COASTAL PROCESSES REPORT

Technical Report H

Greenhithe Bridge Watermain Duplication and Causeway Project

By

Tonkin & Taylor Ltd

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Watercare
An Auckland Council Organisation

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EXECUTIVE SUMMARY

Tonkin and Taylor Ltd (T&T) has been commissioned by Watercare Services Limited (Watercare) to assess the potential coastal process effects related to the construction, operation and maintenance of Watercare's proposed Greenhithe Bridge Watermain Duplication and Causeway project.

The project comprises:

- The construction of a new watermain on the northern side of the Greenhithe Bridge to duplicate the existing North Harbour 1 Watermain already located on the southern side of the bridge, and
- Widening along the northern side of the existing State Highway 18 motorway causeway to accommodate the new watermain, as well as wastewater pipelines and associated facilities which form part of Watercare's proposed Northern Interceptor project.

This report sets out the assessment of effects on coastal process. The area of the route that impacts on coastal processes is along the south-western approach to the Greenhithe Bridge (Phase 1 extent of works as shown in Figure 2) within the Upper Waitemata Harbour.

This area is a relatively low energy environment dominated by tidal flow concentrations through the narrow channel and wind generated wave conditions on the intertidal flats. The intertidal flats are likely to be depositionary areas.

Effects

Based on the results of the coastal process study and hydrodynamic modelling it is shown that the proposed causeway widening a less than minor effect on the coastal processes operating in this area. The extension to the causeway will result in short term and localised effect on coastal processes. The resulting changes are likely to include localised lowering of sand at the tip of the causeway extension and minor levels of deposition on the intertidal areas to the north-west and along the main channel to the south-east. These depositions are unlikely to be measurable and will be within the natural fluctuation of the existing system. Apart from the localised scour around the tip of the causeway extension there will be no effects on wider seabed levels within the main channel. There will be no influence on the existing bridge structures. The effects of the proposed works are unlikely to change with increasing sea level rise.

Mitigation

Short term effects on coastal processes associated with the construction of the causeway are associated with discharge of sediments and contaminants. These effects can be mitigated by good construction techniques, including doing much of the foundation and initial construction works on either side of low tide and by use of a silt curtain where sediment discharge. The construction approaches and management and avoidance of sediment and accidental discharge should be set out in a comprehensive construction management plan and appropriate monitoring.

1 INTRODUCTION

Tonkin and Taylor Ltd (T&T) has been commissioned by Watercare Services Limited (Watercare) to assess the potential coastal process effects related to the construction, operation and maintenance of Watercare's proposed Greenhithe Bridge Watermain Duplication and Causeway project.

The project comprises:

- The construction of a new watermain on the northern side of the Greenhithe Bridge to duplicate the existing North Harbour 1 Watermain already located on the southern side of the bridge, and

- Widening along the northern side of the existing State Highway 18 motorway causeway to accommodate the new watermain, as well as wastewater pipelines and associated facilities which form part of Watercare's proposed Northern Interceptor project.

The proposed water and wastewater infrastructure is required in order to maintain water and wastewater service levels and to provide for future growth.

The proposed Greenhithe Bridge Watermain Duplication and Causeway project requires various resource consents under the Resource Management Act 1991 ("RMA"). This technical report provides specialist input for the *Greenhithe Bridge Watermain Duplication and Causeway – Assessment of Effects on the Environment* report ("the main AEE") report prepared by URS New Zealand and Jacobs New Zealand Limited which supports the resource consent application. The works described in the AEE have been considered in the technical assessment presented in this report

This report provides the following:

- A brief overview of the proposed works (in Section 2);
- A description of the environmental baseline for the physical coastal environment (Section 3);
- Description of specific aspects of the project in relation to the physical coastal processes (Section 4);
- An assessment of the actual or potential effects on the environment, having reference to the statutory framework and any other environmental factors considered relevant. This includes the identification of activities that could result in adverse effects and, in turn, identifying design refinements or construction methodologies that could avoid, remedy or mitigate such effects (Section 5);
- Recommended mitigation and management measures (Section 6).

The new watermain will eventually form part of Watercare's future North Harbour 2 Watermain project. The proposed widening of the motorway causeway will also incorporate wastewater pipelines and associated facilities which form part of Watercare's proposed Northern Interceptor project. Separate technical reports have or will be prepared for the future North Harbour 2 Watermain project and for the balance of the Northern Interceptor project.

2 GREENHITHE BRIDGE WATERMAIN DUPLICATION AND CAUSEWAY PROJECT

The GBWD works assessed in this report are the construction, operation and maintenance of:

- The new watermain from Station Street in Hobsonville, under SH18 to the coastal edge – this could involve open trenching from Station Street to the motorway, and trenchless construction (pipe jacking) under the motorway;
- Causeway widening to accommodate the new watermain and wastewater pipelines – the existing SH18 causeway will be widened along the northern side by approximately 50 metres from the edge of the cycle way for a length of approximately 860 metres;
- The new watermain attached to the underside of the Greenhithe Bridge; and
- A new watermain cross connection chamber close to the eastern abutment of the Greenhithe Bridge.

The proposed works are described in detail in the AEE and the accompanying drawing set. Key drawings showing the proposed works and construction methodology are copied in the AEE, Volume 3 - Drawings. The works described in the AEE and shown on the appended drawings are assessed in this report.

3 ENVIRONMENTAL BASELINE

The area of the route that impacts on coastal processes is along the approaches to the Greenhithe Bridge (Extent of works as shown in Figure 3-1) within the Upper Waitemata Harbour. This section describes the environmental baseline of this area.

3.1 General setting

The Upper Waitemata Harbour is at the head of a complex, deeply indented and infilled drowned-river-valley estuary. Seven shallow tidal creeks (Hellyers, Lucas, Paremoremo, Rangitopuni, Brighams, Rarawaru, Waiarohia) drain into the main body of the Upper Waitemata Harbour, which in turn connects with the relatively broad and open Middle Waitemata Harbour (refer Figure 3-1). The basement rock of the Upper Waitemata Harbour, over which modern sediments have been deposited, is uneven and irregular. Sediments in the upper reaches of the 7 tidal creeks tend to be thinly draped, but in the lower reaches of the creeks, mudbanks exceed 10 m thickness in parts, and tend to be stabilised by mangroves along the landward margins (NIWA, 2004).

The creeks are largely intertidal, with narrow central channels. Approximately 50% of the Upper Waitemata Harbour is intertidal, and the seven tidal creeks account for approximately 50% of the area of the Upper Waitemata Harbour. At high tide, water depth over the intertidal areas is 1–2 m. Tidal currents are typically not strong enough over the intertidal flats to entrain bed sediments. The estuary sits within a valley that is aligned east–west, which crosses the dominant wind directions (southwest and northeast). Hence, it is relatively sheltered, and waves are typically small (NIWA, 2004).

Figure 3-1 Site location

3.2 Site description

The Upper Harbour Corridor (SH18) is the main east/west link between Waitakere and North Shore running from SH16 in the west to SH1 in the east. The 457 m long Greenhithe Bridge and associated causeway crosses the Upper Waitemata Harbour at its narrowest section. The original bridge and causeway reclamation was completed in 1974 and the duplicate bridge and causeway widening was completed in 2006.

3.3 Bathymetry

The bathymetry of the Upper Harbour environs as well as Lucas and Te Wharau Creek is provided in hydrographic charts (refer Figure 3-2). This chart is made up of surveys carried out in 1958-1959 (Upper Waitemata Harbour) and from 1967 to 1987 (Middle Waitemata Harbour). More detailed hydrographic surveys of the main harbour pipe crossing was carried out as part of the present study and is reported in the Coastal Model Study (Appendix C). The additional survey information in the vicinity of the bridge and causeway was carried out by Scantec Ltd. This survey data is presented in terms of AVD-46 datum, some 1.743 m higher than Chart Datum. The more detailed survey shows very similar general trends to the hydrographic chart in this area suggesting no significant change occurred from the 1960's to the present at this location.

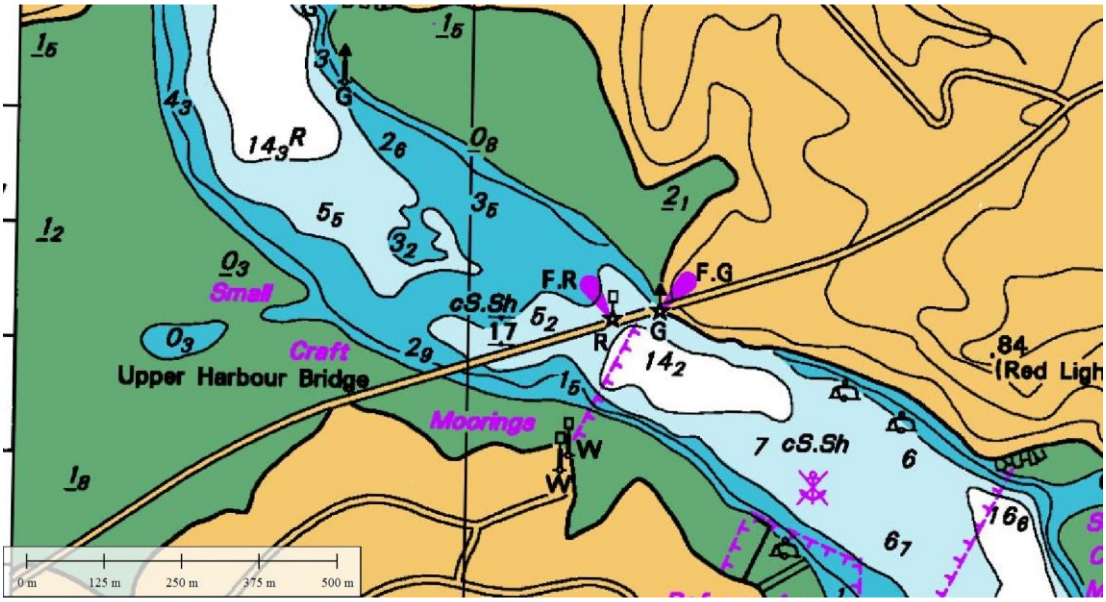


Figure 3-2 Extract from hydrographic chart (NZ 5322), depths to Chart Datum

Both the hydrographic chart and the Scantec Ltd surveys show a narrow channel with the narrowest point between the Hobsonville and Greenhithe headlands being around 250 m wide and the main channel some 200 m wide. The main navigation channel is close to the northern shoreline. There is a relatively deep ebb flow scour hole to the east of the bridge with a maximum depth of around 14.2 m below Chart Datum, although depths below the bridge are typically between 5 and 10 m below Chart Datum. The intertidal sand flats to the west of the causeway are around 0.3 to 1.8 m above Chart Datum.

An unusual feature of the Upper Waitemata Harbour is the rock sills within the intertidal zone of the Rangitopuni and Lucas Creeks. During much of the tidal cycle, these rock sills prevent upstream intrusion of estuarine water (NIWA, 2004).

3.4 Sedimentation trends

Sediments entering the Upper Waitemata Harbour are mainly sourced from soil and rock erosion within the Upper Waitemata Harbour catchment and carried to the estuary by streamflow, mainly during floods.

Additional sources are erosion of stream channels and suspended-sediment entering from the Middle Waitemata Harbour on flood tides, but both of these are relatively insignificant (NIWA, 2004).

Sediment thickness throughout the harbour can be measured against the depth to the underlying Pleistocene basement rock. Depths can vary from zero at stream headwaters to over 10 m in relict river channels formed when sea levels were more than 120 m below present levels (6000 to 16000 years before present). Marine source sedimentation started some 6000 years before present (Hume, 1983). Much of this sediment can probably be attributed to the impact of man from about 1070 years BP (before present), with a major increase in the rate of sedimentation that coincided with the start of European settlement 100 years BP.

Core samples taken by NIWA Ecosystems (1997) show that sedimentation in the Upper Waitemata Harbour before humans colonised New Zealand was around 0.03-0.04mm/year. This increased with the arrival of Polynesian settlers due to clearing of forests, and increased by 2-3 times upon the arrival of Europeans, who logged Kauri forests and carried out gum digging (NIWA Ecosystems, 1997). Large scale land use changes associated with farming and urban development after 1910 further increased sedimentation, to produce current average rates of 2.0 mm/year in Lucas Creek and 1.4 mm/year in Brigham's Creek. The mud flats of these creeks are more prone to sediment accumulation, and show much higher rates (6-9 mm/year) from 1950 onwards.

Pollen dating and radioisotope dating by Swales et al. (2002) found that approximately 300 mm of material has been deposited in the sub-tidal regions of the Waitemata Harbour since the early 1900's, and that the average rate of sediment accumulation since the 1950's has been approximately 3 mm per year.

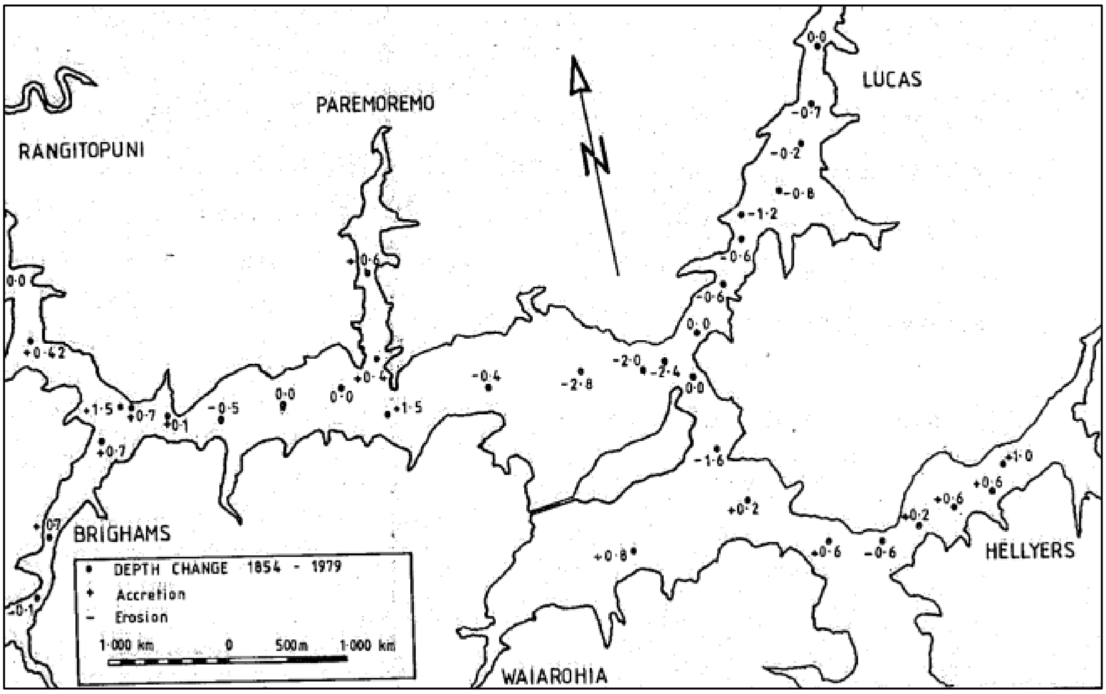


Figure 3-3 Net sedimentation in the Upper Waitemata Harbour between 1854 and 1979 based on a comparison of soundings from 1854 and 1979 (Source: Hume, 1983)

An examination of net sedimentation in the Upper Waitemata Harbour was done by comparing 1854 (refer Figure A-1, Appendix A) and 1958-59 soundings (refer Figure 3-3). Errors are likely to be in the order of 0.7 m for the 1854 survey and 0.3 m for the 1979 data (Hume, 1983). The results show little change along the main channels and Lucas Creek and no change in the vicinity of the Greenhithe

Bridge. However, some of the large erosion values are likely to be attributed to channel migration rather than significant localised erosion. There are generally consistent trends of sedimentation in the majority of the tributaries to the harbour.

3.5 Sediments

Estuarine sediment dispersal and sedimentation are determined by the characteristics of the source material (size, shape, density, mineralogy and organic content) and the dynamics of the estuarine receiving waters. Water motions (principally tidal currents and associated turbulence) mix, disperse and, in places, re-suspend sediment particles. Gravity causes sediment to settle and deposit onto the estuary bed, but opposing this are the random movements of turbulent eddies, which increase in strength as current speed increases. The tidal creeks are the focus of freshwater–saltwater mixing, which promotes flocculation of freshwater-borne suspended sediments. This, in turn, increases particulate settling speed thereby promoting deposition of suspended particulate matter. Hence, deposition of terrigenous sediments and associated contaminants is favoured in tidal creeks (NIWA, 2004).

Sediment is generally sorted throughout the estuary by the ability of finer particles to be more easily transported by weaker currents. As a result, the finer particles are more likely to settle in very sheltered areas, such as the upper reaches of the tidal creeks and mangroves. Once deposited, fine sediment may dehydrate and consolidate, which makes it more resistant to subsequent erosion. Coarser particles, which can be moved only by the stronger currents that typically occur in channels, are more likely to remain within those channels. However, coarser particles are also found at the entrances to the creeks, where they have been deposited in the aftermath of large floods and where tidal currents are not energetic enough to re-suspend and re-disperse them (NIWA, 2004). Figure 3-4 shows the surficial sediment texture of the upper harbour environment from historic analysis.

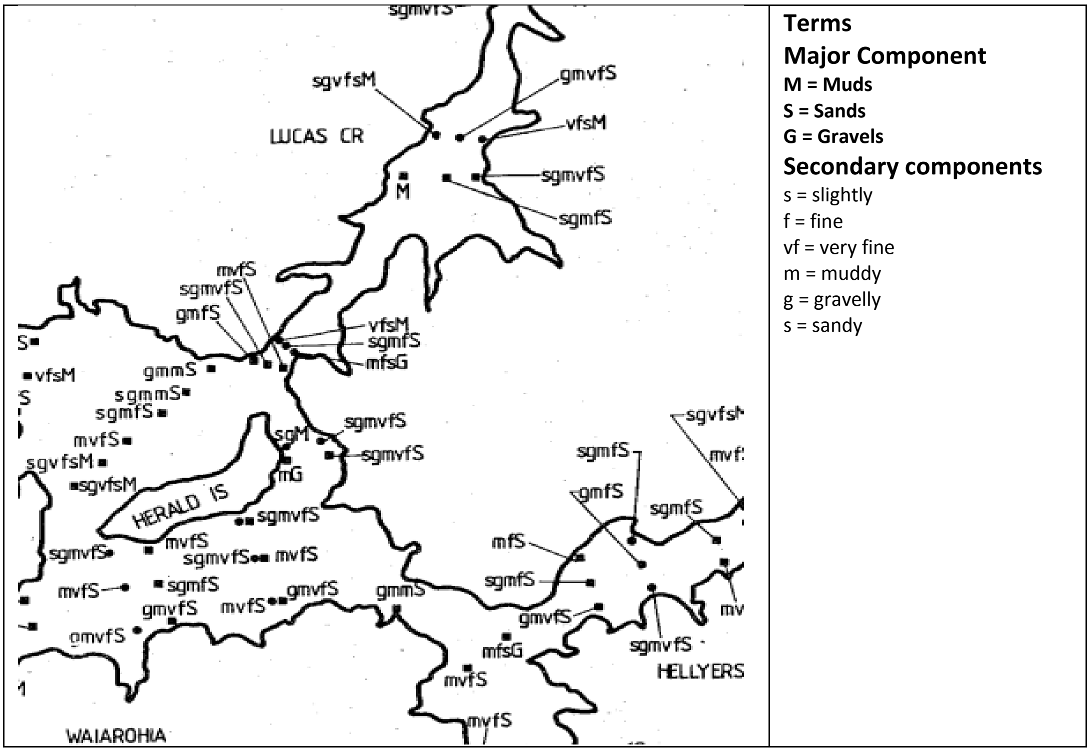


Figure 3-4 Surficial sediment texture (Source: Hume, 1983)

Sediment sampling was carried out during the investigations for the bridge duplication and additional investigations were carried out by Meritec in 2002 and T&T in 2014 and 2015. Locations of the samples are shown in Figure 3-5 and results of surficial and near surficial sediment gradings are shown in Figure 3-6 and Figure 3-7 and tabulated in Table 3-1 and Table 3-2.

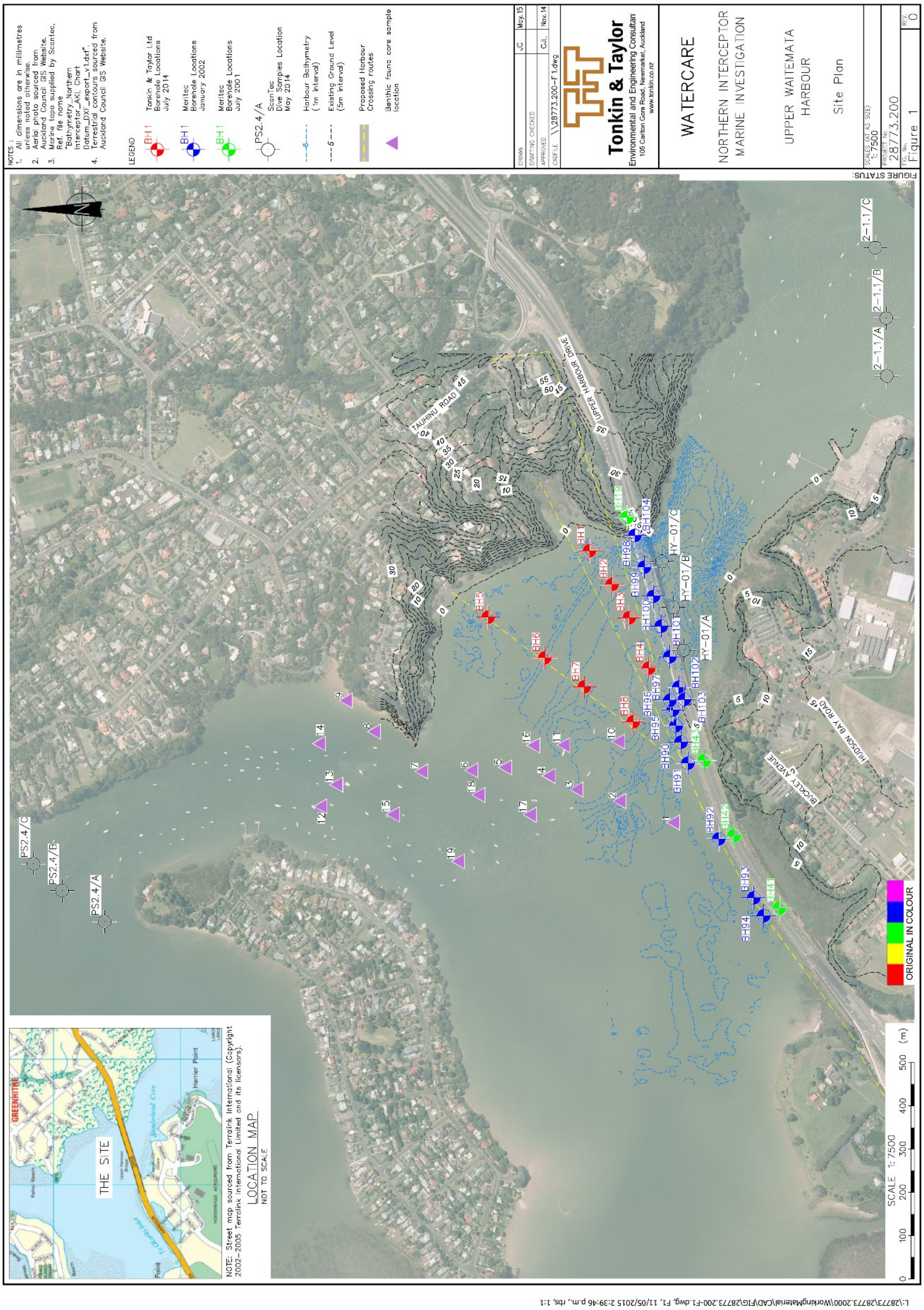


Figure 3-5 Location of seabed testing

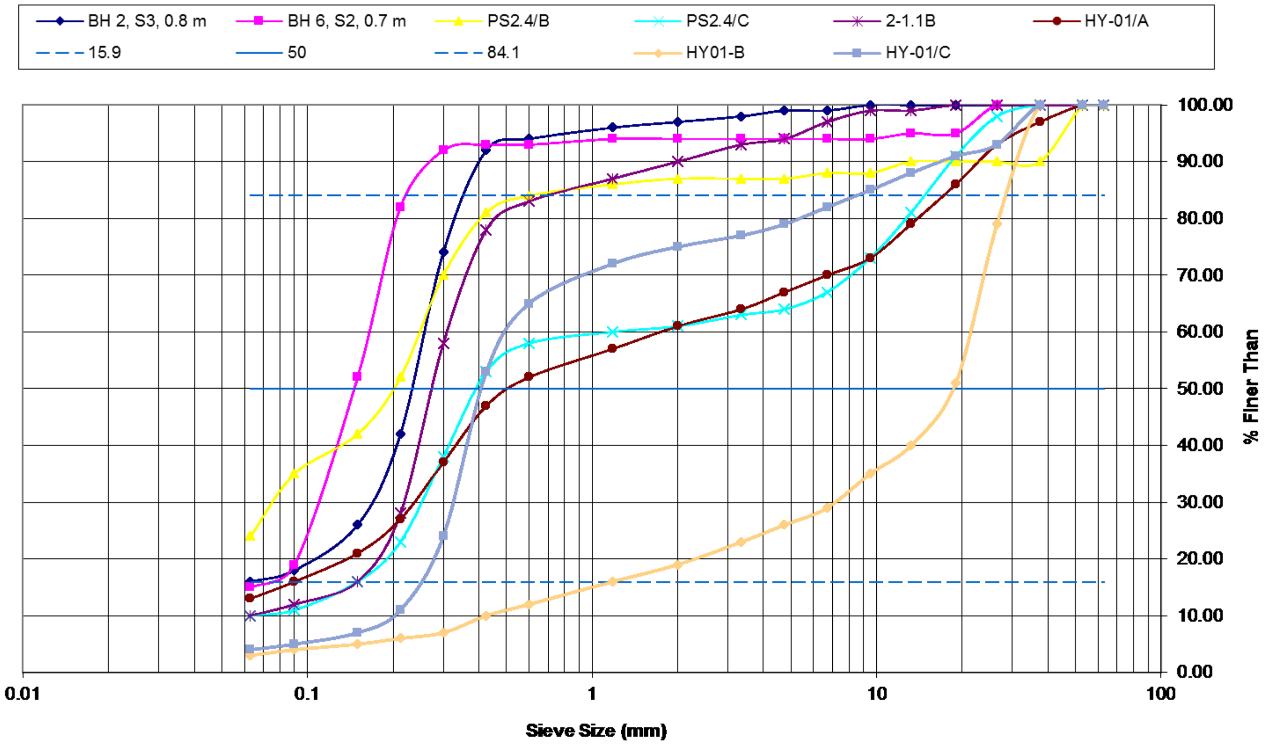


Figure 3-6 Sediment grading information from historic sources

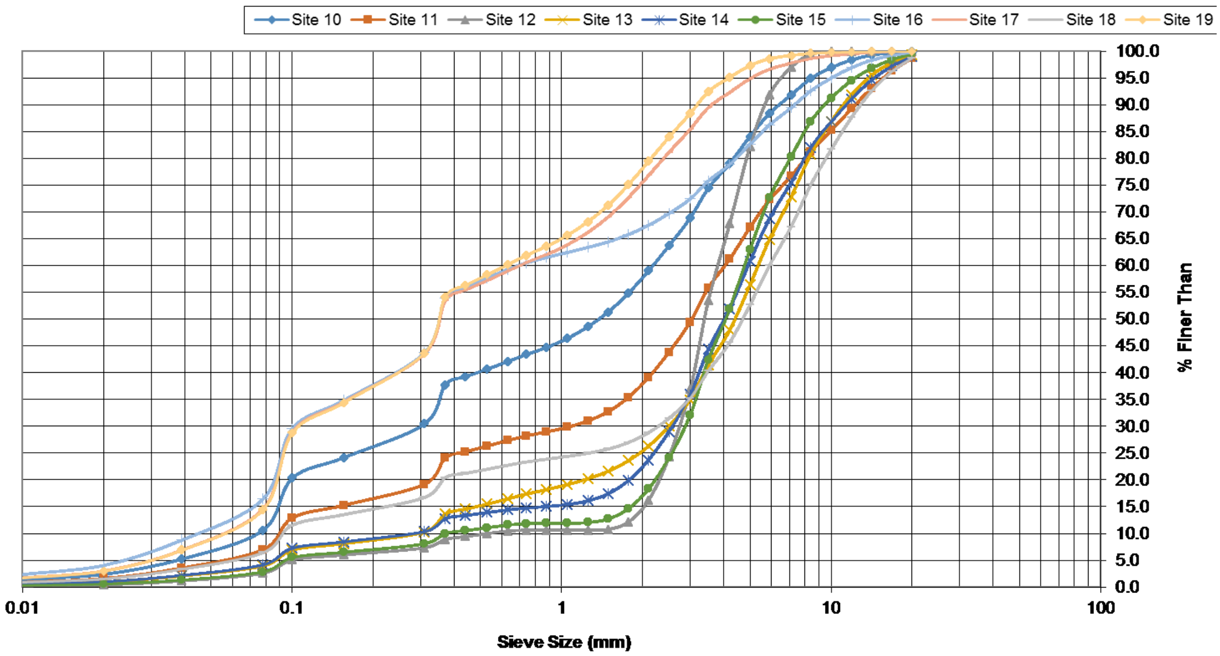


Figure 3-7 Sediment grading curve for marine samples taken in 2014 and 2015

Table 3-1 Percent of sediment passing of marine samples from historic samples

Sieve (mm)	BH 2, S3, 0.8 m	BH 6, S2, 0.7 m	PS2.4/B	PS2.4/C	2-1.1B	HY-01/A	HY01-B	HY-01/C
<0.063	0	0	0	0	0	0	0	0
0.063	16	15	24	10	10	13	3	4
0.09	18	19	35	11	12	16	4	5
0.15	26	52	42	16	16	21	5	7
0.21	42	82	52	23	28	27	6	11
0.30	74	92	70	38	58	37	7	24
0.43	92	93	81	53	78	47	10	53
0.60	94	93	84	58	83	52	12	65
1.18	96	94	86	60	87	57	16	72
2.00	97	94	87	61	90	61	19	75
3.35	98	94	87	63	93	64	23	77
4.75	99	94	87	64	94	67	26	79
6.7	99	94	88	67	97	70	29	82
9.5	100	94	88	73	99	73	35	85
13.2	100	95	90	81	99	79	40	88
19.0	100	95	90	91	100	86	51	91
26.5	100	100	90	98	100	93	79	93
37.5	100	100	90	100	100	97	100	100
53.0	100	100	100	100	100	100	100	100
63.0	100	100	100	100	100	100	100	100

Table 3-2: Percent of sediment sampling marine samples taken in 2014 and 2015

size (mm)	Site 10	Site 11	Site 12	Site 13	Site 14	Site 15	Site 16	Site 17	Site 18	Site 19
20	99.9	98.8	100.0	99.3	99.1	99.5	99.8	100.0	98.8	100.0
16.8	99.7	96.3	100.0	97.9	97.2	98.4	99.2	99.9	96.2	100.0
14.1	99.3	93.1	100.0	95.5	94.6	96.8	98.3	99.8	92.6	99.9
11.9	98.4	89.3	100.0	91.9	91.1	94.5	96.9	99.6	87.7	99.9
10	96.9	85.2	100.0	86.9	86.9	91.3	95.0	99.2	81.7	99.8
8.4	94.9	81.2	99.4	80.8	81.9	86.9	92.6	98.7	75.1	99.6
7.1	91.8	76.7	97.0	72.8	75.5	80.3	89.4	97.8	67.2	99.2
5.9	88.4	72.3	91.9	64.9	68.8	72.6	86.2	96.6	60.0	98.6
5	84.1	67.1	82.2	56.4	60.8	63.0	82.7	94.9	52.7	97.3
4.2	79.0	61.2	67.7	47.9	51.9	51.9	78.9	92.3	45.6	95.1

size (mm)	Site 10	Site 11	Site 12	Site 13	Site 14	Site 15	Site 16	Site 17	Site 18	Site 19
3.5	74.5	55.8	53.4	41.5	44.3	42.4	75.8	89.5	40.4	92.5
3	68.9	49.5	36.9	35.0	35.9	32.1	72.5	85.5	35.2	88.4
2.5	63.7	43.8	24.2	30.0	29.0	24.1	69.7	81.1	31.5	84.0
2.1	59.0	39.1	16.2	26.3	23.7	18.3	67.5	76.8	28.8	79.4
1.77	54.9	35.4	12.2	23.6	19.9	14.7	65.8	72.7	27.0	75.1
1.49	51.3	32.8	10.8	21.6	17.4	12.7	64.5	69.1	25.8	71.3
1.25	48.6	31.0	10.6	20.2	16.1	12.0	63.4	66.2	25.0	68.2
1.05	46.4	29.8	10.6	19.1	15.4	12.0	62.5	63.9	24.4	65.7
0.88	44.7	29.0	10.6	18.2	15.0	12.0	61.5	62.1	23.9	63.6
0.74	43.4	28.2	10.6	17.4	14.8	11.9	60.5	60.6	23.4	61.9
0.63	42.1	27.4	10.5	16.5	14.4	11.6	59.3	59.0	22.7	60.2
0.53	40.7	26.3	10.0	15.5	13.9	11.1	57.7	57.3	22.0	58.2
0.44	39.3	25.3	9.4	14.6	13.4	10.5	56.0	55.5	21.2	56.3
0.37	37.8	24.1	8.9	13.7	12.8	10.0	54.0	53.5	20.4	54.1
0.31	30.5	19.2	7.3	10.3	10.3	8.1	43.7	43.6	16.7	43.7
0.156	24.2	15.3	6.0	8.1	8.4	6.5	34.9	34.7	13.5	34.5
0.1	20.3	12.9	5.1	6.9	7.2	5.5	29.6	29.0	11.6	28.8
0.078	10.6	7.0	2.6	4.0	4.1	2.8	16.4	14.6	6.4	14.5
0.039	5.2	3.6	1.2	2.1	2.2	1.3	8.8	6.9	3.4	6.9
0.02	2.4	1.6	0.5	0.9	1.0	0.6	4.0	3.0	1.5	3.0
0.0098	1.4	0.9	0.2	0.5	0.5	0.3	2.3	1.7	0.9	1.6
0.007	0.5	0.3	0.0	0.2	0.2	0.1	0.8	0.6	0.3	0.6

In the intertidal areas the sediment generally comprised fine sands (mean size range between 0.15 mm and 0.2 mm) although shells were evident in samples in the channel and a greater proportion of silts were observed in the intertidal flat adjacent to the causeway. The more silty sediments suggest lower energy environments where currents are weaker, enhancing settling.

The critical current and bed shear stress for current induced transport was calculated using the methods of Van Rijn (1990). Based on a D50 of 0.15 mm and a D90 of 0.4 mm the resulting critical velocity for initiation of movement for currents with no wave action was 0.64 m/s and the critical bed shear stress was 0.88 N/m^2. It is noted that wave action, particularly on the shallow intertidal areas when water depth is low, will initiate sediment transport at much lower tidal flows.

3.6 Suspended sediment concentrations

Suspended solids concentrations have been measured by the former Auckland Regional Council (ARC), Ports of Auckland Ltd (POAL), and NIWA at various locations around Waitemata Harbour. Analysis of these data is shown in Table 3-3. The sediment-settling rate of suspended material was measured at 1.5 m per hour from a site within the Port of Auckland (Beca, 1997) for a type of material, broadly referred to as marine mud. Settling velocities from sediment samples taken in Bayswater Marina found that the settling velocity of the fine sand fraction was of the order of 36 m/hour while the silt and clay particles settled at a slower rate of less than 3.6 m/hour. The median sediment size from the samples was 0.077 mm and the fraction of sand was 60% (KMA, 1989).

Table 3-3 Suspended sediment concentrations within upper and middle areas of Waitemata Harbour

Location and period	Suspended sediment concentration (mg/L)
NIWA DOBIE gauge data (4 April 2006 to 8 June 2006), entrance to Catalina Channel	20 (mean)
ARC data (April 1991 to May 1997), Chelsea	12 (mean)
POAL data (August 1992 to October 1992), Westhaven Marina	12 (mean)
POAL data (March 1995 to March 1997, Westhaven Marina	9 (mean)
Fletcher Construction, Stage 1 dredging works, Bayswater Marina	8 (mean)
Entrance to Upper Harbour (4 May to 27 July 2007) at 5.1 m mean water depth (Oldman et. al, 2008)	1340 (max) 20 (90^{th} percentile)
Hobsonville Jetty monthly data from 2003 (NIWA, 2004)	7.2 (median), 4.9 (min), 25 (max)
Lucas Creek monthly data from 2003 (NIWA, 2004)	13.4 (median), 8.3 (min), 41.9 (max)

Measurements of suspended sediments on the fine sandy intertidal flats of the Manukau Harbour have identified that waves are the primary mechanism responsible for sediment re-suspension on estuarine intertidal flats at that location, with the highest suspended sediment concentrations occurring in the turbid fringe which occupies the shallow edges of the estuarine water body (Dolphin and Green, 1997). The measurements showed significant suspended sediment concentrations of 289 to 1640 mg/l) in shallow water depths even with small wave heights (less than 0.3 m). Due to the Upper Waitemata Harbour having similar flat sandy intertidal area as the Manukau Harbour these processes are also likely to occur along these intertidal areas.

3.7 Water levels

Tidal and extreme water levels adjacent to the bridge crossing are included in Table 3-4. Based on the nautical definitions, the tide follows a typical spring/neap tidal cycle, with spring range of 2.8 m and neap range of 2 m. Comparing the six-month tide record at the Salthouse Jetty in Lucas Creek at the end of Rame Road (Williams and Rutherford, 1983) with tides in the Commercial Harbour (Queens Wharf), shows that high tide occurs simultaneously at the two locations but is 0.15 m higher in the Upper Waitemata Harbour. Low tide is generally 0.12 m lower in the Upper Waitemata Harbour, and the time of low water is variable compared to Queens Wharf (+/- 20 minutes). This compares well with 0.14 m that was used to derive tide levels at Hobsonville Landing (Beca, 2009). Extreme water level information in the vicinity of the crossing is provided in NIWA (2013) coastal inundation report.

