Geomorphological Baseline Assessment in the vicinity of the Whanganui Rivermouth

A report prepared as part of the Wanganui District Council's Lower Whanganui River Management Strategy

By Dr Roger Shand

COASTAL SYSTEMS LIMITED

Research, Education and Management Consultancy

70 Karaka Street.
Wanganui, New Zealand.

Phone: +64 634 44214 Mobile: +64 21 057 4189

rshand@coastalsystems.co.nz
www.coastalsystems.co.nz

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1 INTRODUCTION

A Geomorphological Baseline Study is required as part of the Lower Whanganui River Management Strategy project being carried out by the Wanganui District Council to manage channel alignment and riverbank stability and to provide an information base for other works. In particular, the study is required to provide a historical context of changes in the lower Whanganui River to assist in understanding the current processes, planning and design work, and predicting future changes both (a) naturally occurring, and (b) as a result of any future management modifications.

Tonkin + Taylor Ltd (T+T) have been commissioned by the Whanganui District Council to undertake a Lower Whanganui River Management Strategy (the Project) and Coastal Systems Ltd (CSL) http://coastalsystems.co.nz/ have been subcontracted to prepare the Geomorphological Assessment (henceforth referred to as the Baseline Study) for which the full Terms of Reference are attached as Appendix A.

Briefly, the study is to extend from the Cobham Bridge, some 3.7 km upstream, to the rivermouth and include approximately 2 km of coast to either side, with the primary focus being the estuary and port area and the rivermouth/coast being the secondary focus. The time of interest is from the late 1800s to the present, with particular emphasis since the 1990s. The study will identify temporal morphology, port and river control structures, notable extreme events, review the literature and undertake higher level (broad) analyses. The study will also summarise available (driving) process information.

Rather than begin with a stand-alone literature review, the various consultant, resident engineer and academic contributions are incorporated within the relevant sections. Sections 2 and 3 cover the primary focus area with Section 2 chronologically describing the morphological nature and management-associated modifications up to the present day, and Section 3 providing the high-level (broad) analysis of these changes. Note, a SUMMARY of harbour development works, morphological change and analysis is included as Section 3.3. The rivermouth and adjacent coast are described in Section 4. Section 5 summarises the energy and sediment regimes (sediment, tides, river hydrology, waves and wind) that drive morphological change. Conclusions and recommendations are contained in Section 6. Pertinent information sources used in the study are summarised in Appendix B, and archive materials are listed in Appendix C with electronic files also provided. Contoured bathymetries between 1893 and 2015 are included as Appendix D and Appendix E contains the sampling density for the two digital surveys (2006 and 2015).

2 ESTUARY and PORT AREA MORPHOLOGY

2.1 Background

The earliest use of the river for colonial navigation is described in, for example, King (1964) with the earliest comprehensive map being the New Zealand Company's Land Settlement Plan of 1842. This map is reproduced as Figure 1 and has had the modern shorelines, port and navigation structures superimposed for comparison. There are several points of note on this earliest map including no Corrlis Island (in Figure 1 the green outline locates the present feature), the large reclamation immediately upstream of the present Port, a shortened South Spit, a very wide rivermouth, and a North Spit with ribbon-like sand bars extending upstream for several hundred metres. This coastal morphology indicates a time of very high littoral sediment supply so likely entrance shoaling and frequent channel changes would be inevitable, making navigation hazardous and challenging, as confirmed by the 1846 Cook's Strait Almanac (King, 1964, p1).

Figure 1 The NZ Company 1842 Land Settlement Plan is the earliest detailed map of the lower Wanganui River including South Spit and coastline. Marked are: the Cobham Bridge in grey, the 2013 riverbanks in green, the 2015 structures in black, and the 2013 coastline in blue.
Information sources: Wanganui Regional Museum, Wanganui District Council

The shoreline on the 1856 cadastral plan is overlaid in Figure 2 along with the 1842 shoreline and demonstrates dramatic change since 1842 with the entrance against the North Head and an extended South Spit, i.e. the typical configuration for this inlet. Greater stability could be expected with the entrance against the North Head and interestingly the Rev. Richard Taylor wrote in the mid-1860s that "Formerly the bar was supposed to be all but impassable, but now vessels of considerable *burthern* manage

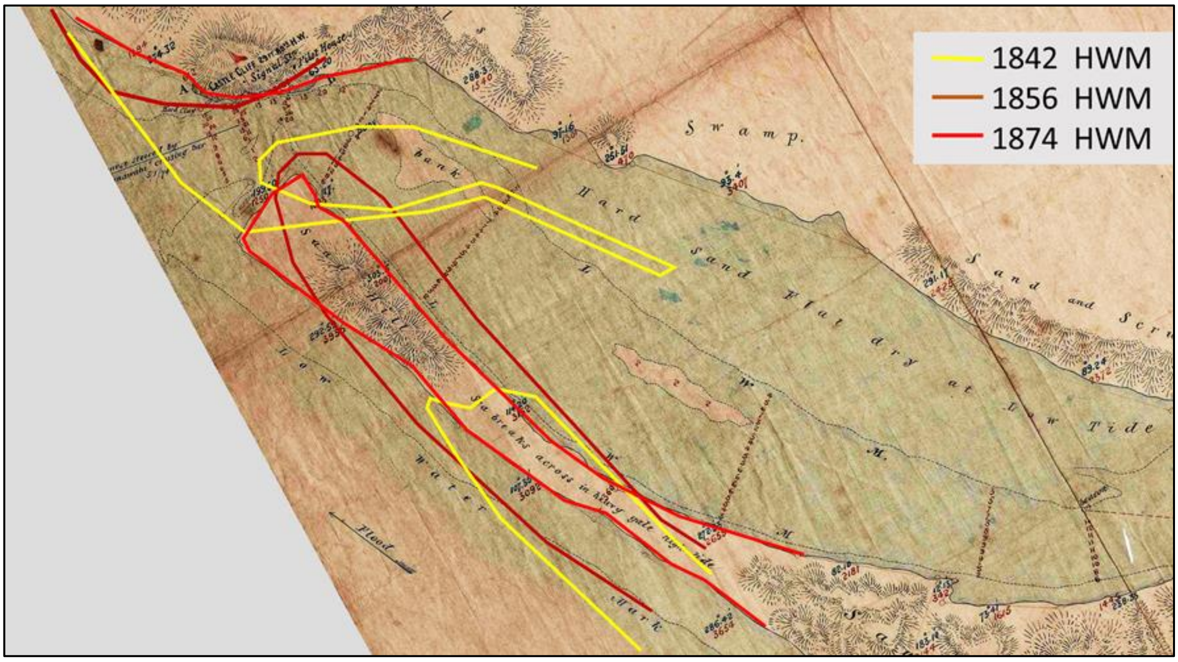


Figure 2 The plan accompanying John Blackett's 1874 report, with the 1842 Wakefield shoreline and the 1856 cadastral (SO 10552) shoreline superimposed.
Information sources: Wanganui Regional Museum and Wanganui Harbour Board Archive

to find their way over it…." (King, 1964, p8), and goes on to note that "the river (nearer the Town wharves) has been greatly improved by snagging and faggoting [bundled branches] the sides and this should be carried on to the Heads").

The first consultant's report was prepared in 1864 by marine engineer James Balfour. His was a descriptive manuscript that noted the need for river training, protection of the eroding North Head and the need for a comprehensive bathymetric survey, but considered the river bar beyond the Heads too exposed to improve.

The next consultant's report was that of John Blackett in 1874 and this was accompanied by a plan (reproduced above as Figure 2) that included some channel depths. The single (south) spit and entrance against the North Head are again evident although the spit had narrowed since the 1856 survey, indicating a reduction in littoral sediment. Blackett was particularly concerned at the state of the spit with waves washing across it in heavy gales and his caution was well founded as the spit breached in February 1877. He also discussed at length the need to train the river and he located the limestone outcrop upriver near Kaiwhaiki which was later used extensively for building river control structures. He again suggested that the North Head needed protection and conceptually discussed construction of a North Mole.

2.2 Stage 1 development (1876 to 1885)

In 1876 the Wanganui Harbour and River Conservators Board was formed with its main concern being to provide sufficient depth of water for vessels to reach the Town Wharf – some 7 km upstream from the Heads. The Board obtained the services of consulting marine engineers Messrs Barr and Oliver to report on works to achieve that objective. A bathymetric survey was undertaken in 1877 and, based on the resulting chart, proposals were prepared for the protection of the (recently breached) South Spit, internal training walls from the Heads to the Town Wharves, protection of the North Head, and a North Mole.

Barr and Oliver's proposals are marked by red lines in Figure 3, with completed works shown in bold and non-completed shown by dashed lines. The spit protection was completed in 1878. The internal training walls were 500 feet (152 m) apart and constructed to half tide level with their rationale being to conserve the tidal prism to ensure scour of the rivermouth bar. Of note is the proposed estuary section (red dashed line) designed to keep the channel well away from the vulnerable South Spit. However, only the upper section (Town Wharves to Landguard Bluff) was constructed, with this alignment modified somewhat upon advice of renowned marine engineer Sir John Coode. The northern wall terminated somewhat beyond Landguard Bluff with a wing groyne on the northern wall (see Figure 3) – apparently to encourage outgoing flows away from the (vulnerable) spit. These internal works were completed in 1880.

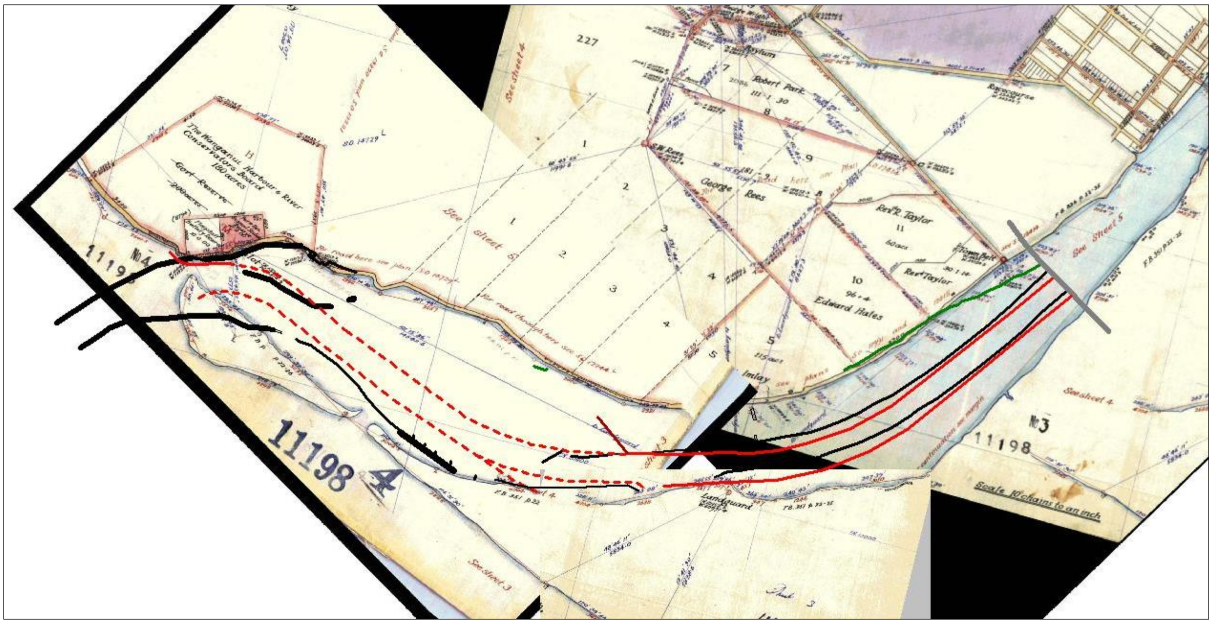


Figure 3 Protection and training works proposed by Barr and Oliver (red) with eventual (current) structures depicted in black. The base map is the 1877 cadastral plan SO 11198 and the February 1877 breach in the South Spit is clearly discernable.

Information Sources: Wanganui Harbour Board Archive, Wanganui District Council and LINZ

The final Stage 1 work was construction of a North Mole in 1884-85. This mole was raised to spring high tide level and was designed to turn the ebb flow into the direction of predominant wave approach (to assist navigation), and to concentrate the flow to ensure maximum scour on the bar located some 500 m out from the heads. However, the original plan was for a 900 m long structure that tapered in height down to low tide level and would terminate at a depth of 4.25 m below low water. The eventual structure was 260 m long (Figure 4) and while reducing lateral migration of the channel, would have had only limited effect on bar scour. The Stage 1 works remained largely unaltered until the onset of Stage 2 in 1908 with investigations beginning in 1894.

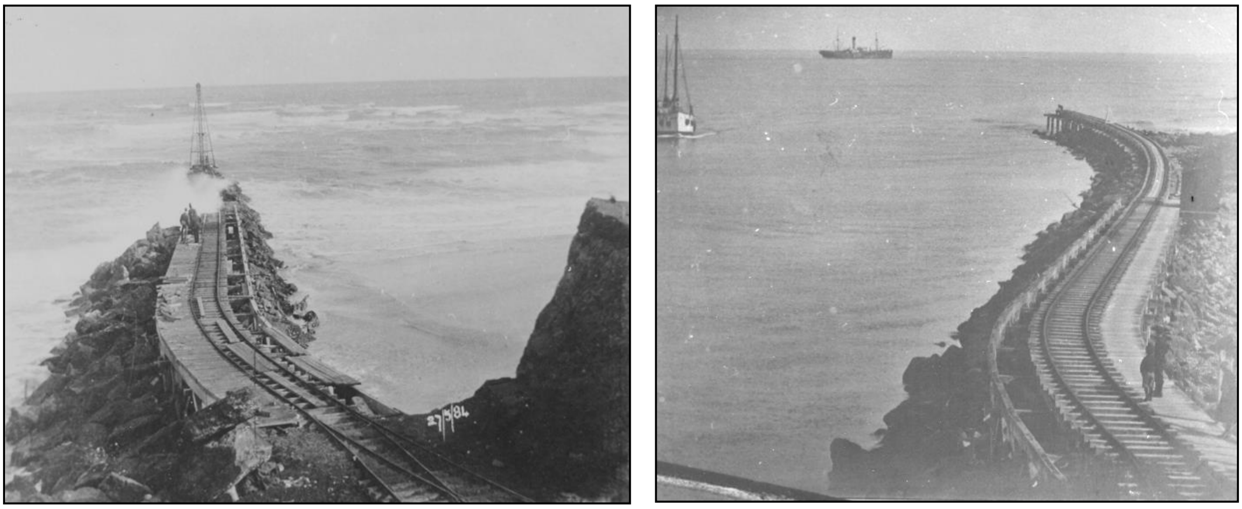


Figure 4 Construction of the original North Mole underway in 1884 on left, and the final structure curving into the prevailing wave direction several years later on the right. Note the beach accretion against the northern (right hand) side of the mole.

Source Alexander Turnbull Library

2.3 Stage 2 development (1894 to 1939)

In 1894-5 consultant marine engineer Leslie Reynolds carried out extensive investigations (including another comprehensive bathymetric survey and hydrological measurements) into the effects of the earlier works, with particular emphasis on loss of tidal prism, and prepared further development plans including the future port at Castlecliff. Reynolds planned to:

- train the main channel against a well-protected South Spit, then
- direct the flow, by means of extending the South Spit protection works, out across the shallows (mud flats) characterising the central/right side of the river, then

- assisted by a curved stone deflection wall extending from the right bank (to become the future turning basin encompassing the Castlecliff wharves), redirect the flow smoothly around and into double moles (keeping with Barr and Oliver's original proposed alignment) with the jet-effect inducing maximum scour of the bar.
- the North Mole was to be raised to spring tide level for the landward section, lowering to mean water for the seaward section, this being to minimise accretion of the Castlecliff coast shoreline while ensuring jetting and scouring across the bar. By comparison, the South Mole was constructed to mean water to facilitate tidal inflow while constraining outflow (jetting).

The Board was impressed, but it would take another 10 years to raise loans (via acts of Parliament) and overcome the political challenges within 3 elections before Reynolds was commissioned to prepare construction drawings. It was then a further 3 years before works would begin.

Reynolds' substantial contribution is depicted in Figure 5, overlaid upon the 1893 bathymetry. Of note, because the channel was redirected out from the spit and across the "shallows", dredgings were used to infill the original channel. Reynolds' photographs illustrating the construction methods are shown in Figure 6.

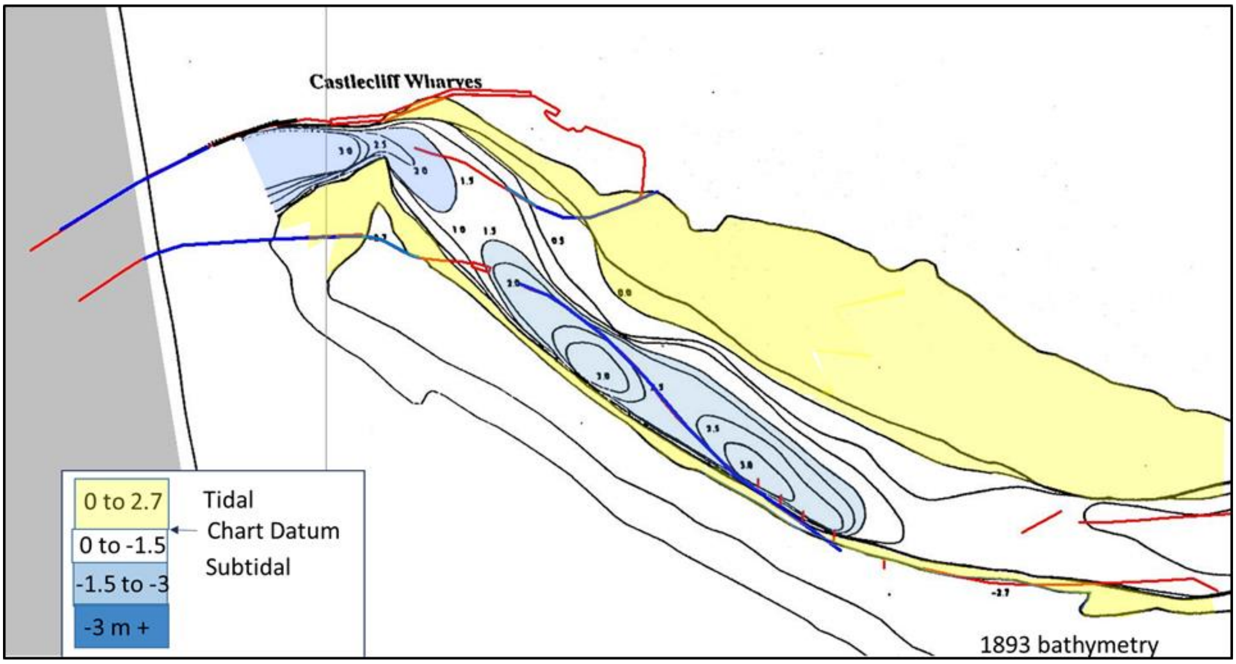


Figure 5 Stage 2 works designed by Leslie Reynolds in 1895 (construction began in 1908) shown by blue lines. The red lines define other works to achieve the current layout. The original (1885) short North Mole shown in black. The underlying map is the 1893 bathymetric chart upon which Reynolds based his design. Information sources: WHB Archive, Macky 1989, Wanganui District Council

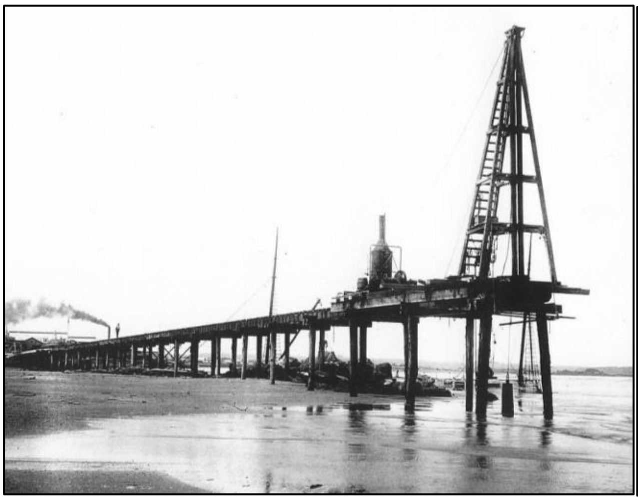
 

Figure 6 Early Stage 2 construction of North Mole c.1909, showing pile driving and wooden staging (left) and side-tipping stone (right). Source: Wanganui Regional Museum

While Reynolds was the primary designer of Stage 2, several other consultants and resident engineers played a role either in development or response to environmental consequence. It is also noted that approval was required from the Marine Department for all structural works in the river and at the entrance. However, it appears that structures on the coast were at the discretion of the Harbour Board's Resident Engineer (RE). The various works and their chronology are illustrated in Figure 7.

In 1918, marine engineer F.W. Furket from the Public Works Department (PWD) described *considerable erosion on the South Spit in 1915-16 may be related to starvation of littoral drift by the moles and recommended beach groynes be constructed*. By 1920, resident engineer H.V.M. Haszard reported the spit had breached and a stone groyne was constructed on the South Beach. Two additional beach groynes were constructed in 1926 as a safeguard against further erosion, again under Haszard's direction. The location of these three groynes are marked in Figure 7.

In 1919, consultant Blair Mason recommended *construction of internal training walls between Landguard and the South Spit, and Basin Wall completion was urgently required.* The half tide training wall along with extensions to the northern wall opposite and downstream of Landguard Bluff appear to have been to keep flow in the Main Channel rather than allowing it to escape across the north of the Estuary. The Basin Wall completion was required to assist in turning flow into the moles.

In 1928 resident engineer Haszard reported completion of an entrance groyne off the South Mole to further constrain the flow and increase jetting across the bar. In 1928 a groyne was also constructed into the river off the wharf at the base of the South Mole (this groyne being referred to the South Mole Wharf [or Jetty] Groyne) to deflect flow across the river toward the basin/wharves where sedimentation was occurring. Subsequent depth measurements presented to the Board (Engineer's Reports) indicated both structures were effective, but both subsequently fell into disrepair and both would be reconstructed, the base groyne in 1950 and the entrance groyne in 1973. In 1928 a wing groyne was also added to the downstream end of the northern half-tide wall; this appears to further direct flow into the Main Channel and along the South Spit rather than allowing it to take a more northerly route.

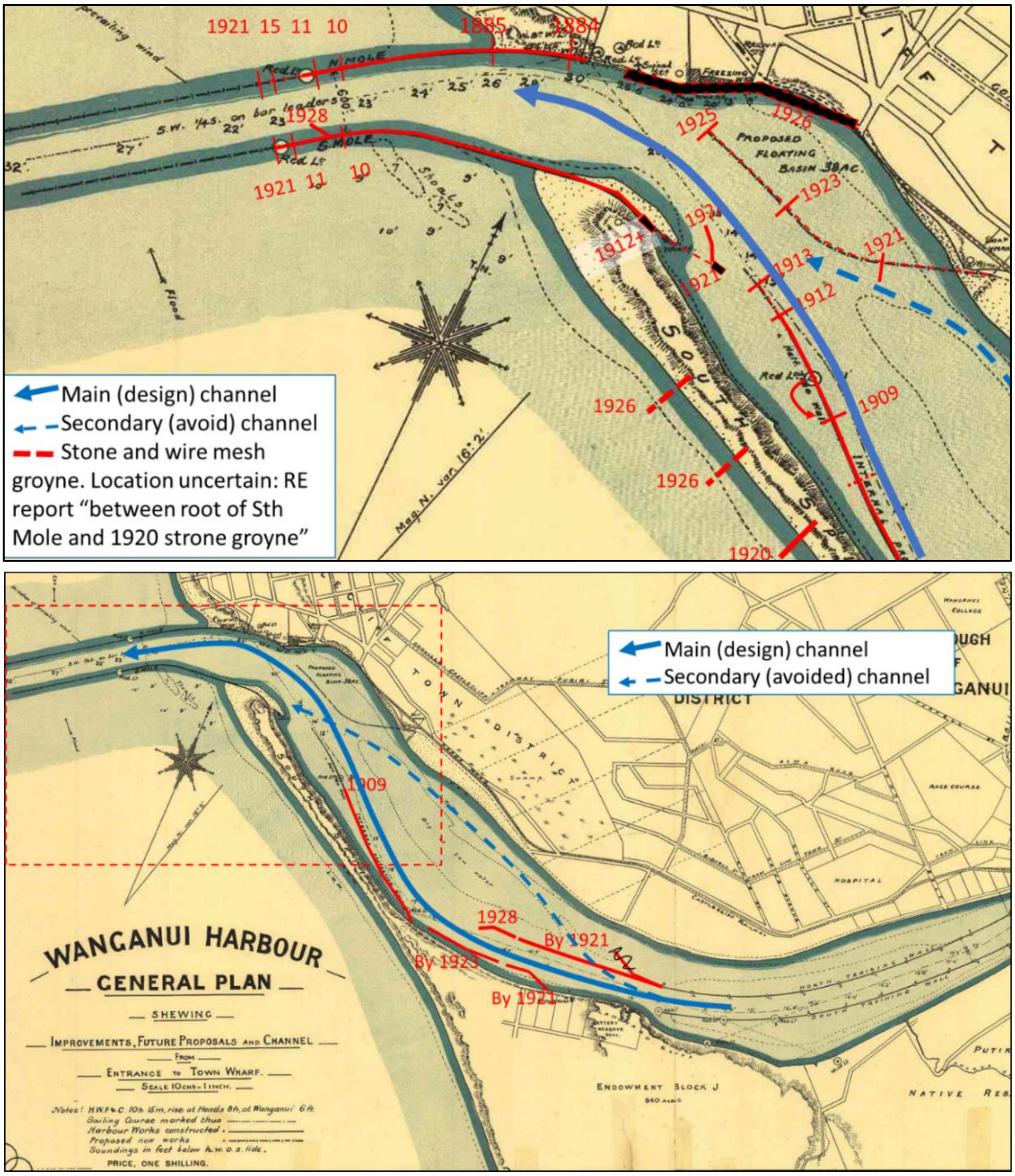


Figure 7 Chronology for Stage 2 works. Upper figure covers area marked by dashed red line in in lower figure. The underlying General Plan appears to be for publicity purposes. Information sources: Gibbs (1962), King (1964); various plans and documents from WHB Archives, New Zealand Nautical Almanacs (see Appendix B).

In 1921, consultant Blair Mason had recommended *raising North Mole above HWM to make it "sand tight"*. However, by raising the North Moles enough to prevent waves washing sand into the river the adjacent shoreline would move seaward, i.e. accretion would occur and with it wind-blown sand hazards. Reynolds had previously recognised such a shoreline response and his design had reduced the mole height to seaward accordingly. These processes are illustrated in the photo and bathymetry depicted in Figure 8. Then in 1929, G.A. Lee recommended raising both moles to above spring high water, to increase the jetting effect.

The mole extensions (wooded staging) were completed in 1921 according to data in King's (1964) thesis (p97). At that time some 64,492 tons of stone had been placed. Subsequent raising used another 76,863 tons and this included increasing the height of the seawardmost 330 m of the North Mole some 2 m above the mean high water spring level. While his raising of the moles may have improved jetting efficiency it was to have a significant effect on both the northern and southern coasts and this will be described later in Section 4. By the mid-1940s, stability of the South Spit was of major concern to the Board.

