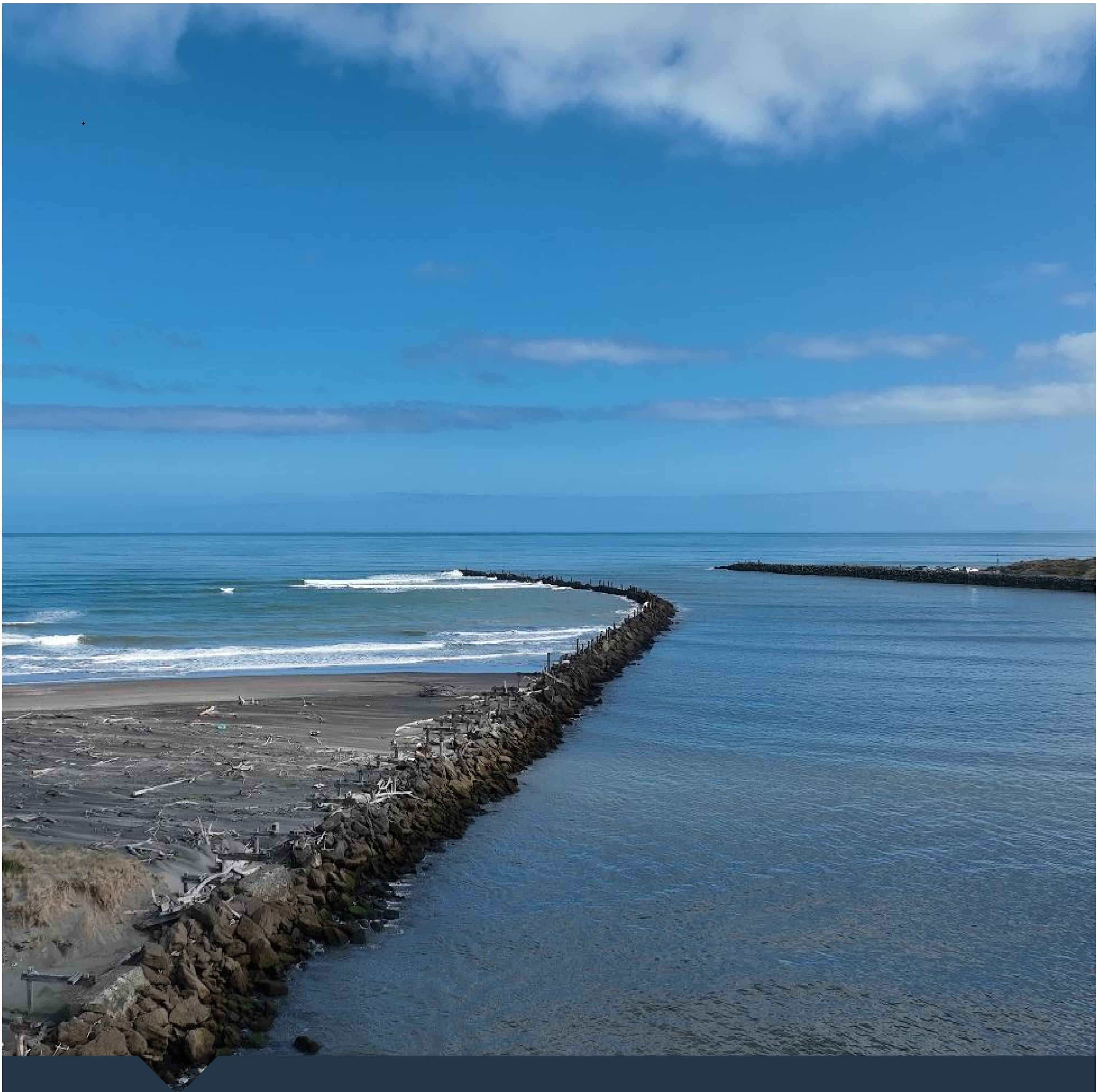

Whanganui River - South Mole Upgrade Options Assessment

June 2022

Prepared for:

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REPORT

Tonkin+Taylor

Whanganui River - South Mole Upgrade

Options Assessment

Prepared for
Horizons Regional Council
Prepared by
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Executive summary

The South Mole was constructed progressively in the early 1900s to stabilise the Whanganui River Mouth and provide improved hydraulic flushing and a more navigable river entrance. Since this time there has been limited work undertaken on the structure and the crest level has progressively lowered over time. Horizons Regional Council (HRC) have identified the South Mole as needing repair and upgrade to function as designed and are interested in possible options.

This report sets out options for consideration in upgrading the existing South Mole structure including possible rock sources, levels of service (i.e., depending on crest level and geometry) and construction methods. Environmental conditions, bathymetry and existing structure geometry and material have been considered in developing these options.

The majority of the South Mole structure is comprised of shell rock with measured average diameter between 0.8 m and 2 m, with the most seaward 200 m topped up by rectangular concrete armour units estimated to have a mean weight of 9T. The crest level of the existing structure ranges from just below 1 m MVD-53 (i.e., around high tide) to roughly 3 m MVD-53 (i.e., 1.5-2 m above high tide), with the most seaward 50 m to below 0 m MVD-53 based on available topography and bathymetry data. The side slopes of the existing structure are approximately 1.5(H):1(V) as a result of the original construction method (side tipped from rail) and slumping occurred at the river side due to riverbed lowering. The bed levels along the river side range from -5 m MVD-53 at the seaward end to -1 m MVD-53 at the landward end. Jet probing site investigations show that rock is buried 1-2 m below the sandy bed levels on the river side. The bed levels on the seaward side are shallower ranging between -2 and 0 m MVD-53 as a result of sand accumulating on this side.

For the upgrade of the existing structure, several rock sources have been considered (i.e., Shell rock, Andesite and Dolomite) with test results provided by HRC being used to assess rock characteristics and suitability as armour rock. Required rock sizes to upgrade the structure depend on the adopted rock source and have been assessed for the head and the trunk including sensitivity around several design parameters. The required rock size is largely dependent on the exposure to wave action with the design wave heights derived from wave model results. The resulting rock sizes for Andesite rock are a mean weight of 7T at the head (chainage 730-850), reducing to a mean weight of 4T for the trunk (chainage 150-730), with the required rock size for the trunk being similar to the measured rock of the existing structure.

The possible crest level and geometry of the South Mole upgrade are largely dependent on the construction methodology. The upgrade works can possibly be undertaken from land or from the river. The marine-based construction method would utilise an excavator mounted on a barge to place rock on the crest and/or river side of the existing structure. Barges would be brought in as close to the structure as possible, though a jack-up barge would likely be required for stability and due to the size of rock and reach required. Due to the limited reach of the excavator, depending on the required rock size, it is likely that rocks can only be place on the crest and on river side of the structure (i.e., the excavator cannot reach the seaward side of the structure). This limits the rocks to be place at the crest only (i.e., just above MHWS) or if a higher crest level is required (i.e., up to 3.3 m MVD-53) a single or double layer armour rock side slope needs to be created. The latter would result in a substantial increase in required rock volumes.

Land-based construction would involve creating a platform on the seaward side of the South Mole, likely around or just above MHWS (i.e., 1.4 or 1.9 m MVD-53) from the beach out to the structure head. This platform would be comprised of smaller rock which can be tracked/driven over, or of larger rock with a smaller blinding layer. The platform may need to be in the order of 4-6m wide depending on the final rock size and size of required construction plant. The platform will be constructed progressively towards the head and once the platform has reached its full extent, the structure can be closed in using armour rock and bringing the crest up to the preferred level as the machinery tracks back landward. This means that the crest levels are limited to a single layer or double layer of armour rock covering the construction platform sitting at 1.9 m MVD-53 and would be 3.3 m MVD-53 and 4.4 m MVD-53 respectively considering Andesite rock at the head of the structure. Slightly lower levels would be possible, such as 2.8 m MVD-53 with the platform at MHWS, however, this would increase the amount of expected construction downtime.

The possible upgrade options taking into account the above options and including estimated rock volumes assuming Andesite rock are set out in the table below.

Option	Rock size[1]	Crest level (m MVD-53)	Construction approach	Required rock volume (m^3)
1	W_{50} = 7T at head W_{50} = 4T at trunk	1.9	Marine-based construction top up existing structure	5,000-10,000
2a	W_{50} = 7T at head W_{50} = 4T at trunk	2.8	Marine-based construction river side slope and crest top up	18,000-28,000
2b	W_{50} = 7T at head W_{50} = 4T at trunk	2.8	Land-based seaward side platform (MHWS) and top up	15,000-25,000
3a	W_{50} = 7T at head W_{50} = 4T at trunk	3.3	Marine-based construction river side slope and crest top up	20,000-30,000
3b	W_{50} = 7T at head W_{50} = 4T at trunk	3.3	Land-based seaward side platform (MHWS + 0.5 m sea level rise) and top up	20,000-30,000
4	W_{50} = 7T at head W_{50} = 4T at trunk	4.4	Land-based seaward side platform (MHWS + 0.5 m sea level rise) and top up	40,000-50,000

[1]Based on Andesite rock

The table above sets out two marine-based construction options and two land-based construction options. For all options W_{50} = 7T rock has been adopted for the head section (i.e., chainage 730-850) and W_{50} = 4T for the trunk section (i.e., chainage 150-730). Adopting another rock source or different design parameters may possibly result in slightly different crest levels and rock volumes.

For the maintain existing level of service allowing for 0.5 m sea level rise option (i.e., top up to 1.9 m MVD-53) a rock volume of 5,000-10,000 m^3 have been estimated, which requires marine based construction. The volume significantly increases if the crest level is raised to 2.8 or 3.3 m MVD-53 for marine based construction as a result of needing to create a stable side slope along the existing structure. The estimated rock volume for a crest level at 2.8 m MVD-53 is 18,000-28,000 m^3 and at 3.3 m MVD-53 is 20,000-30,000 m^3.

For the land-based construction methods the estimated rock volumes are 15,000-25,000 m^3, 20,000-30,000 m^3, and 30,000-40,000 m^3 for crest levels at 2.8, 3.3 and 4.4 m MVD-53 respectively. For all options the larger rock at the head is 15-20% of the total volume with the smaller rock the remaining 80-85%.

1 Introduction

The South Mole was constructed progressively in the early 1900s to stabilise the Whanganui River Mouth and provide improved hydraulic flushing and a more navigable river entrance. Since this time there has been limited work undertaken on the structure and the crest level has progressively lowered over time. Horizons Regional Council (HRC) have identified the South Mole as needing repair and upgrade to function as designed (see extent in Figure 1.1) and is interested in possible options to upgrade the South Mole structure.

This report sets out possible upgrade options including possible rock sources, levels of service (i.e., depending on crest level and geometry) and construction methods. Environmental conditions, bathymetry and existing structure geometry and material have been considered to develop the possible options. Design sketches for typical sections have been included to illustrate the possible options. The purpose of this assessment is to assist HRC with selection of the preferred upgrade solution, which can be used to progress to a consent level design and effects assessment and, following consent approvals, to a detailed design stage suitable for tender.

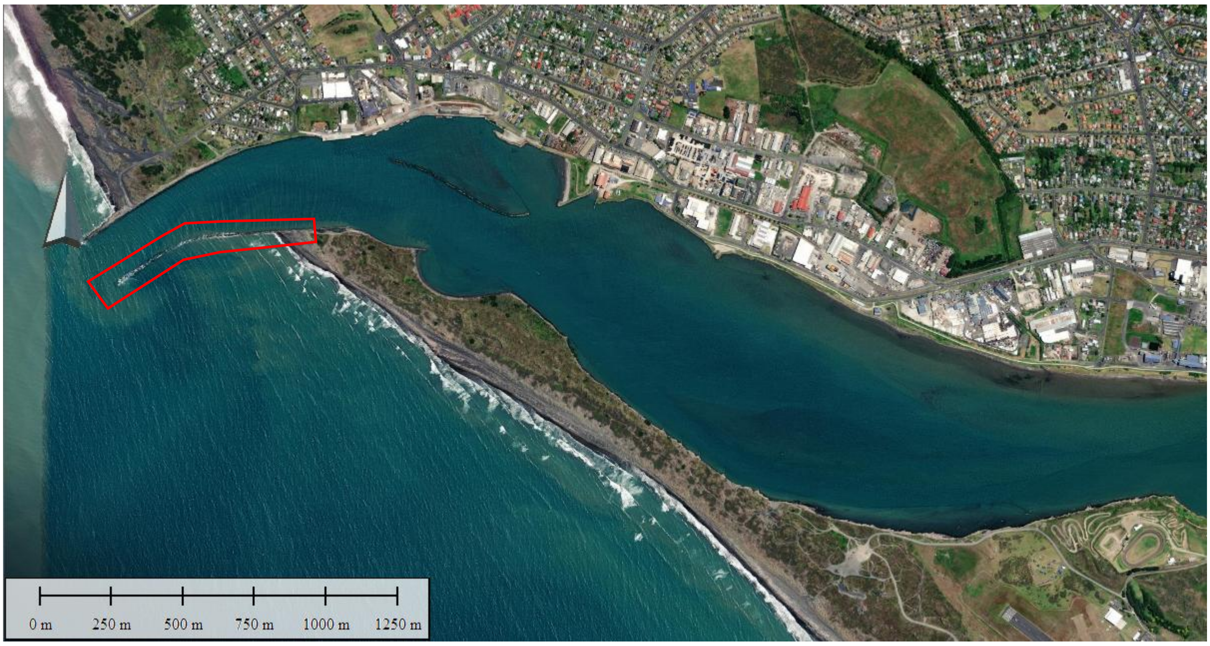

Figure 1.1: Proposed extent of South Mole repair and upgrade works

2 Existing South Mole structure

The South Mole of the Whanganui River mouth was originally constructed comprising limestone shell rock from local sources. It is understood that the mole has been constructed progressively seaward by constructing a train track on piles with the shell rock being side tipped (on the river side) to form the structure. Maintenance and repair have not been well documented, although a description of the South Mole development and modifications is set out by CSL (2016). Based on a review of aerial photographs, in addition to the shell rock there has likely been some top-ups later in the 20th century, using rectangular concrete blocks, identified along 200 m from the seaward tip.

The length of the existing structure is approximately 850 m long, with the seaward 700 m included in this assessment report (see extent in Figure 1.1). The existing structure (see Figure 2.1) is partially submerged in high tides and gradually degrading through displacement of armour units, ongoing settlement, and weathering of the armour units. Ongoing degradation will eventually lead to increased wave penetration, lower control of river channel, eventual breach, and likely shallower bar - posing hazard to navigation.