Water slopes within the Upper Waitemata Harbour are very small. The tidal prism (volume of the estuary at high water minus volume at low water) is 1.86×10^7 m^3 during spring tides and 1.00×10^7 m^3 during neap tides (NIWA, 2004).

Table 3-4 Tidal and extreme water levels adjacent to the bridge crossing

Event (%AEP/ARI)	Level (m AVD[3])
0.5% AEP (200 yr ARI)[1]	2.64
1% AEP (100 yr ARI)[1]	2.59
2% AEP (50 yr ARI)[1]	2.53
10% AEP (10 yr ARI)[1]	2.41
Highest Astronomic Tide (HAT)[2]	2.0 (2.08)[4]
Mean High Water Springs Perigean (MHWSP)[2]	1.8
Mean High Water Springs 10 (MHWS10)[2]	1.7
Mean High Water Springs Nautical (MHWSn)[2]	1.6 (1.68)[4]
Mean Low Water Spring Nautical (MLWSn)	-1.2 (-1.36)[4]
[1]Extracted from Point 87 (1749621E, 5927312N) NIWA (2013) [2]Estimated by adding 0.15 m to Queens Wharf Values from NIWA (2011) [3]Auckland Vertical Datum (= Chart Datum +1.74 m) [4]Tide levels established at Hobsonville by direct measurement (Beca, 2009)	

3.8 Winds

Wind data has been obtained from the Whenuapai Aero club using NIWA's Cliflo website. This site provides some 52 years of daily observations (refer Figure 3-8). While strong wind speeds of around 20 m/s can occur from all directions, predominant wind directions are from the west-north west to the south west. The maximum recorded wind speed of 22.6 m/s originated from the north east.

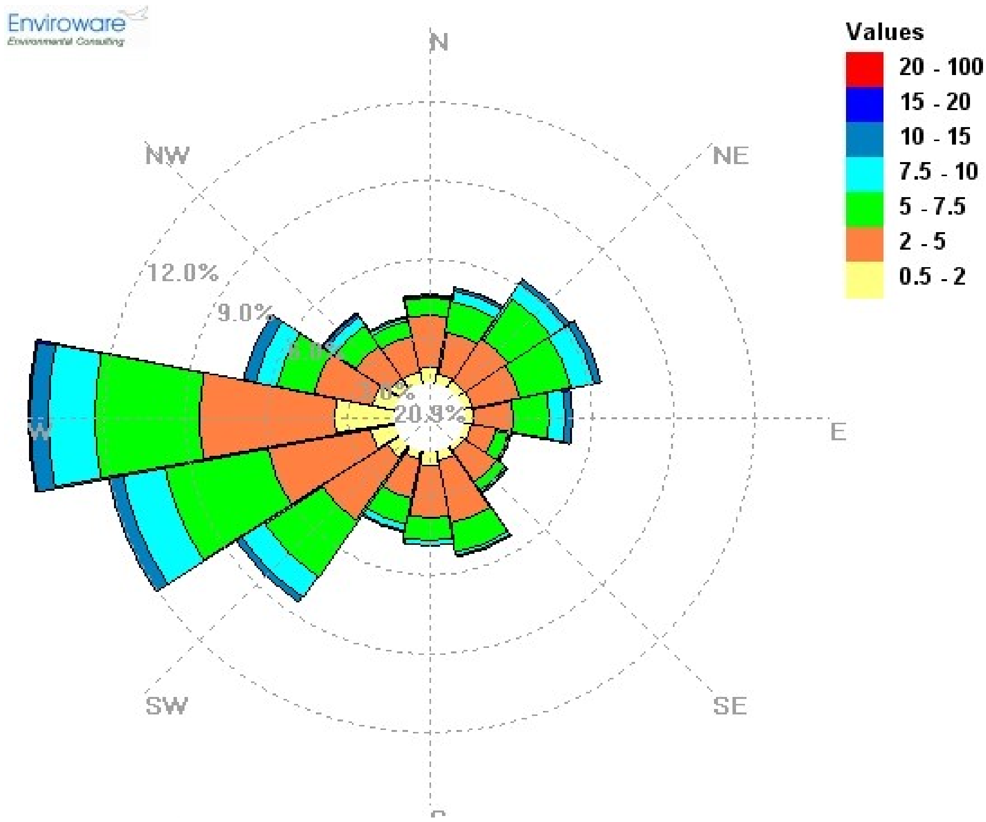


Figure 3-8 Wind rose data from Whenuapai Aero Club, Jan-1960 to Dec 2012 (Source: Cliflo)

3.9 Waves

Waves at this location are fetch and depth limited, with maximum wave heights being generated at high water levels with winds from either the south west to north east (for the causeway area) and less frequently from south-east along the main channel (for the bridge). Based on standard empirical relationships of Wilson (Goda, 2003) the significant wave height and period has been determined for a 100 year return period wind speed at MHWS (refer Table 3-5).

Table 3-5 Extreme nearshore wave heights at MHWS tide level

Wind Direction	Fetch (km)	Average fetch depth at MHWS (m)	Significant wave height, Hs (m)	Significant wave period, Ts (s)
W	1.5	2.8	0.71	2.3
NW	0.8	2.8	0.55	1.9
N	1	7.8	0.65	2.1
NE	0.8	6	0.55	2.0

Based on depth limited empirical relationships, maximum significant wave heights of between 0.5 m and 0.71 m are possible at this location, with periods of between 2 and 3 seconds along the causeway. During south easterly conditions wave heights could reach up to 1.0 m within the channel with periods of around 3.4 seconds (Beca, 2009).

3.10 Currents

Currents within the main body of the Upper Waitemata Harbour are driven principally by the tide and current patterns are governed mainly by local bathymetry (channels, mudbanks, headlands, bays). It is only during large freshwater floods, and even then only in the tidal creeks, that freshwater momentum is a significant factor driving currents. Because the estuary is sheltered, wind typically does not generate strong currents (either as shear-driven surface movements or orbital motions associated with wind waves). However, wind-driven motions may become important under strong winds that blow down the long axis of the estuary (NIWA, 2004). The strong tidal influence and the elongate nature of the Upper Waitemata Harbour means circulation is largely parallel to the major channel systems. Exceptions occur where in the shallow water area north of Herald Island where anticlockwise rotatory circulations appear to be generated on flood flows. A variety of minor water movements occur, particularly over shallow intertidal areas due to wind generated currents and waves (Hume, 1983).

Tidal currents were measured using an Aanderaa RCM4 current meter situated mid depth in the Hobsonville Channel from 4-20 May 1982 in 8 m of water (NIWA, 2000). Higher velocities were observed on the ebb tide supporting the ebb scour hole as shown in Figure 3 2. Additional tidal current measurements were carried out by NIWA between 12 and 16 September 2002 using an S4 current metre deployed in 9 m water depth, 0.7 m above the seabed in the centre of the channel (809495.29N, 392340.49E, Mt Eden Circuit 2000)(see Table 3-6).

Currents are only slack (below 0.05 m/s) for 7% of the time when the tide is turning. The mean period of ebb tide is 5.5 hours compared with 6.9 hours for flood tide. The difference in tide periods explains the higher velocities recorded during ebb tides.

Table 3-6 Measured tidal currents in the Hobsonville Channel (Source: NIWA, 2000)

Tide	Peak ebb velocity (m/s)	Peak flood velocity (m/s)
Spring tide	0.70	0.58
Neap tide	0.42	0.30

Additional tidal current measurements were carried out by NIWA between1 2 and 16 September 2002 using an S4 current metre deployed in 9 m water depth, 0.7 m above the seabed in the centre of the channel (809495.29N, 392340.49E, Mt Eden Circuit 2000).

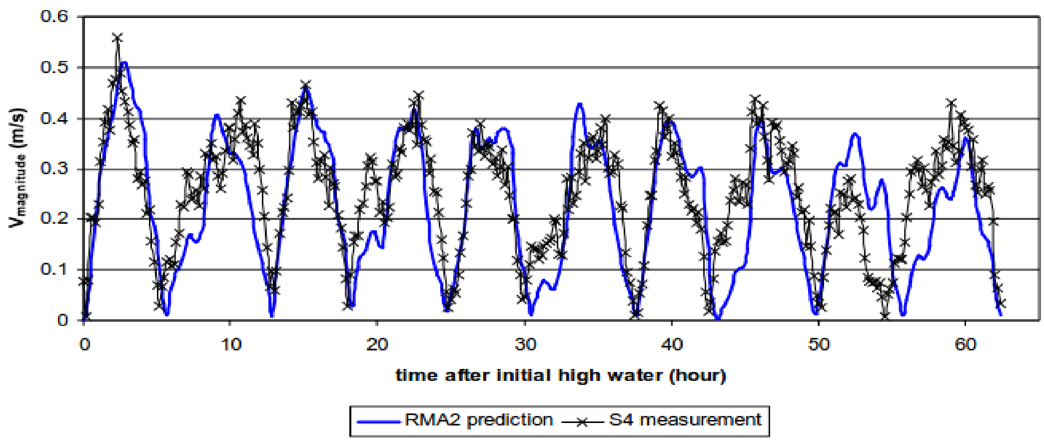


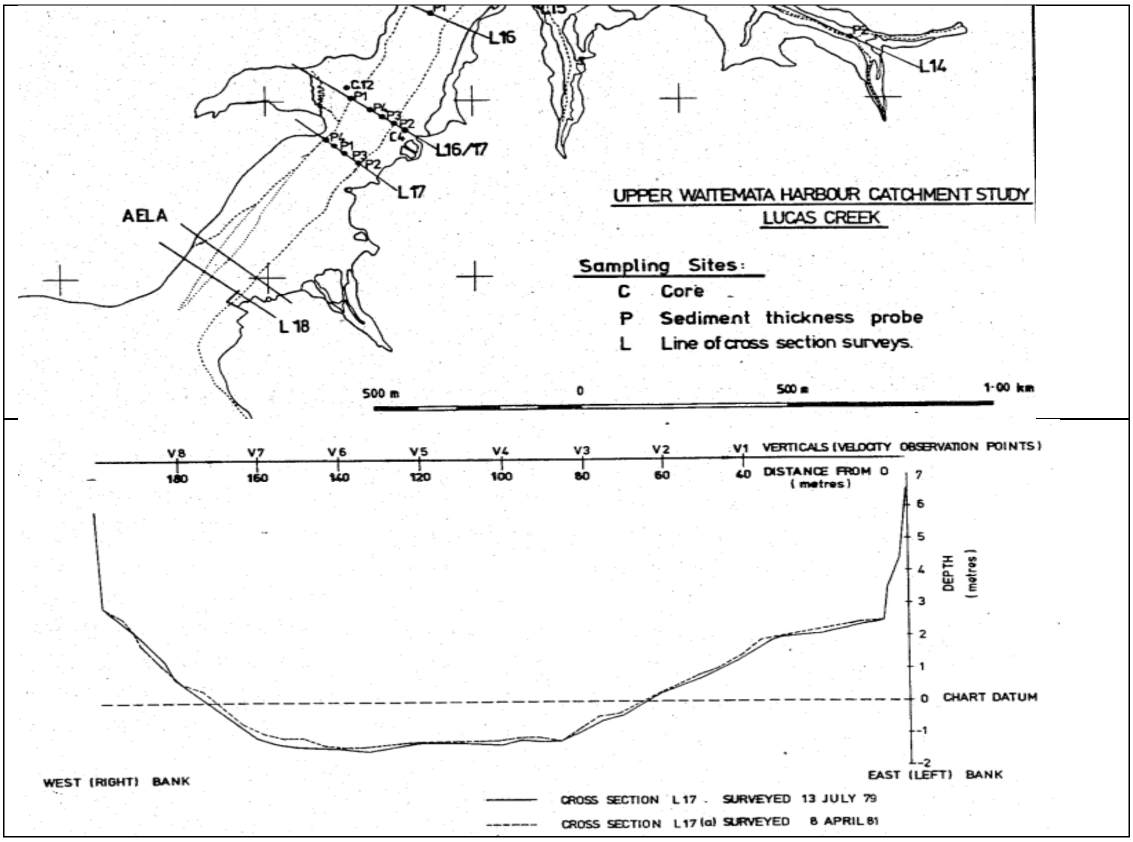
Figure 3-9 Comparison of measured and predicted current velocity in the Catalina Channel between 12 and 16 September 2002 (Source: Beca, 2009)

Williams and Rutherford (1983) measured mid-channel tidal currents up to 0.7 m/s during peak ebb tide in the lower reaches of Lucas Creek and McLachlan and Hume (1981) measured spring tide measurements across Lucas Creek on 11 March 1981 (refer Figure 3-10).

A computational model of the Whau River, Central Waitemata Harbour and Waterview Estuary was constructed using the MIKE 21 FM coastal model. This model was extended to include the Upper Harbour area, Lucas Creek, Rangitopuni Creek and Rarawara Creek (refer Figure 3-11 for full model extents) to provide better representation of the upper harbour. The model grid was refined around the causeway to provide greater resolution (refer Figure 3-12). Details of the model set-up and verification are included in Appendix C. This computational model suite is an industry accepted model for simulating coastal and inland flows. It has previously been successfully applied to several coastal areas of New Zealand, including the Waitemata Harbour.

The hydrodynamic module of MIKE 21 FM solves the vertically integrated equations for fluid flow (conservation of continuity and momentum, i.e., the Saint Venant equations) over a grid consisting of non-uniformly sized triangles and quadrilaterals. This 'flexible mesh' allows for maximum computational efficiency, while maintaining good resolution in areas of interest, such as the Greenhithe Bridge and surrounding area of interest (southern bridge approach and area of proposed rock protection).

The hydrodynamic model provides spatially and time varying information of water levels and current velocities, when provided with the bathymetry and the time-varying forcing (tides, currents) along the model boundaries.



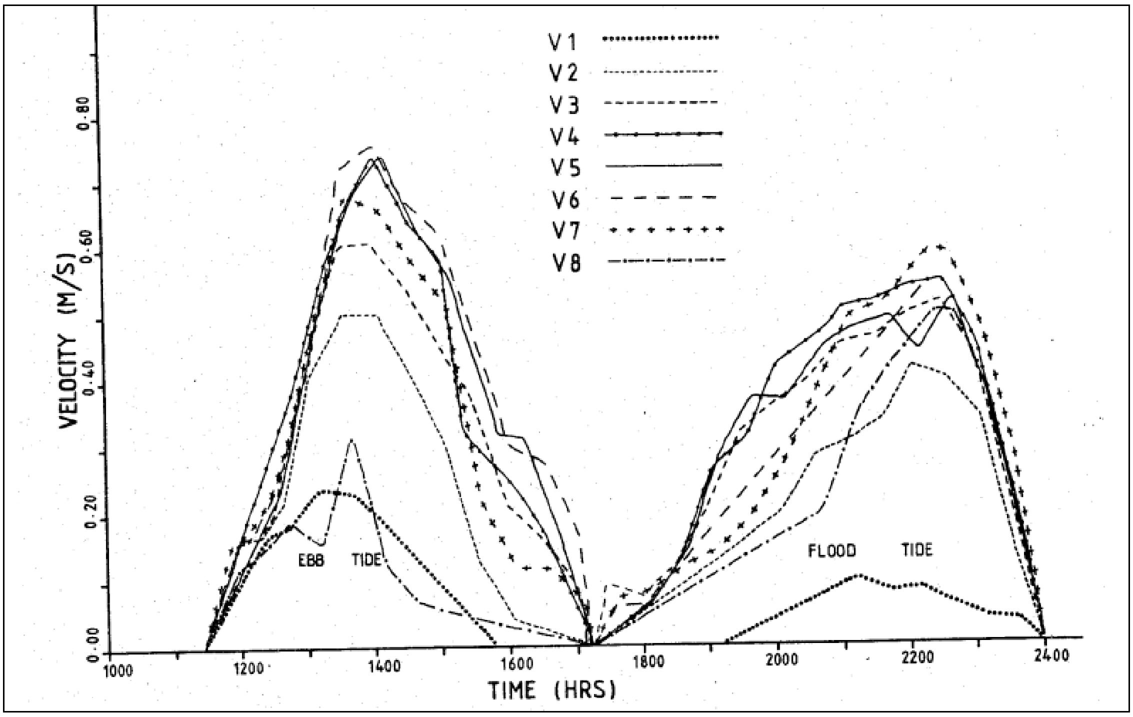


Figure 3-10 Spring tide velocity-time profiles on 11 March 1981 (Source: Hume 1983)

The model grid was refined around the causeway to provide greater resolution (refer Figure 3-12). Details of the model set-up and verification are included in Appendix C. This computational model suite is an industry accepted model for simulating coastal and inland flows. It has previously been successfully applied to several coastal areas of New Zealand, including the Waitemata Harbour.

The modelling result of maximum flood and ebb spring tides are shown in Figure 3-13 and Figure 3-14 and modelled peak currents in the centre of the channel included in Table 3-7. There results show highest currents are confined to the main channel. Typical maximums are in the order of 0.5 m/s for flood and ebb spring tides, consistent with historic observations of Beca, but slightly lower than the observations of NIWA.

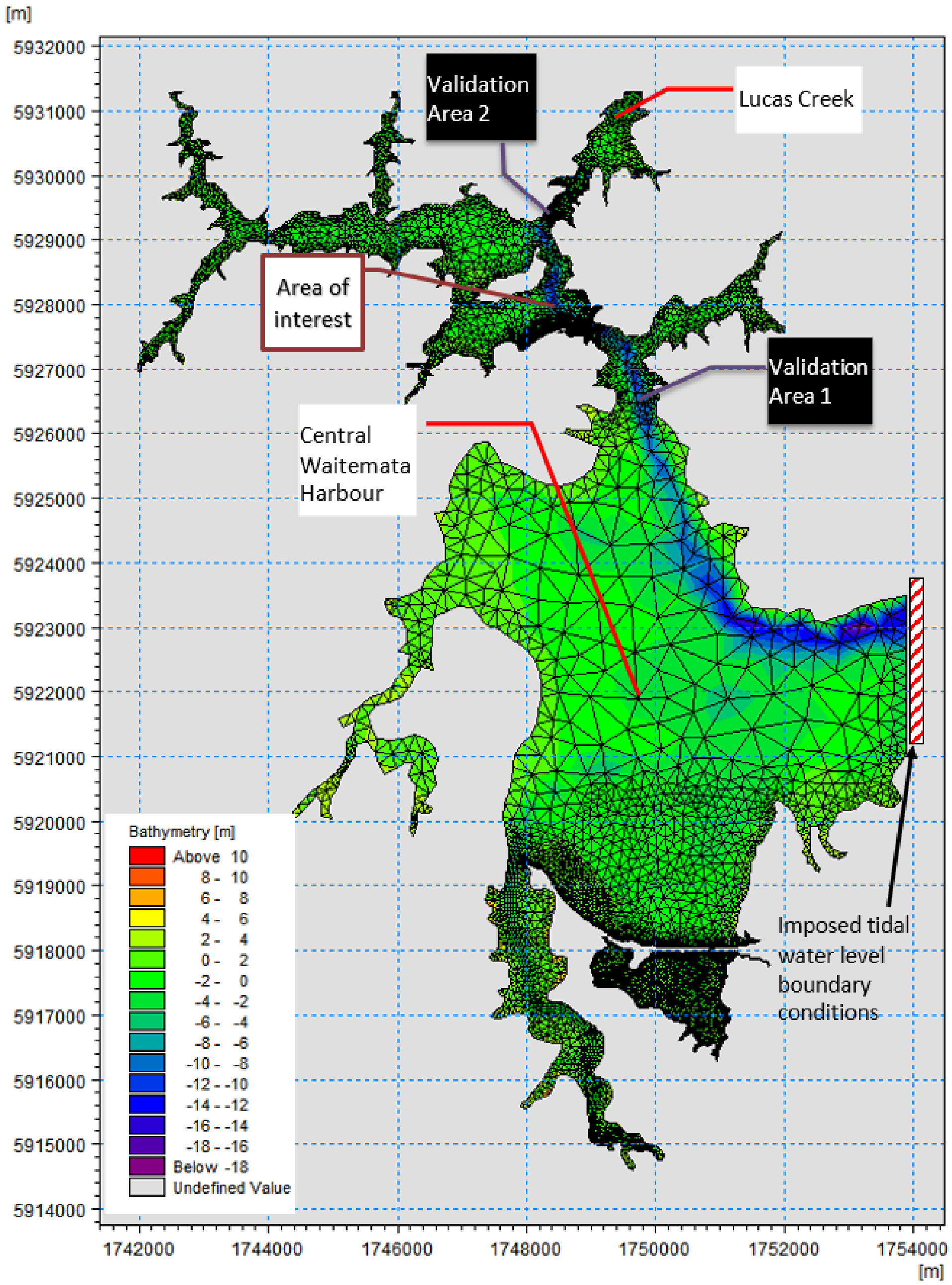


Figure 3-11 Extent of hydrodynamic model

The results show highest currents in the main channel adjacent to the Greenhithe shoreline (greater than 0.8 m/s) and lower currents are present on the intertidal areas, although slightly higher ebb velocities (up to 0.3 m/s) are present than occur during flood tides (up to 0.2 m/s) due to the sheltering nature of the causeway during flood conditions compared to ebb conditions.

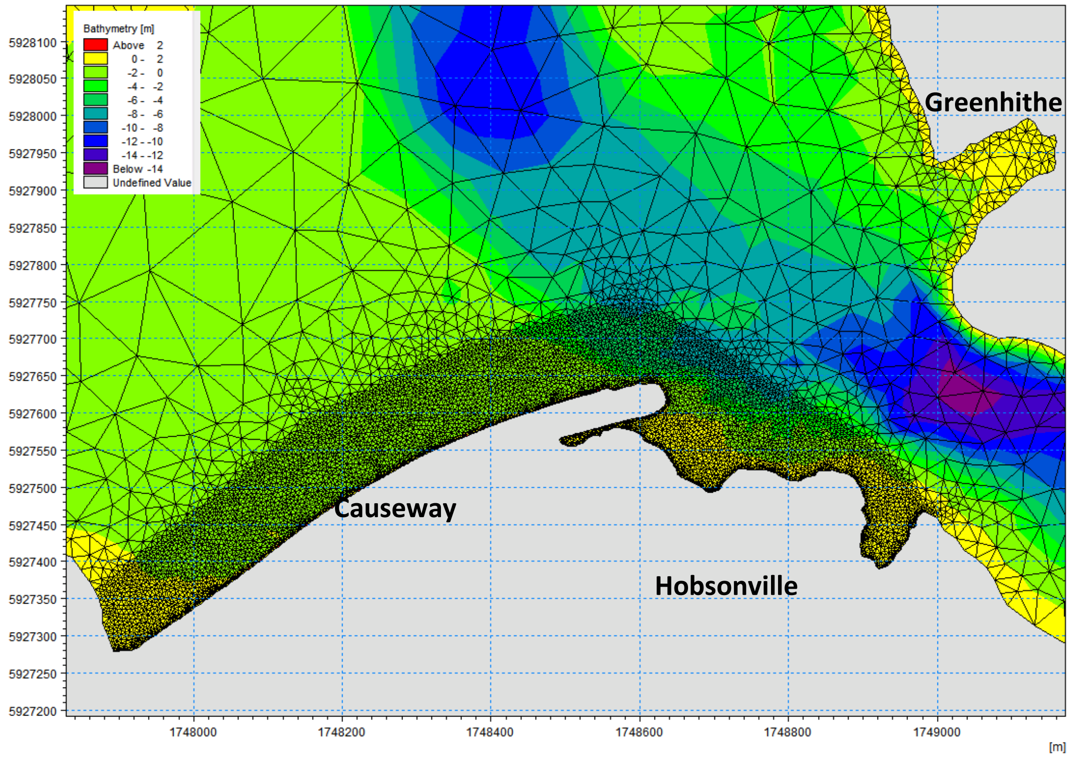


Figure 3-12 Detailed modelling grid around main tidal channel and existing causeway

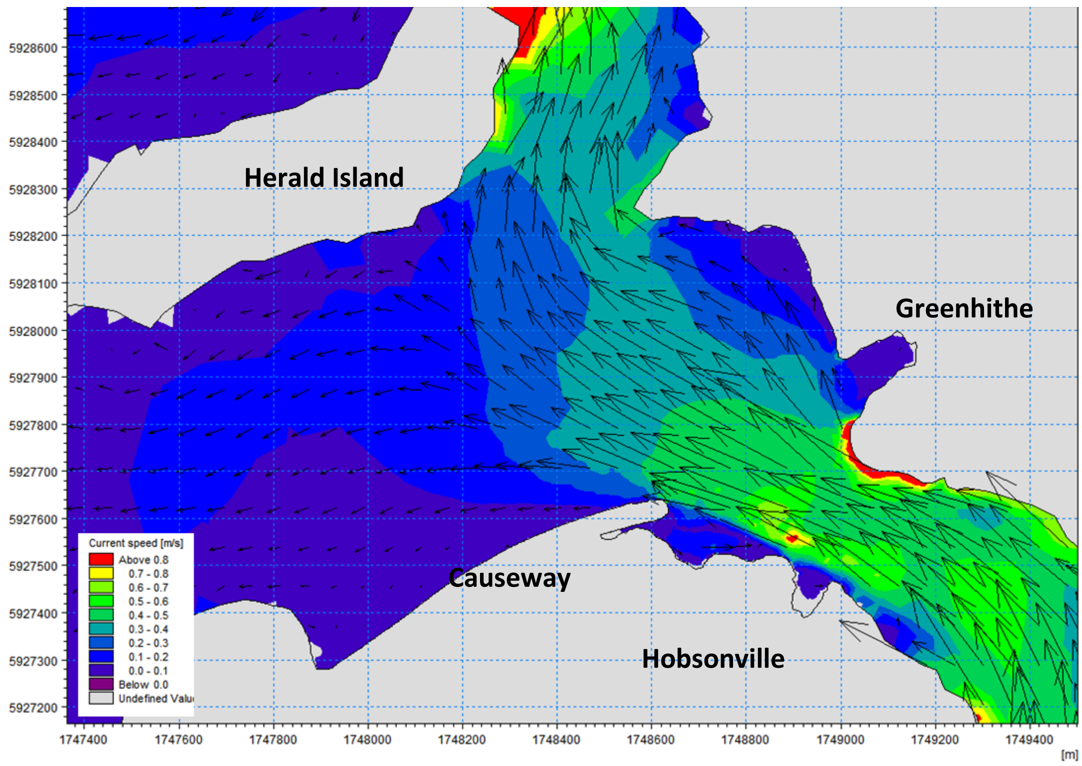


Figure 3-13 Spring tide – maximum flood velocity

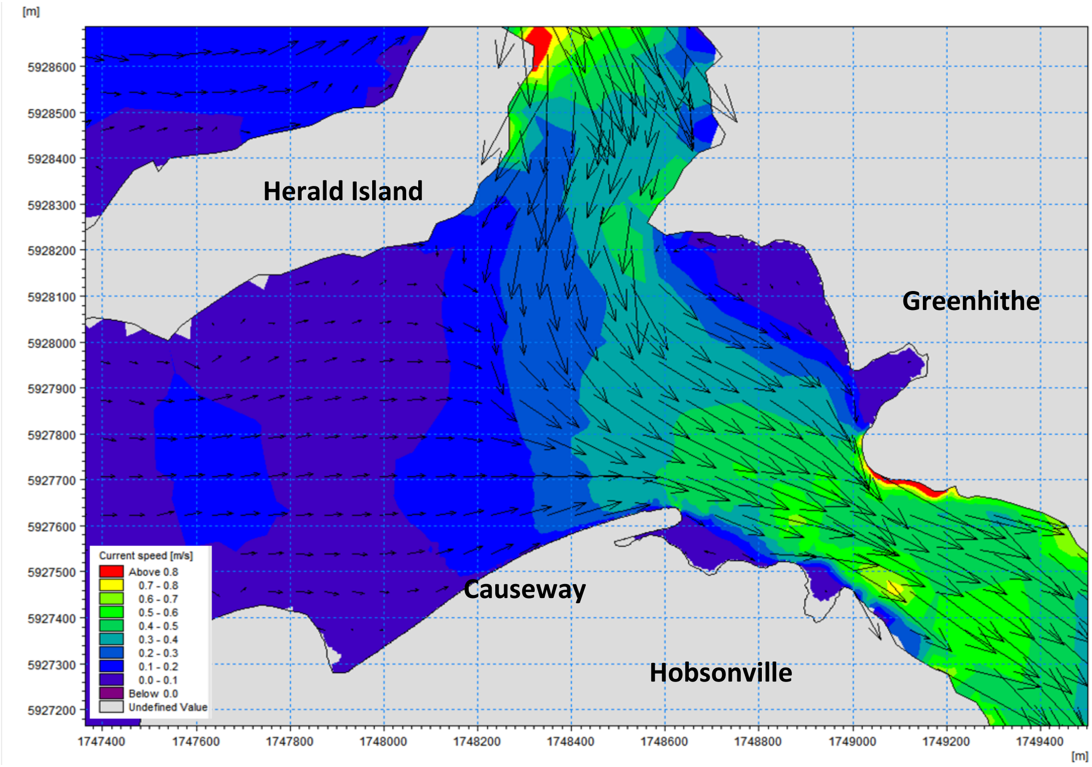


Figure 3-14 Spring tide - maximum ebb tide velocity

Table 3-7 Modelled tidal flows in main channel under existing bridge

Tide	Peak flood velocity (m/s)	Peak ebb velocity (m/s)
Springs	0.497	0.464
Neaps	0.244	0.251

3.11 Coastal processes

The Upper Waitemata Harbour area bounded by Hobsonville Point, Greenhithe and Herald Island is a relatively low energy environment dominated by tidal flow concentrations through the narrow channel and wind generated wave conditions on the intertidal flats. As shown in the comparison of bathymetric charts (refer Figure 3-3) the intertidal flats are likely to be depositionary areas. Historic studies have indicated long term sedimentation patterns in sheltered areas, particularly within the numerous inlets and creeks (Hume, 1983) although in the more exposed areas, subject to higher wave and tidal flows, there is no clear trend.

An analysis of aerial photographs was carried out by NIWA in 2000 using photographs from 1972 (pre original bridge construction) and 2000. Oblique aerials from Whites Aviation are available from 1949 to 1973 (refer Appendix A) and additional aerial images are available from 1959, 1996, 2006 and 2008 and satellite images via Google Earth for 30 August 2004, July 2006 to March 2007, 28 March 2009 and 30 August 2013 (refer Appendix B). It is noted that these images are at different stages of the tide, but they do provide a means of assessing global changes. The original causeway construction can be seen in Figure A-6 (Appendix A) and the causeway widening and duplication of the Greenhithe Bridge is shown in Figure B-3 and B-4 (Appendix B).

The NIWA (2000) assessment identified a slight retreat of the intertidal flat on the southern side of the western bridge abutment that was attributed to the higher flow velocities from the construction formed by the abutment. Additional impacts identified by NIWA included mangrove growth and the development of drainage channels associated with the drainage works through the causeway.

The results of our review of the available photographs, particularly since the duplication of the bridge was completed, show less than minor changes in coastal processes with changes mainly associated with the increase in mangrove growth. The retreat of the intertidal flat on the southern side of the western bridge abutment has not continued. Even from the 2008 high resolution photograph (Figure B-6, Appendix B) the shoal that formed immediately to the south of the abutment has progressively become covered with mangroves. Comparing the 1996 photograph (Figure B-2) with the 2006 aerial (Figure B-4) and the 2013 satellite image (Figure B-8) which appear to be at similar states of the tide, show little net change over this 17 year period.

The only other change evident from the higher resolution photographs available since NIWA's study is the migration of the shell ridge that extends south east of the private land area at the western end of the causeway. It has moved more landward from 1959 to 1999 although has been relatively stable since that time. It is assumed that the causeway modified catchment flows in this area and enabled the shell ridge to migrate landward. However, from 1999 it has appeared to be relatively stable to accretionary.

3.12 Climate change and sea level rise

Historic sea level rise in New Zealand has averaged 1.7 ± 0.1 mm/year (Bell and Hannah, 2012). Climate change is predicted to accelerate this rate of sea level rise into the future. NZCPS (2010) requires that the identification of coastal hazards includes consideration of sea level rise over at least a 100 year planning period. Potential sea level rise over this time frame are likely to significantly alter the coastal hazard risk.

The Ministry for the Environment (MfE, 2008) guideline recommends a base value sea level rise of 0.5 m by 2100 (relative to the 1980-1999 average) with consideration of the consequences of sea level rise of at least 0.8 m by 2100 with an additional sea level rise of 10 mm per year beyond 2100. Bell (2013) recommends that for planning to 2115, these values are increased to 0.7 and 1.0 m respectively. Bell (2013) also recommends that when planning for new activities or developments, that higher potential rises of 1.5 to 2 m above the present mean sea level should be considered to cover the foreseeable climate-change effects beyond a 100 year period.

Modelling presented within the most recent IPCC report (AR5; IPCC, 2013) show predicted global sea level rise values by 2100 to range from 0.27 m, which is slightly above the current rate of rise, to 1 m depending on the emission scenario adopted. Extrapolating the RCP8.5 scenario to 2115 results in a sea level range from 0.27 to 0.47 m by 2065 and 0.62 to 1.27 m by 2115 (Figure 3-15). The RCP8.5 scenario assumes emissions continue to rise in the 21st century.

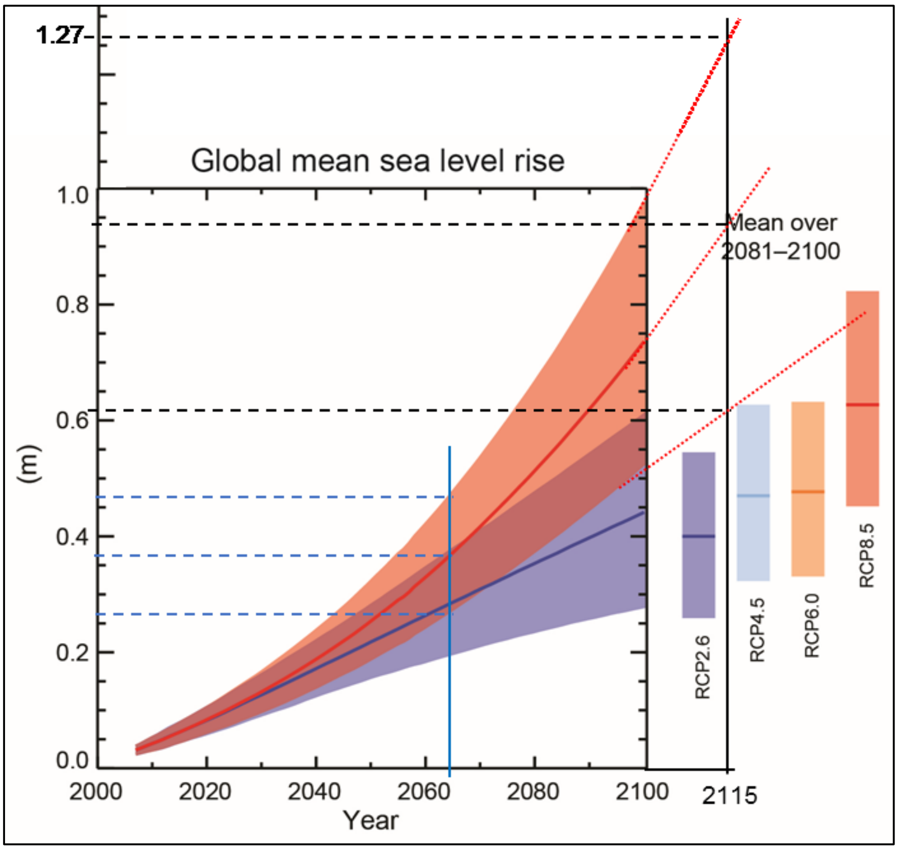


Figure 3-15 Projections of potential future sea level rise presented within IPCC AR5 (IPCC, 2013) with adopted values for this assessment at 2065 and extrapolated to 2115

4 DESCRIPTION OF THE PROJECT

The component of the GBWD works assessed in this report that has a potential impact on the physical coastal environment is the causeway widening to accommodate the new watermain and wastewater pipelines. The existing SH18 causeway will be widened approximately 15 m from the edge of the existing shared path (top of the embankment) for a length of approximately 860 m. As part of the causeway widening, a construction platform will be established to provide room both for the Northern Interceptor pipe facilities and stage 2 of this development (both separate projects). This will require an additional area approximately 150 m long and an additional 38 m wide (so 53 m total width from the edge of the cycle way).

The proposed works are described in detail in the AEE. Key drawings showing the proposed works and construction methodology are copied in the AEE, Volume 3 - Drawings. The works described in the AEE and shown on the appended drawings are assessed in this report.

5 ASSESSMENT OF EFFECTS

The assessment of effects addresses the potential effects of the permanent works (long term effects) as well as potential effects of the shorter term construction activity as it relates to the physical coastal processes.

5.1 Long term effects

There are three components that have potential to affect the physical coastal processes operating in this area. They are:

- The 85 m extension of MHWS by the causeway (100 m extension of the toe) extending into the tidal channel (Drawings 2010673.851, 2010673.854, 2010674.855, 2010674.007 and 2010673.007)
- The general encroachment of the 15 m causeway widening (Drawings 2010673.851, 2010673.852 and 2010673.853)
- The localised effect of the additional 38 m by 150 m widening of the causeway to accommodate the new watermain and pipes for the Northern Interceptor project
- The modifications at the western end of the causeway providing connections from the public road to the causeway embankment (Shown on Drawing 2010673.008).

This development could have an effect on water level and current velocity due to the additional constriction within the main channel as well as the displacement of volume as a result of the reclamation mass.

A numerical model study was carried out to identify and assess the effects of the modifications (refer Appendix A). The assessment of effects was made by comparing the results of tidal current and water level change from the existing to the proposed causeway configuration that included the extension into the tidal channel and the additional reclamation area adjacent to the existing causeway (refer Figure 5-1).