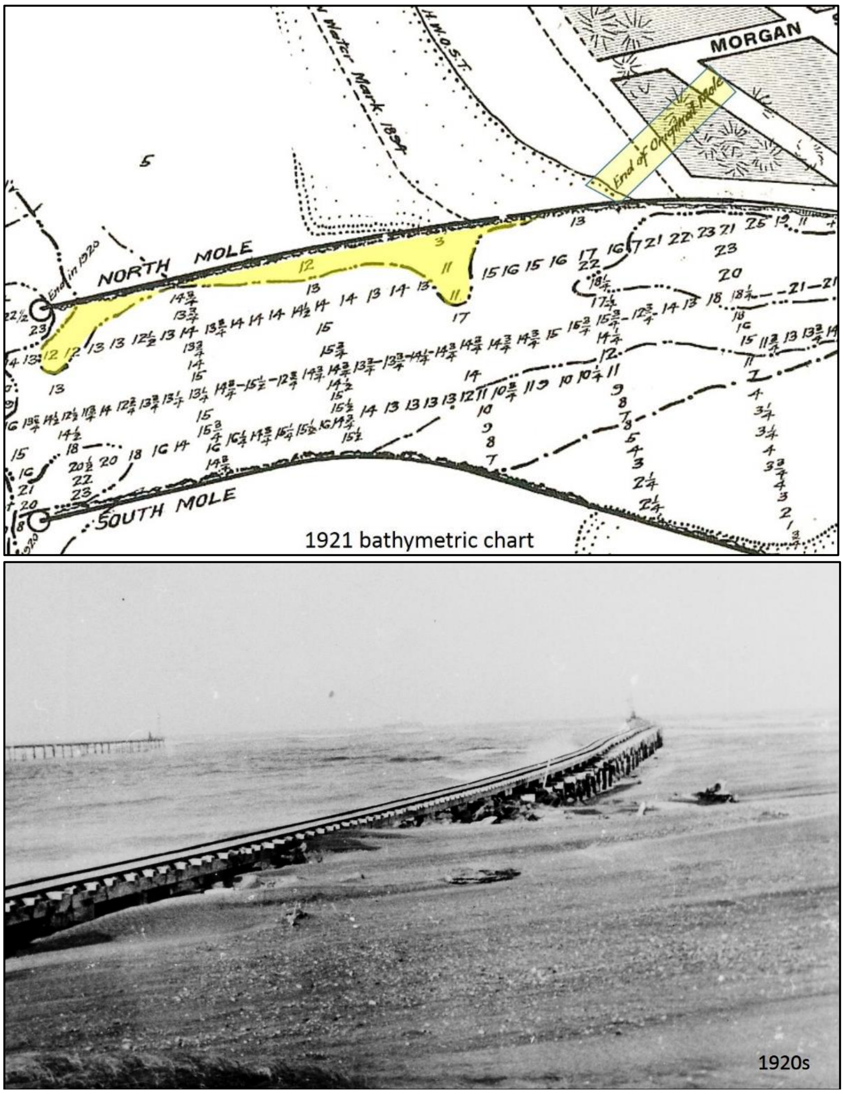


Figure 8 Effects of the low stone levels on North Mole in the 1920s. Upper bathymetry shows shoaling (yellow) in the river adjacent to the North Mole where beach sand has been washed across the low stonework. Bottom photo depicts the wide inter-tidal beach and wave action in the river adjacent to the mole indicating relatively shallow water.

Sources: WHB Archive, John Souter collection

2.4 1940s South Spit Breach

The harbour appears to have come closest to functioning as designed during the 1930s and 1940s, but the late 1940s were dominated by one event – the breach in the South Spit. The following account is compiled from the Harbour Master's monthly and annual reports, Resident Engineer and Supervisors' monthly and annual reports, and accounts from King (1964). The various morphological features referred to are depicted in Figure 9.

Leading up to the breach on 18 August 1946, the spit had become vulnerable along much of its 2.5 km length with actual breaching occurring on occasion but then naturally closing. During this time, Harbour Board staff carried out a range of pro-active conservation measures in an effort to prevent a major breach. What made the 18 August breach different was that the wave event was immediately followed by a flood on 20 August. The resulting opening was 270 m long at MHWS level and its base was 1m below chart datum. Beach and dune sand were carried directly into the Main Channel and this began reducing navigation depth.

By the end of 1946 the situation had worsened and the Public Works Department (PWD) offered assistance. Renewed breaching occurred in July 1947 (Figure 9 shows aerial views at high tide) with overwash sand creating a delta which crossed the navigation lead line in the Main Channel. It is noted that the late 1940s was a period of relentless storms and high seas, likely an El Nino event. In January 1948, the PWD had taken over work closing the breach by constructing a continuous wall using 3 to 5 ton blocks of concrete. By May 1948 the navigation channel had reduced to "gutter" and a large dome-shaped shoal (described by the Harbour Master as an "island", and by the Supervisor as a "sand hill") had formed opposite the breach. River flow was carrying littoral overwash sand as far as the Basin. Shipping to town was now via a new landward channel (the Balgownie or North Channel) around the landward side of the "island" which is henceforth referred to as the *Dome Shoal or Dome*.

In Figure 9 (lower), an arrow from the new "deep" channel to the Turning Basin wall suggests breaching was contemplated, i.e. an entirely new estuary/harbour configuration was considered. In July 1948 stone was removed from the half tide training wall where the new channel meets the Main Channel opposite Landguard Bluff "to encourage improvement of this (North) channel for navigation and to reduce flow in the restricted spit (Main) channel so the amount of sand washing downstream to the Basin would diminish". This stone was then used to help close the breach – a more economical alternative to concrete. In August 1948 the spit breached again but by late 1948 it had been sealed, having been open for over two years.

A substantial amount of littoral sand appears to have entered the Estuary during this time and it took a further two years of dredging (four machines were hired) to return the Main Channel to navigable dimensions for all vessels using the Town Wharves. There is no mention in the records of any reinstatement of the half tide wall opposite Landguard Bluff.

A comprehensive bathymetric survey was undertaken later in 1948 and this is reproduced as Figure 10 which provides scale to the features in the sketch (Figure 9). Sediment transport pathways (inferred by morphological signatures) are included in Figure 10 and show breach sediment moving upstream in the centre of the estuary (along the dome-shoal) and downstream around the margins. The dimensions of the Dome indicates a substantial volume of sediment entered the Estuary via the breach and this was to have longer-term effects on morphological behaviour.

Figure 9 South Spit Breach on 5-7-1947 (upper photos) taken at high/ebbing tide and as sketched by Harbour Board staff in 1948 (lower) with surrounding morphological features referred to in the text having been marked.

Sources Ian Moore (photos) and WHB Archive.

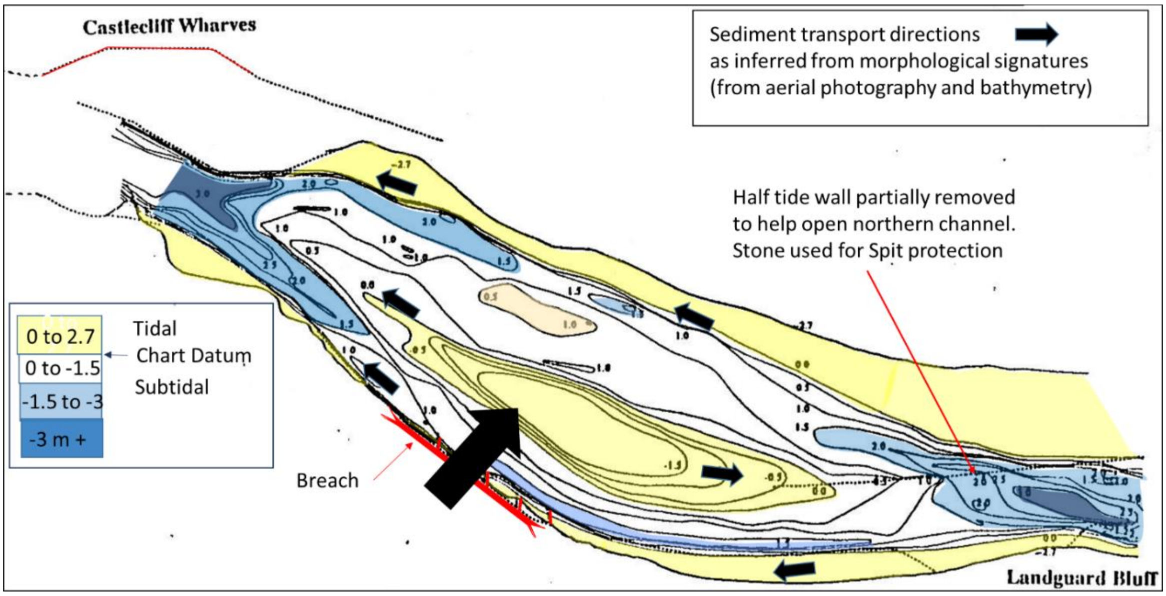


Figure 10 Estuary bathymetry in mid to late 1948 compiled from a survey carried out by the Public Works Department in July to October 1948. The dotted black lines locate training walls and other structures. The concrete protection wall, along with river groynes constructed during the late 1950s, are shown in red.

Data source WHB Archive, Macky 1989

2.5 1950s morphological adjustments and control works

During the 1950s a series of secondary control works were used to either adjust river flow and sedimentation associated with the South Spit Breach effects or to prevent its reoccurrence. The following account of these works, their effectiveness and their fate, is again compiled from Harbour Master monthly and annual reports, Resident Engineer and Supervisor monthly and annual reports, and King (1964).

A consequence of the South Spit breach was littoral sediment washing into the river and being transported downstream where it caused shoaling about the Turning Basin entrance and reduced depths along the adjacent wharves. A structural response to this was re-establishing the wooden pile and waling groyne at the base of the South Mole, i.e. the South Mole Jetty [Wharf] Groyne described earlier in Section 2.3. This groyne was extended some 300 feet (91 m) into the river between May 1949 and July 1950 with the intended effects eventuating: outgoing flow was deflected toward the Basin where depths did increase, while depths decreased in the lee of the groyne, i.e. against the South Mole immediately seaward of the groyne. Continued shoaling about the wharves lead to approval by the Marine Department of a further 250 feet (76 m) extension in September 1951. However, construction was deferred as channel/basin depths began to further (naturally) improve. This groyne is depicted on the right side of Figure 11, a 1955 plan showing existing and proposed works.

To complete the story of the South Mole Jetty Groyne, it was intentionally shorted by 35 feet (10.7 m) in January/February 1958 in an effort to reduce the sand bank (the so-called Tanae Bank) that had developed seaward of the groyne which, as explained below, had become problematically large. Following some reduction in size of the Bank, the South Mole Jetty Groyne was subsequently extended by 60’ (18.3 m) and strengthened in 1959 to help achieve its original purpose. The groyne was still evident in an aerial photo taken in early 1994 (Figure 12); however, it did not appear in subsequent aerial photographs.

By 1951 the Main Channel (adjacent to South Spit) had been reinstated by dredging, but the North Channel, including its opening opposite Landguard Bluff, had been left to adjust to the modified flow regime. As noted above, shoaling about the Basin entrance (the apparent response to the sediment from the Spit breach flowing downstream), had reduced in the early 1950s making a planned extension of the South Mole Jetty Groyne unnecessary. In August 1953 such a ‘reversal’ was further described in the resident engineer’s monthly report when he noted that the sand bank immediately seaward of the South Mole Jetty Groyne and opposite No 1 wharf (i.e. the Tanae Bank) was becoming larger and shallower (location shown in Figure 11) along with the channel moving 200 feet toward the Basin entrance and depths increasing off No1 Wharf.

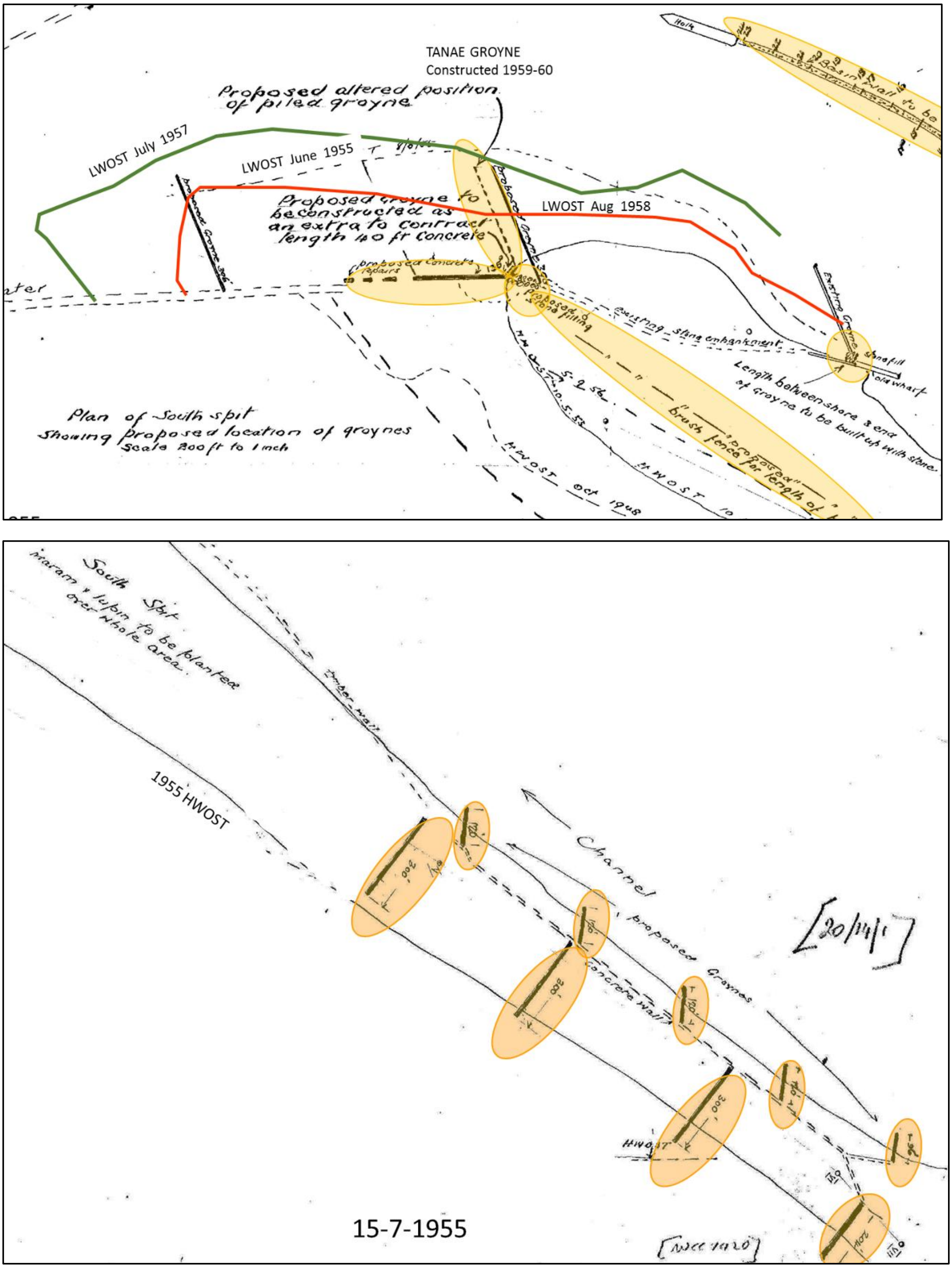


Figure 11 Existing and proposed river control works (upper) and spit control works (lower) control works as depicted in the 1955 Harbour Board drawing. Highlighted features mark structures which were subsequently built. The Tanae Bank, as defined by LWOST, for 1955 is shown by the dashed black line, 1957 by the green line and 1958 by the red line. Note upper and lower parts are from the same drawing.

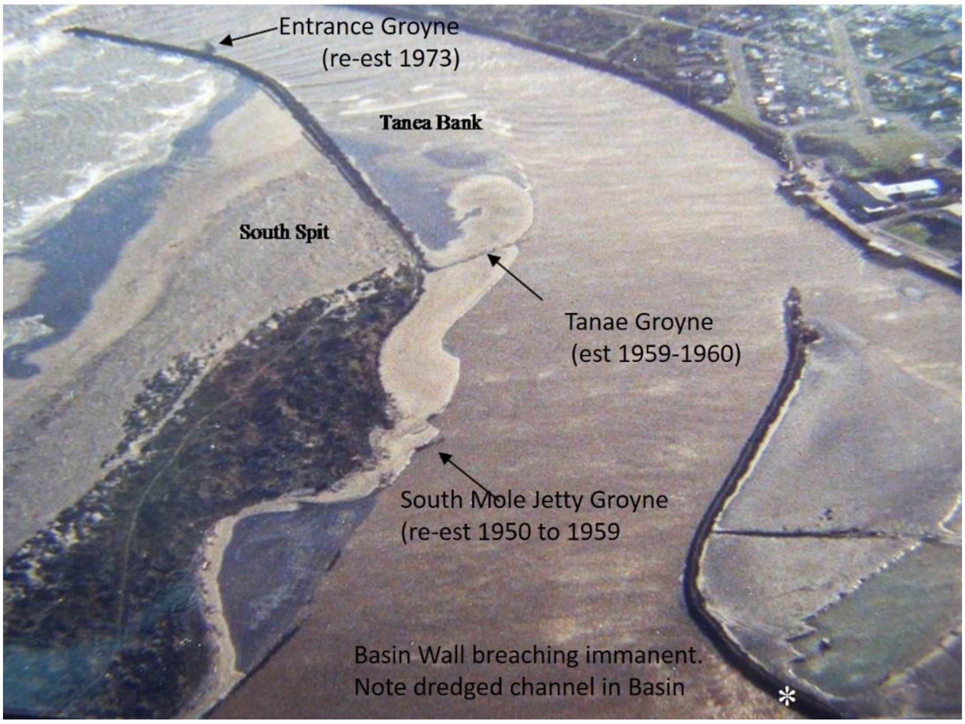


Figure 12 Oblique aerial photo taken on 25 April 1994 at 1500 hrs. The three groynes extending from the South Mole are marked and the asterisk locates the site of the breach in the Turning Basin Wall which occurred about a month later.
Source Coastal Systems Ltd Archive

In August 1956 substantial shoaling was reported in the Main Channel at the seaward end of the South Spit reach with only 0.15 to 0.3 m below low water and the Harbour Masters report noting "It appears almost certain that the deposit has come from the northern bank of the channel, the combined action of ships propellers and freshes of the last month breaking away the sand deposits". This situation subsequently lead to closing of Main Channel for shipping to the Town Wharves. The area of shoaling is marked in Figure 13.

In his report of the following month (September 1956), the Harbour Master noted the increasing size of the Tanae Bank and linked this to improving depths at the wharf opposite. In the March 1957 report, the Tanae Bank was described as extending "a little over half way to the Mole (seaward) end and in width from the South Mole to close to the line of the leading beacon". (The lead line is used to define the navigation channel from the mole ends to the wharves). The report continues "At present this causes no interference to shipping but a close watch should be kept on the sand accumulation". The position of the bank in July 1957 is marked by the green line in Figure 11.

In April 1957 the Resident Engineer reported the "the island [i.e. the Dome Shoal] was building up and encroaching towards South Spit" and the Harbour Master reported in April 1957 that "The course of the river shows a decided change toward South Spit. This is particularly noticeable in the bend below the airport". These comments and the morphology depicted in Figure 13 enable the inferred directions of sediment transport to be identified.

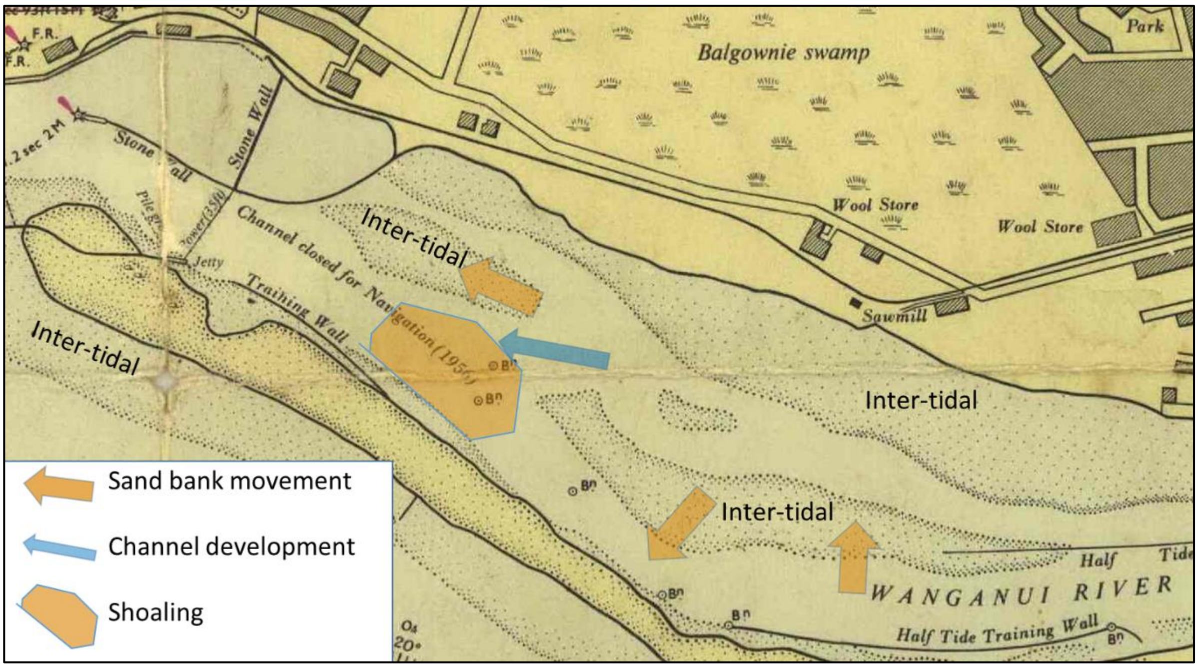


Figure 13 Wanganui harbor and estuary in 1956/57 with area of morphologically inferred channel development (scour) marked along with directions of sediment transport and area of recent shoaling in the main channel.
Source Navy Chart NZ 4612 (survey 1956, additional soundings from WHB 1957).

Possible large-scale relationships between the various morphological developments described above are now discussed. Following closure of the South Spit Breach, the Main Channel had been dredged back to pre-breach dimensions. Seaward flow (ebb tidal plus river flow) through the North Channel would therefore reduce and sedimentation occur. However, with stone from the half tide wall opposite Landguard not being replaced following closure of the breach there would remain a tendency for (higher) flows to still enter the North Channel and follow a less defined channel around the northern side of the Estuary. During higher tides, such flow could rejoin the Main Channel by flowing across the top on the Dome (and hence its encroachment toward the Spit). At lower tides such flow would concentrate around the northern margin of the Dome thereby causing secondary channel development and shoaling in the noted location in Figure 13. The timing of these estuary processes and the development of the Tanae Bank suggests a causal relationship in which this shoal sediment was a contributing source for growth of the Bank.

On 11 August 1957 the M.V. Tanae grounded on the Tanae Bank and the Master (Captain Williams) refused to return to Wanganui until this bank (having now acquired the vessels name) was less hazardous to navigation. Urgent dredging followed along with shortening of the South Mole Jetty Groyne by 4.6 m. A flood in February 1958 removed a further 6.1 m of groyne and the bank was trimmed back enough to satisfy Captain Williams. The reduced size of the Tanae Bank in August 1958 is marked by the red line in Figure 11.

In the early 1950s, gaps began appearing in the landward section of the South Mole with the resident engineer reporting that waves on South Beach were washing sand into the river and contributing to the size of the Tanae Bank. The proposed repair using concrete and stone is marked in Figure 11, along with the proposed raising of the Basin Wall to above flood level. Detailed plans are included on the CD (Appendix C). These works were carried out during 1957-59. Undermining of this base section of the South Mole has been reoccurring in recent years, coinciding with deepening of the Tanae Bank (detailed in Section 3).

Also proposed in the 1955 drawing (Figure 11) are two other groynes extending from the South Mole across the Tanae Bank. Only one groyne was contructed (marked in Figure 11 as the Tanae Groyne), this occurring in 1959-60. This pile and waling groyne was 90 m long and had a 15 m *wing groyne* at the junction with South Mole to deflect waves that propagated upriver and across the bank away from the mole. The Tanae Groyne is clearly evident in the 1994 photo (Figure 12) and is also largely discernible in the 1998 aerial photo, but only 20 m remains in the 2004 photo and just a few piles survive today. Little explanation was found as to the purpose of these Tanae Bank Groynes, but in keeping with the South Spit Jetty Groyne, they were to deflect flow onto the northern side of the river to increase the size of the Main Channel, with accretion on their seaward side protecting the South Mole and dissipating the energy of waves propagating upriver.

As marked in the 1955 drawing (Figure 11), five groynes on the river side of the South Spit were being contemplated for longer-term spit protection in the vicinity of the 1946-48 breach. The need for the groynes came with the Dome Shoal migrating toward the spit and constricting the Main Channel (Resident Engineer's report, April, 1957). Between 1957 and 1960 the five wooded pile and waling groynes were constructed, the upstream groyne being 90 feet (27.4 m) long and the other four were 120 feet (36.6 m) long. While these structures are still effective today, stone strengthening was recently carried out to prevent outflanking (discussed in Section 2.7).

During the 1950s, the South Spit was also under pressure by beach/dune erosion and again groynes were proposed in the 1955 drawing (Figure 11). In 1957, the Resident Engineer's report noted the spit was only 100 feet (30.5 m) wide in places with 16 chains (322 m) of upper beach/dune protection with brush wood undertaken. In May 1958 gales exposed 500 feet (152.4 m) of the concrete wall protecting the old breach, lowered the beach by 4 feet (1.2 m) and removed previous log and scrub protection resulting in substantial lengths of dune erosion. In 1958-59, three pile and waling groynes each 300 feet (91 m) long were constructed from the dunes out across the intertidal beach. A forth groyne at the southern end was 204 feet (62 m) long. The beach was subsequently reported to have risen 1.2 m, indicating that the structures were having some positive effective. However, this beach is subject to large sediment availability/width fluctuations and while the groynes were evident in the 1960 aerial photo and partially evident in the 1965 photo, they were buried thereafter.

Finally, between 1960 and 1962, a brush fence was erected along the length of the spit, from the South Mole to beyond the 1946-8 breach (noted in the 1955 proposal, see Figure 11). Morphological dune signatures from this structure were evident until 1986 in aerial photographs.

2.6 1960s to 1990s: downscaling

This period was characterized by development proposals, the introduction of private company administration in the late 1980s, reducing port operations and maintenance, structural changes in the Basin and recreational boating developments.

Three port development schemes have been proposed since the 1960s: Riddell (1967, 1970), Payne, Sewell and Associates (1978), and WDC (1993), and while these proposals contained some new process information (McLean and Burgess, 1969; Tonkin and Taylor 1983; Patterson 1991, 1992), they did not eventuate so did not lead to any actual morphological change.

Changes that did occur in the basin are shown in Figure 14. In May 1994, a 90 m long breach (excavated down to Chart Datum) was opened in the Basin Wall with a dredged channel linking the breach to the wharves (Figures 12, 14). The initial purpose of this breach was to maintain water movement within the Basin during the falling stage of a flood, thereby reducing backwater sedimentation. However, flood flows were found to take a straighter path across the Basin thereby bypassing Number 2 Wharf and No 3 Wharf so a 350 m pile-and-tyre permeable training wall was constructed to direct outgoing flow toward and along the wharves (Figure 14).