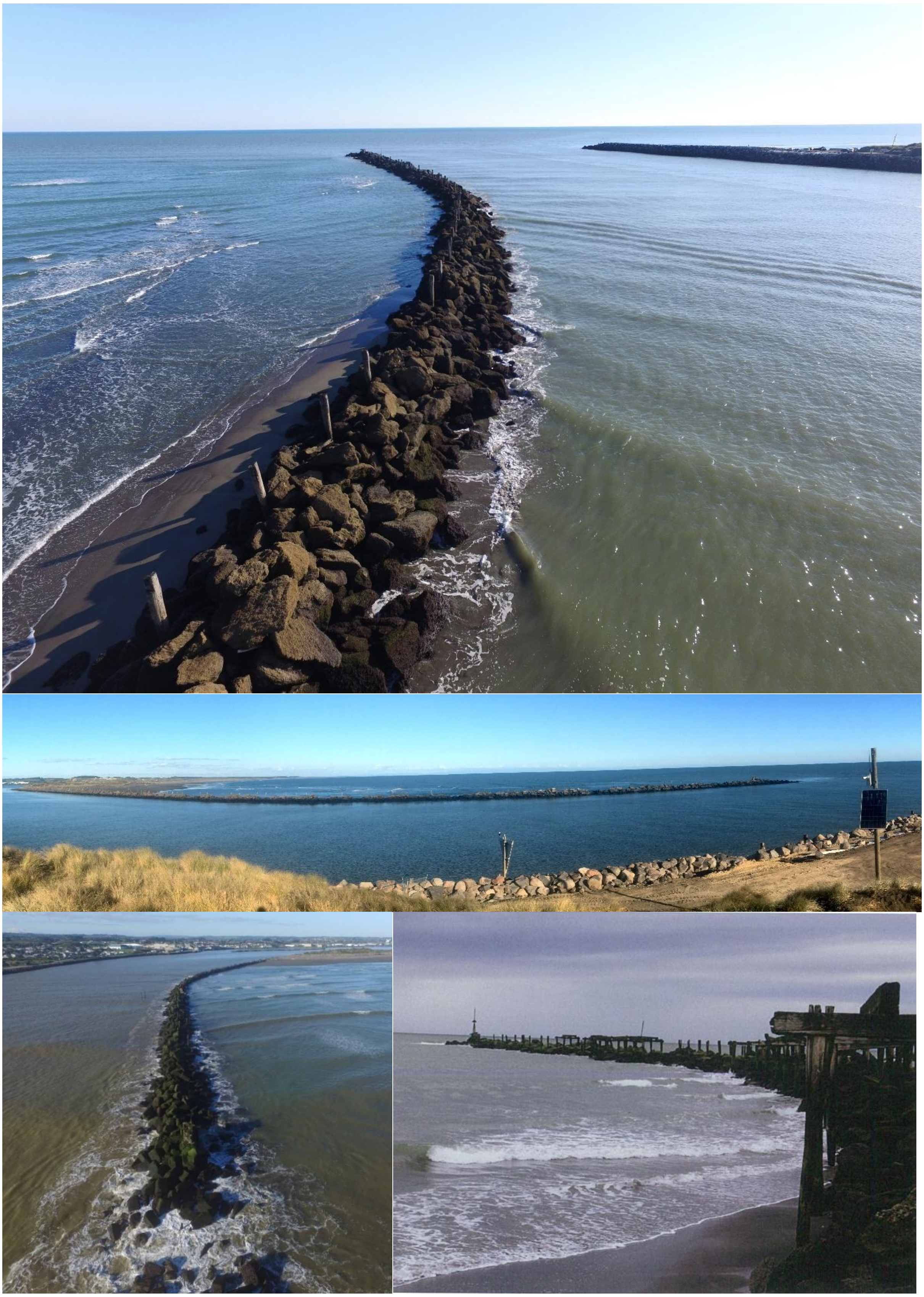

Figure 2.1: Photographs of the South Mole including oblique image from the trunk looking seaward (top, source T+T), from the North Mole (middle, source T+T), oblique image of Mole head (lower left, source T+T), and photograph along south side of South Mole (lower right, source Moore, 2014)

2.1 Rock armour and concrete unit sizes

The majority of the South Mole structure is comprised of shell rock, with the most seaward 200 m topped up by rectangular concrete armour units. The size of armour rock has been assessed at two locations (i.e. one at the seaward end and one at the landward end) using measurement of two dimensions of individual rocks from aerial photograph. A sample size of 30-50 rocks has been measured at each location with a range in averaged diameters of between 0.8 and 2m and a median measured size of D = 1.3. The converted weight is W_{50} = 3,200 kg and ranging from 750 kg to 12,000 kg based on a density of 2,500 kg/m³ (based on Shell rock quarry test results).

The concrete armour units along the seaward 200 m length have likely been cast from the same size mould and have been estimated at 2.5 m x 1.5 m x 1.0 m, which is equivalent to a 9,000 kg unit. However, large amounts of weathering since construction is apparent, with fractures, breakages, and size reductions from abrasion being identified. Measurement of a sample of 30 units shows the converted median weight of W_{50}, = 7,700 kg and ranges between 4,000 and 12,000 kg.

Table 2-1 shows the measured armour rock and concrete armour unit sizes and their converted weights assuming 2,500 kg/m³ for shell rock and 2,400 kg/m³ for concrete.

Table 2-1: Measured armour rock and concrete unit size and converted weights

Type	D_{50} (m)	Median Diameter (m)	Diameter range (m)	W_{50} (kg)	Median weight (kg)	Weight range (kg)
Shell rock	1.33	1.29	0.8-2.0	4,300	3,200	750-12,000
Concrete unit	1.77	1.75	1.4-2.4	8,300	7,700	4,000-12,000

Note that the rock and concrete armour units were measured using LINZ 2017 aerial images, with a resolution of 0.075 m and that the third dimension (depth) could not be measured using this approach. The 2D measurements also have limitations as rocks may be partially buried or shadows may make it difficult to interpret the correct lengths. This gives an estimated margin of error of +/- 0.2 m. However, given uncertainty is likely in both positive and negative, and given our reasonably large sample size, we believe the median size/weight values are a reasonable estimate. Figure 2.2 shows an example of how a rock size is measured in two dimensions and visualises some of the limitations (e.g. buried rocks, shadows, reduced resolution, etc.).

Figure 2.2: Method of measuring rock size

2.2 Topography and bathymetry

Several sources of topographic and bathymetric data are available within the Lower Whanganui River mouth area. These have been combined to create a single topographic-bathymetric model. Overlapping surveys have been ranked in hierarchy based on date, resolution, survey source and quality/applicability, with historical images used to validate differences where possible. Note that due to the changing morphology over time, particularly on the dynamic south beach side, some vertical differences are apparent between surveys. Appendix A shows plans and cross-sections of the different data sources and the vertical differences between the data sources. Table 2-2 sets out a list of topography and bathymetry sources in order of hierarchy, and Figure 2.3 for their respective survey bounds.

Table 2-2 List of topography and bathymetry sources for the Whanganui lower river mouth

Rank	Survey Description & Source	Date	Resolution	Source Vertical datum
1	LiDAR: Whanganui District Council, LINZ	2020/2021	1 m	NZVD2016
2	DML multibeam survey of port basin	02-08-2021	0.1 m	MVD53
3	DML multibeam survey of port basin	24-08-2017	2 m	MVD53
4	HRC Bathymetric Survey of South Mole	26-10-2021	N/A[1]	MVD53[2]
5	HRC Whanganui Riverbed Survey	15-12-2015	N/A[1]	MVD53
6	LINZ Hydrographic Chart 4541	1993	N/A[1]	CD

[1]Gridded resolution is not valid for soundings or single beam surveys, as points/paths are separated by more than 5-10 m, and intermediate values must be interpolated.

[2]The HRC bathymetric survey was vertically offset from overlapping LiDAR points of confirmed heights by ~1.6m when in same vertical datum (MVD53). Survey points were adjusted to match the higher ranked LiDAR survey.

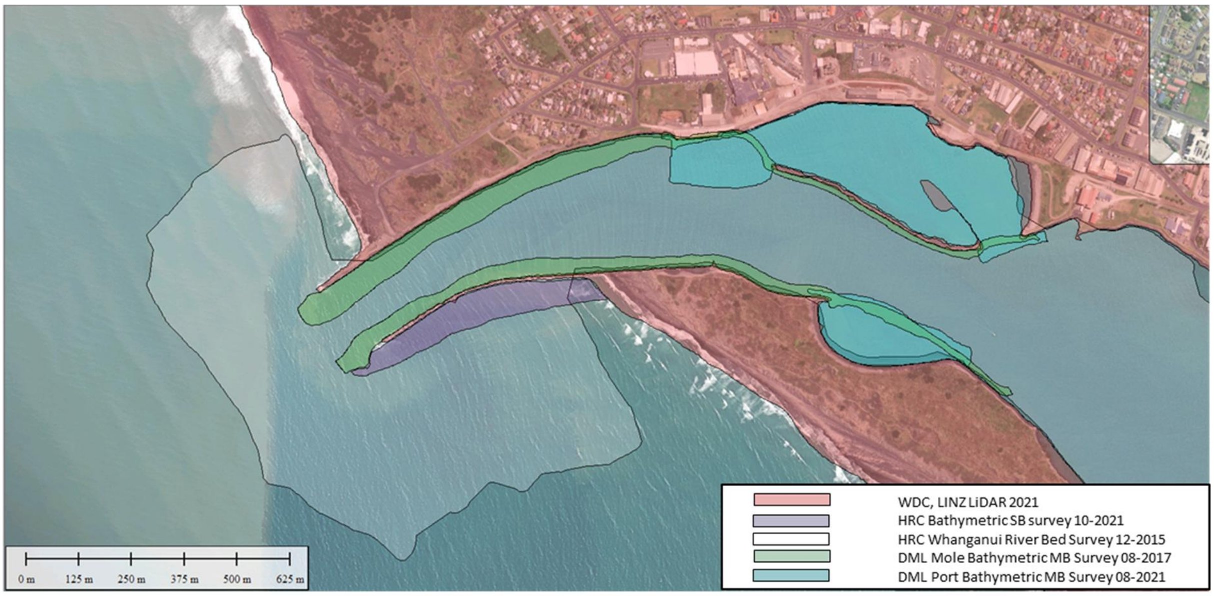

Figure 2.3: Bathymetric survey bounds

From the combined bathymetry elevation dataset (i.e., DEM) cross-sectional profiles have been created at approximately 5m intervals from the landward extent to the tip of the training wall. Figure 2.4 shows the generated cross-sectional profiles extending 50 m to either side of the centre of the training wall. The profiles have been coloured based on similarity in geometry and can be subdivided in seven sections. The magenta profiles are the most landward profiles with an established dune landward of the existing rock armour structure. The subsequent 4 sections show a gradual decrease in bed levels both on the river side and seaward side, and also a decrease in typical structure crest level. The profiles between chainage 800 and 850 m show the structure lowered to around 0 m MVD-53, with the most seaward 50 m lowered to below -2 m MVD-53. The geometry of these profiles will be used to inform the required rock volumes to upgrade the South Mole structure.

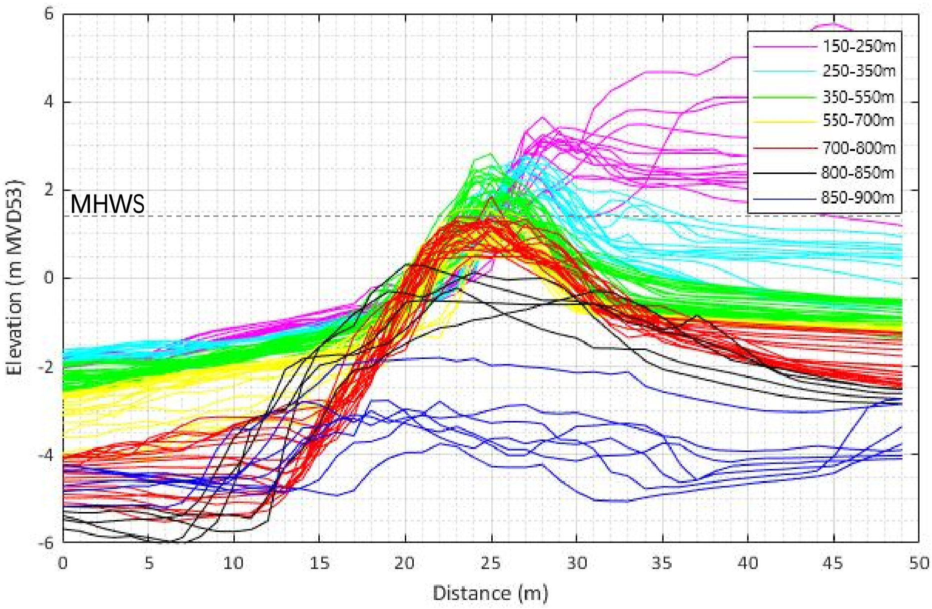

Figure 2.4: Cross-sectional profiles through South Mole at ~5m intervals (see chainage in Figure 3.2)

Figure 2.5 shows a long section of the South Mole structure along the centre of the structure including MHWS line. This shows that the crest at the seaward of chainage 400 is on average around MHWS with some rocks extending above and below MHWS. The crest level rises to 2 m MVD-53 around chainage 400 and further to around 2.5-3 m MVD-53 between chainage 150 and 300.

