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SEDIMENT TRANSPORT AND TIDAL INLET HYDRAULICS

A thesis
submitted in fulfillment
of the requirements for the Degree
of

Doctor of Philosophy
in Earth Sciences

at the
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by

Kerry Peter Black

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ABSTRACT

An investigation into sediment transport and tidal inlet hydraulics was initiated to ascertain the impact of a proposed timber port development at Marsden Point in Whangarei Harbour. The program progressed along two parallel pathways, i.e. (i) an expository description of the relevant harbour characteristics using field measurements and numerical modelling and (ii) a detailed investigation into the physics of sediment transport under tidal flow.

The first of these provided an understanding of the hydraulic and sediment transport characteristics of the inlet while aiming to separate cause and effect in the present harbour configuration. The numerical modelling extended this to provide predictions of future behaviour after port construction. In the tidal environment, methods to isolate the effective component of the flow velocity responsible for the transport of sediment are often inadequately substantiated being mostly derived in rivers or flumes. Accordingly, the second aspect of the program necessarily approached and extended the current state of the art with respect to sediment transport under oscillating, non-steady tidal currents in an estuary.

Bathymetric and sea bed surveys revealed that the harbour is mostly stable in an inlet receiving low inputs of sandy sediment. The entrance is sheltered from most storm wave attack by land masses offshore. Due to preferential scouring of sandy bed material, the channels are typically lined with a predominantly-stable, shell-gravel lag. Zones of deposition primarily occur in areas of tidal current neutrality.

Outside the entrance, with the added wave influences, bathymetric changes are greater. Using the experimental results in conjunction with inferences from standard tidal deltas, the changes were seen to be occurring in a closed or partially-closed sediment loop in northern Bream Bay including ebb-tidal transport around the north and east of Mair Bank, wave-induced, shoreward sediment transport over the southern segments of the ebb-tidal delta and northward longshore transport along Bream Bay Beach and through the marginal flood channel off Marsden Point.

Similar controlling transport directions occur inside the entrance with the two most dominant being an ebb-directed route from One Tree Point along the shipping channel and out of the southern side of the entrance throat. Another passes up the Snake Bank flood ramp and around the north-east flank of the flood-tidal delta.

More than 600 vertical velocity profiles were measured in the estuary. These were examined to derive bed friction coefficients and, over megaripples, the form drag and effective components of the bed shear stress were isolated. This was applied to the determination of the sediment threshold velocity and

the transport of sandy material as bedload. New methods to ascertain the threshold and to derive the bedload transport fluxes over megaripples were presented.

Wave height measurements and sediment trapping experiments revealed that the timber port would have a very limited influence on its environment due to changes of sediment transport patterns on the inter-tidal zone at the port site.

One and two-dimensional numerical hydrodynamic models and a 2-dimensional numerical sediment transport model were produced for the study and applied to the proposed timber port construction and dredge zone. The sediment model provides erosion and sedimentation rates in a tidal inlet.

With the sediment transport model and the 2-dimensional hydrodynamic model in a coupled mode, numerical experiments were undertaken to produce a fully-developed estuary from an initially planar bathymetry. By alternately running each model to obtain water speeds and bathymetry changes, an estuary with inter-tidal flood-tidal deltas, ebb-tidal deltas, a deep main channel, marginal flood and ebb channels and a stable ebb shield developed. The method has considerable potential for the prediction of final equilibrium bathymetry after man-made changes.

An iterative technique was designed which utilises the sediment transport model and the water velocities provided by the hydrodynamic model. The procedure determines the percentage of sand cover at all locations throughout the model region in a stable estuary and would allow an optimisation of the minimum quantities of dumped lag material necessary to prevent scour or accretion by establishing a dynamic equilibrium over large areas of the harbour bed.

After accounting for the shell cover which restricts the transport of sediment, the maintenance dredging requirements for the initial 2 berth port development (Case A7) were small and estimated to be in the range of 10,000 m^3 to 30,000 m^3 per year. The impact of the port in the vicinity of the NZRC wharf is expected to be minimal with some slow sedimentation of the order of a few centimetres per year possibly occurring. In Shoal Bay, the port should have an extremely limited impact. In the shipping channel to One Tree Point, small amounts of erosion may occur in response to an increased flow velocity ($\simeq 5\%$) caused by deepening of the channel in the dredge zone.

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More than two years were spent in Whangarei and NHB personnel were faced with a series of unusual and sometimes inconvenient request which eventually earned them the title of "Northland Oceanographic Institute". The Harbour Board provided personnel and equipment including NHB vessels throughout the study.

Of the Harbour Board staff, Mr Murray Hunter, NHB Senior Civil Engineer, can be accredited with considerable personal responsibility for the success of the work. Mr Hunter always supported the research and undertook the administrative and logistic tasks necessary to fulfil the often difficult experimental design requirements. As the Engineer-in-Charge of the timber port study, the successful large-scale data collection program is a testimony to his efforts. Mr Hunter's support is warmly acknowledged.

Civil Engineer Mr Mike Beazley spent many hours in the field making scientific measurements both in and out of the water. He also acted on Mr Hunter's behalf preparing for field data collection during a period Mr Hunter was working in Denmark with the Danish Hydraulic Institute. Mr Beazley's expertise was appreciated.

NHB Civil Engineering Officer, Mr Warren King was present during most field operations and worked through many long 13-15 hour current measurement days. Mr King personally undertook the inter-tidal beach surveys involving data reduction and graphical presentation. Mr Beazley and Mr King managed the wave probe including calibration testing and final verification of probe behaviour during an extended period in 1983.

NHB divers, Mr Peter Hudson and Mr Don Gregory worked long hours, often in very cold conditions and fast currents and showed strong support by working hard when daily hours were governed more by the state of the tide rather than the time of day. These two gentlemen shared the burden of

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SEDIMENT TRANSPORT AND TIDAL INLET HYDRAULICS

Chapter 1:

INTRODUCTION

CHAPTER 1 INTRODUCTION

1.1 Background

Situated on the east coast of the North Auckland peninsula (Figure 1.1), Whangarei Harbour has one of the deepest ports in New Zealand. At its Marsden Point entrance, a terminal has been built to service the country's sole oil refinery. The wharf, constructed on piles in 1963, was placed in the narrowest part of the entrance throat on the southern shore, which is a prograded, sandy spit separating Bream Bay Beach to the south from the inner Harbour to the west. It is a potentially-unstable site, with strong tidal flows exceeding $1.3 m.s^{-1}$ nearby in mid-gorge.

In response to a projected need for future adequate port facilities, the Northland Harbour Board (NHB) investigated the suitability of Marsden Point for the construction of a new port, principally for the shipment of timber products, although not solely for this purpose. The preferred site was near the existing NHB wharf, some 700m further into the harbour from the NZRC wharf. The port was to be constructed of land-fill forming a permanent barrier to the transport of sediment past the site. Furthermore, for shipping requirements, a considerable dredge zone extending 450m offshore along the length of the port, was required.

The NHB was apprehensive about the impact of this development with particular regard to coastal erosion near the NZRC wharf and Marsden Point, and the general impact on a waterway which has considerable recreational and commercial use. In conjunction with other interested parties, the NHB then undertook preliminary feasibility studies and a full Environmental Impact Report (EIR, 1981).