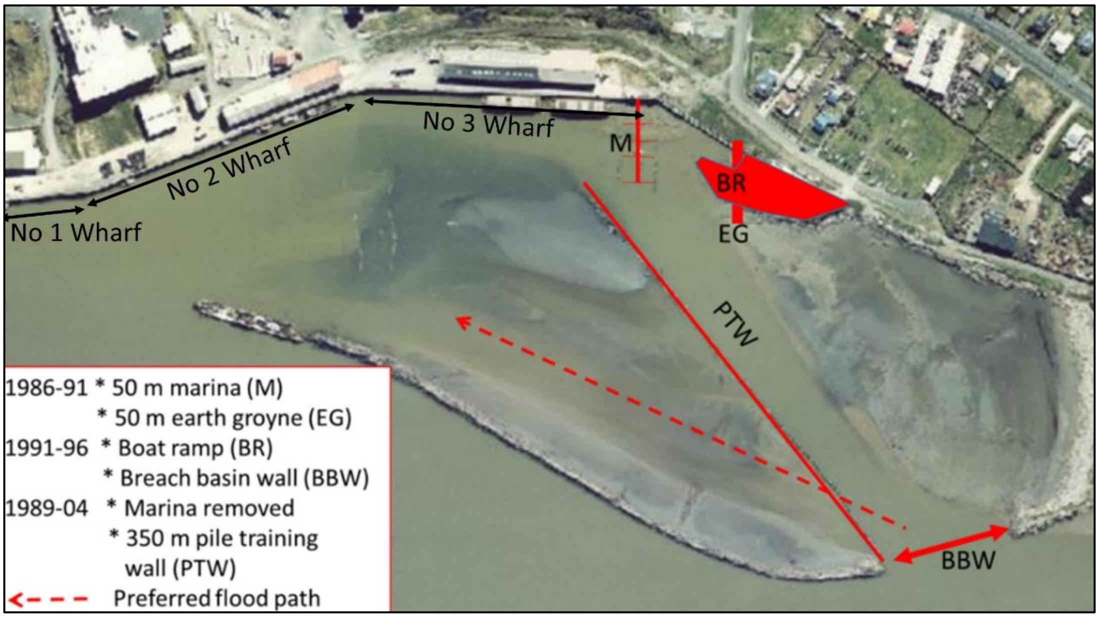


Figure 14 Structural changes from the 1980s to present superimposed on a 2008 aerial photo. The dashed red line locates the preferred flood path.

The remaining structures in Figure 14 were carried out to assist recreational boating and include a 50 m marina that was removed by 2004 although some piles remain. An earth groyne had a brief existence before being replaced by the boat ramp. Improving the boat ramp is the focus of present engineering investigations.

The North Mole had strengthening carried out near the seaward end in the 1960s and early 1970s using concrete blocks that were poured in-situ. From the late 1970s, demolition rubble (concrete with reinforcing steel) was placed against the seaward side of the mole beginning at the beach end as a low cost strengthening/maintenance measure. At the beach end of this mole the 3 m wide crest increasing to about 7 m by 1980, 11 m by 1990 and 16 m by 2000 with the width tapering to seaward.

The South Mole also had some concrete block strengthening carried out in the 1960s/70s after which the staging/railway, used for the concrete making operation, had become inoperative.

In 1973 the original 1920s entrance groyne off the South Mole was reconstructed to improve the jetting effect of the moles. The 1994 remnants are evident in Figure 12 and today just a couple of piles remain. During the 1960s the wharf/jetty at the landward end of the South Mole fell into disrepair and today only some of the piles remain beyond the mole end.

In 1998, Horizons Regional Council carried out a study of the lower reaches of the Whanganui River (Williams, 1998). The study reported on the various protection works present and their condition. The four river groynes in the 1946-8 breach area were sound and various training/protection walls immediately downstream, while in a poor state, were still functioning reasonably effectively. Immediately upstream of the South Mole landward end, Williams described a large and potentially problematic erosion embayment, referred to in the present report as the *South Mole Embayment*.

2.7 2000 to 2015: increasing awareness

This final time period is characterized by an increasing awareness of the role of protection/control structures and the need for their maintenance, along with greater understanding of their purpose within the context of specific estuary and rivermouth processes.

The period began with the Wanganui District Council having concerns as to the stability of the North Mole following the extensive dumping of demolition rubble during the previous two decades. Reports were commissioned from OPUS (2002) and Massey University (2002) covering the integrity of the structure and the environmental implications. Briefly, the former (Opus) report documents the structure as being "marginally stable" and it would be "unwise not to replenish the sides with suitably chosen material". The latter report identified the effects the structure had had on beach/dune processes, and stated that future dune instability was likely and would eventually require management intervention. Since that time the District Council has carried out controlled rubble and stone placement on the mole, albeit primarily for recreational hazard mitigation and visual appearance rather than addressing the base stability of the structure.

Following on from the concerns raised by River Engineer Mr Garry Williams (Williams, 1998), a subsequent report was commissioned by the Wanganui District Council and Horizons Regional Council (Williams, 2004) to investigate user requirements and how these related to remedial measures. Williams noted that "the 1990s and early 2000s was a period of relatively high intensity for floods with exceptionally high activity and channel reworking in 1998". William's findings included the following:

- breaching of the Basin Wall had contributed to siltation upstream of the breach (the Q-West reach) and this situation should be resolved, possibly by sealing the breach.
- while the report speculated as to causes of the erosion embayment at the end of the South Mole, it recommended further investigation.
- a bathymetric survey of the lower estuary was required with possible future numerical and physical modelling, and
- of particular concern was the lack of clarity about management responsibilities and legal status based on current legislation of the harbor and its facilities and assets. The report recommended formulating an asset management plan to define standards and requirements and financial arrangement between parties.

In 2006, the Wanganui District Council commissioned Mr Ian Moore [Geologist, Land Management Consultant and previously Wanganui District Manager for Horizons Regional Council], to carry out periodic ground inspections and repeat photography for the purpose of informing the council of high risk developments/instability that required action. It appears this approach (observation and recording change over time) was suggested by

Dr Jeremy Gibb [Coastal Scientist] who visited the Wanganui Coast at that time and provided general management comment to council officers. Mr Moore's initial report was in February 2006 and provided a general description of the physical state of the spit and potential driving forces, along with numerous photographs. Several recommendations were made for more quantitative monitoring. The following report in January 2010 included comparison photos of 2006, 2008 and December 2009. Erosion fronting structures sealing the 1946-8 breach were noted along with expansion of the South Mole Embayment. Stability issues along the landward end of the South Mole were also mentioned as well as foredune erosion along the South Beach. Moore again recommended more detailed monitoring, along with research into the original protection works and observation of flood effects. The next report was December 2010 along with an update in February 2011 describing observed effects of a flood on 23-24 January 2011. These reports noted ongoing erosion along the spit riverbank with significant loss of vegetation and exposure of early protection works. It recommended that the gap between the river groynes and original breach protection wall needed to be addressed and river dynamics need to be better understood by studying available data and undertaking cross-section sounding between the airport and port at no less than six monthly intervals.

Following on from the 2010/2011 Moore inspection reports, the Wanganui District Council and Horizons Regional Council commissioned river engineer Mr Garry Williams to undertake a further review of erosion and to propose a programme of repair works. The Williams (2011) report focused on the following:

- between the concrete block wall at the site of 1946 breach, and the terminus of the half tide training wall extending downstream from Landguard Bluff, an embayment had formed. To prevent this embayment further enlarging and outflanking the adjacent concrete block wall (fronting the 1946 breach), a low stone groyne was recommended across the river flats to the upstream end of the wall; the plans are included in Appendix C. This work was completed in May 2014.

- erosion was now occurring along the site of the 1946 breach and the original concrete block protection wall was exposed (this area had been stable in 1998) with gaps existing between the wall and the four pile and waling groynes. To prevent the groynes being subject to "landward flow concentrations and associated structure stresses", gaps between the concrete block wall and the groynes should be "plugged" with stone and at the upstream embayment (plans included in Appendix C). This work was completed in May 2014.

- the embayment at the end of the South Mole was described as being more active than in 1998 and posed an increasing risk of a river/sea "breaking though". Control using rock protection was considered, but a lattice of driven piles from a barge was favoured, with numerical modelling being used to finalise the design. This design work has not occurred.

- the South Mole was considered to be subsiding, with gaps occurring in places and undermining occurring where the 1950s concrete repairs (NB Figure 11) had been carried out. The report concluded that maintenance topping up along the mole would be required at some time in the future.

The analysis in Section 3 provides data and process explanation relating to issues raised by Williams and Moore.

Mr Moore's final report dated October 2014, focuses on the moles and records observations from a site inspection with Mr Peter Atkinson (engineer with the Port of Taranaki) on 28 May, 2014, along with subsequent observations and discussions. The most recent work on the North Mole was considered cosmetic work and there was a need for additional maintenance works to ensure structural integrity, in particular the need for a more supportive base. Gaps in the lower South Mole permitted wave penetration and this threatened the integrity of the structure as did ongoing erosion/undermining of the section in the vicinity of the old Tanae Groyne (repaired in the late 1950s). The South Mole Embayment was also noted with some concern. The report concluded/recommended the need for bathymetric soundings were needed to understand (driving) channel changes, and that initial "patchup" works should be undertaken to infill gaps in the moles and provide basal support where needed, with more comprehensive remedial work being staged later. It also recommended that access feasibility to the South Mole along the South Spit be investigated along with investigation of a suitable rock source with a view to stockpiling for emergency and future use.

Note that Mr Atkinson's subsequent retirement resulted in Tonkin and Taylor Limited taking on the role of Port Advisors to the Whanganui District Council in 2015. Their first request was for the present Baseline Study to be carried out, a study that was also suggested by Mr Moore and Mr Williams as a means to proceed with defining harbour protection works and improvements.

3 ANALYSIS and DISCUSSION

3.1 Methods

An initial high-level quantitative analysis carried out in August-September 2015 utilized bathymetric data surveyed in 1893, 1921, 1948, and 2006 along with part surveys from 1956, 1983 and 1993. These seven bathymetric samples are reproduced in Appendix D. The 1893 bathymetric data represents the post-Stage 1 and pre-Stage 2 harbour development works. The 1921 bathymetry was surveyed during Stage 2 - just after the moles had been extended to their final length but before they were raised to their final height with stone and concrete. So the 1921 survey represents both and an intermediate developmental form and also is indicative of the pre-breach (South Spit) form. The 1948 sample was taken as sealing the South Spit Breach was completed so represents the breach-affected bathymetry. The 2006 data represents post-breach adjusted bathymetry. While, the 58 year time-span between the 1948 and 2006 data sets is excessive, the 1957, 1982 and 1993 part surveys are helpful in identifying inter-survey change. Aerial and satellite imagery (1942, 1960, 2065, 1974, 1980, 1982, 1986, 1991, 1996, 1998, 2000, 2004, 2008, 2009, 2010, 2011, 2012, 2013a, 2013b) were also used to assist in defining inter-survey bathymetric change and to further identify sediment transport direction using exposed (inter-tidal) bedforms. Note details of all data sources are summarised in Appendix B

At the time of the initial analysis, only the 2006 bathymetry was available digitally as x,y,z data points, the remainder being contour maps which were found to be too spatially inaccurate for direct digitzation. The alternative was to locate transects or cross-sections (referred to hereafter as *sections*, which best represented the available data coverage and the apparent range in morphology (form). Each bathymetric sample was georeferenced at each section thereby deriving acceptable spatial accuracy (+/- 2.5 m). The location of the 9 sections are depicted in Figure 15, and the profiles at each section have been superimposed upon the corresponding 2006 profile in Figure 16.

At a *progress presentation in October 2015* the advantage of procuring and including a current bathymetric data and corresponding spring low tide/low river flow aerial photography was recognised and new surveys were undertaken by Horizons Regional Council in November 2015 and by Lawrie Cairns and Associates: Survey Services and Aerial Photography on 26 December 2015. Note an aerial survey was also undertaken by the Wanganui District Council on 27 August 2015. Unfortunately, the tide and river flow at the time of photography had been such that they prevented exposure of the lower inter-tidal estuary, although some useful information was still captured. By contrast the 26 December 2015 photography was taken with the water level approaching chart datum.

Figure 17 shows the new 2015 bathymetric contours superimposed upon the 2015 Lawrie Cairns aerial photograph in the centre image. For comparison, 2006 bathymetric contours are superimposed upon the 2004 aerial photograph in the top image, and the net bathymetric change between the two digital bathymetries (2006 and 2015) is shown in the lower image.

The bathymetric sampling density for the 2006 and 2015 surveys are depicted in Appendix E. The 2015 survey clearly has a much higher sampling rate but upper margins along the right side of the river were not always covered. By contrast, the 2006 survey has a much lower sampling density and this required extensive interpolation in places (e.g. Section 3). In addition, the sampling differences resulted in some artifacts as a 10 m wide representative area about data points was required to derive a satisfactory number of net change points. Metadata detailing equipment and processing specifications have not been provided, so typical accuracy is assumed (+/- 0.5 m position and +/- 0.05 m elevation). All data sets were converted to NZMG and Wanganui Chart Datum using official conversions. Finally, we note that inconsistencies are clearly discernable in the 2006 data within the Basin Channel elevations being greater than elevation on the adjacent flats. Analysis of recent change within the Basin could therefore not be included in the present analysis.

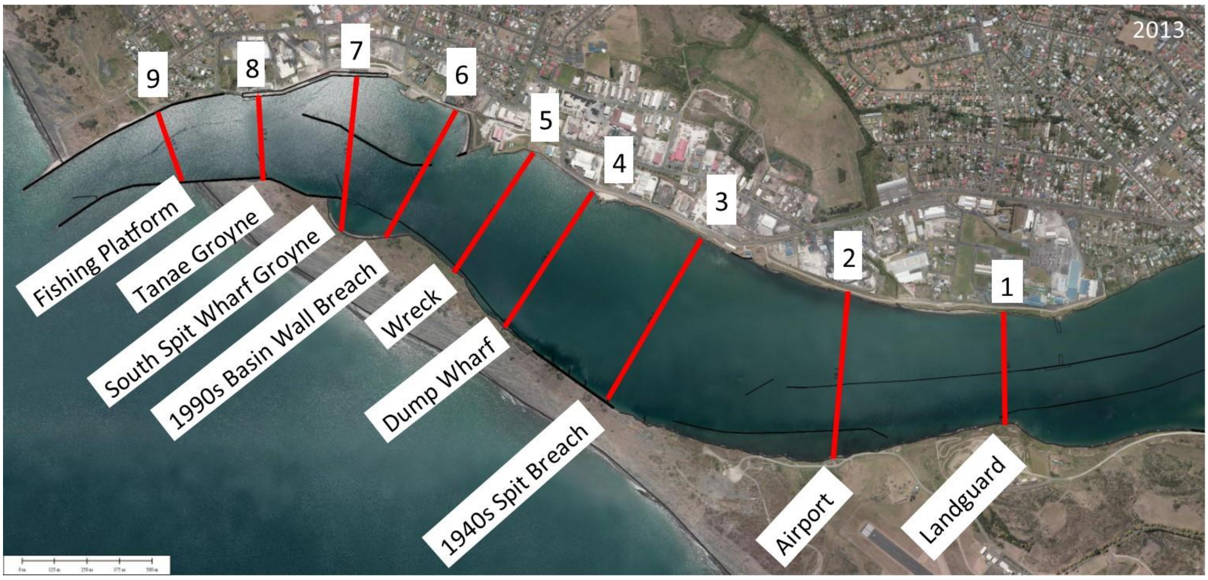


Figure 15 Section locations (red lines), numbers and names superimposed upon a 2013 aerial photograph. Black lines locate river control structures.

3.2 Section Results

3.2.1 Section 1: Landguard

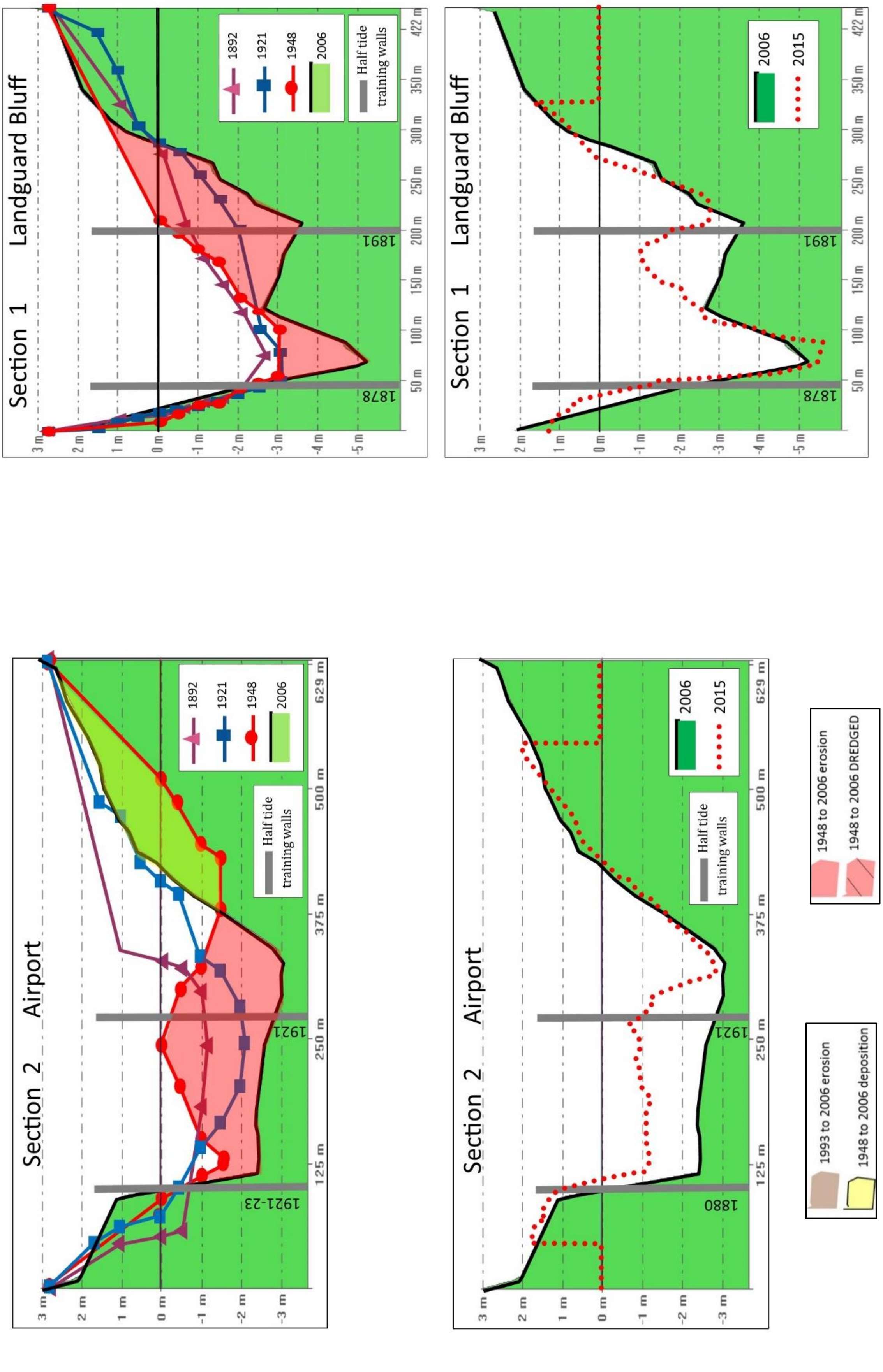
The 1948 contoured bathymetry (Appendix D) indicates sediment that had washed through the South Spit breach had moved upstream (under tidal flows) and affected the channel configuration below Section 1, which in turn has resulted in a build-up of sediment about the northern (right) half tide training wall (Figure 16). The 2006 bathymetry is tending toward pre-breach (1921) form, but with deeper channel(s). Note the 1948 to 2006 erosion is shaded red in the Section graphs. No pre-2006 data is available (since 1921) to define prior riverbed behaviour above Languard.

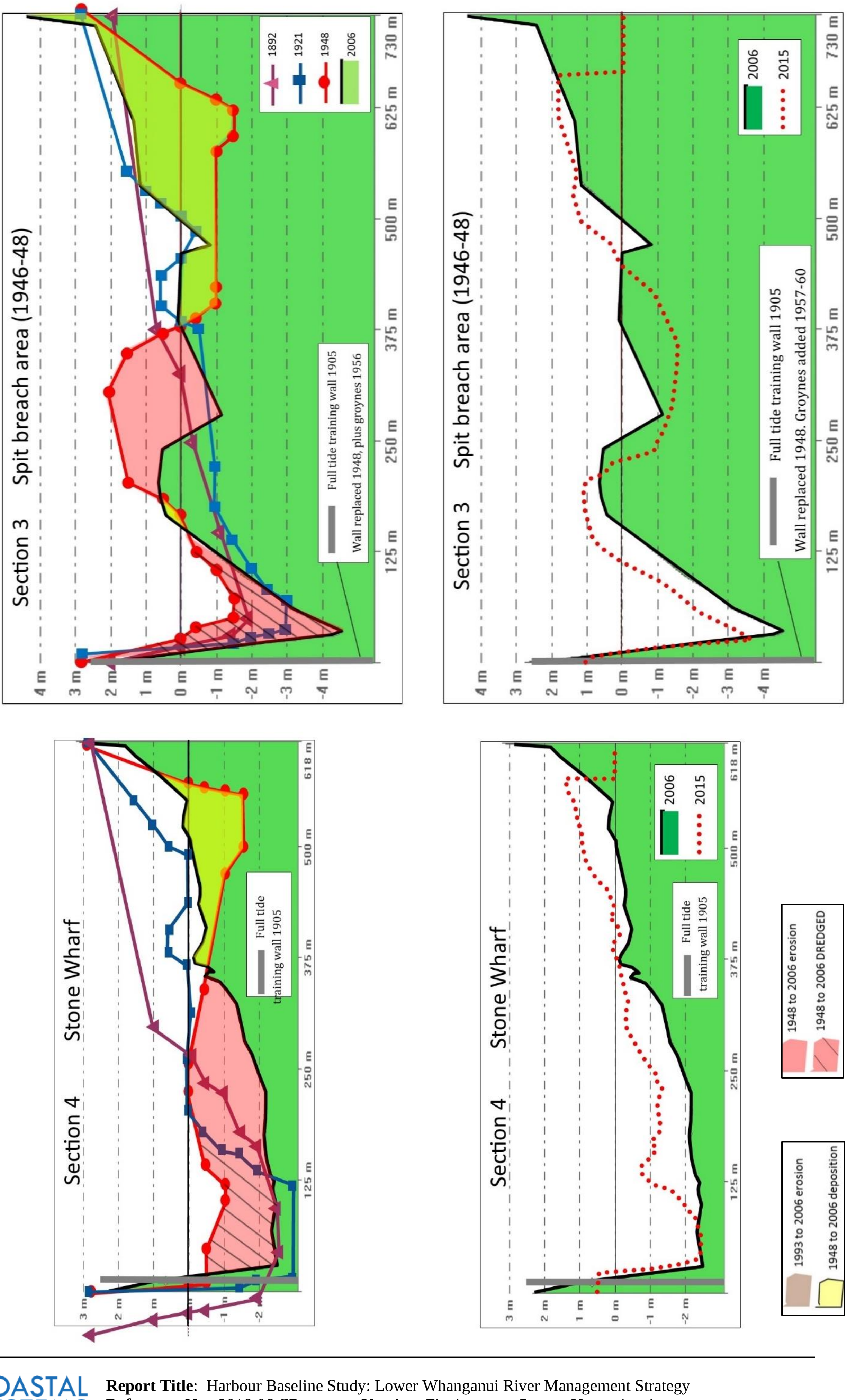
The 2015 profile (Figure 16) is dominated by sediment accumulation within the northern side of the (trained) channel and lesser accumulations along the margins of the sounded area. Net erosion has occurred between these three accumulations. The contoured bathymetric chart and net inter-survey change map (Figure 17) indicates these changes may be associated with river/ebb tidal flows "cutting the corner" (in the old Imlay Wharf area) possibly in response to flow-deflection associated with upstream channel accumulations between the training walls. Morphology and bedforms on the aerial photography indicate river (flood) flows and tidal ebb flows are affecting this area.

3.2.2 Section 2: Airport

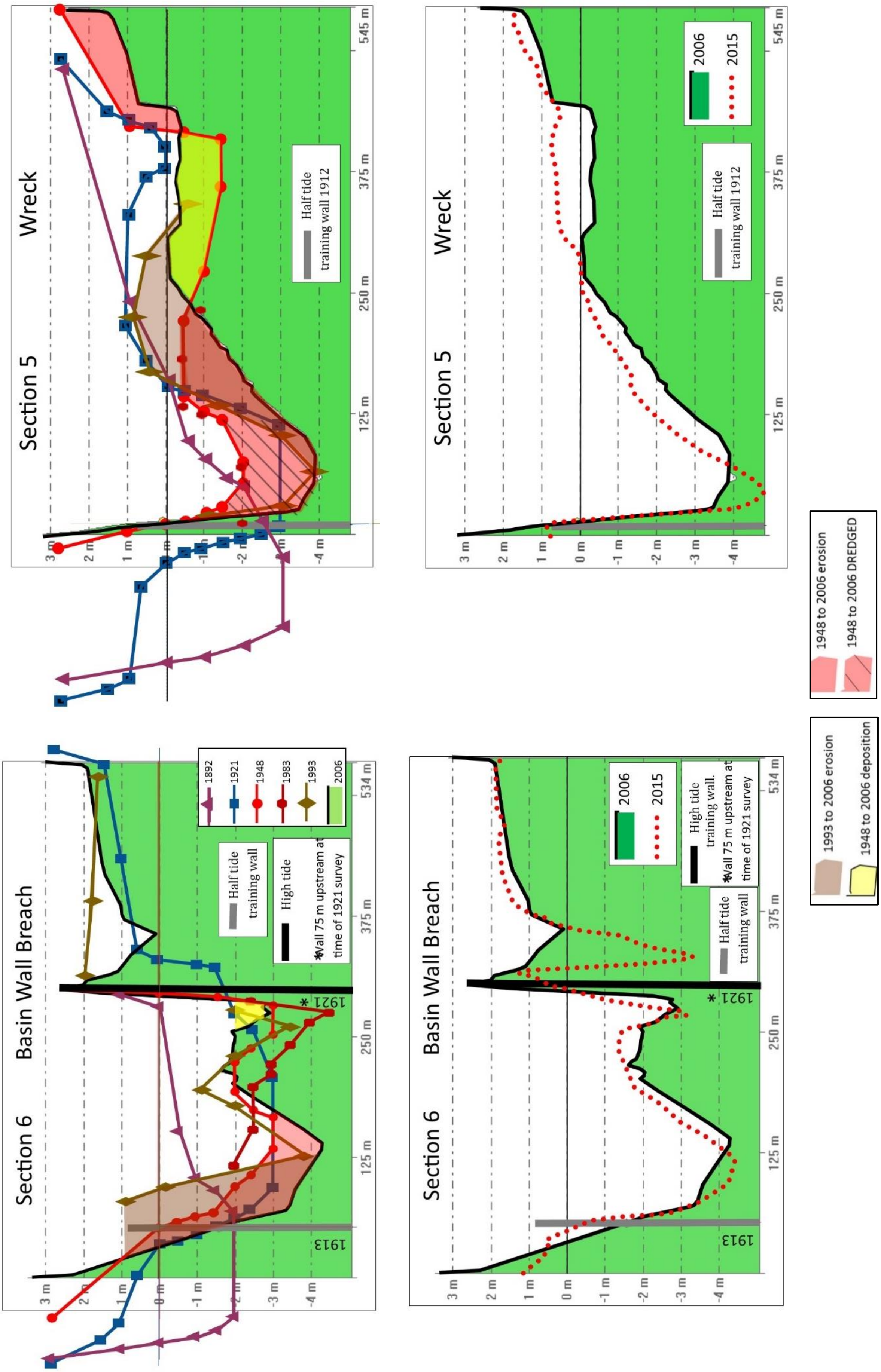
The 1948 profile in Figure 16 shows a substantial shoal has replaced the well-defined channel depicted by the 1921 profile, with shallower channels occurring on each side of the shoal, and the North Channel being substantially larger than the southern channel (referred to as the Main Channel). The shoal appears to be a continuance of the large dome-shaped sand-bar which formed opposite the South Spit Breach (clearly defined in Section 3 profiles). This feature was referred to as an "Island" in Harbour Board records and is referred to as the Dome Shoal or just the Dome in this report. The 2006 profile shows a return to pre-breach (1921) form (1948 to 2006 change is shaded red), but the double channel is still discernible with the northern (right) side being the larger.

