) provides a thorough review of past information relating to coastal processes at MacMasters Beach, along with information that could be used for design purposes.

The suburb of MacMasters Beach was subject to rapid development following subdivision in 1927. Based on interpretation of aerial photography, Worley Parsons (2014a) have classified MacMasters – Copacabana as receding, losing around 1.4m^3/m/yr, translating to a landward retreat of around 0.25m/year.

Conditions at the southern end of the Beach are interesting for several reasons. Firstly, aerial photography of the beach shows that the southern end is extensively underlain by cobble/boulder sized rounded stones. An idea of the extent of these cobbles can be seen in the photograph provided as Figure 4. From that figure, and others presented in Appendix C, it is estimated that this shallow natural armour extends for at least 150m north of the SLSC building. While individual stones are capable of being moved when a coastal storm attacks this length of beach, they still act to provide significant protection to the beach by dissipating wave energy and retarding the erosion of any underlying sand. Based on available aerial photography, the natural armour appears to have been transported northwards and landward from the offshore rocky reefs and adjacent rock platform of Second Head.

Figure 4 also clearly shows that the platform upon which the SLSC building is constructed is artificial, with the fill material apparently sourced from the hill to the rear of the SLSC.

[2] https://beachsafe.org.au/beach/nsw/gosford/macmasters-beach/macmasters. Accessed 14 October, 2016

Figure 4 Eroded Southern end of Beach in 1974, showing extent of protective cobble/boulder armour and construction of the SLSC building (from Worley Parsons, 2014)

It is likely that, near the SLSC building, the natural armour protection continues for some distance landward, below the filled platform. Worley Parsons (2014a) recommended that a subsurface investigation be undertaken to assist with refining an estimate of storm erosion demand. The field investigation undertaken as part of this study (see Appendix C) provides some information that may assist in this regard. Worley Parsons reported that, during the May-June 1974 storm sequence, wave run-up overtopped the “dune crest” at the southern end of the Beach and that the lower part of SLSC building was inundated. Considering the elevations shown in Figure 1 and that Worley Parsons calculate a design wave run-up level of 6.0m AHD, it appears that this “lower part” refers to the storage area immediately landward of the boat ramp. At present, the paved outdoor dining area fronting most of the SLSC is unlikely to be inundated by anything but the most extreme storms (i.e. larger than the 1 in 100-year storm).

Macmasters Beach has been subject to significant erosion at several occasions in living memory. While the southern end of the Beach is somewhat protected from the more frequent erosion experienced at more northern locations along the beach, there is some localised erosion of sand in front of the stormwater outlet immediately to the north of the SLSC boat ramp (Figure 1 shows the location). Even so, the presence of natural armour prevents erosion from scouring significantly downwards at this location.

Substantial analysis of wave propagation, wave setup, storm surge and coastal hazards were undertaken by Worley Parsons. Wave transformation modelling using SWAN was used to propagate offshore waves from inshore to a depth contour of 6.5m below Australian Height Datum (AHD) (around 6.5m deep at mid tide). From these analyses, a “Peak Wave Coefficient” of 0.85 was determined for the southern end of the beach. While not clear from the main project report, it appears that these coefficients account for the effects of refraction and shoaling. However, it should be noted that the value of such coefficients is affected by the wave period (which strongly affects the degree of refraction) and the wave period used in determining the reported coefficients is not stated in Worley Parsons (2014a).

Using similar methods, a design inshore 100yr average recurrence interval (ARI) significant wave height (H_s) of between 6.0 and 6.5m was determined for the southern end of the beach. If a design still water level of around 1.5m AHD was used, this would indicate that the inshore design wave here is governed by wave breaking, assuming that this process was enabled when the SWAN model was executed. Design conditions for this location would reflect the highest wave that could physically break on the structure, governed by a “depth limited” wave breaking condition. The design wave breaking at the structure is expected to be markedly smaller than 6.0 to 6.5m.

The recommended design 100yr ARI water level of 2.4m apparently includes a wave setup of somewhere between 10 and 15% of the offshore significant wave height (Worley Parsons, 2014a). These assumptions have been considered further as part of the present study when determining appropriate design conditions.

In deriving coastal hazard lines for the southern end of MacMasters Beach, noting that the protective effects of the natural armour were not considered and an entirely sandy substrate was assumed, the following were used by Worley Parsons:

1 Recession due to sea level rise: 13.3m between 2011 and 2050, 32.8m between 2011 and 2100;

2 A storm demand of 200m^3/m

The resulting mapped hazard line for the southern end of MacMasters Beach is reproduced in Figure 5. Therein, the SLSC building is subject to ‘immediate’ coastal hazards, meaning that the building could be destabilised during a rare but foreseeable coastal storm, depending on how the building is founded.

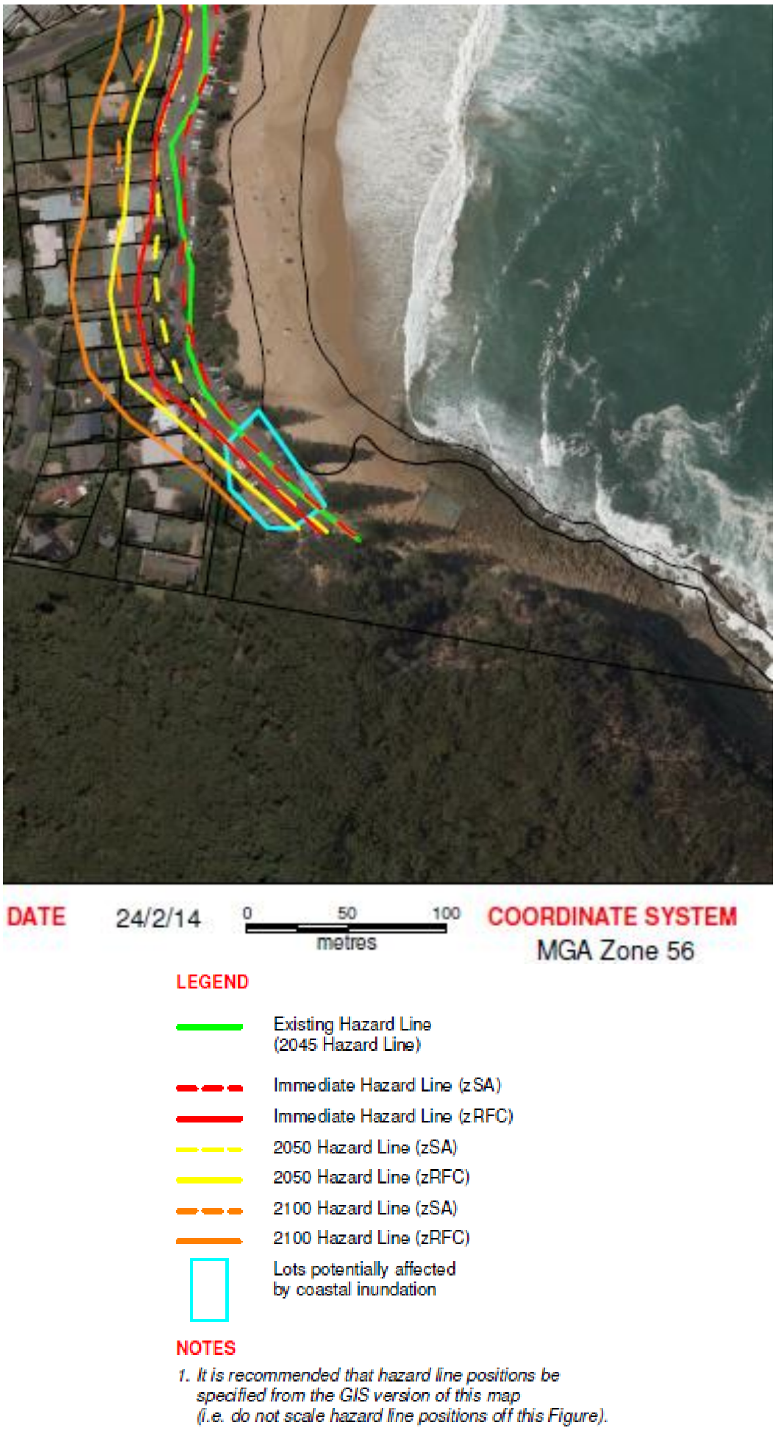

Figure 5 Hazard Lines for Southern MacMasters Beach, as determined by Worley Parsons (extract from Worley Parsons(2014b))

3.2 Available Data

3.2.1 Aerial Photography

Central Coast Council provided a range of high resolution scans of historical aerial photographs (prior to 2000) and access to online aerial photography dating from 2005 onwards. Historical scans were imported to the Quantum Geographical Information Systems (QGIS) software and adjusted through orthorectification and georeferencing such that they could be compared to the online data, which were already presented in a known geographical coordinate system.

Individual maps for each year of photography are presented in Appendix B, and a discussion comparing the photographs is provided in Section 3.3.

3.2.2 Topography

Following the storm of June 2016, Council commissioned survey of the beach using an unmanned aerial vehicle (UAV or "Drone"). Using a combination of high resolution digital photography and photogrammetric techniques, a very detailed digital elevation model (DEM) was derived and provided for use in this study. The DEM is particularly useful, as it captures the beach in a particularly eroded state, with the natural armour fronting the SLSC building exposed.

An accompanying high resolution aerial photograph was also provided and has been used to inform design. That photograph clearly captures construction activities associated with a temporary access path in front of the SLSC building. It is understood that this work was independently organised by the SLSC to facilitate the placement of "bulka bags" full of sand to protect the eroding foreshore of concern. Furthermore, we understand that this work was undertaken a couple of weeks following the June, 2016 storm, i.e. during mid to late June.

The aerial photograph has been used to examine and adjust the digital elevation model. This was required as the supplied DEM contained artefacts related to items such as trees and outdoor dining furniture which needed to be removed to properly capture ground elevations. Those features were filtered out and the resulting DEM and underlying aerial photograph have been used during concept design preparation (for example, in Figure 14).

3.2.3 Bathymetry

Bathymetric data were required as an input to the wave model described in Section 3.4. The primary source of bathymetric data was the hydrographic series map AUS 204 (Australian Hydrographic Service, 1977). Contours as shallow as -16m AHD were digitised into GIS. Additional information was gathered from contours derived from LiDAR elevations collected in 2013 and provided by Council, the drone survey undertaken on behalf of Council in June, 2016, and the aerial photography, also provided by Council. These data sets were all brought into QGIS and used to derive nearshore contours, based on experience and interpretation of the shape of breaking waves, offshore reefs and the shape of the rock platform to the south of the SLSC building.

The resulting digital elevation model was used in derivation of the numerical model, and the resulting representation of bathymetry in the model is presented in Section 3.4.

3.2.4 Wave

The directional wave record for Sydney, extending from March 1992, through September 2016, was provided by Manly Hydraulics Laboratory (MHL) which acts as custodian of these data sets for the Office of Environment and Heritage. In addition, MHL provided the storm history for Sydney's directional record in a summarised form for consideration.

The entire wave record was processed to derive the wave climate, both overall, and seasonal, for the coastline surrounding Sydney. The seasonal wave roses are presented in Figure 6 through Figure 9 for summer through spring respectively and the all seasons wave rose is presented in Figure 10. The figures demonstrate well known features of Sydney's offshore wave climate, namely that it is dominated by swells approaching from the south-east quadrant throughout the year, although this dominance is most pronounced during winter. The contribution of waves from the east and north-east increases as the year progresses through spring and summer. That contribution subsequently decreases again in autumn. Of course, seasonal changes are not the only influences of wave climate with wave storm direction, for example, known to correlate to the El-Nino Southern Oscillation index. However, the pattern indicates that there is a more likely transport of sand towards the Copacabana (northern) end of this embayment during winter, with a greater tendency towards southerly transport during summer months. This southerly transport would tend to result in a sandy beach covering the cobbles/boulders in front of the SLSC building during summer, although this would not necessarily be the case every summer.

Of interest to the present design is the storm in early June 2016. That storm was noted to be peculiar due to the uncommonly high waves approaching from such an easterly direction, particularly during winter. At the peak of the storm, (H_s = 6.52) waves were approaching from around 103 degrees, with earlier waves tending toward being more easterly, and later waves more south easterly.

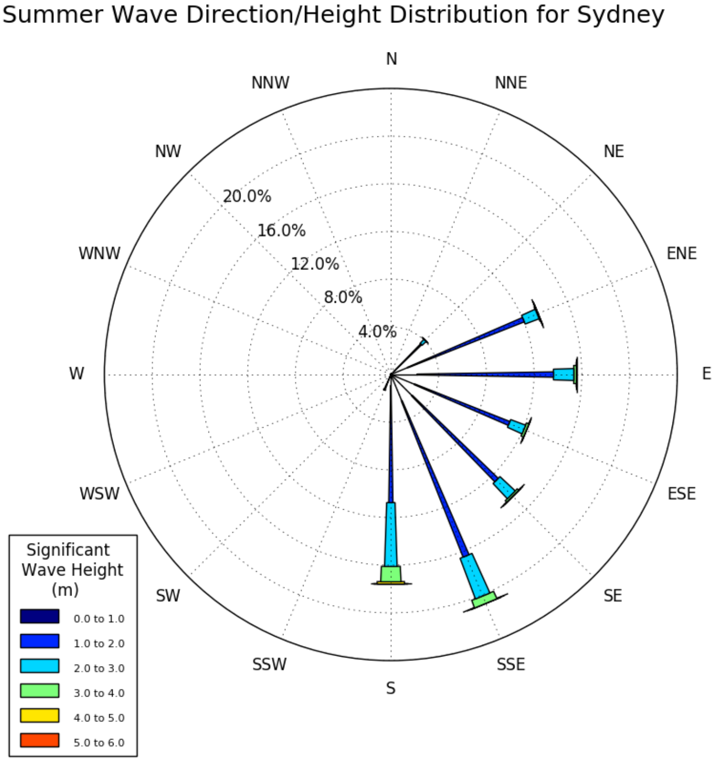

Figure 6 Sydney Offshore Wave Rose (Summer)

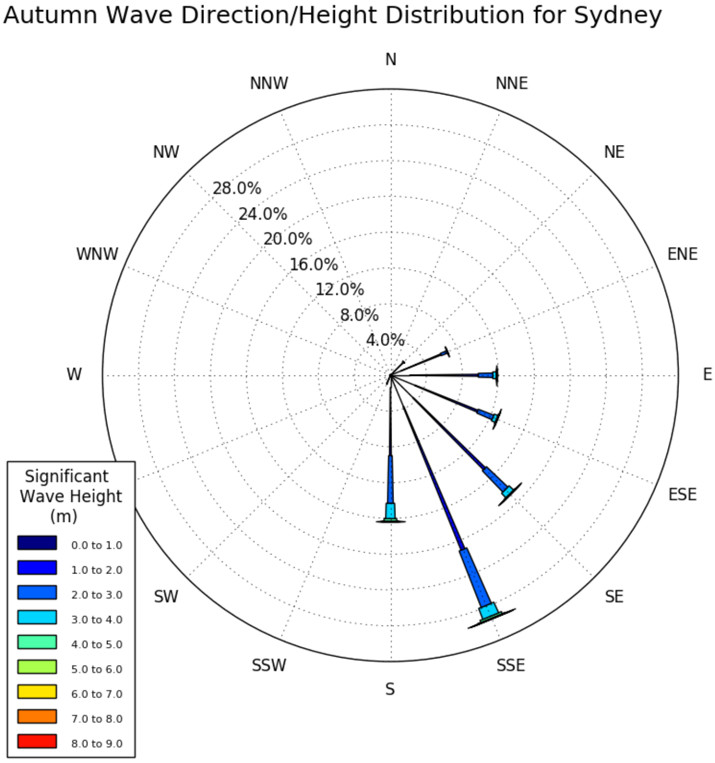

Figure 7 Sydney Offshore Wave Rose (Autumn)

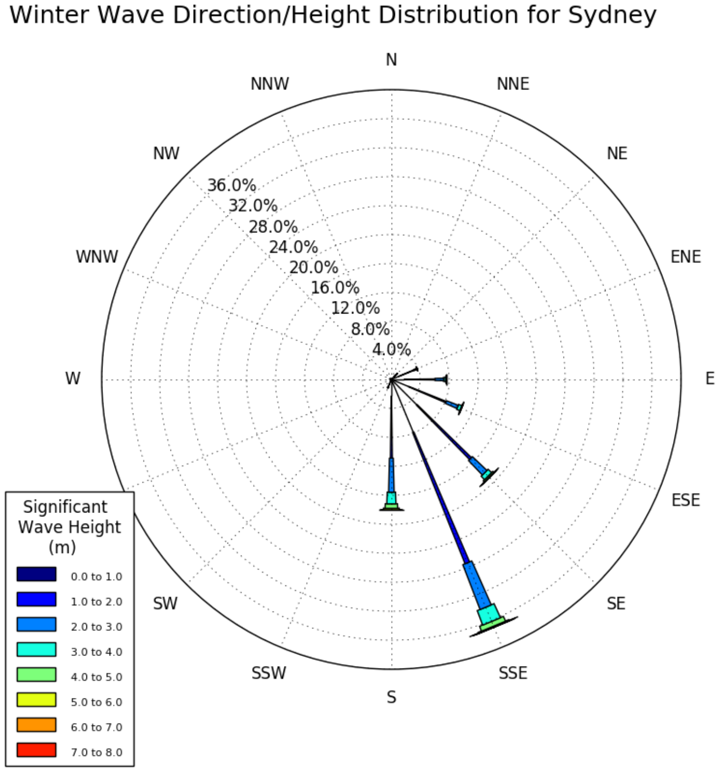

Figure 8 Sydney Offshore Wave Rose (Winter)

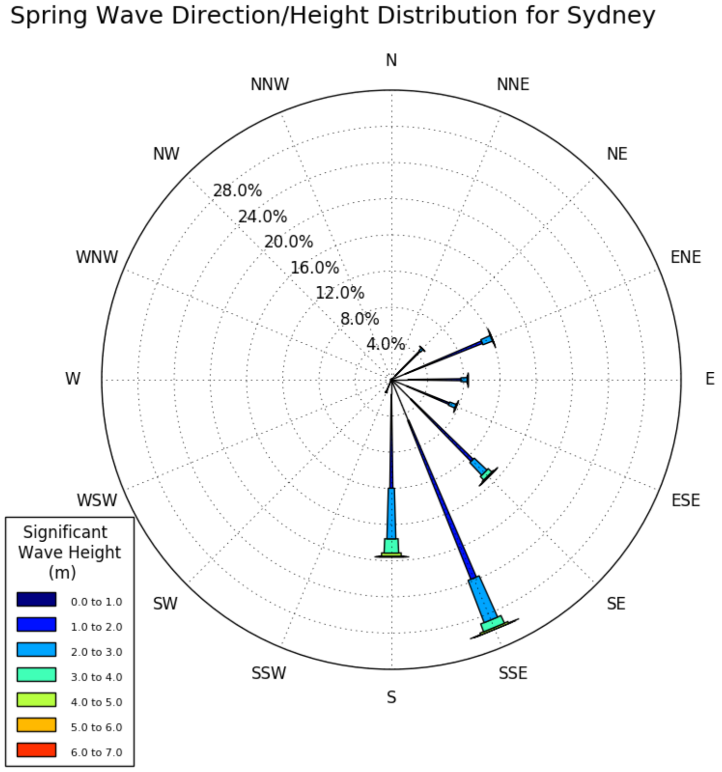

Figure 9 Sydney Offshore Wave Rose (Spring)