Healy, in the EIR section discussing sediments and inlet hydraulics at the port site, made a preliminary analysis of potential sediment transport, and noted that planned dredging would significantly increase the cross-sectional area near the inlet throat. Accepting that tidal inlets are dynamic-equilibrium systems, likely to react to changed hydraulic conditions, the EIR recommended a detailed investigation of sediment transport characteristics and inlet hydraulics incorporating physical and/or mathematical modelling. This writer's study was then established in December, 1980, to investigate Whangarei Harbour and the impact of the proposed port with particular regard to sediment transport. The study was a University of Waikato, Doctor of Philosophy project sponsored by the NHB.

After this project was established, a consulting organisation, the Danish Hydraulic Institute (DHI), were also appointed by the NHB. With physical and numerical modelling they were to:

(i) determine an acceptable range of layouts for the first stage of port development with respect to orientation of the quay front and its distance

from the shore, (ii) determine the width and extent of the dredging basin, (iii) study ship manoeuvring conditions, (iv) estimate the effects of the developments on sediment erosion and/or accretion. The results of the DHI study were presented to the NHB in December, 1982.

Previous major studies of the Harbour have been conducted by Rogers (1959) who constructed a physical model for the NHB, which included waves and currents; Bioresearches (1976, 1978, 1979, 1980) and Smith-McNaught (1976) who have undertaken field research in the Harbour in a consulting capacity; and Millar (1980) who examined the bed sediments in Whangarei Harbour with specific application to the spoils from the Portland Cement Works in the Upper Harbour.

1.2 Thesis aims and experimental design

The general aims of the research program were:

(i) to ascertain the present hydraulic conditions and sediment transport characteristics at Marsden Point.

(ii) to undertake research into sediment transport under oscillatory tidal flow.

(iii) to assess the impact of the port development on the inlet hydraulics with particular regard to sediment transport and including expected maintenance dredging requirements. At a later stage, the problem of a suitable location for the dumping of dredge spoils was raised by the NHB. To make these assessments, this writer proposed to undertake numerical modelling of the harbour as well as a supportive, detailed field data collection and analysis program.

The study took nearly three years to complete starting in January, 1981 and is presented in two volumes with the Text in Volume 1 and Figures and Tables in Volume 2. The thesis deals initially with field data collection and analysis (Chapters 2 to 8) and secondly with numerical computer modelling (Chapters 9 to 11). The two aspects are closely inter-related and both play an important role in the final determinations. Furthermore, the field program made it possible to perform detailed tests of the quality of the numerical simulations in a tidal inlet.

Chapter 2: Fundamental to a sediment transport study is an investigation of the nature of the sea floor to ascertain the size, shape and geometric configuration of in situ bed sediments. This also allows identification of zones of active sediment transport and scoured, shell lag where fluxes are not likely to be as significant. To achieve this, underwater photography, underwater video and side-scan sonar were employed. These aspects are only briefly discussed in this thesis (Chapter 2), because they were fully detailed in two reports prepared by this writer for the NHB (Black, et. al., 1981; Black and Healy, 1982). These reports may be seen as Appendices to the thesis and are

quoted often throughout.

Chapter 3: To determine whether the existing morphology of the harbour is stable, a bathymetric sounding program was established. Soon afterwards, the Royal New Zealand Navy, Hydrographic Branch undertook a complete bathymetric survey of the area. A prior RNZN survey was made in 1961. These are all compared to isolate areas of deposition and erosion. Furthermore, the relationship of channel morphology to tidal hydraulics is discussed. As a result of the sounding program, a computerised sounding reduction procedure was established on the NHB PDP 11/23 computer, detailed in Black (1983).

Chapter 4: The sediment budget in a tidal inlet constantly responds to the wave and current regime outside the entrance. It is not sufficient to examine the up-harbour side of the entrance throat without some concurrent knowledge of the ocean and beach behaviour as the two are closely interrelated. The effect of waves impinging on northern Bream Bay and on the ebb-tidal delta, and their influence on sediment transport was examined primarily using computer models simulating the refraction of surface, gravity waves and wind-wave generation. This writer's computer programs and mathematical algorithms were published in Black and Healy (1981).

Chapter 5: Having made prior determinations of the bed facies and of the zones of bathymetric instability in the harbour, a detailed program of water velocity measurements was established. The flows, driven predominantly by semi-diurnal tidal oscillations, are responsible for the transmission of "cause and effect" within the estuary through the transportation of loose sediment and the carriage of flow momentum. At some 58 sites, vertical velocity profiles were measured throughout the tidal cycle. Numerous drogue tracking experiments and shallow water measurements around the major banks were also undertaken. The data specify the present hydrodynamic regime in Whangarei Harbour and northern Bream Bay.

Chapter 6: Frictional energy losses in the tidal inlet are examined in Chapter 6, for three major reasons:
(i) it is a research subject still being debated, (ii) the loss of energy by the flow over undulating, bedformed sand is intrinsically related to the sediment transport processes and (iii) the derived friction coefficients would have application in the numerical models. More than 600 vertical profiles were measured in the prototype which offered a wide data base to draw conclusions from. A new method to determine the friction coefficients over bedforms was developed, for application in the next Chapter to the determination of sediment transport rates.

Chapter 7: The problem of determining sediment transport rates under reversing tidal flow and over megarippled beds is addressed. As sediment

transport data during the tidal cycle under estuarine flows has rarely been measured because of the logistic and environmental problems, a field program was designed to make measurements of the bedload and suspended load sediment fluxes in conjunction with relevant hydraulic parameters. The data are used to formulate a new procedure to predict sediment transport rates from the water speeds at 1m above the sea bed and the D_{50} and D_{65} grains sizes. The method isolates the skin friction component of the friction velocity which includes a form drag component over megarippled beds. By doing so, it is shown to be possible to use a plane bed transport equation to determine sediment fluxes even over strongly-bedformed, non-cohesive sand. Further to this, a new procedure to predict the sediment threshold over bedforms is also developed. In conjunction with the water velocity measurements (Chapter 5), the threshold and sediment transport equations were applied to ascertain the present sediment fluxes in the harbour including the rates at the proposed port site. This Chapter satisfies the first part of the NHB Brief by specifying the present hydraulic and sediment transport characteristics at Marsden Point. Results in this and the last Chapter were published at the ASCE Conference in Cape Town (Black and Healy, 1982b) in a paper prepared by this writer from a draught of this thesis.

Chapter 8: The proposed port will totally disrupt sediment transport along the inter-tidal zone. Thus, experiments were conducted to ascertain the transport rates and directions there, and the relationships to wave, wind and tidally-induced currents. Once again, very little sediment transport data for an inter-tidal zone subject to low tidal currents and severely fetch-limited wave attack was available, as most effort is concentrated in higher-energy regions such as open-sea beaches. To measure the sea state, a capacitance-type, wave recorder with a data aquisition system for input onto the NHB PDP 11/23 computer was established.

Chapter 9: To further investigate the tidal behaviour of the inlet and port area, a one-dimensional, numerical model was developed and applied. The model uses an explicit, finite-difference scheme to solve the hydrodynamic equations. Behaviour is examined using test rigs and a one-dimensional schematisation of Whangarei Harbour. After successful comparisons were made the one-dimensional model was employed to generate boundary sea levels for the 2-d case.