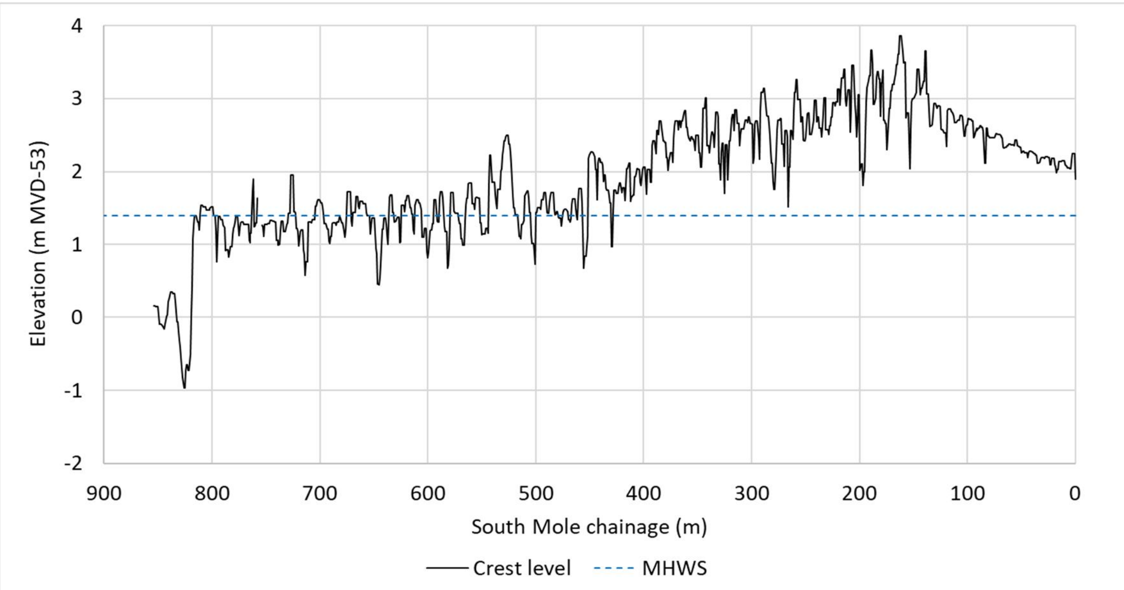

Figure 2.5: Crest level along centre of the South Mole structure

2.2.1 Historic crest level

Based on CSL (2016), the South Mole was originally built to mean sea level and was raised to mean high water (~1.4 m MVD-53) in 1929. The structure was upgraded/topped up several times since 1929, but no accurate records exist. Therefore, a review of historical photographs has been undertaken to estimate the historic crest level. Based on the review it appears that the crest level varies considerably and was between ~0.7 and ~3.5 m below the rail structure in 1970 (see Figure 2.6). The elevation of the piles supporting the rail structure are assumed to be generally consistent over time, with a height of approximately 4.0m MVD53 based on recently surveyed pile levels. It is estimated that the average crest level of the structure in 1970 was approximately 1.7 m below the rail structure or approximately 2.3 m MVD53 (approximately 1 m above MHWS). The lowest estimated crest level is 0.5 m MVD-53 (i.e., approximately MHWN) and the highest estimated crest level is 3.3 m MVD-53 (i.e., approximately 2 m above MHWS). The variable crest levels are likely a result of differential slumping along some sections and it is likely that the structure crest started closer to the maximum level.

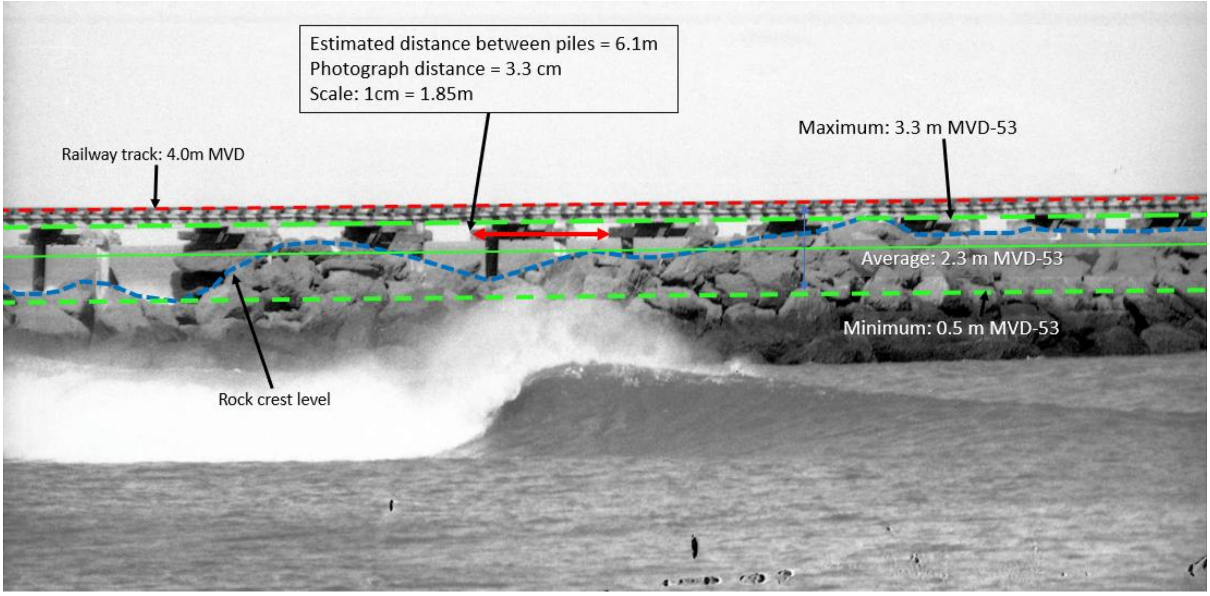

Figure 2.6: Photograph from 1970 showing height difference between railway track and crest of rock structure (source: R. Shand)

2.3 Rock toe levels on river side of structure

In order to assess the depth of armour rocks below existing sand levels a diver survey has been undertaken. The existing rock levels are important to consider for upgrading the South Mole structure, with additional rocks likely placed down to existing rocks. The diver survey has been undertaken by launching a jet from a boat with a probe penetrating the sandy bed layer to measure the thickness of sand covering the rocks. The measurements have been undertaken approximately at 100 m intervals along the structure and 10 m intervals off the toe of the structure. Figure 2.7 shows the locations of the measurements.

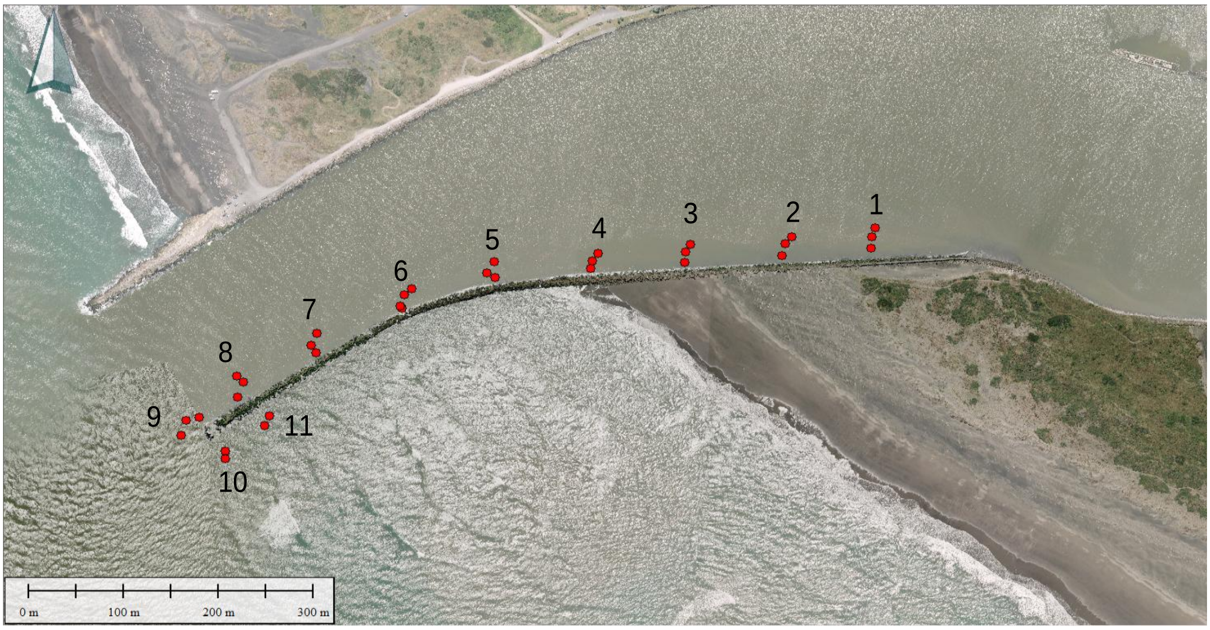

Figure 2.7: Locations of diver survey

The resulting sandy bed levels, rock levels and depths to rock have been set out in Table 2-3. This shows a variable depth to rock ranging from 0 m to 2 m depending on the location, with an average depth of approximately 1 m below the riverbed level.

Table 2-3: Diver survey results

Location	Approx. Chainage	Sandy bed level (m MVD-53)	Rock level (m MVD-53)	Depth to rock (m)
1	100	-1.9 to -2.3	-2.7 to -3.3	0.4-1.0
2	200	-1.7 to -2.3	-2.4 to -3.0	0.6-0.8
3	300	-2.0 to -2.2	-2.7 to -3.0	0.7-0.8
4	400	-1.8 to -2.4	-2.5 to -3.2	0.7-0.8
5	500	-1.9 to -2.3	-2.9 to -3.1	0.8-1.1
6	600	-2.2 to -3.0	-2.3 to -4.0	0.1-1.5
7	700	-3.5 to -3.8	-4.1 to -5.8	0.6-2.0
8	800	-4.7 to -5.1	-6.3 to -7.1	1.3-2.0
9	850	-3.9 to -4.7	-3.9 to -4.7	0-0.1
10	850	-3.3 to -4.2	-3.8 to -5.0	0.5-0.8
11	800	-3.1 to -3.4	-3.6 to -4.0	0.5-0.6

3 Design conditions

3.1 Water levels

Astronomical tide water levels and extreme storm tide water levels have been set out in Table 3-1, with astronomic tide levels based on LINZ (2021) and extreme water levels consistent with levels included in the North Mole design report (T+T, 2018). The spring tide ranges from -1.2 m MVD-53 to 1.4 m MVD-53, with the 100-year ARI (Annual Recurrence Interval) storm tide level assumed to include 0.5 m storm surge. The 100-year ARI water level including 0.5 m sea level rise is considered for as the extreme water level allowing for approximately 50 years of sea level rise.

Table 3-1: Water levels

Water level	MVD-53 (m)	CD (m)
100-year ARI + 0.5 m sea level rise	2.4	4.0
100-year ARI	1.9	3.5
MHWS (Mean High Water Springs)	1.4	3.0
MSL (Mean Sea Level)	0.1	1.7
MLWS (Mean Low Water Springs)	-1.2	0.4

3.2 Waves

The offshore wave climate at a depth of 30 m was extracted from the University of Auckland Wave Data tool (uoa-eresearch.github.io/waves/hindcast.html). This is a satellite calibrated hindcast that provides 3 hourly wave data from 1993 – 2019 at any location in New Zealand (Albuquerque et al., 2018). An overview of wave climate offshore of Whanganui is presented in Figure 3.1, showing the relationships between wave height, wave direction and wave period. The largest waves that could influence design parameters occur in two cohorts, from the SSW and WSW. Since wave direction has an influence on wave behaviour in the nearshore and channel, both these directions were considered important for representing in the nearshore transformation model.

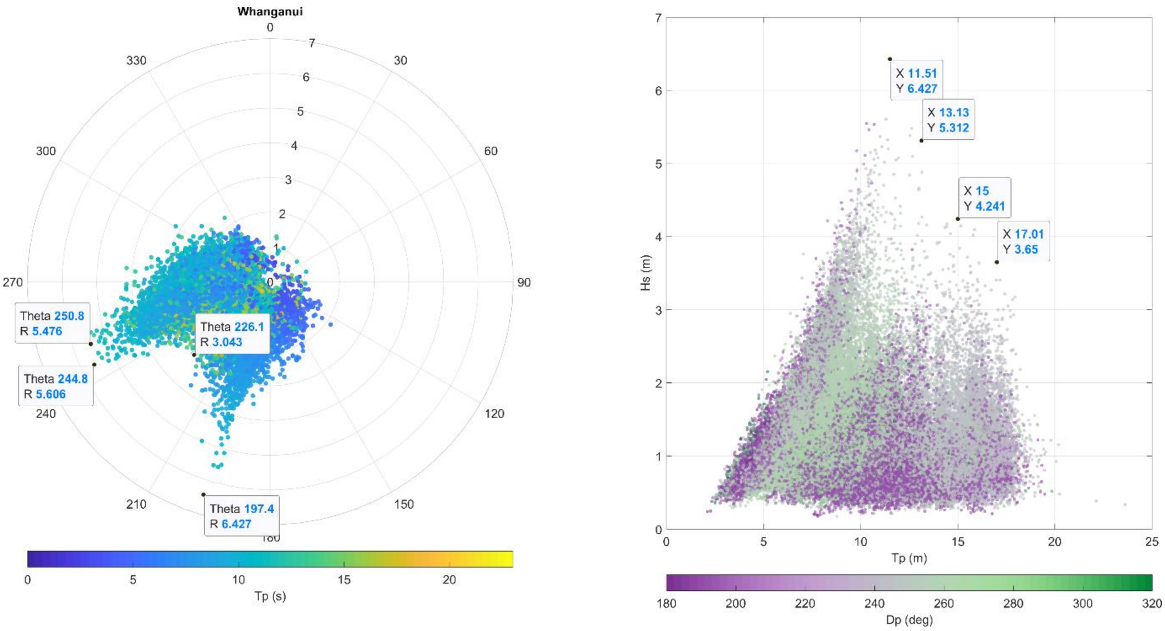

Figure 3.1: Wave climate offshore of Whanganui Port, based on the UoA hindcast including example points for both plots to indicate extreme wave height, period and direction combinations

For the wave modelling the bathymetry data sources as set out in Section 2.2 were used. Due to the changing morphology over time, particularly on the dynamic south beach side, some vertical differences are apparent between surveys. These have been resolved by adopting the latest dataset in preference and linear interpolation between datasets to smooth the vertical jumps. Figure 3.2 shows the final bathymetry DEM used as input into the numerical wave modelling.

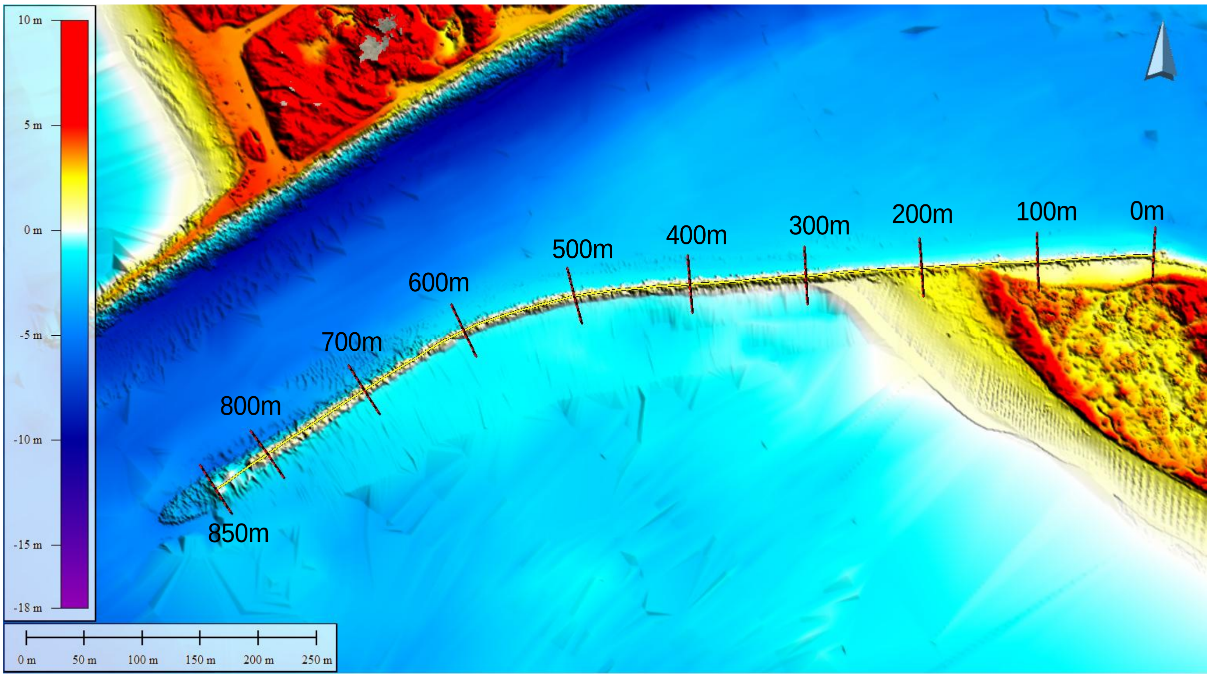

Figure 3.2: Final bathymetry DEM used as input to numerical modelling including chainage

Three steps were followed to understand the wave height around the South Mole for a 100-year ARI design event.