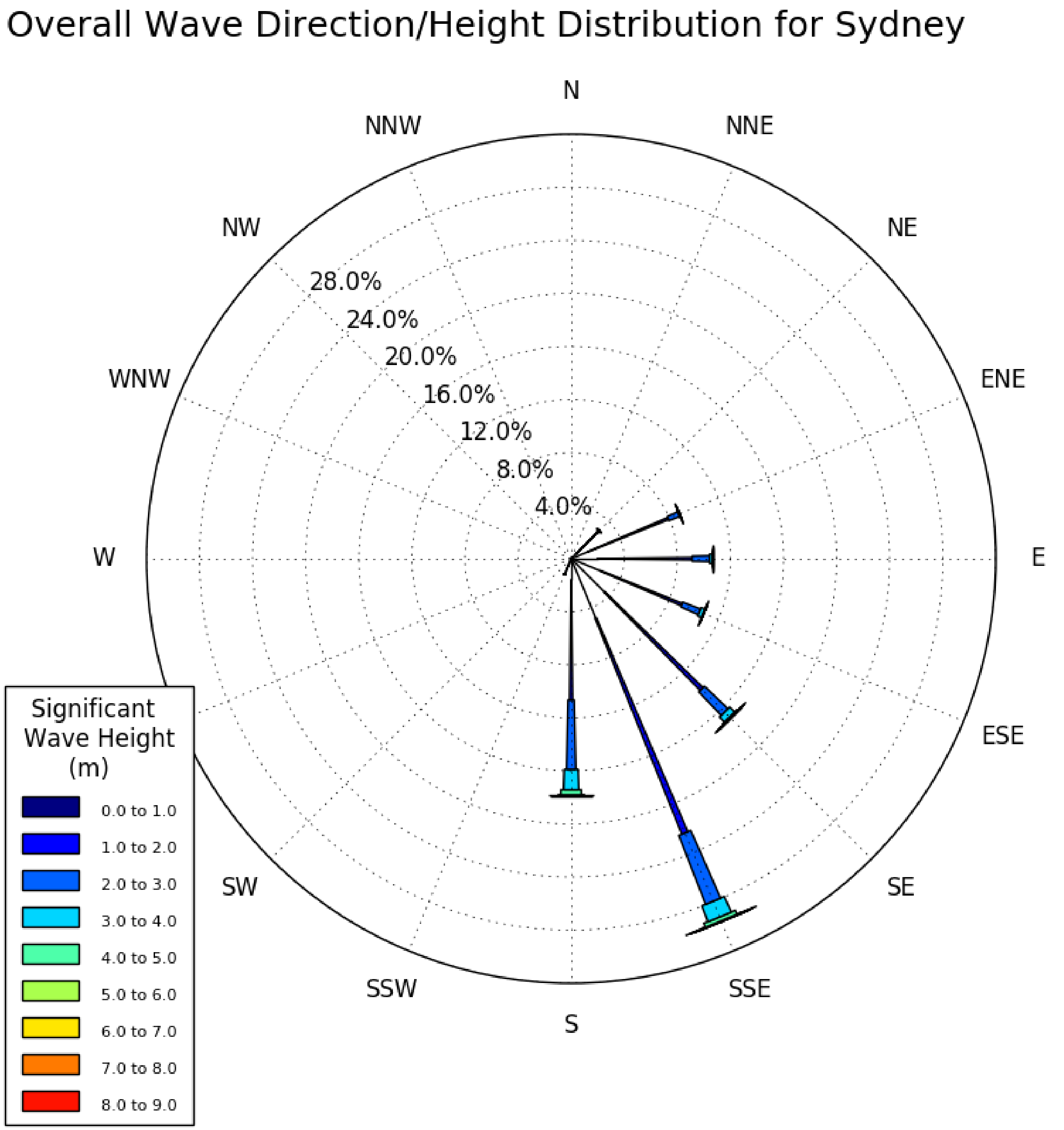

Figure 10 Sydney Offshore Wave Rose (All Seasons)

3.3 Examination of Aerial Photography

Aerial photographs from a variety of years were imported to QGIS and adjusted to enable comparison at a consistent scale and orientation. Figures for all years of available photography are provided in Appendix B and a summary of findings from inspection and comparison between years is presented in Table 1.

Table 1 Summary of Aerial Photography[3]

Year	General Notes	State of Beach
1954	Limited development, unsealed roads. Earlier SLSC building located in northern part of existing building's footprint. Pool not present. Area to south of building (present pool access) appears to be vegetated.	Beach very wide at southern end, compared to more recent times. Extends out to near the present day offshore extent of the pool
1957	Development is almost identical to that present in 1954.	Beach reasonably wide, but has narrowed somewhat since 1954. Cobbles are exposed on beach to north of the present-day pool location
1975	Aerial Photo is high level, quite poor resolution. Previous SLSC building has been replaced with one somewhat larger, but not as big as the present building. Ongoing clearance landward of Marine Parade and upon middle headland to the north. Apparent additional clearance of vegetation to south of SLSC building. Appears that pool may have been constructed but unclear.	Beach width and state at southern end very similar to that shown in 1957.
1983	Very poor resolution aerial photograph. Cannot make out pool or SLSC building. Appears that significant residential development has occurred landward of Marina Parade.	Beach very wide at southern end.
1986	Clear photograph shows that pool has been built and area to south of SLSC building cleared close to its present state. Residential development has approached present day density. Norfolk Island Pines have established along foreshore.	Beach has reasonable width at southern end. Cannot make out cobbles on beach.
1988	No notable developments since 1986.	Beach is very wide at southern end
1992	No notable developments since 1988	Beach is particularly wide and seems flat in intertidal zone.
1995	SLSC building has been extended 10m towards south to its present-day footprint.	Beach in a very accreted state.
1999	No notable developments since 1995	Beach particularly wide and flat, similar to 1992, not as accreted as 1995.
2005	No notable developments since 1999	Beach has narrowed. Cobbles exposed to north of pool, but sandy beach still extends to landward pool edge.
2007	No notable developments since 2005	Sand has been removed from back beach exposing cobbles adjacent to steep embankment below dining area. Continuous cobbles along landward edge of although there is sand present in the intertidal zone to the north of the pool. Overall, less sand than 2005
2010	No notable developments since 2007	Back beach has recovered in front of SLSC and there appears to be more sand in the intertidal zone. Significant sand present
2012	No notable developments since 2010	Southern end of beach is particularly devoid of sand. Cobbles exposed the full length and width of beach south of SLSC, between SLSC and Pool
2014	No notable developments since 2012	Some recovery of beach, although the cobbles are only covered by a thin veneer of sand just above the high tide mark between the SLSC and the Pool
2015	No notable developments since 2014	Beach particularly devoid of sand above tidal range. Exposed cobbles are present between pool and boat ramp, more exposed than any prior aerial photo (but not as severely eroded as in 1974)

[3] Figures showing the photographs are presented in Appendix B

Patterns emerge from Table 1 but the conclusions drawn need to consider that the latter aerial photographs have far more detail than the earlier photographs and are more frequent. However, photos prior to 2005 did not show evidence of significant cobble exposure.

Sand does migrate northwards and southwards along the Beach, but in recent years it appears that sand is less likely to be present at the southernmost end of the Beach. Indeed, following the June 2016 storm, the beach from the boat ramp at the SLSC southwards was devoid of sand within or above the intertidal zone. At the time of writing (December, 2016), that situation is changing, with sand being transported southwards along the Beach, particularly under the influence of the east to north-east swells that are more common during summer.

A lack of sand exposes the foreshore fronting the SLSC dining area to more ready attack by waves and the present trend appears to be towards more frequent removal of the beach sand. This behaviour is commensurate with an overall recession of this beach compartment as identified by Worley Parsons (2014a).

3.4 Numerical Modelling

The Delft-3d modelling software (Version 3.04.01) was used to model wave propagation towards the proposed revetment site. A curvilinear model mesh was constructed of the area offshore of the NSW Central Coast and Sydney's northern beaches. The coverage of that mesh is illustrated in Figure 11, along with the digital elevation model which was used to set depths in the model. Closer detail of the model mesh near MacMasters / Copacabana Beach is presented in Figure 12

At the grid cell resolution shown in Figure 12, each cell typically covers around 50 to 70m of beach length. The model is expected to provide reasonably indicative nearshore wave heights along the beach with meaningful distinction possible every 200 to 250m along the Beach. At this resolution, the model is not capable of replicating the fine scale forces that would drive the development of rips. Indeed, the bathymetry data used to derive the digital elevation model does not resolve features (bars, gutters etc.) that are necessary for driving beach circulation patterns. An even finer resolution would be desirable. However, the absence of detailed nearshore bathymetric survey makes it difficult to justify model refinement at this stage. The model has been used primarily to estimate wave setup along the Beach during design wave conditions.

Importantly, the model is not calibrated. We have used typical values for roughness and other relevant parameters within the ranges recommended in the model manuals. Instead, the model results have been compared carefully against values estimated during previous stages of the project, and wave model results presented in the recent Coastal Processes and Hazards Study (Worley Parsons, 2014a).

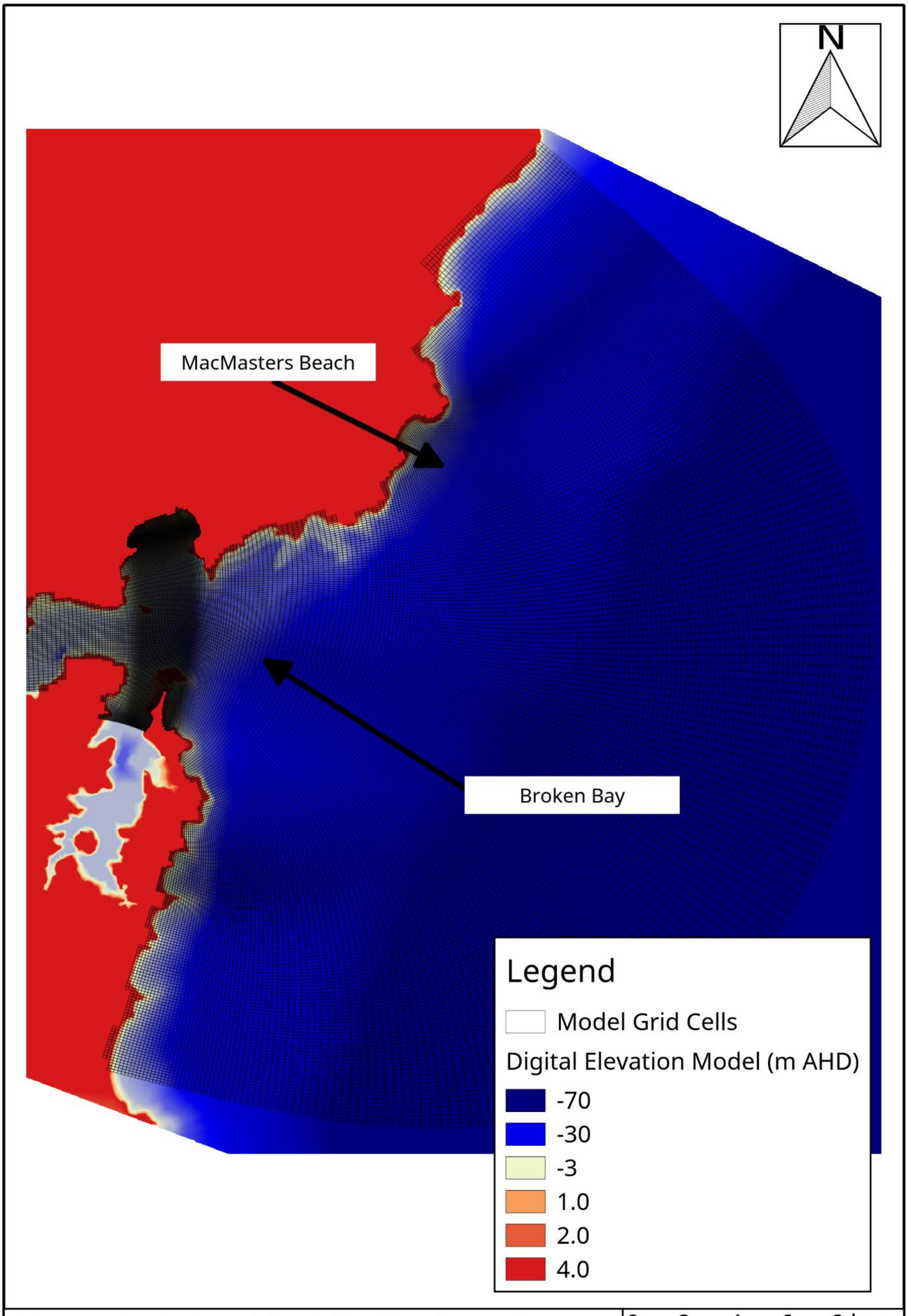

Figure 11: Wave Model Extents and Bathymetry

Conceptual Design of Macmasters Beach Seawall

REV	DRAWN	CHECK
A	DJW	DJW

Salients

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Legend

Model Grid Cells

Figure 12: Wave Model Representation of MacMasters Beach

0 0.1 0.2 0.3 km

APPROX SCALE

Conceptual Design of Macmasters Beach Seawall

REV	DRAWN	CHECK
A	DJW	DJW

Salients

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Model simulations were executed for a range of design offshore waves. Those waves were determined by undertaking an extreme value analysis on the data presented in Section 3.2.4. The waves were grouped according to their direction of approach, using 8 directional bins from north-east clockwise through to south-south-west. A generalised extreme value (block maxima) analysis was undertaken of each individual direction, and all directions combined, resulting in estimates for design waves for different recurrence intervals as presented in Table 2.

Table 2 Design Wave Heights Estimated from Generalised Extreme Value (GEV) Analysis (Hs in m AHD)

Direction	25yr	50yr	100yr
NE	4.0	4.2	4.4
ENE	5.4	5.7	5.8
E	5.9	6.1	6.2
ESE	6.3	6.5	6.7
SE	7.6	7.9	8.1
SSE	8.0	8.2	8.4
S	7.5	7.6	7.7
SSW	5.4	5.6	5.7
All	8.2	9.0	9.9

These values are slightly higher than some previously published values. This would partly be affected by particularly high recently measured significant wave heights (2015-2016) and the method used to determine extreme values for all directions is likely to have differed from that used in other published analyses (i.e. block maxima vs. peak over threshold methods). Regardless, the values are comparable and suitable for design in this instance.

Also of note is the size of the June 2016 peak wave height when compared to the design values for E and ESE directions. The peak measured wave height of 6.52m during that event was around the size of a 100-year recurrence interval wave from those directions.

Considering the nature of the wave climate offshore of Sydney, the highest waves are likely to occur from easterly to southerly directions. Furthermore, future climate change may cause a shift in the dominant storm wave direction and/or intensity offshore of New South Wales. Accordingly, the design values derived using all directions were tested approaching MacMasters Beach from 5 different directions (E, ESE, SE, SSE & S), to determine how much set-up might occur.

Offshore wave heights of 9.0m and 9.9m were tested for 50 and 100 year conditions respectively. A comparison of wave period and height presented in Callaghan et al. (2008) indicates that a wave period of the order of 12.5 – 13.0 seconds could be expected to accompany these wave heights.

In selecting the recurrence intervals to be considered, reference was made to Council's 2013 Development Control Plan which specifies that a 1 in 100-year design storm event should apply to coastal development. That DCP also specifies a design life of 35 years. Considering these parameters, a design "still" water level upon which the wave would propagate was derived using the components shown in Table 3.

Table 3 Components of Design "Still" Water Level

Component	Amount	Description
AHD Adjustment	0.08m	Due to past sea level rise, mean sea level is now around 8cm above AHD offshore of Sydney (Wainwright et al., 2014).
Future Sea Level Rise	0.21m	Based on the adopted projection of Gosford City Council from 2015 to 2051 (35 year design life)
Still Water Level (50yrs/100yrs)	1.40/1.43 m AHD	Based on recent analysis by Manly Hydraulics Laboratory of the Fort Denison Tide Gauge (above mean sea level). Represents astronomical tide and effects such as wind and barometric setup, but not wave set up.
Total	1.69m/1.72m AHD	1 in 50yr / 1 in 100yr still water level. Somewhat conservative as it assumes that the design storm occurs at the end of the design life.

Wave Simulations were executed for a 50yr ARI (1.69m AHD still water level with 9.0m wave) and a 100yr ARI (1.72m AHD SWL with 9.9m wave). Ultimately, the simulations were used to estimate the wave setup component at the shoreline. This is the key wave related consideration which controls the design wave height the shoreline. Waves which reach the shoreline to break on the structure are affected by depth limited wave breaking. A larger value for wave setup enables larger waves to propagate to the foreshore before breaking directly on the structure. Conversely, smaller wave setup results in smaller depths and waves breaking further away from the foreshore.

The modelled wave setup around the shoreline of MacMasters Beach for the condition which gives the worst-case setup scenario near the proposed revetment is shown in Figure 13.

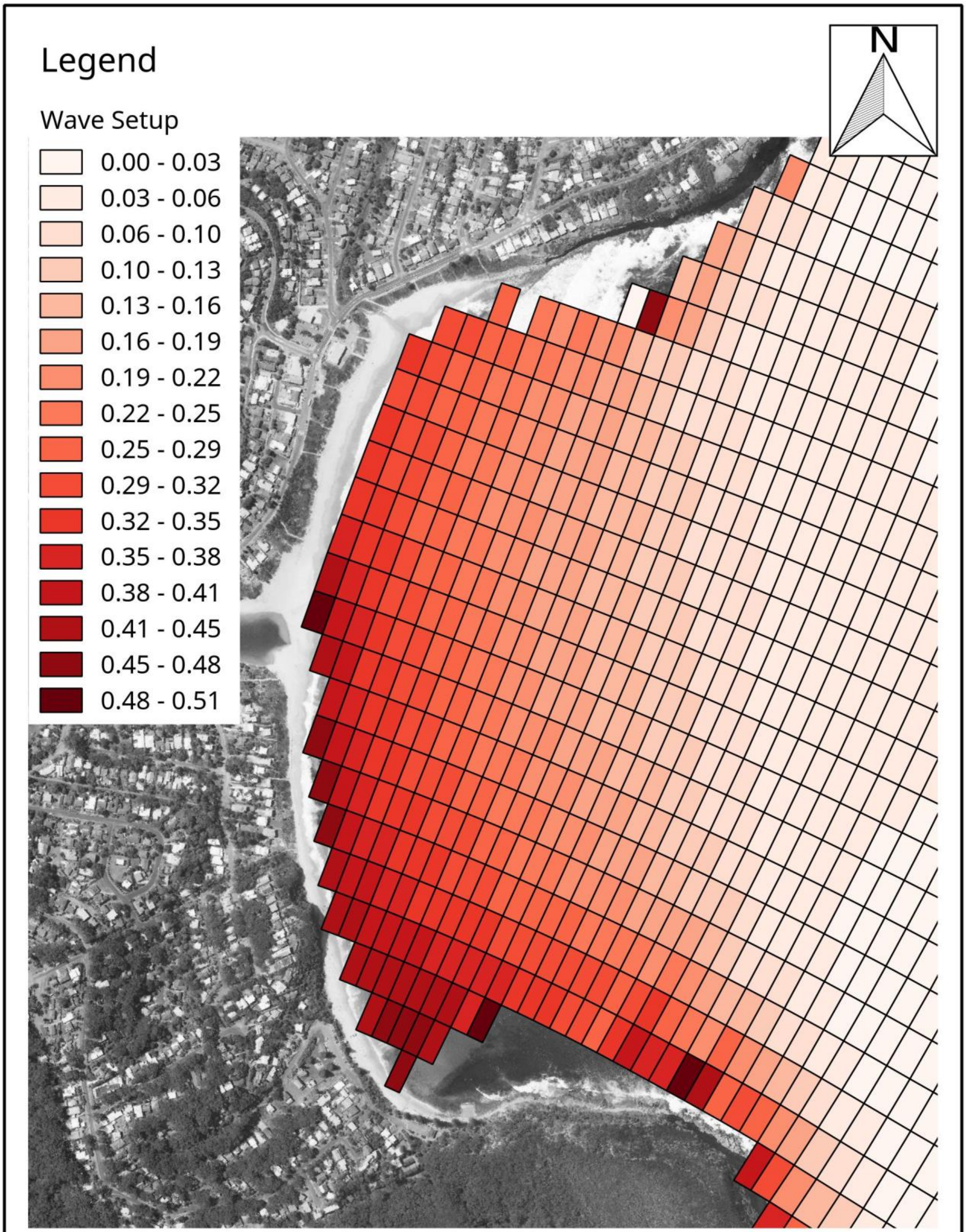


Figure 13: Modelled Worst Case Design Wave Setup - 100yr Wave from East

Conceptual Design of Macmasters Beach Seawall

REV	DRAWN	CHECK
A	DJW	DJW

GIS File:/ZDrive/BusinessShare/Them/Projects/P00016_MacMastersRevetmentConceptDesign/Outgoing/Figures/Figure_12_A.qgs

Salients

For the purpose of checking the model results using an empirical hand calculation, Nielsen (2009) presents the following relationship for setup at the shoreline:

$$\bar{\eta} = \frac{0.4 \times \mathrm{H}_{0rms}}{1 + \left(10 \times \left(\frac{h}{\mathrm{H}_{0rms}}\right)\right)}$$

Herein, the value of H_{0rms} ($\frac{\mathrm{H}_{0s}}{\sqrt{2}} = 7.0m$) is known, but the depth (h) and setup ($\bar{\eta}$) are interrelated and need to be solved iteratively. Assuming that setup is calculated where the bed elevation is 0.0m AHD (still water depth of 1.72m); a setup of 0.64m is calculated. Considering the relative merits of the empirical vs. numerical approaches presented here and a desire to maintain conservatism in the design, a setup of 0.6m was adopted. A rounded water level for calculating depth at the toe of the structure was set at 2.3m AHD.