Chapter 10: To the writer's knowledge, a 2-dimensional hydrodynamic estuarine inlet model has not been previously successfully developed and applied in New Zealand. The satisfied objective of this Chapter, was to develop the 2-dimensional model (2DD). Comparisons are made with the prototype, the DHI S21 model and test rigs. Furthermore, resultant water speeds over the tidal cycle, which map the nett sediment transport potentials, are generated

and compared with field measurements. The comprehensive field program made it possible to do detailed comparisons with the prototype. In many cases because of the prohibitive cost of obtaining information, this cannot be done.

Chapter 11: The 2-dimensional hydrodynamic model is extended to include numerical sediment transport modelling. This is at the apex of a research program designed to derive an understanding of sediment transport processes at Marsden Point. The model uses water speeds from 2DD and the transport equations derived in Chapter 7 to ascertain the sediment erosion or accretion rate in square cells throughout the harbour. As far as this writer knows, this was the first model of its kind which uses a 2-dimensional hydrodynamic model as input to a sediment model. Notwithstanding the research implications, the model is used to satisfy the remaining expectations of the NHB in the initial brief and the general study aims.

Appendices 1, 2 and 3 discuss a computer system written specifically for this project to handle the massive data analysis, reduction and presentation requirements. The system was recently purchased by the Water Quality Centre, Ministry of Works in Hamilton for application to a similar study for the Bay of Plenty Harbour Board in Tauranga Harbour.

Appendices 4 and 5 offer circuit diagrams (Jordan and Schaare, 1980) and the writer's computer program designed to allow direct input of digital data from an Analogue to Digital converter onto a PDP 11/23 computer. The data aquisition system was established for use with the capacitance, wave-height recorder.

Appendix 6 describes a method to find the effective width of sediment traps subjected to currents with varying directions, and utilised during analysis of the sand trapping experiments on the inter-tidal zone at the port site.

1.3 Relationship to DHI study

Some overlap has occurred between the two studies. In both cases, the computer modelling was necessary to obtain a "best" solution to the problem. From the research point of view, this overlap proved to be useful as the 2-dimensional numerical model developed by the writer for this study, could be directly compared with the DHI model and both were examined for their ability to reproduce the measured speeds in the prototype. Comparisons of this nature in the same tidal inlet, have rarely occurred before. Notably, the two models apply different methods (explicit and mixed-implicit schemes) to solve the hydrodynamic equations.

1.4 Thesis structure and style

A brief discussion of the factors influencing the style of this thesis

is perhaps warranted. By undertaking a strongly practical topic which was established to find out an answer to a specific problem, one is immediately in the position of having a "foot in two camps". That is, there is the justified "real world" NHB expectations on the one hand, and on the other, there is what this writer idealistically believes to be a moral obligation on the part of the research organisation to allow senior students to undertake a program of "pure" research.

To define the latter, it is time spent on avenues of investigation instigated by a natural curiosity about the physical world with no necessary concern for the practical applications of the results. By its nature, Science must maintain a link with Philosophy and only the research organisations can uphold this link. Furthermore, in a practical, University department, there is a difficult-to-define division between new research and the rehashing of old research in a new harbour or sea, with the view to solving problems specific to particular areas. The style of this thesis then arises out of several diverse influences. It is hoped that a reasonable balance has been struck.

1.5 Computer installation

Computer analysis and reduction was primarily undertaken on the Northland Harbour Board's Digital PDP11/23 computer with 128K bytes of memory. The installation includes two VT100 terminals with an LA180 line printer as well as an LA120 hard copy terminal. Two RL02 disc drives provide adequate storage for data and programming. An HP7221C 8-colour plotter was acquired by the NHB at the commencement of the study to enable graphics plotting.

An RT11 operating system and FORTRAN language compiler was also purchased by the NHB for this study and was established on the computer by this writer. All subsequent programming was written in a subset (Digital, Version 4) of ANSI standard FORTRAN.

1.6 Study area

The initial aim of the project was to examine the effects of the timber port at Marsden Point, so a principal study region from the Ruakaka River mouth in Bream Bay to just west of One Tree Point (Figure 1.1) was defined. The goal was to keep the physical size to a minimum so that more detail could be entered into, while maintaining a sufficiently large area so that the effect of processes outside the region would be minimally felt at the timber port site. Conversely, the port construction should have very limited influence beyond the boundaries. The interaction expected to occur between Bream Bay and the inner harbour warranted inclusion of northern Bream Bay.

The major banks and channels and the names assigned to previously unlabelled features are defined in Figure 1.2. Included in this Figure are

landmarks numbered by this writer. Their assigned names with Mt Eden Geodetic Metric Coordinates are listed in Table 1.1. This list is stored on computer disc to be used for position fixing programs as well as for map reproduction.

Major banks in the region are Snake and McDonald Banks defined classically as flood-tidal deltas, being located inside the entrance throat. This classification is later questioned when it is shown that McDonald Bank is in fact, subject to dominant ebb currents. Ebb-tidal deltas found outside the throat, are Calliope and Mair Banks with the latter being the major feature separating the main shipping channel to its north from the ocean waters of Bream Bay to its south.

Millar (1980) classified the harbour as being a polygenetic estuary of the drowned river valley type: "The course of the original river valley appears to have been controlled by the Harbour Fault (Ferrar, 1925) and the entrance has been restricted by Holocene sand-spit and dune ridge formations" (Millar, 1980, p. 30). At its narrowest point the entrance is 730m wide and has a depth of 31m below Chart Datum. Millar (1980) summarised the harbour's geology, catchment, climate and vegetation.

1.7 Tidal regime

The tidal curve in Whangarei Harbour at Marsden Point is asymmetrical with half tidal periods averaging 6 hrs 5 mins and 6 hrs 20 mins for the ebb and flood respectively (Millar, 1980). The mean Spring tidal range is 2.2m and the mean Neap is 1.6m with overall average of 1.9m. The Port Whangarei range is greater with a mean of 2.1m. Smith-McNaught (1976) confirmed that the tide at Port Whangarei is dominated by the principal lunar semi-diurnal component, M_2, followed in order of importance by the N_2, S_2, K_1 and M_4 components.

Both Millar and Smith McNaught present graphs, obtained by planimetering the harbour charts, for the determination of tidal prism which is 165×10^6 m^3 for a mean Spring tide according to Millar. An extreme tide has a prism of approximately $200\text{-}210 \times 10^6$ m^3.

Chart Datum (CD) at Marsden Point, which is 1.681m below Mean Sea Level at One Tree Point, is different from CD at Port Whangarei which is 1.907m below the same datum. Thus, Port Whangarei CD is 0.226m below that of Marsden Point. This is especially important for the calculation of sea slope between the two tide gauges but it also causes some difficulty with the interpretation of harbour bathymetric charts. One Tree Point was apparently chosen as a cut-off on the RNZN chart NZ5213 where depths up-harbour are relative to Port Whangarei CD and those downstream were related to Marsden Point. No definite verification of the cut-off has been obtained.