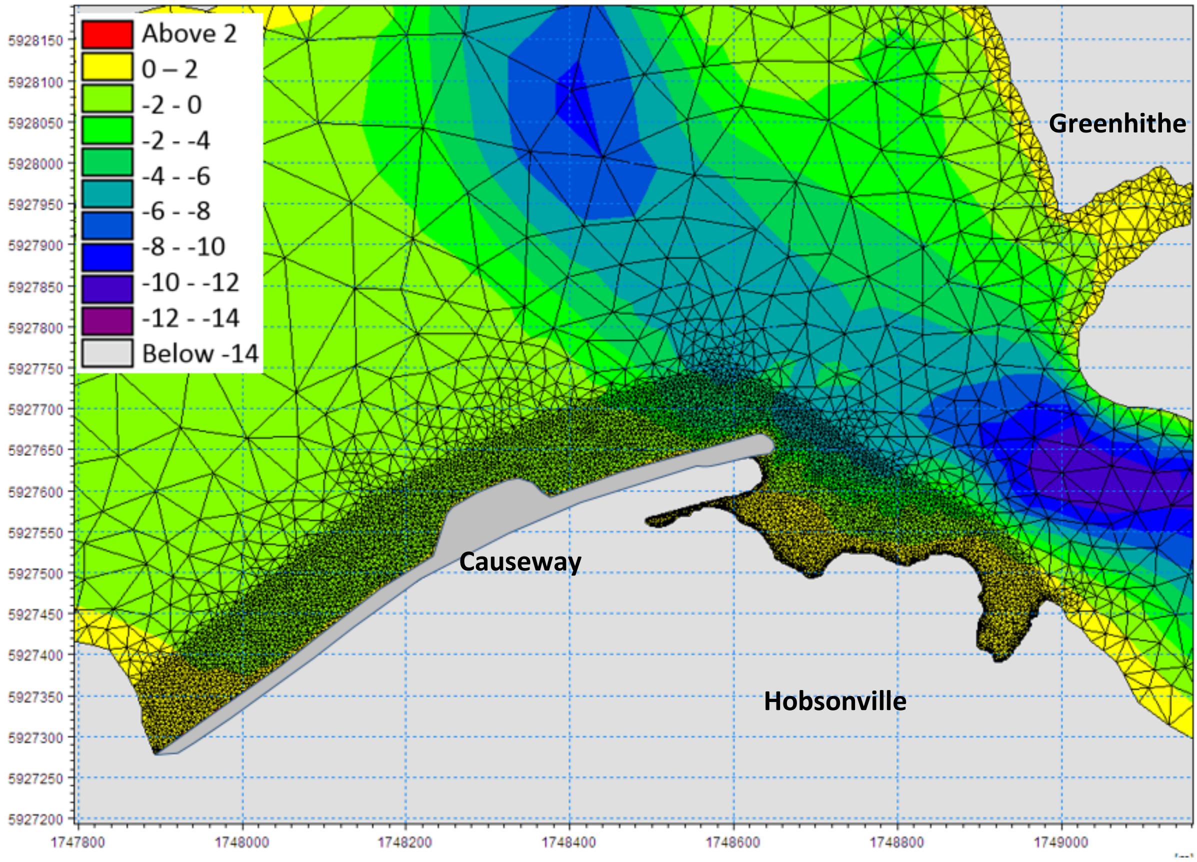


Figure 5-1 Proposed causeway widening and extension included in the hydrodynamic model

5.1.1 Effect on water level

Figure 5-2 shows the difference in the maximum water level with the proposed causeway additions. The results show only very minor changes (generally less than 1 mm) which is not able to be practicably measured. Therefore, the proposed development will not result in any noticeable change in water level.

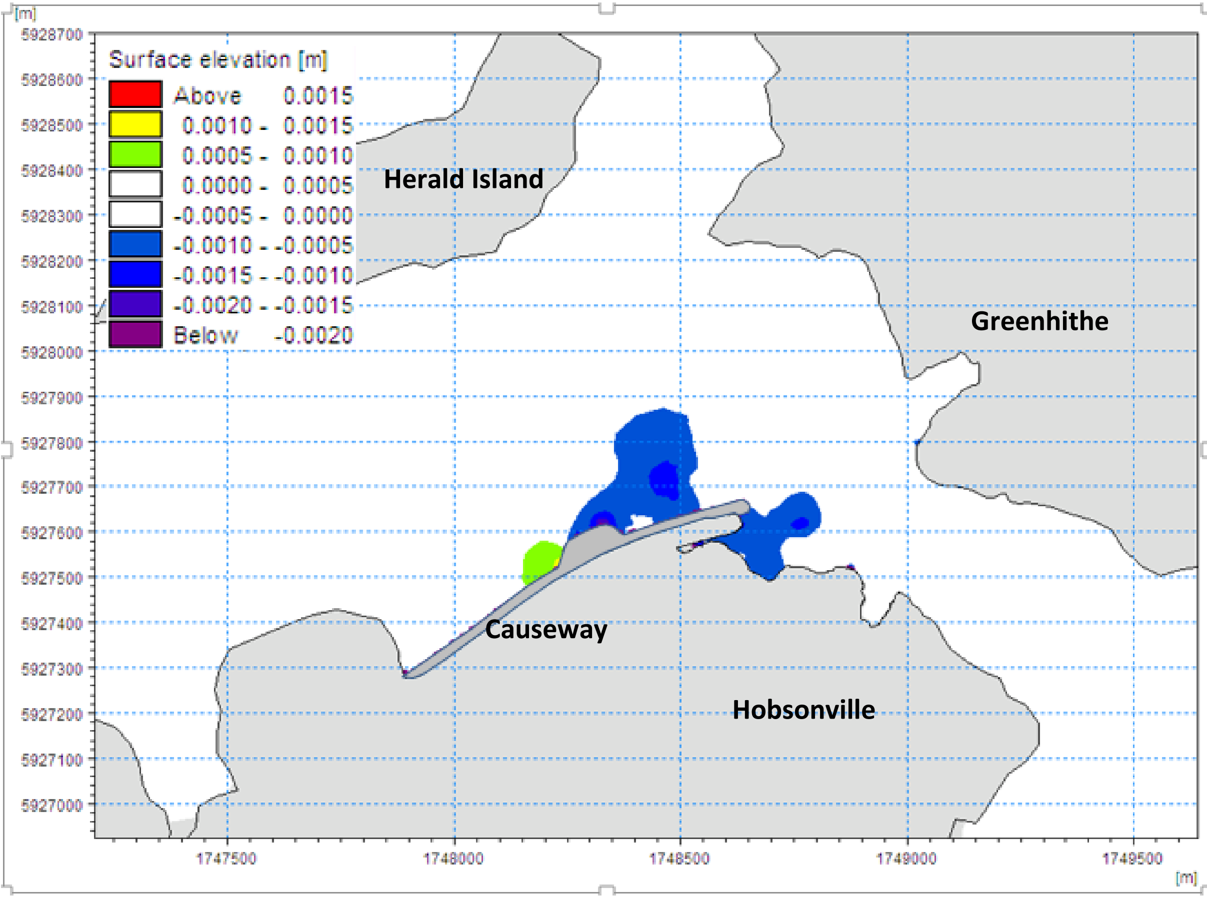


Figure 5-2 Difference in maximum water levels pre and post causeway construction

5.1.2 Effect on tidal currents

Figure 5-3 and Figure 5-4 show the maximum flood and ebb velocity distribution during spring tides. Figure 5-5 and Figure 5-6 show the difference plot between the velocity distribution for the existing situation and with the proposed causeway for flood and neap conditions.

The difference plot for flood tide, springs (Figure 5-5) show a very small area of increased velocities at the seaward tip of the causeway extension into the main channel, with up to 0.2 m/s immediately at the end of the causeway, but generally less than 0.1 m/s at peak tide conditions. The difference plot shows a "shadow" area either side of this location during peak flood spring tides with lower velocities. In the immediate lee of the new causeway velocities are reduced by more than 0.25 m/s. Velocities are reduced along the flank of the new causeway from the spit to the localised widening for the NI project and there are reductions in velocity along the western bank of the main channel due to the flows being pushed out into the main channel.

During peak ebb spring tide flows the differences are less significant, although there are still increased velocities at the north-eastern end of the extension and increased sheltering to the south-east and along the causeway extension.

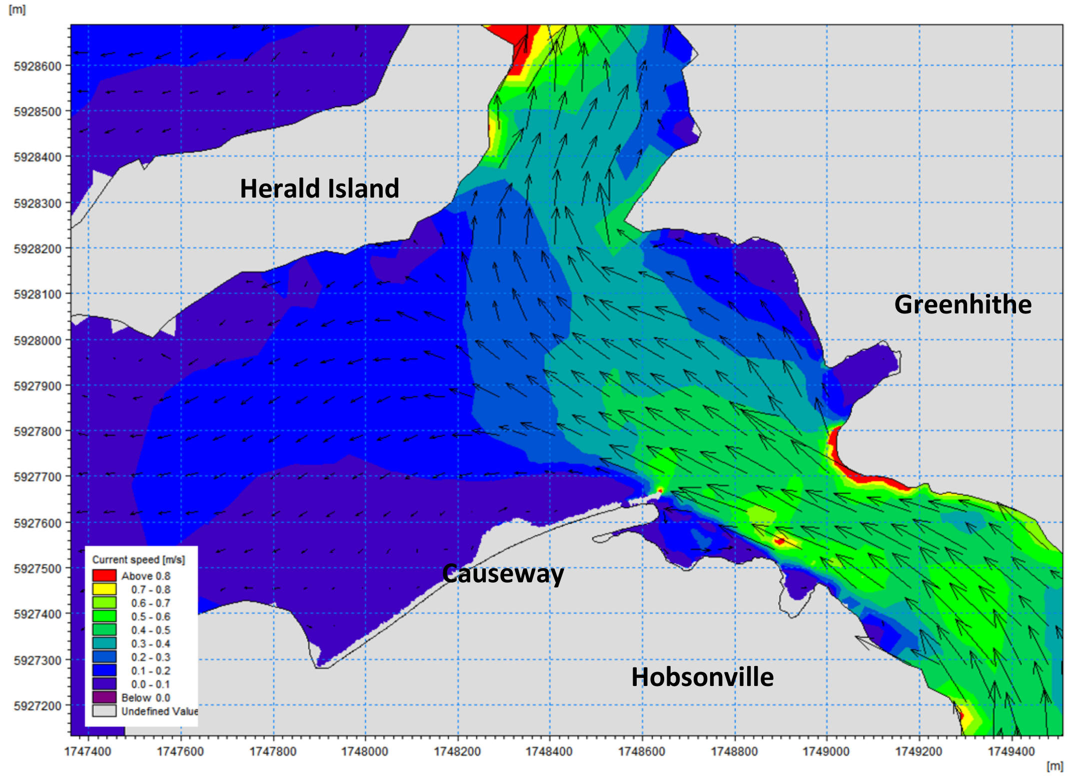


Figure 5-3 Maximum spring tide -flood condition with widened and extended causeway

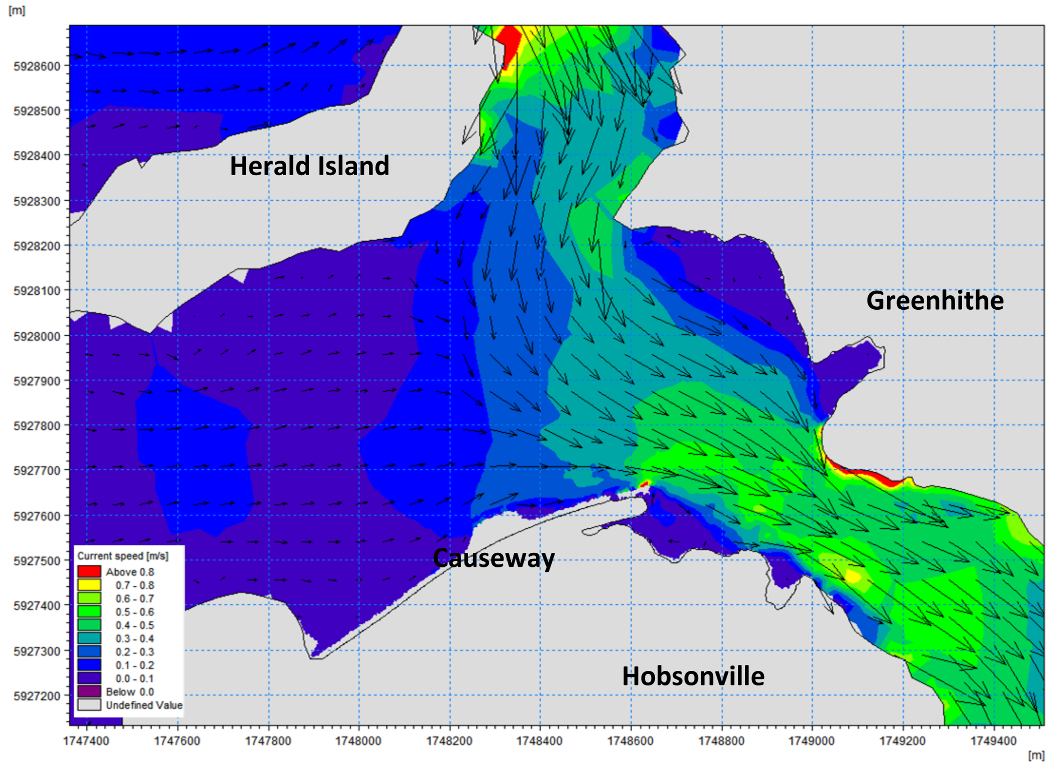


Figure 5-4 Maximum spring tide - ebb condition with widened and extended causeway

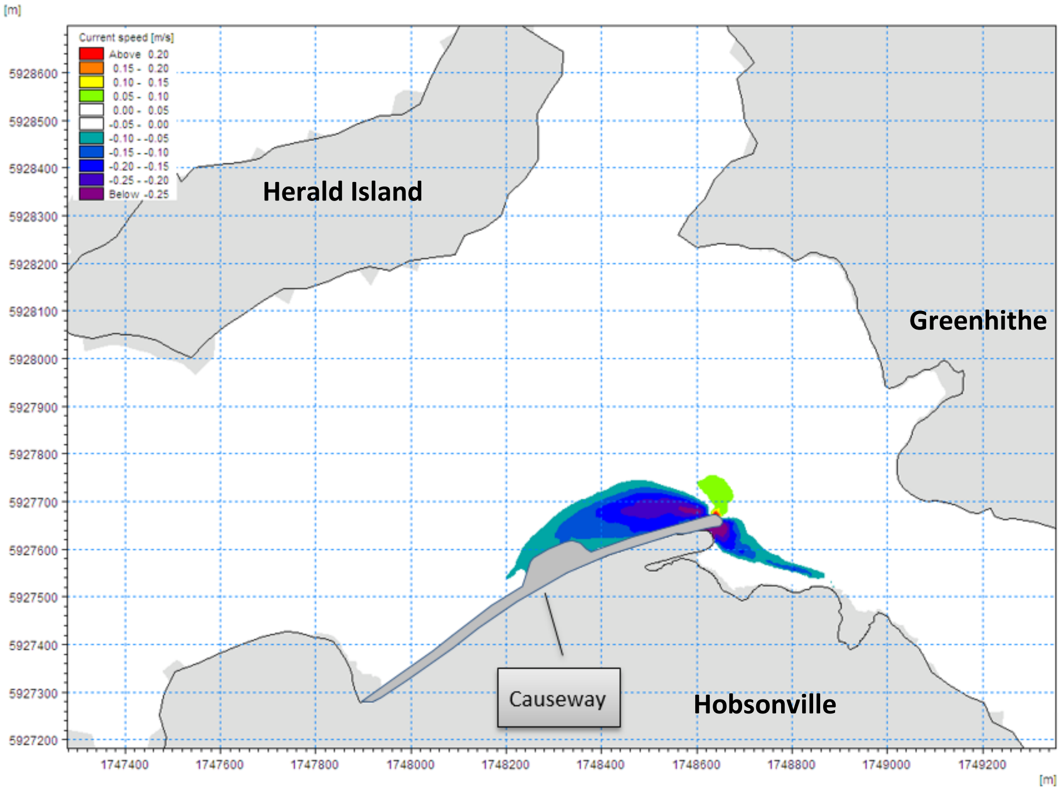


Figure 5-5 Spring tide, difference in flood tide velocities between existing situation and with causeway widening and extension

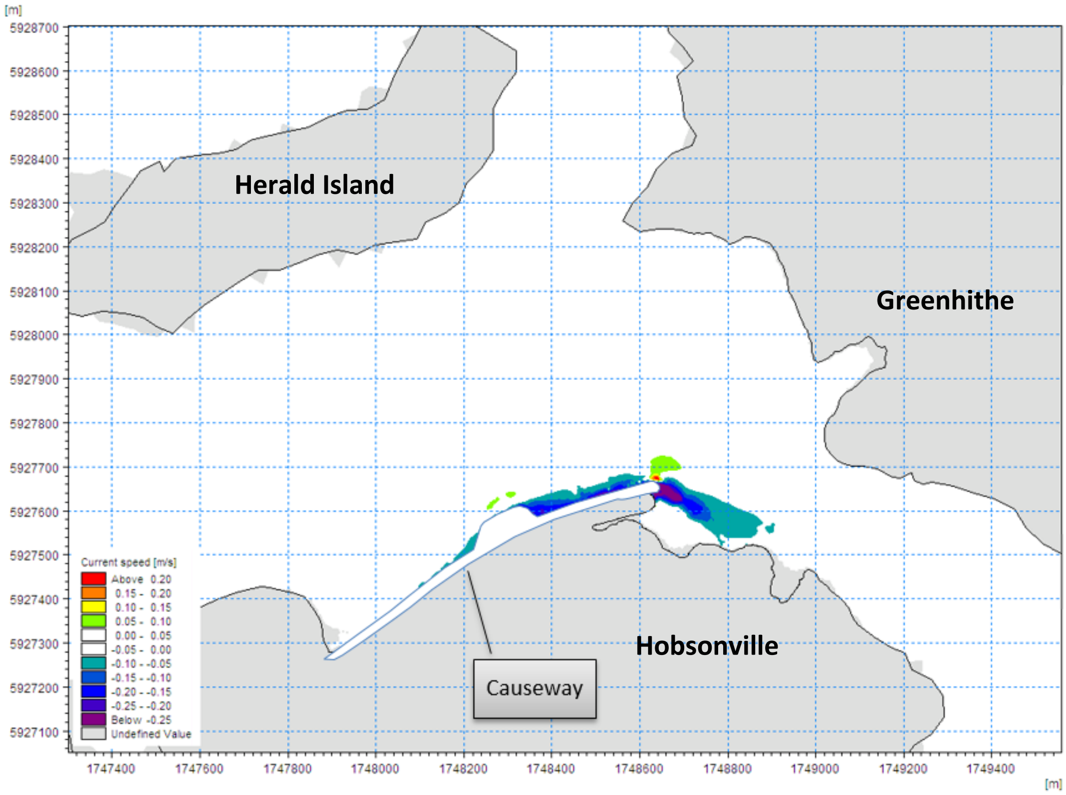


Figure 5-6 Spring tide, difference in ebb tide velocities between existing situation and with causeway widening and extension

Figure 5-7 and Figure 5-8 show the differences in bed shear stress during flood and ebb spring tide conditions and the surface areas of areas of lower (deposition) and higher (erosion) trends.

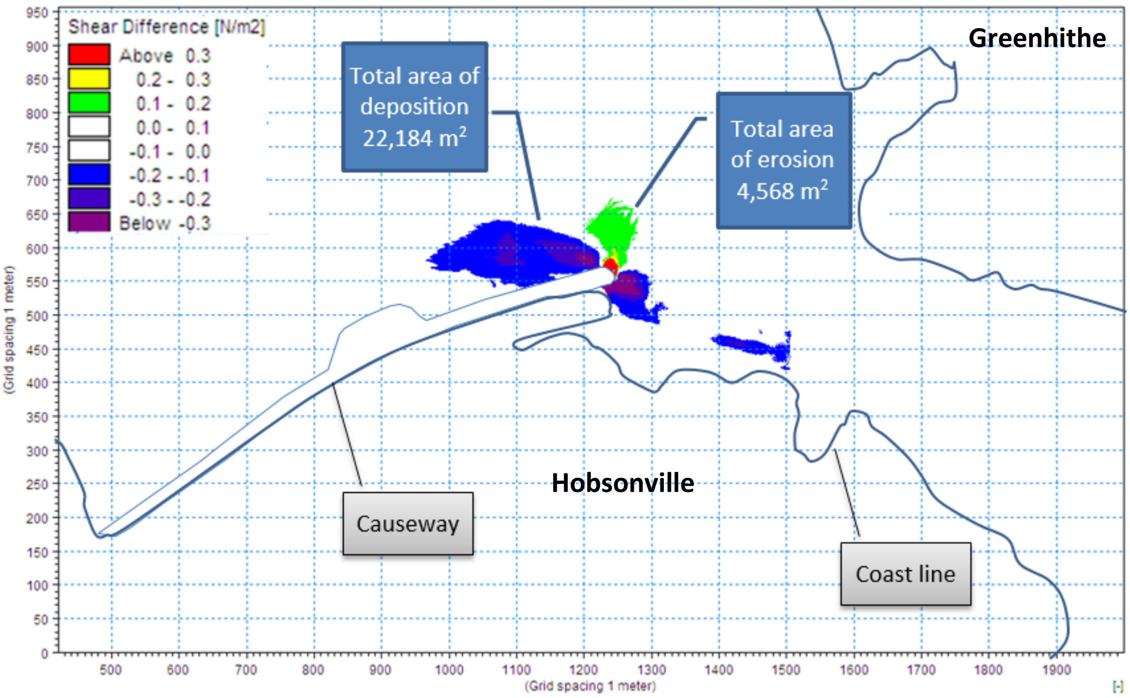


Figure 5-7 Difference in flood shear stress for spring tide conditions

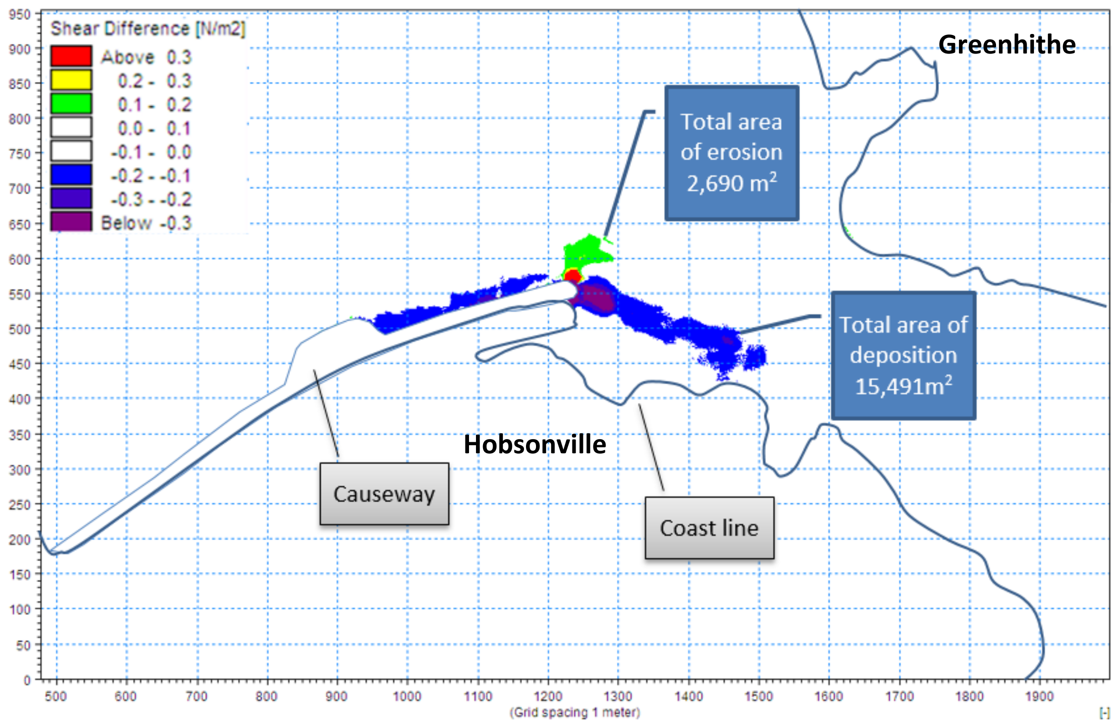


Figure 5-8 Difference in ebb shear stress for spring tide conditions

When examining the potential for the causeway widening and extension to affect coastal processes, the results show that minor scour can be expected at the tip of the new causeway as it extends into the main channel. Scour depth will be limited as the depth of sediment at this location is relatively shallow (less than 1 m) and overlies Waitemata Group sediments.

The scoured sediment will be transported and deposited both on the intertidal area to the north west of the channel and to the south-east in the lee area created by the causeway extension. Deposition is likely to initially occur in areas where bed shear stress is reduced from the existing situation due to the increased sheltering and modification of tidal flows resulting from the causeway extension. It is likely that there will be greater areas of deposition within this area from time to time. This process is likely to occur over a period of weeks to months, with change initiating during the spring tide phases due to the higher energies present.

Based on the results of the coastal process study and hydrodynamic modelling it is shown that the proposed causeway widening and modifications at the western end of the causeway has no significant effect on coastal processes. The extension of the causeway has a minor and localised effect on coastal processes at this location. The resulting changes are likely to include localised lowering of sand at the tip of the causeway extension and minor levels of deposition on the intertidal areas to the north-west and along the main channel to the south-east. There will be no significant effects on seabed levels within the main channel or influence on the existing bridge structures.

5.1.3 Sea level rise

The effect of the occupation of the CMA by the causeway is unlikely to change as a result of existing sea levels as there was no significant change in sea level rise and only minor and local changes in tidal current. The increased sea level will result in wave action occurring higher up the causeway. The design water level taking into account 1 m of sea level rise and a 1%AEP storm is 3.6 m RL. This is lower than the proposed crest along the majority of the causeway of 5 to 5.5 m RL. The extension crest of 3 m RL is based on the crest being 1 m above HAT. Therefore it will be emerged during normal tidal processes even with 1 m of sea level rise. However, as sea levels increase wave overtopping during storm events will occur more frequently over the extension and the crest of this area will need to be designed to take into account more frequent wave overtopping events. For public safety access to this area would need to be controlled during events where overtopping is likely.

5.2 Short term effects

The details of the construction methodology are set out in the description of works in the AEE and summarised in Section 4 of this report. The most significant risk to the coastal marine environment is the accidental discharge into the CMA. The potential effects of sediment discharge from the excavated sediments and stabilization will be initially managed by silt curtains or other appropriate sediment control measures and carrying out the works at low tide using similar approaches to that currently being employed on SH16 causeway extension as indicated on Drawing 2010674.040. A construction management plan will be prepared to outline procedures for refuelling and machine maintenance to reduce risk of spills and discharges from machinery operating within the CMA.

The risk of silt discharge from the causeway extension can be reduced by careful excavation techniques for the formation of foundations, preferably at periods of slack tide and ensuring clean rockfill is used for end-tipping.

6 RECOMMENDED MITIGATION AND MANAGEMENT MEASURES

6.1 Long term effects

The coastal process investigation and hydrodynamic modelling assessment has indicated that the proposed causeway widening and modifications at the western end of the causeway will have no adverse effects apart from the permanent occupation of the CMA, although in principle the size of any occupation of the CMA should be kept as small as practicable, as over time significant occupation of the Waitemata Harbour has occurred that cumulatively has some effect on the way the harbour behaves. The causeway extension will have a minor and localised effect on coastal processes at this location. The resulting changes are likely to include localised lowering of sand at the tip of the causeway extension and minor levels of deposition on the intertidal areas to the north-west and along the main channel to the south-east. This will occur soon after the works are completed and the shoreline and seabed will stabilise taking into account the new structure. There will be no significant effects on seabed levels within the main channel or influence on the existing bridge structures.

6.2 Short term effects

Short term effects on coastal processes associated with the construction of the causeway are associated with discharge of sediments and contaminants. The likelihood of discharge can be limited by doing much of the initial construction works at low tide and by using effective sediment control practices and by adherence to a construction management plan that addresses the operation and control of machinery and practices within the CMA.

7 APPLICABILITY

This report has been prepared for the benefit of Watercare Services Ltd with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

8 REFERENCES

Beca (1997) Coastal permit application: Dredging at the Port of Auckland. For Ports of Auckland Ltd.

Beca (2009) Coastal processes in the Upper Waitemata Harbour. Report prepared for Hobsonville Land Company. Beca Infrastructure Ltd, August 2009.

Bell, R.G. (2012) Sea Levels for New Zealand: Give us a Number Please! *Presentation at Sea level Rise: meeting the Challenge Conference*, Wellington May 10-11, 2012.

Bell, R.G. (2013) Submission on the Proposed Northland Regional Policy Statement of Evidence, 26 May 2013. 22p.

Dolphin, T.J. and M.O. Green (1997) Sediment dynamics of an estuarine "Turbid Fringe". Combined Australasian Coastal Engineering and Ports Conference, Christchurch September 1997.

Hume, T. (1983). Upper Waitemata Harbour sediments and inferred impact of future catchment and estuary use change. Water Quality Centre Report No. 3, Hamilton, 1983.

Goda, Y., (2003) Revisiting Wilson's Formulas for Simplified Wind-Wave Prediction. J Waterway, Port, Coastal, Ocean Engineering. 129 (2), 93-95.

Hannah, J. and Bell, R.G. (2012) Regional sea level trends in New Zealand. Journal of Geophysical Research 117: C01004.

Hume, T. (1983). Upper Waitemata Harbour sediments and inferred impact of future catchment and estuary use change. Water Quality Centre Report No. 3, Hamilton, 1983.

IPCC (2013). Working Group I contribution to the IPCC 5th Assessment Report "Climate Change 2013: The Physical Science Basis". DRAFT report by Intergovernmental Panel on Climate Change. June, 2013.

KMA (1989) Assessment of the disposal of dredged material at the North Rangitoto spoil ground. Kingett Mitchell and Associates report for Ports of Auckland Ltd.

McLachlan, M.J. and T.M. Hume (1981). Data file – Lucas Creek tidal survey, 11 March 1981, Upper Waitemata Harbour Catchment Study, Hamilton Science Centre Internal Report No. 80/6

MfE (2008) Coastal Hazards and Climate Change. A Guidance Manual for Local Government in New Zealand. 2nd edition. Revised by Ramsay, D, and Bell, R. (NIWA). Prepared for Ministry for the Environment.

NIWA Ecosystems (1997). Effects of urbanisation in the catchment of the Upper Waitemata Harbour/ three reports prepared by NIWA Ecosystems for Auckland Regional Council. Auckland, New Zealand: Auckland Regional Council

NIWA (2004a). Prediction of contaminant accumulation in the Upper Waitemata Harbour. Niwa report ref HAM2003-087/1 report prepared for Auckland Regional Council, North Shore City, Rodney District Council, Waitakere City Council and Transit New Zealand, June 2004.

NIWA (2004b). Water quality survey of Mahurangi Harbour, Upper Waitemata Harbour and Tamaki Estuary Annual Report. Niwa report ref HAM2004-045 report prepared for Auckland Regional Council, October 2004.

NIWA (2013) Coastal inundation by storm tides and wave in the Auckland region. NIWA report ref. HAM2013-059 prepared for Auckland Council, September 2013.

NZCPS (2010) New Zealand Coastal Policy Statement

Oldman, J, N. Hancock, M. Lewis (2008) Central Waitemata Harbour Contaminant Study. Hydrodynamics and Sediment Transport Fieldwork. Prepared by NIWA for Auckland Regional Council. Technical Report 2008/036

Swales, A., Hume, T. M., McGlone, M. S., Pilvio, R., Ovenden, R., Zviguina, N., Hatton, S., Nicholls, P., Budd, R., Hewitt, J., Pickmere, S., & Costley, K. (2002). Evidence for the physical effects of catchment sediment runoff preserved in estuarine sediments: Phase II (field study).

Van Rijn, L. (1990) Principles of sediment transport in rivers, estuaries and coastal seas. Aqua Publications.

Williams B.L. and J.C. Rutherford (1983). The flushing of pollutants and nutrients from the Upper Waitemata Harbour. Upper Waitemata Harbour Catchment Specialist report, Auckland Regional Authority, Auckland.

APPENDIX A HISTORIC CHARTS AND OBLIQUE PHOTOGRAPHS

Figure A 1 1854 Bathymetric chart (Source: Governor Greys Collection Auckland Library)

Figure A 2 White Aviation Photograph 1949 03 17

Figure A 3 White Aviation Photograph 1955 11 07

Figure A 4 White Aviation Photograph 1955 11 17

Figure A 5 White Aviation Photographs 1963 06 06

Figure A 6 White Aviation Photograph 1973 07 04

APPENDIX B HISTORIC AERIALS AND SATELLITE IMAGES

Figure B 1 Aerial from 1959 (Source: Auckland Council GIS)

Figure B 2 Aerial from 1996 (Source: Auckland Council GIS)

Figure B 3 Satellite image, 30 August 2004 (Source: Google Earth

Figure B 4 Aerial from 2006 (Source: Auckland Council GIS)

Figure B 5 Satellite image, Jul 2006 to Mar 2007 (Source: Google Earth)

Figure B 6 Aerial from 2008 (Source: Auckland Council GIS)

Figure B 7 Satellite image, 28 March 2009 (Source: Google Earth)

Figure B 8 Satellite image, 30 August 2013 (Source: Google Earth)

APPENDIX C COASTAL MODELLING REPORT

REPORT

Greenhithe Bridge Watermain Duplication and Northern Interceptor

Coastal Modelling Report

Prepared for:

Watercare Services Ltd

March 2015

Job No: 29719.v2

Tonkin & Taylor

ENVIRONMENTAL AND ENGINEERING CONSULTANTS

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1 Introduction

A computational hydrodynamic model was developed of the middle and upper portions of the Waitemata Harbour to assist in the assessment of effects on the physical coastal processes operating within the harbour area between Hobsonville and Greenthithe. The model was used the calibrated model used to establish effects of the Waterview Connection Project extended to include the upper harbour area.

This computational model suite is an industry accepted model for simulating coastal and inland flows. It has previously been successfully applied to several coastal areas of New Zealand, including the Waitemata Harbour.

2 Numerical model

The hydrodynamic module of MIKE 21 FM solves the vertically integrated equations for fluid flow (conservation of continuity and momentum, i.e., the Saint Venant equations) over a grid consisting of non-uniformly sized triangles and quadrilaterals. This 'flexible mesh' allows for maximum computational efficiency, while maintaining good resolution in areas of interest, such as the Upper Harbour Bridge and surrounding area of interest.

2.1 Model domain and resolution

The hydrodynamic model provides spatially and time varying information of water levels and current velocities, when provided with the bathymetry and the time-varying forcing (tides, currents) along the model boundaries. For this assessment, the inputs to the model were tidal water levels (with horizontal velocity) at the boundary of the computational domain, and sources of mass and momentum representing flow into harbour.

The computational model covers the area of the Waitemata Harbour, from the Auckland Harbour Bridge in the east to the Rangitouni Stream in the north, and includes the Whau River, Waterview Estuary and Oakley Inlet to the south as shown in Figure 2-1.

Mesh size has been refined around the proposed causeway with grid size of approximately 2 m^2 – 5 m^2 at the coastal boundary, increasing to approximately 20 m^2 at the Lukas Creek mouth and 50 m^2 further west (as shown in Figure 2-2 and Figure 2-3). Mesh resolution south of the Upper Harbour Bridge (old model domain) has not been altered.

Due to the mesh generation processes, results within 2 -3 grid size cells of the proposed SH18 causeway widening should be treated with caution and compared with results of the adjacent cells for consistency.