The 2015 profile (Figure 16) is dominated by significant accumulation of sediment between the training walls with depths below Chart Datum having more than halved (to about 1 m) since 2006. Bedforms between the southern half-tide training wall and riverbank do not indicate a dominant transport direction. The main channel now clearly lies on the landward side of the northern training wall with minor erosion (<0.5 m) across the adjacent Northern (or Balgownie) Bank (or Flat). Flow concentration into the centre of the river and across the Balgownie Bank appears to be driven by upstream accumulation processes described above in Section 1 as well as outgoing flow deflection of Landguard Bluff coupled with likely ongoing subsidence of the northern training wall following its artificial lowering in the late 1940s. Bedforms across the Balgownie Bank are indicative of net downstream sediment transport.


Section 1 Landguard Bluff
1892
1921
1948
2006
Half tide training walls
1878
1891
2006
2015
Section 2 Airport
1921-23
1921
1880
1993 to 2006 erosion
1948 to 2006 deposition
1948 to 2006 erosion
1948 to 2006 DREDGED



Section 3 Spit breach area (1946-48)
1892
1921
1948
2006
Full tide training wall 1905
Wall replaced 1948, plus groynes 1956
Section 3 Spit breach area (1946-48)
2006
2015
Full tide training wall 1905
Wall replaced 1948. Groynes added 1957-60
Section 4 Stone Wharf
Full tide training wall 1905
Section 4 Stone Wharf
2006
2015
Full tide training wall 1905
1993 to 2006 erosion
1948 to 2006 deposition
1948 to 2006 erosion
1948 to 2006 DREDGED


Section 5
Wreck
Half tide training wall 1912
2006
2015
1948 to 2006 erosion
1948 to 2006 DREDGED
Section 6
Basin Wall Breach
1892
1921
1948
1983
1993
2006
Half tide training wall
High tide
*Wall 75 m upstream at time of 1921 survey
High tide training wall.
1913
1921 *
1993 to 2006 erosion
1948 to 2006 deposition

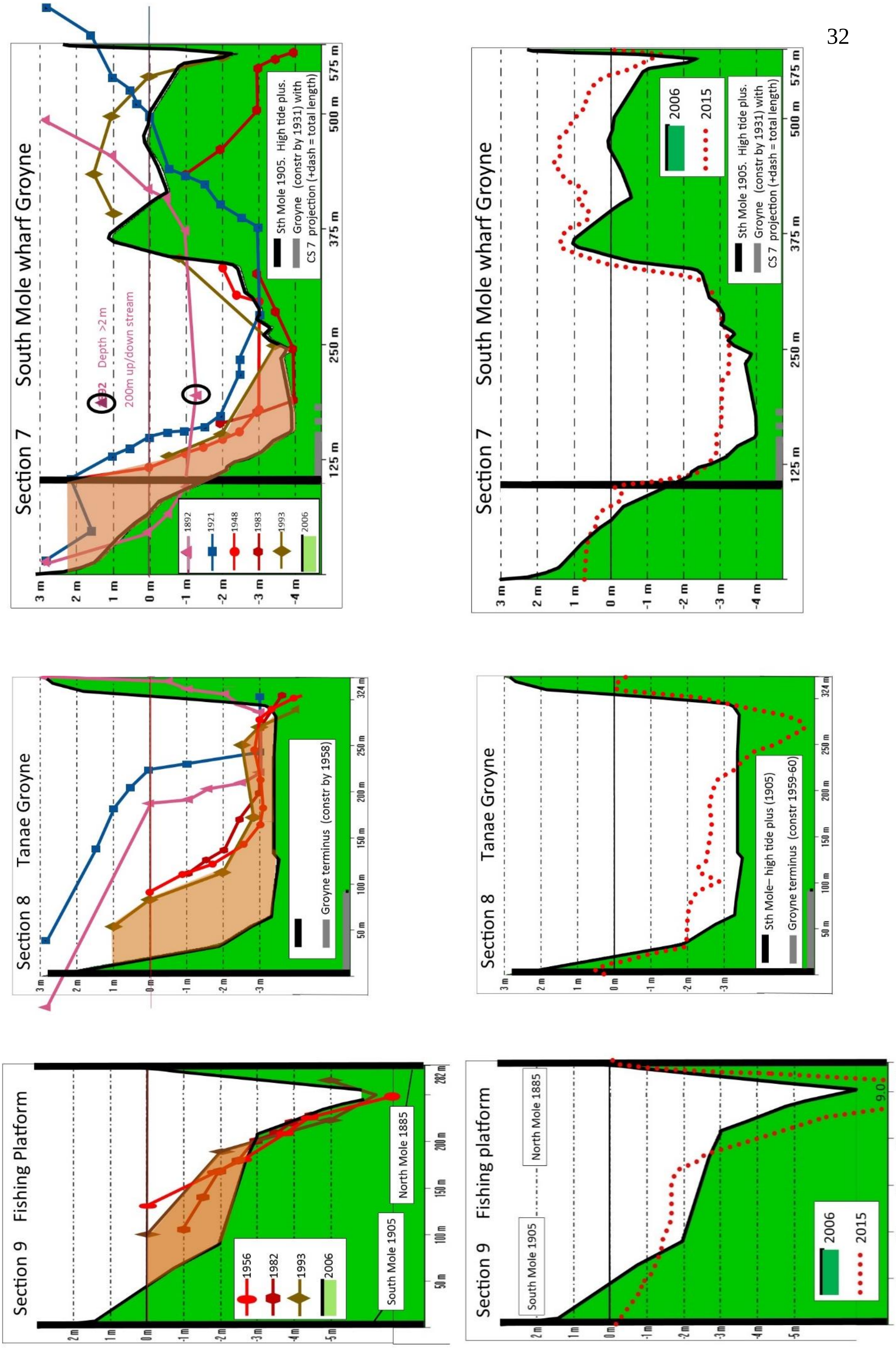


Figure 16 Section graphs superimposed upon the 2006 bathymetry on left, and 2015 bathymetry superimposed upon 2006 on right. Note constant scales in all graphs enable direct comparisons.

3.2.3 Section 3: 1940s South Spit Breach

Comparing the 1948 breach-affected profile with the 1921 pre-breach profile shows littoral sediment washed through the gap in the South Spit and infilled the Main Channel and the extensive Dome Shoal was able to grow to some 1.5 m above Chart Datum. Scour then occurred across to the right bank with the (now predominant) Northern or Balgownie Channel located close to right bank. The 2006 profile shows a return toward the pre-breach form (c.f. 1921 profile) with infill of the northern channel (shaded yellow), removal of much of the Dome and deepening of the Main Channel (shaded red). However, the remnant Dome and adjacent Northern Channel are still evident. The size of the Dome appears to have reduced by erosion of its northern side (shaded red in the Section 3 graph) by ebb tide and river flow down the Northern Channel, with some (small) deposition on its spit side (shaded yellow in the graph), indicating the dome is also migrating toward the spit.

The 2015 profile in Figure 16 show the Dome has migrated further into the Main Channel (toward the South Spit) as well as its upper surface rising about 0.5 m. The upper left bank of the river is eroding. The Balgownie Channel appears to have grown in cross-sectional area and, as with the Dome, its location has moved toward the Spit. The Balgownie Bank (between the Balgownie Channel and the right bank of the river) has had a net accumulation of sediment; however as at Sections 1, 2 and 4, recent data (detail) is lacking close to the riverbank.

The spatial data (Figure 17) shows the accumulation within the Main Channel extends upstream to Section 2 and beyond and this, together with movement of the Dome into this channel, is probably constricting river flood flows that drive bank erosion along this section of the South Spit, resulting in the need for the groyne strengthening in 2012. Excavation of the Balgownie Channel appears to be associated with increased flows eroding the downstream end of the Dome prior to rejoining the Main Channel. As with the upstream sections, bedforms across and along the Balgownie Bank indicate net downstream sediment transport and migration of discrete sand bodies which appear to grow in size closer to the Turning Basin Wall.

To better define the Dome's behaviour, its outline was located on aerial photographs and its centre of mass tracked. The resulting plot (Figure 18) shows the Dome's net location has moved 124 m toward the spit and 239 m downstream since 1948. Note that only the initial part of this behaviour is well represented by the Section 3 data. Furthermore, the movement is nonlinear with a net locational change of 31 m (18 m towards the spit, 25 m downstream) between 1948 and 1991, then 284 m (70 m towards the spit and 275 m downstream) between 1991 and 2013, and 20 m toward the Spit coupled with 64 m upstream between 2013 and 2015. Bedforms observed on the Dome (2008 aerial photo) typically infer down-stream sediment transport with bedforms on the December 2015 aerial photo indicating flood flow diverted to the north across the upstream end of the dome (consistent with flow constriction noted above), and to the south across the downstream end as the Balgownie channel flow seeks to rejoin the Main Channel.

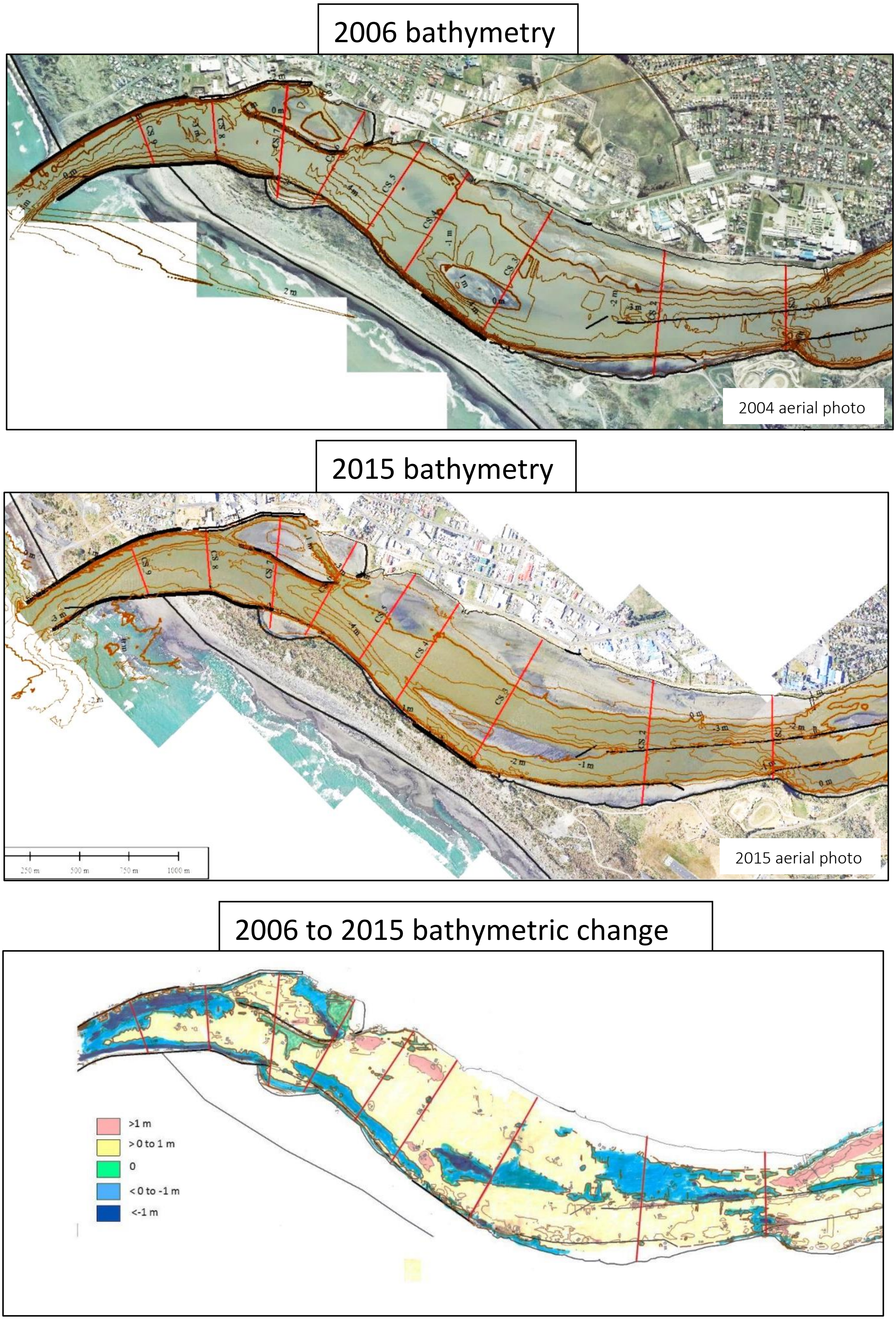


Figure 17
Spatial bathymetric data; 2006 at top and 2015 centre, each superimposed upon relevant aerial photographs. Net inter-survey differences depicted in lower image.

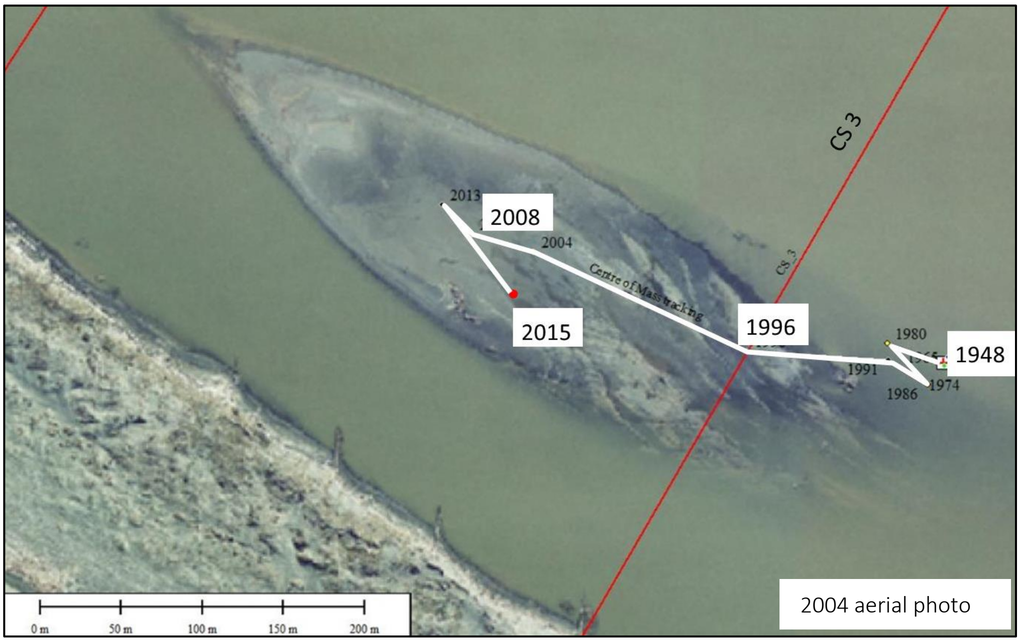


Figure 18 Migration tracking of the Dome-Shoal based on aerial photo centres of mass between 1948 and 2015 superimposed upon the 2004 aerial photograph

The Dome's most recent upstream-directed centre of mass relocation appears to reflect change in the wider sediment transport regime (rather than depicting a change in net sediment transport direction), and while this change may be ongoing/systematic, it may alternatively have been triggered by the June 2015 flood. It is noted that while the flood caused significant flooding upstream of the Cobbam Bridge, it was relatively insignificant in the Estuary due to coastal control parameter values, e.g. neap tide range. However, sustained moderate events are capable of driving significant morphological change in coastal settings.

3.2.4 Section 4: Dump (or Stone) Wharf

The 1921 profile shows the South Spit river bank at Section 4 had moved out to the half tide wall established in 1912; this was assisted by *dumping* dredged sediment excavated from the realigned Main Channel which passed through a *shallow* area comprising part of the Ebb Tide Delta (see Sections 2.3 and 3.2.5). Spit-breach sediment does not appear to have affected the centre of the 1948 profile which had the same elevation in 1948 as in 1921. However, the spit-breach did cause infilling of the Main Channel and scouring of the Balgownie Bank on the right side of the river to accommodate the then predominant North (Balgownie) Channel. The 2006 profile is tending toward the pre-spit breach form on the right and left sides, but there is a significant sediment deficit in the centre of the profile (shaded red in the Figure 16 profile graph). This 2006 bathymetry indicates the Balgownie Channel may be rejoining the main channel about this location after flowing around the northern side of the Dome.

The 2015 profile in Figure 16 is characterised by an overall accumulation of sediment apart from erosion against the riverbank on the spit side and a slight deepening of the Main Channel possibly associated with accumulation-associated narrowing of this channel. The spatial data (Figure 17) suggests the narrowing of the main channel is associated with deposition related to the scour occurring along the Balgownie Channel's Dome margin. Note, in sediment transport systems, concentrated scour is invariably followed by (downstream) deposition, this being so regardless of the fluid medium (water or air). Indeed, this sequence can be seen repeatedly in the deposition/erosion diagram (Figure 17) under the seaward directed river flow/ebb tidal flow regime. Finally, it is noted that the Balgownie Bank at Section 4 has accumulated up to 1 m of sediment since 2006.

3.2.5 Section 5: Wreck

The 1921 profile shows the South Spit river bank at Section 5 had moved out toward the half tide wall established in 1912 (assisted by dredge dumping from the shallows) with the Main Channel being well formed. A broad shoal (flood tide delta) characterised the centre-profile in the 1921 cross-section together with a relatively small North Channel along the right hand side of the river. The 1948 profile shows infill across to the training wall, and also significant infill within the adjacent Main Channel – this apparently the result of littoral sediment that had washed through the breach and been subsequently carried downstream. The 1948 profile also shows net scour of the flood tide delta and replacement with a broad North Channel. The 1982 profile shows little recovery, although accuracy of these data at this Section is questionable. The (more reliable) 1993 profile shows the main channel and adjacent flood tide delta had regained their pre-breach (1921) form. However, a relatively smaller North Channel is still evident.

By contrast, the 2006 profile shows a dramatic regress from that (1993) recovery (shaded light brown) and bed level had actually deepened to below breach level closer to the Main Channel. This 2006 bathymetry indicates the Balgownie Channel may still be rejoining the Main Channel about this location - at is was at Section 4. In addition, the 2006 profile depicts a better defined Northern Channel than at any of the other 2006 Sections, and the tidal flat on the far right had deepened to below both breach (1948) level (shaded red) and also pre-breach 1921 level. Unfortunately, the 1982 and 1993 surveys did not extend to the right side of Section 5 thereby limiting the information-base on pre-Basin Wall Breach morphology. However, such deepening (in the 2006 section) closer to the right bank indicates strong seaward directed currents and this is supported by bedform orientation evident on the 2008 aerial photo (Figure 19 upper image). This contrasting 2006 morphology of a well-defined North channel and inter-tidal bed erosion on the right of the profile, compared with the other sections, may have been associated with locally strengthened river flood flows resulting from the 1994 Basin Wall Breach.

The 2015 Section 5 profile in Figure 16 shows the spit riverbank is eroding and the main channel has deepened on the spit side. The bathymetric chart and aerial photo in Figure 17 also support the Balgownie Channel joining the main channel at Section 5. As at Section 4 the rest of the Figure 16 profile is characterised by deposition, including infill of the previously (2006) well-defined Northern channel with up to 1 m of sediment. The bedforms in the 2015 aerial photo (Figure 19) depict substantial sand-bodies migrating downstream and it seems such sediment availability has overwhelmed any localised scour tendency associated with the Basin Wall Breach. Indeed, the more recent upstream erosion/deposition pattern described at upstream sections indicate such sediment inputs are likely to continue into the future.

3.2.6 Section 6: Basin Wall Breach

The 1921 profile shows the left bank had also shifted out toward the 1913 training wall at this section, and the partly constructed Turning Basin Wall (still 80 m upstream) was having an effect on the right side of the riverbed.

The 1948 profile shows deposition on the seaward side and base of the Main Channel – again likely to have resulted from breach overwash littoral sediment migrating downstream. Shoaling is evident mid channel and scour adjacent to the Basin Wall – the Basin Wall Channel, formed by ebb flow convergence against the Basin Wall. The 1983 profile shows a deeper Basin Wall Channel and shallower Main Channel while the 1993 profile shows a reversal indicating the occurrence of a range of shorter-term fluctuations. Of particular note in the 1993 profile is a significant build-out from the South Spit into the Main Channel beyond the 1913 half-tide training wall.

The 2006 profile shows considerable change with the Main Channel increasing in size to the largest on record with the spit side of the channel retreating to behind the training wall (net erosion shaded brown and darker red in Figure 16). In contrast, the Basin Wall Channel reduced in size to the smallest of all samples (yellow in Figure 16), possibly related to the Basin Wall Breach diverting outgoing flow through the Basin Channel and thus reducing flow along the Basin Wall Channel. The 2006 bathymetry therefore indicates upstream (deepening) and downstream (shallowing) effects of the 1994 Basin Wall Breach. However, as noted earlier in Section 3.1, the 2006 data within the Basin itself appears to have been corrupted and this rules out analysis into the effects the breach may have had within the Basin itself. It is noted that a site-specific study using the various soundings the Port Company has taken over the years may provide evidence; however, this was beyond the scope of the present study.

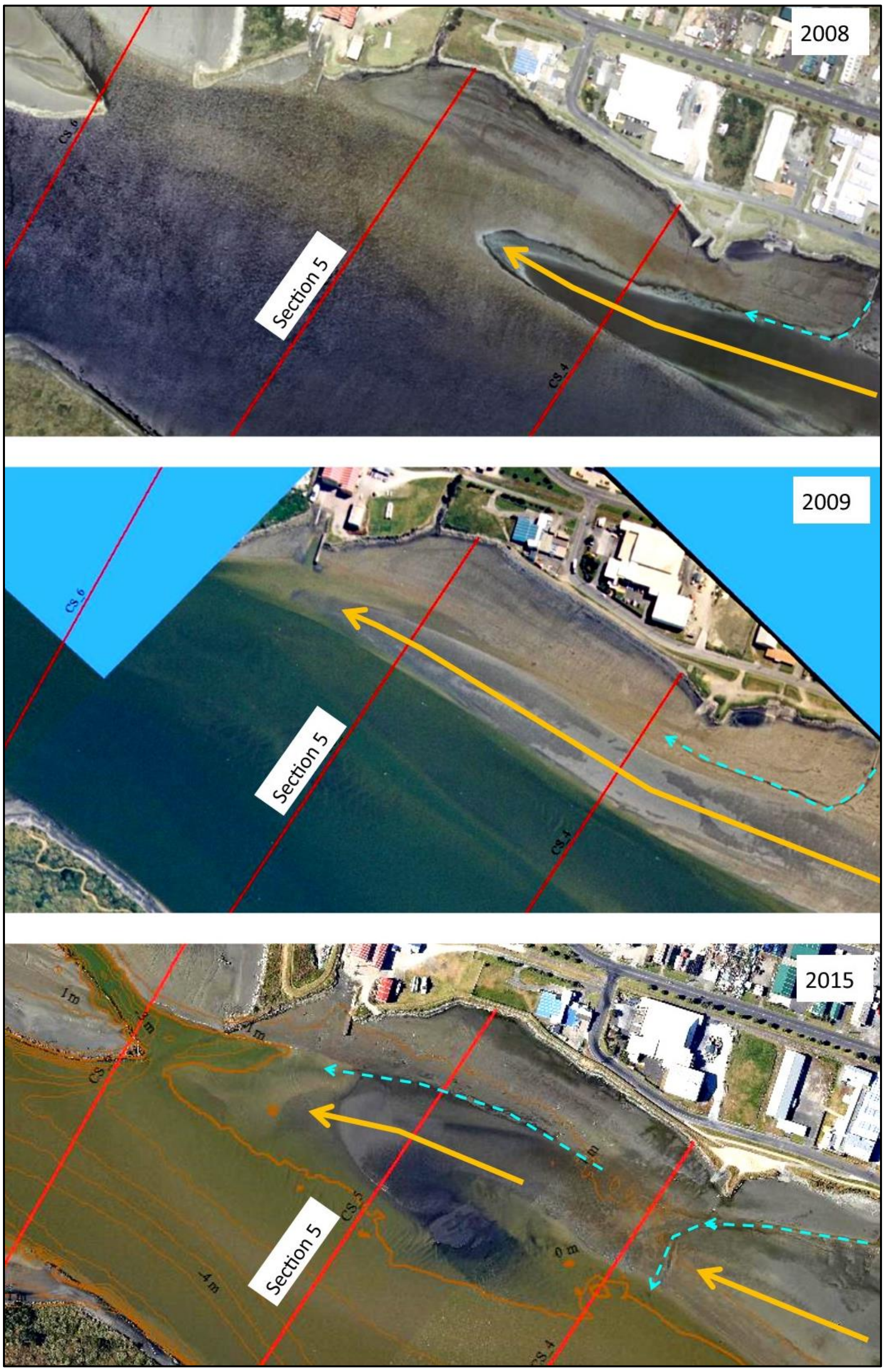


Figure 19 Downstream-migrating (yellow arrows) discrete sand bodies upstream of the Basin Wall Breach. Note the subsurface extension across the Basin Wall Breach as depicted by the zero level contour in the lower photo. Dashed light blue lines locate drainage channels deflected by such sediment movement.

The 2015 Section 6 profile in Figure 16 shows similar erosion and accumulation patterns as the sections immediately upstream: eroding left (spit) bank, accumulation about the half tide wall, erosion at the base of the main channel, accumulation along the right side of the main channel and across to the right side of the river (in this case within the Basin itself). However, of note at Section 6 is the continuing small size of the Basin Wall Channel, further indicating this is an adjustment to flow regime change resulting from the Basin Wall Breach. Finally, it is noted that the dramatic increase in size/depth of the Basin Channel, on the basin side of the Basin Wall, is probably overestimated given the already described deficiencies in the 2006 Basin data.

3.2.7 Section 7: South Mole (Wharf or Jetty) Groyne

The 1921 profile shows that the left (spit) bank adjusted quickly to the construction of the South Mole, with the right bank, at that time beyond the influence of the short Basin Wall, also shifting further right to conserve the cross-sectional area. By contrast, the 1948 profile is constrained by the Basin Wall and the left side of the channel has been scoured back toward the mole. While greater retreat may have been expected (from a cross-sectional area perspective), sediment from the spit breach had reportedly washed downstream and reached this area. The 1983 and 1993 profiles fluctuate adjacent to the Basin Wall. The left side of the channel is located no closer to the South Mole than the 1948 profile. Note this transect approximately defines the upstream end of the Tanae Bank (discussed further for Sites 8 and 9 below). As occurs at Section 6, the left side of the Main Channel has moved significantly landward, in this case landward of the now dilapidated South Mole Groyne and Wharf, i.e. the upstream extent of the Tanae Bank had been greatly reduced. This area is shaded brown in the Section 7 profile plot.

This dramatic retreat of the southern side of the Main Channel as defined by the 2006 profile was accompanied by the growth of the so-called South Mole Embayment (see Figure 20). The graph depicts high water marks taken from 1980 to 2015 aerial photos, with the most dramatic recession (45 m) occurring between 1998 and 2000. This probably resulted from increased wave penetration (and refraction) following reduction in size of the Tanae Bank which the bathymetry for Sections 7 to 9 shows to have occurred sometime between 1993 and 2006. The increase in erosion associated with the 2015 flood event (Figure 20) further suggests that this event may have had an undue influence on the 2015 bathymetry including the recent Dome behaviour at Section 3.

The 2015 Section 7 profile (Figure 16) is characterised by upper bank (South Spit) erosion (expected as the Embayment is still expanding, albeit at a reduced rate). Overall accumulation has occurred across the embayment beach to the South Mole remnants. Erosion has also occurred at the base of Main Channel. In each of these cases the changes are greater than at the upstream sections. The right side of the main channel is unchanged, providing further evidence of completed adjustment to the Basin Wall Breach. Within the Basin itself the 2006 and 2015 data show some 50,000 m^3 of sediment has accumulated; however, this value is questionable given the accuracy issue of the 2006 data in this area.