SEDIMENT TRANSPORT AND TIDAL INLET HYDRAULICS

Chapter 2:

SEA BED DESCRIPTION

CHAPTER 2 SEA BED DESCRIPTION

2.1 Introduction

As a first step in a sediment transport investigation it is necessary to ascertain the nature of the sea floor and the bed sediments present. To achieve this, underwater still photography, underwater video, side-scan sonar and depth sounder surveys, together with a detailed surficial bed sediment sampling program were implemented. The collection of this data is an essential prerequisite to the study of sediment transport in the tidal inlet.

2.2 Sea floor photography

Sea floor photographs at 67 sites were taken by the NHB and RNZN divers for an initial investigation of the sea floor. The results of this survey were published in Black et. al. (1981) which details the experimental procedures and offers two photographs from each location with relevant comments about the sediments and biological features. Maps were produced which summarised the results and these only will be reproduced here in Figures 2.1 to 2.3. Throughout this Chapter, "Photo Sites" will refer to the above publication.

2.3 Underwater video investigation

More than 100 locations were examined with an underwater video camera operated with the assistance of the RNZN diving tender "Manawanui" commanded by Lt Gary Collier, RNZN. Pre-selected sites were located with Motorola EDM equipment, and the video was lowered unattended to the sea floor where either (i) a 2-3 minute examination of the bed was made before moving to the next site or (ii) a long, slow drift along a predetermined course was made to establish the continuity of bed facies, and to ascertain the location of boundaries between shelly and sandy regions. Adjustments to the camera height above the seabed were made while watching video monitors on board ship. A lead weight was tied at a known distance below the camera for scale and to facilitate easier height adjustments. The important pictures were recorded on video film for later examination.

The video inspections were summarised by this writer in Black and Healy (Appendix II, 1982). Corresponding maps of the bed have been reproduced here in Figures 2.4 to 2.5.

2.4 Diver verification of bedformed regions

As a result of the underwater video and photography investigations, megarippled sandy beds - a primary indicator of active sediment transport - were identified along distended regions aligned longitudinally with harbour flow orientations (i) along the 5m contour around the north-east side of Snake Bank in Shoal Bay, and (ii) along the south-east sub-tidal extension of Snake Bank towards the NZRC wharf.

A diver verification of the existence of these insular pathways flanked by stable beds was sought, because of their importance to the overall harbour sediment budget. Moreover, the latter loose-sediment "pathway" passes across the northern edge of the proposed timber port dredge zone.

Working from the NHB oil spill vessel "Maunu", divers were attached to the surface by a rope which acted as a signal line for bed descriptions and were asked primarily to follow the active sediment pathways, to be identified by the presence of bedforms. Three distinguishing bed types were to be discerned (i) non-active i.e. any region without bedforms and could include flat sand and gravel beds (ii) rippled and (iii) megarippled beds, the last two being distinguished by a difference in scale of the features. The boat positions were recorded with Motorola EDM equipment and reduced and plotted as described in Appendices 1 and 2.

The divers covered a total of about 7km and their observations were compiled in Figure 2.6.

2.5 Side-scan sonar survey

A detailed side-scan sonar survey of the study region was undertaken with assistance from the Royal New Zealand Navy, Hydrographic Branch and the results were published in Black and Healy (1982). The inner harbour from One Tree Point in the west, through the tidal inlet gorge to the outer channels and Mair Bank ebb-tidal delta were surveyed.

The sonographs were interpreted with simultaneous consultation of the previous photography, video and diver verification data, in conjunction with a concommitant echo sounder record. Figures 2.7 and 2.8 classifying the sea floor facies were constructed from the side-scan charts, while inclusively compiling information from all the above studies.

2.6 Bed sediment sampling

The Royal New Zealand Navy, Hydrographic Branch undertook a bed sediment sampling program for application to this writer's and the DHI's study. Initial samples were taken by divers during the photography survey in Bream Bay. Others were remotely sampled with a towed dredge. The site locations were reduced and recorded by Lt Cdr J. Stoakes, (RNZN, Hydrographic Office) and the collected samples were analysed by the University of Waikato under the direction of Dr T. Healy. Samples were sieved at 0.25ϕ intervals, with the shell content left intact. Computer output of the grain size analyses using the program of de Lange and Kamp (1980) was supplied by the University. Figures 2.9 and 2.10 respectively show the D_{50} grain size and the percentage by weight exceeding 1ϕ (P(1ϕ)) with their contours plotted by this writer. The site locations were digitised from the RNZN maps (NZ5213, Serials 2a to 6a) and converted to the Mt Eden Geodetic Metric Grid using the methods outlined in Appendices 1 and 2.

2.7 Sea floor facies

Integration of the preceeding investigations permitted identification of the following sea floor characteristics summarised in Figure 2.11.

2.7.1 The channel floors

These are typically covered with a shelly lag, formed of Dog Cockles, Scallops, Cockles, Tuatua, Turret shells and rock fragments as major constituents (see also, Bioresearches, 1976). Some shell clasts are coated with pink, corraline, algal encrustations and anenomes. These grow best if the shell is undisturbed by tidal currents and if the suspended sediment load is low, thus minimising the "sand-blasting" abrasive effect and allowing weed to grow. The algae is therefore an indicator of bed stability and the suspended sediment transport rates, whereas the presence of the shell lag itself implies an erosive current/sediment balance such that the more easily eroded sand is carried away and not replaced from an upstream direction, leaving the shell deposits.

There is a wide variation in scoured bed facies e.g. the up-harbour progression from the inlet throat at Photo Sites 5 to 19 to 26 changing from clean to algae-coated shell. Different again is the bed in the shipping channel south of Snake Bank at Photo Sites 11 and 24, which is described in Black, et. al. (1981) as "40-50% cobble, shelly-gravel and biomass cover; some loose sand with broken shell". In the scoured holes off Darch and Reserve Points, Photo Sites 27 and 3 respectively, the beds were covered by a conglomeration of shell and rock cobble constituents, with a biomass including sponges, algae and starfish and bear little resemblance to the clean shell deposits in the entrance throat.

At Photo Site 14, in the channel behind McDonald Bank around McLeod Bay, algal growth suggests bed stability, in the presence of significant fine-sand quantities. The paradox can be resolved by examination of the flow velocities which are later shown to be small through this region. A similar explanation is offered for the occurrence of fine sand, impregnated with myriad marine animal burrows, again testifying to the bed inactivity, found along the channel north of McDonald Bank near Munro West Point during video inspection. Subsequent drogue experiments (Chapter 5) revealed very low flow velocities. In all these cases, the side-scan sonograph depicted a gravel bed evidenced by a speckled trace. Interpretation was only made possible with supplementary information. Outside the entrance throat, the main channel floor is lined mostly with clean or lightly-encrusted shell, see Photo Sites 35, 36, 34, 33 and Navy 28 and 27.

2.7.2 Bedformed sediments

Throughout this thesis, bedforms will be classified according to their

wavelengths only (Table 2.1) after Harms et. al. (1975). Strictly, a bedform classification should strongly relate the feature to its hydraulic environment. The Harms et. al. classification has limited physical justification especially the wavelength distinction between megaripples and sandwaves. It is, however, easily applied and allows simple description of the scale of the features under discussion. The term megaripple will be used in preference to "dune" which has two common meanings, and will not be used to describe subtidal bedforms.