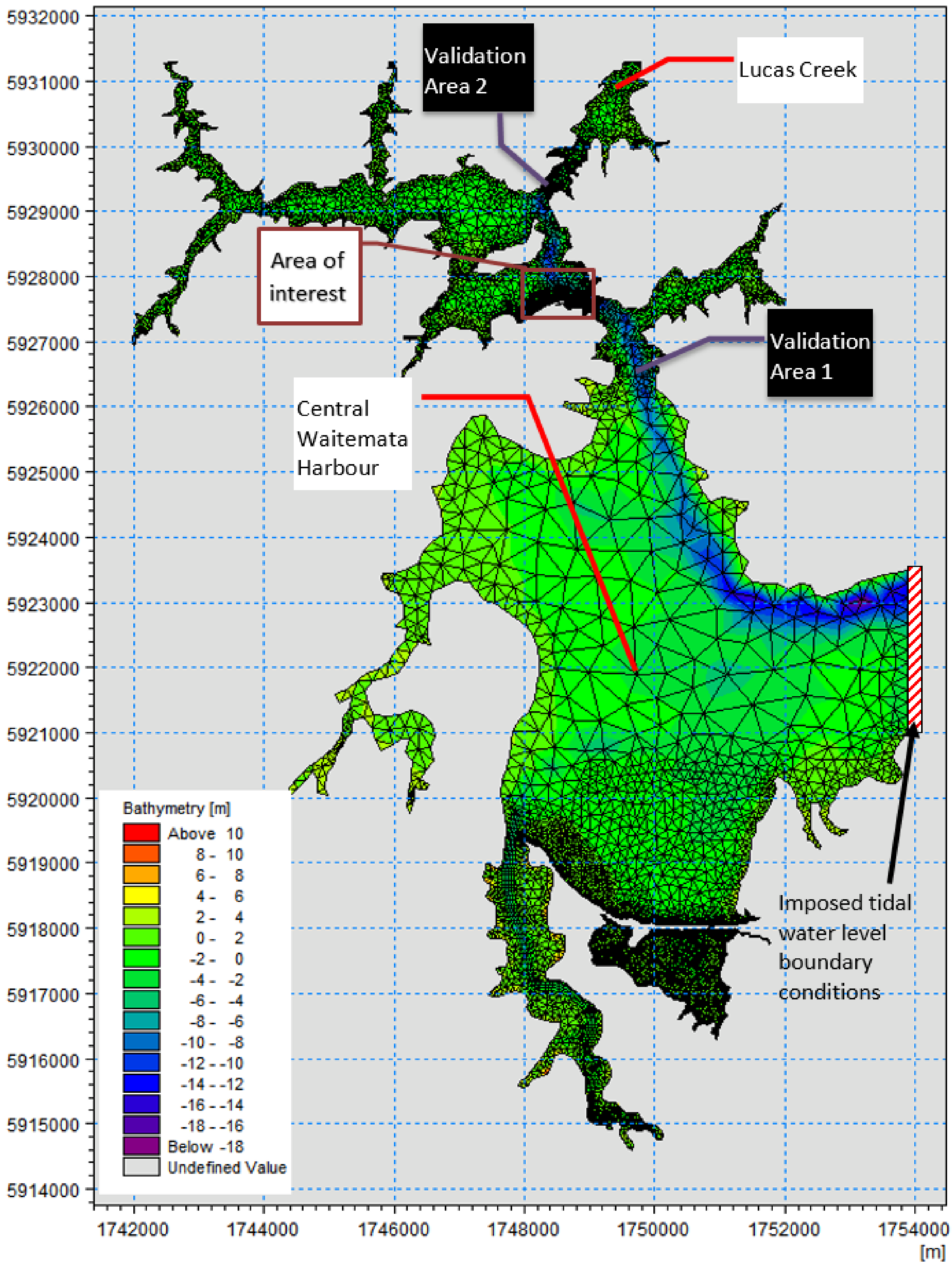


Figure 2-1: Model domain, boundaries and validation points

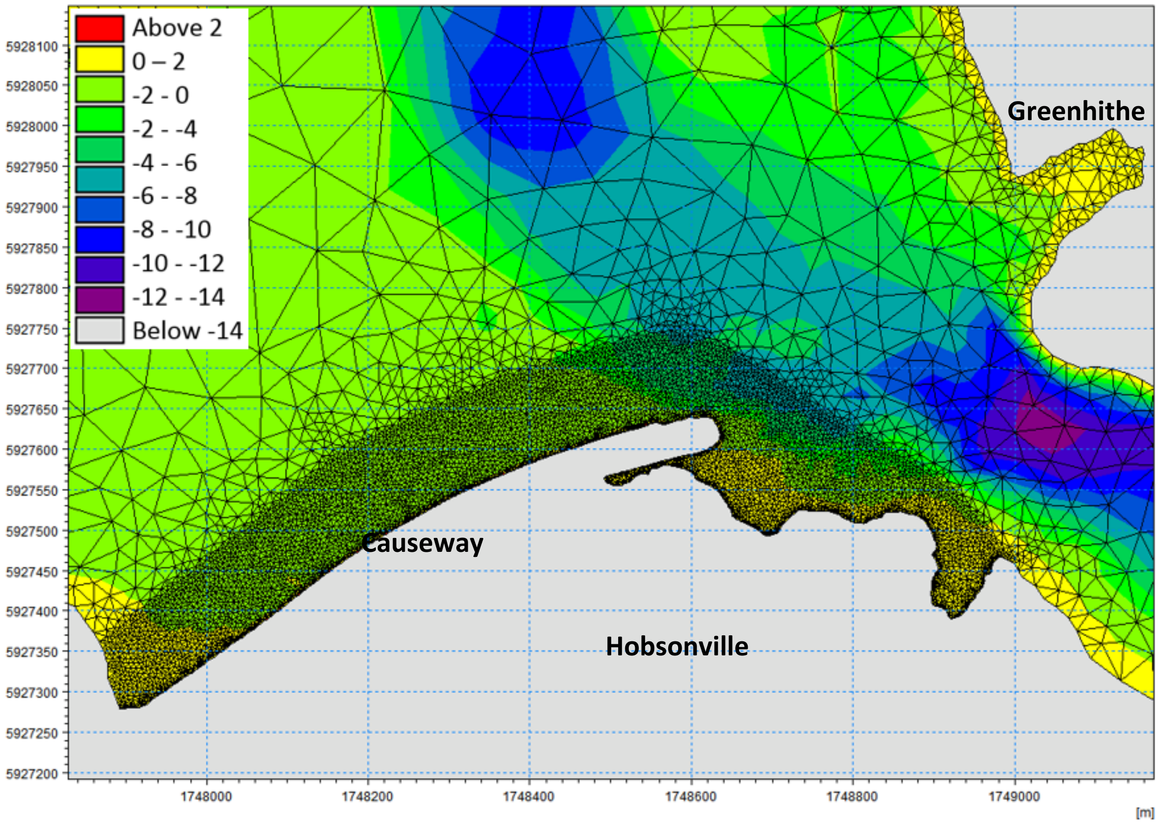


Figure 2-2: Flexible mesh grid configuration at area of interest for existing bathymetry

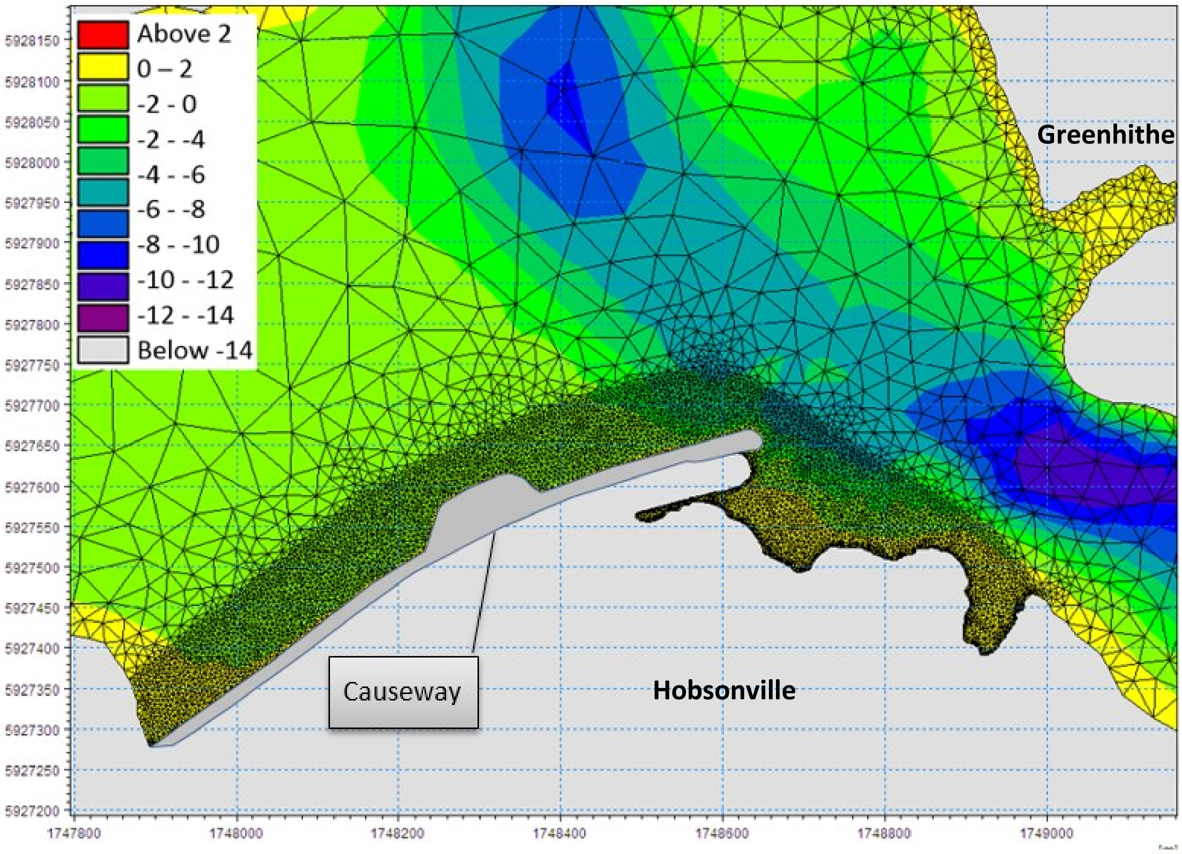


Figure 2-3: Flexible mesh grid configuration at area of interest for proposed bathymetry

2.2 Bathymetric data

2.2.1 Sources

The model bathymetry was obtained from several sources, these are:

- The existing MIKE 21 'Regional Harbour Model' (RHM) of the complete Waitemata Harbour and inner Hauraki Gulf
- Boat surveys provided by NIWA and Marine Discovery Ltd (in the Waterview Estuary, Oakley Inlet and Whau River areas)
- Causeway drawing for existing and proposed scenarios, provided by Watercare Services Ltd
- Bathymetric Survey provided by Scantec Geophysical Consultants (for Upper Harbour Bridge area)
- LiDAR (provided by Auckland Local Government Geospatial information), in the upper harbour region and aerial photographs
- LINZ depth contours.

These data sources were combined to generate the bathymetric grid shown in Figure 2-1.

2.2.2 Hierarchy

Data sources for the original Waterview model is described in detail in the Tonkin & Taylor, NIWA, 2010 Waterview model report. The table below provides more detailed description of the additional data used to assess effects of the proposed SH18 causeway widening

Layer number	Layer name	Comment	Exceptions
1	MIKE21 RHM	Used in all areas of the original Waterview model (excluding areas of higher resolution data)	
2	Boat surveys (NIWA and Marine Discovery Ltd)	Used in the Waterview Estuary, Oakley Inlet and Whau River areas	
3	Causeway drawing for existing and proposed scenarios, (WaterCare)	Used to reflect levels adjacent to the coast line for the existing and proposed scenarios	Around the bridge piers the existing depth has not been used due to it reflecting the level of the bridge, in this area the intertidal LiDAR data set is found to be more appropriate and is used
4	Bathymetric survey (Scantec Geophysical Consultants)	Used in the vicinity of the Upper Harbour Bridge area and southern bridge approach	In areas where there is an overlap between the Causeway drawings (in 3) and the bathymetric survey, the Causeway drawings data set has been used
5	Intertidal Lidar	Used in the Upper Harbour Bridge area (e.g. around piers and areas north of the upper harbour bride	
6	Depth contour (LINZ website)	Used at locations with no other data sets. i.e. not intertidal LiDAR nor any bathymetric survey data, e.g. Lucas Creek	

2.2.3 Datum and projection

The datum utilised in the model is Auckland Vertical Datum 1946 (AVD46). The projection is in New Zealand Transverse Mercator (NZTM).

2.3 Boundary conditions

The boundary conditions used in the model correspond to a typical spring tide and neap tide (Figure 2-4). All simulations used a nominal start date, as opposed to an actual period. Water flow in the model is driven by a time series of tidal water levels at the eastern boundary of the model. This time series was obtained from the calibrated RHM for both spring and neap tides.

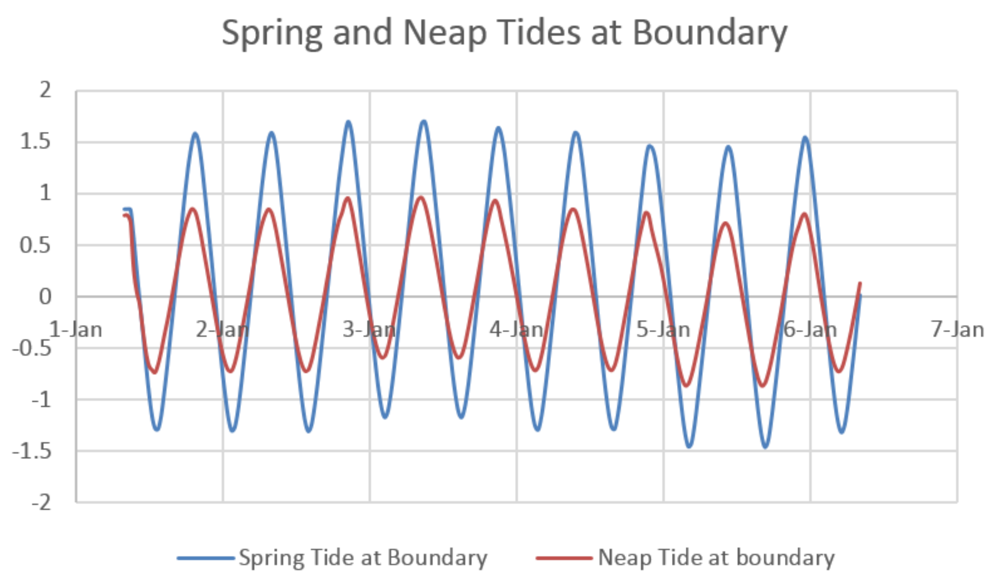


Figure 2-4: Spring and neap tides at model boundary

2.4 Model validation

The results of the model were verified using the 2010 Waterview model data set.

Surface elevation data near the 2010 northern boundary in the 2014 model (south of the Upper Harbour Bridge), was compared with the 2010 numerical model boundary condition used for the Waterview project.

A difference plot (Figure 2-5) provides a comparison between the time series. The RMSE for this data is 0.06 m showing a reasonable match between the two datasets.

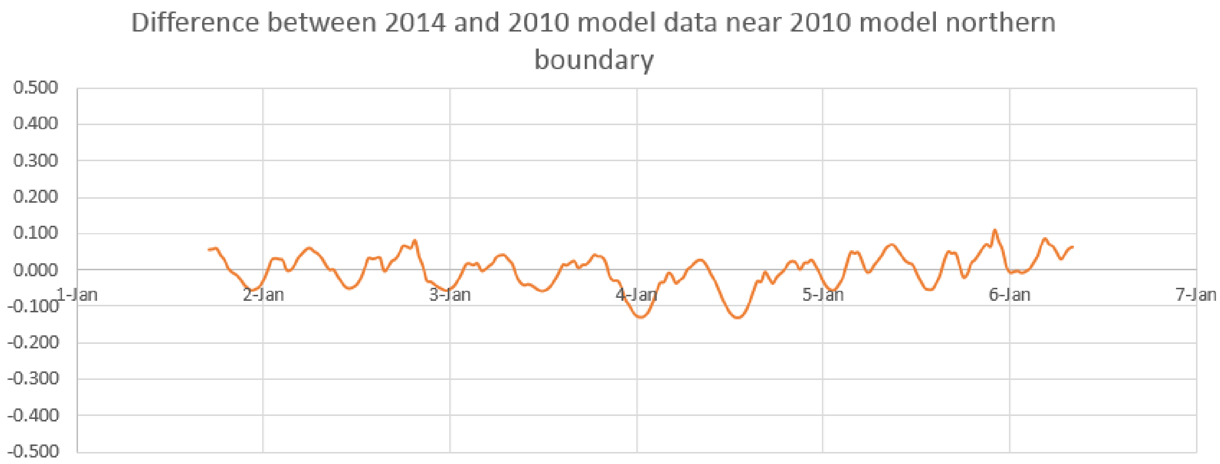


Figure 2-5: Validation - surface elevation showing the difference between the 2014 and 2010 model.

The original Waterview model validation can be found in Tonkin & Taylor, NIWA 2010 Waterview model report.

3 Results and analysis

3.1 Scenarios

Four scenarios were simulated, these are:

1. Existing bathymetry with spring tide
2. Proposed bathymetry (including causeway widening and extension) with spring tide
3. Existing bathymetry with neap tide
4. Proposed bathymetry (including causeway widening and extension) with neap tide.

Sections 3.2 to 3.4 of this report describe the results, including spatial plots for surface elevation, flood and ebb current speed, and flood and ebb shear stress for the spring tide.

In addition to the spatial plots provided in the sections above, data sets in specific locations as shown in Figure 3-1 have also been obtained. These are included in Appendices A and B. Data sets in appendices include:

- Tables showing maximum current speed at flood and ebb tides as well as corresponding depth and surface elevation for spring and neap tides (Appendix A)
- Time series showing surface elevation, current speed and total water depth for spring and neap tides (Appendix B).

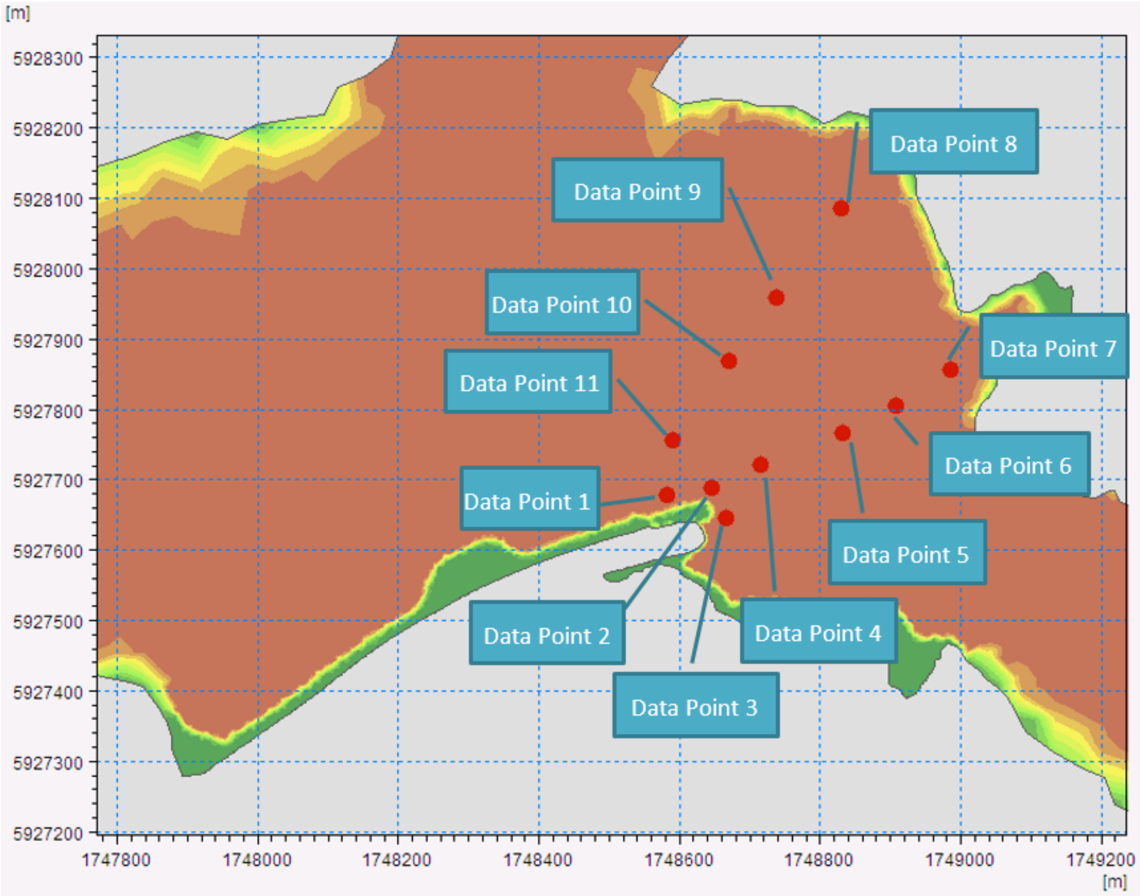


Figure 3-1: Location of output points for comparison of changes

3.2 Surface elevation

Model results include surface elevation (water level). For the existing scenario, in the vicinity of the Upper Harbour Bridge, the spring tide maximum surface elevation (shown in Figure 3-2) is 1.76 mRL to 1.78 mRL.

For the proposed scenario, in the vicinity of the Upper Harbour Bridge, the spring tide maximum surface elevation (shown in Figure 3-3) is also 1.76 mRL to 1.78 mRL.

Existing and proposed spring tide maximum surface elevation was compared through a difference plot. The maximum difference in water level (proposed scenario - existing scenario) is approximately - 0.0015 m to 0.001 m as is shown in Figure 3-4. This very small difference in levels suggests negligible changes to surface water levels as a result of the causeway widening and extension and these changes are unlikely to be measurable.

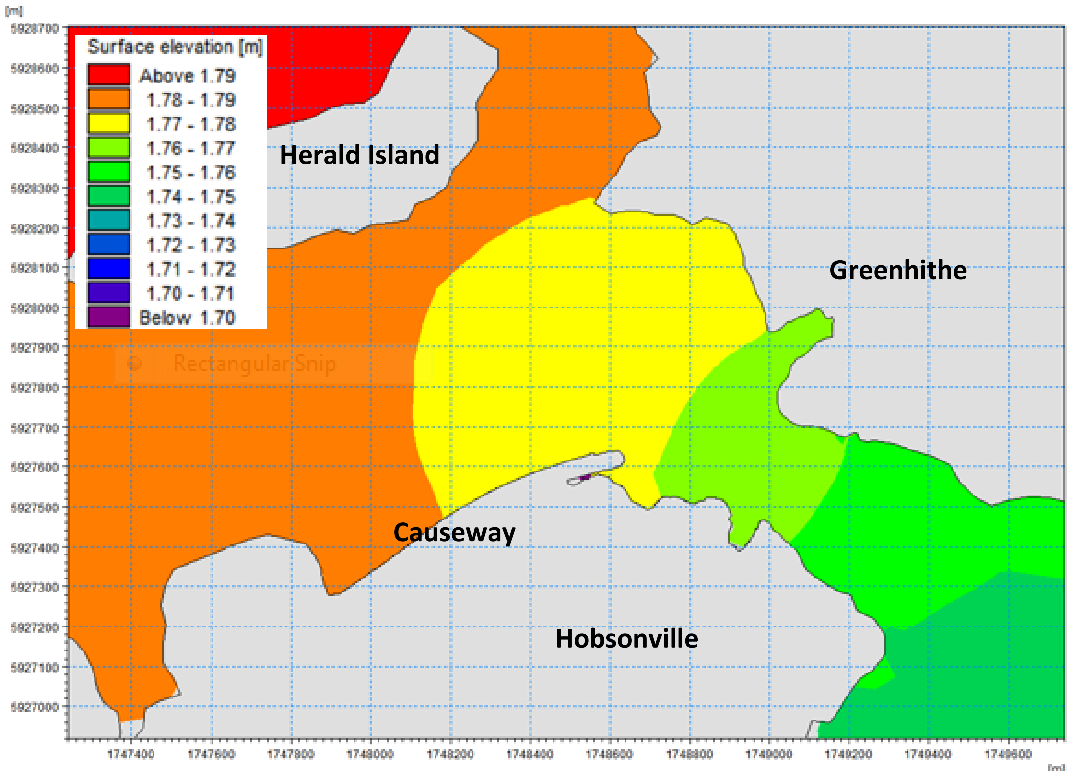


Figure 3-2: Existing scenario spring tide maximum surface elevation (Scenario 1)

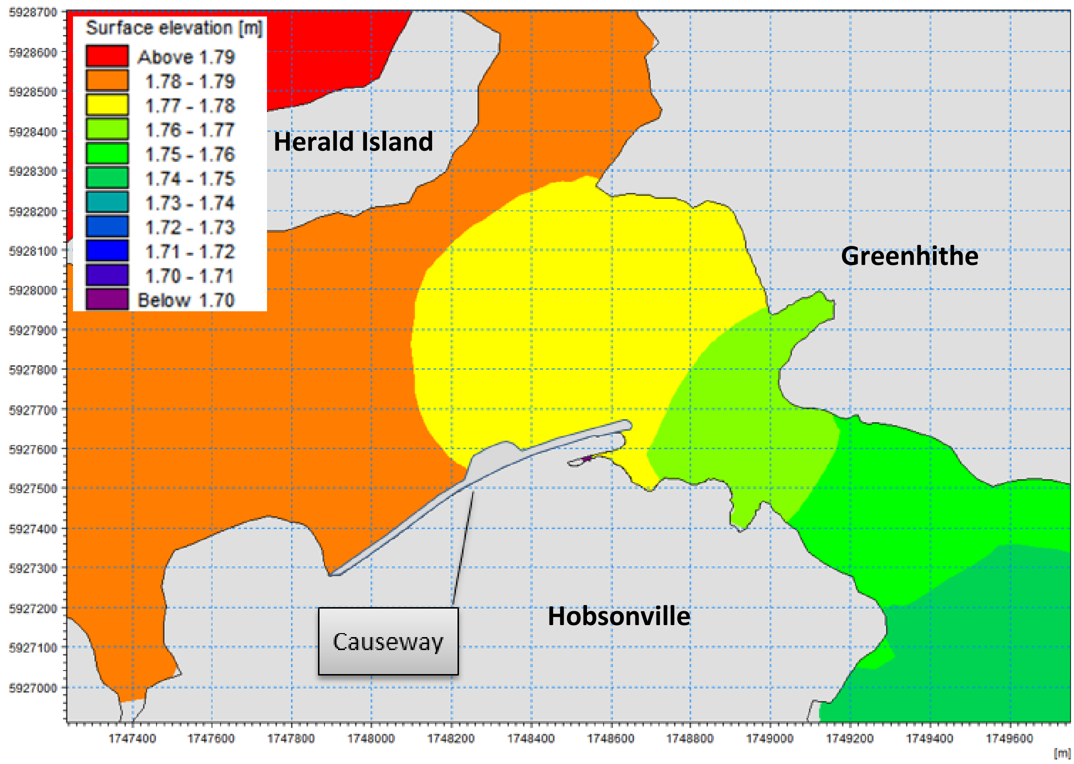


Figure 3-3: Proposed scenario spring tide maximum surface elevation (Scenario 2)

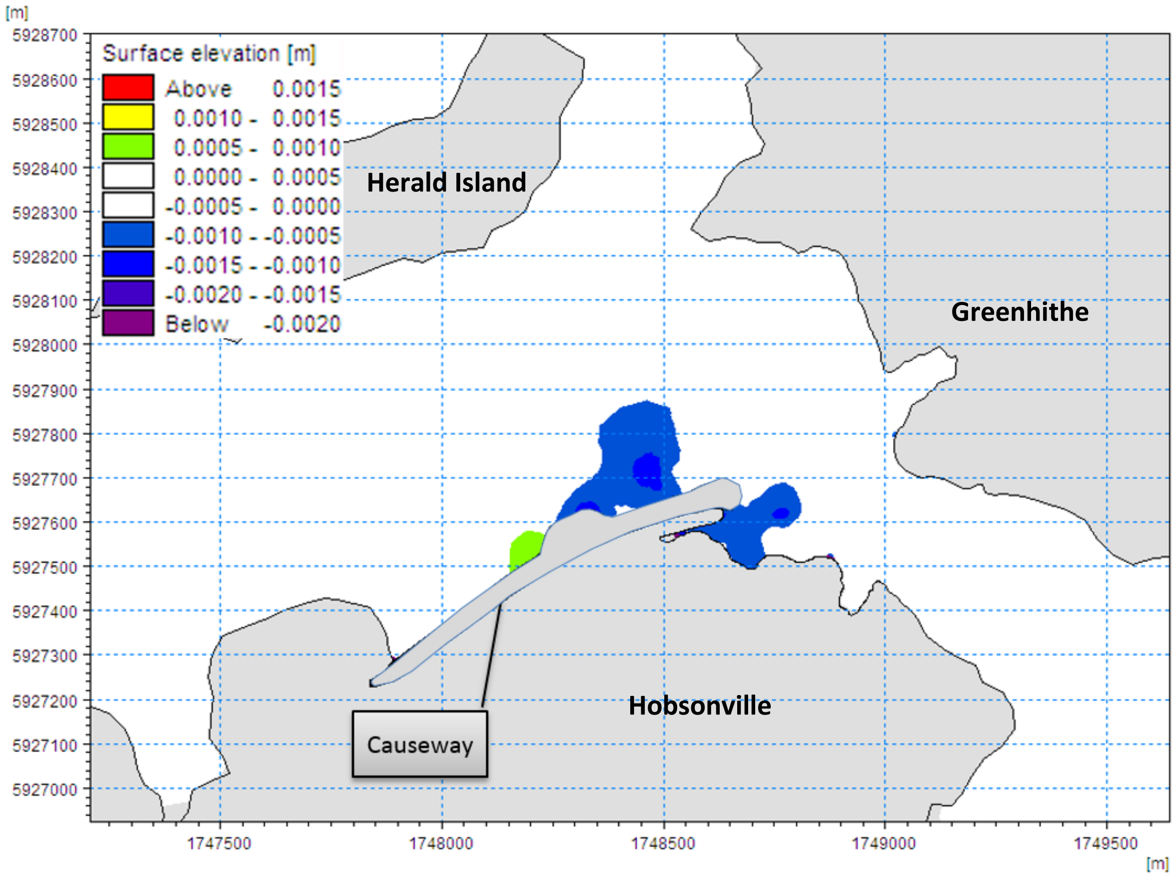


Figure 3-4: Difference between Scenario 1 and 2 in spring tide maximum surface elevation

3.3 Peak current speed

Model results also include peak current speed information (depth averaged velocity). Current speed was extracted and compared between existing and proposed scenarios, for typical flood and ebb tides.

3.3.1 Current speed – flood tide

In the vicinity of the Upper Harbour Bridge, the peak spring flood tide current speed ranges between:

- 0.1 m/s and 0.8 m/s for the existing scenario (shown in Figure 3-5) and the proposed scenario (shown in Figure 3-6).

Although this is essentially the same for both scenarios, minor differences in current speed are observed in several locations. This is shown in Figure 3-7, where:

- An increase in current speed is shown in the model results near the tip of the proposed causeway and is up to 0.1 m/s
- A decrease in current speed is shown in the model results immediately north and south of proposed causeway tip and is up to -0.25 m/s.

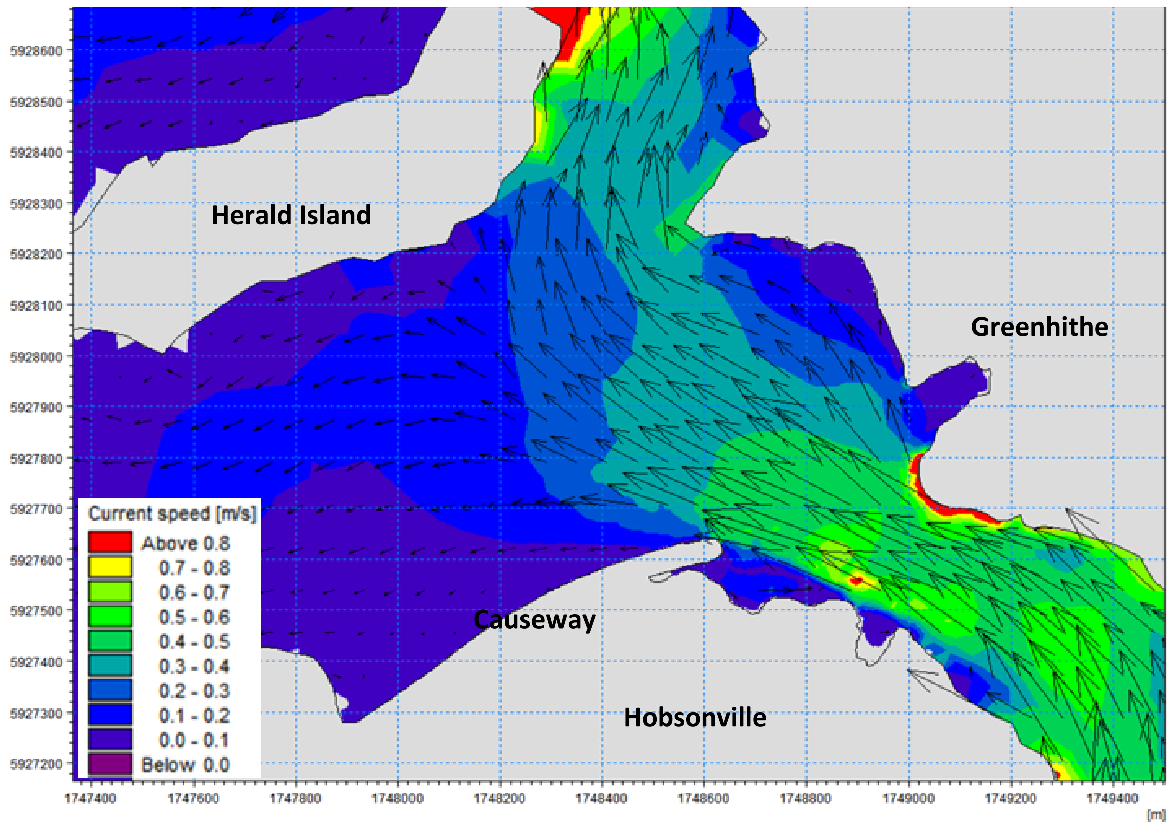


Figure 3-5: Existing situation (Scenario 1) - spring tide maximum flood current speed

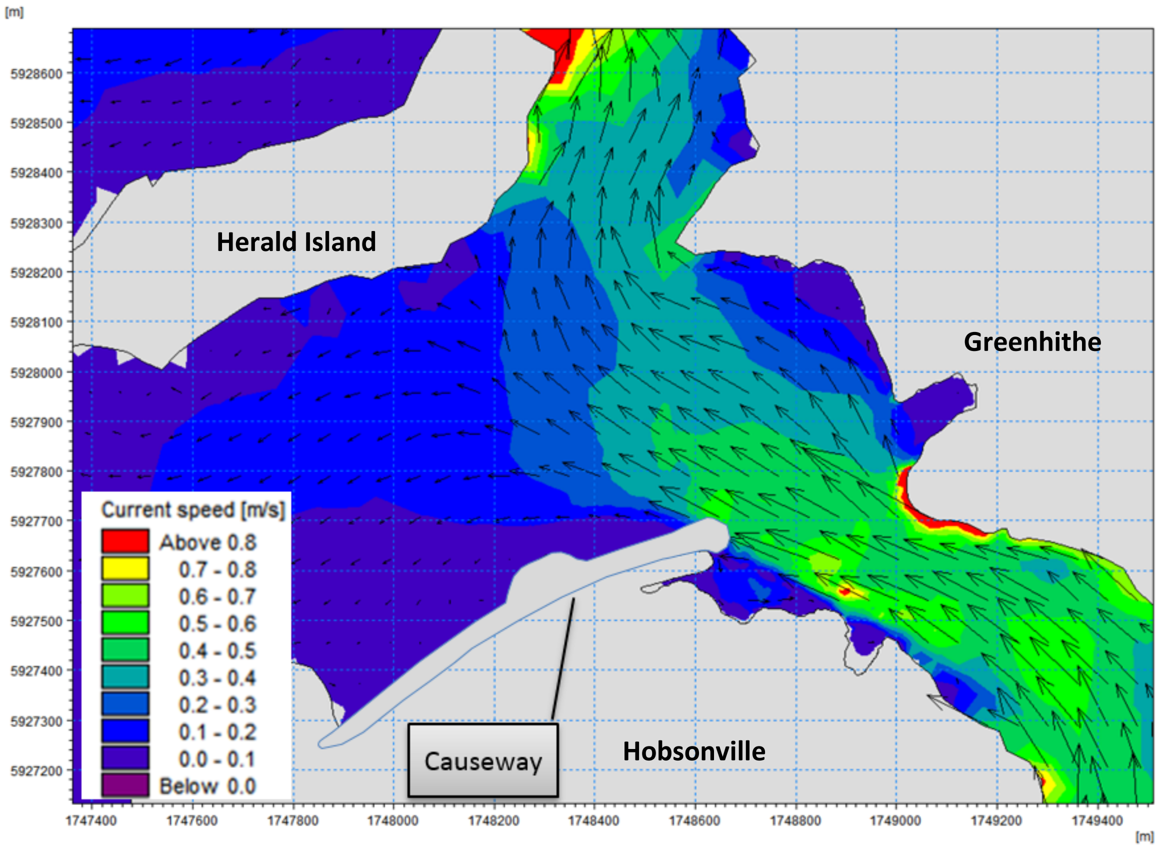


Figure 3-6: Proposed bathymetry (Scenario 2) - spring tide maximum flood current speed

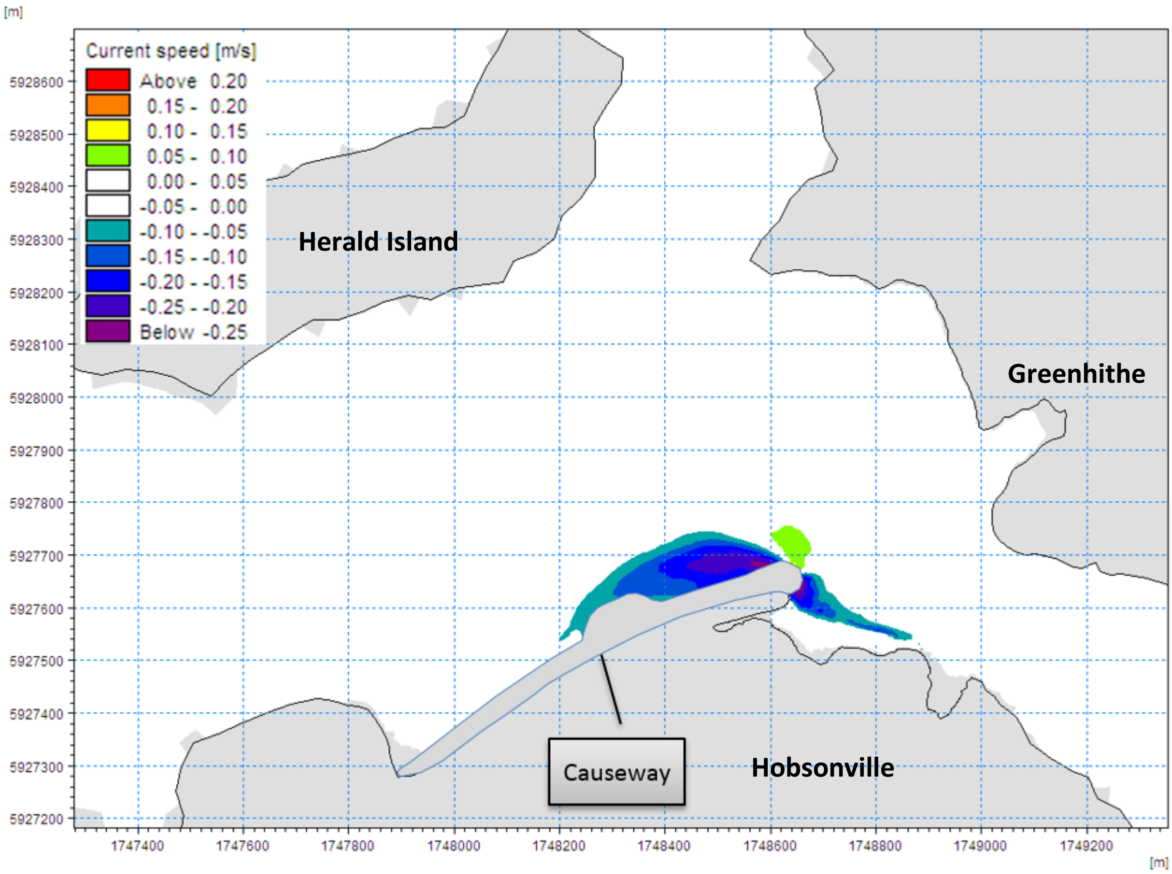


Figure 3-7: Difference in flood current speed for spring tide

3.3.2 Current speed – ebb tide

In the vicinity of the Upper Harbour Bridge, the peak spring ebb tide current speed ranges between:

- 0.1 m/s and 0.8 m/s for both Scenario 3 (shown in Figure 3-8) and Scenario 4 (shown in Figure 3-9).