1. An extreme value analysis (EVA) was undertaken for the full hindcast dataset for significant wave height, and a directional EVA was undertaken to return period wave height from the SSW and WSW (Table 3-2).
2. The 100-year wave height for the SSW and WSW directions were transformed from the offshore to nearshore using a spectral wave model (SWAN) to account for refraction, shoaling and depth limiting to the 15 m depth contour. Wave outputs from SWAN resulted in approximately 5 degrees of directional change and up to 0.5 m reduction in wave height between the 30 m depth and 15 m depth. The resulting wave height and direction from SWAN was then used as a boundary condition in a higher resolution nearshore model.
3. The phase resolving wave model XBeach-NH (non-hydrostatic) was used to simulate wave transformation from the 15 m depth shoreward. XBeach-NH resolves the velocity and water level of individual waves, providing accurate representation of wave shoaling, breaking, refraction, diffraction, reflection, and surf-zone dynamics such as currents and wave setup. The model was implemented in quasi-3D mode, with 2 vertical layers to improve dispersion behaviour. A total of six simulations were used as set out in Table 3-3. Simulations include waves from the SSW and WSW for the 100-year ARI event, with different wave periods and sea level scenarios that could influence armour sizing. Wave output points were included to extract time-series data on water level at 10 Hz, at 100 locations around the South Mole. Timeseries data were used to calculate wave height in the sea-swell frequency band (periods below 25 s), and mean water level from wave setup. Gridded outputs for wave height, water level and velocity were also extracted to QA the model behaviour (e.g. Figure 3.3).

Table 3-2: Extreme wave heights from different offshore wave directions

ARI	All Directions	SSW	WSW	WNW
1	4.40	3.44	4.27	3.05
10	5.57	5.03	5.26	3.67
50	6.34	6.35	5.90	4.10
100	6.67	6.95	6.17	4.27

Table 3-3: Wave model scenarios

Simulation	Offshore significant wave height (Hs, m)	Peak wave period (Tp, s)	Peak wave direction (Dp, °)	Water level (m MVD-53)
1	6.5	12	255	100 yr ARI
2		12		100 yr ARI + 0.5m SLR
3		10		100 yr ARI + 0.5m SLR
4	6.0	12	285	100 yr ARI
5		12		100 yr ARI + 0.5m SLR
6		10		100 yr ARI + 0.5m SLR

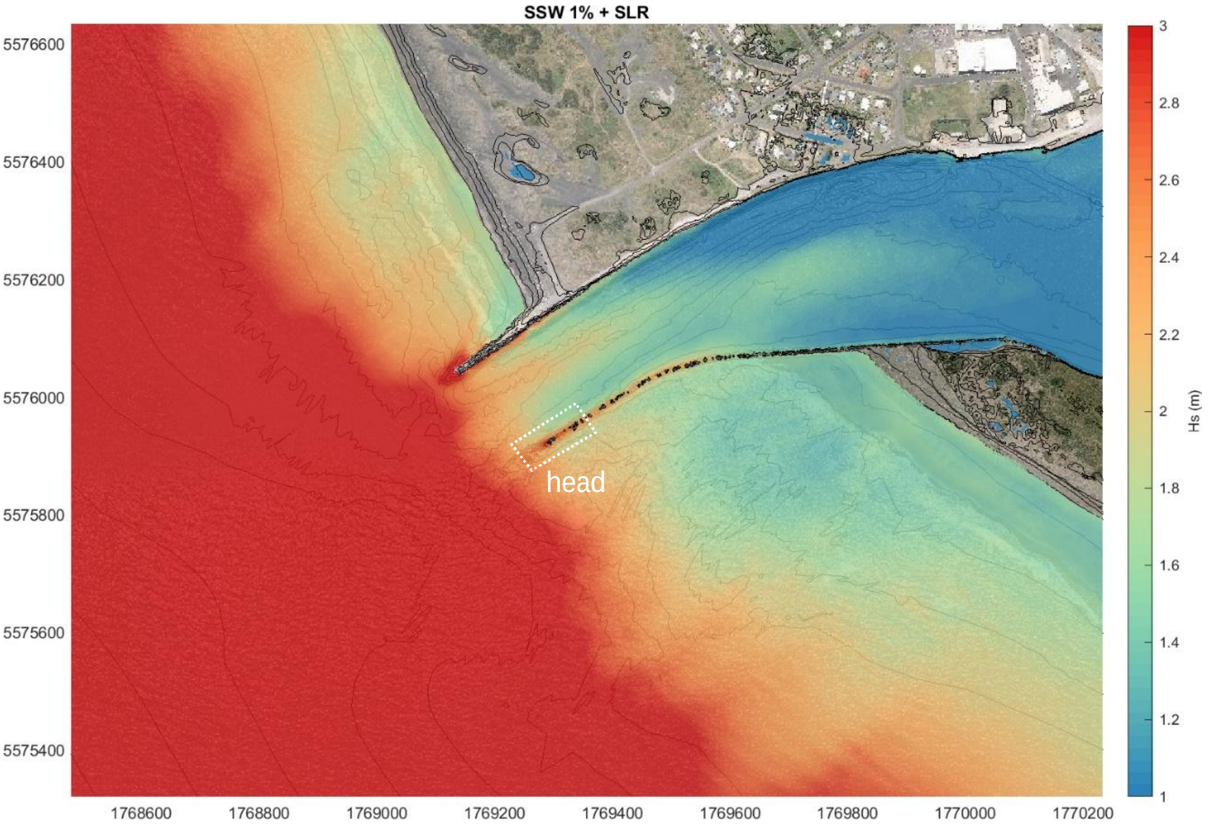

Figure 3.3: Example of the XBeach model, showing significant wave height near the Whanganui Harbour entrance for scenario 2 (see Table 3-3)

Wave height calculated from time-series output data along the seaward edge and channel edge of the South Mole are presented in Figure 3.4 and were used to inform armour sizing for the design options. This shows that the maximum significant wave height at the head is 2.9 m, and were found in similar order for all the scenarios that allow for sea level rise (i.e., scenario 2-3 and 5-6). The significant wave height reduces further landward to just below 1 m at the base of the South Mole (i.e., chainage 150). For design purpose the most seaward ~150 m has been considered the most exposed section with waves between 2.5 m and 2.9 m. This section has been considered the head of the structure with a design wave height of 2.9 m. The remaining length of the structure has been considered the trunk of the structure with a design wave height of 2.4 m.

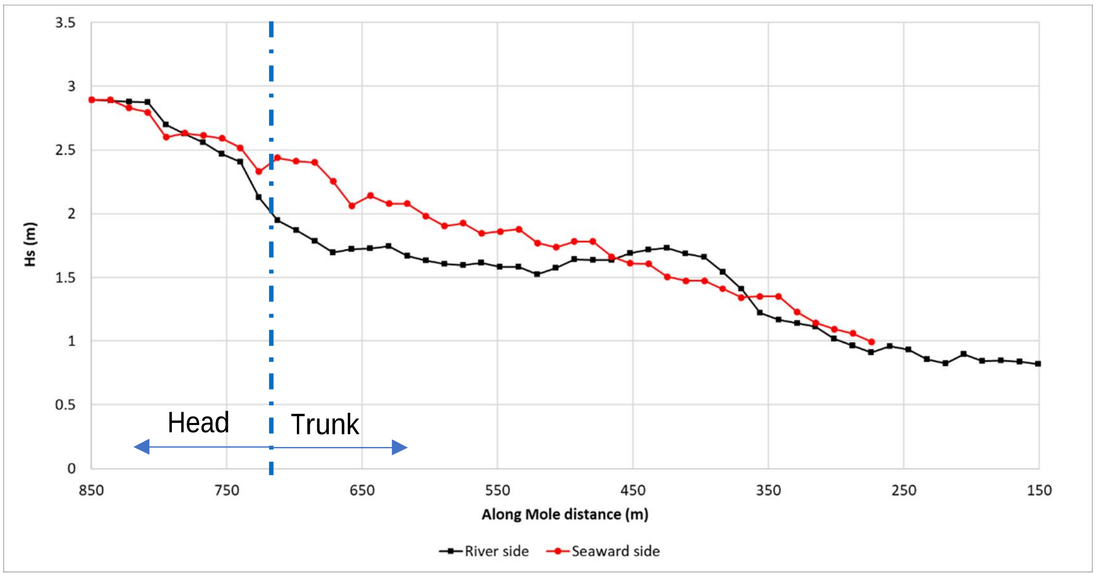

Figure 3.4: Maximum significant wave heights calculated from all scenarios from time-series outputs along the South Mole (see chainage in Figure 3.2)

3.3 Currents

Measured flow data in the vicinity of the South Mole is not available. A high-level hydrodynamic modelling exercise undertaken by T+T in 2016 shows a resulting depth-averaged velocity of up to 1 m/s near the inner bend on the river side of the South Mole. Velocities were modelled up to 2.5 m/s at the centre of the River. Figure *3.5* shows resulting depth-averaged velocity plots for the flood and ebb flow scenarios. It should be noted that an uncalibrated model was used, and the actual river flows may be higher or lower than the modelled flow velocity.

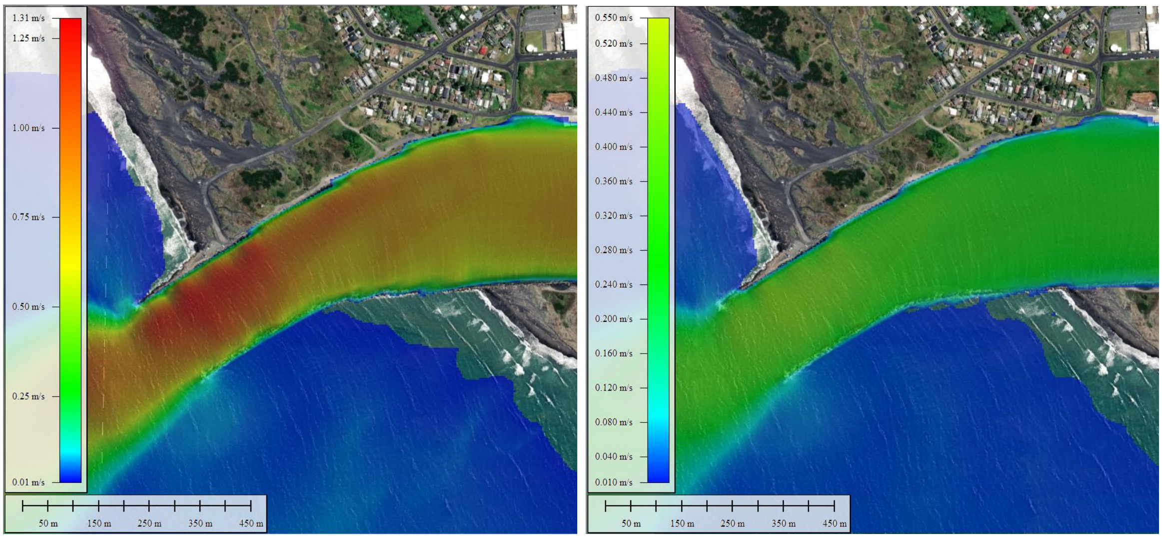

Figure 3.5: Depth-averaged current velocities for flood and ebb flows. The velocity magnitude is coloured (m/s), with direction indicated with arrows. Source: T+T (2016).

4 South Mole design options and considerations

Possible options to upgrade the existing South Mole are dependent on:

- the rock source,
- required rock size and volumes,
- preferred crest level and geometry, and
- possible construction methods.

These considerations have been set out in this section and can used as the basis for selecting a preferred option (set out in Section 5).

4.1 Rock sources

Several rock sources (i.e., quarries) are situated within a reasonable distance to supply rock for the South Mole upgrade works. The following rock types are available:

- Shell rock
- Andesite rock
- Dolomite rock

Quarries producing shell rock are generally located nearest to Whanganui, quarries supplying Andesite rock are typically located in South Taranaki and the Central Plateau, and quarries supplying Dolomite rock is in the Golden Bay (South Island). A summary of rock parameters for possible quarries based on testing data provided within this and previous projects is set out in Table 4-1.

Table 4-1: Rock sources including quality parameters

Parameters	Recommended Specifications	Shell rock		Andesite rock				Dolomite
		Bins Quarry	Waitahinga	Waiteika Road	Nukumaru	Wiremu Road	Byfords-Raetihi	GB Dolomite
Density (t/m³)	2.53	2.47	2.53	2.6-2.77	2.6-2.71	2.67	2.68	2.84
Absorption	<2%	3.8	2.9	0.4-2		1.2	1.1	
Soundness	<9%	15.5				1.1	2.9	
Abrasion	<25%	41	55			23	25	
Weathering	AA, AB, BA	CA		BA		AA	BA	CA

Table 4-1 sets out the quality parameters for the possible rock sources including recommended minimum requirements (i.e. general rock specifications) typically used for coastal structure projects. The parameters for each of the sources have been colour coded based on whether they meet the minimum requirements (i.e. green hatch) or not (i.e. yellow hatch), with a grey hatch where data is unavailable. Results show that both the Wireme Road and Byfords – Raetihi sources meet the five requirements included in the table and would be suitable to be used to upgrade the structure with minimal degradation of the rocks required over the next decades. It is understood that HRC have used both quarries for upgrading the North Mole. The two other Andesite rock quarries also meet the requirements for the parameters for which data is available but additional tests for water absorption, soundness and abrasion may be required to assess the properties of these materials.