The bedform zones in the inner harbour were located: (i) along the south side of the shipping channel between One Tree Point and Blacksmith's Creek, (ii) from the south-east extension of Snake Bank towards the NZRC wharf (verified by divers), (iii) flanking the 5m contour on the south side of Shoal Bay and leading towards the entrance throat beyond Darch Point where distinctive sand waves with amplitudes greater than 1m were found off Reotahi Bay (verified by divers) and, (iv) in the shipping channel off One Tree Point.

A major finding is the narrow, distended nature of the sediment pathways and the relatively small percentage of bed area covered by loose, bedformed sediment. Both the pathway in Shoal Bay and to the north of the NHB wharf are no more than 140m wide. Thus, the total quantities of sediment in motion are severely limited by the physical extent of the loose sand. The Snake Bank extension has a bipartite structure (Figure 2.7) being megarippled to the south and covered by sand waves to the north (Photo Site 23). Furthermore, the transition from bedformed to a more stable shell lag is very abrupt on the southerly side (Figure 2.12) and is a feature of these regions, with sudden, distinct changes in bed facies. Due to this and because of the detailed, overlapping video and side-scan sonar observations, the boundary of the active sediment transport zone could be accurately placed along much of the pathway's length and this is shown on Figure 2.7.

The sand waves on the south side of the shipping channel off Point Sinclair (Fix 543, Figure 2.7) are unusual in that they are located in a high flow area and typically in Whangarei Harbour the bed is scoured, shelly lag in such zones. In respect of this, divers were asked to verify their existence, by recording observations along the cross-channel power cable between the Shell Cut Reach Front Lead and the One Tree Point Beacon (11s Beacon) which served as a course indicator. No position fixing gear was used and approximate visual positions of a buoy attached by rope to the divers were recorded with the time of day. This was later matched with observations written on a slate by the divers. A definite assertion cannot be made because of the potential positional errors, however the divers did not confirm the existence of the sand waves, even though they did find large megaripples flanking the channel on both sides. There is thus, some doubt associated with the side-scan interpretation at this specific location.

Figure 2.13 shows the side-scan and sounder records for the region in question, and the undulatory topography is clearly evident. There is no doubt that an active belt of sand runs along the edge of the channel on the Point Sinclair side, the major dispute is in the channel-ward extent or width of this zone. Mid-channel is certainly covered by a broken, shell lag.

Outside the entrance throat, bedformed regions (summarised on Figure 2.11) were found (i) on Marsden Point and flanking the south side of the main channel nearby, (ii) on the western side of the main channel near the eastern end of Mair Bank, (iii) surrounding the main channel to the south and south-east of Busby Head where flow diverges in Bream Bay, (iv) (distinctive, confined bands of finger megaripples) on an east-west transect crossing the outer ebb-tidal delta, and (v) near the 2m contour to the south of Mair Bank.

At location (i) off Marsden Point, casual (not mapped) diver inspection revealed abundant sediment quantities worked into megaripples and sand waves by the tidal currents.

In the strongly, bedformed region to the south and south-east of Busby Head, large-scale sand waves with wavelengths of approximately 50m and backed by smaller megaripple features (Figure 2.14), cuspate megaripples indicative of strong current flows (Figure 2.15) and linear crested mega-ripples, were all found on the subtidal, active flanks of the ebb-tidal delta and main channel.

The fingers of megaripples shown on Figure 2.8, are zonal being surrounded by otherwise featureless sand (Figure 2.16). Bed photographs nearby, but off the megaripple fields (Photo Site Navy 7), showed fine sand overlaid with bioturbation casts with no bedform development and this distinguishes much of the ebb-tidal delta in Bream Bay.

The photography survey revealed sharp-crested ripples typical of those formed by surface-wave, bed-orbital currents along the southern extent of Mair Bank near the 2m contour. The lengths were 0.10-0.15m and heights were 0.03-0.04m. These are noteworthy because Mair Bank is thus attacked by surface waves with sufficient energy to disturb the bed sediments.

2.7.3 The proposed port site

The port site is distinguished by a heavy, shell lag (Photo Site 20) over much of the dredge zone (Figure 2.5) with some patches of sand and interspersed shell occurring offshore from the NHB wharf (Photo Site 16). Video observations at Sites 17-22 on Figure 2.5, were summarised in Black and Healy (1982) as, "Total shelly lag starts near Fix 17. The shell is algae-coated, and some sponge and taller weed was sighted. The lead weight bounced across much of the terrain which was solid and compacted". At Fixes 23-32, "Mainly a shell bed. Some sand patches, especially near 25-26. A fine particulate material was thrown into suspension when the lead weight touched the bed.

Much of the shell has an algae coating. Anenomes were sighted at Fix 31." The side-scan sonar survey revealed no bedforms over much of the dredge zone and the trace was interpreted as, indicating gravel established by a shell lag.

Based upon these field studies the bedload transport rates for the existing bathymetric configuration at the port site is evidently small over much of the region, with little deposition occurring and most sediment passing through as suspended load. Thus, the major loose-sediment reserve is the pathway extending southeast from Snake Bank which crosses the northern side of the dredge zone.

2.8 Shell sand waves

Sounding revealed notable undulations on the channel floor off Marsden Point (Figure 2.17), and this rhythmic topography would normally be interpreted as being due to sand bedforms. For example, working independently with the same sounding record, the DHI (1982, p.17) made this interpretation. However, after examination of evidence by this writer suggesting that the channel floors were shell-covered, diver observations of the bed were undertaken in this region. Positions were fixed with Motorola EDM.

The sea floor was found to be composed of shell (Black and Healy, 1982, Figure 28) with rhythmic undulations forming "shell sand waves", having wavelengths of about 20m and heights of 0.2m. Shell sand waves are not well documented, but Dahm (1983) has identified similar features in Tauranga Harbour. He suggests that they are lower regime features of a sediment which requires much higher speeds for entrainment than typical sands.

2.9 Sea floor implications for sediment transport and inlet stability

There are only minor quantities of loose sediment in any of the expected fast-flow regions. The channel floors are typically lined with shell. Thus, even though the flows may be strong in the channels, appraisal of the sea floor characteristics indicates that total volume of sediment moving in the harbour may be quite small because of the predominance of shell in the deepest parts of the channels.

The banks serve as storage reservoirs which contain most of the loose sand in the harbour, and typically because of the lower speeds on their crests the banks are mostly stagnant. However, they are flanked by narrow distended regions of active megaripples along the channel margins where most bedload sediment transport is evidently occurring. The potential for sediment transport, correctly noted by Healy (1981) after examining water speeds in the vicinity of the proposed port, is shown by this data to be significantly less than expected because of the shell lags.

Because of its abundance in the deep channels, the shell must be important to the overall stability of the harbour and disruption of its growth

could have detrimental effects. Major sources for the shell were noted by Bioresearches (1976) and their distribution map has been reproduced in Figure 2.18.

The most extensive cockle (Chione stuchburyi) bed is on Snake Bank which supports densities of 30mm cockles of up to 848/m^3 (Bioresearches 1980). Mair Bank supports a sizable population of pipis (Paphies australe).

2.10 Heights and wavelengths of bedforms in Whangarei Harbour

Two forms of analysis utilising the depiction of bedforms on the side-scan sonar charts was undertaken. The relationship of height to wavelength was firstly examined and then a probability density function describing the distribution of wavelength in two megarippled regions was sought.