The spatial bathymetric data in Figure 17 shows a well-defined main channel located on the spit side of Section 6 which crosses to the northern side of the river at Transect 8.

Geometrically speaking, the 2015 bathymetry shows a channel saddle point between Sections 7 and 8. The 2006-2015 differencing map (erosion/deposition) shows net accumulation between the Turning Basin Wall and the South Mole in the vicinity of Section 7, behaviour that could be expected given the upstream erosion in the Main Channel characterising Sections 4-6.

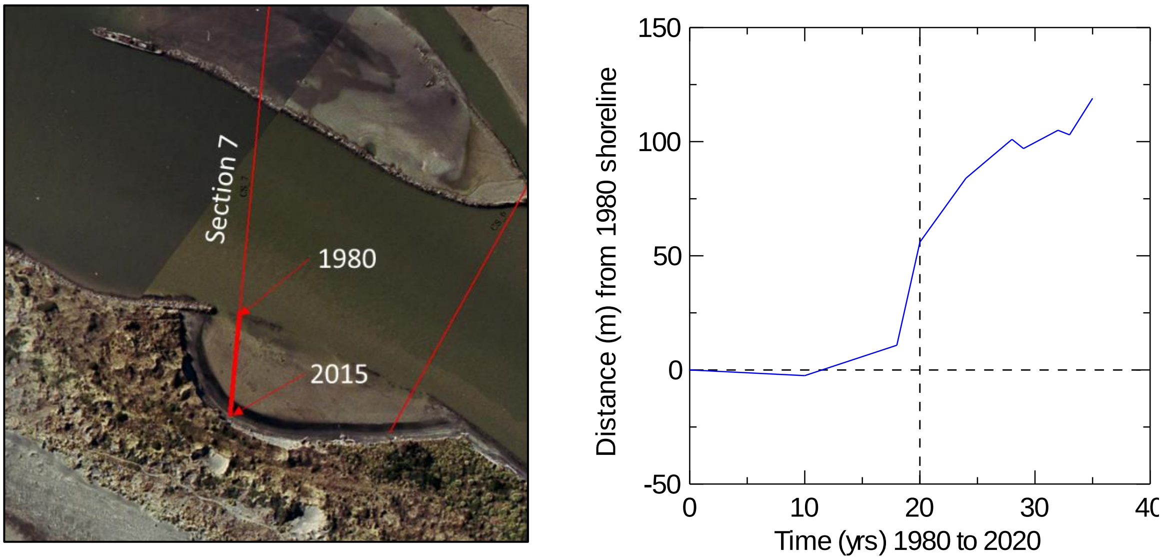


Figure 20 South Mole Embayment on Transect 7 (left), and shoreline graph for the HWM as defined on aerial and satellite image between 1980 and 2015 (right).

3.2.8 Section 8: Tanae Bank Groyne

The 1921 profile depicts the most constrained channel configuration of all samples. In 1921 the partly completed Basin Wall was still upstream of Transect 7 so probably having little, if any, effect at Transect 8. The 1948 profile depicts a considerably wider Main Channel, this being a product of the Turning Basin Wall concentrating seaward flows. In addition, in 1921 the South Mole had yet to be raised and South Beach (littoral) sand was able to enter the rivermouth system about this location. Between 1948 and 1993 the left side of the channel remained relatively stable. The 2006 channel displayed the greatest cross-sectional area of all samples and, as with Sections 6 and 7, significant retreat had occurred on the south side of the channel (shaded brown in Figure 16) – the period in which the Tanae Bank reduced in size and the Tanae Groyne disappeared. The most recent (2015) profile shows some erosion adjacent to the South Mole, deposition across the centre of the river bed and sour closer to the North Mole.

The spatial difference data in Figure 17 show the mid-channel accumulation and South Mole erosion are continuations of the Section 7 behaviour and originated immediately seaward of Section 7 respectively. The Main Channel erosion on the North Mole side is a new behaviour and possibly a morphodynamic response to the upstream accumulations, as may be the more limited erosion along the South Mole. It is also noted that the observed sediment accumulation (between the end of the Turning Basin Wall and South Mole and adjacent erosion in the vicinity of Number 1 Wharf) is consistent with the inverse deposition/erosion relationship recorded throughout the Harbour Board records and which resident engineers sought to replicate by construction of the South Mole Wharf Groyne in 1928.

3.2.9 Section 9: Fishing Platform

Data only from 1956 were available for this site (which had earlier been rivermouth rather than riverbed). While the channel running adjacent to the North Mole remained relatively stable in the earlier samples, the recession of the Tanae Bank since 1993 is again evident (shaded brown) in the 2006 profile. The 2015 profile shows a similar pattern to the Section 8 profile, but is more extreme with more extensive erosion along the South Mole, less extensive mid channel deposition and more extensive enlargement and deepening of the main channel along the North Mole.

The spatial difference data in Figure 17 indicate that the accumulation slug extending downstream had greatly reduced in extent by Section 9 and only continues another 150 m with the erosion on each side, i.e. along each of the moles, increasing in spatial extent.

3.3 Summary

The lower Whanganui River has a natural tendency to bifurcate immediately downstream of Landguard Bluff. The bathymetric charts show the Bluff deflects river/ebb tide flow to the north resulting in a channel across the northern estuary (the Northern or Balgownie Channel). Flow in the other branch (the Main Channel) is toward, and then along, the South Spit. The early marine engineers decided that from Landguard the navigation channel (Main Channel) should be directed toward the South Spit and northern flow should be minimised, so internal training walls were implemented. Once the Main Channel met the South Spit an additional training wall moved the channel toward the centre of the original estuary. This enabled it to follow a smooth curve past the wharves and towards the sea, thereby conserving the momentum of river and ebb tidal flow before jetting through the entrance and scouring the bar (NB Figure 5). Subsequent engineers built river groynes along the South Mole to further constrain the flow and minimise shoaling about the Port/Basin (NB Figure 11). They also raised both moles above MHWS and constructed an entrance groyne in 1928 (rebuilt in 1973) to increase the jetting effect (Figure 12).

Between 1930 and the mid 1940s this system appears to have operated most effectively. However, the process of creating such a system disturbed the coastal sediment budget to such an extent that consequent South Beach erosion caused the spit to be significantly breached in August 1946 and it was not permanently closed until late in 1948 (Figures 9 and 10). During this time wave action washed a substantial amount of littoral sediment directly into the estuary. River flow transported this sediment downstream with effects in the Turning Basin noted within a year or so. Opposite the breach, sand infilled the channel and formed a large dome-like shoal (the Dome), sediment from which appears to have migrated upstream under tidal flows and its effects can be detected at Section 1. The constrained Main Channel together with large Dome (Sections 2 and 3) resulted in the expansion of the Balgownie Channel (Section 2 to 5) and this became the navigation route to the town wharves. Stone was removed from the half tide training wall opposite Landguard Bluff to assist navigation and protect the Spit. Once the breach was sealed, two years of dredging using up to 4 hired dredges re-established the Main Channel, but the training wall opposite and downstream of Landguard was not reinstated. The Main Channel was closed for navigation in 1956 following further South Spit instability. While ongoing stability of the spit has been maintained, only minimal channel maintenance occurred thereafter.

Following the 1948 bathymetric survey, it was 58 years before another full survey of the Estuary was undertaken (2006) and a comparison shows a greatly reduced Dome Shoal, substantial infill of the Northern Channel and significant erosion along the length of the South Mole, i.e. loss of the Tanae Bank. Limited bathymetric data available between the 1948 and 2006 bathymetric surveys, coupled with aerial photography, gives some insight as to what happened during those 58 years.

The Tanae Bank expanded in the 1950s and the Northern Channel appeared to be infilling. However, during the mid-late 1990s a *morphodynamic shift* seems to have occurred that lead to downstream and spitward displacement of the Dome, development of the North Channel to join the Main Channel downstream of the Dome, loss of the Tanae Bank, and an erosional embayment forming at the landward end of the South Mole. This shift or *change in state* was possibly caused by a series of floods as correspondence with Port staff in the late 1990s indicated a morphological-changing flood event had occurred in 1996, and the NIWA's hydrological record shows several significant floods occurred during the mid to late 1990s which could have driven the observed changes. The flood events may also have coincided with a *threshold exceedance* caused by the somewhat inevitable ongoing increase in the size of the North Channel where it splits from the Main Channel near Landguard, i.e. that area where the training wall had not been reinstated.

A possible third influence was the Basin Wall Breach given its coincidental establishment in 1994. However, the broad analysis has indicated only localised effects, with possible channel enhancement immediately upstream and some erosion downstream.

The high resolution 2015 bathymetric data shows some significant changes and further illustrates the dynamic nature of the Estuary and Port areas. There is some evidence (noted above in Sections 3.2.3 and 3.2.7) that the June 2015 flood event may have had an undue influence on the November 2015 data, so interpretation needs to be mindful of possible bias where differences between the 2006 and 2015 data are small.

The 2006 to 2015 bathymetric change Figure (17) shows infill of the Main Channel from well above Landguard (Section 1) to the 1946 spit breach area (Section 3). Development of a Northern Channel is evident opposite Landguard (both up and downstream) and around the northern margin of the Dome Shoal where it meets the Main Channel about Sections 4 to 5. Within the Centre/Lower Estuary net sediment accumulation has been occurring, in places over 1 m deep. Net sediment accumulation is also occurring along the northern side of the Estuary and is moving toward the Basin Wall within discrete sand bodies, one of which is about to reach the Basin Wall Breach. The Main Channel from Section 3 to the Turning Basin Wall/base of the South Mole (i.e. Sections 6/7) has deepened slightly as it is forced toward the Spit by the estuary sediment accumulations. Also of particular note is the seaward extension of the Estuary accumulation beyond the Turning Basin, i.e. into the Tanae Bank area, coupled with channel deepening from near the Wharves toward the Rivermouth. The present data also shows an ongoing loss of sediment next to the South Mole (from Section 7 toward the Rivermouth).

4 RIVERMOUTH and COAST

This section describes the physical nature and changes that the secondary study area, i.e. the rivermouth and adjacent coast, has undergone during the period of port and harbour development. Since 1989 Coastal Systems Ltd have undertaken extensive monitoring of the Wanganui Coast and some materials in this Section were derived from that source.

4.1 Introduction

The Wanganui Rivermouth and adjacent coast can be classified in terms of several ratios:

(1) Wright and Coleman's (1973) discharge effectiveness index: *the river discharge to wave power ratio*. The Wanganui ratio is relatively low and comes between the wave-dominated Shoalhaven River (NSW) with little deltaic development (Wright, 1985), and the Westport Rivermouth which has distinct deltaic development (Kirk et al 1986).

(2) Boothroyds (1985) *tide or wave dominated entrance index is* somewhat similar but was developed for tidal inlets rather than freshwater dominated rivermouths. As with Wright and Coleman's index, Wanganui lies on the boundary.

(3) Brunn and Gerritsen's (1959) littoral drift bypass mechanisms.

(a) <u>inlet bypass</u> where littoral sediment moving into the inlet and is then jetted out to join the downdrift coast, or

(b) <u>bar bypass</u> where littoral sediment crosses the mouth by way of the seaward bar, with the latter mechanism being inherently difficult for navigation except for small vessels.

The mechanism applying to a particular inlet/coast depends on the ratio of *littoral drift to tidal flow*, with a higher value, as applies to the Wanganui setting, indicating the bar bypass mechanism predominates.

These classifications appear consistent with observation at Wanganui, subdued deltaic morphology being evident in Figure 21. In addition the inlet is asymmetric with the southerly offset being produced by the net NW to SE directed wind and wave energy as described in the process section (5). While the persistence of a well defined rivermouth bar supports the bar bypass mechanism, littoral sediment does move into the inlet (Macky 1988), indicating some inlet bypass occurs.

4.2 Morphology

Approaching the rivermouth along the northwest coast, the foreshore narrows and steepens toward the North Mole from ~87 m (slope = 0.033) some 3 km alongshore, down to 44 m (slope = 0.064) some 200 m from the North Mole (Shand, 1990). The pre-mole (1879) beach width was ~200 m (slope = 0.015) (Gibb, 1962), showing the moles have had a temporal as well as spatial impact. By comparison the South Beach foreshore has maintained a greater width closer to the Mole (150 m recent c.f. 160 m in 1885) which reduces to approximately 90 m some 1.5 km southeast of the rivermouth.

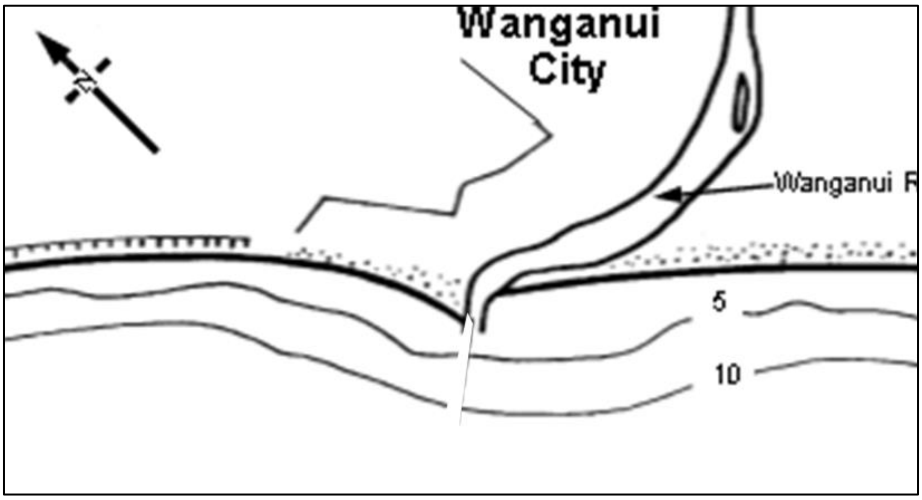


Figure 21 Extended study area with 5 and 10 m bathymetric contours shown to define the offset subaqueous delta (Navy Chart NZ 46).

Typical rivermouth morphology is illustrated in Figure 22 and rivermouth morphology and behaviour is described in detail by Shand (1990). Briefly, at times an inner bar lies between the mole ends, a main bar is always present about 150 m beyond the mole ends, and at times a more seaward outer bar is present, this latter bar typically having subdued form, i.e. little difference if any in crest and landward trough height. A well defined and relatively deep channel exists between the end of the North Mole and the main bar and a primary inlet channel originating near the end of the North Mole is typically offset to the south as it intersects the main bar. These features are evident in Figure 22, and their spatial variations, which can be considerable, are defined and illustrated in Figure 23.

The coastal morphology to the north of the rivermouth typically consists of an inner bar at times crossed by rip channels, a main bar that is typically linear in the longshore direction and separated from the inner bar by a well defined longshore trough, and at times a (subdued) outer bar exists (Shand et al., 2004). Overall, the morphological configuration is linear in the longshore direction and described as two-dimensional. These features merge with the corresponding rivermouth morphology as illustrated in Figure 22.

As Figure 22 also indicates, the morphological makeup of the southern coast is markedly different, consisting of tranverse bars and intervening channels (i.e. features orientated in a more seaward rather than alongshore direction) which often link to large embayments on the intertidal beach that can result in localised foredune erosion. Indeed, Harbour Board records include a drawing showing the South Spit shoreline characterised by embayments during the 1940s breaching (Appendix C). This three-dimensional morphology diminishes

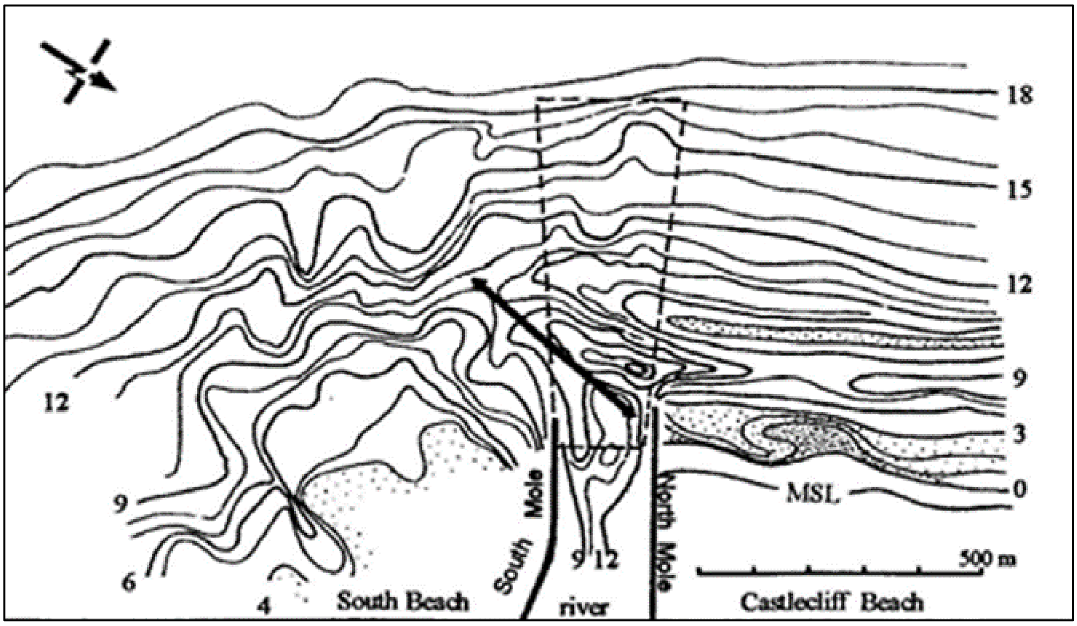


Figure 22 Bathymetry surveyed on 23-1-1968 showing typical rivermouth morphology merging with adjacent coastal morphology. The arrow located the primary inlet channel and the area enclosed by the dashed line marks the area sounded at 2 to 4 weekly intervals. The southern margin defines the southern lead, the northern margin defines the northern lead and the centerline of the dashed area defines the central lead line. Depths are in feet below Chart Datum.
Source: Shand (1990)

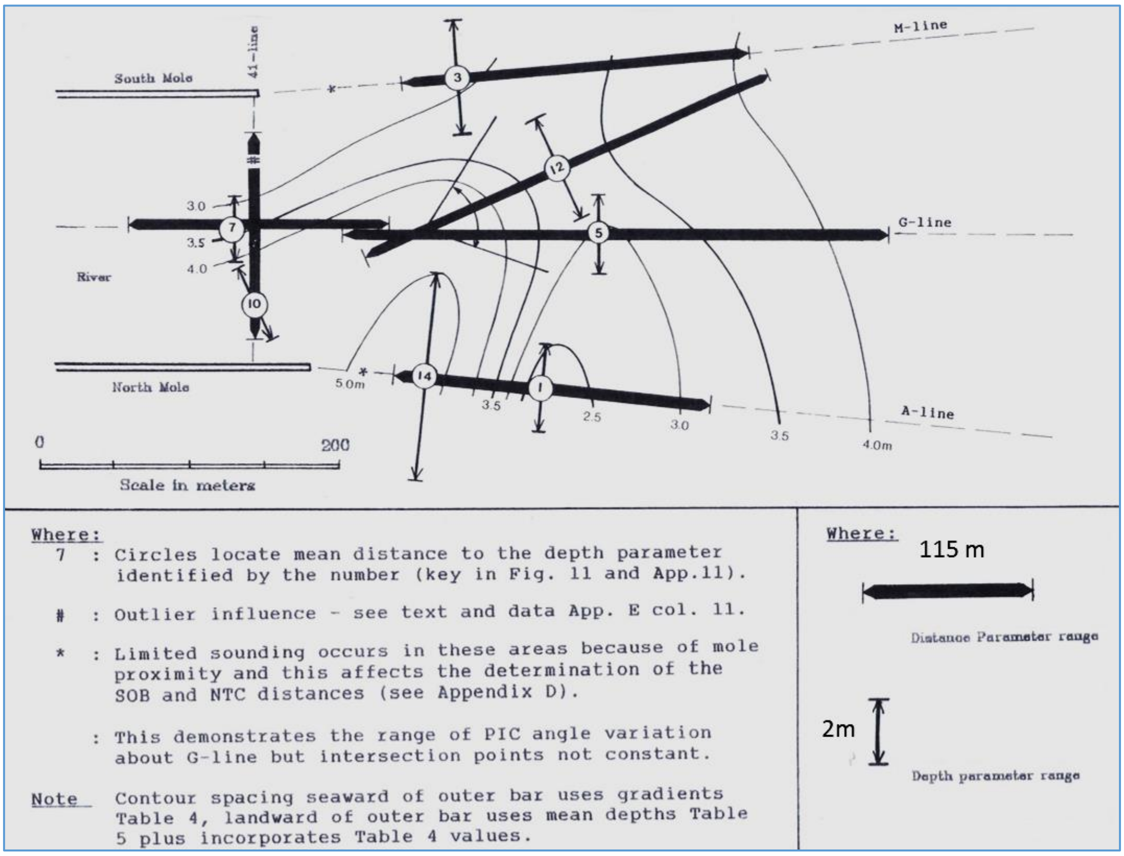


Figure 23 A qualitative model of the Wanganui Rivermouth morphological components based on statistical analysis of parameter values derived from bar chart data.
Source Shand (1990)

with increasing distance southward of the rivermouth and pre-1980 aerial photos show the change to linear form occurred from about 2 km. However, the 1800 m long marine outfall constructed in the early 1980s some 3 km south of the rivermouth, has had a significant effect upon nearshore processes (Shand et al,. 2005) such that 3D morphology now extends some 4km south of the rivermouth (see Figure 24).

Figure 24 Wanganui coast (1993) with marine outfall (approximately) located. Linear bars and troughs (2D morphology) are evident on the far side of the rivermouth (Castlecliff coast). Two-dimensionality is also evident in bottom of photo (beyond 4 km from rivermouth), with transverse (three-dimensional) morphology from the rivermouth to beyond the marine outfall.

Source CLS Archive

4.3 Longer-term behaviour

Of particular relevance is the longer-term behaviour related to the rivermouth jetties. Before describing the effects on the adjacent coasts, the regular bar soundings (surveys at two to four weekly intervals since 1926) can illustrate how the rivermouth itself has adjusted to these structures. Figure 25 depicts mean annual minimum depths along the North Mole line, i.e. the minimum depth of the main rivermouth bar. Depths reduced from approximately 6 m (relative to Chart Datum) in 1926 to a little over 4 m by 1970 and remained approximately stable thereafter. NB Chart Datum is ~1.5 m below mean sea level. However, these annual data show considerable fluctuation about the fitted equilibrium line, even more so if the annual variation shown in Figure 23 was included.

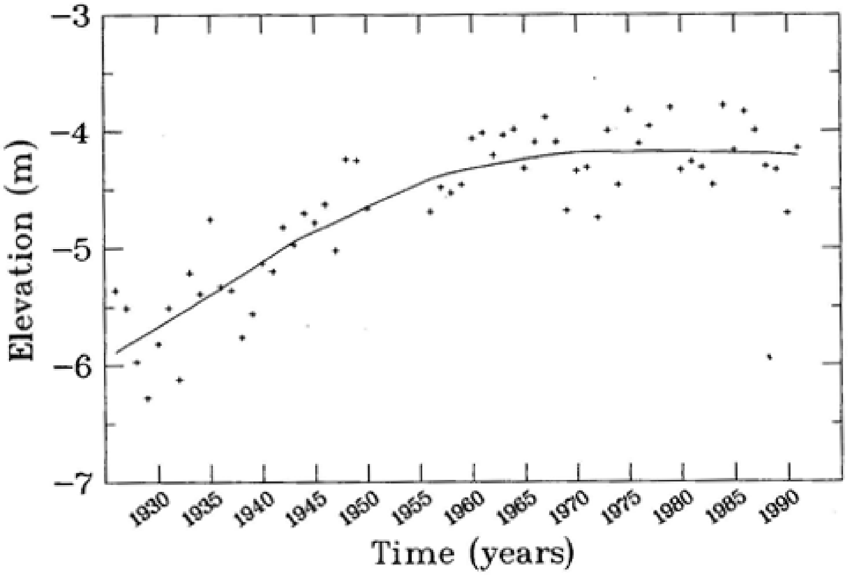


Figure 25 Mean annual minimum depths between 1926 and 1992 along the line of the North Mole extended seaward (this being the line of most regular soundings) with the fitted line being a locally weighted (Lowess) filter. Elevation datum is MSL. Source: McLean and Burgess (1974)

The effects of the mole construction phases on the adjacent coast are illustrated spatially in Figure 26 in which shorelines for 1877, 1911, 1921 and 1942 are superimpose upon the 1942 aerial photo. The white dashed arrow depicts systematic accretion on the Castlecliff side of the rivermouth and erosion along the South Spit side. Both these behaviours are attributed to disruption of the net NW to SE littoral drift which is numerically defined in Section 5.

Three phases of mole development are represented by these shorelines with the initial time period (1877 to 1911) encompassing the Stage 1 (short North Mole) effect, the 2nd time period (1911 to 1921 encompassing Stage 2 lengthening of the moles, and the third period (1921 to 1942) encompassing Stage 2 raising of the moles.

On the north coast the shoreline rapidly built outwards to the end of the short mole whereupon sediment bypass resumed. Relatively little response is evident on the southern shoreline. Lengthening the moles to 1921 and raising to low water level limited seaward migration of the northern shoreline; however, entrapment was still occurring and the southern shoreline receded with breaching reported in 1920 in the vicinity of the 1877 breach. Raising of the moles to at least HWM (1921 to 1939) caused the northern shoreline to undergo a significant seaward adjustment while further shoreline erosion occurred along the South Spit which became critically unstable during the 1940s and breached in 1946 (red shaded area in Figure 26).

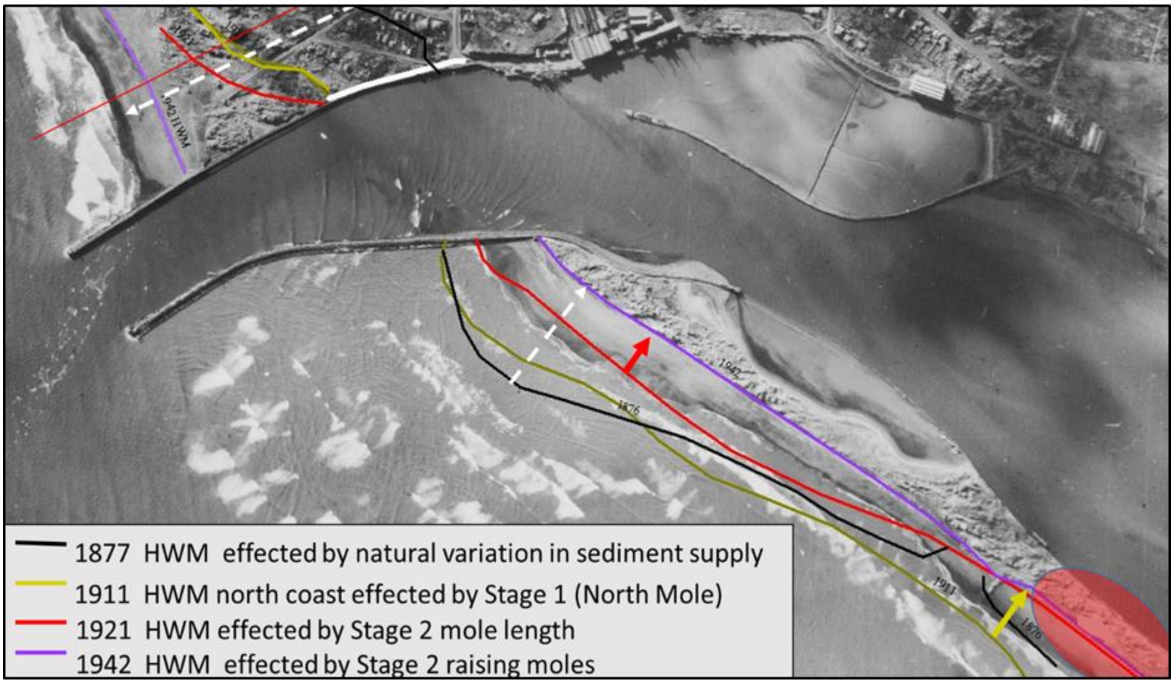


Figure 26 Shoreline change from 1877 to 1942 (underlying aerial photo). The bold white line defines the original 1885 North Mole. The dashed white arrows define net change, while the yellow arrow denotes change between 1911 and 1921, and the red arrow denotes shoreline change between 1921 and 1942. The shaded red area locates the 1946-48 spit breach.