3.5 Geotechnical Assessment

A geotechnical investigation of subsurface conditions was completed by Pells Consulting and is attached as Appendix C.

Pells Consulting postulated that the cobbles present on the beach have originated from the ocean/cliff interface and subsequently weathered and moved by waves into their present location.

Four test pits along the base of the structure and a borehole adjacent to the SLSC building were excavated.

Based on four test pits excavated along the base of the structure, Pells Consulting found that the cobbles are underlain by a layer of silty sand, like that used to construct the bench upon which the clubhouse is built. The depth of this silty sand was variable. In some instances, bedrock was not encountered before the maximum depth of excavation was reached, whereas at others (i.e. close to the Pool) bedrock was almost immediately beneath the layer of cobbles.

Pells Consulting concluded that the existing cobbles have provided a reasonable historical robustness, even though they are capable of being moved around by significant storms. Accordingly, a toe design which "tucks" under these cobbles was postulated as being reasonable. Salients supports this reasoning and proposes a design where a small structural toe is constructed just below the existing cobble level with the existing cobbles replaced to maintain as much of the existing beach profile as possible. Where bedrock is shallow, that toe may be founded directly on bedrock.

3.6 Summary and Consideration of Impacts on Coastal Processes

MacMasters Beach appears to have been receding in recent decades. Because of this, the southern end of the beach, near the SLSC building and the ocean pool tends to have less sand than it has had historically. From time to time sand will still move back into this area from the north, particularly during summer when waves from the east and north east are more prevalent off the New South Wales coast.

Regardless, the thin layer of sand which comprises the Beach in this area is readily removed by storm action and transported towards the north. Along the southern end of the beach, if sand is present, it is underlain by a layer of smooth cobble/boulder sized rocks which are apparently derived from the nearby rocky headland, platform and nearshore reefs. Those rocks have been moved across the foreshore and continue to be mobile during significant storms.

In turn, these rocks are underlain by a layer of orange silty sand with some clay, possibly of Pleistocene origin. Given this stratigraphy, it appears that the boulders have been moved by the action of waves into this area following the last glacial maximum as sea levels rose to their present level around 6,000 years ago. It also follows that the silty sand layer has been effectively protected from erosion by the boulders for at least a few thousand years. It is reasonable to expect that these boulders will continue to serve this function, and that the overlying beach sand will continue to move backwards and forwards along the beach in this area, intermittently covering and uncovering the boulders, to varying extents, from time to time.

Construction of the revetment in this location is not expected to have a significant impact on the movement of the beach sand. The extent to which any impact would be felt will depend on the degree to which the slope of the revetment extends seaward to intersect the existing cobble/boulder layer. In undertaking design a steeper slope which results in less landward projection of the structure is desirable. However, this is not an overriding concern and needs to be balanced against stability considerations and the relative cost of constructing steeper/flatter slopes.

The degree to which the revetment projects seawards is also of interest in terms of public access to the Beach. The further seaward that the structure projects, the more likely it is that the structure will extend into the intertidal area, noting that the actual location of the intertidal area will vary depending on the presence, or otherwise, and width of a sandy beach at any given time. Again, lesser seaward projection is desirable. However, when the beach is devoid of sand, public foot based access along the foreshore is already difficult, as the public needs to climb over the uneven boulder surface. A much easier way for the public to gain access along the Beach at these times, is to walk up the boat ramp at the SLSC building, across the crest of the foreshore and back down to the Pool using the stairs which have been recently refurbished in this location. Therefore, only small impact on public foreshore access is expected to result from construction of the revetment and, while a reduced seaward projection is preferable, this needs to be balanced against cost and stability implications.

Finally, a function of the revetment will be to halt recession of the foreshore in this location. The revetment will effectively prevent the edge of the artificial platform on which the SLSC building is constructed from being eroded further. When eroded, that sand would potentially be transported elsewhere along the beach as part of normal sand redistribution during and following storm events.

It needs to be recognised that the sand in this location is not part of the naturally occurring Holocene sand barrier, instead seeming to have been placed loosely here during fill operations prior to construction of the present day SLSC building. Based on site investigations and available historical information, we expect that the 'natural' surface of the Beach comprises the layer of cobbles and boulders, extending for some distance underneath and probably beyond the landward edge of the SLSC building. A borehole augered adjacent to the building indicated that this layer is at significant depth (>4.5m in the borehole). Accordingly, during a significant storm, it is unlikely that this end of the beach would have acted as a significant natural source of erodible dune sand to meet storm demand along the beach.

An 'edge effect' resulting from erosion during storms has also been considered. The area immediately to the north of the location where the revetment would be constructed contains a boat ramp and stormwater outlet. The crest of the foreshore here is around 2m lower than in front of the southern end of the SLSC building. That area does not seem to have been as badly affected by erosion during the June, 2016 storm, indicating that the concentrated erosion area, where sand filled 'bulka' bags have been placed, was affected by a rip which could foreseeably have formed naturally along the southern end of the beach during that storm. This rip would be strongest when storm waves approach from more easterly directions. In comparison, when waves approach from a more southerly direction, the area where the revetment is proposed would be protected to a significant degree by the extensive rock platform and walls of the pool to the south. For these reasons, it appears unlikely that edge effects would be significant during and immediately following a storm event.

This apparently low likelihood of edge effects needs to be considered against the alternative, allowing foreshore erosion to continue and eventually undermining the SLSC building, making continued use of that building impractical. It is expected that, on balance, the community services provided by the building will be considered valuable enough to justify protection of the foreshore in the face of this uncertainty.

Ongoing monitoring of the foreshores to the north and south of the revetment would seem prudent following construction, particularly following storms to identify when further assets (car parking, Marine parade) become threatened. This is expected to happen in future as the Beach is receding.

4 Design Conditions

4.1 Water Depth at Structure

The design still water level is taken as that derived by adding the 100yr still water level from Table 3 (1.72m AHD) and adding a 100yr wave setup (~0.6m) as discussed in Section 3.4. A resulting water surface elevation of 2.3m AHD was adopted. Inspecting the nearshore characteristics on aerial photography, it appears that the actual depth to which the bed could scour in the nearshore sits at no higher than -1.0m AHD, noting the presence of a rocky reef offshore of the area of interest.

Common engineering practice is to adopt the depth of water at the toe of the structure in determining the breaking wave height. The ground contours captured by drone survey in June 2016 indicated an elevation at the back of the cobble beach at around 2.5m AHD. If the toe is tucked below the cobbled layer with two layers approximating a thickness of around 1.5m, the design bed elevation before the toe is undermined (albeit unlikely with the protective cobble layer) was assumed at 0.5m AHD. Considering the design water level of 2.3m AHD, the depth at the toe of the structure was set at 1.8m.

4.2 Breaking Wave Height

A first pass estimate of breaking wave height would arise from applying a breaker index of 0.78, resulting in an incident wave height of 1.4m. However, a variety of research has demonstrated that the breaking wave height is related to the slope of the beach as well. Again, the elevation of the rocky reef at around -1.0m was assumed from aerial photography and measured to be around 75m offshore of the toe of the revetment. The slope of the eroded beach was, accordingly, established at 1 in 50.

An iterative process for determining breaking wave height with consideration of slope is attributed to Weggel (1972) and subsequently adopted in a variety of design manuals (CERC, 1984; US Army Corps of Engineers, 2014).

That process was adopted, resulting in a breaking wave height of 1.64m.

Examining the nature of waves breaking across the rocky reef in aerial photographs, it appears that the platform seaward of the pool is somewhat higher than that offshore (eastward) of the northern end of the SLSC building. Adjusting the beach slope accordingly (Flatter than 1 in 100), a breaking wave height of 1.50m results.

4.3 Armourstone Size

Recent experience with stone sourced for protection of the repaired stairs at the southern end of MacMasters Beach has shown that Hawkesbury Sandstone with a density of 2.6t/m^3 can be sourced locally. This density has been assumed for design.

Using relationships recommended by CIRIA (2007), the primary armourstone size options for various slopes were calculated. These relationships have assumed damage of between 5-10% of armour units for the design storm event and are presented in Table 4. Eight (8) conditions are presented, comprising four different revetment slopes (1 in 1.0, 1 in 1.5, 1 in 2.0 and 1 in 3.0) and two different still water levels (1.64m AHD and 1.50m AHD).

Table 4 Design Armourstone Sizing Options

Condition	1	2	3	4	5	6	7	8
H_s	1.64	1.64	1.64	1.64	1.5	1.5	1.5	1.5
Slope (1 in X)	1	1.5	2	3	1	1.5	2	3
D_{n50}[4] (m)	0.94	0.82	0.75	0.65	0.86	0.75	0.68	0.60
M_{50} (tonnes)	2.17	1.45	1.09	0.72	1.66	1.11	0.83	0.55

4.4 Run-up and Overtopping

Worley Parsons (2014a) indicated a design run up elevation of 6.0m AHD for the southern end of MacMasters Beach and, based on experience during the 1974 storm, it is only the lower levels of the SLSC building that would be affected by run up during a design storm. The platform on which the building is constructed is at around 7.0m AHD which would allow for up to around 1m of sea level rise before becoming problematic. It presently appears highly unlikely that this much sea level rise would occur during the 35-year design period of the structure.

If the structure were to be designed for overtopping, it would be customary to assess the risk to the public and, to potentially modify the design of the revetment crest to accommodate / mitigate against any damage landward of the crest. In this instance, the risk would appear minimal and no further consideration of run-up is considered as part of the present design. A crest width of a single armour stone is appropriate.

4.5 Summary

A summary of the adopted design conditions is presented in Table 5

[4] D_{n50} is the side length of an 'equivalent' cube.

Table 5 Components of Design Water Level and Wave Height

Component	Value	Description
Design Life	35 years	Gosford City Council DCP, Section 6.2
Design Event	1 in 100 years	Gosford City Council DCP, Section 6.2
AHD Adjustment	0.08	Due to past sea level rise, mean sea level is now around 8cm above AHD offshore of Sydney (Wainwright et al., 2014).
Future Sea Level Rise	0.21m	Based on the adopted projection of Gosford City Council from 2015 to 2051 (35-year design life)
Design Historical Still Water Level	1.43m AHD	Based on recent analysis by Manly Hydraulics Laboratory of the Fort Denison Tide Gauge (above mean sea level). Represents astronomical tide and effects such as wind and barometric setup, but not wave set up.
Adjusted Still Water Level	1.72m AHD	Includes Sea Level Rise allowance and AHD Adjustment.
Design Offshore Wave Height (H_s)	9.9m	Extreme value analysis of Sydney Wave Record
Wave Set up	0.60m	Assessed using numerical modelling and comparing against equation of Nielsen (2009).**.**
Depth at Toe of Structure	1.80m	SWL + Wave Setup, minus base of likely toe elevation scour (~0.5m AHD)
Design Breaking Wave Height	1.64/1.50	Based on eroded beach slope of 1 in 50 and data of Weggel (1972). 1.64 for less protected, northern end of revetment. 1.50 for southern end.

5 Design, Rationale and Cost Estimates

5.1 Design Rationale

Figure 14 illustrates the area of concern and contains the following features:

- An alongshore chainage line, used in the descriptions that follow.
- A series of representative cross-section locations, for which conceptual design profiles are provided in the following pages. On those profiles, offsets are measured from the chainage line. In the design profiles, the landward edge of the revetment crest is assumed to be just seaward of the fence line at the foreshore. This fence line is typically offset some 5-10m from the chainage line and comprises a mixture of glazed panels and tubular pool type fencing (in front of the SLSC building) and standard post and wire beach fencing (towards the south).
- Assuming the landward edge of the revetment crest as described above, the approximate location where revetments of varying slopes would intersect the beach has been plotted. These were derived using QGIS and the digital elevation models developed as part of this study and are provided for revetment slopes of 1 in 1.0, 1 in 1.5, 1 in 2.0 and 1 in 3.0.

For chainages of greater than around 50.0m, the intersection locations for a 1 in 1.0 and 1 in 1.5 sloped revetment have been truncated in Figure 14. In those locations, revetment this steep with crests set back against the fence line would be steeper than the existing foreshore. Accordingly, it has been assumed that a revetment slope of at least 2.0m would be adopted between 60.0m to the southern end of the revetment.

For chainages between 0.0 and 50.0, it has been assumed that a slope of 1 in 1.0 is more likely to have issues with stability. However, slopes of 1 in 2.0, and 1 in 3.0 particularly are encroaching on the width of dry trafficable beach, particularly at high tide, which is of some concern to public access. It appears that an optimal solution here would comprise a slope of around 1 in 1.5.

In terms of staging, the area fronting the SLSC building (Chainages 0.0m to 50.0m) is the foreshore length of most concern. This area also experienced the most severe erosion during the June 2016 storm (i.e. between ~ 25.0 and 40.0m). It is recommended that the length of foreshore between 0.0 and 50.0m be set as the priority for protection. The works can be completed in a staged manner, with the priority area protected first, although it is recommended that the foreshore to the south be monitored to ensure that no edge effects arise.

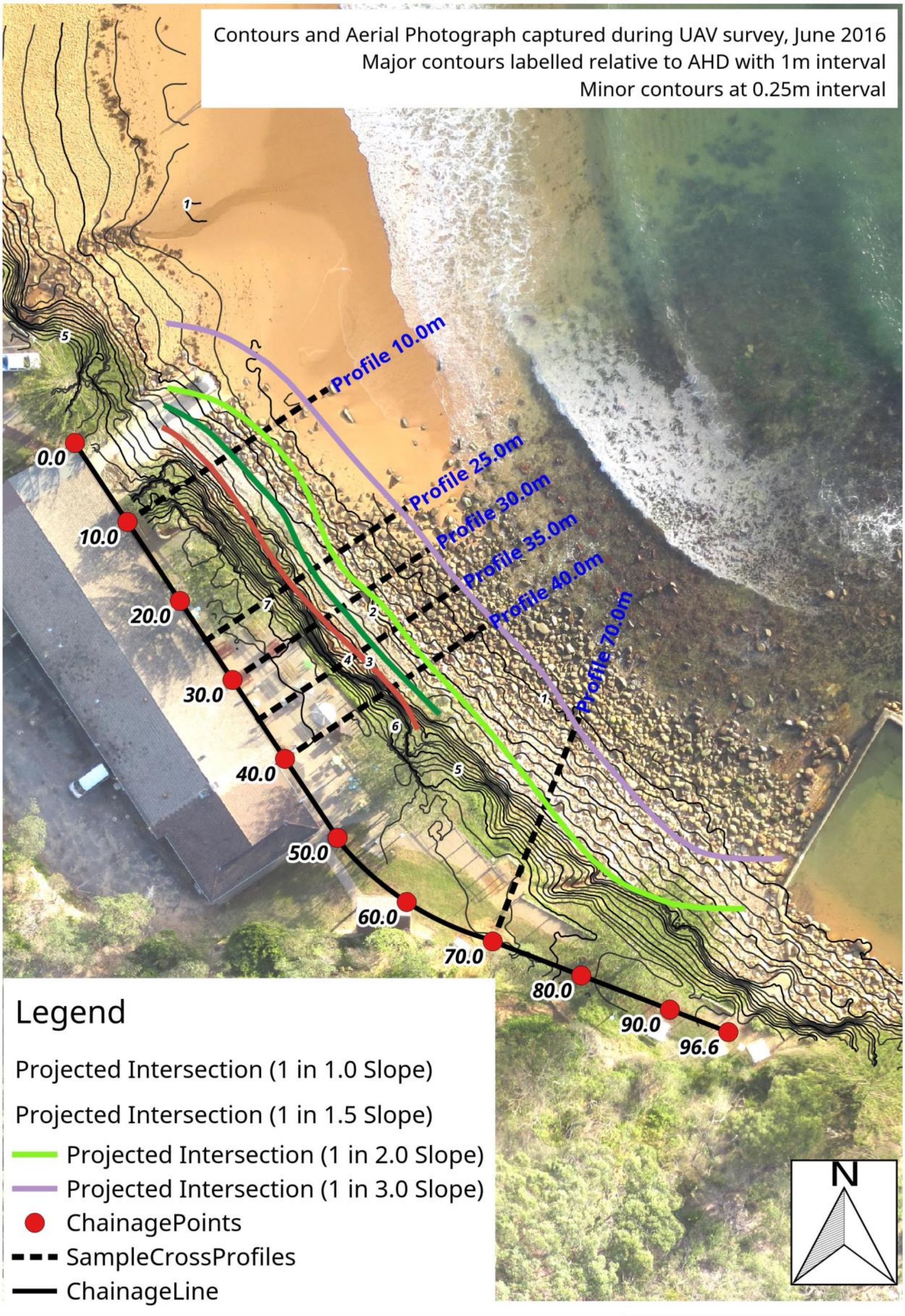


Figure 14: Design Layout Considerations

Conceptual Design of MacMasters Beach Seawall	REV B	DRAWN DJW	CHECK DJW	Salients

GIS File: /Projects/P00016_MacMastersRevetmentConceptDesign/Outgoing/Figures/Figure_14_B.qgs

Finally, some consideration is required as to whether the Norfolk Island Pines existing at the foreshore should be removed. It is possible that they are regarded as having some heritage value, although Figure 4 indicates the presence of only one relatively juvenile specimen in 1974. Presumably the pines along the foreshore are younger than 40 years old. If the trees are to be retained for heritage or other reasons, their incorporation into the foreshore revetment would need to be considered during detailed design. For conceptual design and cost estimation, it has been assumed that these trees would be removed. The cost of retaining the trees may be slightly more expensive, but not significantly so and it is recommended that discussion with the community and a suitably qualified arborist be obtained before finalising detailed design.