2.10.1 Height to wavelength relationship

Measurements of bedform wavelengths were taken from the side-scan charts and heights from the depth sonographs. Individual bedforms could not be isolated. However, megaripple fields were examined as a group by assigning average wavelengths (Δ) and heights (Λ) to each zone. These values are plotted in Figure 2.19.

Although the data scatters, a clear increase in height with wavelength is evident. Moreover, the data compares favourably with the relationship,

$$\Delta = 0.124\ \Lambda^{0.44} \qquad 2.1$$

for inter-tidal megaripples and sand waves in Cobequid Bay, Minas Basin (Knight, 1977) and supported by data from Mirimichi estuary, New Brunswick (Reinson, 1979).

Notably, the Whangarei bedforms are not inter-tidal which implies that flow depth is not an important parameter even though many researchers (e.g. Allen, 1968) have obtained a correlation. In an estuary, the dominant variable is considered to be the flow speed or excess shear stress (Figure 4.14) which also shows a correlation with water depth. Thus, the casual relationship is more likely to be between flow speed and megaripple steepness. Accordingly, the correlation to the simple power function states as much about the hydraulic character of the three estuaries involved as about the relationship of bedform height to wavelength and suggests that bedforms have a typical steepness in estuaries and tidal inlets.

2.10.2 Distribution of bedform wavelengths

Fifty-one consecutive bedform wavelengths were measured off three transects drawn perpendicular to the crests in a megarippled zone (Field 1) on the sub-tidal spit extending off Snake Bank (Figure 2.12). A second measurement of 48 consecutive wavelengths was also made on a second transect at Fix 588-590 (Figure 2.7), over megaripples off Reotahi Bay (Field 2).

The results are summarised in Tables 2.2 and 2.3. Mean wavelengths were,

$$\Lambda = 5.2 \pm 1.2 \quad \text{Field 1.}$$
$$\Lambda = 4.5 \pm 1.6 \quad \text{Field 2.} \qquad 2.2$$

where the errors are 1 standard deviation and the wavelengths varied over the range (2.8, 7.4m) and (2.0m, 9.0m) for the two cases. Thus, the wavelength is not constant but varies over a wide range in both cases. Accordingly, a probability density function was sought to describe this variation.

The distribution of the measured wavelengths is shown in Figures 2.20 and 2.21 and these were compared with the two density functions,

(i) Normal,

$$f(\Lambda) = \frac{1}{\sigma\sqrt{2\pi}} \; e^{-(\Lambda-\mu)^2/2\sigma^2} \qquad 2.3$$

where μ is the population mean and σ is the population standard deviation.

(ii) Weibull,

$$f(\Lambda) = \alpha\beta\Lambda^{\beta-1} \; e^{-\alpha\Lambda^{\beta}} \qquad 2.4$$

where α, β are unknown constants.

The sample mean and standard deviation were utilised in the Normal distribution. A best fit to the Weibull distribution was undertaken using the method described in Appendix 3. In both cases, a Chi-squared goodness of fit test (Freund, 1974) was applied with the result that both functions fitted the data at the $\alpha = 0.05$ level of significance. The Weibull fit was marginally worse but not significantly.

For Field 2, the best fit Weibull probability density function was,

$$f(\Lambda) = 0.021 \quad \Lambda^2 \; e^{-0.007\Lambda^3} \qquad 2.5$$

The $\alpha = 0.007$ coefficient will vary with the mean wavelength. However, the important parameter which may be general, is the value of $\beta = 3$ representing the peakedness (Kurtosis) of the distribution.

In comparison, the distribution of surface gravity wave periods has been described by Lee and Black (1978) with the Weibull distribution and β was found to average about 1.8. $\beta = 2$ is a special case, known as the Rayleigh distribution and is often used to describe wave heights. The higher value of β over megaripples corresponds with a more peaked distribution.

Raudkivi (1976, p. 89) discusses the treatment of megaripple wavelengths as a stochastic process. He states that frequency histograms of the zero-crossing intervals from laboratory data, can be fitted by a two-parameter gamma distribution. As the wavelengths obtained from the side-scan must nearly approximate the zero-crossing distances, the fit of the data to the Weibull distribution could be expected.

SEDIMENT TRANSPORT AND TIDAL INLET HYDRAULICS

Chapter 3:

BATHYMETRIC STABILITY AND INLET MORPHOLOGY

CHAPTER 3: BATHYMETRIC STABILITY AND INLET MORPHOLOGY

3.1 Introduction

A program of bathymetric soundings was established to ascertain bathymetric stability along selected transects in lower Whangarei Harbour and on Upper Mair Bank. The degree of stability is a measure of the total fluxes of sediment being transported which reflects the general sediment transport characteristics of the region and, more specifically, the region's suitability for port construction. To meet the Chapter's aims, the writer's sounding program, as well as two complete RNZN bathymetric surveys performed in 1959/61 and 1981 (including all of the Mair Bank ebb-tidal delta) are compared.

In addition, the relative importance of tide and wave effects on the throat geometry, and inlet morphology are assessed. After an analysis of the entrance throat cross-sectional parameters, a suitable location of the quay wall for the proposed timber port is determined. The position minimises the effect of the construction on the mean cross-sectional flow velocities.

3.2 Field design of sounding program

Soundings were made along representative cross-sections from the University of Waikato jet boat with a Raytheon 719B depth sounder. Single sextant angles were shot at 30-60 second intervals for positional information. The tide levels were read from the NHB gauge on the Marsden Point wharf, and soundings were reduced relative to the Marsden Point Chart Datum (CD). During the survey days, 1-Jul-81 and 3-Jul-81, the weather was calm with wind speeds of less than 3 $m.s^{-1}$ recorded on the Marsden Point anenometer at 13m above MSL.

As some of the sounding lines were more than 4000m long, survey accuracy and repeatability was ensured by the use of marker piles placed on Snake, McDonald, Mair and Calliope Banks. Mr J. Harvey, on the NHB barge "Maui", placed the piles at buoyed locations previously positioned by a person on the bank in radio contact with a shore-based theodolite operator sighting along the transects. The precise poisition of each pile was later obtained by the NHB surveyor Mr E.N. King, using Mini Ranger electronic distance measuring equipment.

3.3 Data reduction

3.3.1 Sounding chart reduction

The formidable task of reducing, plotting and analysing sounding records covering more than 40km of the harbour floor was managed by application of the software package developed by this writer and described in the most part in Black (1983). The programs employ the digitising capability of the NHB Hewlett Packard 7221C plotter and for ease of manipulation, cross-sections are stored on computer disc for later retrieval and analysis. The capabilities

incorporated in this package were:

(i) automatic disc file production which was compatible with other management and analysis programs.

(ii) the option to add positional or general information to any depth data point.

(iii) the tide level and transducer depth could be entered for the computer to produce a file of depths below Chart Datum.

(iv) a rotation of the axes correction, which assessed when the chart was not placed horizontally on the plotter.

(v) variable density of digitised depths which could be changed to suit the complexity of the bathymetry.

(vi) no restriction on vertical and horizontal sounding chart scales.

(vii) an ability to handle a change of depth range set on the sounder.

(viii) options on the units of depth (feet, fathoms, metres).

(ix) ability to reduce soundings on straight lines or on arcs.

Supplementary support programming allows the operator to:

(i) enter depths at the keyboard if required.