Sources: New Zealand Aerial Mapping, LINZ, various plans and documents from WHB Archives.

Coastal shoreline behaviour up to the present is depicted graphically in Figure 27 for three discrete locations, specifically 200m north of the North Mole, 100 m south of the South Mole and 1400 m south of the South Mole which covers the site of the 1946-48 breach

Since 1942, Castlecliff Beach closer to the North Mole has continued accreting at a decreasing rate indicating almost complete littoral bypass. The two South Beach sites show overall accretion following the erosion that characterised these sites up until the 1940s. The site 100 m from the South Mole shows marked shoreline fluctuations with the 1400 m site showing smaller fluctuations and there is no particularly obvious correlation in fluctuations between the two sites. This pattern, along with the lack of fluctuations on the Castlecliff site suggests discrete sediment inputs from the river rather than variation in littoral drift. Interestingly, the timing of the two peaks (early to mid 60s and late 90s) follow periods when sediment slugs may have been released from the river (based on the results and analysis in Section 3). In particular, following the late 1950s increase in size of the Tanae Bank – a time when considerable sediment was passing through the lower river/mouth area, and secondly the onset of sediment loss in the lower river and Tanae Bank areas in the late 1990s.

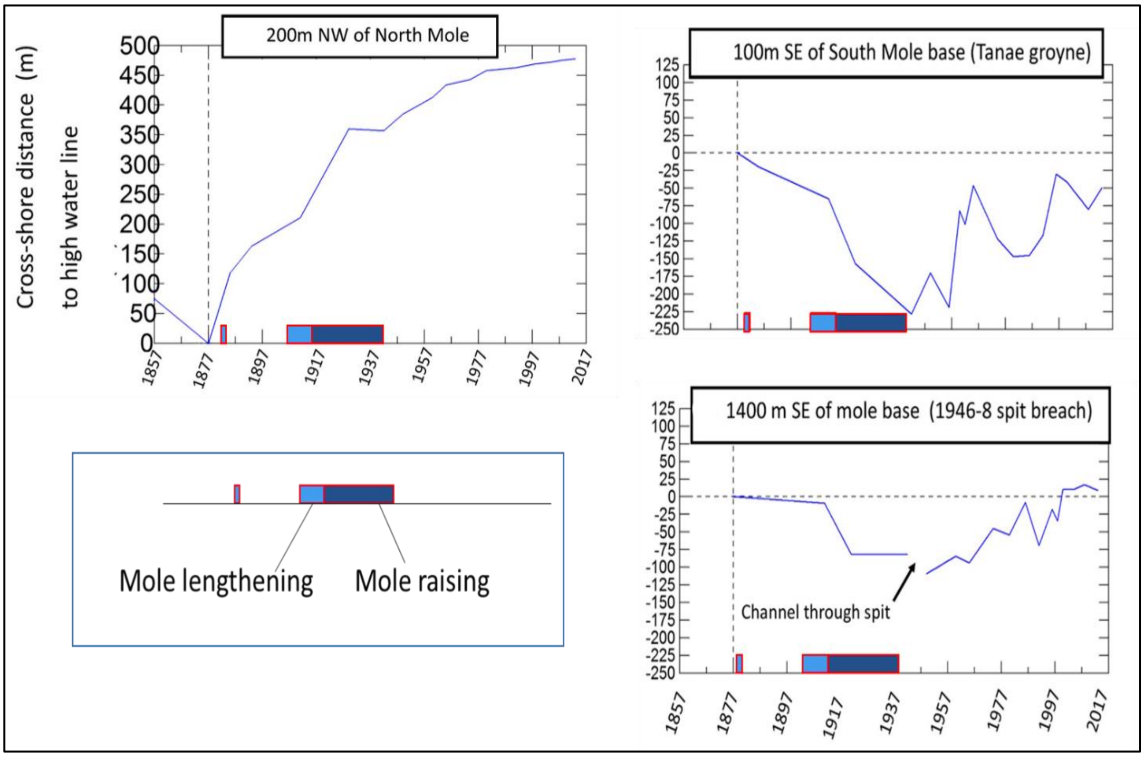


Figure 27 Coastal shoreline behavior from 1877 to the present for three coastal sites. North Mole development is marked along the baseline of each graph. Note the graphs for the South Spit sites have the same vertical axis scale so are directly comparable.

Data sources: various maps, plans and aerial photographs listed in Appendix B

4.4 Medium term behaviour

At a time scale of several years, bars and troughs on Castlecliff Beach have been shown to undergo cycles of net offshore migration, i.e. bars form on the lower beach, systematically migrate seaward then disperse (disappear) in the outer surf zone several years later, this is graphically depicted in Figure 28 for a site some 1500 m northwest of the North Mole. The system appears to be a consequence of this high energy environment, with seaward bar migration during storms exceeding landward migration during intervening periods of fair weather. As different morphological configurations (plan shape) accompany bars at different cross-section locations (discussed further in Section 4.5), there is a systematic change in configuration during the offshore bar-migration process.

The rivermouth morphological configuration is an extension of the north coast morphology (NB Figure 22), so it follows that an underlying offshore migration pattern could be expected also to occur at the rivermouth. Indeed, such a pattern has been observed in bar soundings (Shand, 2000), and this is shown in Figure 29. It also follows that the rivermouth will likely also be predisposed to certain configurations at different times, i.e. as bars migrate to different offshore locations, and this could well influence shorter-term behavioural response to the primary forcing agents (river flow, waves and tide). In addition, navigation could be impeded/facilitated during this morphological cycle.

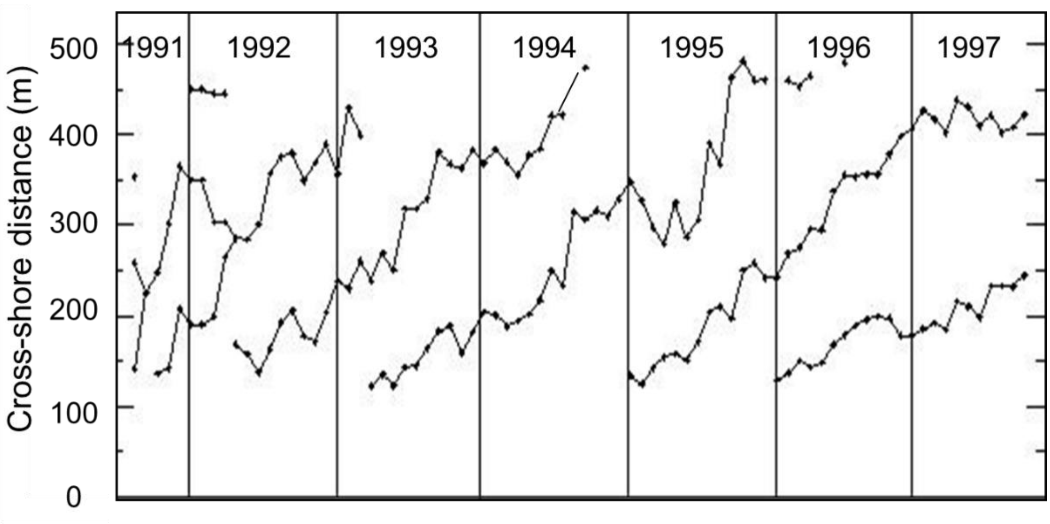


Figure 28 Sandbar-crest behavior along a cross-shore transect on Castlecliff Beach some 1500m northwest of the Whanganui Rivermouth. The origin for the vertical axis is the base of the foredune. The pattern of underlying offshore migration is clearly depicted. Source: Shand (2000)

No medium-term analysis of the South Beach has been undertaken; however, the parallel bar system which re-establishes further south is likely also characterised by net offshore bar migration, a process now recognised on many high energy sandy coasts and adjacent rivermouths worldwide.

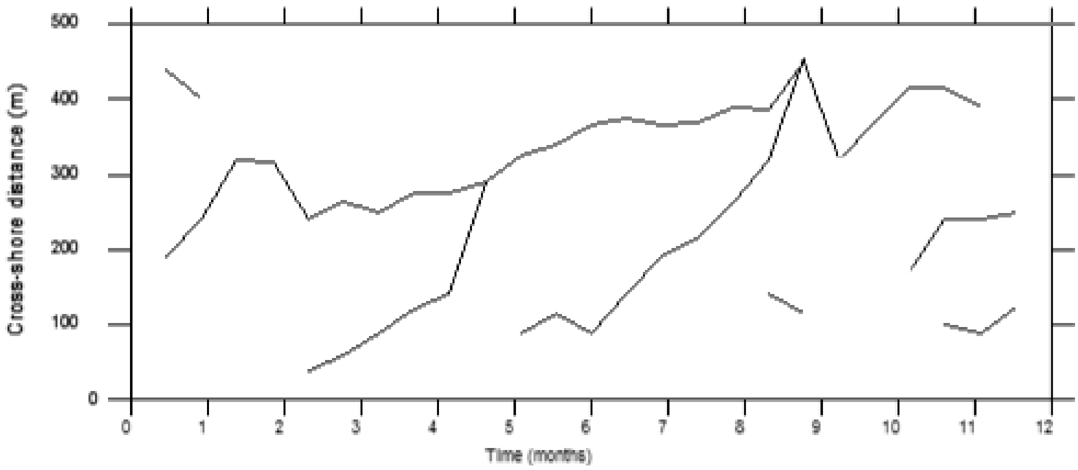


Figure 29 Rivermouth bar-crest time-series for the period 27-7-1990 to 11-7-1991. Systematic seaward migration is evident. These data are from the central lead (sounding) line and the mole ends correspond to approximately 100 m on the vertical axis. Source: Shand (2000)

4.5 Short-term system behaviour

Short-term behaviour of the multiple sand bar system at Castlecliff Beach is a product of forcing agents coupled with existing (antecedent) morphology including cross-shore location. In the mid/outer surfzone, the bar/trough morphologies are typically linear and migrate seaward under high energy conditions and landward under low energy conditions. Inner bar morphologies are more variable and as noted earlier include three-dimensional configurations (rip channels separated by transverse bars) which can be particularly resistant to environmental forcing. Inner bar configurations can also be linear, undulating and subdued (see Figure 30) and alternate between different configurations in response to environmental conditions (Shand et al., 2003, 2004).

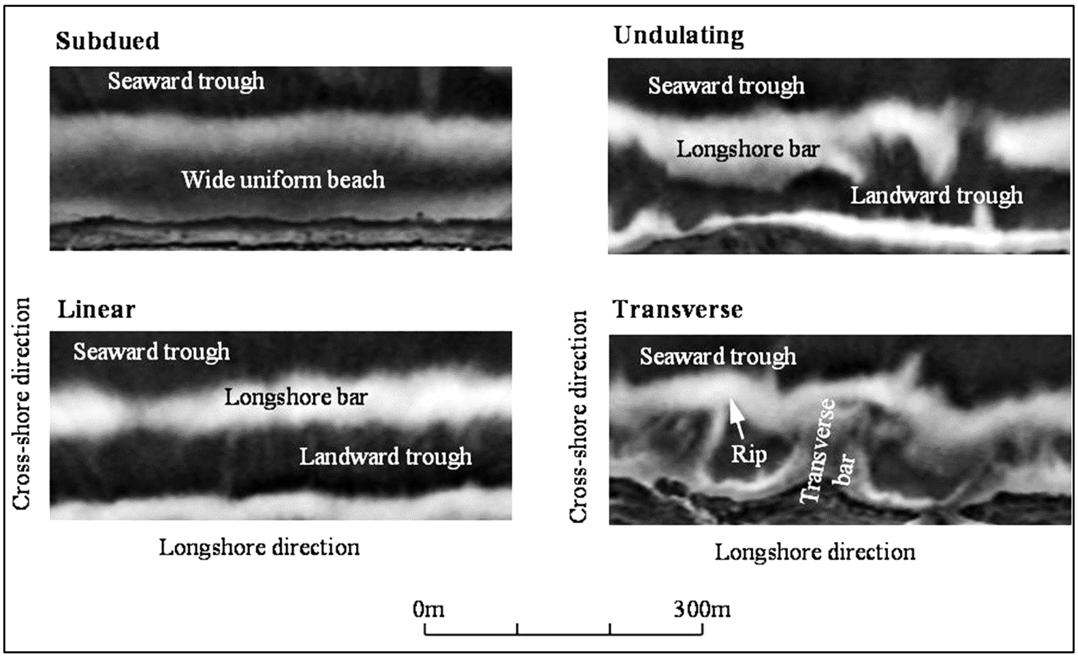


Figure 30 Inner surf zone/beach configurations on Castlcliff beach. Images are time-lapse photographs that have been digitally transformed (rectified) such that the oblique perspective has been changed to a corresponding vertical view. Source: Shand et al., 2003

Short-term rivermouth morphological behaviour was investigated by Shand (1990) using the two to four-weekly bar charts surveyed by the Harbour Board between 1981 and 1987. Using statistical component analysis the configurations were found to be bounded by two extremes related to the angle of the primary inlet channel (see Figure 31). Firstly, a low angle configuration (lower image) occurs where there is no inner bar (between the mole ends) and the main bar extends into the channel from both the northern and southern sides. And secondly, a high angle configuration occurs (upper image) where the inlet channel is offset to the southeast (upper right in diagram), the main bar extends across the mouth from the northern side (base of image) and an inner bar, attached to the South Mole, is also present. The primary inlet channel angle time-series (Figure 32) shows that the configuration oscillates between these extremes at annual, biennial and longer periodicities.

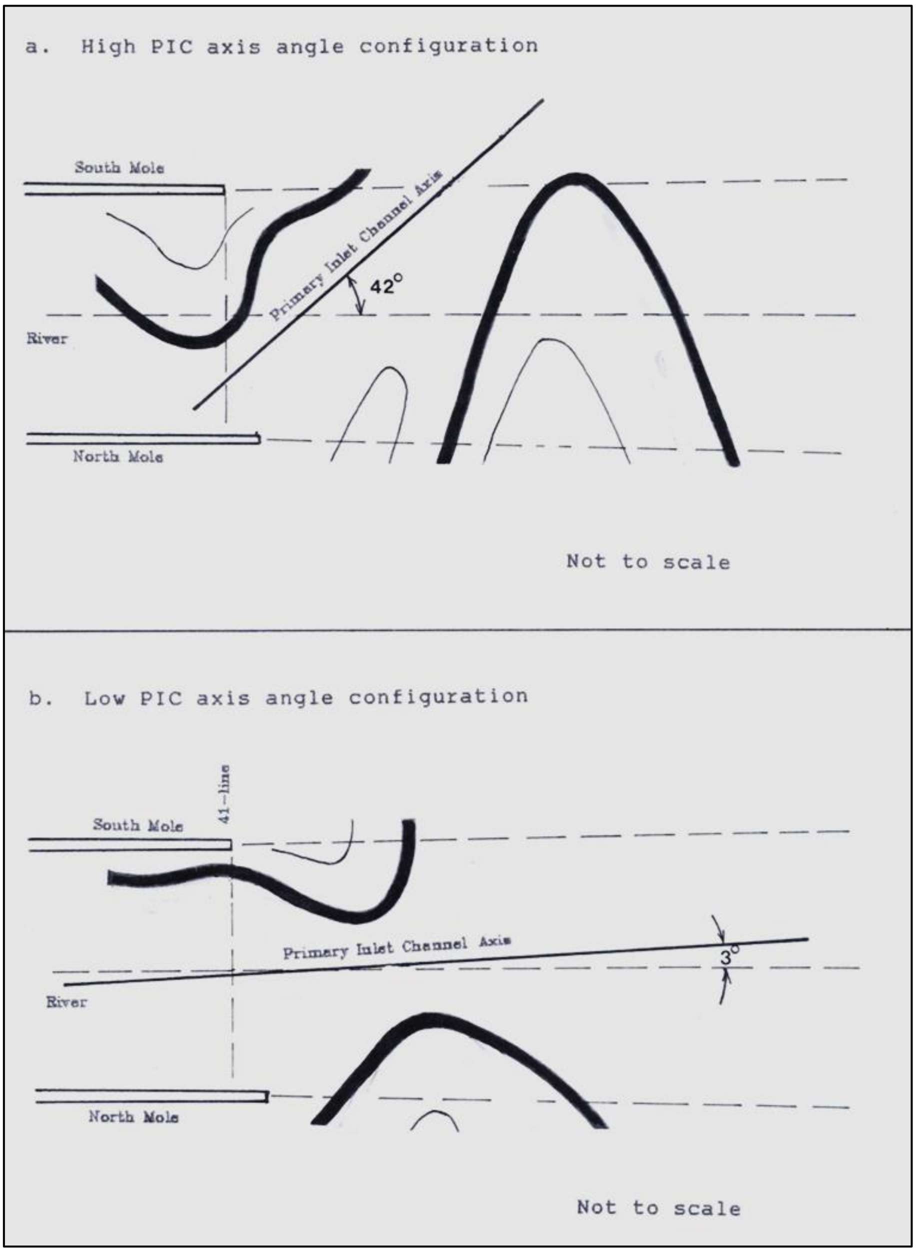


Figure 31 Morphological configuration extremes. These configurations relate to high (upper image) and low (bottom image) primary inlet channel angles (offsets). Source: Shand (1990)

Process-associations that drive the short-term behaviour were investigated by Gibb (1962) by relating cross-shore bar location and relative depth along the same three transects used by Shand (1990) in Figure 22, to wind (a surrogate for waves), river flow (synthesised from rainfall before 1957) and tide, for the periods January 1948 to December 1950 and August 1957 to March 1959. Conditions improving bar for navigation (increased depth or movement seaward) were summarised as follows (with the bar deterioration occurring under opposing conditions):

(a) Southern side of bar deepens under strong northerly blow;

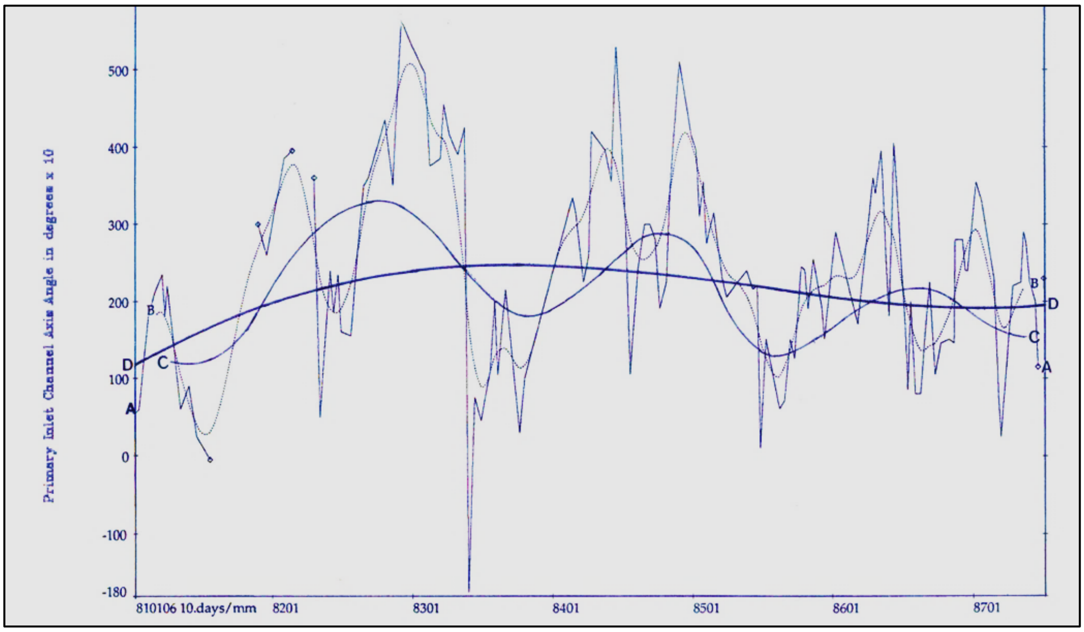


Figure 32 Primary inlet channel angle time-series (1981 to 1987) indicating oscillations at a range of temporal scales. Source: Shand (1990)

(b) Northern side of bar deepens under strong southerly blow;

(c) Frequent changes in wind direction are most favourable;

(d) Spring tides at times of low river flow are favourable, but winds can have over-riding influence;

(e) Spates of river flow > 850 cumecs are favourable;

(f) Prolonged medium to high river flows (>550 cumecs) are favourable as long as accompanied by wave action capable of preventing deposition

The findings of Shand (1990) supported these associations. For example, westerly wave/wind approach will carry bar sediment along the bar and into the channel, i.e. extend the bar across the channel increasing channel offset, with the flow then forced against/eroding the bar on the southern side; this produces the high primary inlet channel-angle configuration. Conversely, prolonged higher river flow will straighten and deepen the channel; this produces the low primary inlet channel configuration.

Gibb also noted that the flow of 850 cumecs (which was associated with bar scour) approximates the spring tide discharge and the spring tide discharge was notably more beneficial (bar improving) that lesser tide ranges. Gibb thus speculated that 850 cumecs approximates the critical flow for effective bar scour.

Macky (1991) tested Gibb's findings using 1987 Waverider data, with temporal cover extending through 1988 by synthesising wave data from a wave/wind correlation model) and corresponding river flow data. In general, Macky also found that floods erode the bar (unless accompanied by low wave energy), while waves build it up on the updrift side.

Most recently, Williams (2007) analysed the association between flood flows and corresponding tidal conditions and port/ rivermouth bed levels to determine seaward boundary conditions for the Horizons Regional Council lower river flood (MIKE11) model. The investigation used the largest 9 floods between 1998 and 2004 and three of these floods fitted each of three tidal states: large decreasing range, large increasing range, and small steady range. Williams found that the water level between the Port and mole ends varied for tide range but was relatively constant for different sized floods, a significant result for upriver flood modelling. Beyond the moles, the sea-bed appeared to respond differently to floods under different tidal conditions (range and change). However, it is noted that some of the morphological change Williams attributes to water level appears to be consistent with the wind and wave effects noted in the earlier research.

5 PROCESS INFORMATION

5.1 Introduction

Early harbour developments were accompanied by hydrodynamic measurements and analyses. Consulting Marine Engineers Barr and Oliver (1887 to 1885) and later Leslie Reynolds (1895 to 1905) placed particular emphasis on tidal characteristics (a primary navigation control) and on the tidal prism, designers being mindful of the need to maintain tidal volume to maximise scour on the bar at the rivermouth. In addition, wind and waves conditions also influenced sedimentation and navigation so were incorporated into the design of the rivermouth moles. However, it was not until the work of Sir Alexander Gibb and Partners investigating possible environmental consequences of the proposed Tongariro Power Development Scheme (TPD) in the late 1950s/early 1960s, that sedimentation and hydrodynamics in the lower river and coast were subject to more thorough measurement and analysis.

The first wave data were measured in the late 1960s using a theodolite, with these data being subsequently used to estimate littoral drift and rivermouth sediment volume. Wave data was collected by the DSIR, using a Waverider buoy between January 1986 and February 1987. In the 1990s wave data were collected daily over a 6 year period, again using shore-based survey equipment; these data were used to further estimate littoral drift and also for use in coastal process studies. Most recently (since 1997) MetOceans Solutions Limited began predicting (at 3 hourly intervals) wave and weather conditions around the New Zealand coast including Wanganui, using a numerical model. This section now summarises available process information to define the sediment, tide, river hydrology, waves, and wind regimes.

5.2 Sediment regime

5.2.1 Fluvial sediment and basin sedimentation

Fluvial sediment charateristics have been described in Tonkin and Taylor (1978). Briefly, the Whanganui River headwaters are located in the mountains of the Central Plateau, an area of some 730 km^2 which firstly provides the river system's bedload, estimated by Thompson (1988) to be 114 kt/yr plus some 13,000 m3/yr diverted through the TPD. By contrast, the majority of the catchment (6,390 km^2) drains steep hill country which provides finer (suspended) sediment. Suspended sediment measurements carried out by the Ministry of Works between 1956 and 1967 derived a suspended sediment rating curve for Paetawa which gives an annual (total) sediment discharge of 486 t/km^2 (Tonkin and Taylor 1978). Assuming a bulk density of 2 $tonnes/m^3$, this equates to 1.7×10^6 m^3 sediment discharged annually into the Tasman sea.

Tonkin and Taylor (1978) note that although the Wanganui River appears to carry a high suspended sediment load due to the fine colloidal particles, this load is at least 10 times lower than the Poverty Bay catchments; this was put down to a more mature catchment, greater tectonic stability, lower rainfall intensities and less development. None-the-less, post flood deposition in the Wanganui Estuary and Port areas have been an underlying problem for past and present port engineers and managers. A permanent dredge had to operate in the basin during the years of Harbour Board administration to ensure wharf and turning basin depths adequate for full port operation. Dredging (bucket) volumes for the period 1951 to 1983 are provided in Appendix C and averaged 80,533 m^3/yr (40,917 to 108,600 m^3/yr).

During the late 1980s, Port Company Ocean Terminals Ltd tried to further develop the port by leasing a suction dredge and spending nearly $1M improving bar and basin depths (WDC, 2004), but this was not sustainable. A Blue Water Port (diversion of the river through South Spit and using artificial sediment bypassing across the moles) was investigated; however, investors could not be found for the $100 m capital development and $1M of annual dredging. The company then breached the Basin Wall in 1994 in an effort to reduce dredging (Appendix F).

City Port Ltd took over the lease in 2004 and pursued a minimalist approach to dredging, allowing for only the depth required for existing shipping and boating, i.e. there is no Turning Basin. The dredge was a hydraulic excavator operating from a barge or directly from the wharves. The WDC (2004) report stated that having private companies operate the port had saved the community $6 m between 1988 and 2004. However, the matter of deferred maintenance remained contentious and the Wanganui District Council itself took over the lease in 2010.

5.2.2 Textural characteristics

The textural characteristics of surface sediment for the 6 kms of coast to the north of the Rivermouth were identified in Shand (2000) after analysing samples collected along the high tide line, and also along cross-shore transects extending some 3000 m offshore (to ~18.5 m below MSL) located 500 m and 3000 m northwest of the rivermouth.

While the beach sediments comprise fine sand (2 to 2.5 phi), the sediment coarsens toward the river where values of ~1.5 phi were obtained. This coarsening appears to be a local phenomenon that has been previously identified by Burgess (1971), Gibb (1979) and Lithgow (1986) and thought to be related to stronger current closer to the rivermouth removing the finer fraction.

Both sets of cross-shore samples have textural characteristics that change with increasing distance offshore. Sediment coarsened from high tide to low tide (2.3 to 0.9 phi). Further seaward the sediment fined across bars and coarsened within troughs before stabilising at ~3 phi (fine to very fine sand) beyond about 1000 m (~ 9 m below MSL). Sorting values were higher near the rivermouth indicating the (expected) fluvial influence.

South Beach sediments were shown by Burgess (1971) to be somewhat finer than on the northern side of the river (2 phi c.f. 1.8) and this fitted within a regional trend of fining toward the southeast

Burgess (1971) analysed 18 samples of rivermouth sediment. The range in sediment size was similar to that found in the surf zone samples. However, sorting values varied dramatically and ranged between 0.28 and 1.7 phi standard deviations. Such variation is expected in an area characterised by wave, tidal and fluvial interaction.