Following discussions with Council, and geotechnical stability analyses (Appendix D), a preferred conceptual design has been selected. Example design cross section profiles are provided in Figure 15 and Figure 16 Between 0.0 and 50.0m, the design adopts a 1 in 1.7 slope (1.3 tonnes primary armour) with a transition to a 1 in 2.0 slope by 60m (0.8 tonne primary armour). The flattened slope and reduced armour size is assumed along the foreshore to the stairs behind the pool. The extent of the preferred option is shown in Figure 17.

5.2 Construction Sequence and Preliminary Cost Estimate

For preparing a preliminary cost estimate, the following construction sequence has been assumed:

1 Site Establishment

2 Remove and stockpile landscaping works, if required, with re-instatement following completion. Alternatively, depending on the related costs and the capability of the selected contractor, this may be avoidable with all works occurring landward of the fenced paving area.

3 Manage Trees. The cost estimate, assumes that the trees will be removed from the site and disposed. Should an arborists report indicate that they may be saved, the final design of the revetment will incorporation retention of those trees.

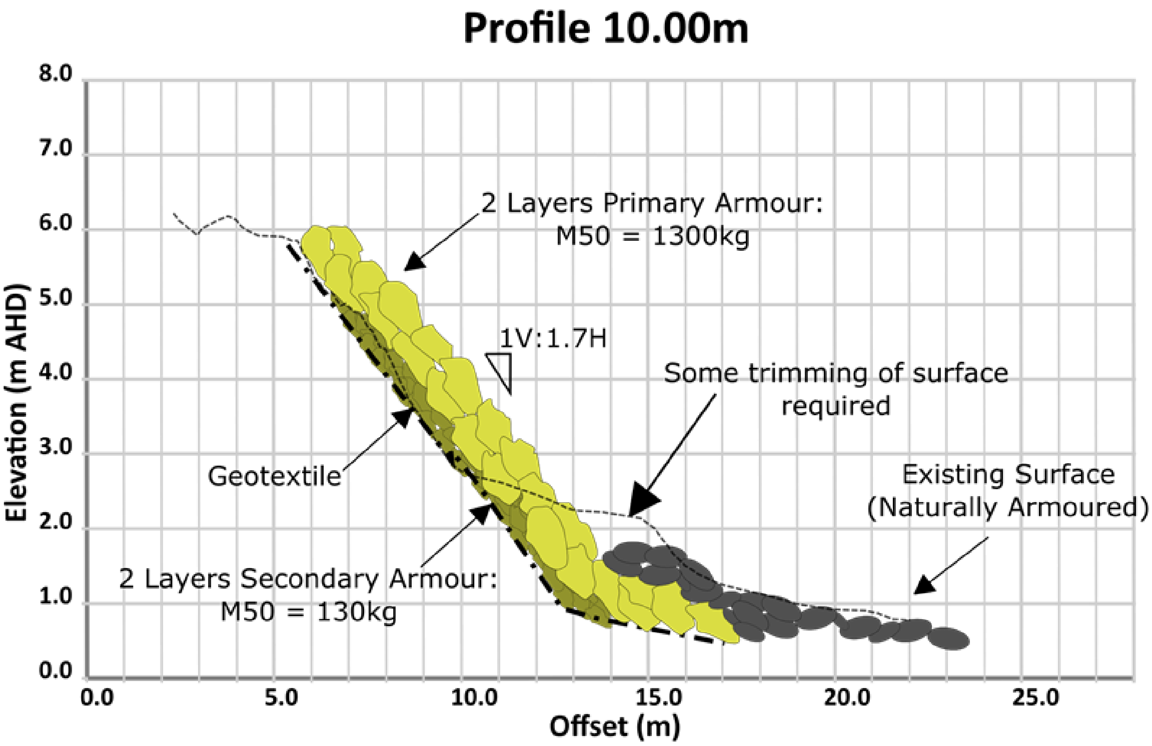
4 Prepare Slope. Herein, it is assumed that there would be an (approximate) cut and fill balance across the site.

5 Place Geotextile. A specialised coastal/marine geotextile is assumed.

6 Import and Place Secondary Armour (Filter)

7 Import and Place Primary Armour

8 Site Disestablishment Reinstate Paving Area and Fence using stockpile materials.



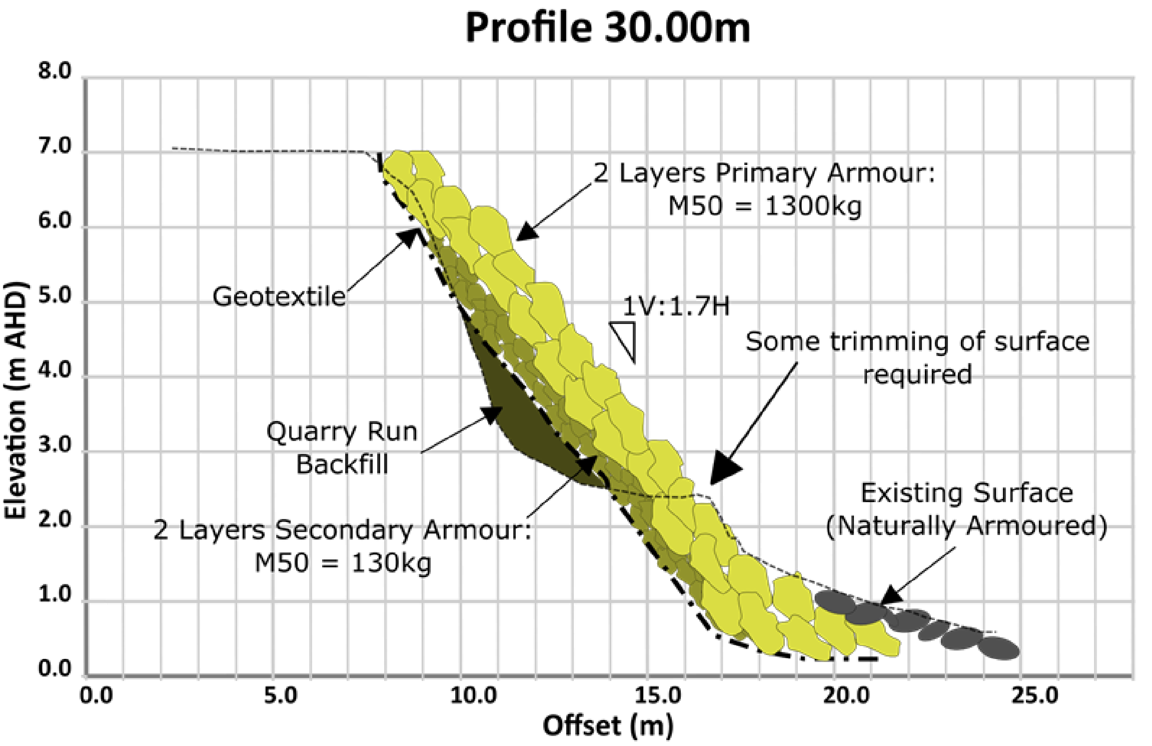


Figure 15: Design Profiles 10.00m and 30.00m	**Do Not Scale. Approx. 2 times Vertical Exaggeration.**			
Conceptual Design of MacMasters Beach Seawall	REV B	DRAWN DJW	CHECK DJW	Salients
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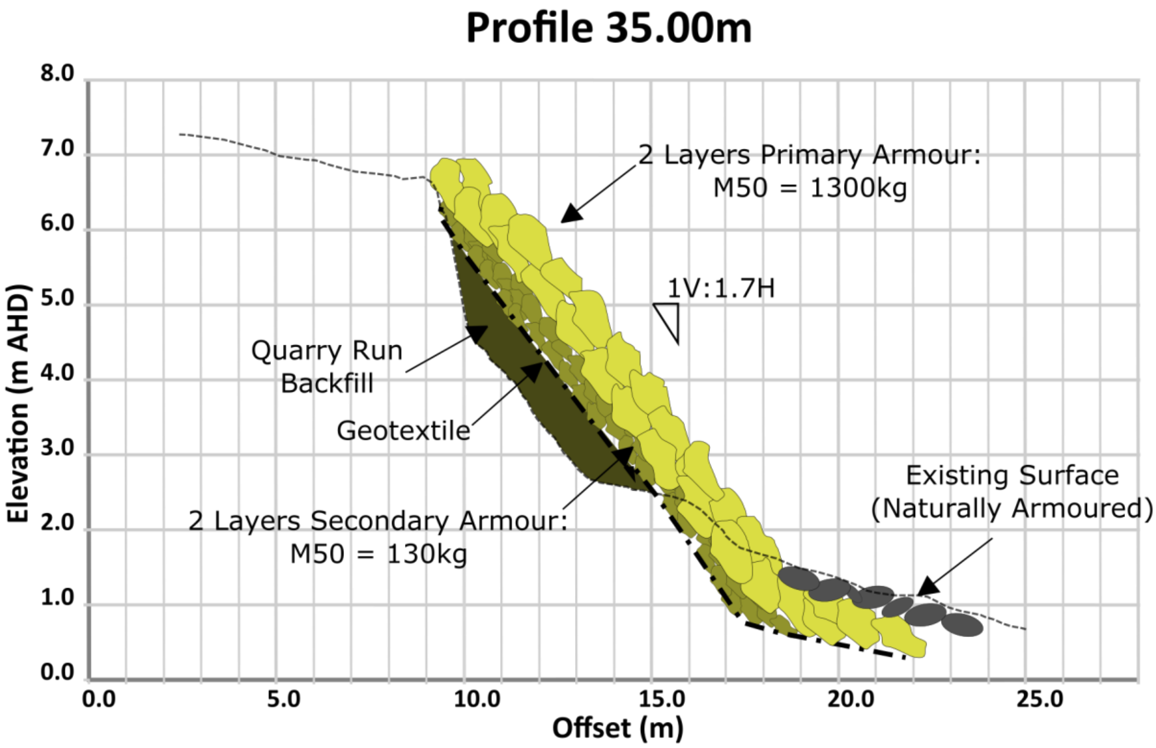

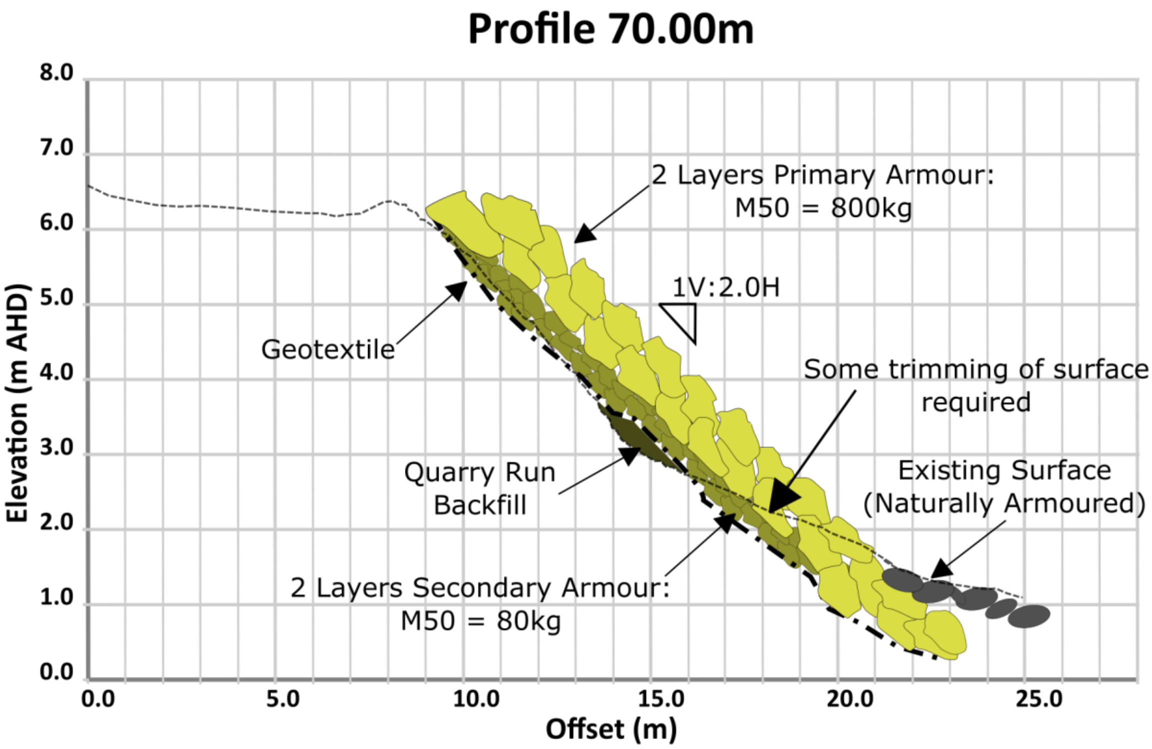

Figure 16: Design Profiles 35.00m and 70.00m	**Do Not Scale. Approx. 2 times Vertical Exaggeration.**			
Conceptual Design of MacMasters Beach Seawall	REV B	DRAWN DJW	CHECK DJW	Salients
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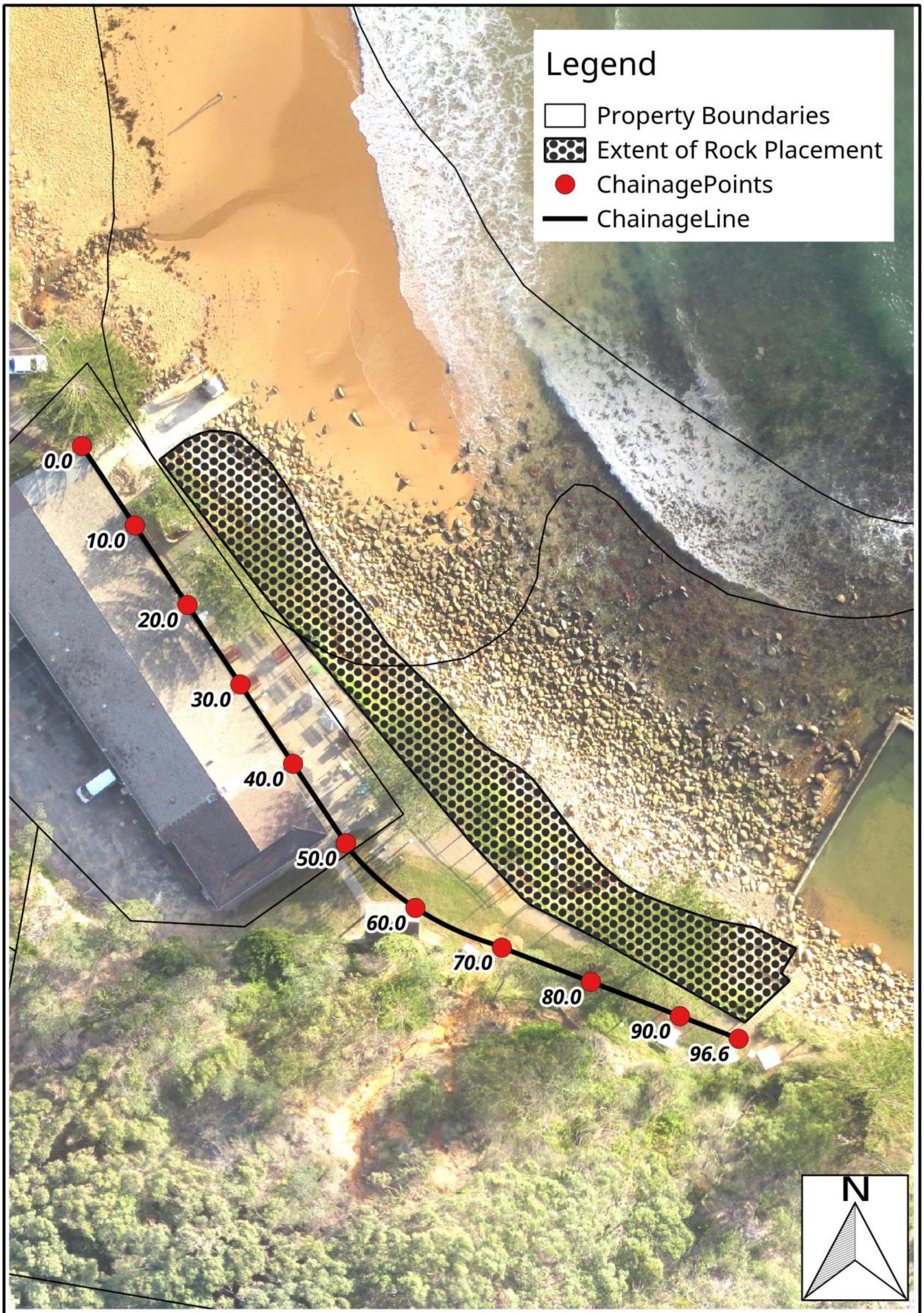

Figure 17: Plan Extent of Preferred Option

0 5 10 15 20 m

APPROX SCALE

Conceptual Design of MacMasters Beach Seawall

REV	DRAWN	CHECK
B	DJW	DJW

Salients

GIS File: /Projects/P00016_MacMastersRevetmentConceptDesign/Outgoing/Figures/Figure_17_B.qgs

A corresponding preliminary cost estimate is presented in Table 6. While the ultimate cost is for the entire revetment, it is split into two sections: Chainage 0.00 to 50.00, and 50.00 to 90.00; assuming that Chainage 0.00 to 50.00 is likely to be constructed first.

The cost estimate needs to be treated as indicative only. The estimate does not include the cost of overheads such as design fees, environmental assessment or project management. At the time of construction, particularly with smaller projects that use local contractors, the cost of construction may be significantly influenced by contractor availability and other market forces. Future inflation will also impact the ultimate cost of construction.

Table 6 Preliminary Cost Estimate

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, install sediment curtain/environmental controls. Establish Stockpile	1	Item	5000	5000	
1.2	Remove and clean up at end of work	1	Item	5000	5000	
Subtotal						$10,000.00
2	**Remove and Stockpile Paving and fence**					
2.1	Pull up Paving	300	sq.m	30	9000	
2.2	Remove Fence	100	m	30	3000	
Subtotal						$12,000.00
3	**Remove Trees**					
3.1	Remove Trees	4	Trees	3000	12000	
Subtotal						$12,000.00
4	**Prepare Slope**					
4.1	Trim Slope	600	sq.m	10	6000	
Subtotal						$6,000.00
5	**Revetment Construction: Chainages 0.00-50.00**					
5.1	Geotextile	370	sq.m	20	7400	
5.2	Acquire and Place Secondary Armour	440	T	137.5	60500	
5.3	Acquire and Place Primary Armour	950	T	150	142500	
	Revetment Construction: Chainages 50.00-90.00					
5.4	Geotextile	310	sq.m	20	6200	
5.5	Acquire and Place Secondary Armour	310	T	125	38750	
5.6	Acquire and Place Primary Armour	670	T	137.5	92125	
Subtotal						$347,475.00
6	**Reinstate Fencing and Paving**					
6.1	Reinstate Fencing	100	m	125	12500	
6.2	Reinstate Paving	300	sq.m	75	22500	
Subtotal						$35,000.00
Intermediate Total						**$422,475.00**
Add Inflation (To End 2016 3.5%)						$437,261.63
Contingency (Conceptual Stage, 30%)						$568,440.11
Grand Total						**$570,000.00**

Note 1: The final design is likely to retain the trees, providing an arborists report indicates that this is possible. If the trees are not removed, It is likely that a commensurate expense will be required for special measures to protect and retain the trees

The intermediate total in Table 6 is based on published rates from Rawlinsons *Australian Construction Handbook* from 2016, experience and rates for rockwork acquired from a local quarry. The level of inflation is based on Rawlinson's Quarterly update to the handbook from October, 2016. The resulting estimate is current for construction at the end of 2016.