Although this is essentially the same for both scenarios, minor differences in current speed are observed several locations. This is shown in Figure 3-10, where:

- An increase in current speed between is shown in the model results near the tip of the proposed causeway and is up to 0.1 m/s
- A decrease in current speed is shown in the model results immediately north and south of proposed causeway tip and is up to -0.25 m/s.

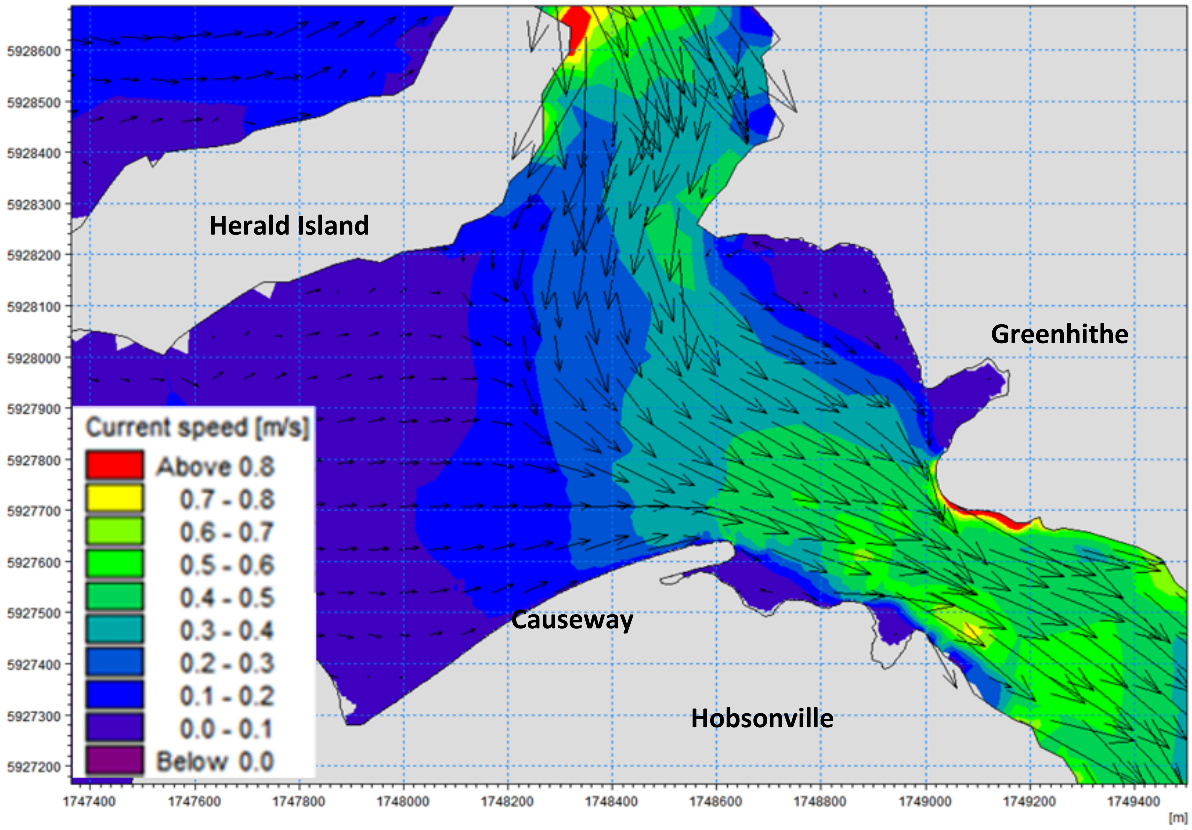


Figure 3-8: Existing bathymetry (Scenario 1) - spring tide maximum ebb current speed

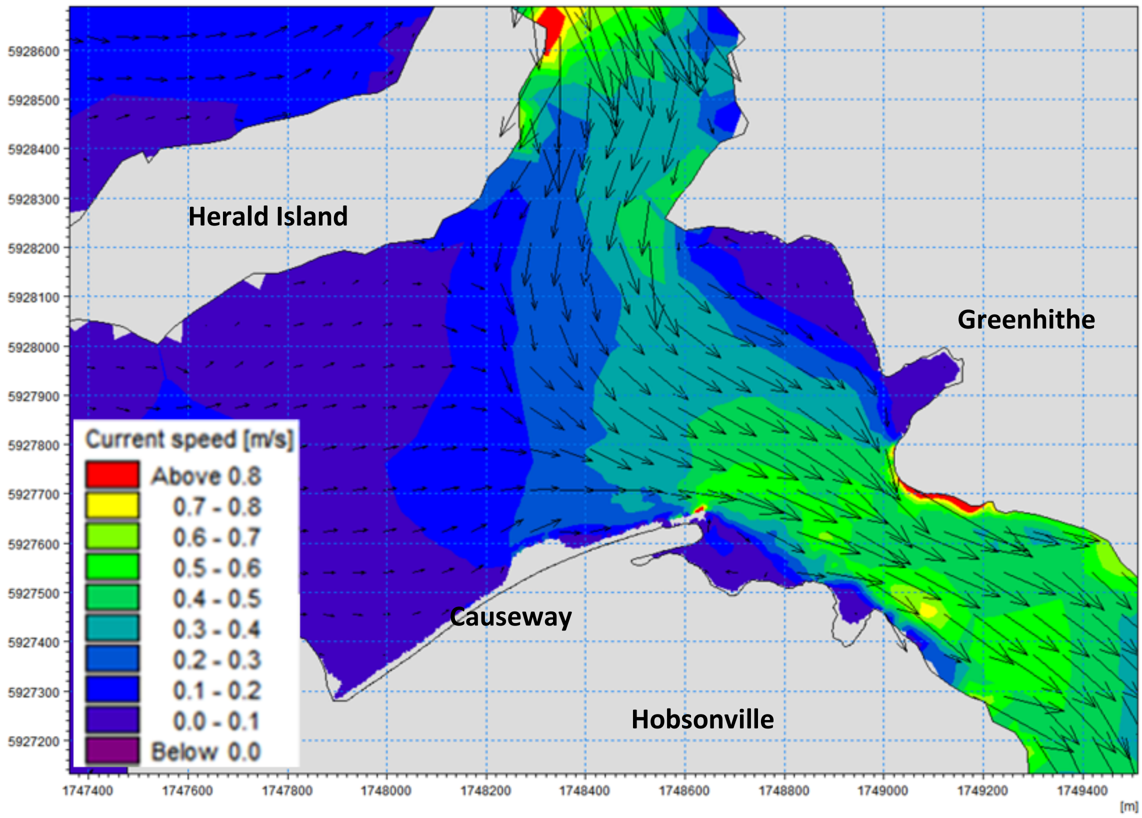


Figure 3-9: Proposed bathymetry (Scenario 2) - spring tide maximum ebb current speed

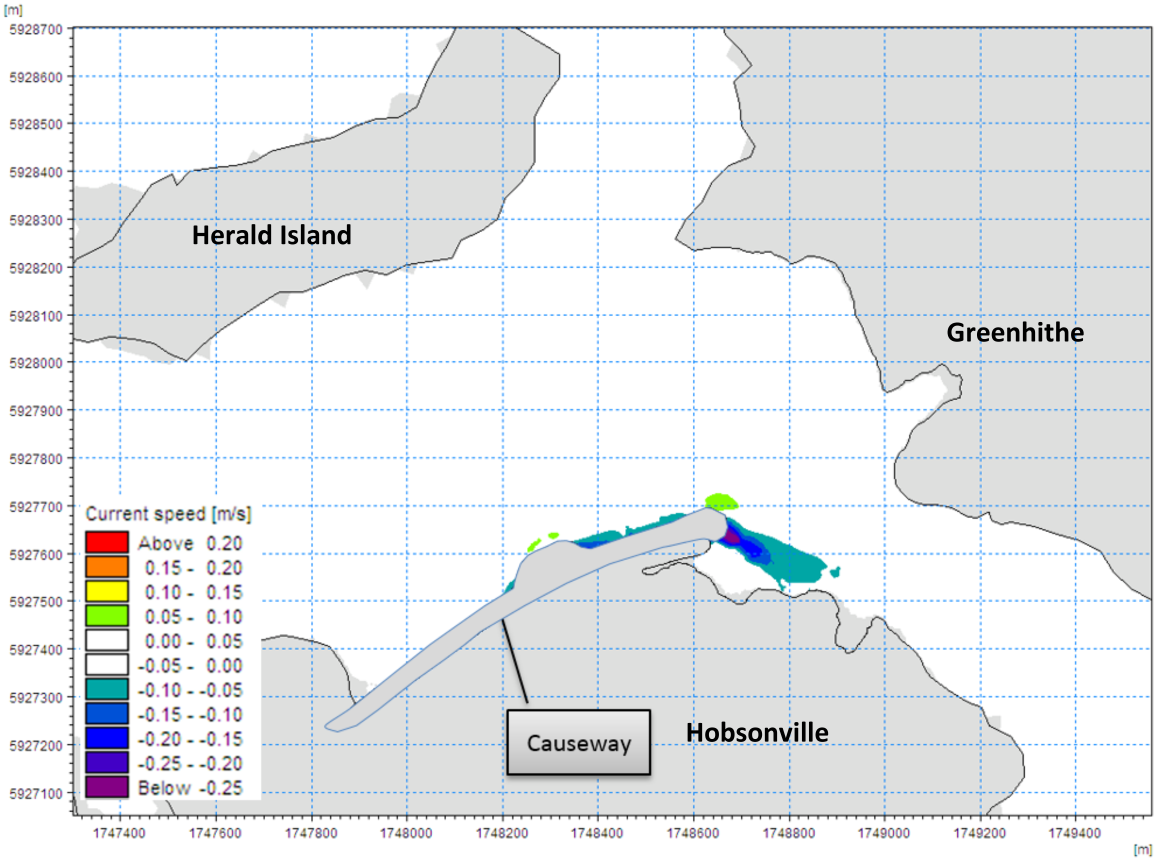


Figure 3-10: Difference in ebb current speed for spring tide

Current speed information at two specific locations for both spring and neap tides are presented below. This is to further illustrate the effects of the proposed causeway on current speed in the vicinity of the bridge. The two data points chosen for this discussion are:

- Data point 2: this is chosen as it represents an area affected by the proposed causeway
- Data point 5: this is chosen as it is considered representative of the channel under the bridge.

Time series and peak current speed information from all data points are included in Appendix A and B.

Table 3.1: Maximum current speed for spring and neap tides during flood and ebb tides

Location (refer Figure 3-1)	maximum current speed (m/s)							
	Flood existing	Flood proposed	Difference in flood tide (m/s, %)		Ebb existing	Ebb proposed	Difference in ebb tide (m/s, %)	
Data Point 2 spring tide	0.478	0.565	0.087	18.2%	0.462	0.548	0.086	18.6%
Data Point 2 neap tide	0.234	0.272	0.038	16.2%	0.245	0.285	0.041	16.7%
Data Point 5 spring tide	0.497	0.504	0.007	1.4%	0.464	0.48	0.016	3.4%
Data Point 5 neap tide	0.244	0.247	0.003	1.2%	0.251	0.26	0.01	4.0%

Table 3.1 shows that:

- The differences in current speed between the proposed and existing scenarios for a typical flood tide at representative locations under the bridge and in the vicinity of the causeway are:
  - Increases of up to 0.087 m/s near the proposed cause way and 0.007 m/s under the Upper Harbour Bridge for the spring tide
  - 0.038 m/s near the proposed cause way and 0.003 m/s under the Upper Harbour Bridge for the neap tide.
- The differences in current speed between the proposed and existing scenarios for a typical ebb tide at representative locations under the bridge and in the vicinity of the causeway are:
  - 0.086 m/s near the proposed cause way and 0.016 m/s under the Upper Harbour Bridge for the spring tide
  - 0.041 m/s near the proposed cause way and 0.01 m/s under the Upper Harbour Bridge for the neap tide.

As expected at this location peak current speed is higher during spring tide than during neap tide for both flood and ebb tides. The difference between the existing and proposed scenario is also larger during neap tides than flood tides. Changes at Data Point 5 are negligible (less than 4%). With physical measurement accuracy limited to around ±0.008 m/s the modelled changes under the Upper Harbour Bridge are unlikely to be measurable.

Changes in velocity at other data points are shown in Table A1-3 and A1-6 (Appendix A) for spring and neap tides respectively. Table A1-3 table shows the reduction in velocity at Data Points 1 and 3 and negligible changes at Data Points 5, 6, 7 and 8. Within the main channel there are slightly greater velocities during flood tide conditions (Data Points 9, 10 and 11) and a slight increase at Data Point 8 on the edge of the main channel. The remaining points show negligible change (less than 4%). This confirms the effects of the causeway extension are localised around the end of the extension as shown in Figure 3-7 and Figure 3-10. During neap tide conditions, velocities are lower but the percentage changes are in a similar order (Table A1-6). The only difference in trend can be seen at Data Point 11 where neap ebb tide conditions change to a very slight reduction in velocity compared to no change during spring tide.

Overall, velocity changes are not likely to significantly affect sediment transport regimes. In the main channel existing velocities are able to transport sediment, while on the intertidal areas velocities are generally lower than the threshold to move sediment and the proposed changes do not change the velocity regimes. However, this is examined in more detail in the following section which looks at erosion and accretion potential via an analysis of shear stress.

3.4 Shear stress

The shear stress at the sediment-water interface generated by water flowing over the bed surface is a primary determinant of the extent to which materials settling out of the water column are deposited on the bed or are eroded from it, and whether particles in motion are transported as suspended load or bed load. Given maximum the flood and ebb current speed (and corresponding water depth), peak shear stress due to tidal currents can be calculated using formula 2.2.17 from Leo C. van Rijn (1993):

$$\tau_{b,c} = \rho \, \mathrm{g} \, \mathrm{h} \, \mathrm{I} = \rho \, g \, \frac{\overline{\mathrm{u}}^2}{\mathrm{C}^2} = \frac{1}{8} \rho \, \mathrm{f}_c \, \overline{\mathrm{u}}^2$$

Where:

$\mathrm{h} =$ water depth (m)

$\mathrm{I} =$ energy line gradient (-)

$\overline{\mathrm{u}}$ = depth-averaged peak velocity (m/s)

C = Chezy-coefficient ($C^2 = 8g/f_c$), ($m^{0.5}/s$)

$\mathrm{f}_c =$ Fraction factor of Darcy Weisbach (-) $= \mathrm{f}_c = 0.24 \left[\log\left(\frac{12h}{k_s}\right)\right]^{-2}$

k_s = effective bed roughness height (m), for a plane bed $k_s = 3d_{90}$*1

ρ = fluid density (kg/m^3) = 1024

$g =$ acceleration of gravity (m/s^2)

*1 Assuming $d_{90} =$ 0.4 mm based on PSD for 'Upper Waitemata Harbour Borehole 3, Sample 3, Depth 0.8 m. This is a conservative close to lower bound based on sediment sampling where values ranges from 0.25 mm to 40 mm and the average value was 2 mm.

The resulting shear stress plots are shown in Figure 3-11 to Figure 3-16.

Erosion occurs when grain shear stress at the sediment surface exceeds the critical shear stress. This can be estimated from the formula of Guo (2002):

$$\tau_{ce} = (G_p - 1)\rho g d_p \left(\frac{0.23}{d_*} + 0.054\left[1 - exp\left(-\frac{d_*^{0.85}}{23}\right)\right]\right)$$

And

$$d_* = d_p \left[\frac{(G_p - 1)g}{\vartheta^2}\right]^{-1/3}$$

Where:

τce = critical shear stress for erosion [ML-1T-2]

Gp = sediment particle specific gravity ≈ 2.65 [dimensionless]

dp = sediment particle diameter

d* = dimensionless particle diameter [dimensionless]

v = kinematic viscosity [L2T-1]

Using this formula the critical bed-shear stress for the lower bound of sediment grading is around 0.2 N/m2 and is around 0.8 N/m2 for average grain size properties. This means that where shear stress values are less than 0.2 N/m2 no erosion due to tidal currents are expected and between 0.2 N/m2 and 0.8 N/m2 some erosion of finer sediments can be expected.

3.4.1 Shear stress – flood tide

The shear stress for the existing situation is shown in Figure 3-11. Figure 3-12 shows the shear stress for the proposed development and Figure 3-13 shows the difference in shear stress. The intertidal areas all have shear stress values of below 0.2 N/m^2 indicating no erosion due to tidal currents are likely in these areas. Within the tidal channel shear stress values range from 0.2 N/m^2 to 2.1 N/m^2. There are no areas within the immediate vicinity of the proposed causeway showing shear stress larger than the critical shear stress for average sediment gradings of 0.8 N/m^2. The only exception is at the very tip of the proposed causeway which is likely to be due to small scale bathymetric inaccuracies under the existing bridge.

In the vicinity of the Upper Harbour Bridge and proposed causeway, the spring flood tide shear stress ranges between:

- 0.4 N/m^2 in the channel to 2.1 N/m^2 near the western bridge abutments and western coastal boundaries, for the existing scenario (shown in Figure 3-11)
- 0.4 N/m^2 in the channel to 2.1 N/m^2 near the bridge abutments and western coastal boundaries, for the proposed scenario (shown in Figure 3-12).

Although this is essentially the same for both scenarios, the difference in shear stress is observed at several locations. This is shown in Figure 3-13, where:

- An increase in shear stress suggests increased erosion potential, this is shown near the tip of the proposed causeway and is up to 0.2 N/m^2, covering an area approximately 4,100 m^2
- A decrease in shear stress suggests increased deposition potential, this is shown immediately north and south of the proposed causeway tip and is up to -0.3 N/m^2, covering an area approximately 21,000 m^2.

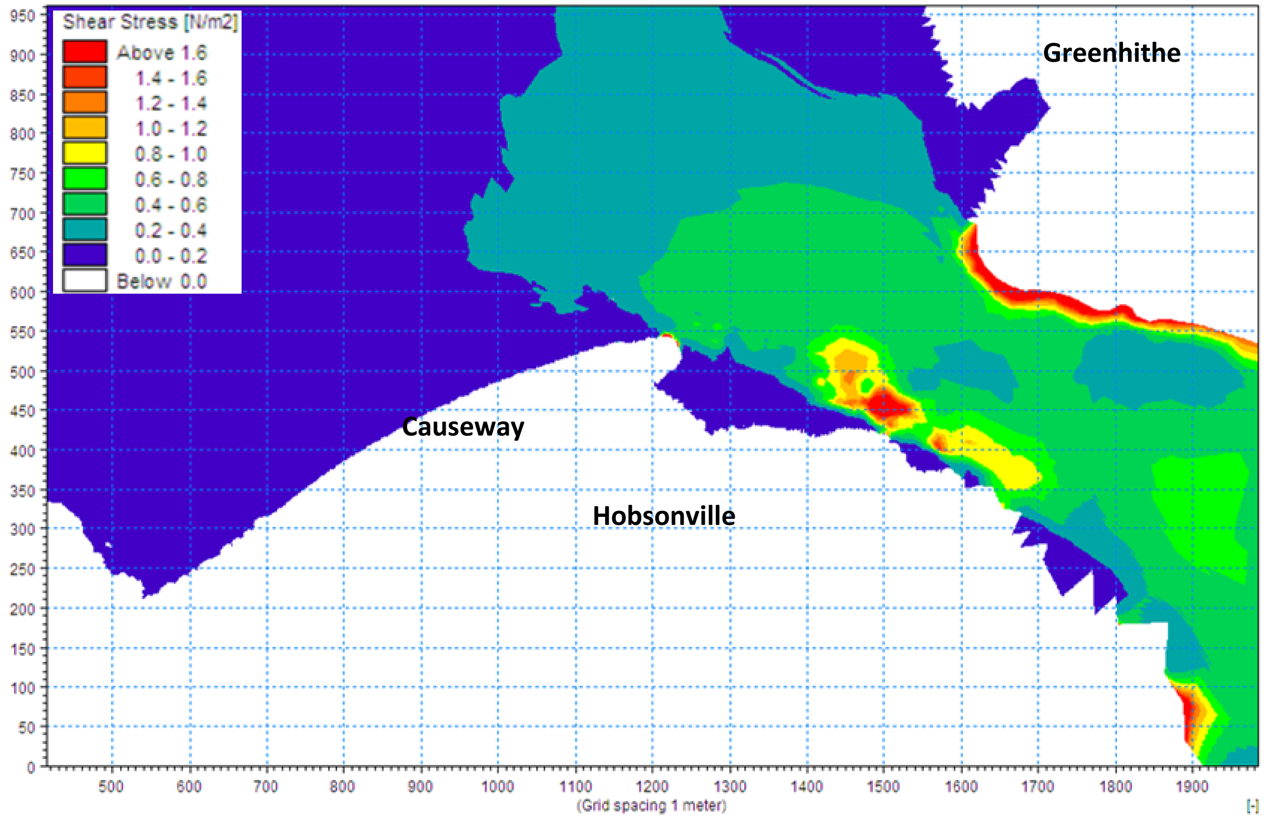


Figure 3-11: Existing scenario – spring flood tide maximum shear stress

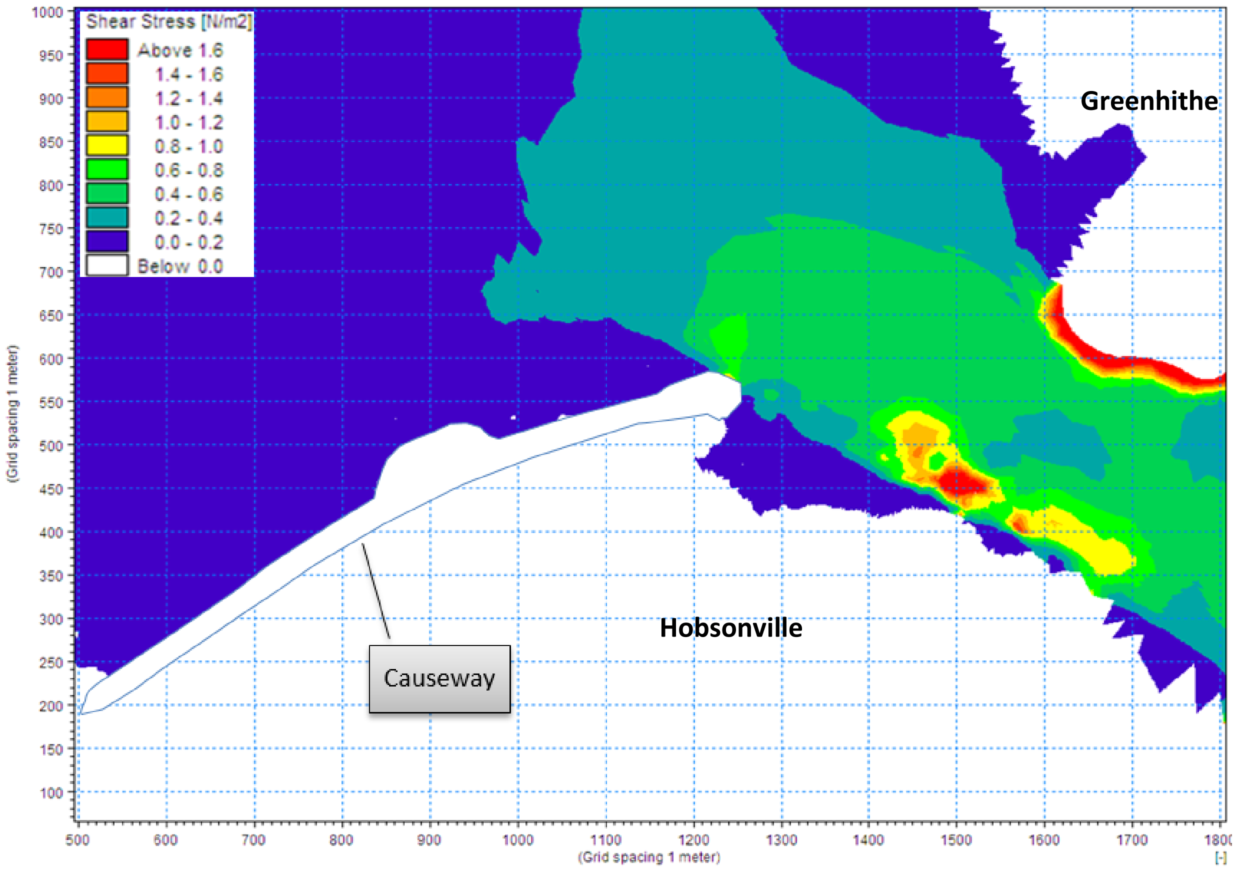


Figure 3-12: Proposed scenario spring flood tide maximum shear stress

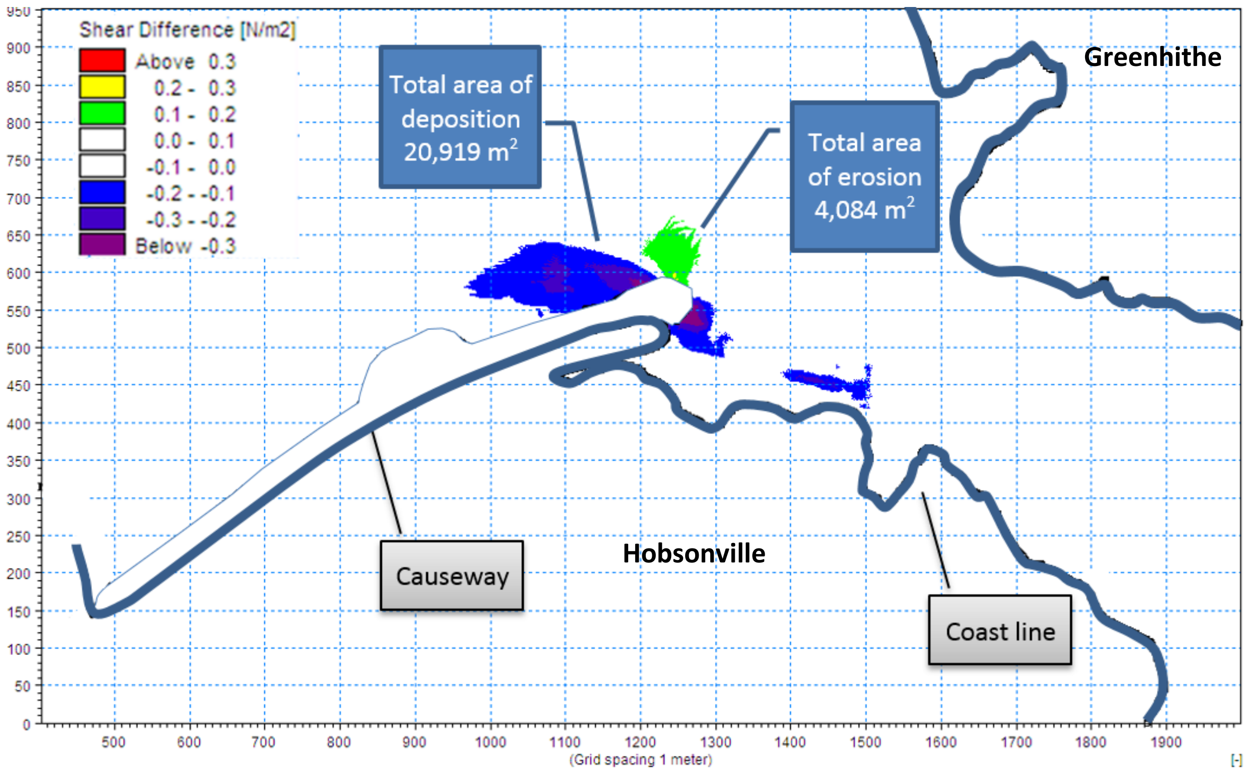


Figure 3-13: Difference in shear stress for spring flood tide showing areas of potential erosion and deposition

3.4.2 Shear stress – ebb tide

The shear stress for the existing situation is shown in Figure 3-14. Figure 3-15 shows the shear stress for the proposed development and Figure 3-16 shows the difference in shear stress. Shear stress was calculated using the formula in Section 3.4 for a typical spring ebb tide. There are no areas within the immediate vicinity of the proposed causeway showing shear stress larger than the critical shear stress of 0.8 N/m^2.

In the vicinity of the Upper Harbour Bridge and proposed causeway, the spring ebb tide shear stress ranges between:

- 0.4 N/m^2 in the channel to 1.8 N/m^2 near the western bridge abutments and western coastal boundaries, for the existing scenario (shown in Figure 3-14)
- 0.4 N/m^2 in the channel to 1.8 N/m^2 near the bridge abutments and western coastal boundaries, for the proposed scenario (shown in Figure 3-15).

Although this is essentially the same for both scenarios, the difference in shear stress is observed at several locations. This is shown in Figure 3-16, where:

- An increase in shear stress suggests increased erosion potential, this is shown near the tip of the proposed causeway and is up to 0.2 N/m^2, covering an area approximately 2,200 m^2
- A decrease in shear stress suggests increased deposition potential, this is shown immediately north and south of the proposed causeway tip and is up to -0.3 N/m^2, covering an area approximately 14,400 m^2.

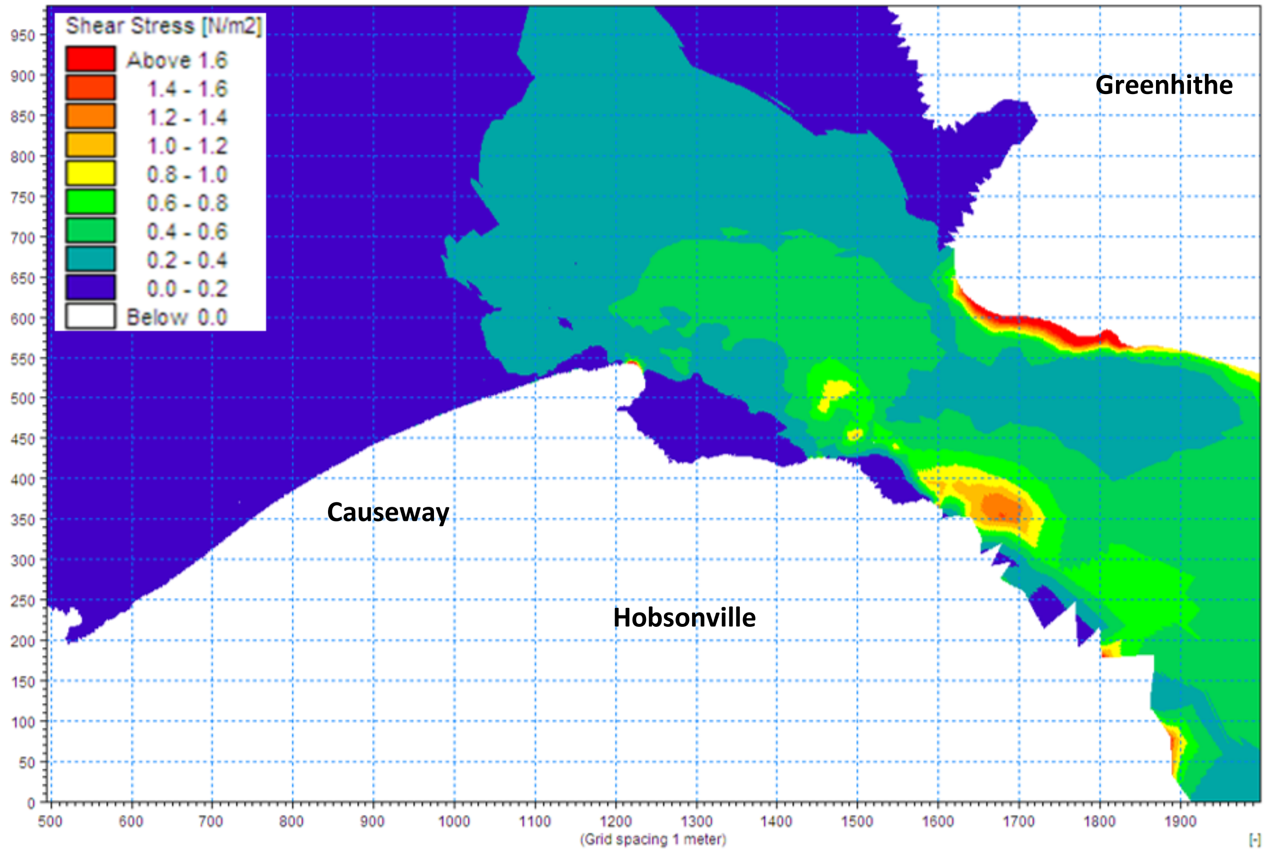


Figure 3-14: Existing scenario - spring ebb tide maximum shear stress

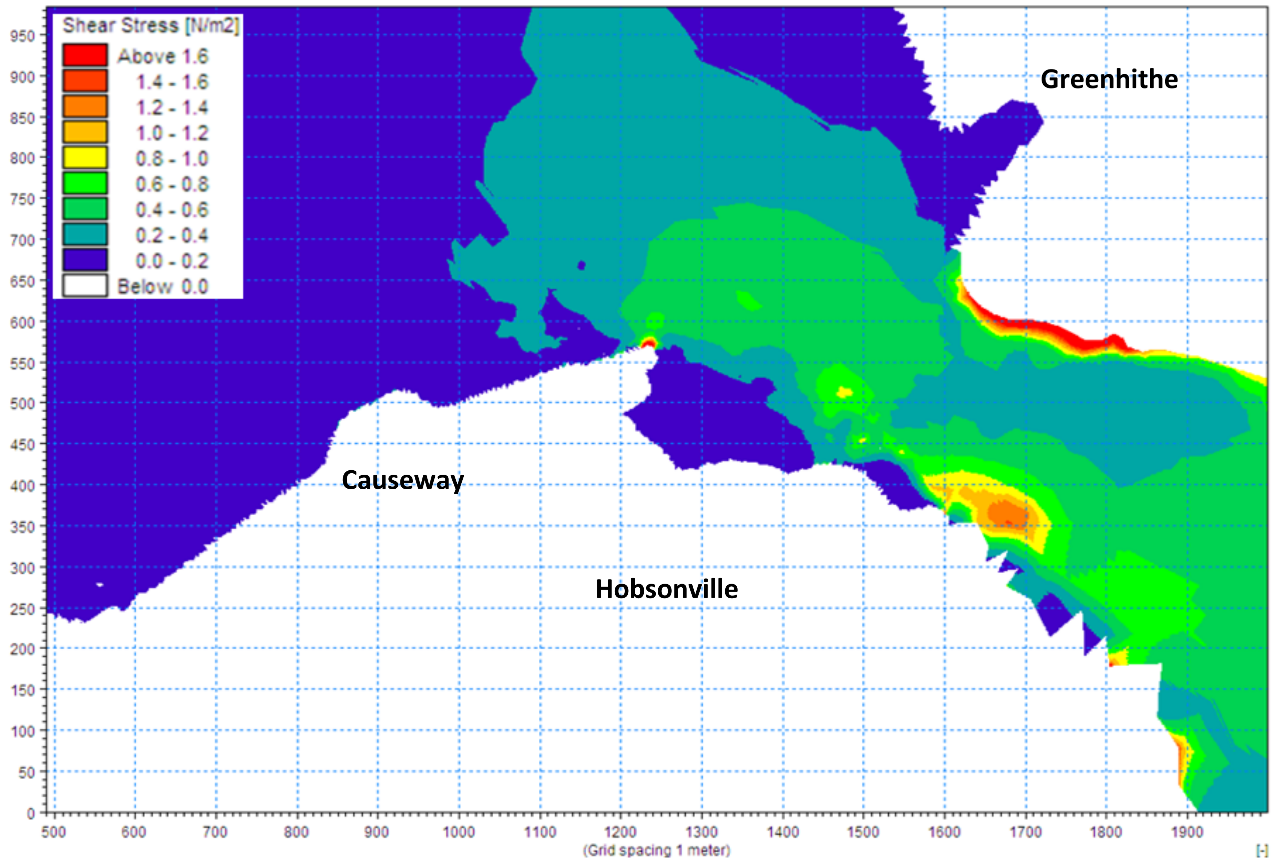


Figure 3-15: Proposed scenario - spring ebb tide maximum shear stress

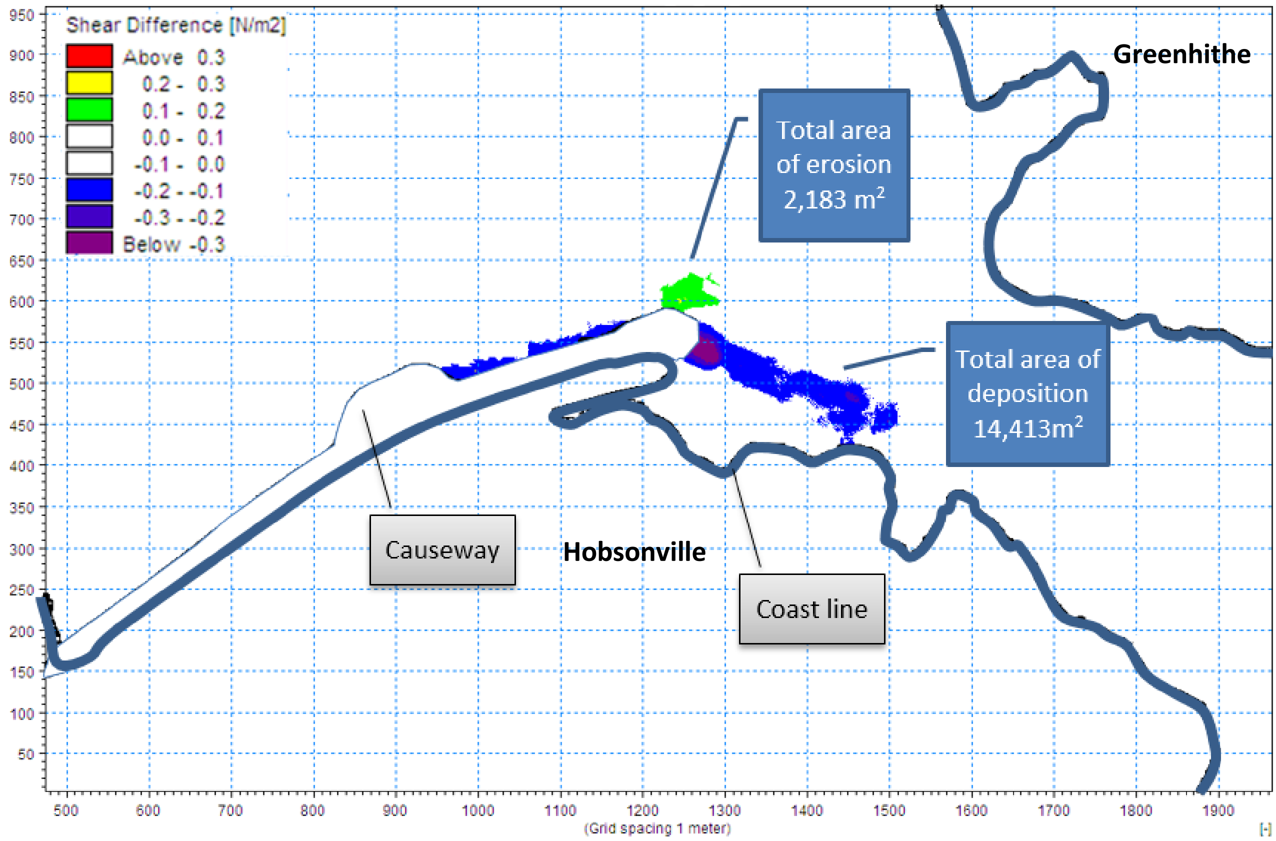


Figure 3-16: Difference in shear stress for spring ebb tide showing areas of potential erosion and deposition

4 Summary

The model built is based on data from the 2010 Waterview model (extended north using various data sets to include the Upper Harbour Bridge area and northern parts of the Waitemata Harbour).