The Shell rock and Dolomite rock sources do not meet the minimum requirements for one or more parameters. However, these rock sources could still be considered if HRC is willing to accept the lower parameter and potential impact on the structure over its life span. It is noted that shell rock has been used in the current structure which has remained in place for over 100 years, although some breakage and abrasion of material has likely occurred.

4.2 Rock size

4.2.1 Design parameters

Rock armour sizing was undertaken using the method of Van der Meer (1988) as set out in the Rock Manual (CIRIA/CUR, 2007). This is the same method as used for sizing rock for the North Mole, with design formula set out in the North Mole design report (T+T, 2018). Table 4-2 sets out the adopted and considered values for sizing the armour rock.

Table 4-2: Possible design parameters

Parameter	Possible values	Comment
Wave height	· 100-year ARI	39% likelihood of occurring within 50 years
Water levels	· 100-year ARI +0.5m sea level rise	A 100-year ARI +0.5m sea level rise has been adopted for a 50 year design life
Storm duration	· 6 hours	A 6 hour storm duration has been assumed to occur over a single high tide, with sensitivity including 4 and 12 hours duration
Rock density, ρs	· 2,500 kg/m³ (Shell rock) · 2,650 kg/m³ (andesite rock) · 2,840 kg/m³ (Dolomite rock)	Adopted rock densities for 3 rock types
Notional permeability factor, P	· 0.4 for non-homogeneous armour rock, including underlayer rock · 0.6 for homogeneous armour rock	Assumed a non-homogeneous structure adopting P = 0.4, as the concrete blocks and potential use for underlayer material (i.e., to create construction platform) likely reducing permeability compared to homogeneous structure (i.e., P = 0.6). P = 0.6 has been assessed for sensitivity purposes. No core or geotextile is expected to be used on the South Mole and so a lower permeability value (i.e. as used on the North Mole) has not been considered.
Damage criteria, S	· 2 (minor damage) · 4 (dynamic reformation)	<2% damage during design event has been adopted, with 5% damage (S = 4) to allow for some displacements of rocks, but keeping the structure dynamically stable to check sensitivity
Slope	· 1.5(H):1(V) · 2(H):1(V)	Existing slope is approx. 1.5(H):1(V) has been adopted including a sensitivity check for a 2(H):1(V) slope

4.2.2 Resulting rock sizes

The resulting rock sizes, using the input parameters as set out in Table 4-2 with the Andesite rock as control scenario, have been set in Table 4-3 and Table 4-4 for the head and trunk sections respectively. Results have been provided for the head and trunk sections as set out in Section 3.2. Note that the actual rock sizes may reduce along the two sections as a result of the decreasing wave height.

Table 4-3: Resulting rock sizes and weights for head section of the training wall

Sensitivity	H_s (m)	T_m (sec)	Depth (m)	Permeability	Damage (-)	Duration (hrs)	Density (kg/m³)	D_{n50} [m]	W_{50} [kg]	Diff Dn_{50}	Diff W_{50}
Control	2.9	9.5	6	0.4	2	6	2650	1.38	7021		
Slope (2:1)	2.9	9.5	6	0.4	2	6	2650	1.20	4609	87%	66%
Permeability	2.9	9.5	6	0.6	2	6	2650	1.07	3272	78%	47%
Damage	2.9	9.5	6	0.4	4	6	2650	1.20	4632	87%	66%
Duration	2.9	9.5	6	0.4	2	4	2650	1.33	6217	96%	89%
	2.9	9.5	6	0.4	2	12	2650	1.48	8644	107%	123%
Density	2.9	9.5	6	0.4	2	6	2500	1.52	8857	110%	126%
	2.9	9.5	6	0.4	2	6	2840	1.24	5400	90%	77%

Table 4-4: Resulting rock sizes and weights for the trunk section of the training wall

Sensitivity	H_s (m)	T_m (sec)	Depth (m)	Permeability	Damage (-)	Duration (hrs)	Density (kg/m³)	D_{n50} [m]	W_{50} [kg]	Diff Dn_{50}	Diff W_{50}
Control	2.4	8.5	*4.1*	0.4	2	6	2650	1.14	3927		
Slope (2:1)	2.4	8.5	4.1	0.4	2	6	2650	0.98	2465	86%	63%
Permeability	2.4	8.5	4.1	0.6	2	6	2650	0.89	1848	78%	47%
Damage	2.4	8.5	4.1	0.4	4	6	2650	0.99	2591	87%	66%
Duration	2.4	8.5	4.1	0.4	2	4	2650	1.09	3478	96%	89%
	2.4	8.5	4.1	0.4	2	12	2650	1.22	4835	107%	123%
Density	2.4	8.5	4.1	0.4	2	6	2500	1.26	4954	110%	126%
	2.4	8.5	4.1	0.4	2	6	2840	1.02	3021	90%	77%

The results show that the required rock weight for Andesite rock is W_{50} = 7T (range 3-14T) at the head, with W_{50} = 9T for Shell rock and W_{50} = 6T for Dolomite rock. The rock weight could be reduced up to 34% by allowing intermediate damage with dynamic reformation of the structure.

The required rock size along the trunk (i.e., 150-730 m) for Andesite rock is W_{50} = 4T, with W_{50} = 3T for Dolomite rock and W_{50} = 5T for Shell rock. We note that the existing measured rock along the trunk section are in the order of W_{50} = 3-4T and this rock size would therefore be suitable for upgrading the South Mole structure. Using notably smaller rock to augment the structure is not recommended as the smaller rock would not interlock well with the larger rock, potentially decreasing stability and leading to loss of material. Therefore, smaller rocks are not considered. However, an option could be to use Shell rocks that are slightly smaller than the existing rocks (i.e., W_{50} = 2-3T and D_{50} = 1.1-1.3 m) landward of chainage 500.

4.3 Crest level

The required crest level of the South Mole depends on the required performance of the structure in functions such as training/constricting river flows, reducing wave energy from the south beach side, and providing navigation guidance to vessels entering the river and also the proposed construction methodology. At present the crest of the structure (discounting the most seaward submerged section) ranges from approximately -0.5m MVD-53 (~mean low water neap) to 3.5m MVD-53, or around 2 m above MHWS (refer to Figure 2.4 and Figure 2.5).

At a minimum, the existing level of service could be maintained (i.e., on average 1.4 m MVD-53 seaward of chainage 500) and allow for future sea level rise (e.g., 0.5 m for ~50 years). The 0.5 m sea level rise is based on MfE (2017) for at least a 50-year timeframe and adopting the RCP8.5M scenarios. The crest level at 1.9 m MVD-53 would ensure that the structure is visible for navigation in most conditions and continues to train the river during most tidal conditions. However, this would still allow waves to overtop the structure from the South Beach side, potentially be hazardous for navigation during a high tide with storm surge or elevated river levels or future sea level rise, and if construction is undertaken using land-based machinery, would be difficult to construct.

Another option may be to raise the existing crest level to the 1970 average crest level (estimated at 2.3 m MVD-53 in 1970) with an additional allowance for sea level rise (e.g., 0.5 m for a ~50-year design life). This would result in a crest level of 2.8 m MVD-53. It should be noted that it is unknown whether the original structure was sufficient in providing an adequate level of service. Also, the fact that it has been topped up several times since it was raised to above MHWS either indicates that it was not providing sufficient protection or the rock was degrading and getting dislodged, or a combination of both.

It may therefore be preferable to raise the crest to higher levels. This could possibly range between 3.3 m MVD-53 (i.e., maximum 1970 crest level), and 4.4 m MVD-53 (i.e., maximum 1970 crest level including 1.1 m of sea level rise). The 1.1 m sea level rise is based on MfE (2017) for at least a 100 year timeframe and adopting the RCP8.5M scenario. Note that crest levels within this range could be considered and higher crest levels could possibly be considered but would likely significantly increase the required rock volume.

In summary, several crest levels that could be considered include:

1) Maintain existing level of service including allowance for 50 years sea level rise:
   - MHWS (current seaward level) + 0.5 m sea level rise= 1.9 m MVD-53
2) Average 1970 crest level including allowance for 50 years sea level rise
   - ~2.3 m MVD-53 + 0.5 m = 2.8 m MVD-53
3) Maximum 1970 crest level excluding allowance for sea level rise
   - 3.3 m MVD-53
4) Maximum 1970 crest level including allowance for 100 years sea level rise
   - 3.3 MVD-53 + 1.1 m = 4.4 m MVD-53

Figure 4.1 shows a sketch of minimum crest level and possible maximum crest level including MHWS and extreme water levels to illustrate possible crest levels with respect to these water levels.

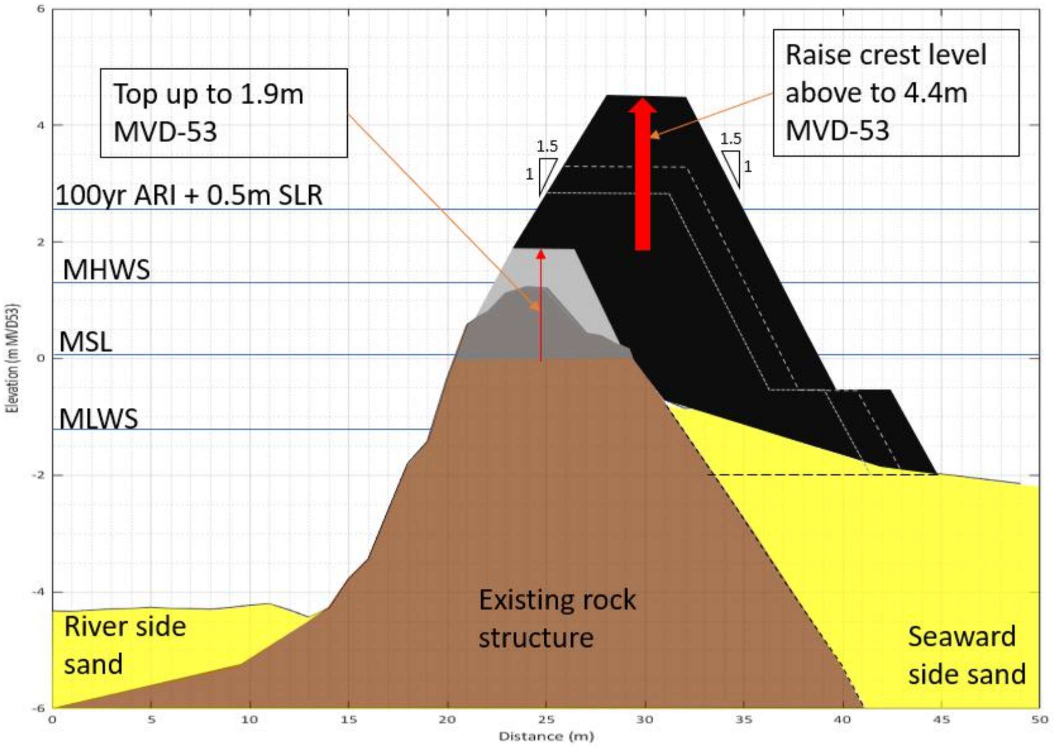

Figure 4.1: Schematic showing possible crest levels on top of existing example profile (chainage 700-800)

4.3.1 Theoretical wave transmission for different crest levels

Wave transmission through rubble mount structures depend on a number of variables including the structure crest level. Lower crest levels typically have increased wave transmission during storm events, with higher crest levels typically reducing wave transmission. It is understood that there is limited wave penetration through the South Mole structure compared to what is coming up the river. However, to provide an estimate on theoretical wave penetration/transmission for various crest levels and water levels, the van der Meer and d'Angremond (1991) empirical formula has been utilized. Theoretical wave transmission for various crest levels are set out in Table 4-5, including the MHWS and 100 year ARI water levels using 100 year ARI wave heights. The existing crest level along the seaward ~400 m is approximately 1.4 m MVD-53 has been included for reference.

Table 4-5: Theoretical wave transmission based on van der Meer and d'Angremond (1991)

Crest level	Water level	Freeboard (m)	Wave transmission
1.4	MHWS	0	74%
	MHWS + 0.5m SLR or 100yr ARI	-0.5	82%
1.9	MHWS	0.5	66%
	MHWS + 0.5m SLR or 100yr ARI	0	74%
2.8	MHWS	1.4	52%
	MHWS + 0.5m SLR or 100yr ARI	0.9	60%
3.3	MHWS	1.9	44%
	MHWS + 0.5m SLR or 100yr ARI	1.4	52%
4.4	MHWS	3	25%
	MHWS + 0.5m SLR or 100yr ARI	2.5	34%

This shows that theoretical wave transmission is relatively high for crest levels around or just above MHWS (i.e., 1.4 and 1.9 m MVD-53) and reduces with the increase of the crest level. Note that actual wave transmission may differ depending on structure composition, varying bed levels and crest levels.

4.4 Construction

4.4.1 Construction approach

There are likely two approaches for construction; marine-based (using barge-mounted excavator) and land-based (using truck and excavator). The construction methodologies are set out below at a high-level including the main limitations and sketches shown in Figure 4.2. Both options have been discussed with several contractors and their feedback has been considered in the summaries below.

- Marine based construction

  This would utilise an excavator mounted on a barge to place rock on the crest and/or river side of the existing structure. Barges would be brought in as close to the structure as possible, though a jack-up barge would likely be required for stability and due to the size of rock and reach required. A second barge would be required to supply rocks brought in from nearby stockpile areas.

  Advantages with this approach would be the potential for less material to be placed if a top-up only is required, and to minimise disturbances on south beach, the south spit and on the seabed on the south side of the structure.

  Disadvantages with this approach would be the high cost of marine plant, limitations in the size and reach of barge-mounted excavator limiting the size and location of rock that could be placed, potential difficulties deploying spuds with rock located at varying depths below the seabed and therefore potential for the jack-up barge needing to be located some distance off the structure, shallow depths at low tide limiting access for the barge containing rock and the likely significant downtime due to wave and river flow conditions.

  Overall this approach was not preferred by the contractors spoken to (including contractors specialising in both marine and land-based construction) and it seems likely that while material volumes may be lower using this approach for topping up the structure only, costs could be higher due to the limitations and uncertainties outlined above.