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Worley Parsons, 2010. Tidal Modelling of Lake Macquarie.

Appendix A Photographs of the Existing Foreshore and Surrounds

Figure A1 Panoramic Capture of Foreshore, 30th November, 2016
(Left most images are of the southernmost end of the foreshore of interest)

Figure A2 Panoramic Capture of Foreshore, 30th November, 2016
(Right most image is of the northernmost end of the foreshore of interest)

Appendix B Historical Aerial Photography

Figure B1: 1954	0 25 50 75 100 m APPROX SCALE			
Conceptual Design of Macmasters Beach Seawall	REV A	DRAWN DJW	CHECK DJW	Salients
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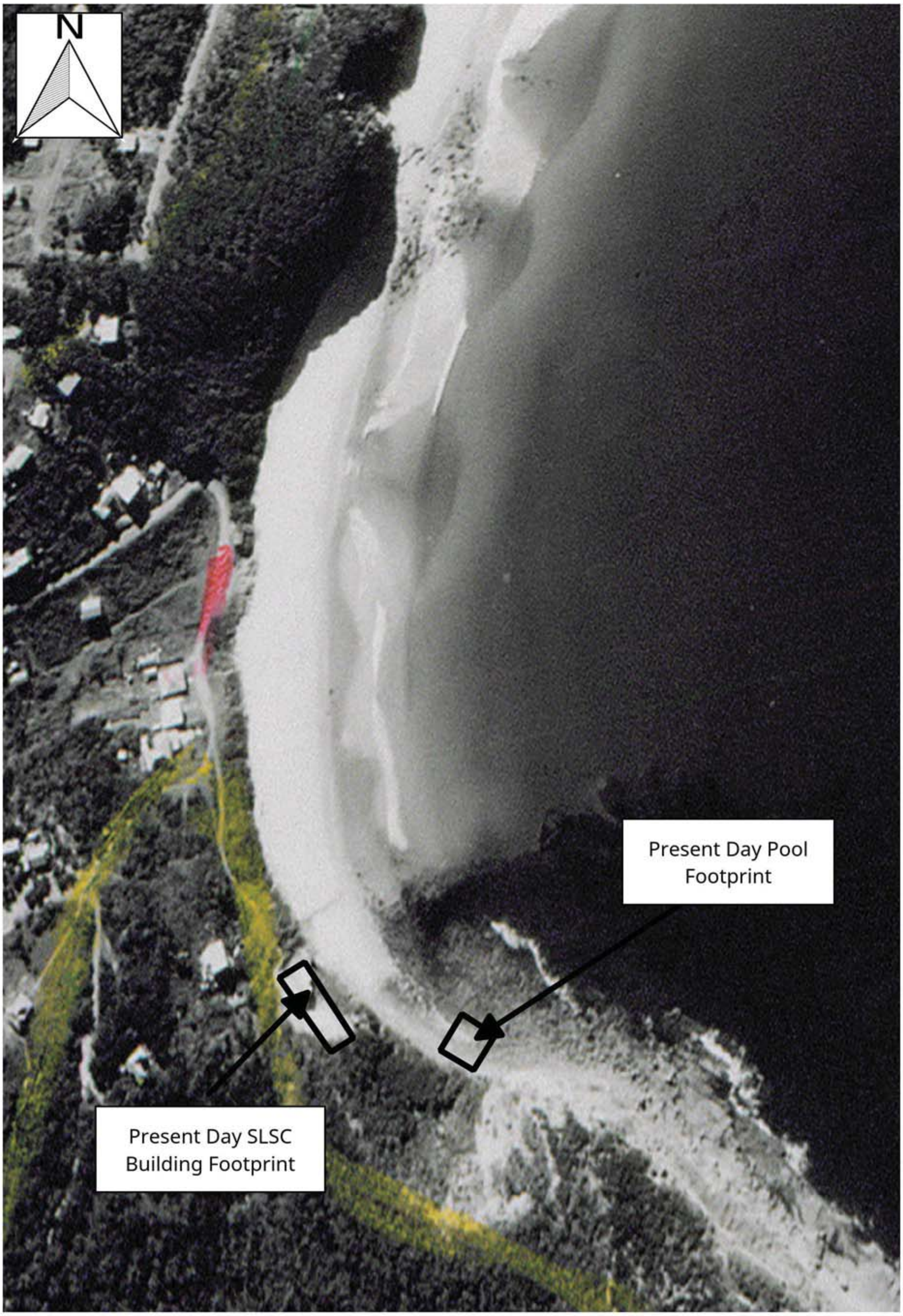


Figure B2: 1957

0 25 50 75 100 m

APPROX SCALE

Conceptual Design of Macmasters Beach Seawall

REV	DRAWN	CHECK
A	DJW	DJW

Salients

GIS File: /Them/Projects/P00016_MacMastersRevetmentConceptDesign/GIS/HistoricalAerials/MacMasterssouth_AppendixBPhotos.qgs

0 25 50 75 100 m

APPROX SCALE

Figure B3: 1975

Conceptual Design of Macmasters Beach Seawall

REV	DRAWN	CHECK
A	DJW	DJW

Salients

GIS File: /Them/Projects/P00016_MacMastersRevetmentConceptDesign/GIS/HistoricalAerials/MacMasterssouth_AppendixBPhotos.qgs

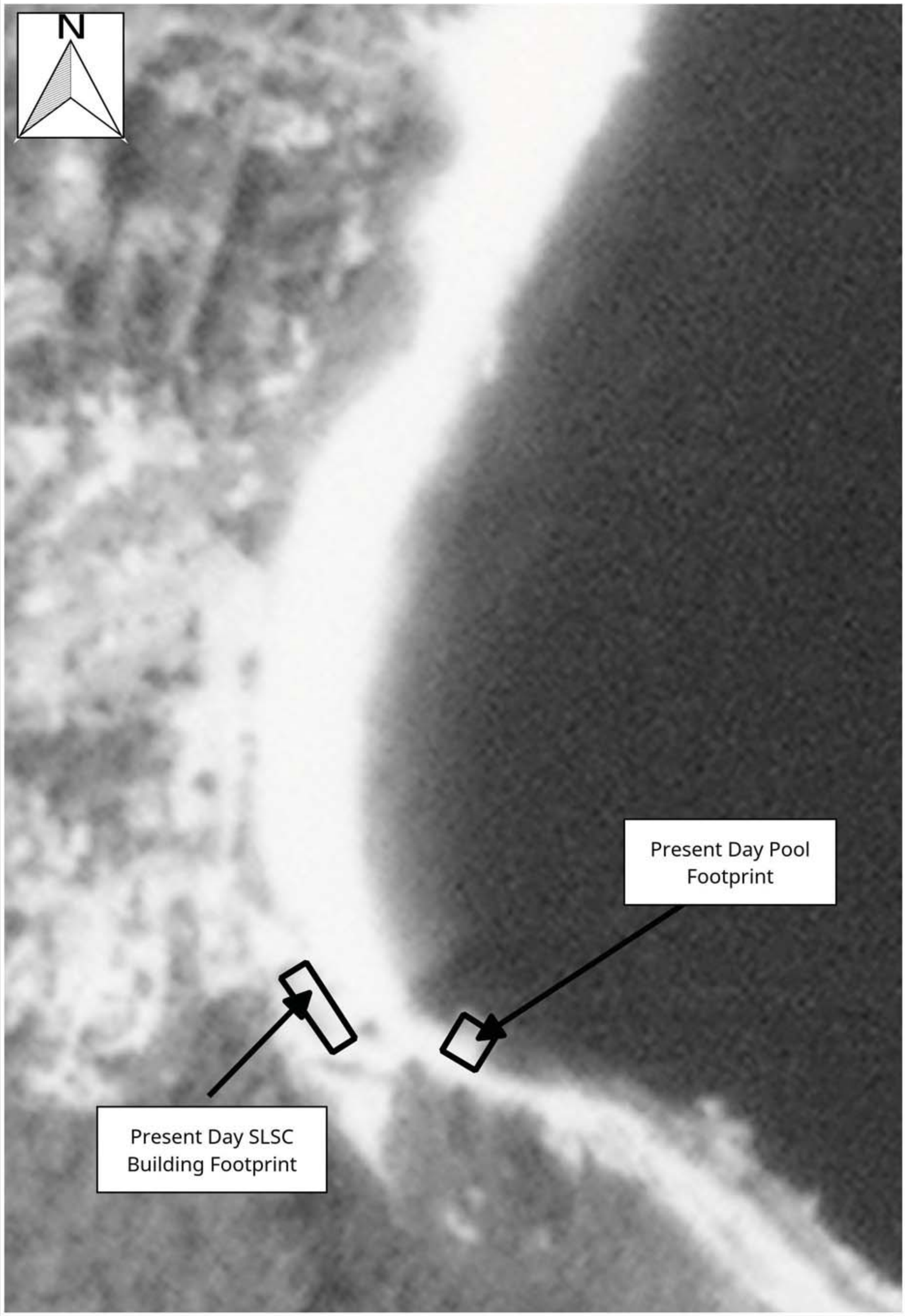


Figure B4: 1983

Conceptual Design of Macmasters Beach Seawall

REV	DRAWN	CHECK
A	DJW	DJW

Salients

GIS File: /Them/Projects/P00016_MacMastersRevetmentConceptDesign/GIS/HistoricalAerials/MacMasterssouth_AppendixBPhotos.qgs

0 25 50 75 100 m
APPROX SCALE

Figure B5: 1986

Conceptual Design of Macmasters Beach Seawall

REV	DRAWN	CHECK
A	DJW	DJW

Salients

GIS File: /Them/Projects/P00016_MacMastersRevetmentConceptDesign/GIS/HistoricalAerials/MacMasterssouth_AppendixBPhotos.qgs

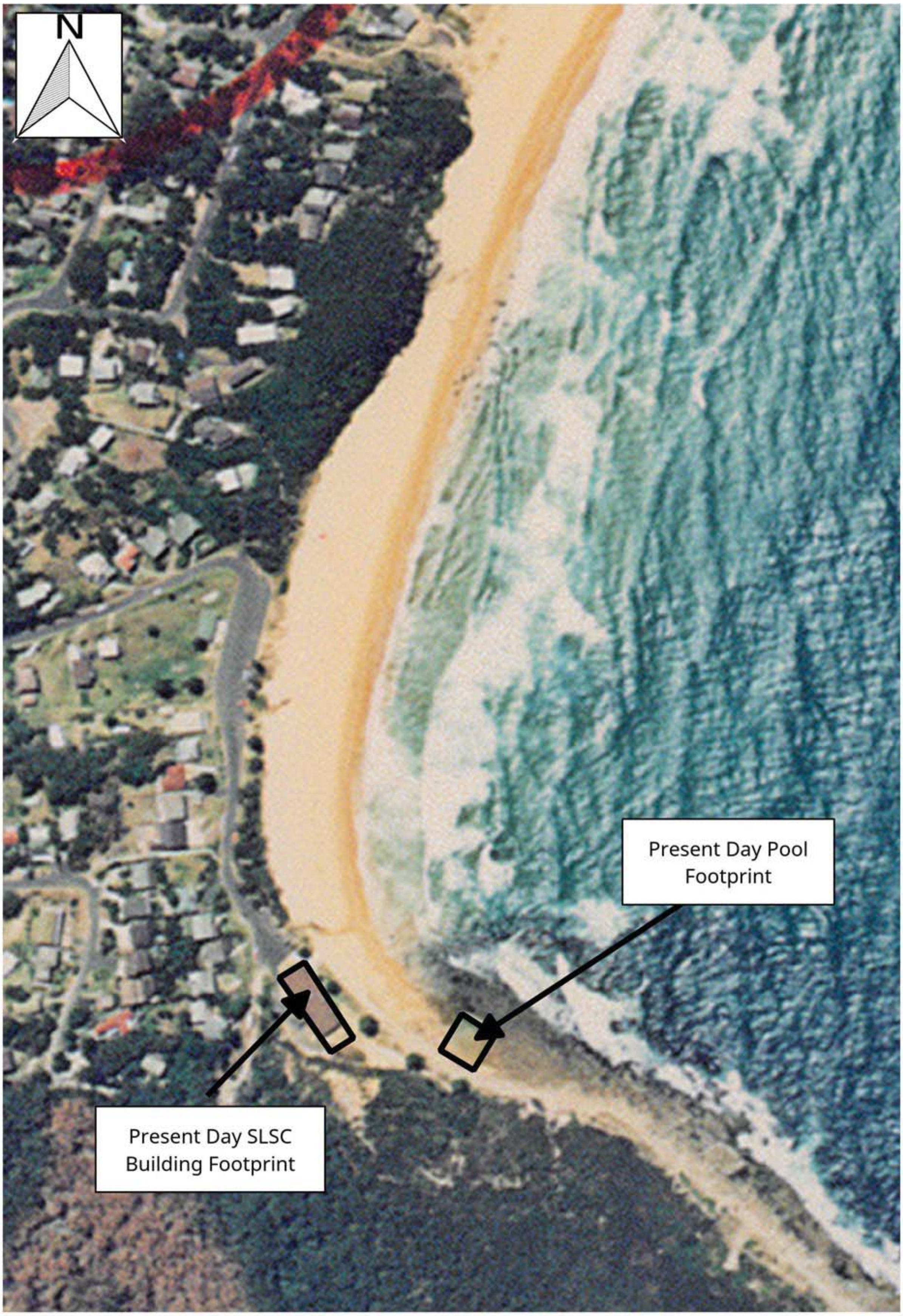


Figure B6: 1988

Conceptual Design of Macmasters Beach Seawall

REV	DRAWN	CHECK
A	DJW	DJW

Salients

GIS File: /Them/Projects/P00016_MacMastersRevetmentConceptDesign/GIS/HistoricalAerials/MacMasterssouth_AppendixBPhotos.qgs

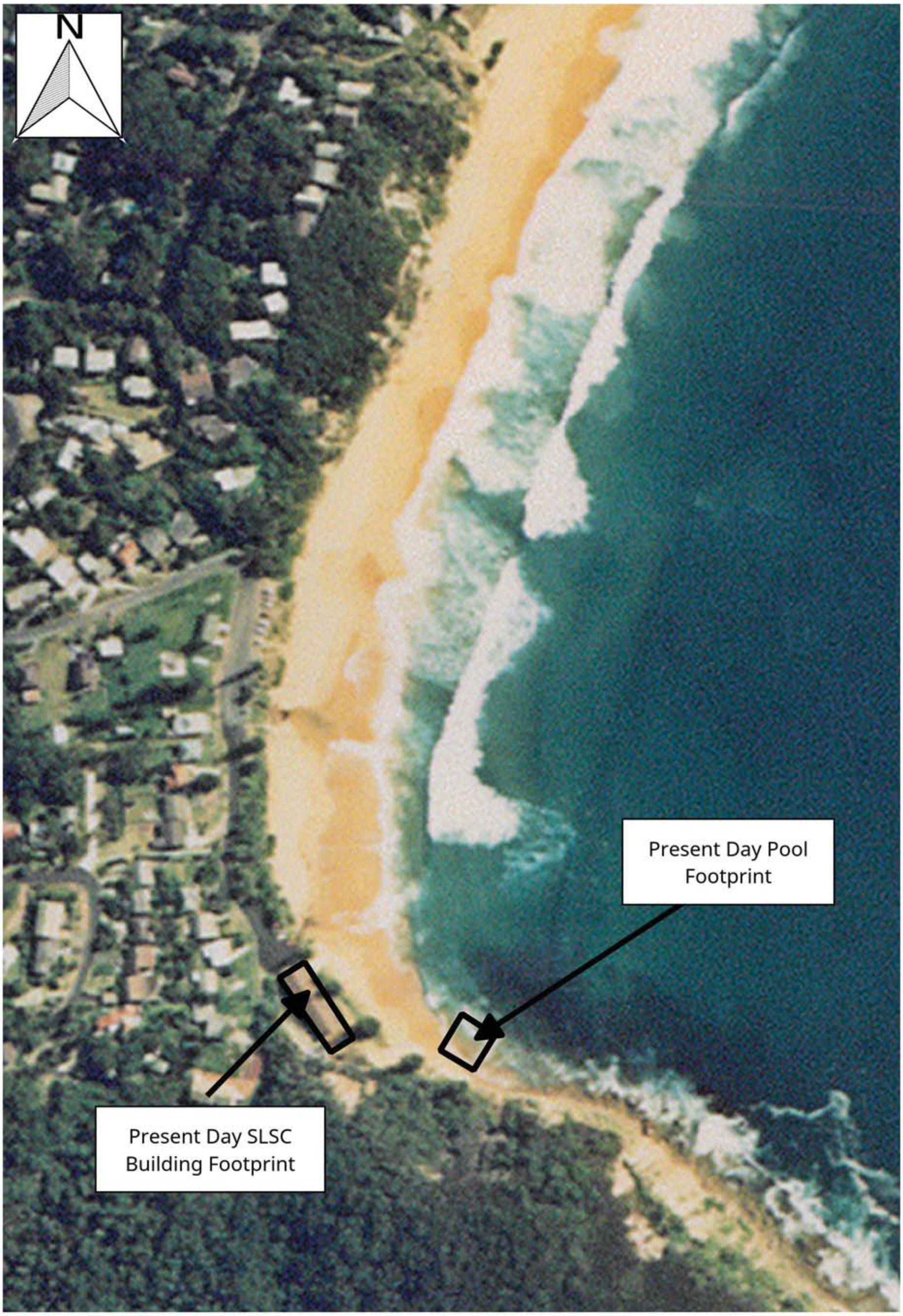


Figure B7: 1992

Conceptual Design of Macmasters Beach Seawall

REV	DRAWN	CHECK
A	DJW	DJW

Salients

GIS File: /Them/Projects/P00016_MacMastersRevetmentConceptDesign/GIS/HistoricalAerials/MacMasterssouth_AppendixBPhotos.qgs