(ii) plot the data as a cross-section or in the format of a bathymetric chart.

3.3.2 Cross-section preparation and determination of relative statistics

Cross-sections were taken from bathymetric maps for comparison with the writer's sounding cross-sections and for general analysis. This was undertaken with the writer's Map Digitising Computer Package and cross-sections were manipulated with an extension to the package - Sounding Line Cross-section Analysis (Appendix 2). In most cases, the map projections were Transverse Mercator and digitised coordinates were converted to the Mt Eden Metric Grid as described in Appendix 1.

To ascertain cross-sectional areas, wetted perimeters, and hydraulic radii, Program CHANMN was specially written to find these parameters below selected depths and between selected horizontal coordinates. Areas were obtained numerically by dividing the digitised cross-section into narrow strips of width Δ_ω. Thus,

$$\Delta A_i = d_i \Delta_\omega \qquad 3.1$$

where d_i was the mean depth in the strip found by linear interpolation of bounding known depths and ΔA_i is the area of the strip. The total area, A, was a summation between user-selected horizontal coordinates, where for n strips,

$$A = \sum_{i=i}^{n} d_i \Delta_\omega \qquad 3.2$$

The wetted perimeter was found from the right angled triangle which can be constructed using the base of each strip as the hypoteneuse. The length of the hypoteneuse was obtained from the gradient of the linearly interpolated line joining the bounding measured depths. The wetted perimeter was the sum of these lengths.

3.3.3 Reduction of sextant fixes

The boat positions were found from the simultaneous solution of the intersection of a circle (described by the sextant angle with a straight line (the boat path). The central coordinates and radius of the circle defined by the sextant fix, are derived in Appendix 1. The circle has general equation,

$$(E - a)^2 + (N - b)^2 = r^2 \qquad 3.3$$

where (a,b) are at the centre, r is the radius. N and E are north and east coordinates respectively. The straight line can be represented as,

$$N = m.E + c \qquad 3.4$$

with gradient, m, and intercept, c, to be determined from the known location of the sounding line.

Introduction of 3.4 into 3.3, and after algebraic manipulation produces the quadratic expression,

$$A.E^2 + B.E. + C = 0 \qquad 3.5$$

where,

$$C = a^2 + b^2 + c^2 - r^2 - 2bc, \quad B = 2mc - 2bm - 2a, \quad A = 1 + m^2 \qquad 3.6$$

with solution,

$$E = (-B \pm \sqrt{B^2 - 4AC})/2A \qquad 3.7$$

Substitution into equation 3.4 gives,

$$N = (-mB \pm m\sqrt{B^2 - 4AC})/2A + c \qquad 3.8$$

Computer solution of equations A1.2, A1.4, 3.4, 3.7 and 3.8 identified the boat positions in Mt Eden Geodetic Metric Coordinates.

3.4 Error limits

There are several factors which can cause errors, some of which are general, while others are specific to the writer's survey. They belong to two categories.

(i) sounding errors: These can be caused by: (a) sea surface gradients. One-dimensional modelling (Chapter 9) revealed that during Spring tides 0.15m differences in sea level between One Tree Point and Marsden Point occur at high tide. This can establish a 0.3m sounding error as the deviation is positive on the flood and negative on the ebb; (b) inaccurate transducer depths which vary with the speed and planning lift of the boat; and (c) instrument deviations.

(ii) Position fixing errors: These can be caused by: (a) sideways movement

of the boat off the straight line course in transverse currents. This produces an apparent forward or backward position shift along the assumed course because of the position fixing method of solution; and (b) sextant angle inaccuracies.

From the above, the case (a) errors were estimated to be ±0.5m. In rough agreement with this, Lt. Cdr. J. Stoakes (Hydrographic Office, RNZN) was quoted (DHI, 1982) as estimating this error to be about ±0.3m. Type (ii), case (a) errors can be very significant, especially on steeply-sloping topography where deviations are more important and where course-longitudinal, current gradients from banks to channels hamper accurate boat handling.

Sounding lines L6H and L6M (Figures 3.8 and 3.15) were sounded twice to test the accuracy and repeatability of the experimental procedure. The major error was due to horizontal alignment inaccuracies arising out of poor position fixing. Sextant angles were then examined by comparing the ratios of change in consecutive angles over the distance moved along the chart and abnormal fixes were dropped. Under this test, more than 60% of all fixes were rejected for suspected or known inaccuracies. This figure is apparently very large, but it is noted that in many cases where fix accuracy was at all suspect, the fix was rejected. After this, if fixing information was unavailable, no comparison with a prior Navy survey was made. The remaining data is considered to be accurate within the usual error limits of sounding surveys. This is assured mostly because, where comparisons were made, the position fixing depends primarily on the accurately-fixed locations of the sounding piles. Placed on the banks at intervals along the cross-sections, these effectively reduced the horizontal position fixing errors.

3.5 Other available bathymetric surveys and comparisons made

The RNZN Hydrographic Branch undertook complete bathymetric surveys of (i) Mair Bank and environs from September to October, 1959; and (ii) the One Tree Point to Marsden Point region from October to December, 1961. These were part of the NZ5213 series, used by the RNZN to compile the 1974 bathymetry map of Whangarei Harbour and the Mair Bank ebb-tidal delta. The data is referred to as the "early" Navy survey in this Chapter. After the writer's sounding program, the RNZN undertook another complete bathymetric survey of the study region in late 1981 from just west of One Tree Point to beyond the southern and western limits of the Mair Bank ebb-tidal delta in Bream Bay.

In this Chapter, three types of bathymetry comparisons are made:

(i) cross-sections taken off the early Navy bathymetric charts are compared with the writer's sounding program. The former were obtained from charts at 1:10,000, made available by NHB draughtsman Mr Bob Aickin in a Transverse Mercator Projection. They were converted to Metrics using the programming previously described.

(ii) using the 1974 Navy chart (NZ5213), photographically enlarged to 1:10,000 by the RNZN and made available for this study, marked depths are compared with the 1981 survey. After consideration of the error limits, a region was defined as stable if the absolute change in depth was less than 0.3m. Erosive and accretive zones exhibiting changes greater than or equal to 0.3m were noted and contoured. As is always the case, this method of depth comparison has a particular weakness. The confidence in the result depends on the accuracy of the photographic enlargement, copying distortions and tracing paper stretching or compression. These deviations are usually insignificant over flat terrain, but on the steep, channel flanks, small positional errors may introduce misleading results. In these cases, a comparison was not made. The procedure is described in the chapter as the "depth comparison".

(iii) isobaths from the 1981 RNZN survey and the 1974 chart (NZ5213) were computer digitised for comparisons. This extra analysis was performed mainly because of the potential errors in the previous method, and a thorough verification was sought. The digitised data contains none of the inherent weaknesses of the photographically enlarged overlays, as paper distortions are mostly corrected for by the computer program. Thus, digitising the old bathymetry, specifically for the comparison, was considered to be worthwhile because of the accuracy of the method. The procedure is described in the chapter as the "contour overlay" or "contour comparison".

Notably, the early Navy depths were charted to the nearest foot for depths less than 10 fathoms, and to the nearest fathom at deeper locations. Thus, the error limits are ±0.1m and ±1.0m for respective depths being less or greater than 18m.