- Land based construction

  This would involve creating a platform on the top and seaward side of the South Mole from the beach out to the structure head, with a platform level likely just above MHWS (i.e., 1.9 m MVD-53) or around MHWS (i.e., 1.4 m MVD-53) if some downtime is acceptable. This platform would be comprised of smaller rock which can be tracked/driven over, or of larger rock with a smaller blinding layer. The platform would need to be armoured on its seaward side with larger armour rock though will generally be protected on the river side by the existing structure (though some top ups may be required in low locations). The platform may need to be in the order of 4-6m wide depending on the final rock size and size of required construction plant. The platform will be constructed progressively towards the head and once the platform has reached its full extent, the structure can be closed in using armour rock and bringing the crest up to the preferred level as the machinery tracks back landward. It is possible that part of this platform could be located on top of the existing structure, although the structure is generally too narrow to accommodate the whole structure and existing piles would need to be removed. Furthermore, the existing rock slope on the river size is relatively steep and could be de-stabilised by the additional loading.

  Advantages of this option are more certainty on construction timelines and potentially less weather downtime (although large storm events may still limit work or cause damage to the partially complete structure), less impact on the existing structure preserving ecological values and limiting risk of geotechnical instability, a wider final structure providing more resilience in the long-term and being less reliance on interlocking with existing rock and concrete.

Disadvantages include the requirement for significantly larger volumes of rock to create the working platform, a larger footprint in the CMA (Coastal Marine Area) on the seabed adjacent the south mole, more impact on land with construction plant (and likely rock) being moved along south beach and potentially set down above MHWS on the south spit.

Overall this approach was preferred by the contractors spoken to and is likely to be the more dependable and likely cost-effective option of the two.

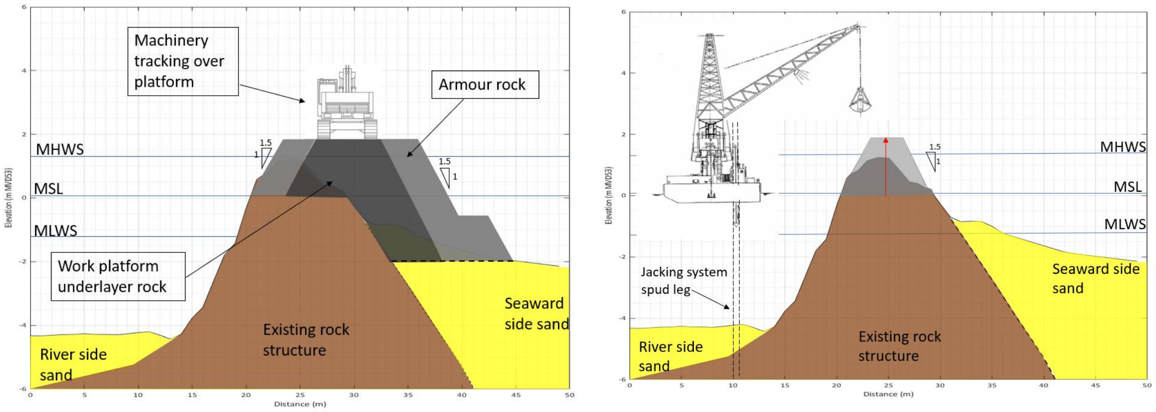

Figure 4.2: Construction methodology sketches for land-based construction (left) and marine-based construction (right)

4.4.2 Constructability

This section sets out the constructability of achieving the crest levels set out in Section 4.3.

1) Maintain existing level of service including allowance for 50 years sea level rise
   Crest level 1.9m MVD-53:

   If a crest level of 1.9 m MVD-53 (i.e., existing seaward crest level +0.5 m sea level rise) is preferred, this would require restacking existing rocks to form a foundation and placing imported rocks to raise the crest. Note that this would be limited by the existing crest level and geometry and would likely only be possible for crest levels up to around MHWS at the seaward end of the structure. This option would require *marine-based construction* (i.e. from barges along the river side) as the width of the existing structure is likely insufficient for land-based construction plant. Figure 4.3 (top left) shows the likely top up including re-stacking of existing rocks for an example profile at chainage 700-800 m. As noted in Section 4.4.1 marine-based construction is challenging with the constructability of this option considered very difficult to achieve.

2) Average 1970 crest level including allowance for 50 years sea level rise
   Crest level 2.8m MVD-53:

   In order to achieve a crest level of 2.8 m MVD-53 marine-based or land-based construction approaches could be considered.

   a) If *marine-based construction* is selected a side slope on the river side may need to be created to provide a stable structure. This would include placing double layer of armour rock along existing river side slope from the seabed up to the existing crest, then raising the crest to the preferred crest level. As existing rocks are buried between 1-2 m under the sandy riverbed, the armour rock would be required to extend below the seabed to the buried rocks.

As the bed levels on the river side of the structure are generally 2-3 m lower than on the seaward side this option would require a substantially higher structure. Figure 4.3 (top right) shows a sketch of topping up armour at crest including the riverside side slope. As noted in Section 4.4.1 marine-based construction is challenging with the constructability of this option considered very difficult to achieve.

b) If *land-based construction* is selected a platform on the seaward side of the existing structure is required, possibly comprised of smaller (underlayer) rock for machinery to track over (as barge access is not likely possible on this side). As shown in Figure 2.1 remnants of the piled rail track are still part of the existing structure, which would make it difficult to create a platform over the existing crest. A double layer armour rock would be placed on the outer sides of the platform, with armour rock stacked on top of the platform and adjacent existing structure to reach the preferred crest level. Information is not currently available on whether rock is buried below the sandy seabed, although given rock was side tipped towards the river side it is less likely to exist at depth here. A falling apron toe is therefore recommended on the seaward side to accommodate changes in seabed levels. This structure would therefore be lower in elevation compared to the river side, but likely much wider.
Based on the required armour rock size (i.e., W_{50} = 7T at the head) the platform would need to sit at 1.4 m MVD-53 (i.e., MHWS) to overlay a single layer of armour rock to achieve a crest level of 2.8 m MVD-53. This would likely result in downtime during high tide with some wave action overtopping the structure. The constructability of this option is therefore considered moderately difficult to achieve. Figure 4.3 (bottom left) shows a sketch of the concept of this option

3) Maximum 1970 crest level excluding allowance for sea level rise
Crest level 3.3m MVD-53:

In order to achieve a crest level of 3.3 m MVD-53, both *marine-based* (3a) or *land-based* (3b) construction approaches could be considered. Constructability of this option is largely the same as set out for Option 2 above apart from an increase in crest levels and rock volumes. For the land-based construction approach the platform can be raised to 0.5 m above MHWS (i.e., 1.9 m MVD-53), which would reduce the downtime compared to option 2. This would allow construction during high tides with some minor wave action. Therefore, the difficulty of constructability for the land-based construction approach would reduce to likely achievable. Figure 4.3 (top right and bottom right) shows the concept of these options.

4) Maximum 1970 crest level including allowance for 100 years sea level rise
Crest level 4.4m MVD-53:

In order to achieve a crest level of 4.4 m MVD-53, the *land-based construction* approach is required. Constructability of this option is largely the same as set out for options 2b and 3b above apart from an increase in crest levels and rock volumes, with the platform at 1.9 m MVD-53 and a double layer of armour rock overlaying the platform. Figure 4.3 (bottom right) shows the concept of this option.

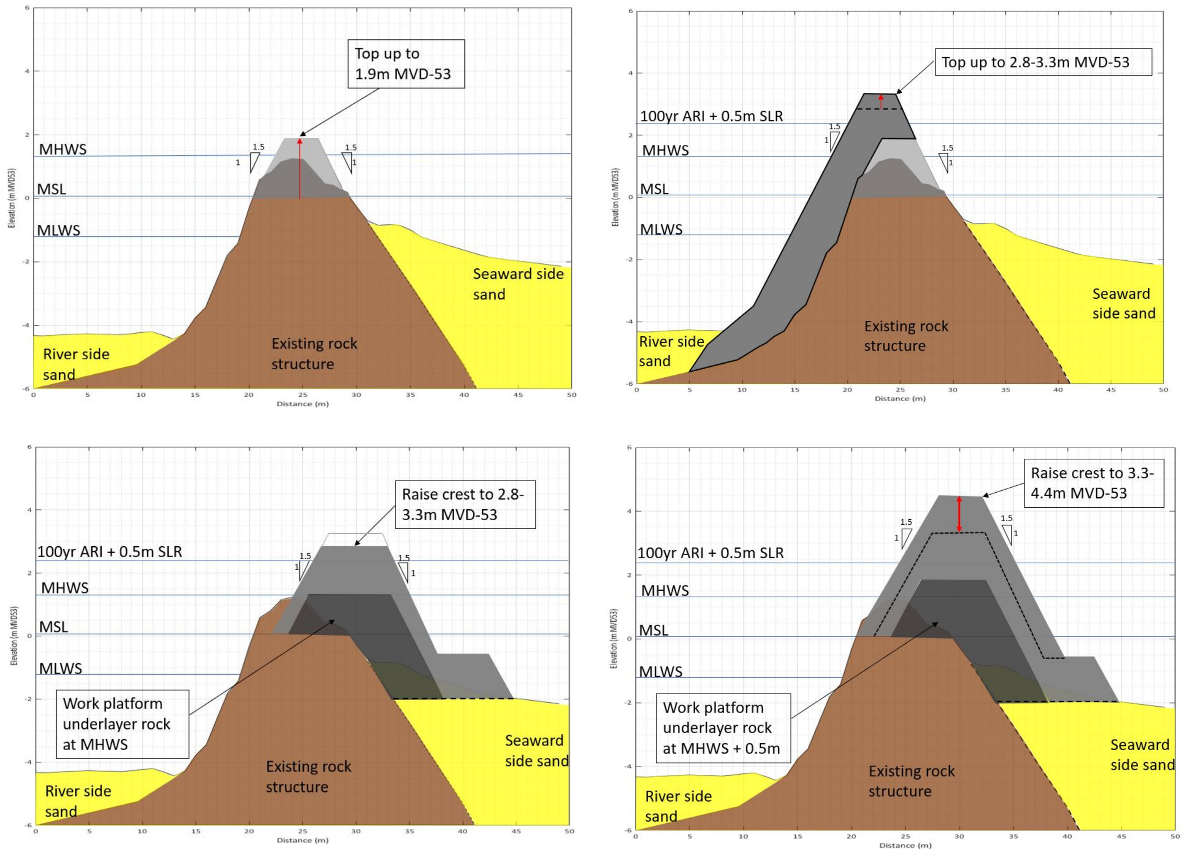

Figure 4.3: Geometry sketch of maintaining existing level of service including allowance for 50 years of sea level rise (top left), marine-based upgrade to average or maximum 1970 crest level including allowance for 50 years of sea level rise (top right), land-based upgrade to average or maximum 1970 crest level including allowance for 50 years of sea level rise (lower left), and land-based upgrades to maximum crest level including allowance for 100 years of sea level rise (lower right)

4.5 Rock supply and logistics

Method of transporting rock to site, stockpiling and supplying to the works site will depend on a number of factors including source of rock, construction methodology and environmental and stakeholder considerations and preferences. It is likely that rock could be supplied to site in the following ways:

- Transport by barge

  Rock could be loaded onto small barges, likely from Whanganui Port, but potentially also from further up river, and landed near the base of the South Mole, potentially using the Tanae groyne when completed (see Figure 4.4). This material could then be stockpiled at the base of the South Mole above MHWS for placement. This option would have the advantage of avoiding potentially ecologically sensitive areas along the south spit, but could be subject to weather delays including floods or large storms, may result in increased handling and would likely require a main set down area elsewhere before the barge is loaded.

- Transport by along the beach

  Rock supplied by truck could be stockpiled at an easily accessible location near the works – likely in an area west of the airport (see Figure 4.4). Rock would then be delivered to site by smaller 6 wheelers (or similar) tracking along the beach and delivering to the base of the South Mole either for direct placement by an excavator (assuming a land-based construction methodology) or stockpiled again above MHWS. This approach would likely provide the most certainty in delivery (provided the 6-wheelers can navigate the soft sand), though transport may be limited to lower tides and beach close to the South Mole is known to be very soft. Additionally, there may be environmental, community and cultural considerations.

- Transport along south spit.

  An alternative route through the dune system may be possible, with an existing, informal track already running most of the way from the informal car park (and previous rock set-down area) northwest of the airport runway, along the back of the foredunes (see Figure 4.4). The track would likely need to be widened and potentially surfaced with a metal to allow truck access. This route would provide the certainty of land-based transport without the uncertainties associated with the beach access. It is acknowledged that there may be environmental, community and cultural considerations with this option.

Figure 4.4: Site map including potential stockpile areas and access routes

4.6 Options for ecological enhancement of the structure

Several options existing for ecological enhancement of the structure. This section addresses selected possible options and comments on the engineering implications and suitability.

a Preferential selection of rock type

Lower strength rock such as Shell rock could be used in preference to harder Andesite or Dolomite. We understand (EOS pers. comm) that this may provide some ecological benefit as lower strength rock becomes more easily colonised, as evidenced by the existing shell rock material

b Selective placement of rock

We understand that the existing structure currently provides marine habitat. Rather than disturbing (by removal and re-packing) or covering the existing structure, the new rock could be placed adjacent to the existing structure (i.e. Figure 4.3, lower panels) to minimise disturbance.

c Addition of ecological units

EOS Ecology assessed possible ecological features that could be incorporated in the South Mole upgrade that would ecologically enhance the structure (refer to EOS Ecology, 2021). Three types of rock pools are discussed including (see examples in Figure 4.5):

1. Tide pools (ECOncrete or similar)
2. Ecologically enhanced armour units (ECOncrete Coastalock or similar)
3. Tide pools drilled into existing rock

The ECOncrete tide pools for example are 1.4T units that need to be placed between MHWS and MLWS and can be spread out along the structure clustered in groups of up to 5 tide pools. As the weight of the unit is 1.4T these units should only be placed where the wave energy is low enough (i.e., likely towards the base of the structure). As no design formula is available for this unit the exact location along the structure cannot be provided. Interlocking with armour rocks is likely limited as a result of the smooth unit shape. However, the risk of affecting the integrity of the structure as a result of single units being dislodged is low as the units are surrounded by armour rocks.

The ECOncrete Coastalock units are 3.4T units that need to be placed between MHWS and MLWS in lines of interlocking units in order to provide a stable structure. These units could possibly be placed along the trunk of the structure. However, these units need to be placed on a smooth, prepared surface which may be difficult to achieve given the existing rocks and voids within the existing structure. Similar to the ECOncrete tide pools no design formula is available, so the stability of the units cannot be reviewed. The risk of affecting the integrity of the structure is significantly higher compared to the ECOncrete tide pools as the Coastalock units rely on interlocking. If a single unit gets dislodged the risk of other units getting dislodged increases.