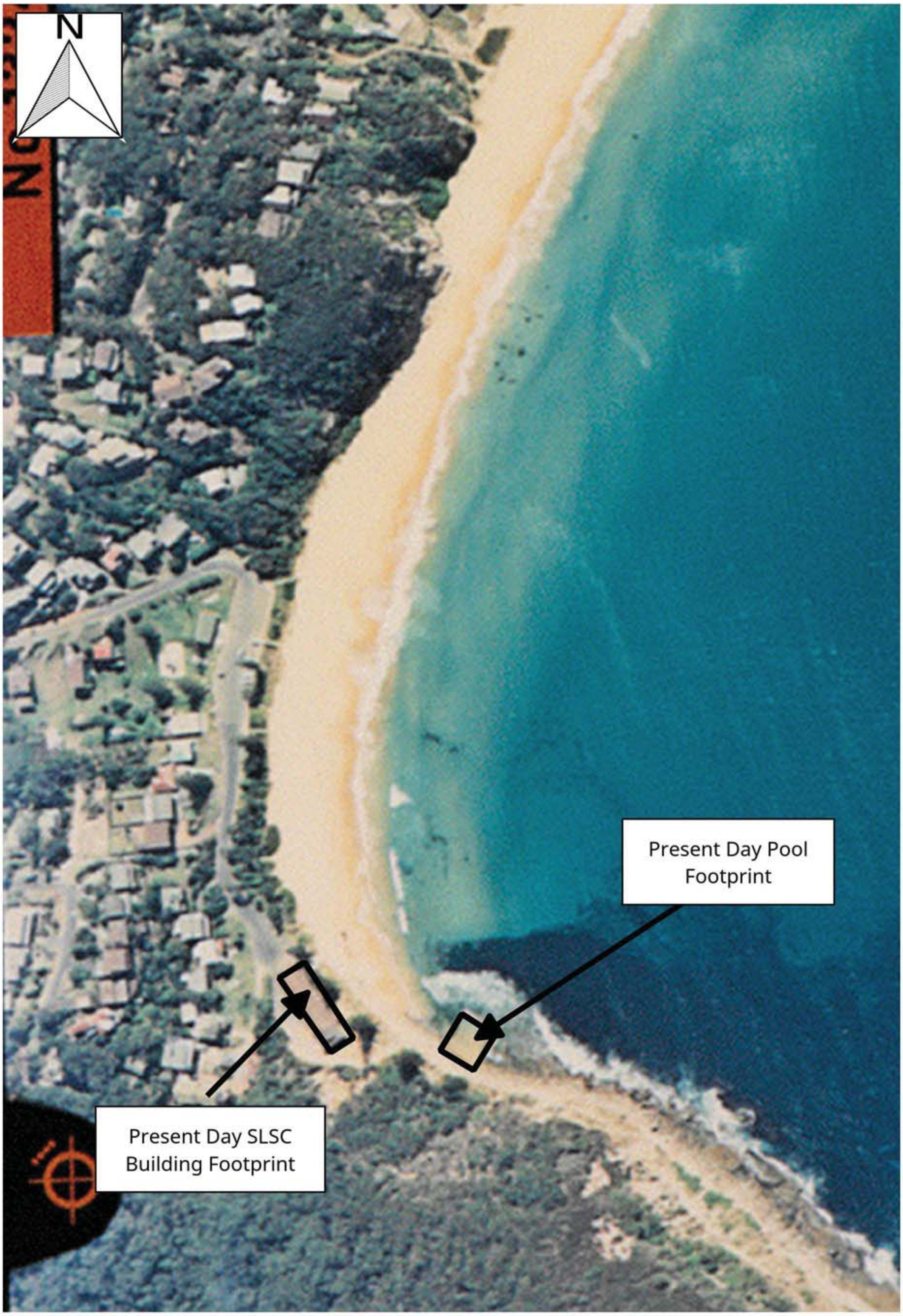


Figure B8: 1995

0 25 50 75 100 m
APPROX SCALE

Conceptual Design of Macmasters Beach Seawall

REV	DRAWN	CHECK
A	DJW	DJW

Salients

GIS File: /Them/Projects/P00016_MacMastersRevetmentConceptDesign/GIS/HistoricalAerials/MacMasterssouth_AppendixBPhotos.qgs

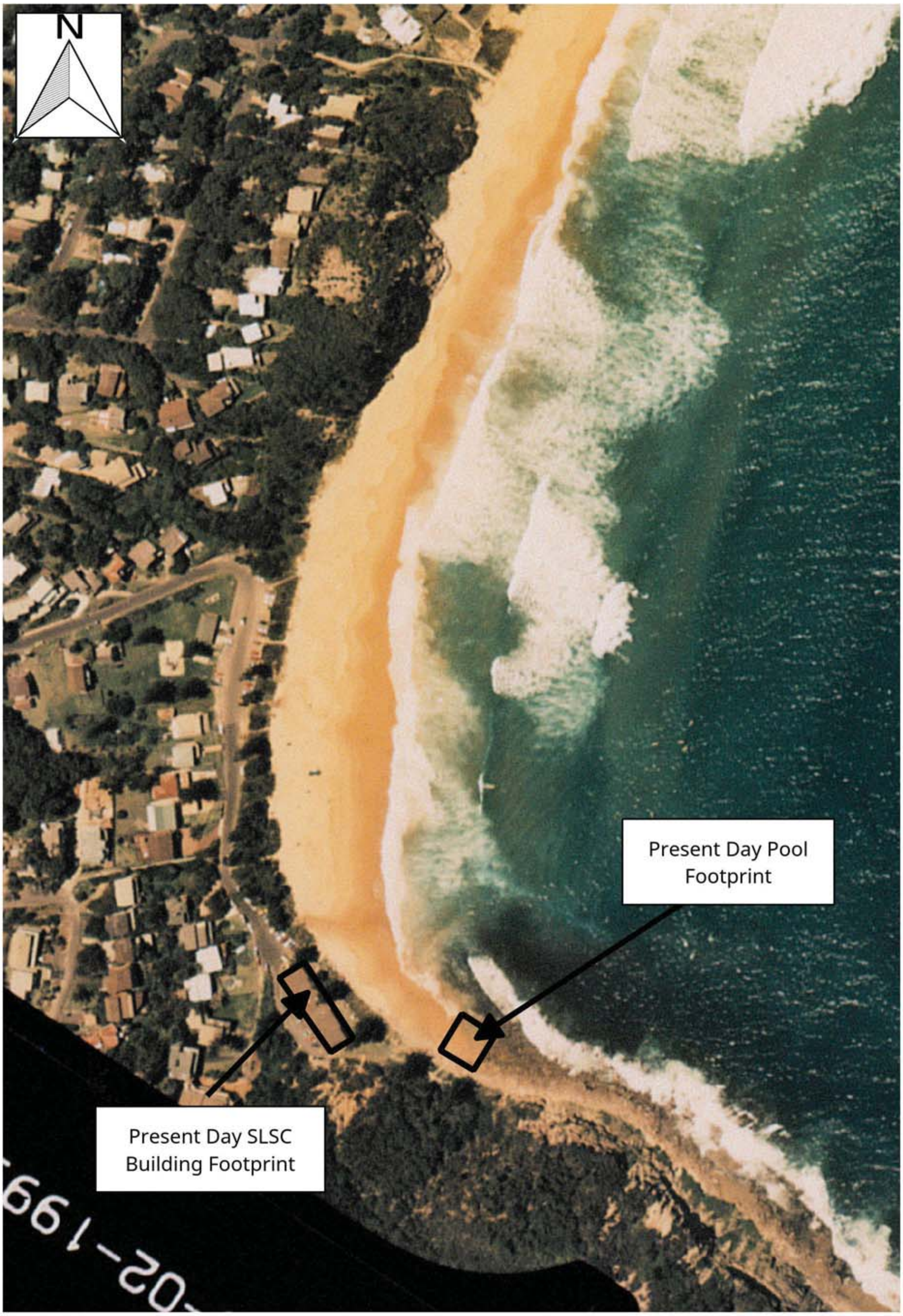


Figure B9: 1999

0 25 50 75 100 m

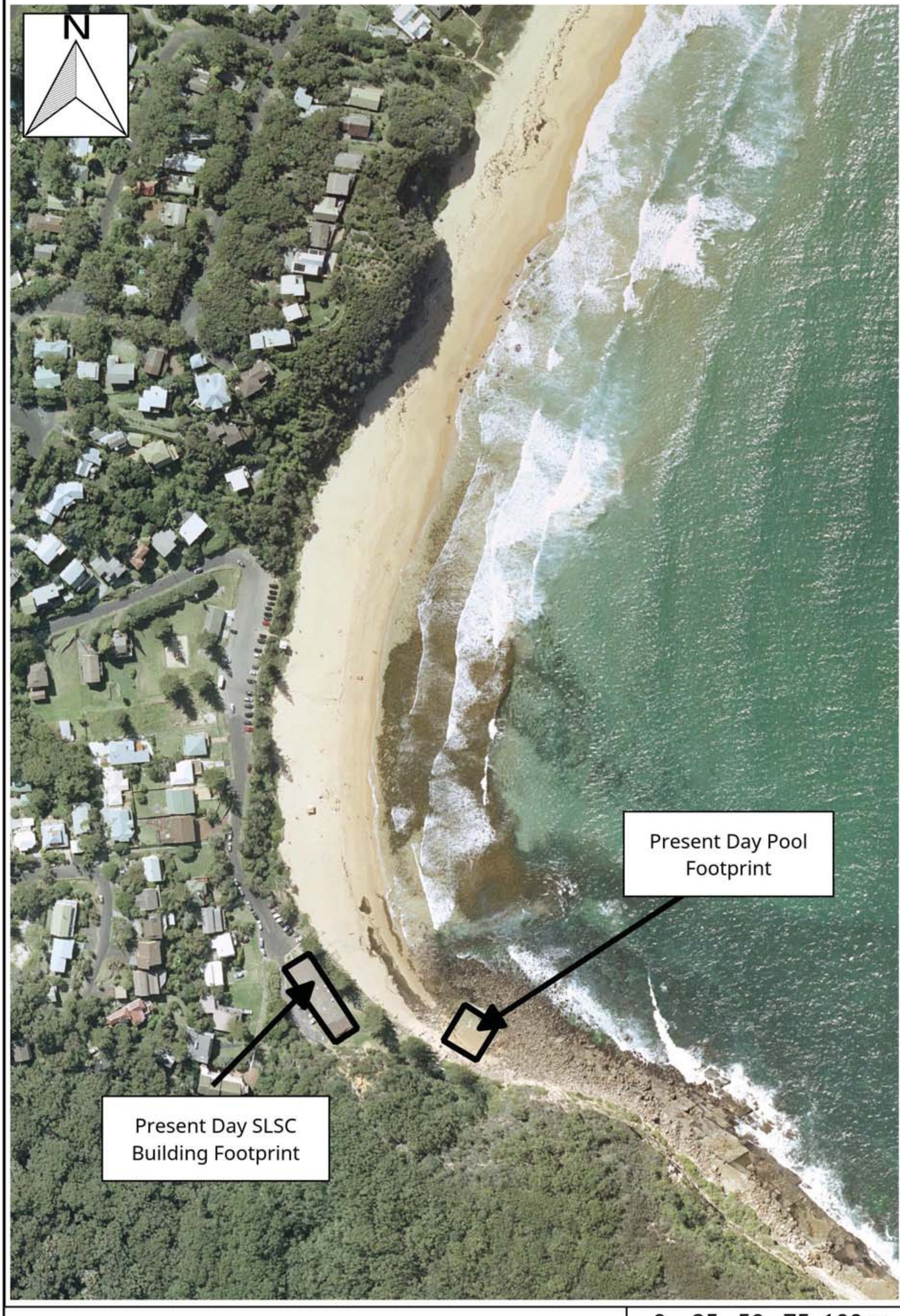
APPROX SCALE

Conceptual Design of Macmasters Beach Seawall

REV	DRAWN	CHECK
A	DJW	DJW

Salients

GIS File: /Them/Projects/P00016_MacMastersRevetmentConceptDesign/GIS/HistoricalAerials/MacMasterssouth_AppendixBPhotos.qgs



0 25 50 75 100 m
APPROX SCALE

Figure B10: 2005

Conceptual Design of Macmasters Beach Seawall

REV	DRAWN	CHECK
A	DJW	DJW

Salients

GIS File: /Them/Projects/P00016_MacMastersRevetmentConceptDesign/GIS/HistoricalAerials/MacMasterssouth_AppendixBPhotos.qgs

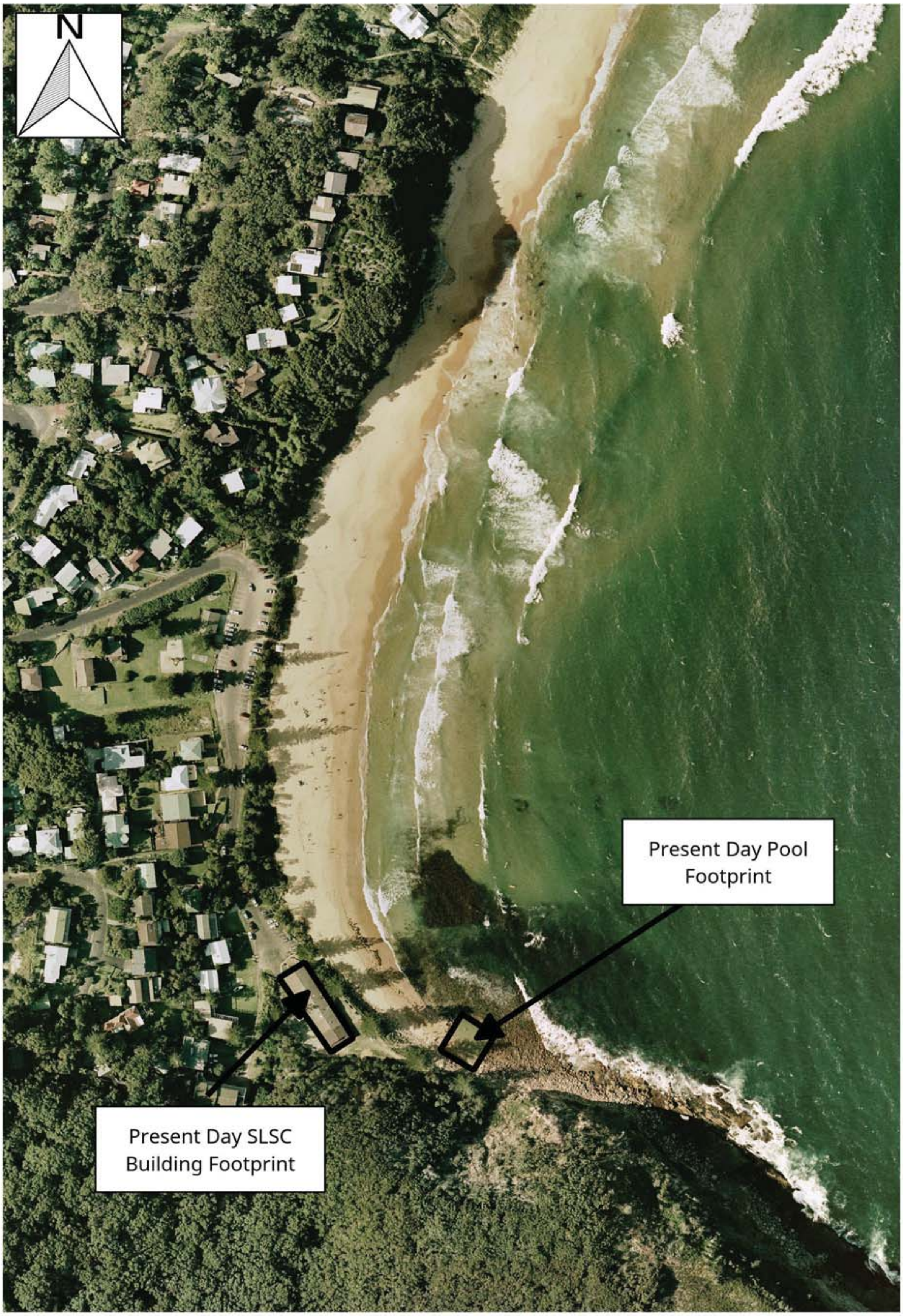


Figure B11: 2007

Conceptual Design of Macmasters Beach Seawall

REV	DRAWN	CHECK
A	DJW	DJW

Salients

GIS File: /Them/Projects/P00016_MacMastersRevetmentConceptDesign/GIS/HistoricalAerials/MacMasterssouth_AppendixBPhotos.qgs

0 25 50 75 100 m
APPROX SCALE

Figure B12: 2010

Conceptual Design of Macmasters Beach Seawall	REV A	DRAWN DJW	CHECK DJW

GIS File: /Them/Projects/P00016_MacMastersRevetmentConceptDesign/GIS/HistoricalAerials/MacMasterssouth_AppendixBPhotos.qgs

Salients

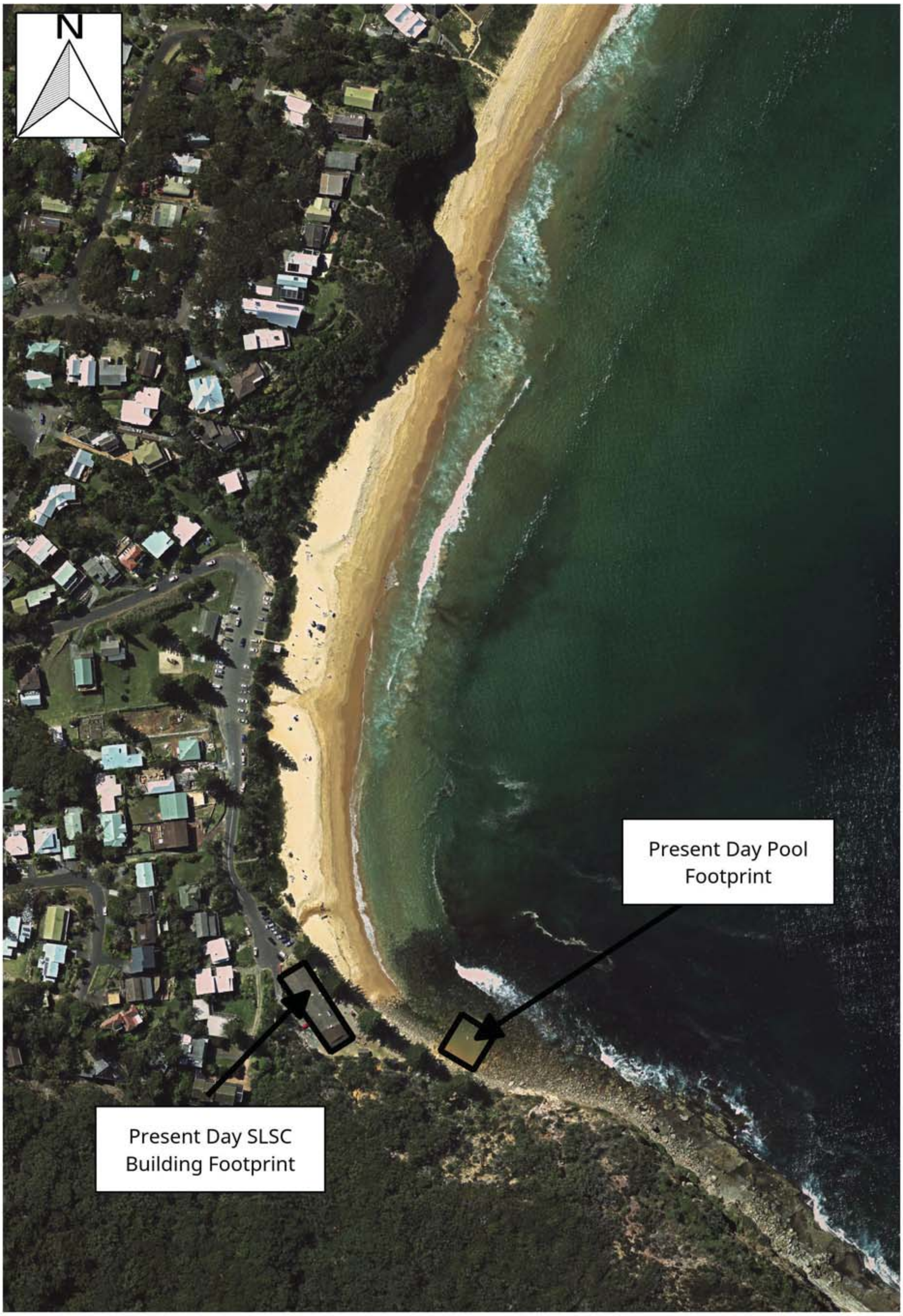


Figure B13: 2012	0 25 50 75 100 m APPROX SCALE			
Conceptual Design of Macmasters Beach Seawall	REV A	DRAWN DJW	CHECK DJW	Salients
GIS File: /Them/Projects/P00016_MacMastersRevetmentConceptDesign/GIS/HistoricalAerials/MacMasterssouth_AppendixBPhotos.qgs				