Model validation shows a reasonable match between the Upper Harbour model and the Waterview model.

Model scenarios were:

1. Existing bathymetry with spring tide
2. Proposed bathymetry (causeway incorporated) with spring tide
3. Existing bathymetry with neap tide
4. Proposed bathymetry (causeway incorporated) with neap tide.

Results show that:

- the difference between the existing and proposed scenarios for a spring tide are:
  - surface elevation: between -0.0015 m and 0.001 m. These are very small changes and unlikely to be detectable
  - flood current speed: between -0.25 m/s (reduction in velocity) and 0.1 m/s (increase in velocity)
  - ebb current speed: between -0.25 m/s (reduction in velocity) and 0.1 m/s (increase in velocity)
- the differences in current speed between the proposed and existing scenarios for flood tide at representative locations (under the bridge and in the vicinity of the causeway) are:
  - 0.087 m/s near the proposed causeway and 0.007 m/s under the Upper Harbour Bridge for the spring tide. These results show away from the immediate vicinity of the causeway extension there is only minor changes in tidal velocity during spring tide.
  - 0.038 m/s near the proposed causeway and 0.003 m/s under the Upper Harbour Bridge for the neap tide. Due to the lower currents during neap tide changes are less than for spring tide.
- the differences in current speed between the proposed and existing scenarios for ebb tide at representative locations under the bridge and in the vicinity of the causeway are:
  - 0.086 m/s near the proposed causeway and 0.016 m/s under the Upper Harbour Bridge for the spring tide. These changes are in a similar order of magnitude near the causeway, but identify slightly greater effects during ebb tide than observed for flood tide under the bridge.
  - 0.041 m/s near the proposed causeway and 0.01 m/s under the Upper Harbour Bridge for the neap tide. These changes are in a similar order of magnitude near the causeway, but identify slightly greater effects during ebb tide than observed for flood tide under the bridge.
- The shear stress analysis shows that the effect of the velocity changes are only detectable in changing the existing seabed erosion potential in the immediate vicinity of causeway, but even at these locations the change in shear stress is small and that due to reductions in velocity any sediment eroded is likely to be deposited along the channel or intertidal areas immediately adjacent to the causeway.
- For a typical spring flood tide, shear stress analysis shows that :
  - the area of deposition potential is approximately 21,000 m^2
  - the area of erosion potential is approximately 4,100 m^2.

- for a typical spring ebb tide, shear stress analysis shows that :
  - the area of deposition potential is approximately 14,400 m^2
  - the area of erosion potential is approximately 2,200 m^2.

5 Conclusion

The hydrodynamic model study examining changes to tidal currents and erosion potential has shown that the proposed causeway widening and extension has minor and localised effects on tidal currents and shear stress. The causeway widening has no discernible effect on coastal processes while the extension will result in localised small scale erosion in the vicinity of the causeway extension and deposition in the areas of reduced tidal currents within the intertidal area adjacent to the causeway and along the south-western bank of the main channel.

6 Applicability

This report has been prepared for the benefit of Watercare Services Ltd with respect to the particular brief given to us to support an Assessment of Environmental Effects to assist in characterising the existing coastal environment and understand the potential effects of widening and extending the existing causeway. It may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

Report prepared by:	Authorised for Tonkin & Taylor by:
....	....
Sarah Basheer	Richard Reinen-Hamill
Water Resources Engineer	Project Director

SGB; RRH
p:\29719\workingmaterial\report\appendix c - coastal modelling report\2015-03-03.rrh.coastal modelling report.r2.docx

7 References

Guo, J. 2002. Hunter Rouse and Shields diagram. Proceedings of the 13th International Association of Hydraulic Research. Asian and Pacific Division Congress, Singapore, Malaysia, August 6-8.

Van Rijn, L.C. (1993). Principles of sediment transport in rivers, estuaries and coastal seas. Aqua publications, the Netherlands.

Tonkin & Taylor, NIWA (2010) Western Ring Route – Waterview Connection, G.14 Assessment of Coastal Processes.

Appendix A: Data Point Tables for maximum current speed at flood and ebb tides with corresponding depth and surface elevation

Table A1.1: Data point locations

Data Point 1	Easting	Northing
Data Point 2	1748582	5927679
Data Point 3	1748643	5927705
Data Point 4	1748688	5927664
Data Point 5	1748714	5927721
Data Point 6	1748831	5927766
Data Point 7	1748908	5927805
Data Point 8	1748986	5927856
Data Point 9	1748830	5928087
Data Point 10	1748739	5927959
Data Point 11	1748671	5927868

Table A1.2: Surface elevation at maximum current speed – spring tide (flood and ebb)

	Spring tide surface elevation at maximum current speed for flood and ebb tides (mRL)					
	Spring flood existing	Spring flood proposed	Difference in flood tide	Spring ebb existing	Spring ebb proposed	Difference in ebb tide
Data Point 1	0.893	0.891	-0.002	0.991	0.993	0.002
Data Point 2	0.891	0.887	-0.004	0.988	0.984	-0.004
Data Point 3	0.892	0.895	0.003	0.987	0.984	-0.002
Data Point 4	0.891	0.891	0.000	0.987	0.985	-0.002
Data Point 5	0.891	0.891	0.000	0.985	0.984	-0.001
Data Point 6	0.891	0.891	0.000	0.985	0.984	-0.001
Data Point 7	0.887	0.887	0.000	0.992	0.991	0.000
Data Point 8	0.894	0.894	0.000	0.991	0.990	0.000
Data Point 9	0.891	0.891	0.000	0.991	0.990	0.000
Data Point 10	0.891	0.891	-0.001	0.991	0.990	0.000
Data Point 11	0.894	0.893	-0.001	0.993	0.993	0.000

Table A1.3: Maximum current speed – spring tide (flood and ebb)

	Spring tide maximum current speed for flood and ebb tides (m/s)							
	Spring flood existing	Spring flood proposed	Difference in flood tide		Spring ebb existing	Spring ebb proposed	Difference in ebb tide	
Data Point 1	0.368	0.1	-0.268	-72.8%	0.351	0.292	-0.059	-16.8%
Data Point 2	0.478	0.565	0.087	18.2%	0.462	0.548	0.086	18.6%
Data Point 3	0.49	0.442	-0.049	-10.0%	0.43	0.361	-0.069	-16.0%
Data Point 4	0.482	0.501	0.02	4.1%	0.483	0.52	0.037	7.7%
Data Point 5	0.497	0.504	0.007	1.4%	0.464	0.48	0.016	3.4%
Data Point 6	0.44	0.447	0.007	1.6%	0.405	0.417	0.012	3.0%
Data Point 7	0.166	0.171	0.005	3.0%	0.198	0.207	0.008	4.0%
Data Point 8	0.126	0.134	0.008	6.3%	0.05	0.055	0.005	10.0%
Data Point 9	0.325	0.346	0.021	6.5%	0.344	0.358	0.014	4.1%
Data Point 10	0.406	0.43	0.024	5.9%	0.395	0.41	0.015	3.8%
Data Point 11	0.382	0.424	0.043	11.3%	0.344	0.343	0	0.0%

Table A1.4: Water depth at maximum current speed - spring tide (flood and ebb)

	Spring Tide Total water depth at maximum speed for flood and ebb tides (m)					
	Spring flood existing	Spring flood proposed	Difference in flood tide	Spring ebb existing	Spring ebb proposed	Difference in ebb tide
Data Point 1	5.608	5.583	-0.024	5.707	5.686	-0.020
Data Point 2	6.755	6.759	0.004	6.852	6.856	0.004
Data Point 3	7.255	7.253	-0.002	7.350	7.342	-0.008
Data Point 4	6.958	6.957	-0.001	7.054	7.051	-0.003
Data Point 5	7.379	7.387	0.008	7.473	7.480	0.007
Data Point 6	6.388	6.396	0.008	6.482	6.490	0.008
Data Point 7	3.104	3.104	0.000	3.208	3.208	0.000
Data Point 8	3.143	3.092	-0.052	3.240	3.189	-0.052
Data Point 9	4.758	4.758	0.000	4.858	4.857	0.000
Data Point 10	7.205	7.205	0.000	7.305	7.305	0.000
Data Point 11	8.265	8.272	0.008	8.364	8.373	0.009

Table A1.5: Surface elevation at maximum current speed – neap tide (flood and ebb)

	Neap Tide Surface Elevation at maximum current speed for flood and ebb tides (mRL)					
	Neap flood existing	Neap flood proposed	Difference in flood tide	Neap ebb existing	Neap ebb proposed	Difference in ebb tide
Data Point 1	0.667	0.666	0.000	0.347	0.348	0.001
Data Point 2	0.666	0.665	-0.001	0.346	0.345	-0.001
Data Point 3	0.667	0.667	0.001	0.346	0.345	-0.001
Data Point 4	0.666	0.666	0.000	0.346	0.345	-0.001
Data Point 5	0.666	0.666	0.000	0.346	0.345	0.000
Data Point 6	0.666	0.666	0.000	0.346	0.345	0.000
Data Point 7	0.665	0.665	0.000	0.348	0.348	0.000
Data Point 8	0.667	0.667	0.000	0.348	0.348	0.000
Data Point 9	0.666	0.666	0.000	0.348	0.348	0.000
Data Point 10	0.666	0.666	0.000	0.347	0.347	0.000
Data Point 11	0.667	0.667	0.000	0.348	0.348	0.000

Table A1.6: Maximum current speed – neap tide (flood and ebb)

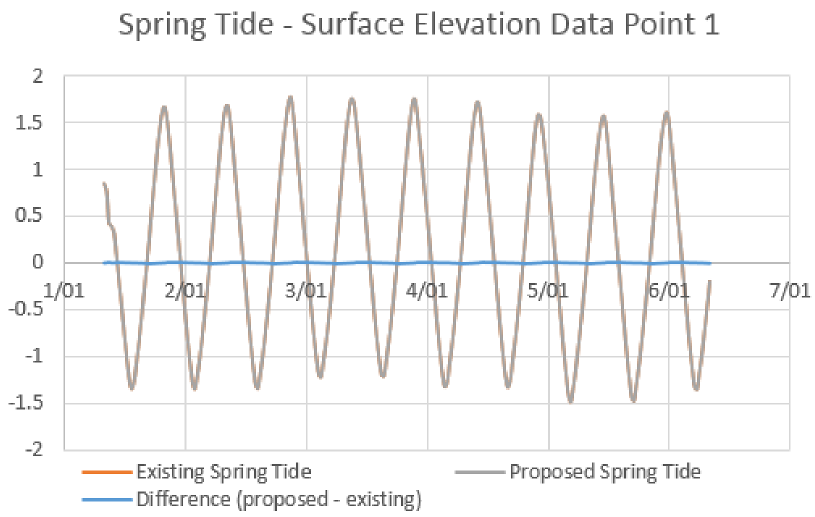
	Neap tide maximum current speed for flood and ebb tides (m/s)

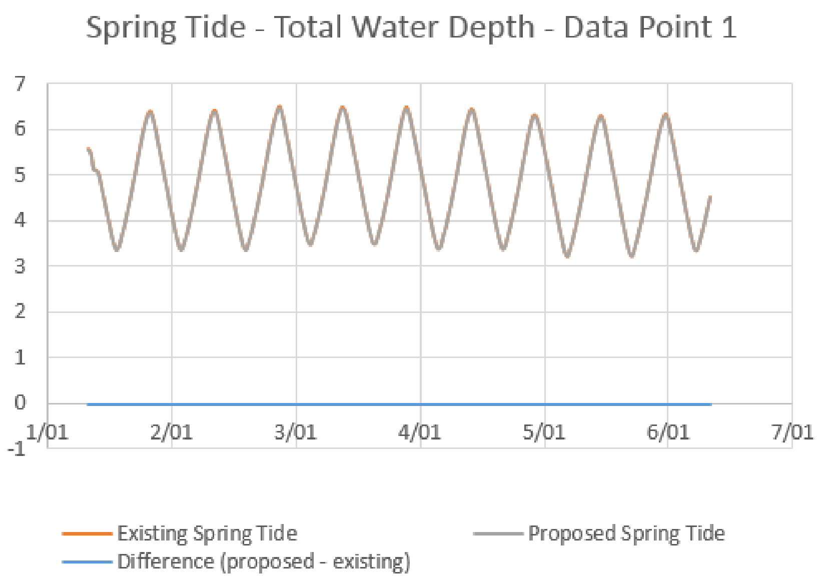
	Neap flood existing	Neap flood proposed	Difference in flood tide		Neap ebb existing	Neap ebb proposed	Difference in ebb tide	
Data Point 1	0.175	0.039	-0.136	-77.7%	0.171	0.131	-0.04	-23.4%
Data Point 2	0.234	0.272	0.038	16.2%	0.245	0.285	0.041	16.7%
Data Point 3	0.239	0.214	-0.025	-10.5%	0.224	0.167	-0.057	-25.4%
Data Point 4	0.237	0.247	0.01	4.2%	0.262	0.285	0.024	9.2%
Data Point 5	0.244	0.247	0.003	1.2%	0.251	0.26	0.01	4.0%
Data Point 6	0.214	0.217	0.003	1.4%	0.218	0.224	0.007	3.2%
Data Point 7	0.077	0.079	0.002	2.6%	0.104	0.108	0.004	3.8%
Data Point 8	0.057	0.061	0.004	7.0%	0.026	0.029	0.002	7.7%
Data Point 9	0.159	0.17	0.011	6.9%	0.186	0.194	0.008	4.3%
Data Point 10	0.201	0.214	0.013	6.5%	0.214	0.222	0.008	3.7%
Data Point 11	0.187	0.21	0.022	11.8%	0.185	0.182	-0.003	-1.6%

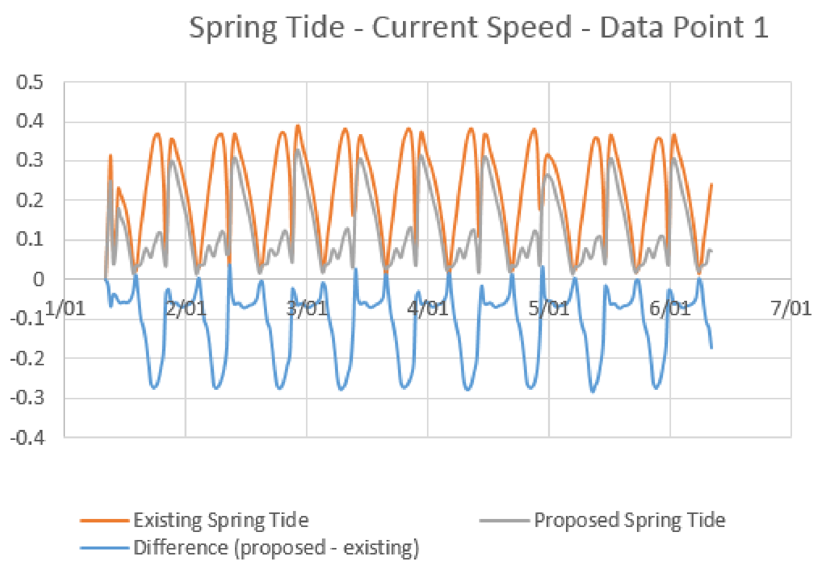
Table A1.7: Water depth at maximum current speed - neap tide (flood and ebb)

	Neap tide total water depth at maximum speed for flood and ebb tides (m)					
	Neap flood existing	Neap flood proposed	Difference in flood tide	Neap ebb existing	Neap ebb proposed	Difference in ebb tide
Data Point 1	5.382	5.359	-0.023	5.063	5.041	-0.022
Data Point 2	6.530	6.537	0.007	6.210	6.218	0.007
Data Point 3	7.030	7.025	-0.005	6.709	6.703	-0.007
Data Point 4	6.733	6.732	-0.001	6.413	6.411	-0.002
Data Point 5	7.154	7.162	0.008	6.834	6.841	0.008
Data Point 6	6.163	6.171	0.008	5.843	5.851	0.008
Data Point 7	2.881	2.881	0.000	2.564	2.564	0.000
Data Point 8	2.916	2.865	-0.052	2.597	2.546	-0.052
Data Point 9	4.533	4.533	0.000	4.214	4.214	0.000
Data Point 10	6.980	6.981	0.000	6.661	6.662	0.001
Data Point 11	8.038	8.046	0.009	7.719	7.728	0.009

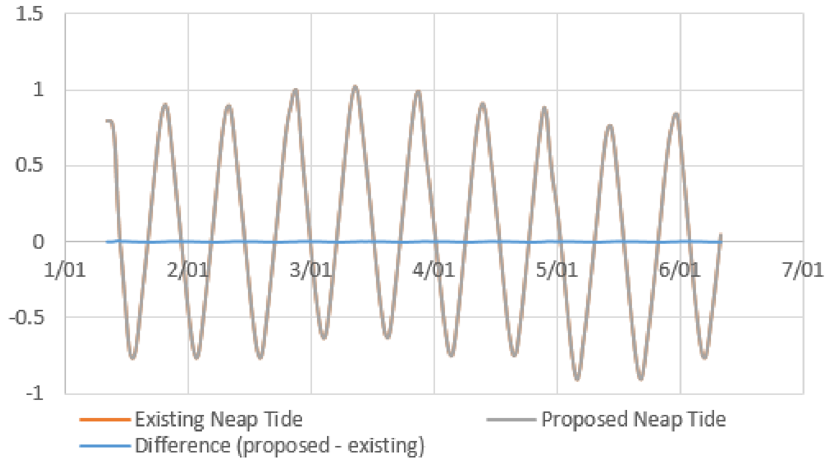
Appendix B: Timeseries for surface elevation, current speed and total water depth at data point locations


Spring Tide - Surface Elevation Data Point 1
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

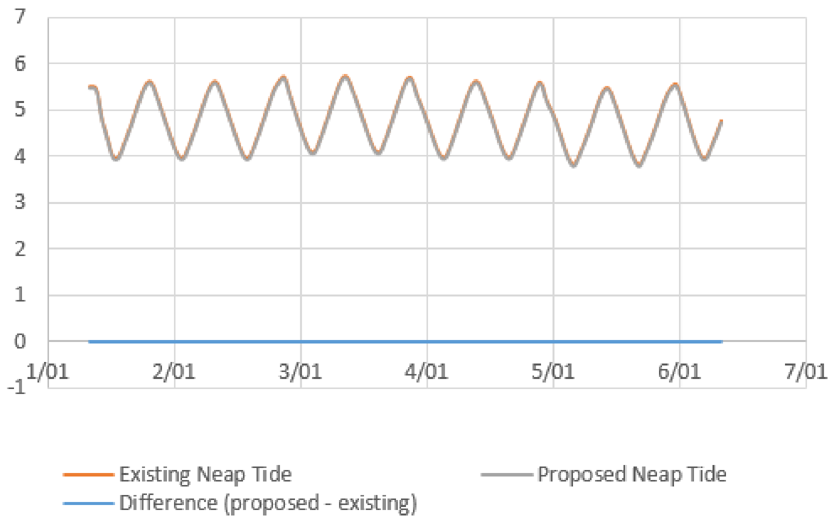

Spring Tide - Total Water Depth - Data Point 1
7
6
5
4
3
2
1
0
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)


Spring Tide - Current Speed - Data Point 1
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

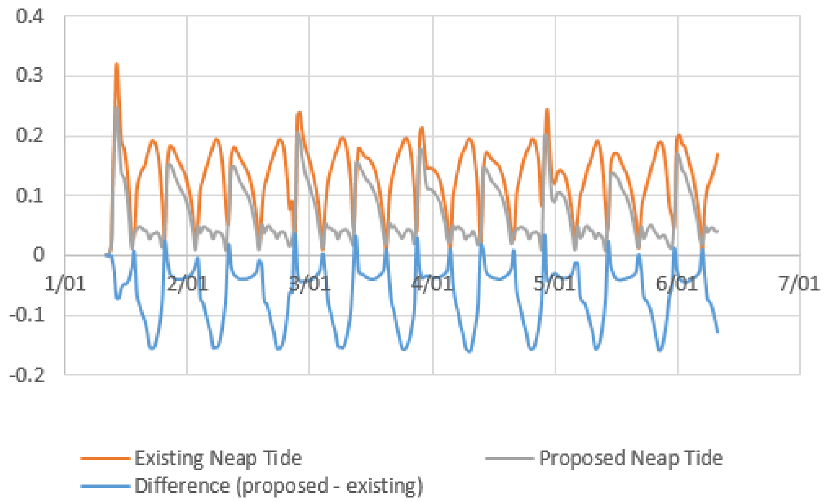
Neap Tide - Surface Elevation - Data Point 1

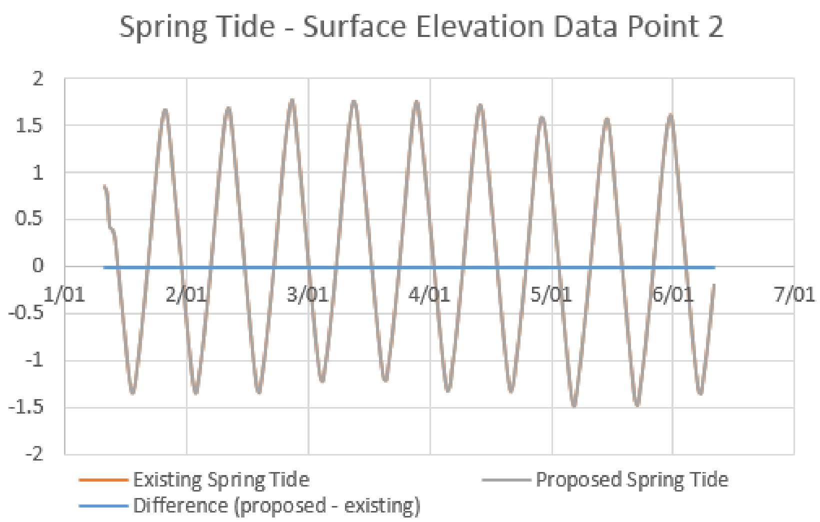


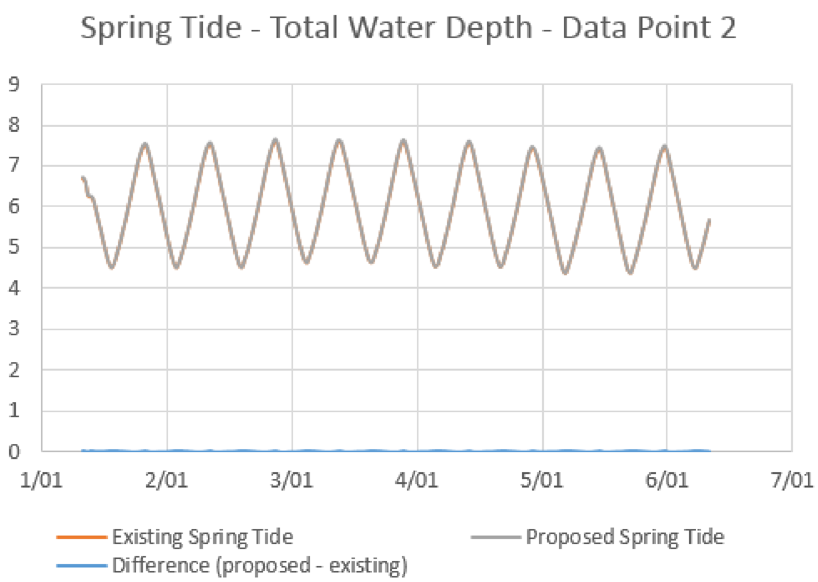
Neap Tide - Total Water Depth - Data Point 1

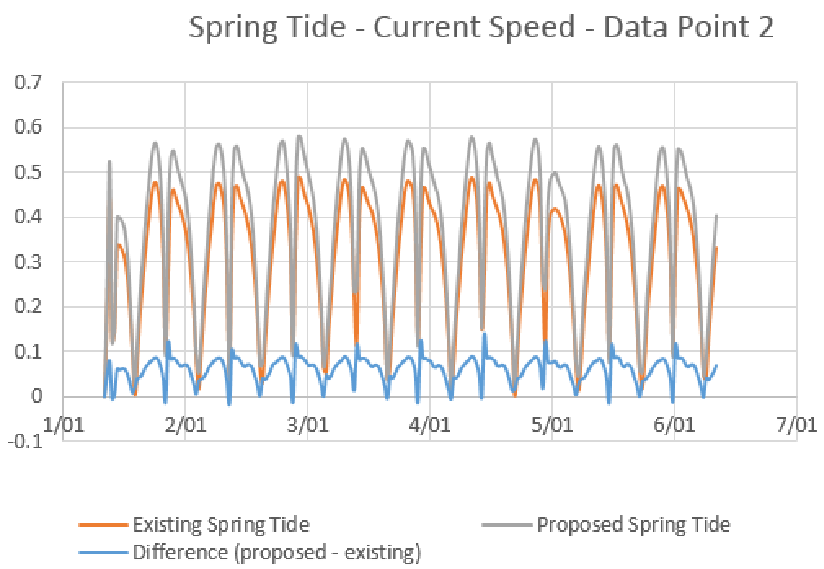


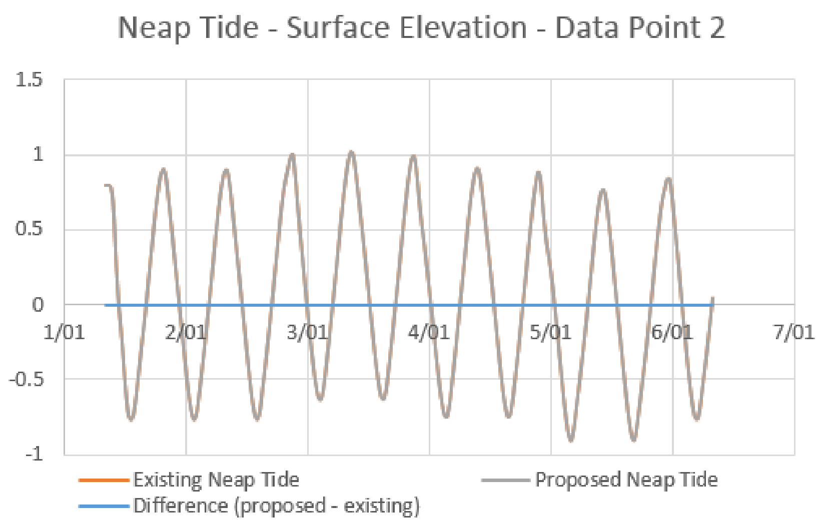
Neap Tide - Current Speed -Data Point 1



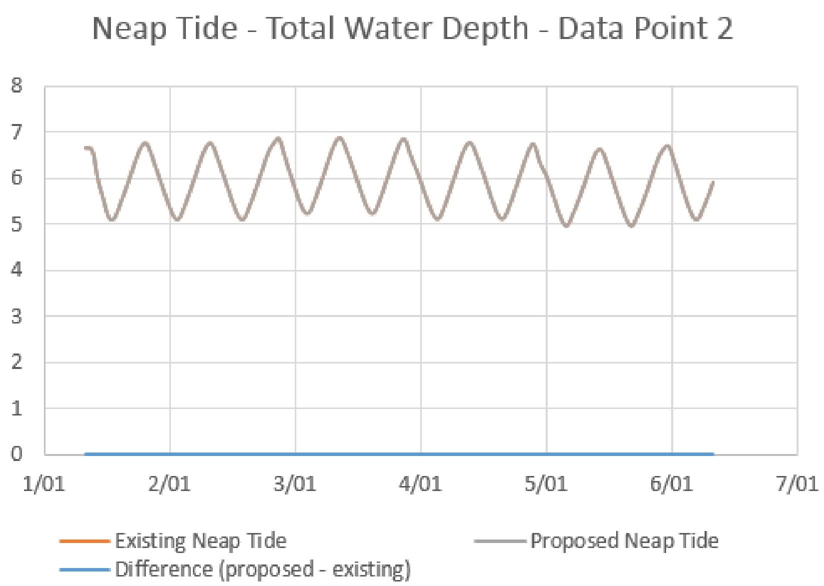

Spring Tide - Surface Elevation Data Point 2
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)


Spring Tide - Total Water Depth - Data Point 2
9
8
7
6
5
4
3
2
1
0
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

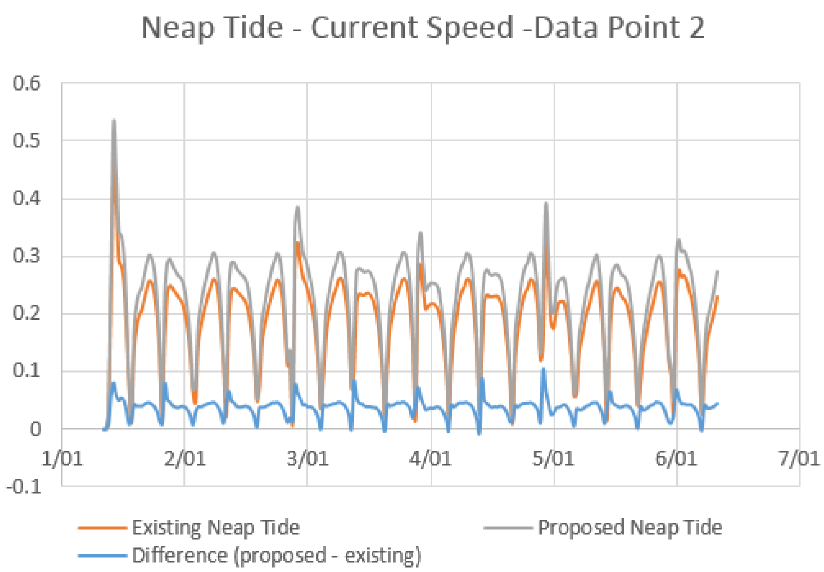

Spring Tide - Current Speed - Data Point 2
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)



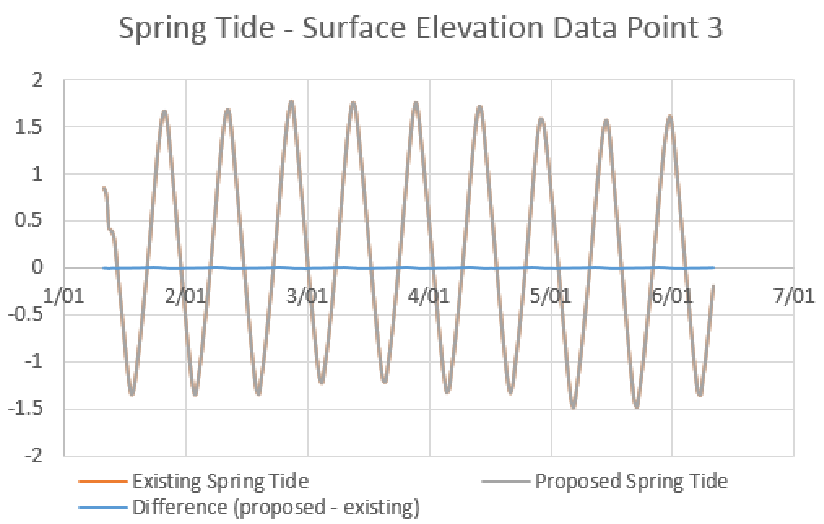
Neap Tide - Surface Elevation - Data Point 2
1.5
1
0.5
0
-0.5
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

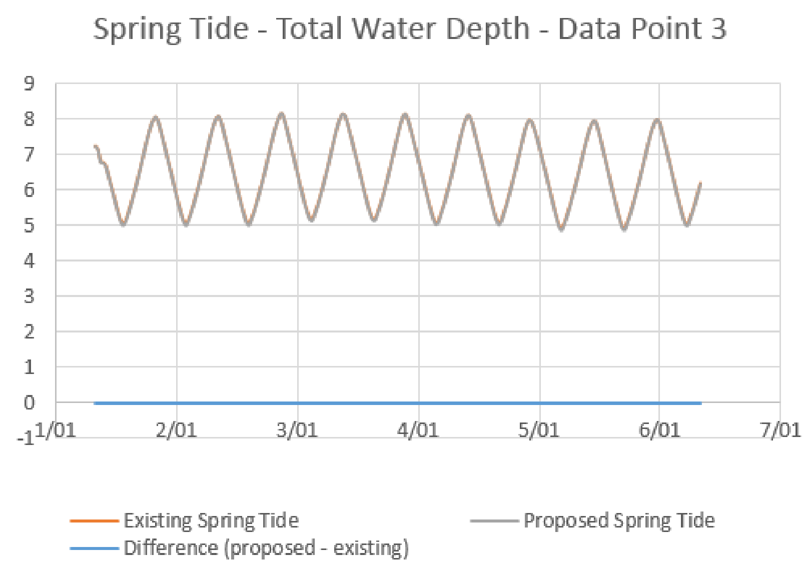


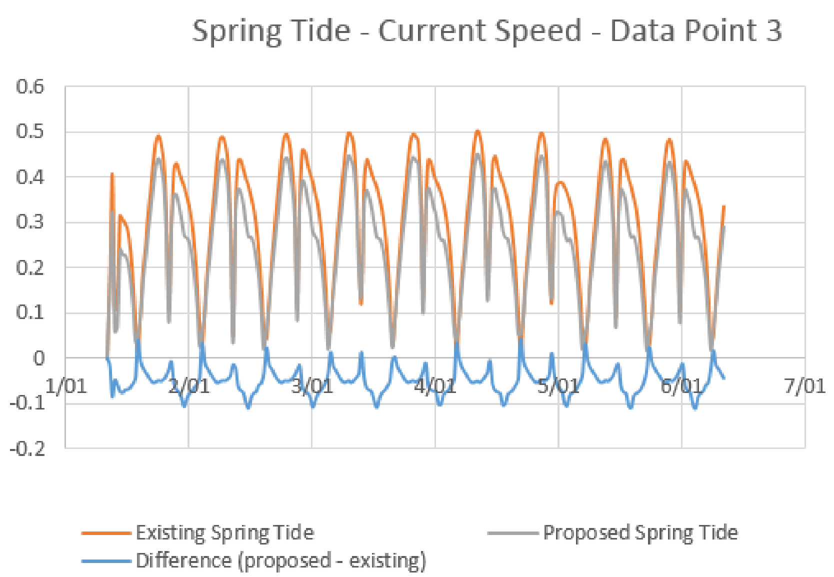
Neap Tide - Total Water Depth - Data Point 2
8
7
6
5
4
3
2
1
0
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