Drilled tide pools are created by drilling holes into armour rocks larger than 450 mm that sit within the intertidal zone (i.e., MLWS to MHWS). The drilled holes are approximately 150 mm in diameter with a depth of 50-120 mm, with up to 4 holes drilled into a single rock. EOS Ecology (2021) suggest drilling holes in 10-25% of the rocks. As armour rocks are typically larger than 450 mm, holes could potentially be drilled in rocks along the entire length of the structure. However, it should be noted that drilling holes in armour rock could potentially affect the integrity of the rocks and result in fracture or splits in the rocks if lower strength rocks are used (e.g., Shell rock).

Figure 4.5: Photo of ECOncrete tide pools (left), artist impression of ECOncrete Coastalock (middle) and photo of drilled rock pool (right) (source: EOS Ecology)

5 Summary of South Mole upgrade options

This section sets out the possible upgrade options taking into account the options and considerations set out in Section 4. A summary of the possible options including estimated rock volumes assuming Andesite rock is set out in Table 5-1. Table 5-1 sets out two marine-based construction options and two land-based construction options. For all options W_{50} = 7T rock has been adopted for the head section (i.e., chainage 730-850) and W_{50} = 4T for the trunk section (i.e., chainage 150-730). Adopting another rock source or different design parameters may possibly result in slightly different crest levels and rock volumes.

Required rock volumes have been estimated using a GIS tool (i.e., Global Mapper) by creating surfacing including selected crest levels, crest width (i.e., minimum 3 rocks wide) and 1.5(H):1(V), and subtracting this from the bathymetry DEM. A manual check was undertaken using basic geometry and math. Note that the estimations are limited to the quality of the spatial data available, with uncertainties around rock levels along the seaward side of the structure and voids between the rocks (not picked up in DEM) reducing the accuracy of the estimations. The estimated volumes therefore include 10-20% additional volume to allow for uncertainty.

For the maintain existing level of service allowing for 0.5 m sea level rise option (i.e., top up to 1.9 m MVD-53) a rock volume of 5,000-10,000 m^3 have been estimated, which requires marine based construction. The volume significantly increases if the crest level is raised to 2.8 or 3.3 m MVD-53 for marine based construction as a result of needing to create a stable side slope along the existing structure. The estimated rock volume for a crest level at 2.8 m MVD-53 is 18,000-28,000 m^3 and at 3.3 m MVD-53 is 20,000-30,000 m^3.

For the land-based construction methods the estimated rock volumes are 15,000-25,000 m^3, 20,000-30,000 m^3, and 30,000-40,000 m^3 for crest levels at 2.8, 3.3 and 4.4 m MVD-53 respectively. For all options the larger rock at the head is 15-20% of the total volume with the smaller rock the remaining 80-85%.

Table 5-1: Summary of possible upgrade options including required rock volume

Option		Crest level (MVD53)	Rock size[1]	Construction approach	Constructability considerations	Required rock volume (m^3)
1	Maintain existing level of service including allowance for 50 years sea level rise:	1.9 m	W_{50} = 7T at head W_{50} = 4T at trunk	Marine based construction top up existing structure	· Marine-based construction would likely incur significant downtime · Marine-based construction considered very difficult due to limited reach, large rock sizes and difficulty with anchoring barges	5,000-10,000
2a	Average 1970 crest level including allowance for 50 years sea level rise	2.8 m	4-7T	Marine-based construction river side slope and crest top up	· Marine-based construction would likely incur significant downtime · Marine-based construction considered very difficult due to limited reach, large rock sizes and difficulty with anchoring barges	18,000-28,000
2b		2.8 m	4-7T	Land-based seaward side platform (MHWS) and top up	· Working platform at MHWS would restrict construction windows to low tides and low wave conditions · Single layer of armour rock overlaying platform	15,000-25,000
3a	Maximum 1970 crest level excluding allowance for sea level rise	3.3 m	4-7T	Marine-based construction river side slope and crest top up	· Marine-based construction would likely incur significant downtime · Marine-based construction considered very difficult due to limited reach, large rock sizes and difficulty with anchoring barges	20,000-30,000
3b		3.3 m	4-7T	Land-based seaward side platform (MHWS + 0.5 m sea level rise) and top up	· Platform 0.5 m above MHWS would reduce construction downtime during high tide compared to Option 2b · Single layer of armour rock overlaying platform could be susceptible to damage in future.	20,000-30,000
4	Maximum 1970 crest level including allowance for 100 years sea level rise	4.4 m	4-7T	Land-based seaward side platform (MHWS + 0.5 m sea level rise) and top up	· Platform 0.5 m above MHWS would reduce construction downtime during high tide compared to option 2b · Double layer of armour rock overlaying platform improving long-term resilience	40,000-50,000

[1]Based on Andesite rock

6 Applicability

This report has been prepared for the exclusive use of our client Horizons Regional Council, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

Tonkin & Taylor Ltd

Report prepared by:	Authorised for Tonkin & Taylor Ltd by:
....	....
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Dr Eddie Beetham Senior Coastal Scientist	
Seth Smith Coastal Engineer	

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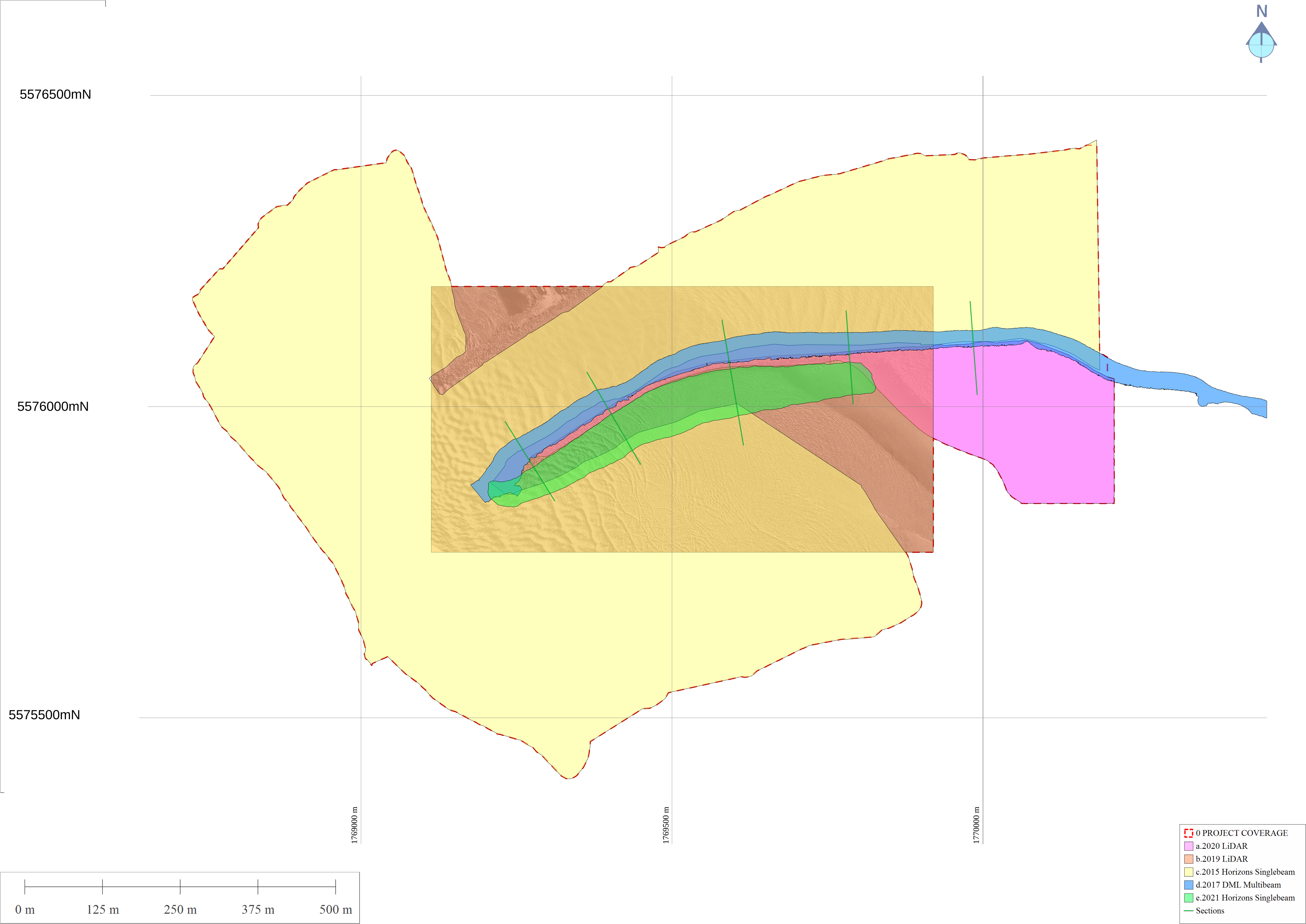
Appendix A: Bathymetry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5576000mN
5575500mN
1769000 m
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1770000 m
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125 m
250 m
375 m
500 m
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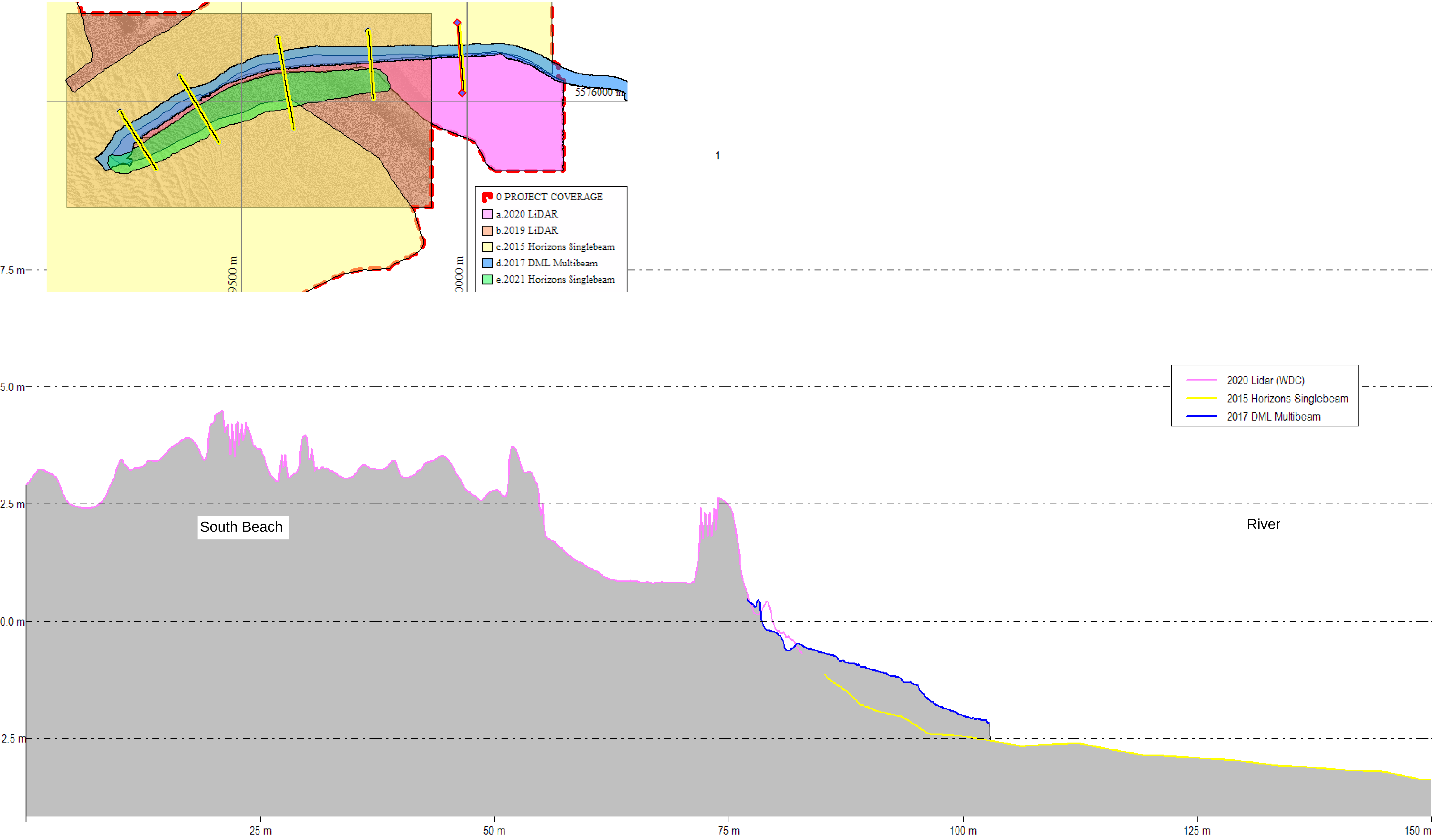
b.2019 LiDAR
c.2015 Horizons Singlebeam
d.2017 DML Multibeam
e.2021 Horizons Singlebeam
Sections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c.2015 Horizons Singlebeam
d.2017 DML Multibeam
e.2021 Horizons Singlebeam
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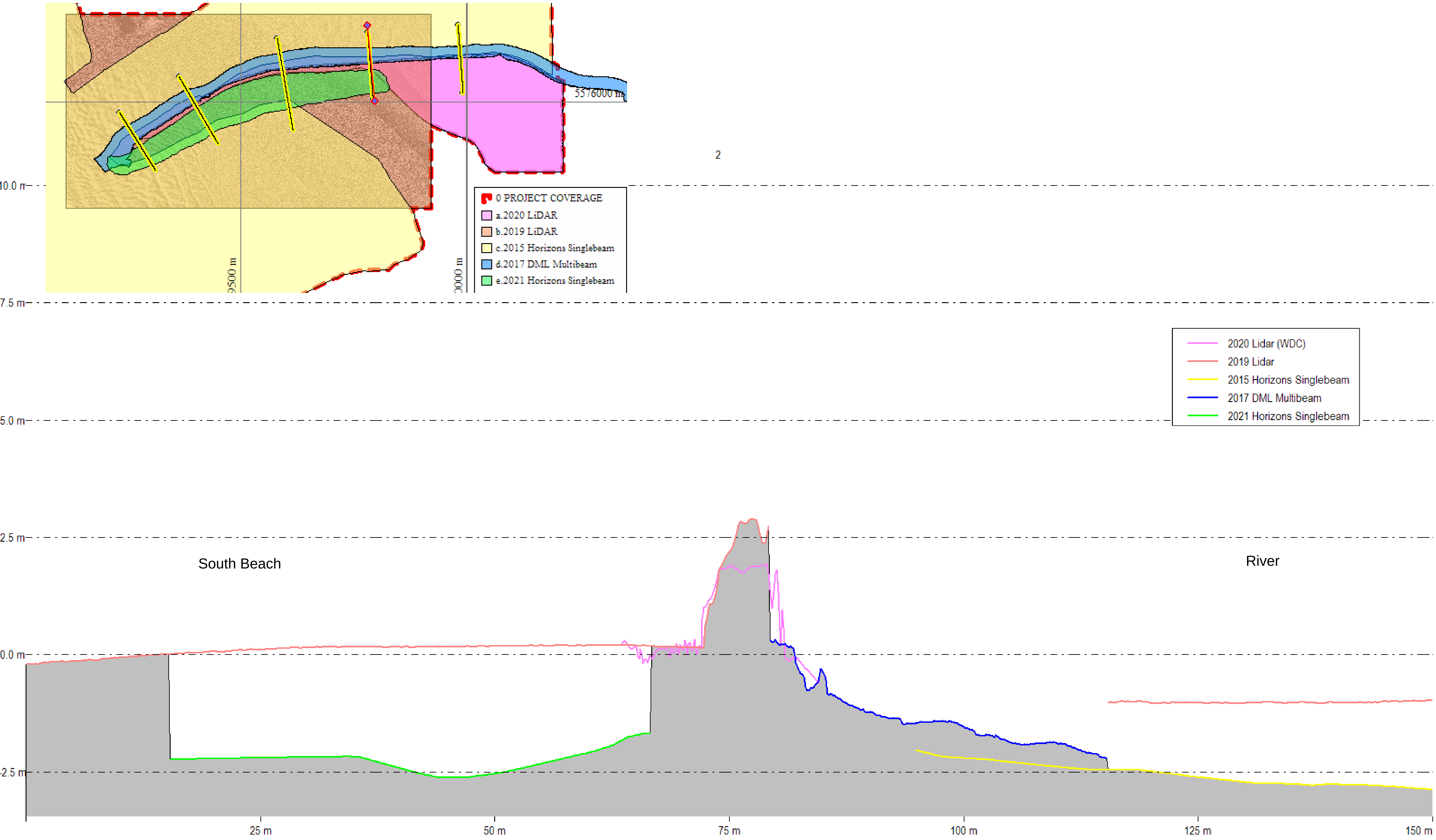
1
7.5 m
5.0 m
2.5 m
0.0 m
-2.5 m
2020 Lidar (WDC)
2015 Horizons Singlebeam
2017 DML Multibeam
South Beach
River
25 m
50 m
75 m
100 m
125 m
150 m