The southeasterly directed net littoral drift, the regional fining of sediment to the southeast and the reduction of heavy minerals southeast of the rivermouth (see below) indicate most river sediment is deposited southeast of the mouth.

Willett (1959) analysed sediment samples collected within the lower Wanganui River and size analysis found river sand to be marginally finer than coastal sand.

5.2.3 Mineralogy

The mineralogy of beach (foreshore) sediments in the Wanganui region has been reported by Fleming (1953), Gibb (1979) and Lithgow (1986). Gibb's (1979) results are reproduced as Figure 33. These studies all found that sediments to the west of the Wanganui Rivermouth were dominated by heavy minerals (60 to 80%) such as augite and opaques (mainly titanomagnetite) which originate from the Egmont Volcanic Region. The percentage of magnetic minerals declines dramatically at Wanganui. Fleming (1953) attributed this reduction firstly to increased distance from the source, and secondly to dilution caused by the high concentration of non-magnetic minerals entering the beach system via the Wanganui River. While most hypersthene is carried to the southeast from the Wanganui River, some deposition occurs along the northwest coast indicating that counter to expected net littoral currents may occur at times.

5.2.4 Littoral drift

The sediment distributions together with the coastal wind and wave regime indicate NW to SE net littoral drift can be expected. Shoreline longshore currents were measured by Patterson (1992), who found that the median value of 0.43 m/s and maximum value of 1.1 m/s. Offshore current meter measurements (10 m depth) reached 77 m/s and averaged 0.15 m/s, with 96% of the kinetic energy associated with the longshore component (Bell, 1990, 1991). Littoral drift estimates for the Wanganui Coast are summarised in Table 1 and show NW to SE rates vary between 142,000 and 597,000 m^3/yr, while SE to NW rates vary between 60,000 and 277,000 m^3/yr.

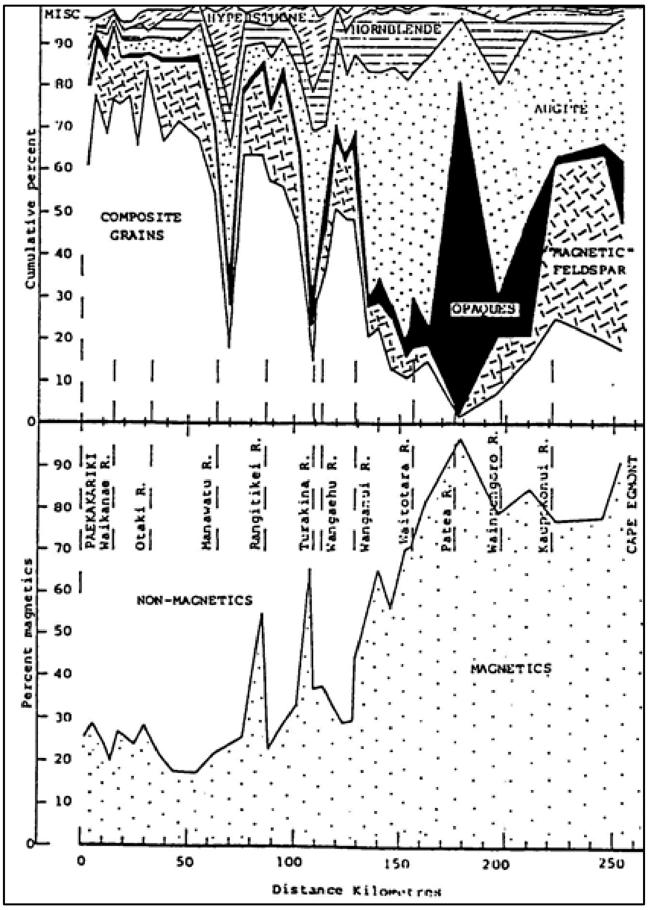


Figure 33 Longshore dispersal pattern of minerals in beach sand between Cape Egmont and Paekakariki. Source Gibb (1979)

Table 1 Littoral drift estimates for the Wanganui Coast. Source Shand (2000)

Method of estimation	Sampling interval	NW → SE (m^3)	SE → NW (m^3)	Gross (m^3)	Net (NW → SE) (m^3)
Sediment accumulation *	1885-95	275 000			
Sediment accumulation *	1921-29	142 000			
Calculation: wave energy #	1968/9	306 000	122 000	428 000	184 000
Calculation: wind & wave @	1968/9	355 000	60 000	415 000	295 000
Calculation: wind & wave @	1986/7	597 000	277 000	874 000	320 000

Where NW → SE indicates sediment transport directed from northwest to southeast. Information sources given below and further details are provided in text.

Sources: * Reported in Wallingford Research Station (1967),
Burgess (1971),
@ Patterson (1992).

5.3 Tidal regime and level datum

Chart Datum (CD) is used as the elevation datum both for New Zealand Hydrographic Charts and for charts prepared by the former Wanganui Harbour Board and later the Port of Wanganui. Chart datum refers to the lowest astronomical tide and early values tend to be higher as the record was shorter.

Gibb (1962) provides the following information on level datums:

- Earlier Admiralty Chart Datum is 95.99 feet to Wanganui City Datum.
- The 1958 Admiralty chart (NZ4612) datum is 95.91 feet to Wanganui City Datum
- Gibb (1962) used Wanganui Harbour Board tide gauge zero = 95.29 feet to Wanganui City Datum
- Wanganui City Datum Chart (1973) [copy in Appendix C] provides the following conversions:
  Moturiki Datum (1953) MSL = CD +1.5 m,
  Wanganui City Datum = CD+101.44 m
  Wanganui City Datum = Moturiki Datum +100.06 m
  Note there are inconsistencies with the datum levels given on the Wanganui District Council web site and an associated recommendation is included in Section 6.

Tides at Wanganui are semi-diurnal with a spring tidal range of 2.1 m (6.9 feet) and a neap range of 0.9 m (3 feet), see Table 2.

Table 2 Tide levels for the Port of Wanganui from LINZ 2014[1], in terms of Wanganui Chart Datum (lowest astronomical tide) and Moturiki Vertical Datum 1953 (MSL). In parenthesis are the parameter values provided in the 1973 Wanganui City Datum Chart (Appendix C)

Tidal state	Chart Datum (m)	Moturiki Vertical Datum (m)
Mean High Water Springs (MHWS)	2.8 (2.74)	1.3
Mean High Water Neaps (MHWN)	2.2 (1.98)	0.7
Mean Sea Level (MSL) approximated	1.75 (1.55)	0.25
Mean Low Water Neaps (MLWN)	1.3 (1.16)	-0.2
Mean Low Water Springs (MLWS)	0.7 (0.34)	-0.8
Chart Datum (CD)	0	-1.5

1. Sourced from Land Information New Zealand, Crown Copyright Reserved.
http://www.linz.govt.nz/sea/nautical-information/new-zealand-nautical-almanac-nz-204

Numerical modelling of oceanic circulation in the northern region of the western embayment of Cook Strait by Bowman et al. (1982), showed that surface wind stress and topography could cause substantial deviations in the time of high and low water, in the amplitude of the tide, and in the speed of the tidal stream.

The tidal stream is orientated parallel to the coast and sets to the northwest during the flooding tide and to the southeast during the ebbing tide (Ministry of Transport, 1989) at an approximate speed of 0.2 m/s (Williams, 1985). Between the wharves and mole ends tidal currents (under low river flow) reach 0.8 m/s during spring ebb tides and 0.75 m/s during spring flood tides (Shand, 2005). Furthermore, Macky (1991) reported peak tidal flows at the mouth of ~1000 m^3/s for the spring range and ~300 m^3/s for the neap range.

The tidal reach (upstream distance affected by the tidal-induced elevation change) is 37 km (Gibb, 1962). The river system has a salt wedge with a slope of about 12 degrees and intrudes upriver as the tide rises. Under spring tide range and 1.6 x mean river flow the wedge penetrated 10.6 km compared with 1.9 km under neap tide and 2.2 x mean river flow (Gibb, 1962). Flood tide gauging (Gibb, 1962) showed maximum bottom velocity of 0.45 m/s with a tidal range of 1.8 m and flocculation was not observed. Salinity measurements by Dahm (1988) found that under very low flows (37 to 61 m^3/s) the wedge penetrated over 11 km under a spring tide range and about 5-8 km under neap range. By contrast, Dahm (1988) did find evidence of flocculation based on sediment size analysis.

Historical tidal compartment measurements were undertaken in 1876, 1895, 1921 and also on 10 occasions between 1958-59 and all results are presented in Gibb (1962). The results of the 1958-59 measurements ranged between 3 and 10.2 x 10^6 m^3 (mean 5.2) under different tidal range and river flow conditions and are summarised in Table 2. Gibb noted that the 1958 volume approximated the 1877 volume, with values in 1895 and 1921 being somewhat lower, and speculated that this may relate to effectiveness of the river half tide training walls which were in optimal condition during the intermediate years.

Table 2 Tidal compartment volumes and relative control conditions

	Volume (10^6 m^3)	Relative tide range	Relative river flow (m^3/s)
Minimum	3.0	1.2 * neap range	0.5 * mean flow
Maximum	10.2	1.1 * spring range	0.44 * mean flow
Mean	5.2	1.4 * neap range 0.6 * spring range	0.95 * mean flow

Source Gibb 1962, Plate VI.7

5.4 River flow

The Wanganui River is 305 km long and has a catchment area of 7120 km^2. Mean river flow at Paetawa (above the tidal reach and with a catchment area of 6643 km^2), is 224 to 231 m^3/s and the annual flood about 10 times higher. Flow parameter values, including percentage loss through the TPD diversion, are summarised in Table 3.

The 730 km^2 mountain headwaters sustains base flow during drier periods and this is evident in the flow duration curves (Figure 34). The pre/post-TPD curves demonstrate the effect in terms of reduced time of equivalent flow. The effects on the lower river have been discussed qualitatively, in terms of increased sedimentation associated with the longer times associated with flood (incoming) tidal flow, in Dahm (1988).

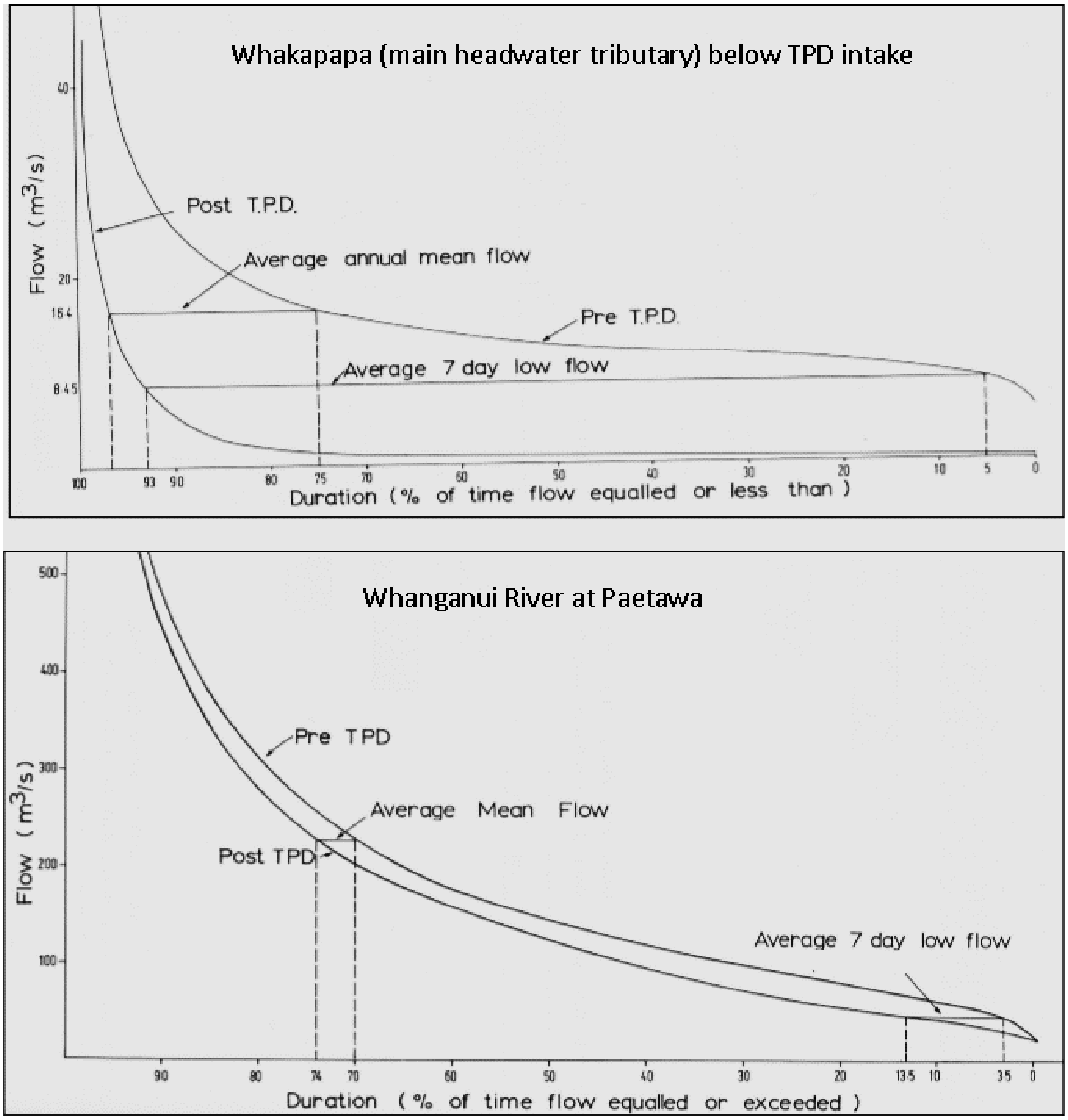


Figure 34 Flow duration curves for Whakapapa, the main mountain tributary, are shown in the upper graph, and for Paetawa above the tidal reach in the lower graph. Pre and post TPD curves are superimposed with parameter differences marked by horizontal lines.
Source: Shand (1988)

Fall (change in water surface elevation) and associated slope between the Town Wharf and Castlecliff Wharves, a distance of 4.26 miles (6.8 km) are summarised in Table 4. The fall is a product of tidal elevation, river discharge, coastal surge (effects of barometric pressure, wind setup, wave setup) and the condition of river training walls. The values in Table 4 were made at LWOST unless otherwise noted. Low tide values range between 0.49 m and 1.07 m with the former before training walls were constructed and the latter when the walls were in good repair. Higher flow and higher tide increase the fall to 1.6 m.

Table 3 Flow parameter values for Paetawa. Parenthesis indicates reduction caused by the TPD headwater diversion.

Parameters (m^3/s)	1957 to 1978[1]	1993 to 2004[2]
Mean annual low flow	43.5 (30%)	41.3
Mean annual flow	224 (8%)	231
Mean annual flood flow	2400 (2%)	2684
10 yr return period flow	3600	
50 yr return period flow	4800	
100 yr return period flow	5300 (1%)	

1. Tonkin and Taylor (1978) 2. Horizons Regional Council

Table 4 Surface fall between Town Whaves and Castlecliff Wharves.

Date	Fall (m)	Slope	Flow (m^3/s)	Comment
1877	0.49	1 in 13,898		No training walls
1885	1.07	1 in 6,355		Walls good repair
1895	0.91	1 in 7,473	147	Walls breached
1905	1.48	1 in 4,595	6,201	High tide
1921	0.74	1 in 9,189		
1954	1.60	1 in 4,250	4,956	
1958	0.67	1in 10,149	99	
1958	0.73	1 in 9,315	142	

Source: Adapted from Gibb (1962) Plate VI 2, Column 20

5.5 Wave regime

Four sources of wave data are available at Wanganui, of which three have been obtained and analysed with results for wave height summarised in Table 5 and results for wave period in Table 6.

1) Between 27 September 1968 and 31 December 1969 the Wanganui Harbour Board collected daily wind and wave data from a site approximately 2.5 km northwest of the Wanganui Rivermouth. The purpose of this monitoring was to correlate process data with morphological change at the river entrance with results being reported in McLean and Burgess (1969) and Burgess (1971). The maximum wave height was determined using a buoy moored in the outer surf zone and a nearby cliff-top theodolite.

2) Between 31 January 1986 and 25 February 1987 a Datawell non-directional waverider buoy was deployed off the Wanganui coast by the Ministry of Works and Development to provide data for future coastal investigations. The buoy was located 9 km offshore at a depth of 30 m. The water surface was sampled at four Hertz (Hz) for nine minute durations every three hours. These data were analysed for deepwater wave height characteristics and reported in Macky et al. (1988).

3) Daily wave conditions were recorded between 27 July 1989 and 10 January 1998 by shoreline observations by aligning a survey staff with the breaking wave crest and horizon beyond (described in Patterson and Blair (1983), and Patterson (1985). These (line-of-sight) data were initially obtained for port development design purposes (Sections 2.6 and 5.2.1), and later to correlate wave parameter values with morphological change along the Wanganui coast, with results reported in Shand (2000).

Mean wave heights varied between 1.25 m and 1.45 m with the 1% AEP (100 year return period) varying between 2.9 m and 3.23 m. Differences in wave height between the three data sources must be expected due to different record lengths, different sampling methods and different monitoring locations. Mean wave period ranged between 6 and 10.3 seconds, minimum = 3.5 seconds and maximum = 19.2 seconds. The Waverrider data spectra showed bimodality with the mean sea[1] period at ~5 seconds, the mean swell[2] period at ~14 secs, and a well-defined cutoff at 10 to 11 seconds. The width of the wave spectra indicates most waves were generated within 320 km. In addition to wave height and period, the wave approach direction was included in the Harbour Board data-set, with results showing 43% approach from the west, 35% from shore-normal and 22% from the east.

1. Sea: waves generated by local wind and characterised by shorter periods.
2. Swell: waves generated by distant weather systems and characterised by longer periods.

Table 5 Significant wave height statistics for the Wanganui Coast.
Source Shand (2000)

Name of data-set	Harbour Board	Waverider	Line-of-sight
Collector	Wanganui Harbour Board	Ministry of Works and Development	Author
Method	Theodolite & buoy	Waverider buoy	Staff & horizon
Collection dates	27.9.68 to 31.12.69	31.1.86 to 25.2.87	27.7.89 to 10.1.98
Duration (days)	460	420*	3439
Mean wave height (m)	1.3	1.25	1.45
50% exceedence (m)	1.2	1.15	1.35
10% exceedence (m)	2.0	2.15	2.32
5% exceedence (m)	2.2	2.45	2.64
1% exceedence (m)	2.9	3.2	3.23

Table 6 Available wave period statistics for the Wanganui Coast.
Source Shand (2000)

Name of data	Harbour Board	Waverider	Waverider	Line-of-sight
Method of period determination	Breakpoint counting	Spectral peak	Zero upcrossing	Breakpoint counting
Duration (days)	460	420*	420*	2053
Minimum (seconds)	5	3.5	4	3.5
Maximum (seconds)	18	19	11	19.2
Mean (seconds)	9	10.1	6	10.3

* 37% records lost or unrecoverable

The fourth source of wave data comes from MetOcean Solutions Limited's numerical (SWAN) model which inputs data from the National Oceanographic and Atmospheric Administration's (NOAA) NewWaveWatch3 (NWW3) wave model, a third generation ocean wave propagation model. NWW3 uses wind fields to solve the spectral action density balance equation for wave number-direction spectra. MetOcean output includes significant wave heights (average of highest 1/3 of waves), period and direction at specified grid locations at 3 hourly intervals. An example of a graphed forecast for Wanganui is shown in Figure 35. MetOcean provides a free forecast and hazard warning service to local authorities. In addition to its prediction services, MetOcean produce archived hindcast data and modeled extreme event statistics (mean heights, return periods etc) for use in hazard assessments and design work. Experience in the South Taranaki Bight suggests the model output needs to be calibrated against instrumented data (wind and wave) to derive dependable results. This is especially so at Wanganui where the southerly wave component tends to be underestimated.

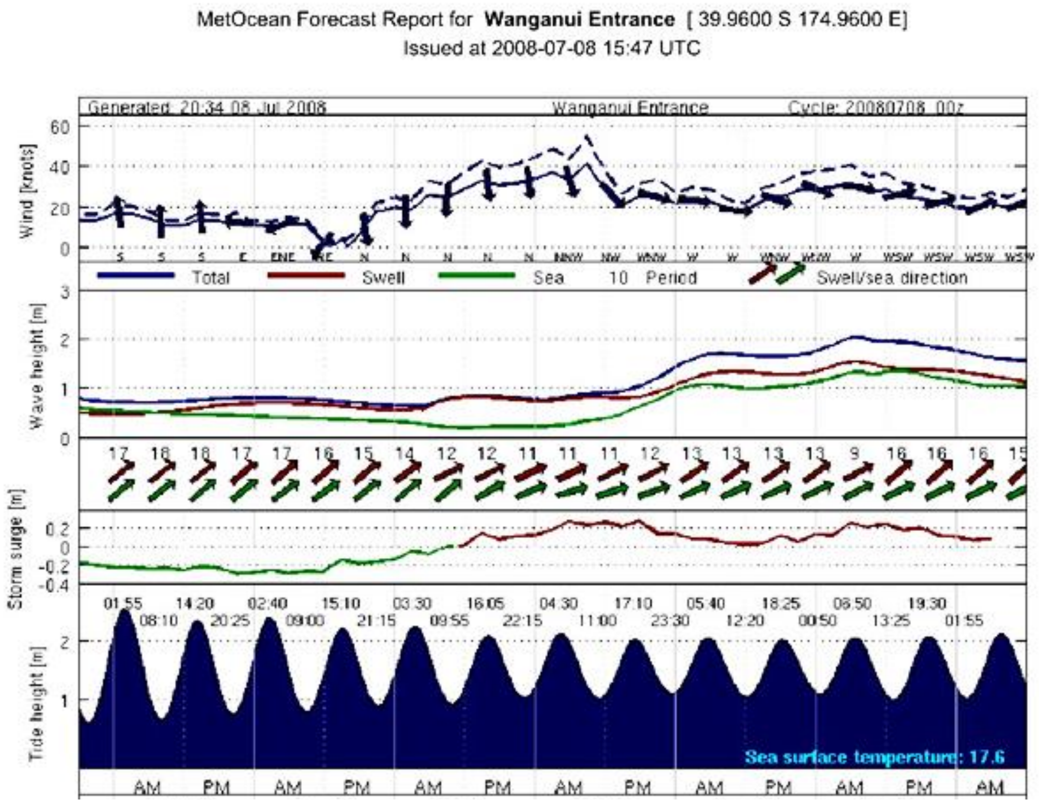


Figure 35 An example of a 7 day forecast for the Wanganui coast from the MetOcean Solutions Ltd numerical model.

Source: www.metocean.co.nz/

5.6 Wind regime

As indicated in the previous sections, wind on the Wanganui coast plays a significant role in wave generation and in driving coast currents.

A wind rose with eight directional sectors for Wanganui Airport data is shown in Figure 36. Winds with speeds > 20 knots (10.4 m/s) approach mainly from west to northwest and to a lesser extent from the south. Seasonality in the wind climate is well recognised with winds in excess of 20 knots occurring most frequently in the spring; however, strong winds can occur at any time of year.

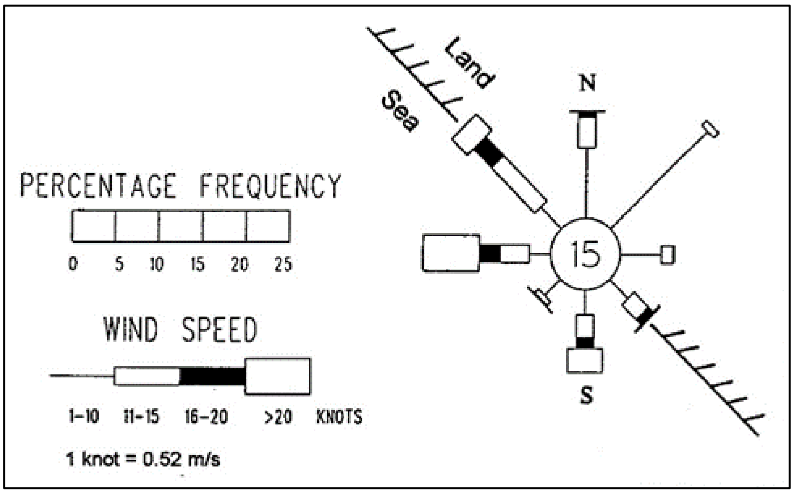


Figure 36 Wind rose for Wanganui Airport data 1.1.90 to 1.7.95
Source: NIWA

Additional descriptive statistics were derived for airport data and showed the dominant wind direction was 290 degrees and the 90th percentile (10% exceedance) value was 10.8 m/s. Cross-shore and longshore wind speed components are shown in Table 7. These results show that while longshore wind speeds are similar from both directions, two thirds (70.4%) were from the northwest. This result is consistent with the distributions for waves, currents and littoral drift supporting the assertion that wind is of fundamental importance as a forcing agent on the Wanganui Coast.

It is also noted that stronger westerly quarter winds elevate water level at the shoreline while stronger wind from the southeasterly quarter lowers shoreline water level (Goring 2007). The pileup of water against the shoreline by onshore winds is referred to as wind set-up, which, when coupled with sea-level elevation or depression associated with barometric pressure, constitutes storm surge, values of which appear in the MetOcean graphs in Figure 35. Wind-driven values are typically 0.2 to 0.3 m but when combined with low barometric pressure can exceed 0.5 m. Water-level setup or set-down at the coast has a directly impact water level in the port.

Table 7 Wind statistics for longshore and cross-shore components based on wind data from Wanganui Airport (1.1.90 to 10.11.96) provided by NIWA and derived in Shand (2000).

Statistic	Offshore (m/s)	Onshore (m/s)	Longshore northwesterly (m/s)	Longshore southeasterly (m/s)
Number	8708	7898	11731	4875
Minimum	0	0	0	0
Maximum	9.85	16.35	18.09	18.48
Mean	2.43	4.63	4.63	4.46

6 CONCLUSIONS AND RECOMMENDATIONS

- Early reports and bathymetry show the natural Whanganui Estuary was dynamic and subject to extensive snags, hence the need for channel control works during the late 19th and early 20th centuries to enable reliable depths and hazard-free navigation up to the Town Wharves.

- The lower river/estuary and port/rivermouth appear to have functioned most effectively during the 1930s and 1940s before the South Spit was breached in August 1946; this was a consequence of modified coastal sediment dynamics associated with the rivermouth moles, themselves part of the harbour development.

- During the two year breach period, a substantial volume of coastal (littoral) sediment washed directly into the estuary infilling the Main Channel adjacent to the South Spit and creating a Northern Channel configuration. After breach-closure, the Main Channel was reestablished after two years of dredging. However, the northern training wall, partially removed to facilitate navigation via the Northern Channel during and immediately following the breach period, was not restored, while channel and structure maintenance ceased in the late 1950s; consequences of these omissions upon sedimentation would be far reaching.

- Analysis carried out as part of the present study indicates an ongoing tendency toward development of the North Channel from where the channel branches just downstream of Landguard Bluff to where it rejoins the Main Channel about 500 m upstream from the base of the South Mole. Also identified were narrowing of the Main Channel and riverbank erosion where the 1940s breach had occurred, sedimentation along the Balgownie side of the Estuary, deepening of the sand bank opposite the port (Tanae Bank), and erosion adjacent to, and about the landward end of, the South Mole.

- The acquisition of detailed bathymetry and customized aerial photography in November and December 2015 respectively has enabled areas of sediment accumulation and erosion within the greater harbor area to be identified to an extent not possible in the past and several critical areas of change have been identified by the present study. This analysis now provides explanation for various observed effects made over the past decade or more by other parties.