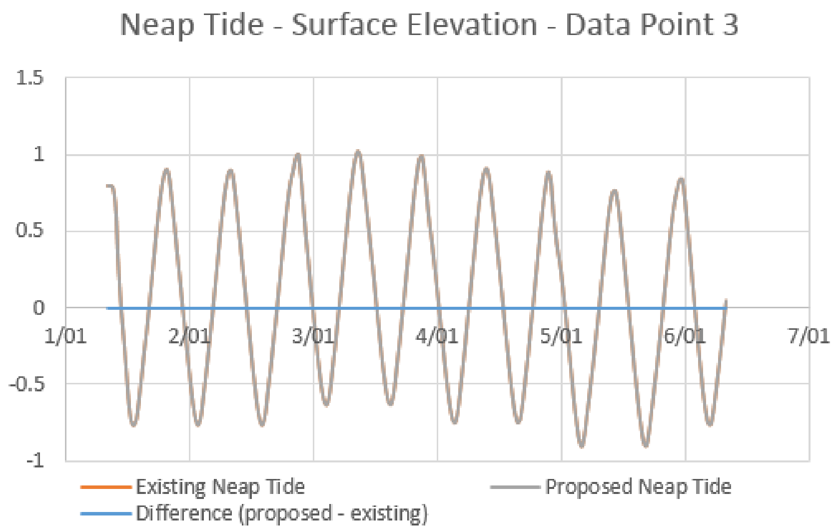


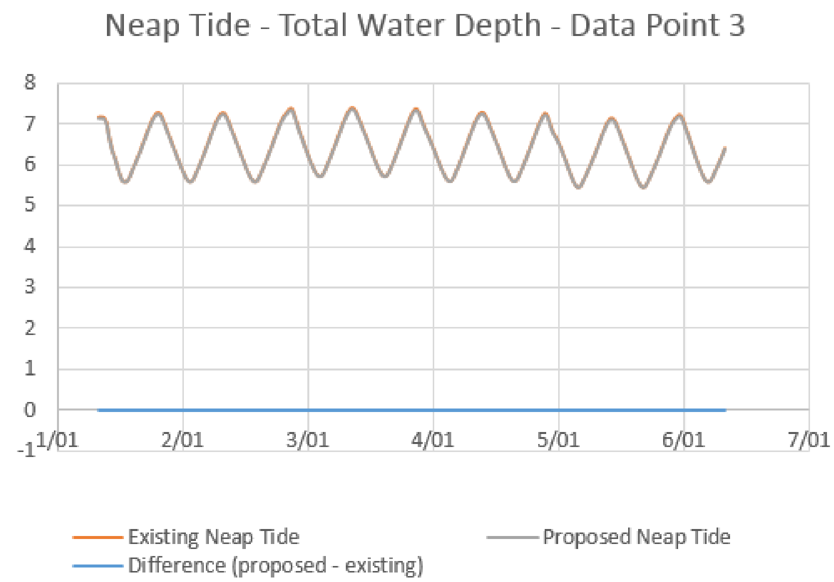
Neap Tide - Current Speed -Data Point 2
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

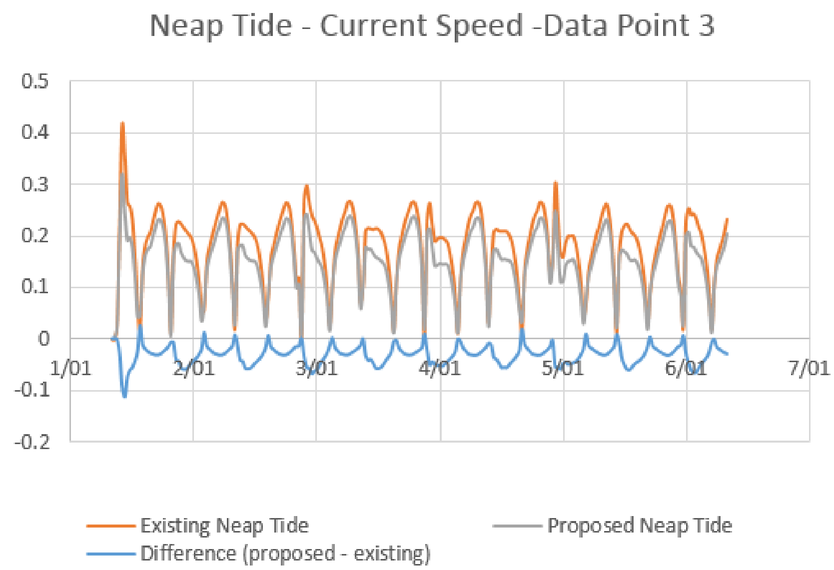

Spring Tide - Surface Elevation Data Point 3
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

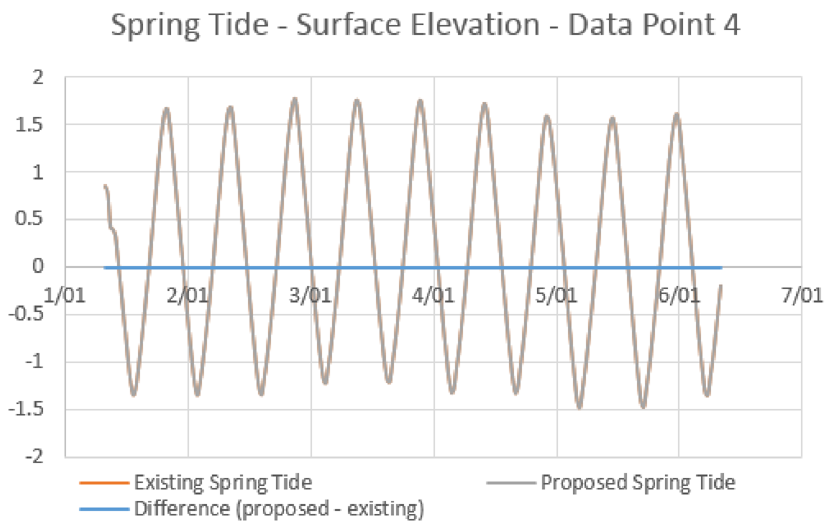

Spring Tide - Total Water Depth - Data Point 3
9
8
7
6
5
4
3
2
1
0
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

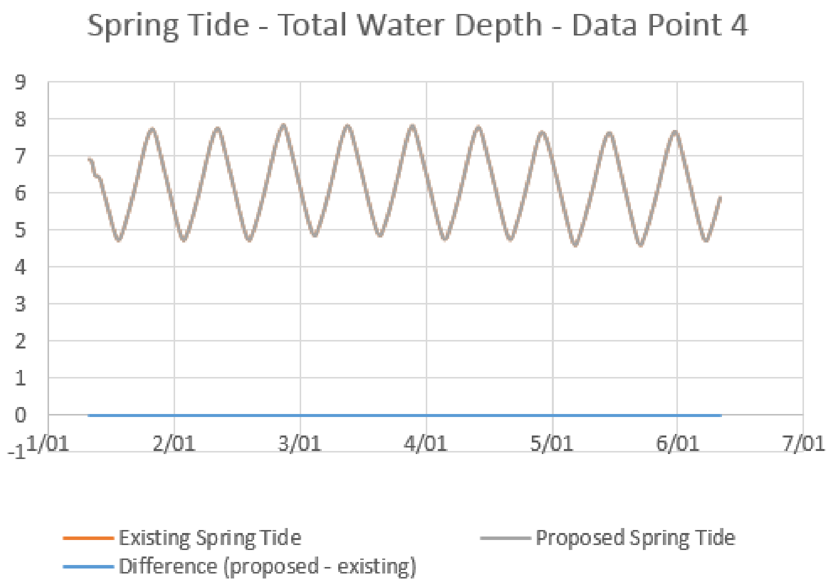

Spring Tide - Current Speed - Data Point 3
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

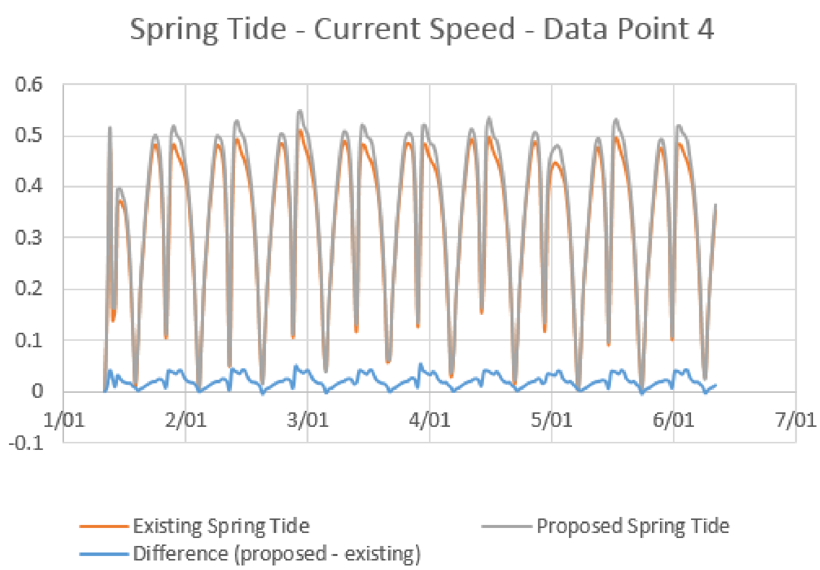

Neap Tide - Surface Elevation - Data Point 3
1.5
1
0.5
0
-0.5
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)


Neap Tide - Total Water Depth - Data Point 3
8
7
6
5
4
3
2
1
0
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

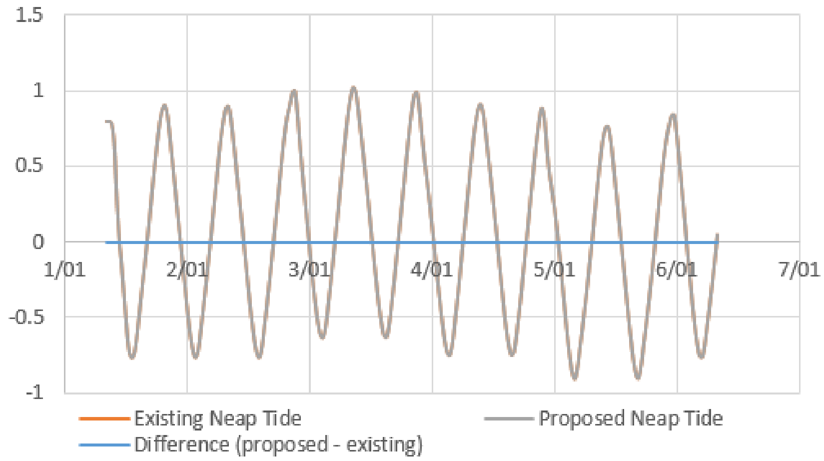

Neap Tide - Current Speed -Data Point 3
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)


Spring Tide - Surface Elevation - Data Point 4
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

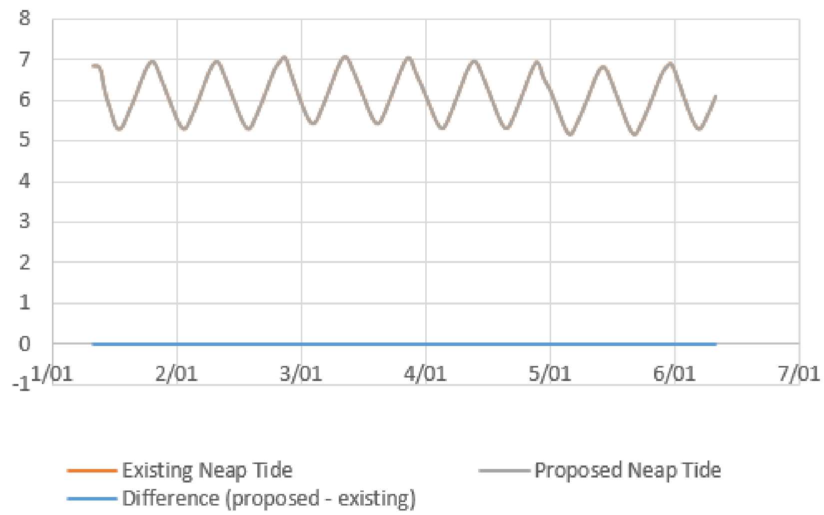

Spring Tide - Total Water Depth - Data Point 4
9
8
7
6
5
4
3
2
1
0
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)


Spring Tide - Current Speed - Data Point 4
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

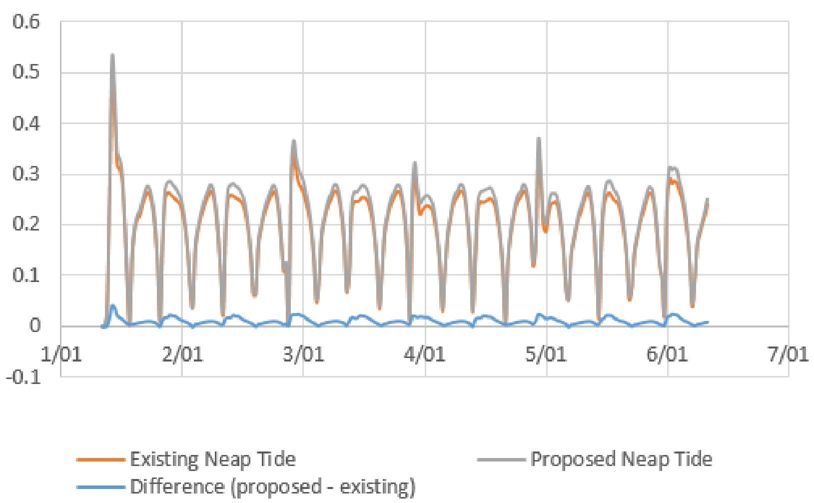
Neap Tide - Surface Elevation - Data Point 4

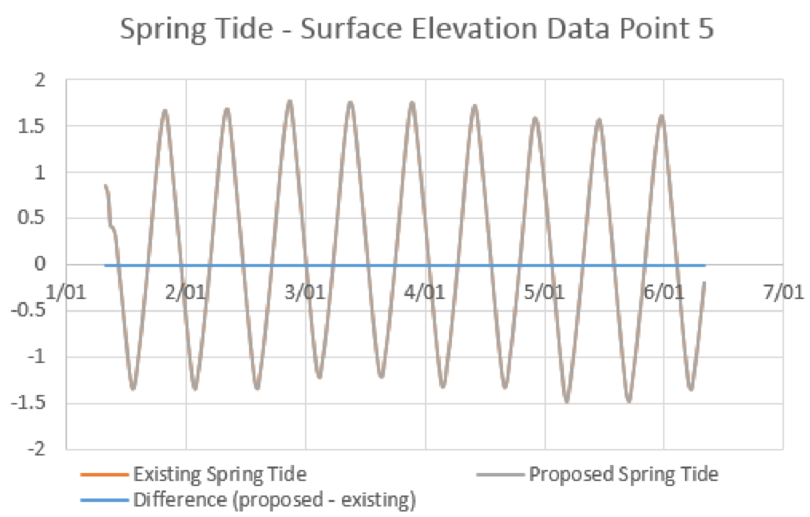


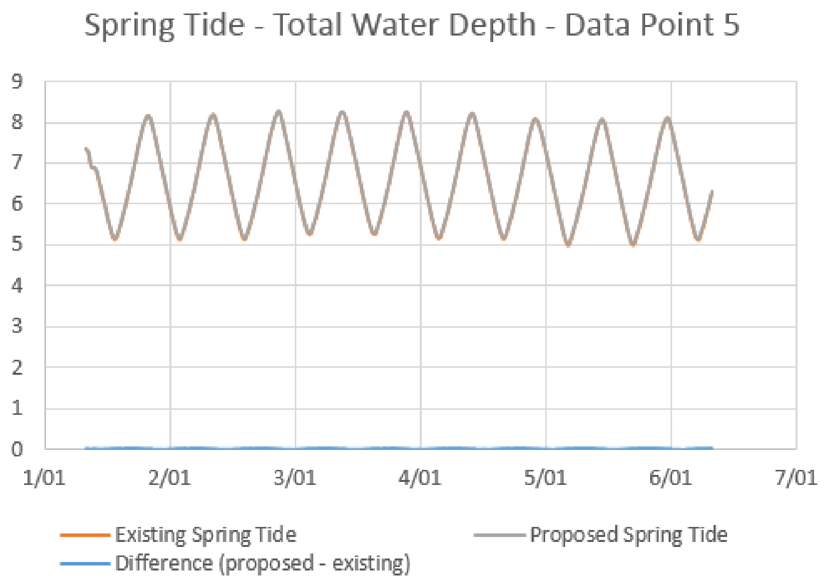
Neap Tide - Total Water Depth - Data Point 4

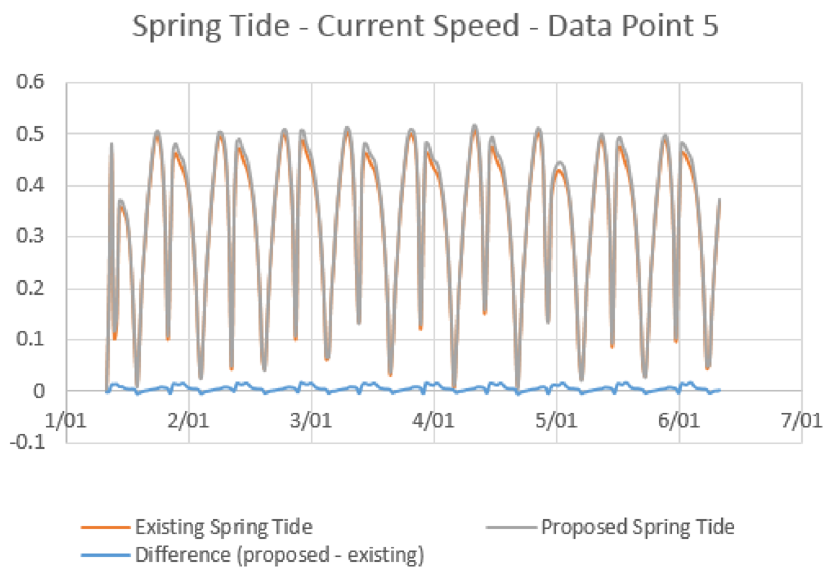


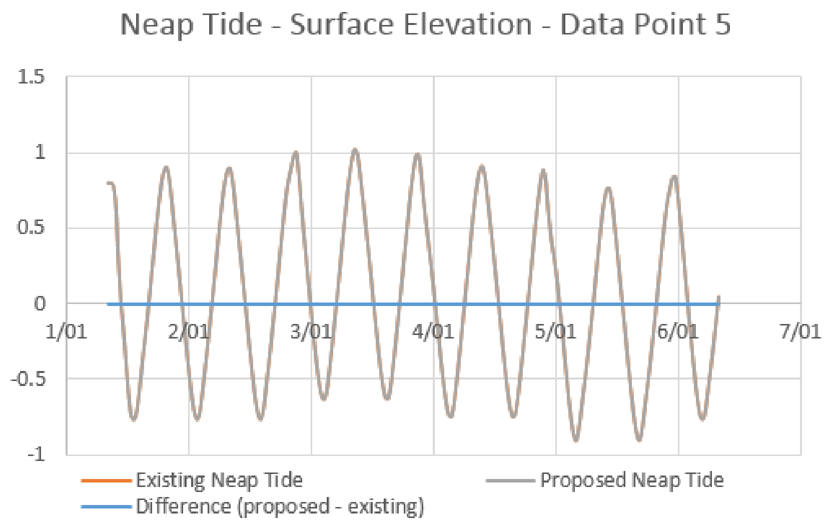
Neap Tide - Current Speed -Data Point 4

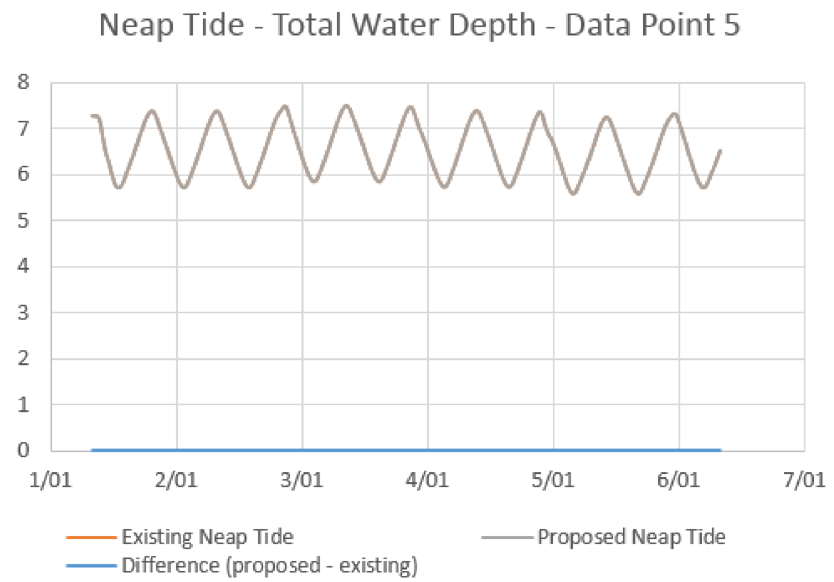


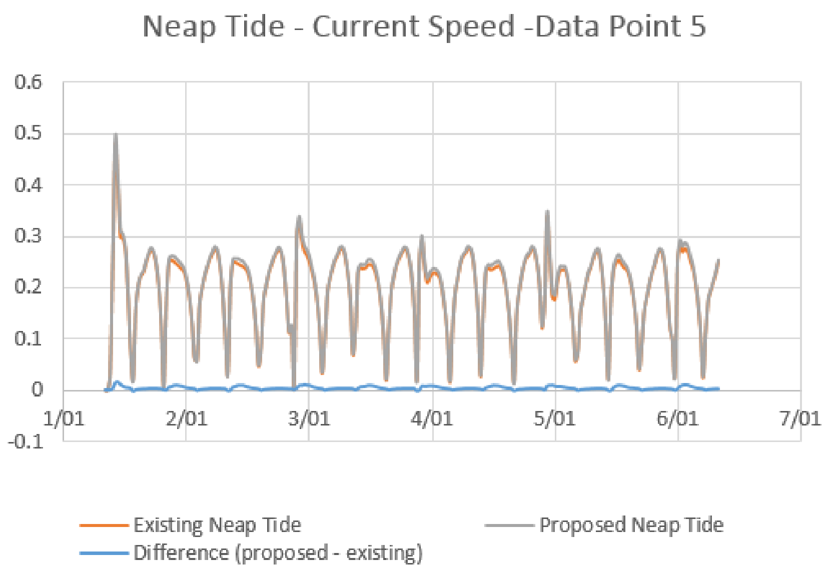

Spring Tide - Surface Elevation Data Point 5
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

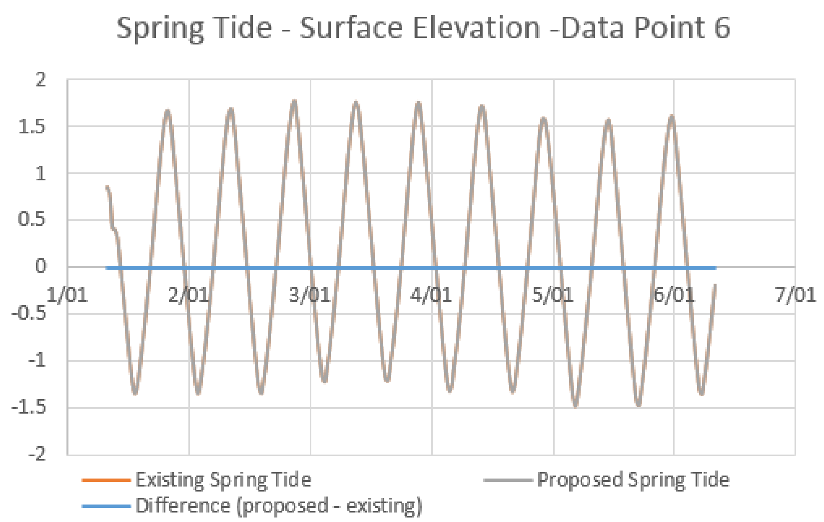

Spring Tide - Total Water Depth - Data Point 5
9
8
7
6
5
4
3
2
1
0
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)


Spring Tide - Current Speed - Data Point 5
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

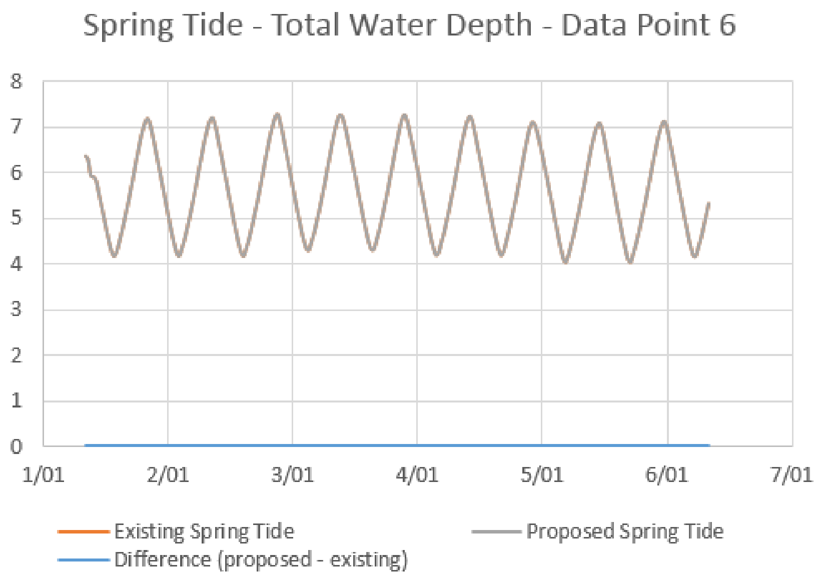

Neap Tide - Surface Elevation - Data Point 5
1.5
1
0.5
0
-0.5
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)


Neap Tide - Total Water Depth - Data Point 5
8
7
6
5
4
3
2
1
0
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

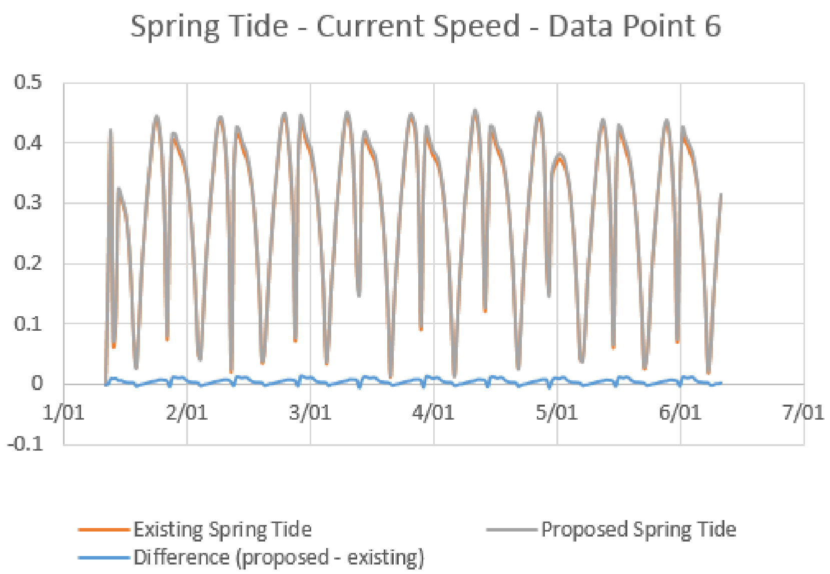

Neap Tide - Current Speed -Data Point 5
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)



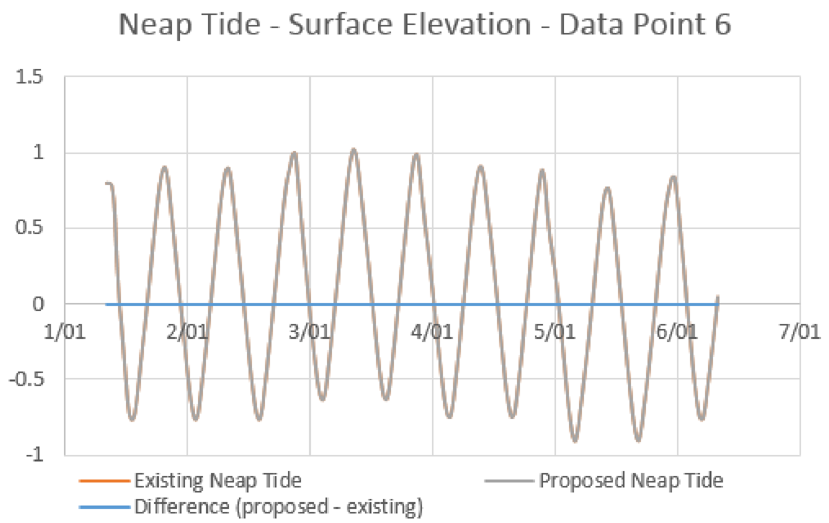
Spring Tide - Surface Elevation -Data Point 6
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

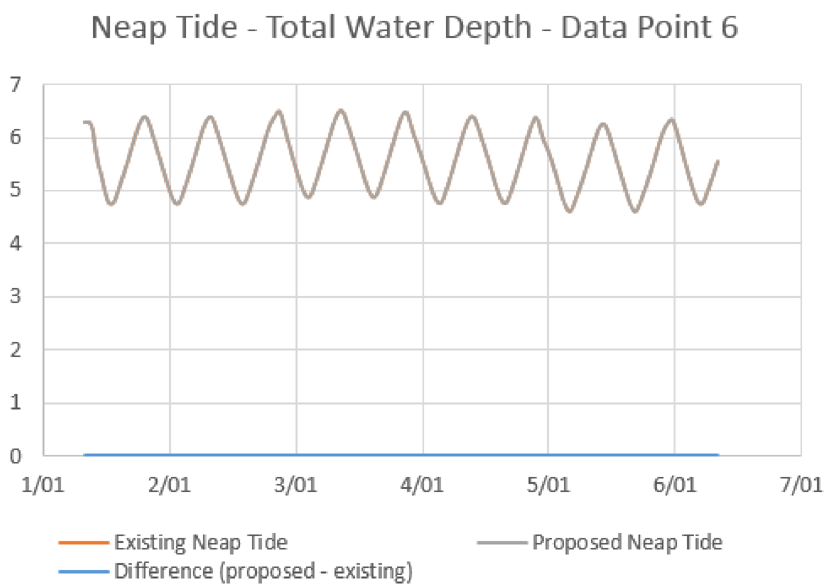


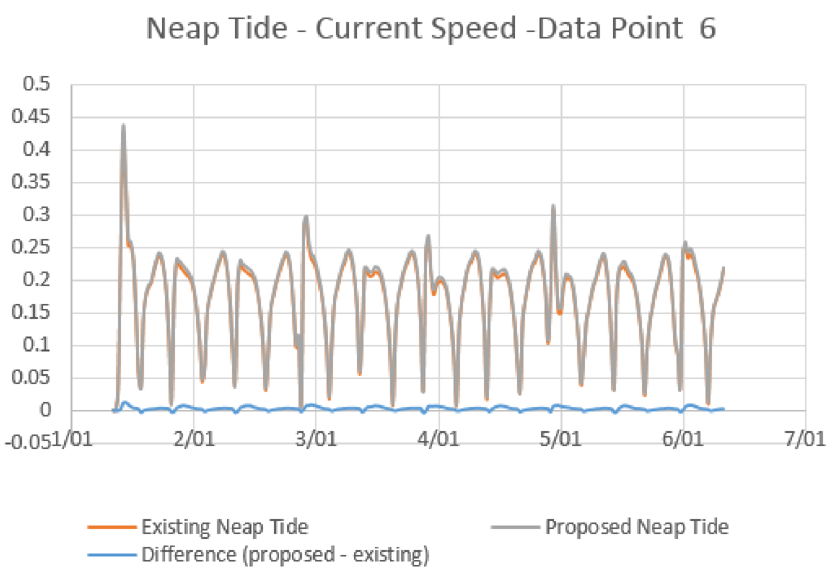
Spring Tide - Total Water Depth - Data Point 6
8
7
6
5
4
3
2
1
0
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)



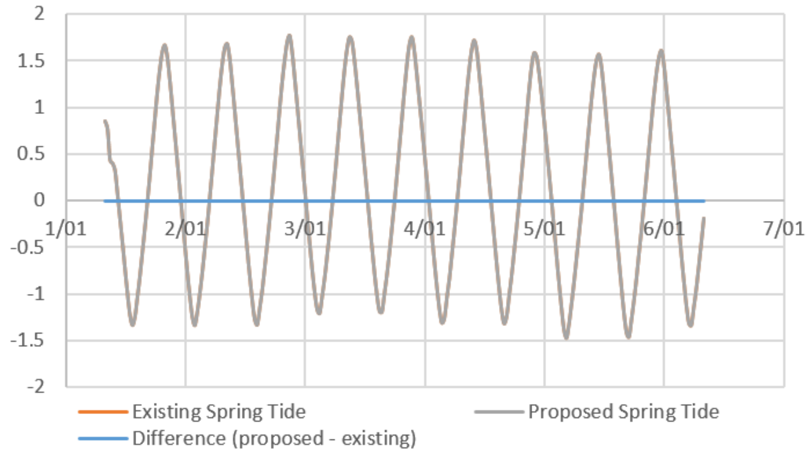
Spring Tide - Current Speed - Data Point 6
0.5
0.4
0.3
0.2
0.1
0
-0.1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)


Neap Tide - Surface Elevation - Data Point 6
1.5
1
0.5
0
-0.5
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

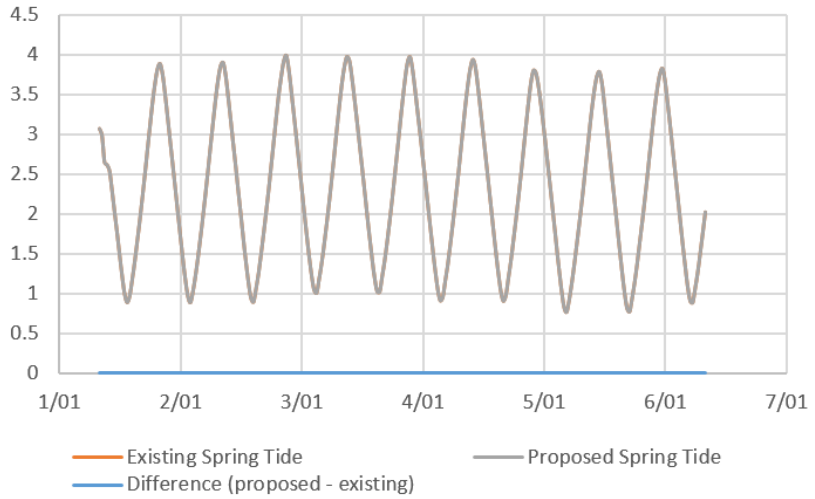

Neap Tide - Total Water Depth - Data Point 6
7
6
5
4
3
2
1
0
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)


Neap Tide - Current Speed -Data Point 6
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
-0.05
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

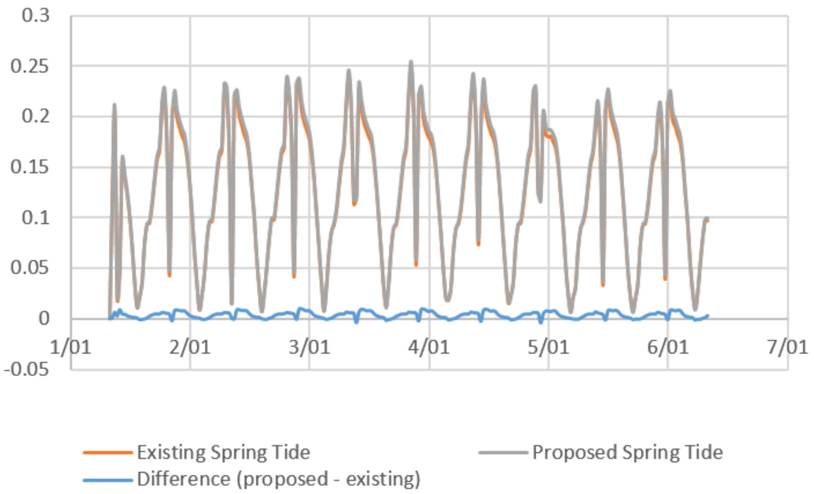
Spring Tide - Surface Elevation - Data Point 7

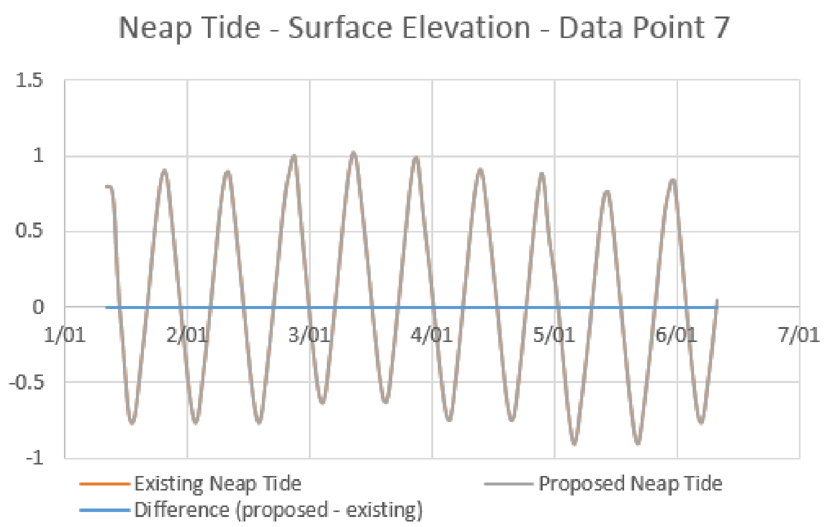


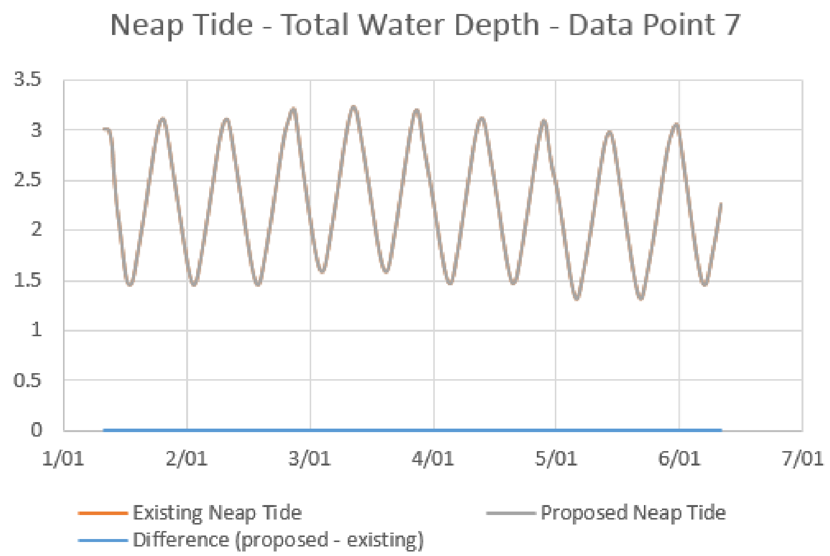
Spring Tide - Total Water Depth - Data Point 7