Section 1



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0 PROJECT COVERAGE
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c.2015 Horizons Singlebeam
d.2017 DML Multibeam
e.2021 Horizons Singlebeam
2
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2.5 m
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2020 Lidar (WDC)
2019 Lidar
2015 Horizons Singlebeam
2017 DML Multibeam
2021 Horizons Singlebeam
South Beach
River
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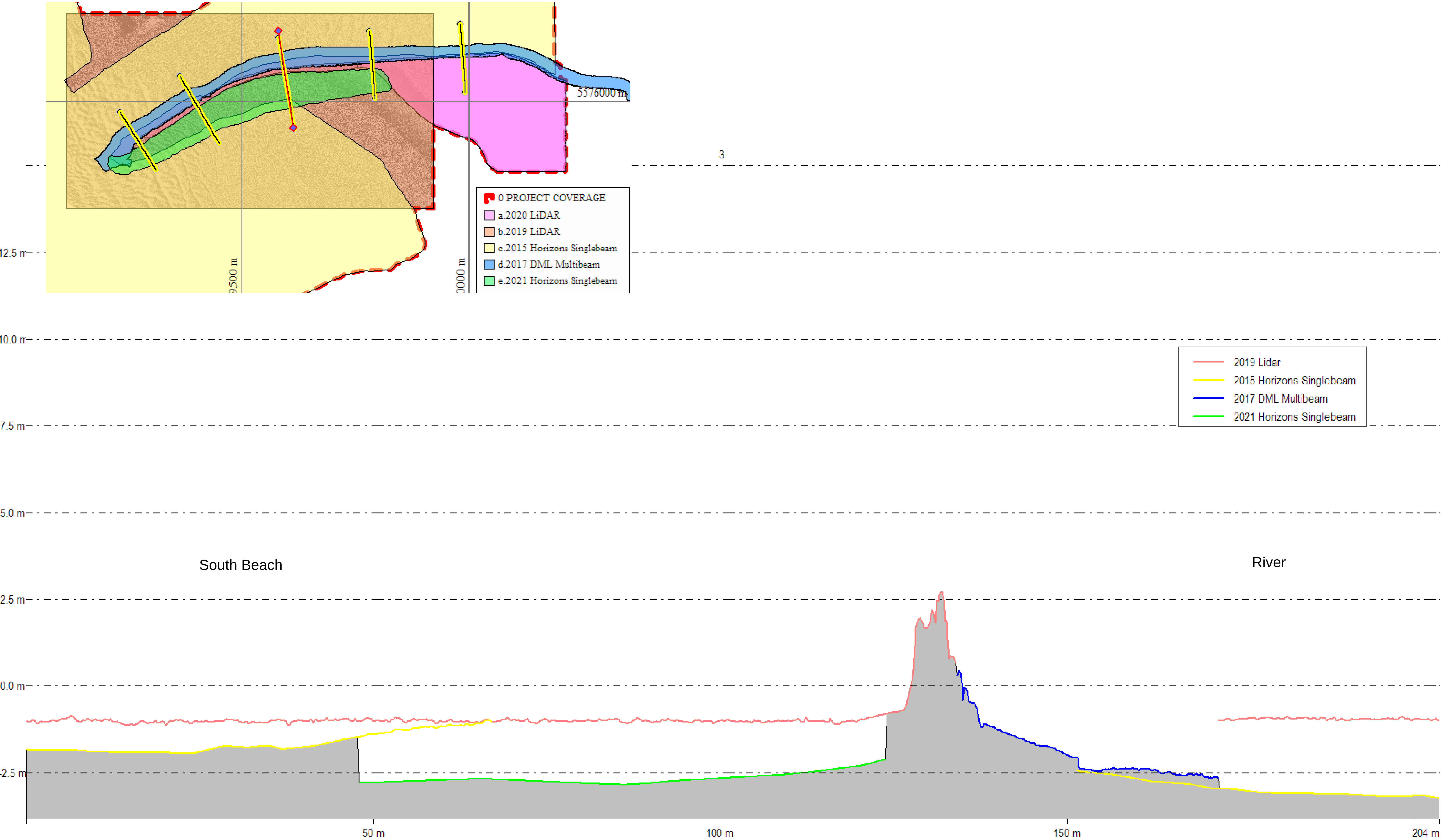
Section 2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d.2017 DML Multibeam
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5576000 m
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2019 Lidar
2015 Horizons Singlebeam
2017 DML Multibeam
2021 Horizons Singlebeam
South Beach
River
50 m
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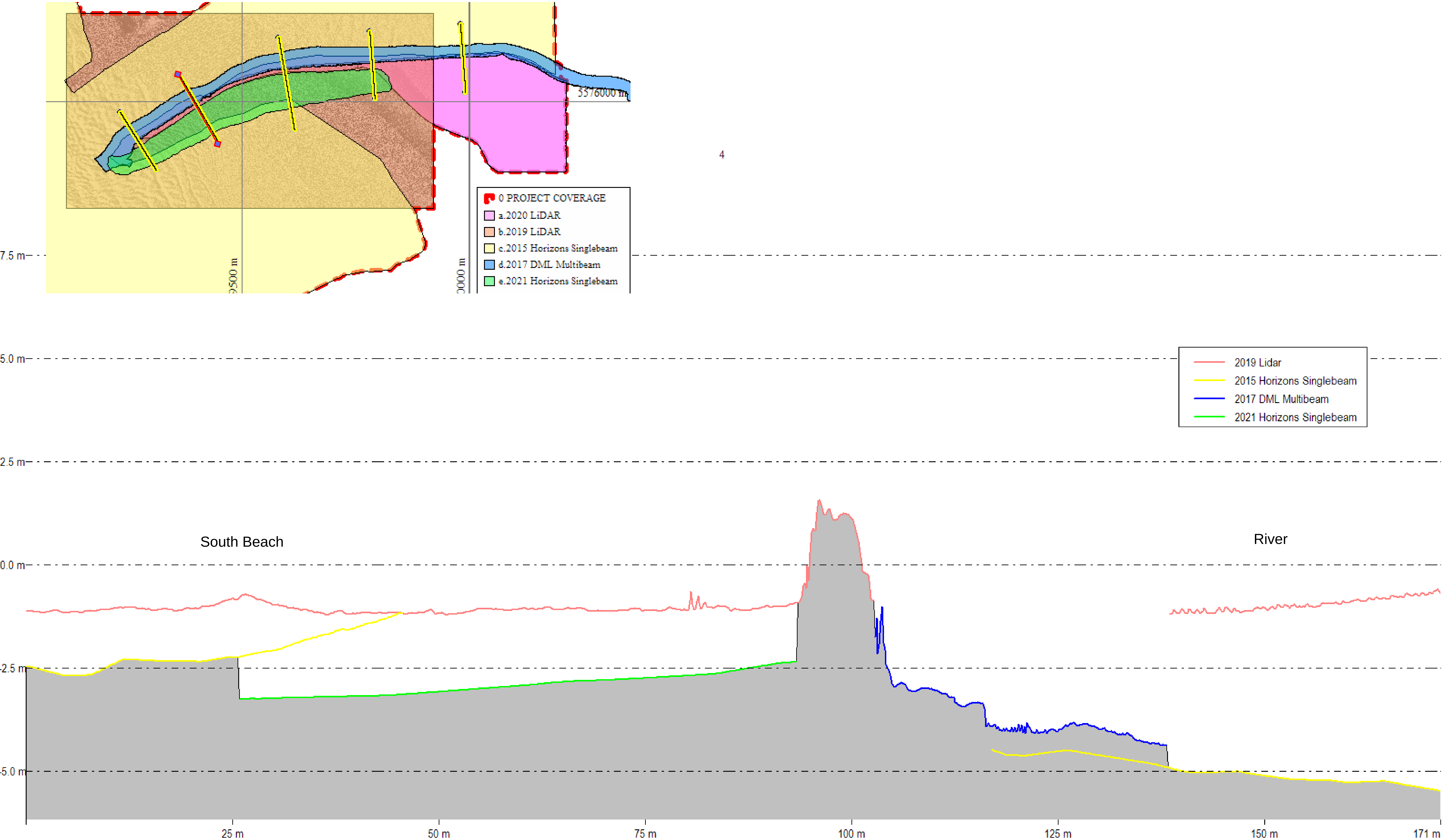
204 m

Section 3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e.2021 Horizons Singlebeam
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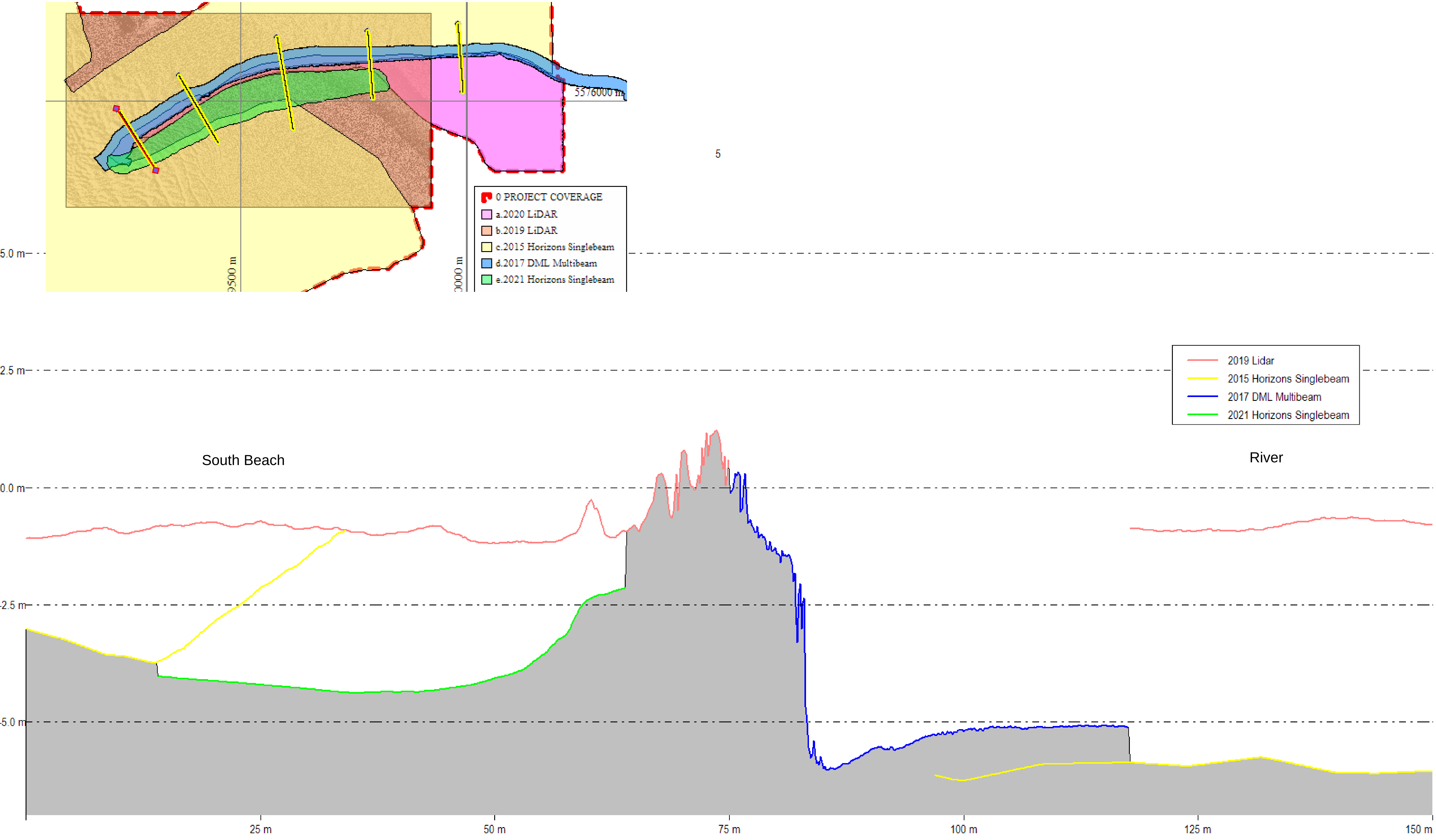
2019 Lidar
2015 Horizons Singlebeam
2017 DML Multibeam
2021 Horizons Singlebeam
South Beach
River
25 m
50 m
75 m
100 m
125 m
150 m
171 m

Section 4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2019 Lidar
2015 Horizons Singlebeam
2017 DML Multibeam
2021 Horizons Singlebeam
South Beach
River
25 m
50 m
75 m
100 m
125 m
150 m

Section 5

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