- In particular, these 2015 data show that the sedimentation occurring within the Main Channel upsteam of Landguard Bluff may be exacerbating both North Channel development and sediment accumulations further downstream along the Balgownie side of the river to the Turning Basin Wall. These surveys also show sedimentation in the Main Channel down to the 1940s breach area on the South

Spit. Sediment accumulation is also affecting much of the Central/Lower Estuary (opposite the spit and along to the Basin Wall), and this accumulation is extending seaward into the Tanae Bank area along with deepening of the Main Channel near and seaward of the wharves. These recent data also show continued erosion adjacent to the South Mole.

- Morphodynamic effects of the Basin Wall Breach upon the adjacent estuary were assessed and found to consist initially of localized deepening in the upstream approach channel, and shallowing of the downstream channel (against the Turning Basin Wall). However, the recent enhanced accumulation along the northern side of the Estuary appears to have now overwhelmed any localized upstream channel scour response, and associated impacts of such an accumulation at the breach itself and in the downstream channel are imminent, albeit unpredictable in form.

- This study excluded a general assessment of change within the Turning Basin itself associated with the 1994 Basin Wall Breach because of inaccuracies in the 2006 bathymetric data. It is recommended that the 2006 hydrographic surveyor be identified and approached in an attempt to recover/correct these (important) data.

- It is recommended that hydrographic and aerial surveys be repeated to build upon the developing knowledge-base to track morphological change and enable a more complete understanding of sedimentation processes within the lower river and port area. Such data/understanding can help identify the need, nature and optimal timing of management intervention. The options and timing of partial and full bathymetric surveys coupled with aerial surveys (including LIDAR) should be worked through and a future monitoring program developed as a priority.

- It is recommended that an investigation be undertaken into the state (including dimensions) of the internal training walls. The findings of the present study indicate that the condition of the training walls, especially in the vicinity of Landguard Bluff, are a fundamental control in bar and channel form and change within the estuary.

- It is recommended that the 1973 City Datum Chart (copy in Appendix C (CD)) be updated by a registered surveyor as there are inconsistencies in the published Wanganui District Council datum levels. Included in the update sould be more recent mean sea-level datum and also Wanganui Chart Datum with historical variations included and benchmarks checked. The former to enable accurate interpretation of archival materials and the latter as this is the base for present surveys and development (design and construction).

- It is recommended that calibrated hindcast wave data be obtained from MetOcean Solutions Ltd and an extreme value analysis subsequently undertaken using these data. Accurate wave data is important for a range of rivermouth and coastal users as well as for hazard and harbour/port design work. Presently available data is spatially and temporally fragmented, and collected using different techniques, making it unsuitable for wave modelling.

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Mr Darrel Couper. Information Systems Manager. Wanganui District Council.
Mr Barry Waugh. Wanganui Office Manager. NIWA.
Mr Ian Moore. Consultant. Moore and Associates.
Mr Garry Williams. Consultant. G & E Williams Consultancy.
Mr Lawrie Cairns. Aerial Photographer and surveyor. Lawrie Cairns and Associates.
Mr Mike O'Sullivan. Registered Surveyor. Harrison and O'Sullivan Ltd.
Dr Peter McComb. Oceanographer. MetOceans Solutions Ltd.
Dr Mike Shepherd. Geography Lecturer (retired). Massey University.
Dr Donald Bailey. Lecturer, Image Processing. Massey University.

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COASTAL SYSTEMS LTD
Hazard, Management and Research Consultants

R D Shand
…………..…………………………….

Dr Roger Shand
Coastal Scientist

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APPENDICES

APPENDIX A Terms of reference

(i) The Baseline Study will identify historic shoreline positions, channel and bar locations, port layout and structure locations (training walls, groynes, etc), notable events such as south spit and basin breaches), and notable changes in channel/bank alignment from available sources including: historic aerial photographs; cadastral maps; bathymetric charts; port layout figures, and past reports.

(ii) The Baseline Study will summarise available process information such as tide range, tidal prism, riverflow and wave regimes and sediment transport.

(iii) The area of interest will extend from the Cobham Bridge to the rivermouth and include approximately 2 km of coast to either side, but will focus on the lower river and port area.

(iv) The timeline of interest is from the late 1800s to present, but the assessment will concentrate on the 1990s to present.

(v) The output of this study will be a stand-alone report that:

* Reviews the available literature as pertains to the Baseline Study

* Summarise the physical changes in the lower river and adjacent coast through time including a description of notable adjustments, and noting any gaps in the data/knowledge.

* Describe the main physical processes occurring in the lower river and on the adjacent coast, noting any gaps in the data/knowledge.

* Include a metadata table detailing the available resources (type, source, date, etc.)

* Include (raw) resources on CD

APPENDIX B Summary of information sources

Note these are not exhaustive but were most readily located and considered sufficient to achieve the Terms of Reference

TYPE of INFORMATION	SAMPLES OBTAINED	SOURCE - COMMENT
Technical Reports	from 1865 to 2012 (48)	WHB Archive, CSL Archive. See References
Technical drawings & Port Devel Plans	From 1878 (48)	WHB and Museum Archive. See Appendix C
Harbour Masters Reports	From 1924 to 1958 (39)	WHB Archive. Obtained relevant copies
Resident Engineer/Supervisor Reports	From 1920 to 1960 (64)	WHB Archive. Obtained relevant copies
Official memos re 1946-48 breach	1946 to 1948 (8)	WHB Archive. Obtained copies
Navigation: Nautical Almanacs, Port , Info Manuals, Admiralty Charts	1884 to 1995 (19)	Turnbull & Massey Uni Library, CSL
Bathymetric charts:		
Wide coverage (incl estuary)	1877, 93; 1921, 48; 2006,15	2006 WHB Archive, 2006 Opus (xyz) 2015 Horizons Regional Council (xyz)
Local coverage (mouth-port)	from 1874, 85,93, 1914, 21,48, 1956,68,82,93, 2006,15	WHB Archive, CSL Archive
WHB Bar charts	1926 to present @ ~2 to 4 wks	WHB Archive, Port Company
WHB Basin charts	1925 to present @ ~3 to 6 mths	WHB Archive, Port Company
WHB other charts	eg 93-96 for Basin breach (8)	Port Company, Horizons RC
LIDAR	2005, 2013	2005 Horizons RC, 2013 WDC
Settlement maps, Survey plans	1842, 56, 76/77, 82, 96	Wng Regional Museum, LINZ, CSL
Aerial photographs	1942,49,60,62,65,74,80,82,86,91 1996,98,00,04,08,09,11,13,15,15	Wanganui District Council, CSL
Satellite images	2005, 10, 12, 13,	Horizons, CSL

APPENDIX C List of raw (electronic) resources provided by the client and available (on CD or similar).

1. Materials from the CSL archive listed in Appendix B, or materials held in the CSL Archive, are not included in Appendix C, but key output are included within the body of the report or in Appendix D.

2. Scans of *Wanganui Harbour Board* materials (held in the Wanganui District Council Archive) were made at Archives Central in Fielding. Large maps/plans were scanned or photographed in parts.

3. List of maps/plans (file names)

1874 Blackett Plan with some depths
1877 Barr and Oliver Bathymetric Chart (Roll Plan) (STITCHED)
1878 Proposed protective works Sth Spit (plan views)
1878 Proposed protective works Sth Spit (cross sections)
1878 Proposed training wall long section Town to Landguard
1878 Proposed training walls and depths between Wanganui and Landguard (STITCHED)
1880 Proposed extension of north training wall Landguard to Imlay
1881 Proposed North Mole (Stage 1 drawings)
1891 Existing and proposed protection works Heads to Town
1893 Depth survey from Rivermouth to Town
1894 to 1966 Beach profiles Bamber St transect (2 sheets)
1910 Accretions at Castlecliff
1910 Rivermouth with existing and proposed mole extensions
1911 Plan shewing proposed extensions
1911 Harbour works and proposed extensions
1912 General Plan improvements and proposals Heads to Town
1914 Proposed harbour works mole extension cross sections
1921 Soundings coast and river to Motua
1926 Port of Wanganui outline plan
1927 Proposed groynes (2) off Sth Mole Jetty and South Spit
1928 Proposed groynes (2) at downstream end of northern training wall
1929 Rivermouth soundings and proposed mole extensions
1942 Proposed south training wall extension
1942 Soundings Mitchells Reach
1946 Plan of South Spit break-through
1947 Soundings in Estuary
1948 Soundings Castlecliff to Imlay
1950 to 1983 Basin dredging volumes
1951 Proposed scheme improvements Castlecliff to Imlay
1953 Proposed extension of South Mole Jetty groyne
1955 57 58 60 soundings Tanae Bank to Wharfs (5 sheets)
1955 Proposed protection works South Mole and Spit (2 sheets)
1955 Proposed protection works Spit and South Mole (2 sheets)
1956 Navy soundings (coast)
1958 Navy Chart 4612 Approaches to Wanganui
1958 Proposed protection works South Spit, Mole and Basin (3 sheets)

1967 Port Development Proposal
1968 Proposed deflection wall South Mole end (3 sheets)
1970 Port Development Proposal (2 sheets)
1973 Wanganui City Datum Chart
1995 Navy Chart 4541 Approaches to Wanganui

4. List of aerial photos (file names) 1949, 1996, 2000, 2004, 2008, 2013, 2015a, 2015b
   1949 WDC non-ortho_ merged
   1996 WDC ortho
   2000 WDC ortho
   2004 WDC ortho
   2005 HRC ortho
   2008 WDC ortho
   2011 HRC ortho
   2013 WDC ortho
   2015 (August) WDC ortho
   2015 (December) WDC non-ortho_merged

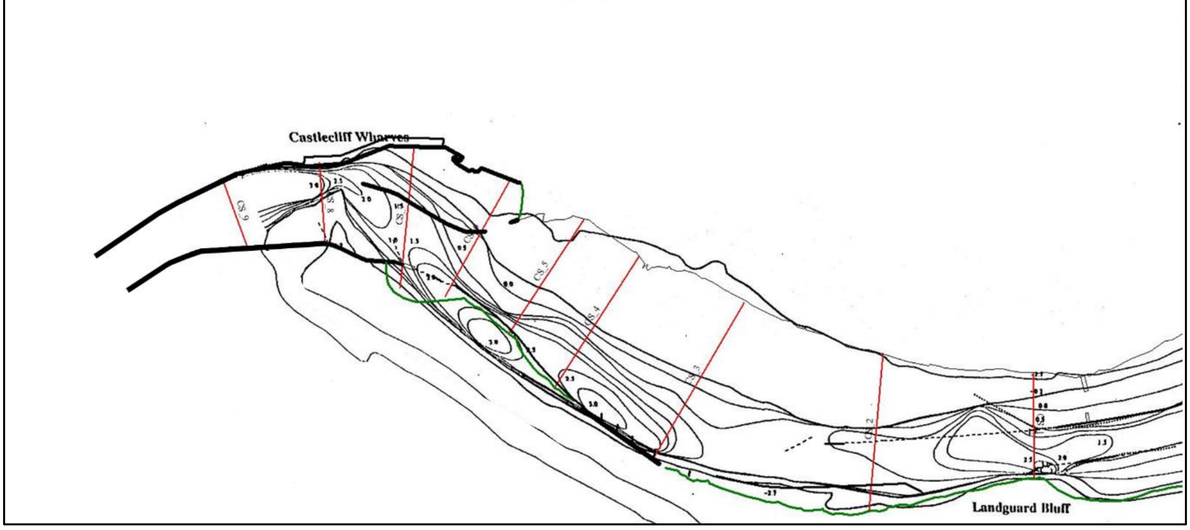
5. List of recent reports/documents (file names)
   1998 Williams. Whanganui River Report
   2002 Massey University. Environmental implications of North Mole
   2002 Opus. Integrity of the North Mole
   2003 Shand. Inner bar modelling CS_03
   2004 Shand. Beach cut and NOM ICS_04
   2004 WDC Memorandum for Harbour Committee Meeting of 2 February
   2004 Williams. Whanganui Estuary Report
   2006 Williams. Whanganui Flooding Report
   2007 Shand. Bar splitting and NOM ICS_07
   2007 Williams. Whanganui River Mouth Report
   2011 Williams. Whanganui River South Spit Report
   2014 Moore. Whanganui River, North and South Mole Update Report

Note Moore 2006, 2010 (January), 2010 (December), 2011 (February) are only available in hard copy.

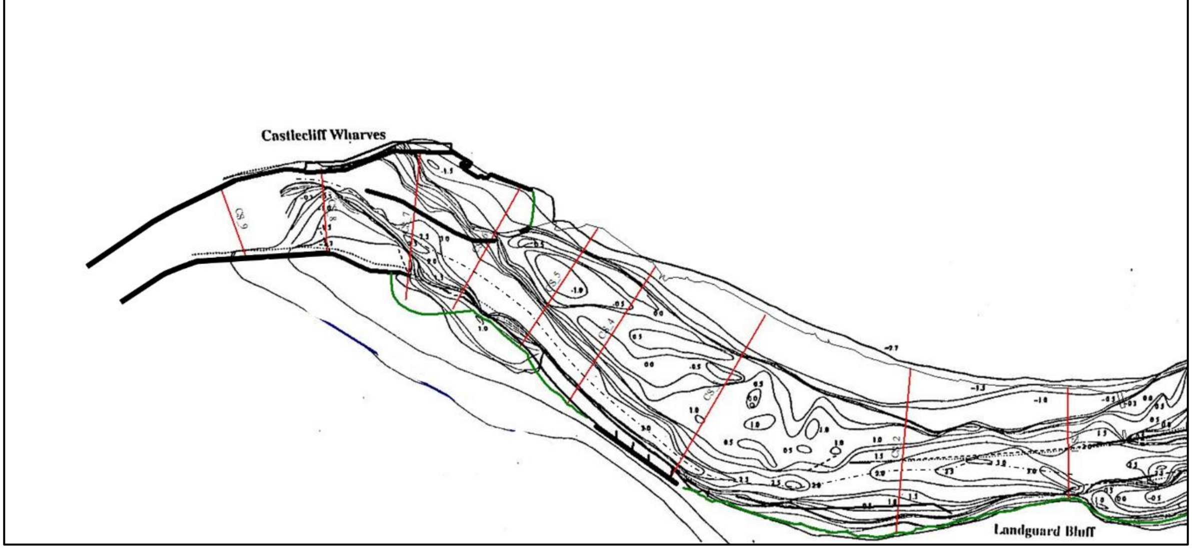
APPENDIX D Bathymetries used in Section 3 cross-section analyses

Analysis transects and existing structures marked. Sources listed below.

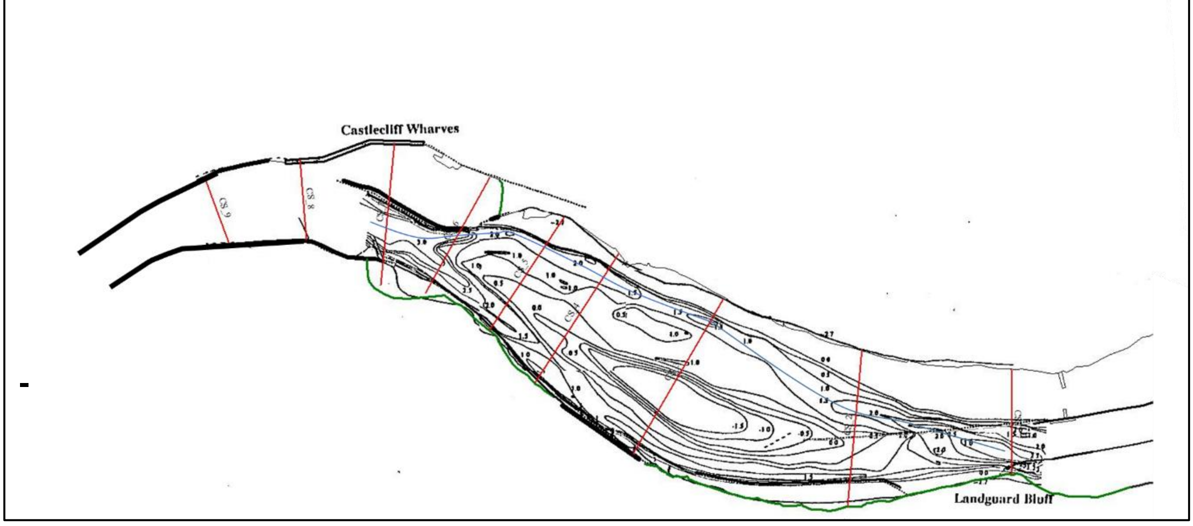
1893



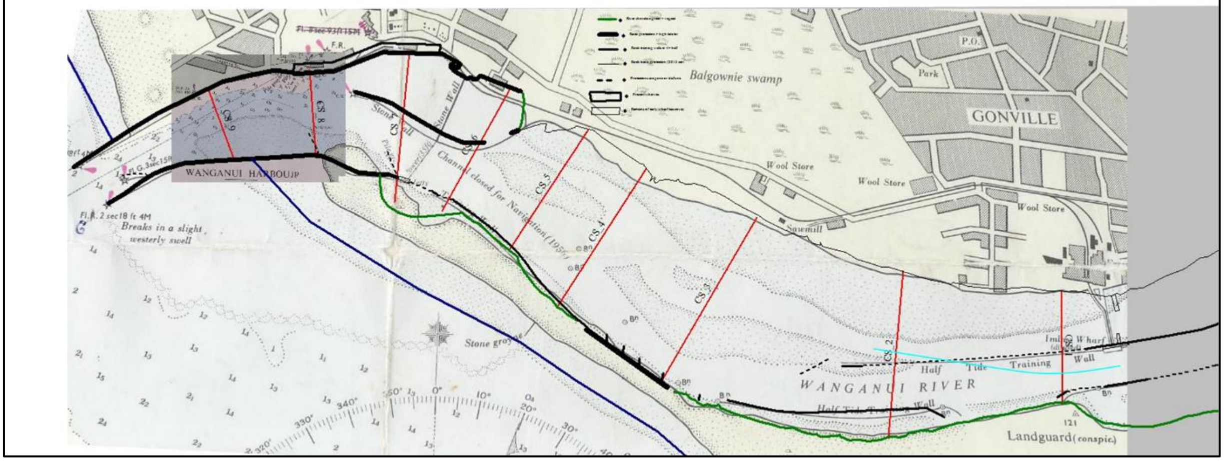
1921



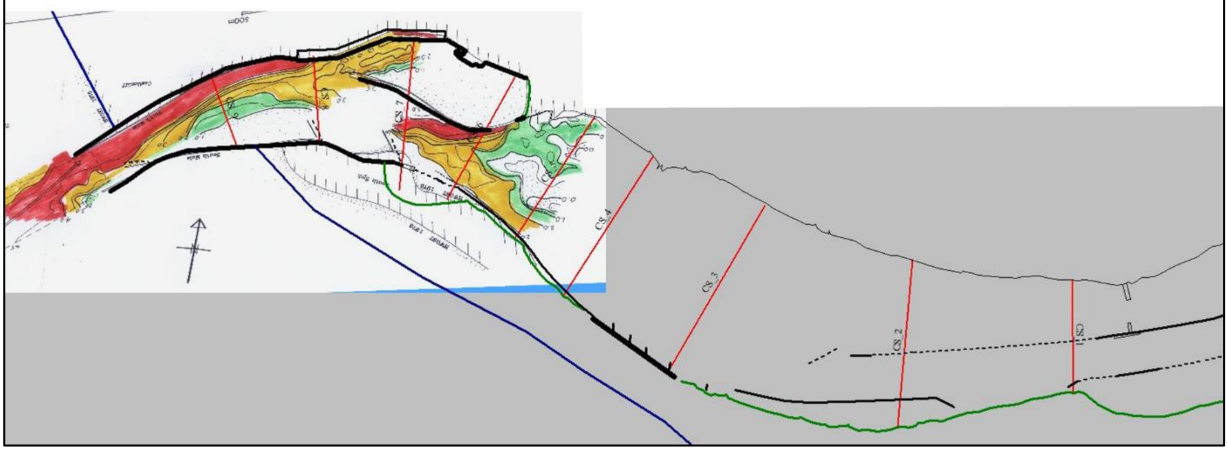
1948



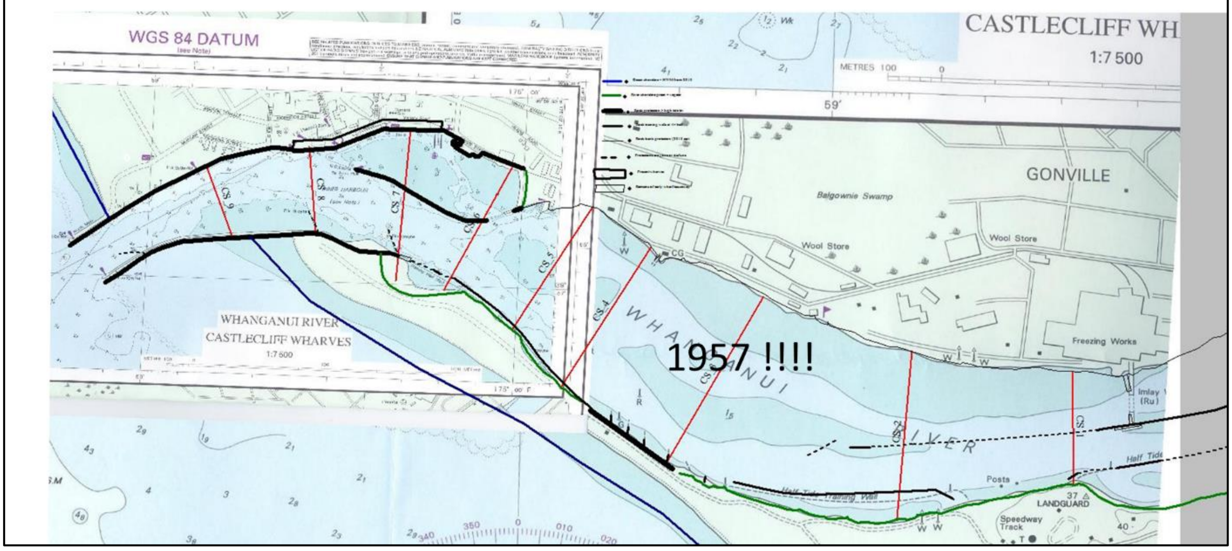
1957

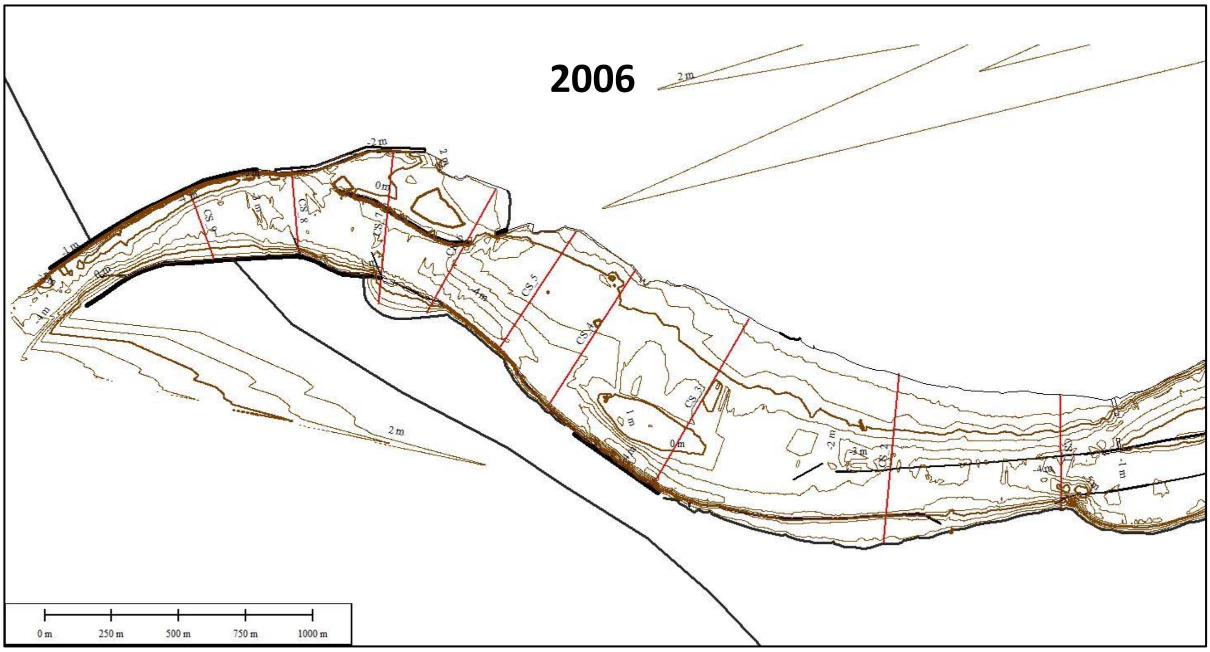


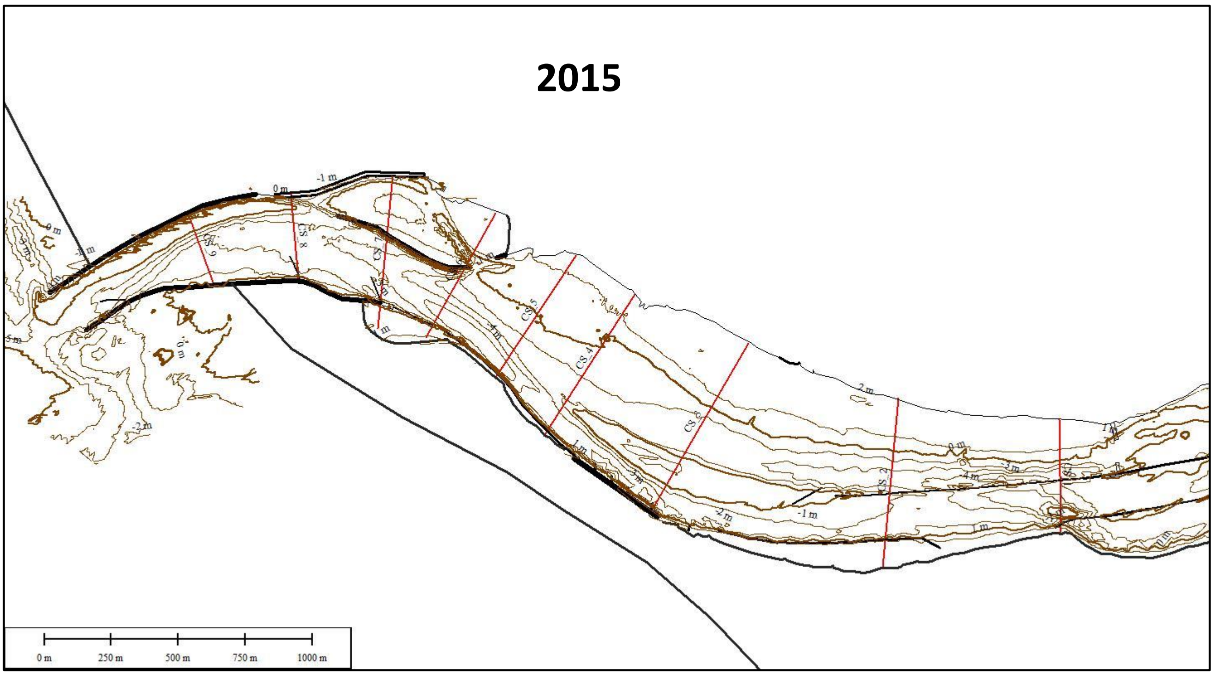
1982



1993







Information sources

1893 WHB Archive, Macky (1989)
1821 WHB Archive, Macky (1989)
1948 WHB Archive, Macky (1989)
1957 Navy Chart NZ 4612
1982 WHB Archive
1993 Navy Chart NZ 4541
2006 Opus (Wanganui)
2015 Horizons Regional Council

APPENDIX E Sampling points for the 2006 and 2015 bathymetric surveys

Superimposed upon 2004 aerial photos (upper) and 2015 lower.