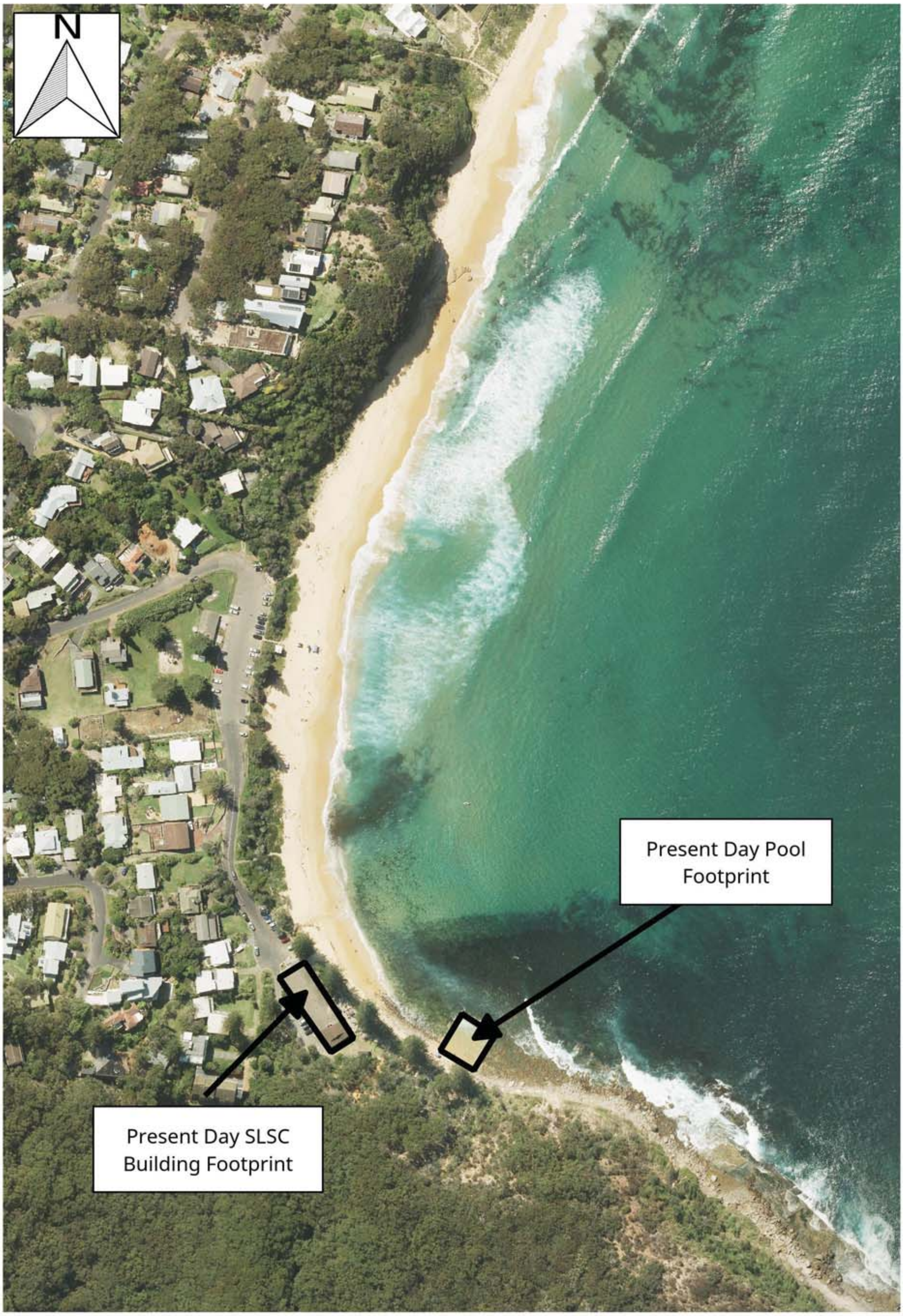


Figure B14: 2014	0 25 50 75 100 m APPROX SCALE		
Conceptual Design of Macmasters Beach Seawall	REV A	DRAWN DJW	CHECK DJW

GIS File: /Them/Projects/P00016_MacMastersRevetmentConceptDesign/GIS/HistoricalAerials/MacMasterssouth_AppendixBPhotos.qgs

Salients

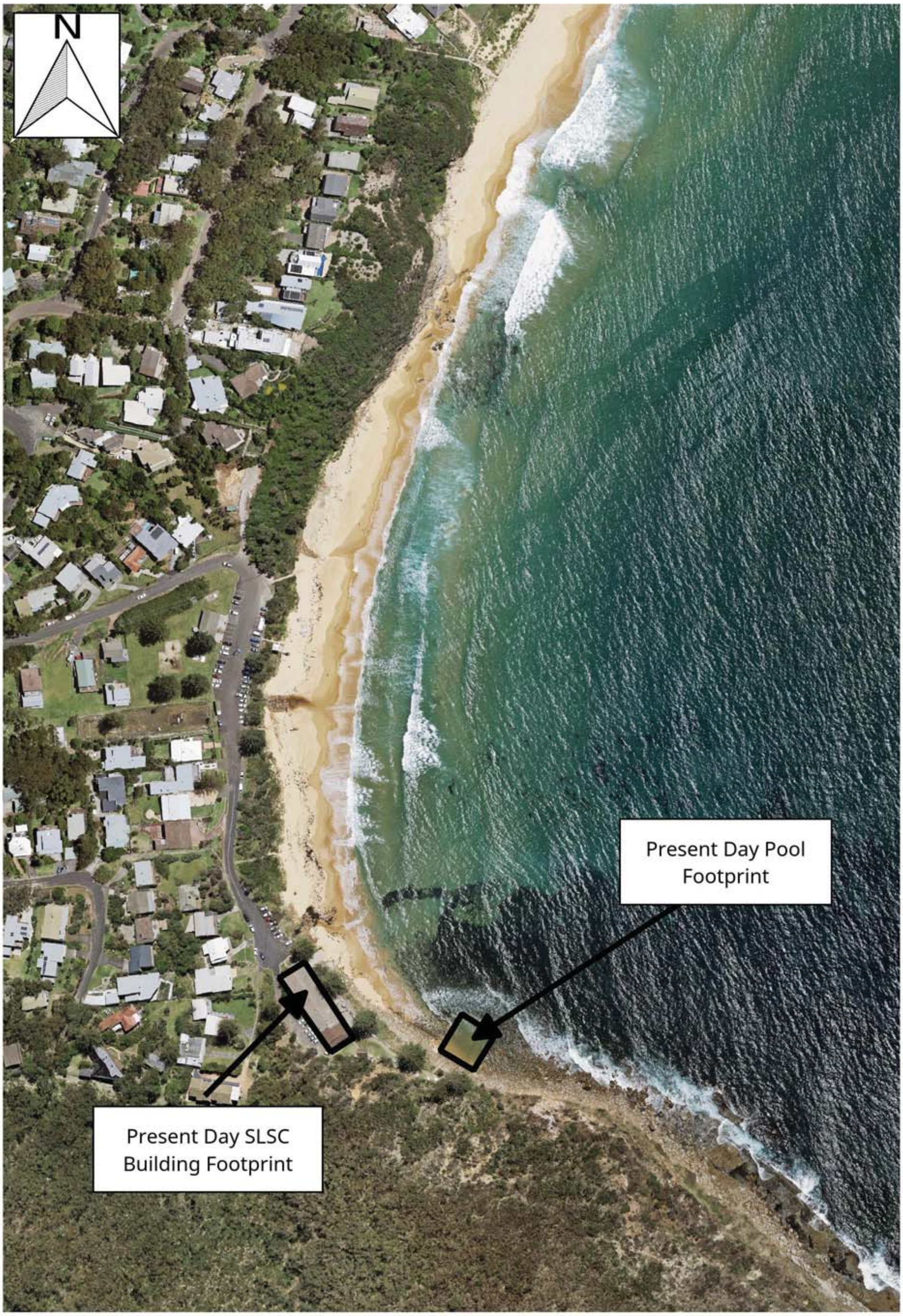


Figure B15: 2015 (December)

Conceptual Design of Macmasters Beach Seawall

REV	DRAWN	CHECK
A	DJW	DJW

Salients

GIS File: /Them/Projects/P00016_MacMastersRevetmentConceptDesign/GIS/HistoricalAerials/MacMasterssouth_AppendixBPhotos.qgs

Appendix C Geotechnical Assessment: Pells Consulting

ABN 74 978 620 434
Phone: 02 4381 2125
Fax: 02 4381 2126
The Old Post Office
49 Lakeside Drive
MacMasters Beach NSW 2251
www.pellsconsulting.com.au

Our Ref: S013_L1

19 September 2016

Dr David Wainwright
Salients Pty Ltd
PO Box 566
WALLSEND NSW 2287

Dear David

EXPECTED GEOTECHNICAL CONDITIONS – PROPOSED REVETMENT AT MACMASTERS BEACH

Following the coastal storm events of June 2016, ongoing erosion at the southern end of Macmasters Beach threatens the stability of the foreshore adjacent to the Macmasters Beach surf club. Temporary and unofficial works were undertaken in the attempt to stabilise a section of the earth batter directly in front of the clubhouse (Figure 1).

Figure 1

Gosford Council has requested Salients Pty Ltd to prepare a conceptual design for a revetment along the foreshore at this location, from approximately the boat ramp to the pool, to guide interim designs and future planning. Salinents Pty Ltd has, in turn, engaged Pells Consulting to provide guidance on geotechnical design issues.

Pells Consulting have undertaken some site investigations, including hand augering adjacent to the clubhouse, and supervision of test pitting along the toe of the proposed structure. Our findings and interpretation of expected geotechnical conditions are presented in this letter report. It is anticipated that a conceptual design for the revetment will be developed by Salients with reference to this information. Subsequent to development of a conceptual design, Pells Consulting will (in a future report) undertake geo-mechanical analysis for evaluation and perhaps optimisation of the design.

It is understood that the surf club is constructed on a bench of fill that has been obtained from the adjacent slopes – being silty sand, possibly of Pleistocene origin. The fill is understood to be un-engineered, and its depth is unknown. The original clubhouse was replaced in the early 1970's. During construction of the new clubhouse, the beach was impacted by significant coastal storms of June 1974. Historical photographs of the beach and clubhouse following the storms (Figures 2 and 3) indicate that the layer of cobbles and boulders that are typically visible around the MacMasters Beach pool are extensive, underlying a surficial layer of beach sand over most of the Macmasters beach embayment.

The extent of cobbles / boulders precludes the possibility of artificial placement. We postulate that the boulders have their origin from the ocean / cliff interface, and have been rolled (and worn) by wave action to their current position, although but we have been unable to find a published geological explanation for their formation.

Figure 2

Figure 3

Test pitting was undertaken at 4 locations along of alignment of the toe of the proposed structure (Figure 4). It was found that the boulders / cobbles do not typically lie directly on an underlying bedrock layer. Rather, the boulders are poised on a layer of silty sand of the same composition as material that was used for the bench of fill that supports the clubhouse. Photographs of test pits TP1 to TP4 are shown in Figures 5 to 8 below. Note that bedrock, below the silty sand layer, was encountered only at TP2 and TP3 (it was below the ~ 3m reach of the excavator at the remaining locations).

The borehole adjacent to the clubhouse (BH1) was augered to 4.5 m below ground level, encountering the same orange coloured silty sand throughout its depth. Its location is shown in Figure 9. A screened standpipe piezometer was installed in the bore and, despite the materials appearing damp during excavation, there was no standing water observed in the standpipe, even by the following day.

Cross sections, at the locations shown on Figure 4, were prepared with reference to the site investigations observations, and using contour data provided by Salients. These cross sections are shown in Figures 10 and 11.

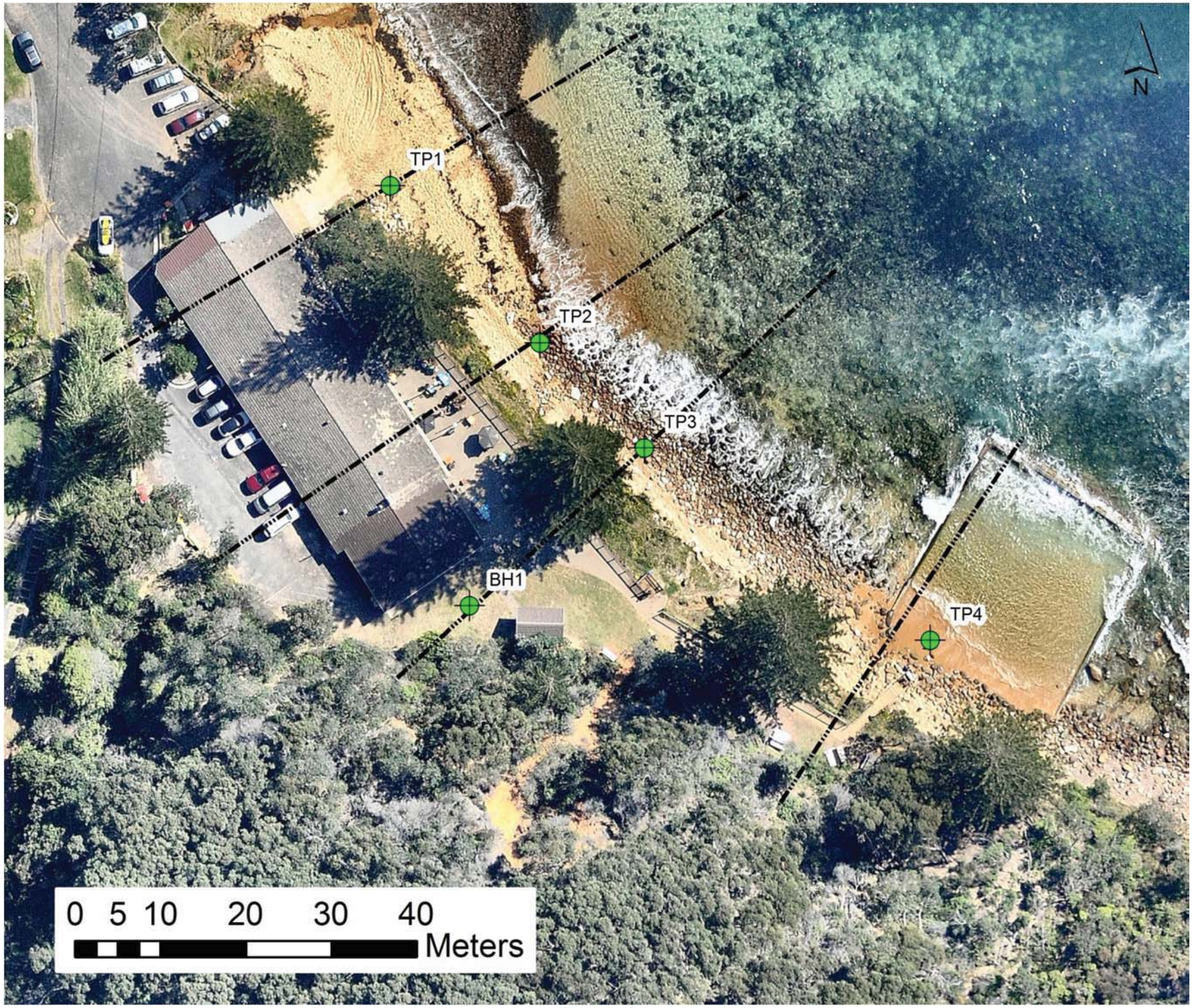


Figure 4

Figure 5 - Photographs from TP1

Figure 6 – Bedrock encountered at TP2

Figure 7 – Photographs at TP3

Figure 8 – The landward edge of the concrete bed of the pool was found to intersect bedrock.

Figure 9 – Location of bore augered to 4.5 m depth.