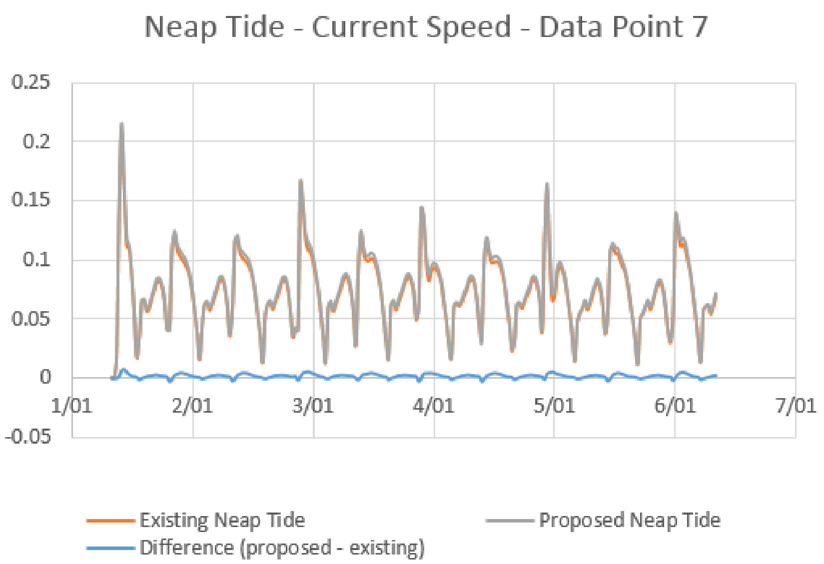


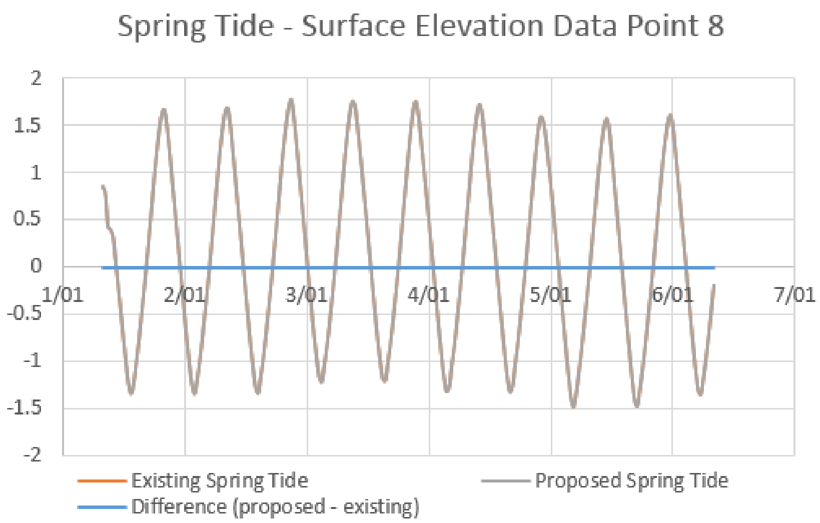
Spring Tide - Current Speed - Data Point 7

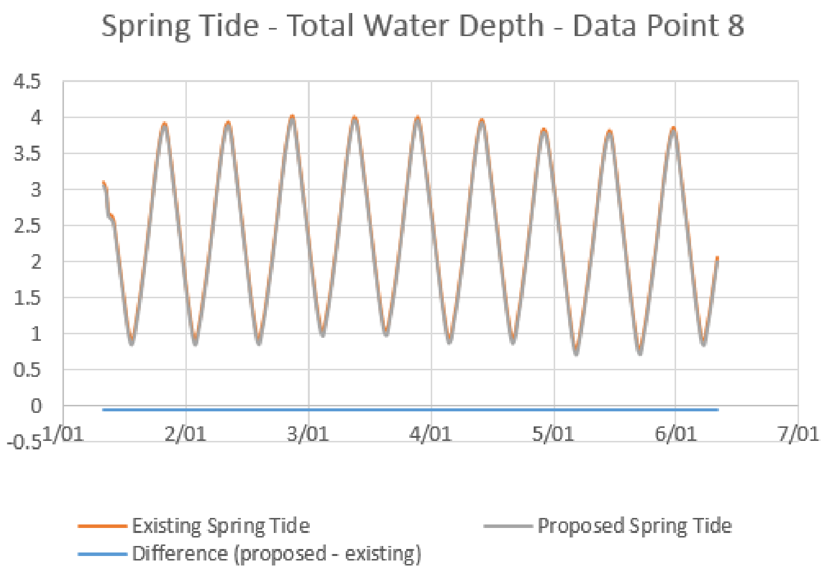


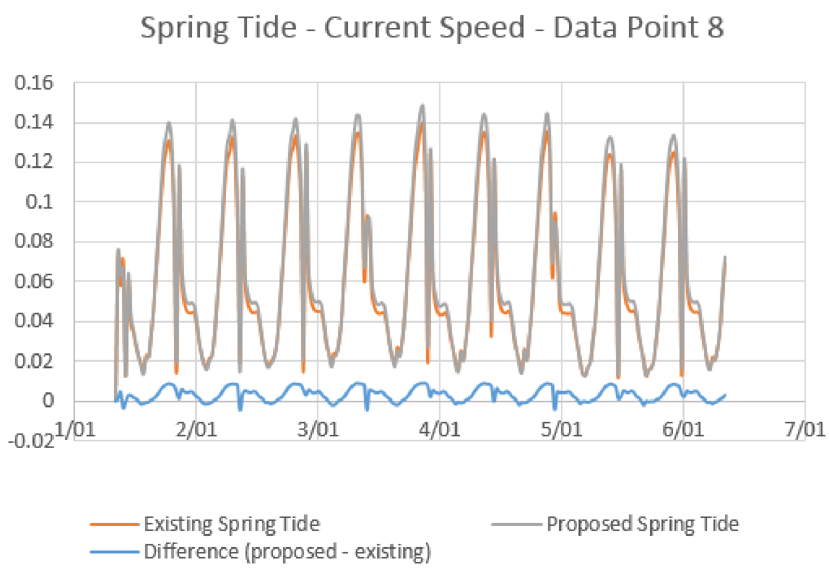

Neap Tide - Surface Elevation - Data Point 7
1.5
1
0.5
0
-0.5
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

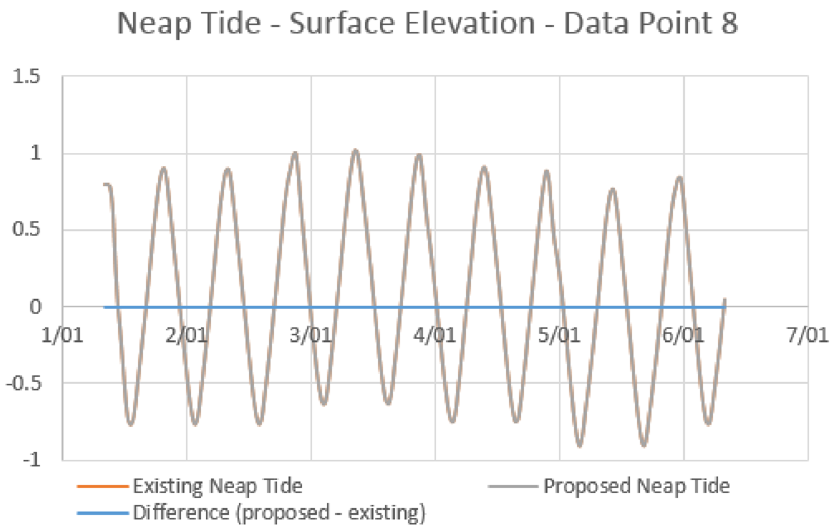

Neap Tide - Total Water Depth - Data Point 7
3.5
3
2.5
2
1.5
1
0.5
0
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

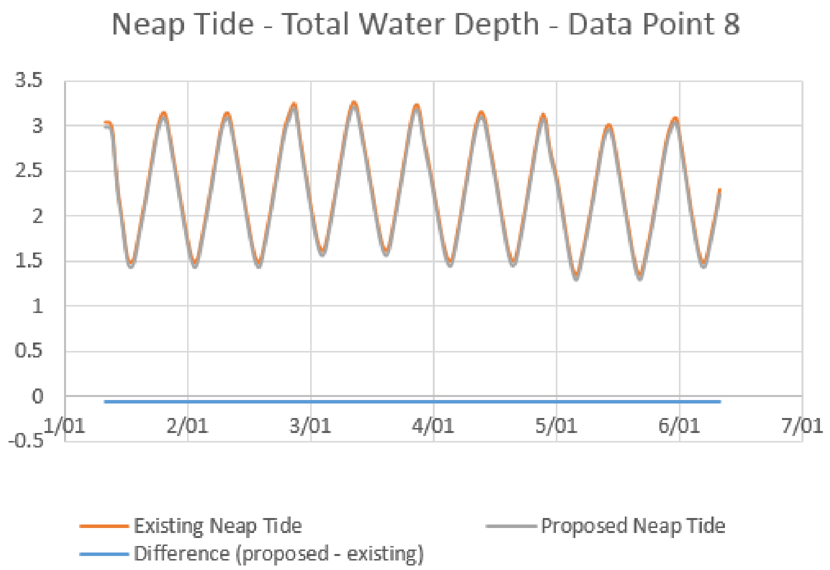

Neap Tide - Current Speed - Data Point 7
0.25
0.2
0.15
0.1
0.05
0
-0.05
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

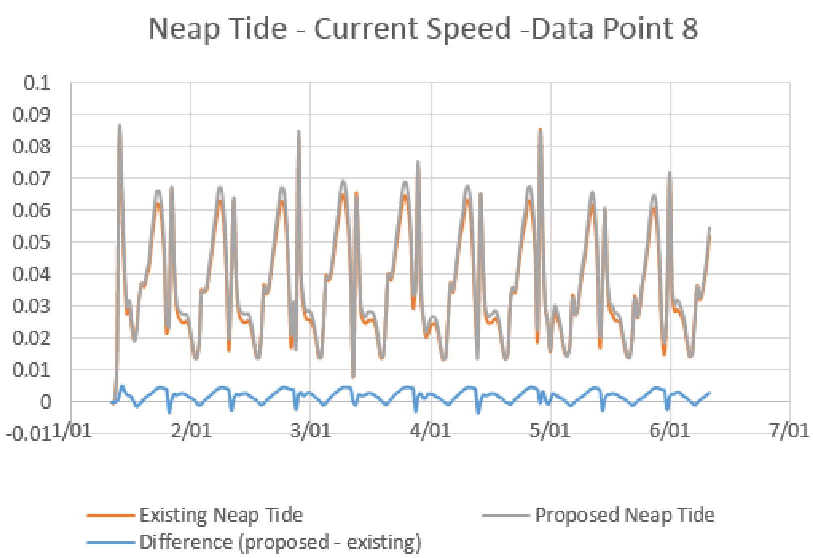

Spring Tide - Surface Elevation Data Point 8
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

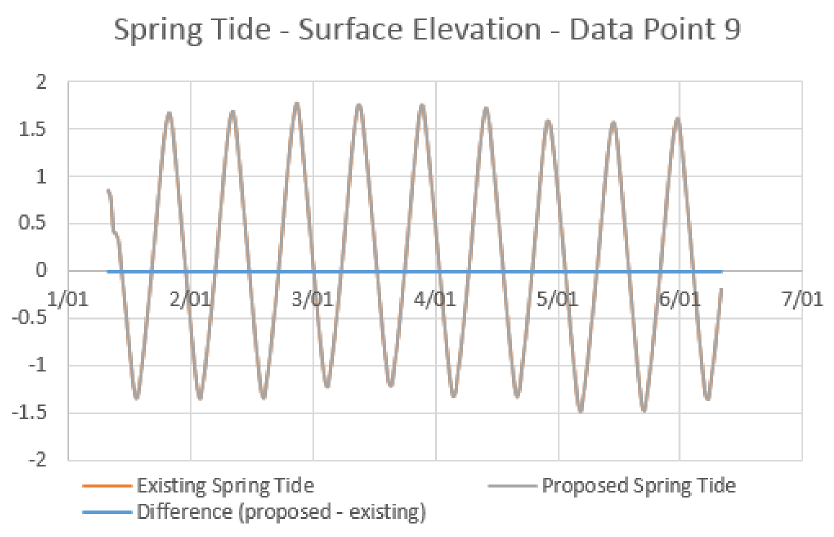

Spring Tide - Total Water Depth - Data Point 8
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
-0.5
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

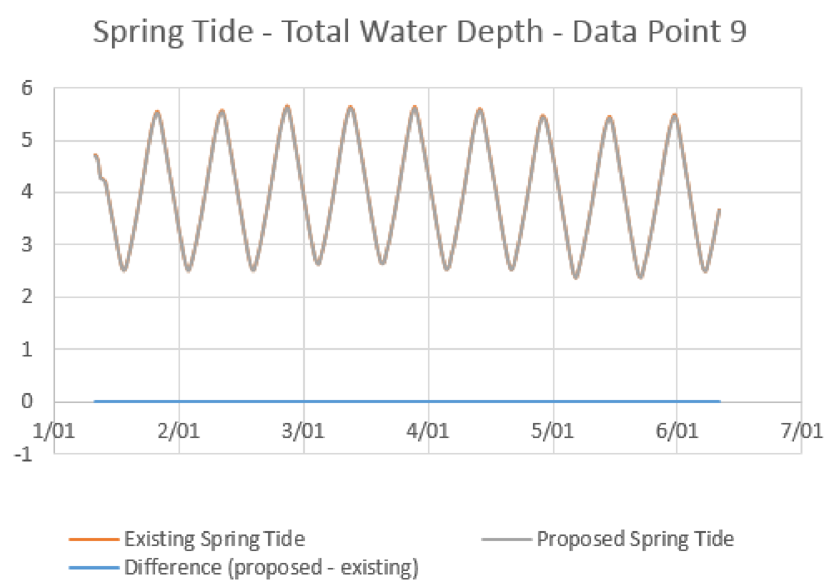

Spring Tide - Current Speed - Data Point 8
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
-0.02
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

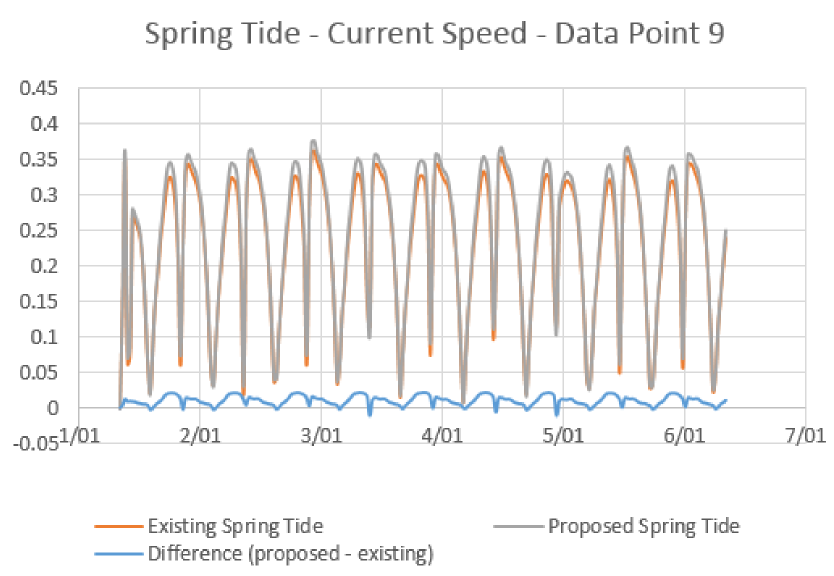

Neap Tide - Surface Elevation - Data Point 8
1.5
1
0.5
0
-0.5
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

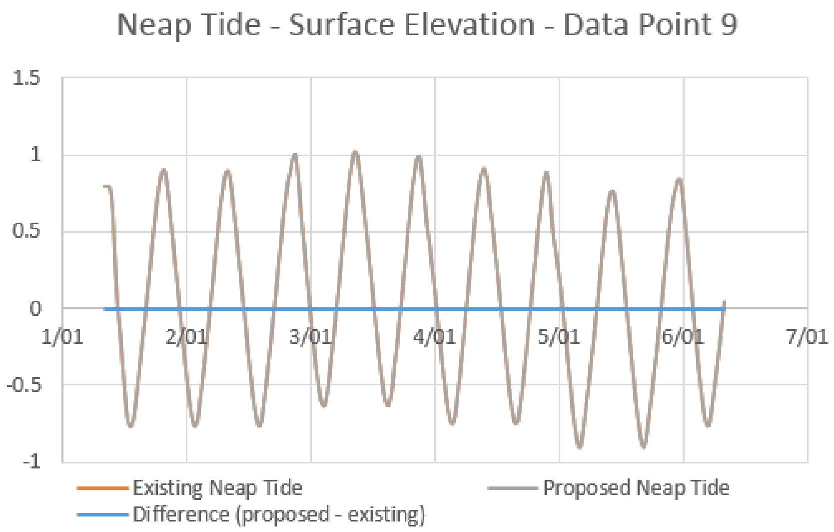

Neap Tide - Total Water Depth - Data Point 8
3.5
3
2.5
2
1.5
1
0.5
0
-0.5
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

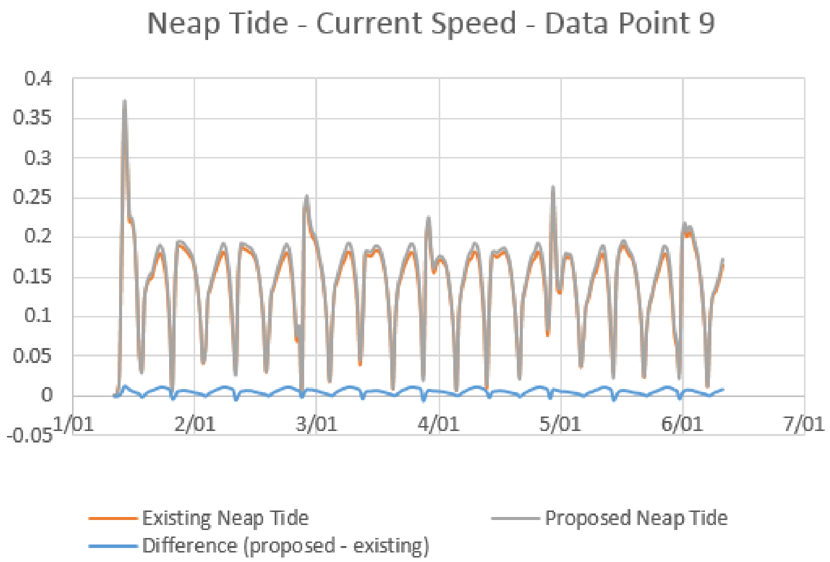

Neap Tide - Current Speed -Data Point 8
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
-0.01
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

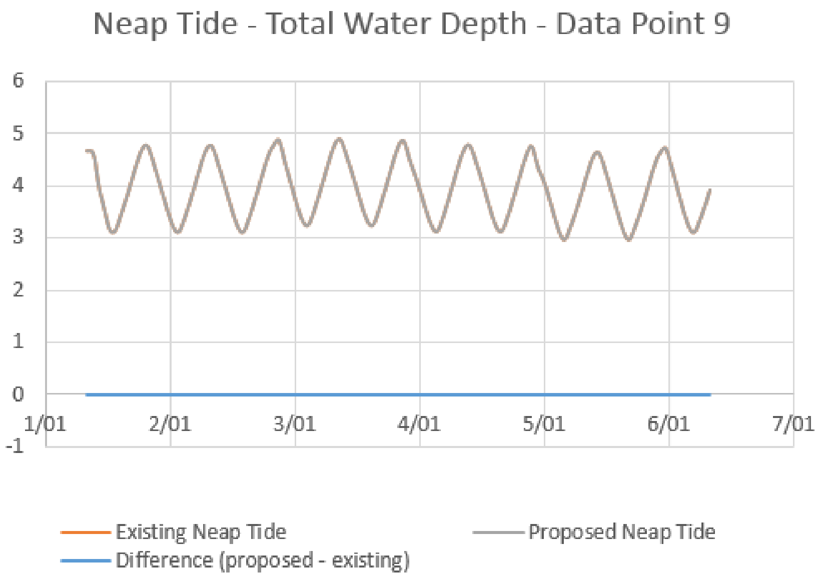

Spring Tide - Surface Elevation - Data Point 9
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)


Spring Tide - Total Water Depth - Data Point 9
6
5
4
3
2
1
0
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

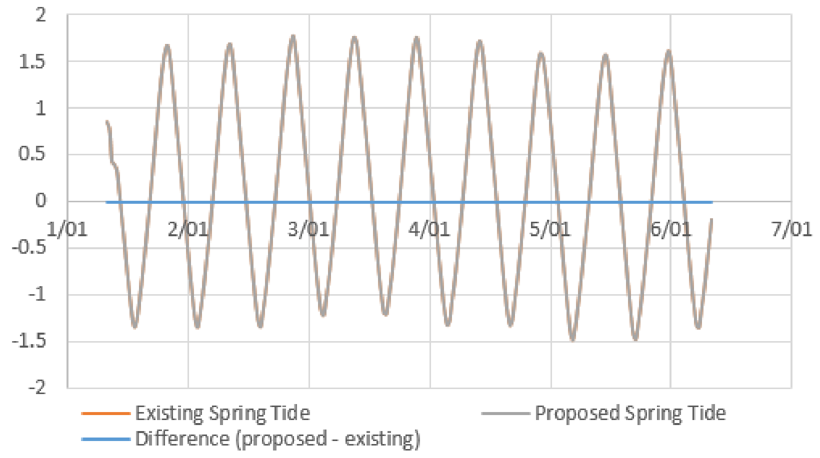

Spring Tide - Current Speed - Data Point 9
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
-0.05
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)


Neap Tide - Surface Elevation - Data Point 9
1.5
1
0.5
0
-0.5
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

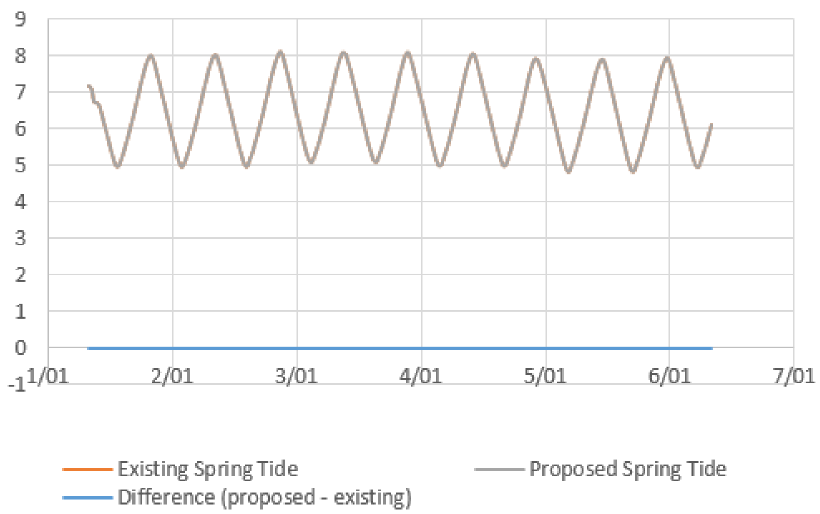

Neap Tide - Current Speed - Data Point 9
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
-0.05
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)


Neap Tide - Total Water Depth - Data Point 9
6
5
4
3
2
1
0
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

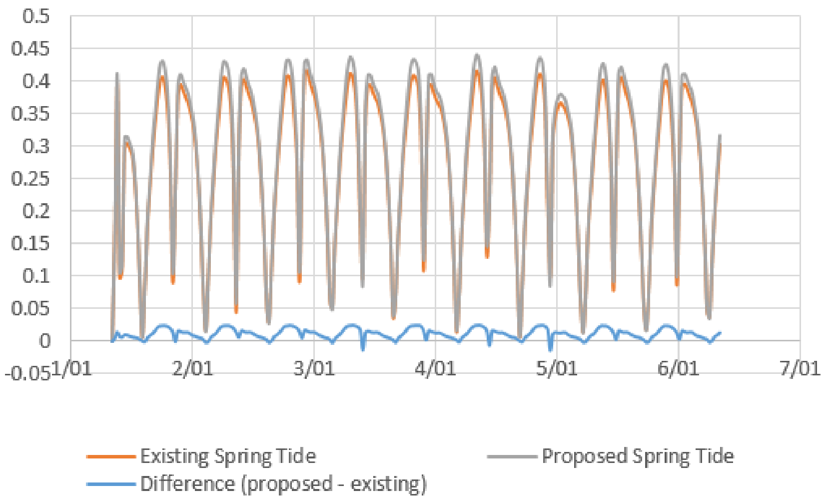
Spring Tide - Surface Elevation - Data Point 10



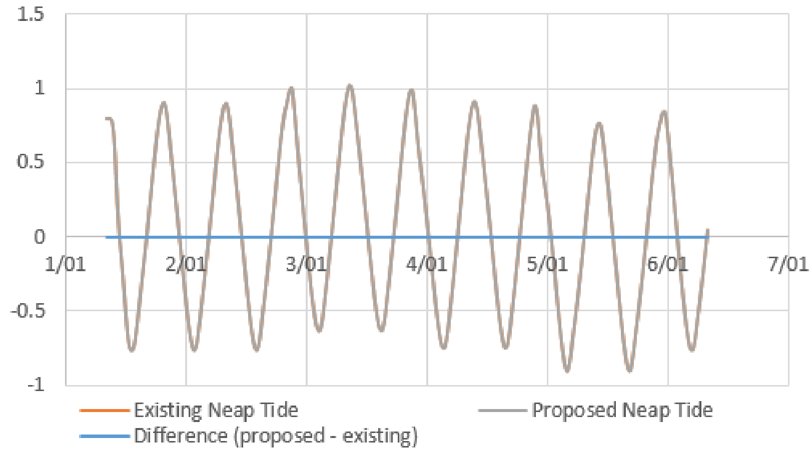
Spring Tide - Total Water Depth - Data Point 10



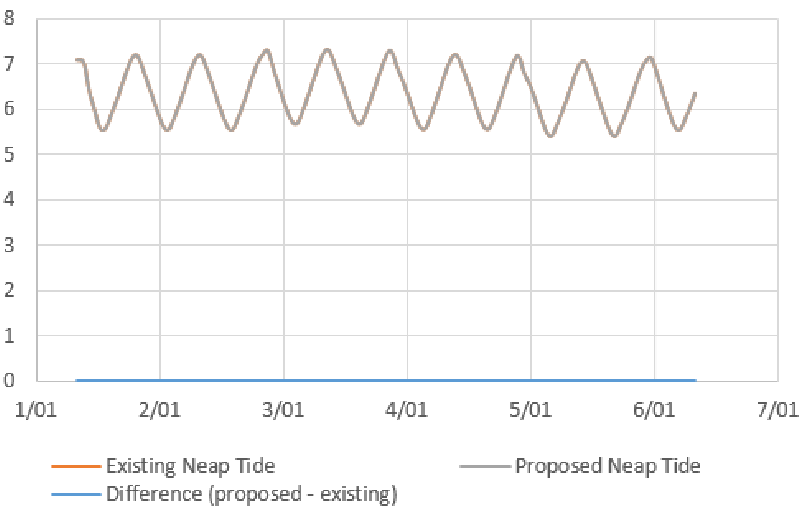
Spring Tide - Current Speed - Data Point 10

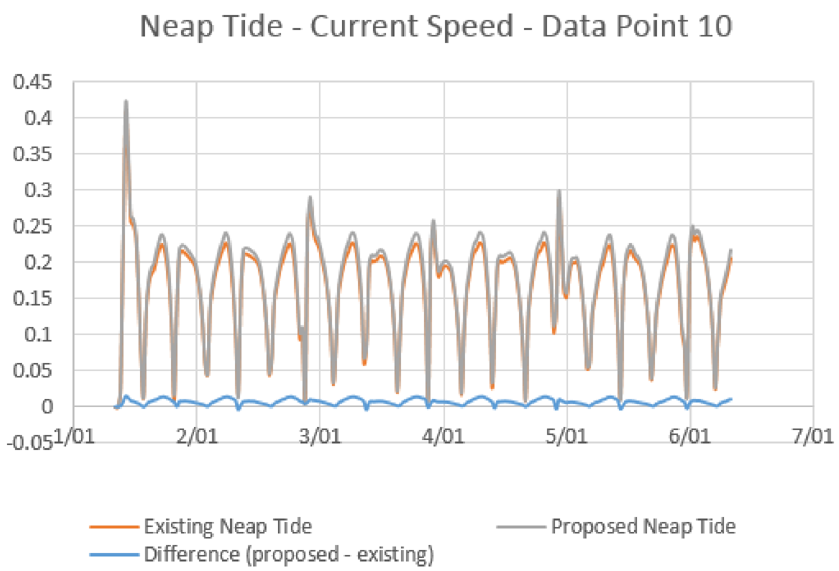


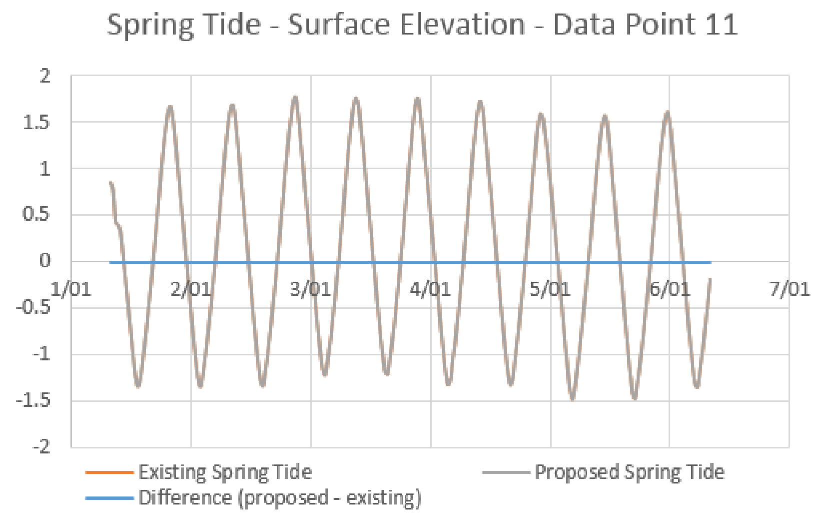
Neap Tide - Surface Elevation - Data Point 10

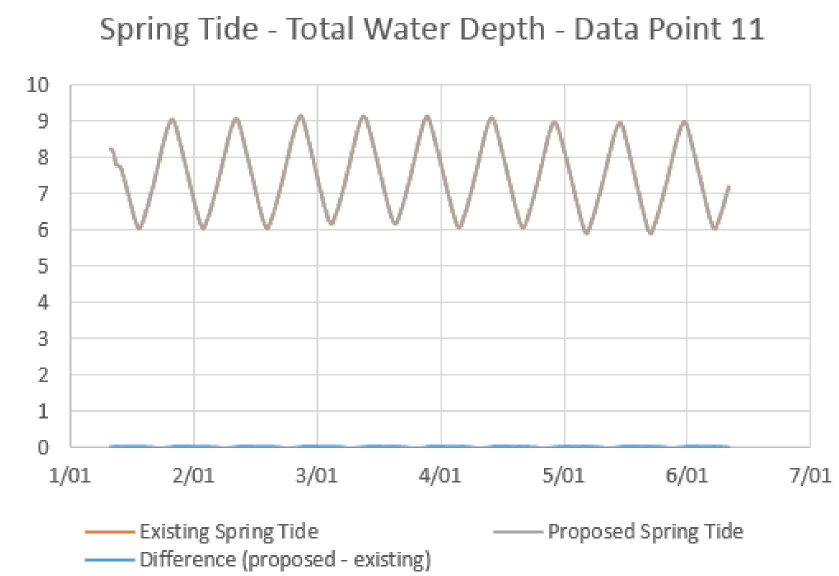


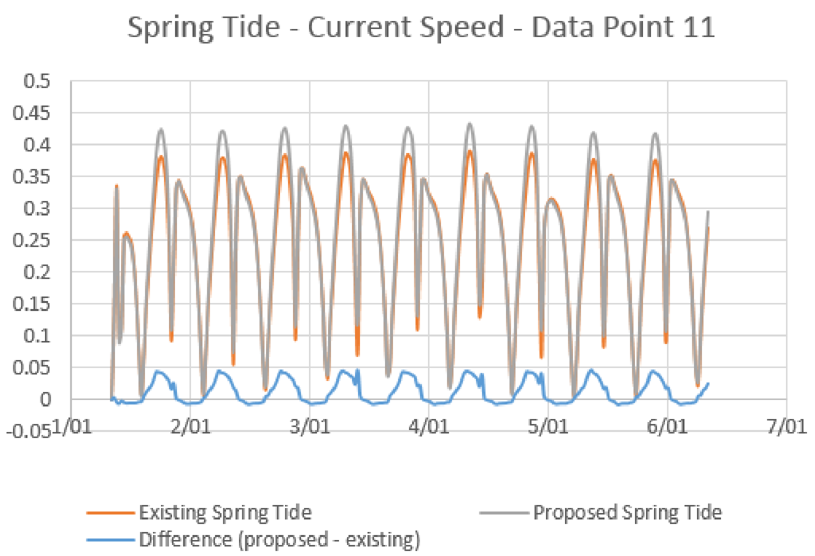
Neap Tide - Total Water Depth - Data Point 10

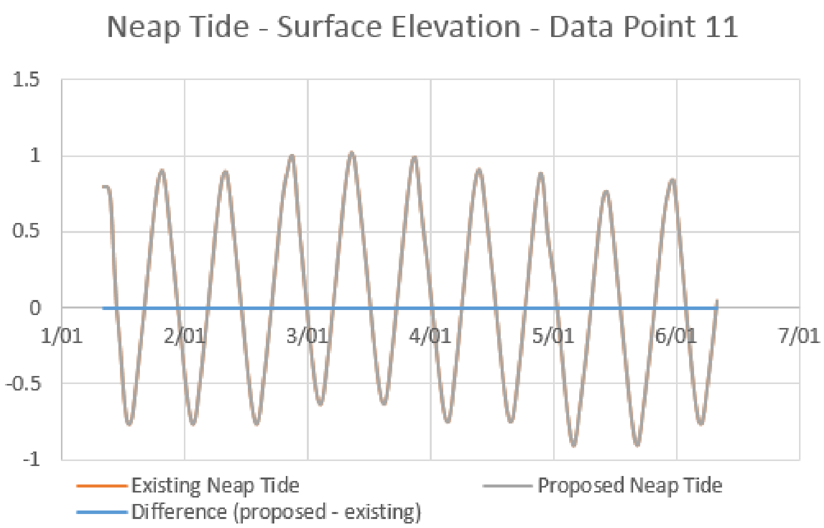


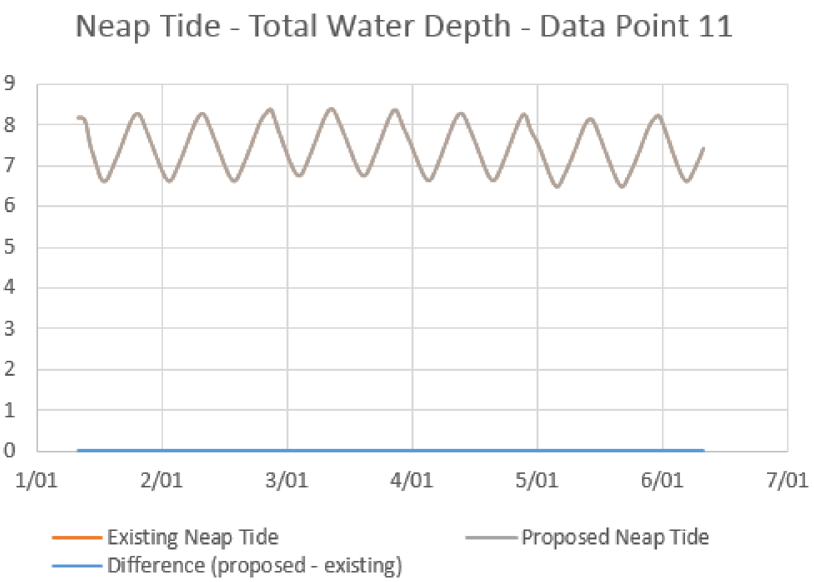

Neap Tide - Current Speed - Data Point 10
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
-0.05
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

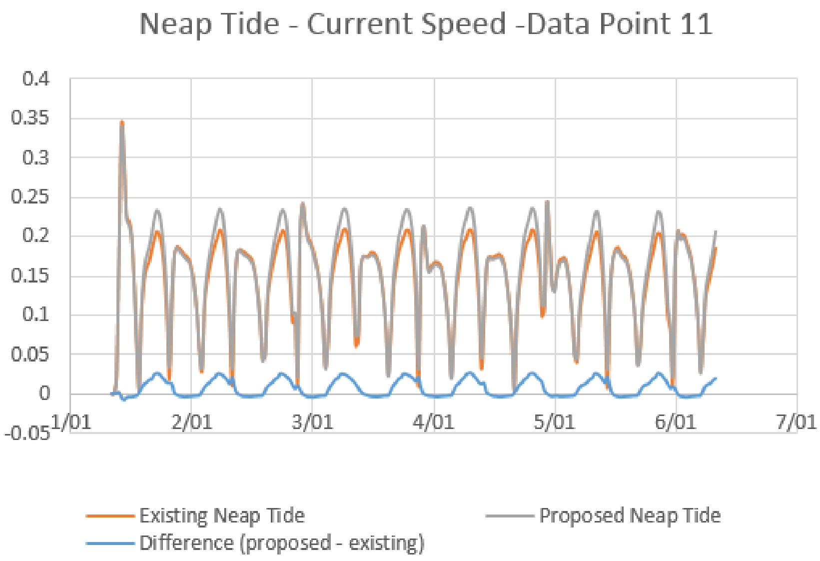

Spring Tide - Surface Elevation - Data Point 11
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)


Spring Tide - Total Water Depth - Data Point 11
10
9
8
7
6
5
4
3
2
1
0
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)


Spring Tide - Current Speed - Data Point 11
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
-0.05
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)


Neap Tide - Surface Elevation - Data Point 11
1.5
1
0.5
0
-0.5
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)


Neap Tide - Total Water Depth - Data Point 11
9
8
7
6
5
4
3
2
1
0
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)


Neap Tide - Current Speed -Data Point 11
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
-0.05
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

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