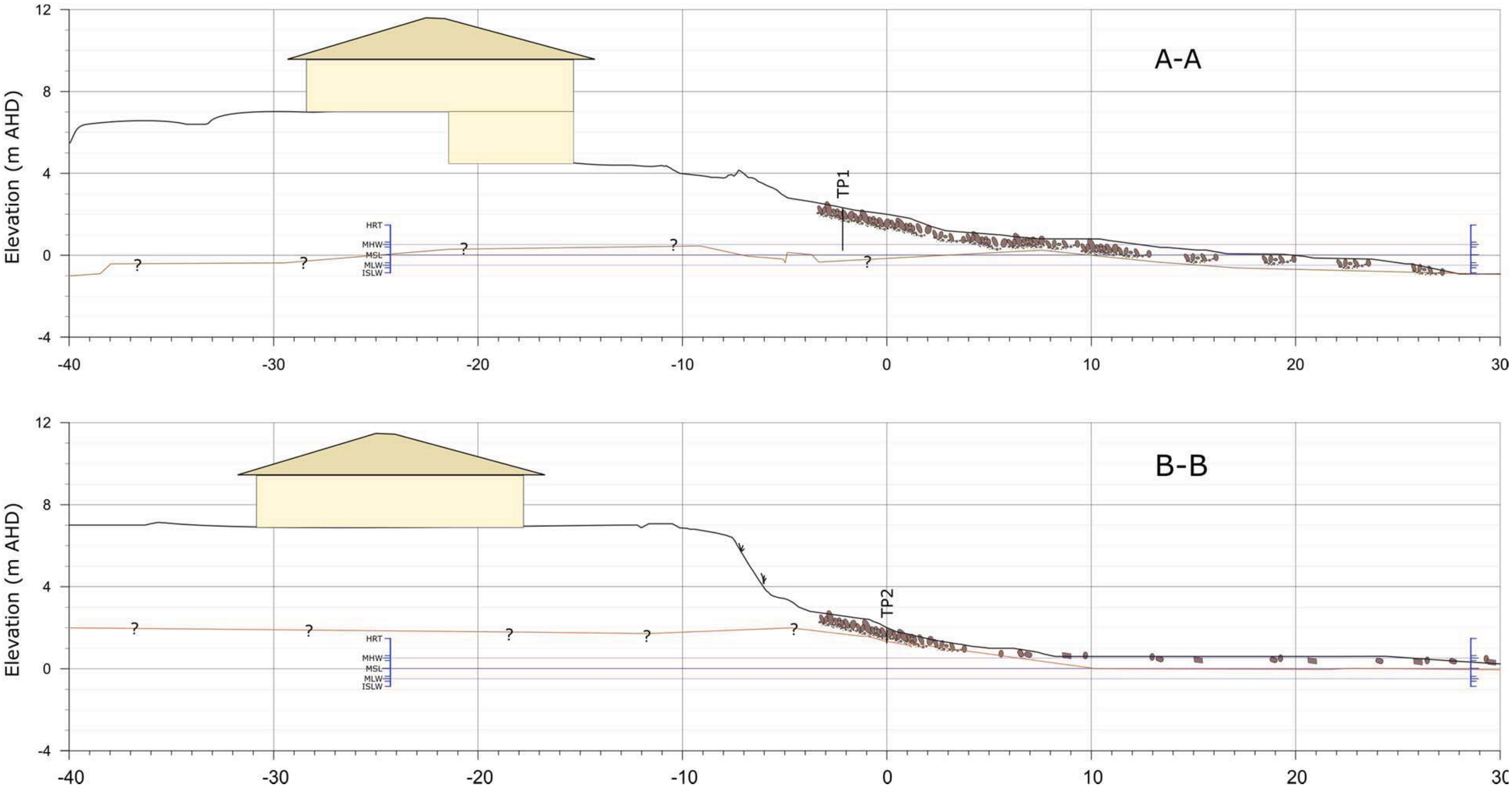


Figure 10 - Cross Sections A-A and B-B

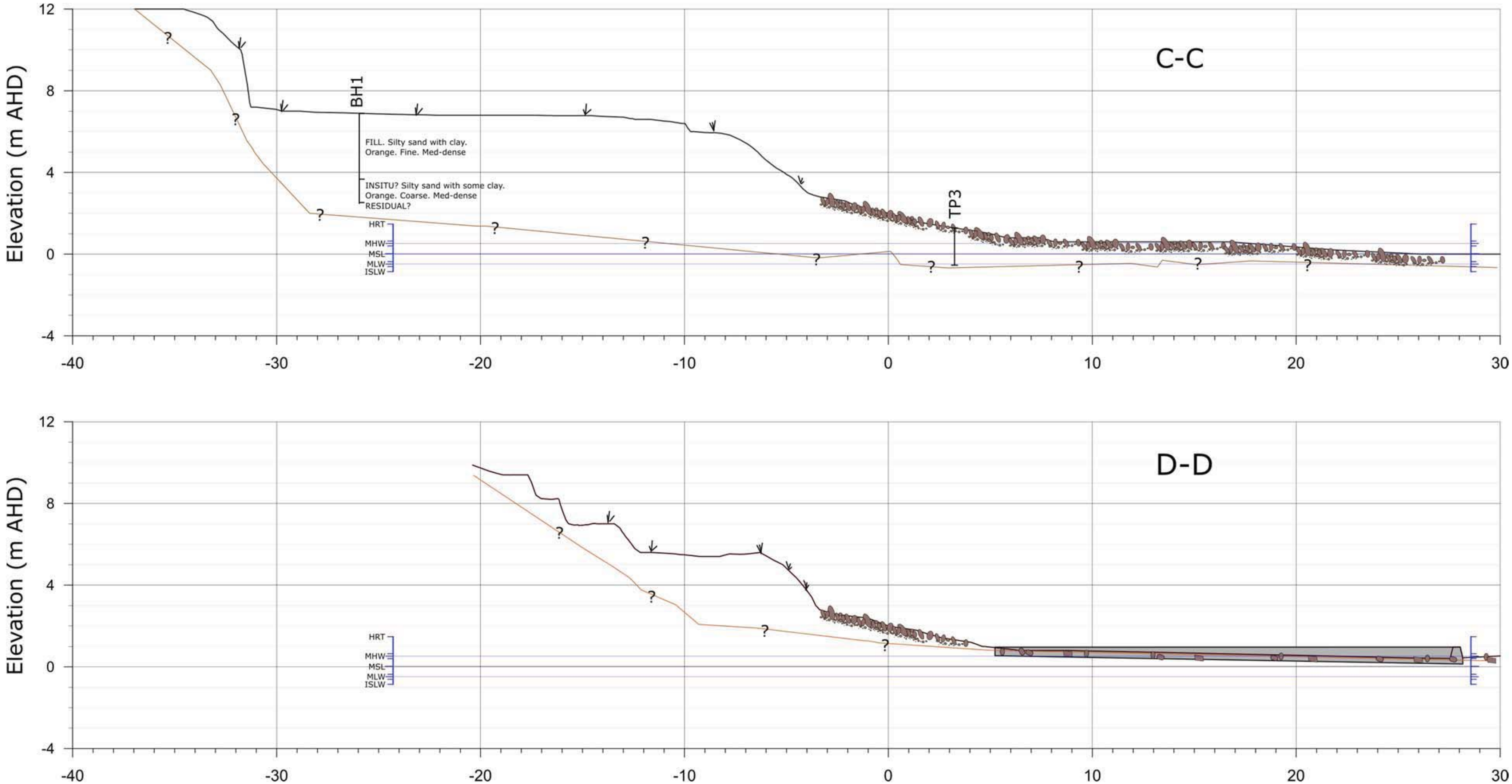


Figure 11 - Cross Sections A-A and B-B

Although the bedrock was inconsistently observed in test pit locations, it is maintained that the majority of the proposed wall is underlain by bedrock of a wave-cut platform. It is anticipated that the wave-cut platform would be around 0 to +1 m AHD, but the significant localised variation may be incurred by shifting of large blocks of bedrock, along fracture and bedding lines under historical wave action. This can be observed, for instance, at the rock platform east of the pool (Figure 12).

The presence of the creek that runs behind the clubhouse, and passing its northern extents, may have historically eroded a channel into the bedrock. It is possible that depth to bedrock could be significantly deeper along the northern section of the proposed wall.

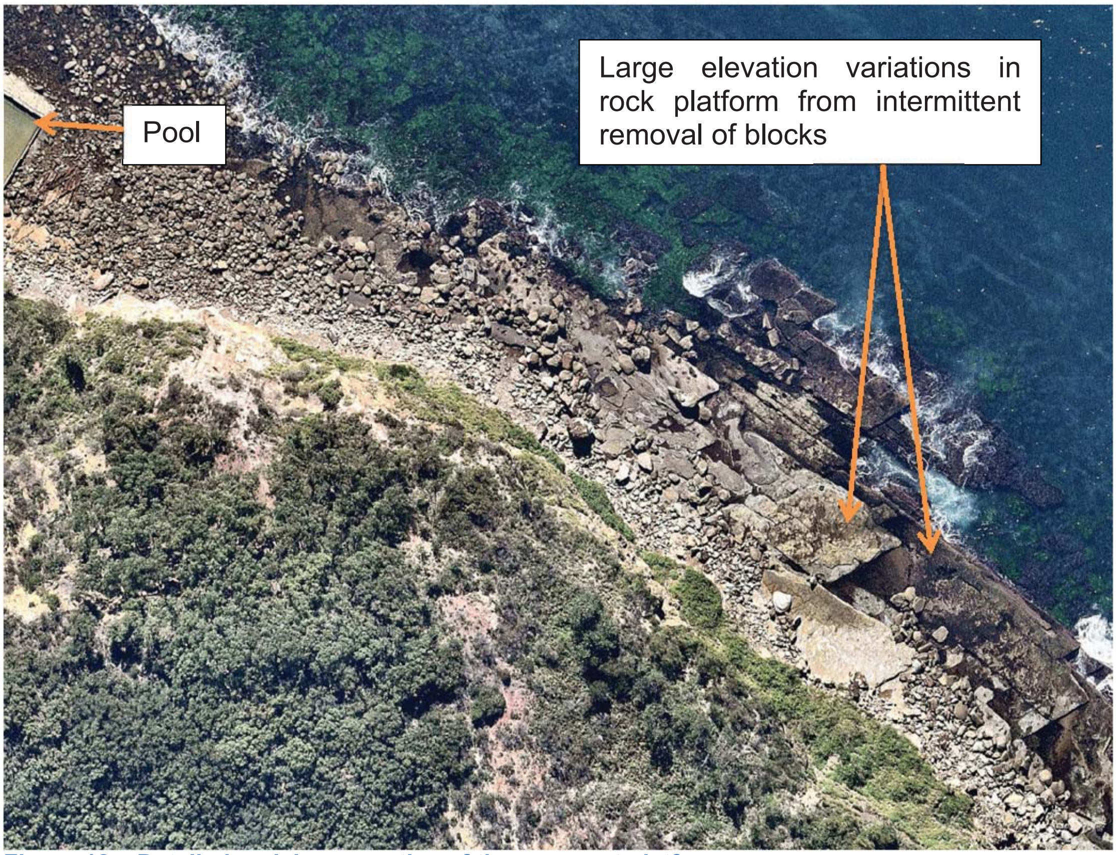


Figure 12 – Detailed aerial perspective of the wave-cut platform

The orange silty sand found in BH1 and in all test pits underneath the boulders is also observed to cover the forested slopes to the south west of the clubhouse (Figure 13). This material is distinguished from local ‘beach sand’, aside from its orange colour, by being slightly silty (<20%) and including some or a trace of clay (perhaps ~ 5%).

Figure 13 – Insitu residual orange silty sand

Preliminary Recommendations

Although the boulders that extensively line the beach are probably 'undersize' as coastal protection works, when designed according to current engineering practices, their spatial and historical persistence, and also their orientation (being generally dipped against wave action) suggest a proven robustness against wave loading, including the benchmark 1974 historical events. Although it is the role of Salients to make this judgement, it seems unnecessary to replace these boulders with artificial materials for the sake of toe stability – ie if the toe is designed to be 'tucked in' under the boulders, the natural armouring may be sufficient for toe protection.

Where the wave cut platform extends close to the surface (eg TP2), it makes sense that the toe of the proposed wall is founded on bedrock. However, there are indications that the bedrock level may vary considerably, to depths that are unnecessary to excavate down to. The design must therefore be flexible to allow founding of the toe on a depth of orange silty sand encountered across the site.

Yours sincerely,

STEVEN PELLS
BE(Civil) MEngSc PhD

Appendix D Slope Stability Assessment: Pells Consulting

Pells Consulting

ABN 74 978 620 434
Phone: 02 4381 2125
Fax: 02 4381 2126
49 Lakeside Drive
MacMasters Beach NSW 2251
www.pellsconsulting.com.au

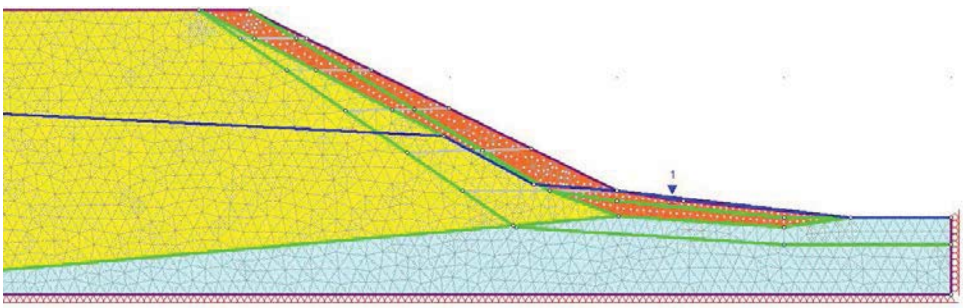
MEMORANDUM
MACMASTERS BEACH STABILIY ANALYSES

TO:	**DAVID WAINWRIGHT**
FROM	PHILIP PELLS
OUR REF:	**M021.M2**
DATE:	23 January 2017

1. INTRODUCTION

We have undertaken stability analyses of draft designs given in the draft Salients Report of 9 January 2017.

We have concentrated on the length of wall between Ch10m and Ch 35m where the draft design has a face angle of 1(v):1.5(h), and a height of about 5.5m. We have undertaken the analyses using finite element methods whereby the failure surface is not prescribed but is determined by means of stress analysis. We have studied the sensitivity of the computed safety factor to face slope, and assumed internal water pressures. We note that little is known as to the water pressures in the sand fill beneath the surf club under extreme rainfall events when the creek, which was diverted around the building, is in flood conditions for a sustained period of several days. Our finite element model assuming a slope angle of 1(v):2(h) is shown in Figure 1. This model is for what we assume to be a high piezometric level.



Material Name	Color	Initial Element Loading	Unit Weight (MN/m3)	Elastic Type	Young's Modulus (MPa)	Poisson's Ratio	Failure Criterion	Material Type	Tensile Strength (MPa)	Tensile Strength (residual) (MPa)	Dilation Angle (deg)	Friction Angle (peak) (deg)	Friction Angle (residual) (deg)	Cohesion (peak) (MPa)	Cohesion (residual) (MPa)	Properties Staged?	Piezo Line	Hu
Rock fill		Field Stress and Body Force	0.024	Isotropic	50	0.2	Mohr Coulomb	Plastic	0	0	0	42	42	0.0005	0.0005	Yes	Staged	1
Sand Fill		Field Stress and Body Force	0.02	Isotropic	100	0.2	Mohr Coulomb	Plastic	0.001	0.001	0	36	36	0.002	0.002	Yes	Staged	1
Beach Sand		Field Stress and Body Force	0.02	Isotropic	100	0.2	Mohr Coulomb	Plastic	0.001	0.001	0	33	33	0.002	0.002	Yes	Staged	1

Figure 1: Finite element model

2. RESULTS

Our computed safety factors are as follows:

- Slope 1 in 2 low piezometric surface (Fig 2) = 1.9
- Slope 1 in 2 high piezometric surface (Fig 3) = 1.7
- Slope 1 in 1.5 high piezometric surface (Fig 4) = 1.5
- Slope 1 in 1.5 high piezometric surface plus 5kPa surface loading = 1.4

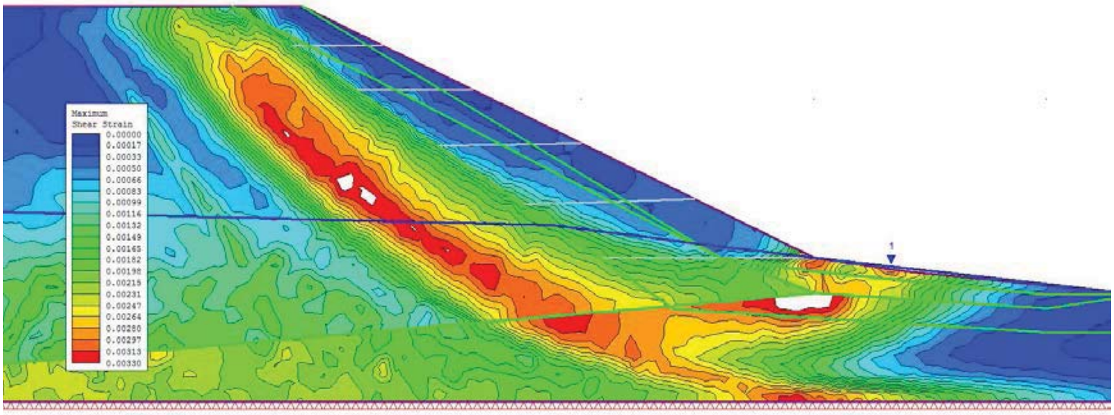


Figure 2

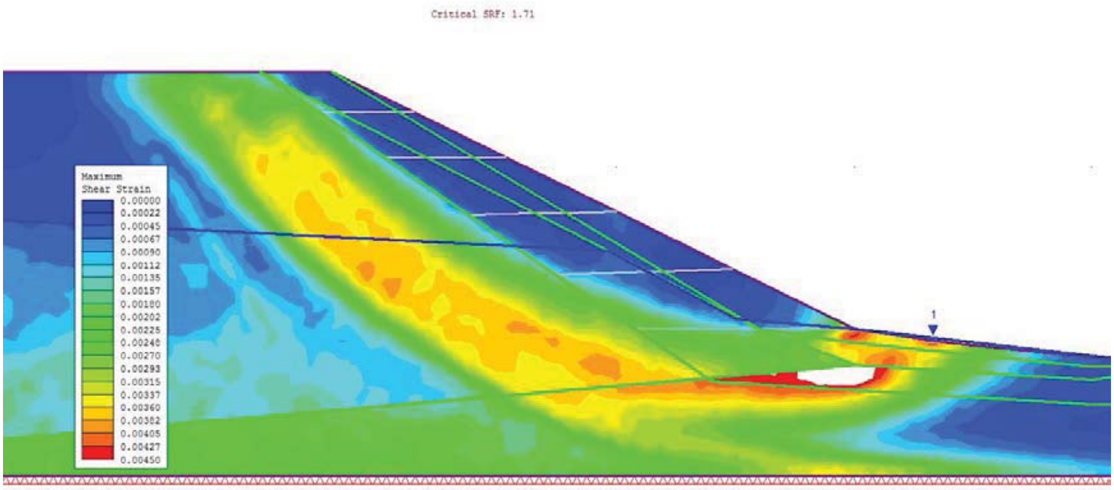


Figure 3

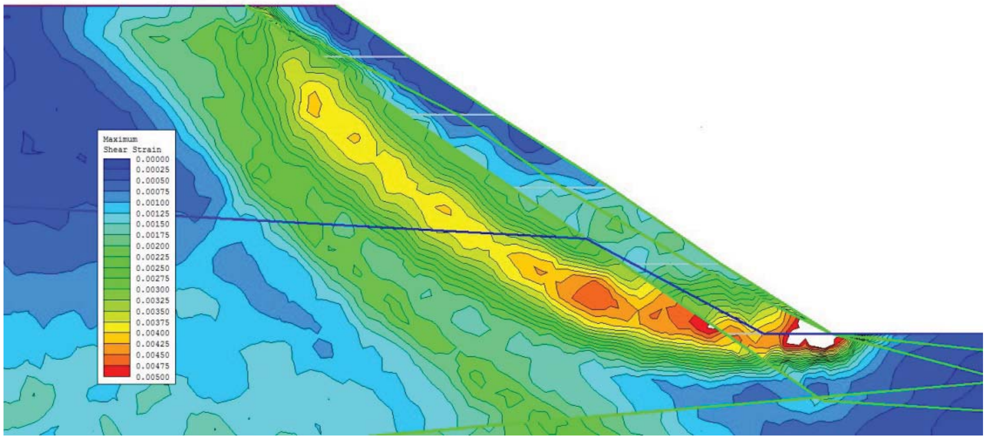


Figure 4

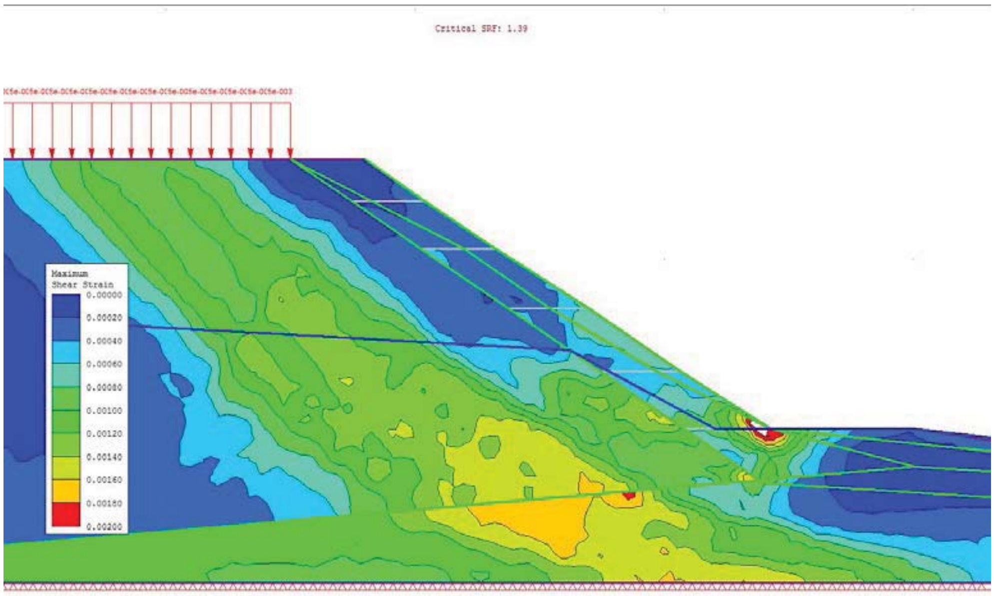


Figure 5

3. DISCUSSION

Normally a safety factor of 1.5 is considered adequate. However, we cannot be sure that our assumed ‘high’ piezometric surface is a reasonable expectation in a major storm event such as 1974, when there was high rainfall and heavy wave attack.

In our view it may be wise to adopt a maximum slope angle of about 30 degrees (i.e. about 1 in 1.7)

Yours faithfully,

PHILIP PELLS