3.6 Sounding program and analysis summary

Thirteen representative cross-sections transecting the major banks and channels between One Tree Point and the Mair Bank ebb-tidal delta were chosen for the writer's sounding program. Cross-section locations with the positions of the sounding piles are shown on Figures 3.1 and 3.2. Each cross-section was defined relative to selected origins and the Mt Eden Geodetic Metric Coordinates of these are listed in Table 3.1 The gradients and intercepts of the equations defining the sounding lines (equation 3.4) are given in Table 3.2. Figures 3.3 to 3.15 show the sounding cross-sections and the comparisons with the early Navy survey. Cross-sectional areas below CD are listed in the Figures.

The depth comparisons are shown in Figure 3.16 (inner harbour) and Figure 3.17 (Mair Bank and northern Bream Bay). The contour overlays are presented in Figures 3.1 and 3.2 for the same regions and the writer's survey lines were included so that direct comparisons could be made.

It is noted that for the following discussions and throughout the thesis Mair Bank has been divided into three regions as shown on Figure 1.2. That is, (i) upper Mair Bank, which includes Bank levels above the 3m isobath, (ii) central Mair Bank, which is bounded by the upper Bank to the north and extends to the south opposite Busby Head. (iii) southern Mair Bank, which includes the sub-tidal expanse of the ebb-tidal delta south of Busby Head.

3.6.1 Sounding program results and comparison with the early Navy bathymetric survey

On sounding Line L1H, dredging of the Shell Cut Reach shipping channel near One Tree Point and the dumping of spoils in the scoured hole off Reserve Point, highlight the bathymetric changes seen in Figure 3.3. In addition, approximately 0.3m of erosion has occurred on both the gently-sloping (0.5^{o} gradient) southern face and the crest of Snake Bank. This erosion may have resulted from an alteration to the local hydraulic regime as a consequence of the dredging. Notably, the change is mostly within the sounding error limits of 0.3m. Bedforms depicting sediment transport activity, are evident on the shipping channel floor and its One Tree Point flank.

On Line L2H, the bank crests have apparently eroded in the 20-year interval between surveys (Figure 3.4). However, this erosion was not confirmed by the comparison of RNZN surveys nor is it evident on Line L3H (Figure 3.5). The Shoal Bay channel remained unchanged.

Line L3H is almost wholly unchanged within the sounding limits except for a significant scouring of the sand flat near Point Sinclair and small changes to the detail of the cross-section. The Shoal Bay channel is stable. Megaripples envelop the 5m contour on the southern side of Shoal Bay, and are associated with the active sediment pathway verified by divers around the northern flank of Snake Bank (Chapter 2). Others top McDonald Bank and flank both steeply-sloping sides of the shipping channel at 10m below CD. The gradient on the northern side of this channel is 10.9^{o} which is about one third of the sediment repose angle of approximately 30^{o}.

The Shoal Bay profile on the Snake Bank side, exhibits adjoining channels isolated by intervening deposition (near 2200m), and is typical of cases when ebb and flood currents occupy neighbouring regions (Ludwick, 1974). Between these lies a neutral zone, evidenced by the ridge of sediment. A second example is seen on Figure 3.6 with the ridge at 700m. A further significant geomorphic feature is the sub-tidal horizontal platform below McDonald Bank near 2800m (Figure 3.5). This platform may have developed as a result of erosion retardation over semi-consolidated sediments as proposed by DeAlteris and Byrne (1975). Their model applied to this particular case suggests that the shells retard the scour on the platform, so preferential horizontal erosion of the less-resistant bank sediments occurs instead. The side-scan

sonar trace along the platform was interpreted as gravelly. A similar platform constituted of scoured shell lag, encompasses a large area in front of and up-harbour from the NHB wharf, with depths of 8-9m throughout (Figure 3.6). Its genesis may be similarly explained.

The cross-sections in the entrance throat and environs (L4H to L7H, Figures 3.6 to 3.9) exhibit great stability, testifying to the sedimentary balance in the region and/or to the "inertia" of the shell beds which do not respond easily to change. Notable among what small differences there are, is a deepening near the NHB wharf, (Figures 3.6 and 3.7), which may have resulted from increased ship-induced turbulence or the altered hydraulic regime with the presence of the ships and wharf. Erosion of 1.3m in mid-gorge on L7H is nearly within the 1961 survey accuracy and as neighbouring regions are very stable, this change is not considered to be significant.

On L4H and L5H the strongly bedformed sand ridge off Darch Point was previously identified during the side-scan sonar survey and is a continuation of the sediment pathway along the north-east of Snake Bank at the 5m contour. The adjoining scour is related to flow convergence as the tortuous current is forced to turn by the imposition of Darch Point. Flow into a bend converges towards the outer bank and can excavate a deep hole into the bed (Raudkivi, 1976). Similar scour also occurs at Reserve and Home Points.

Outside the entrance, the western end of upper Mair Bank and the marginal flood channel separating it from Marsden Point, have accreted with level changes of more than 2m in places (Figures 3.10, 3.11, 3.13 and 3.14), reducing the distributary channel width at CD by more than 50% along section L2M (Figure 3.11). However, depths remained relatively unchanged across the eastern end of the Bank (Figure 3.11) which is totally covered by a shell-lag bed. The accretion on the western upper levels is reflected by erosion to the south of upper Mair Bank between the 0.5m and 2m contours (Figure 3.14). Further south again, in 2-3m of water, L5M exhibits great stability (Figure 3.15).

The main channel along the eastern end of Mair Bank, has moved eastward below depths of 10m with accretion on the Marsden Point side and erosion below Calliope Bank (Figure 3.11). This was confirmed by the comparison of Navy surveys. On all relevant cross-sections, the mid channel depths have generally not changed by more than the ±1.0m accuracy limit of the early Navy survey.

Notable features on the cross-sections are bedforms off Marsden Point at CD (e.g. Figure 3.11), and the shape of the main channel floor across sections L1M, L4M, and L5M (Figures 3.10, 3.13 and 3.14), which is deepest at the edges with a sediment "hump" between. The latter is reminiscent of a profile subjected to opposingly dominant currents on opposite sides, having a relatively, accretive zone between.

Cross-sectional areas throughout are commonly 12,000-14,000m^3, noting, however, that in some cases the section is not flow normal.

3.6.2 Comparisons of the 1959/61 and the 1981 RNZN surveys

3.6.2.1 *Inner harbour*

A large area of moderate deepening (0.3-1.0m) involving the shipping channel and its flanks from the north of Blacksmiths Creek up to One Tree Point and the western end of Snake Bank (Figures 3.1 and 3.16) is evident on both the depth and contour comparisons. Limited accretion along the south-western side of the channel is also suggested by the comparison of the 5m contours in this region (Figure 3.1), though not evident on the depth comparison. The contours show that the channel has moved north-eastwards by approximately 50m at the 5m isobath.

The erosion on the sand flat at Point Sinclair and the general erosion in the channel and off Snake Bank are evident on all surveys. In the shipping channel, some of the large changes in depth are directly due to dredging. The neighbouring bed level differences however, are most likely associated with the adjustment of the bed to the changed hydraulic regime after dredging.

In front of the NHB wharf and in the entrance gorge, there are no significant changes to bed levels, as was also found by the author's sounding program. There appears to be some erosion around Passage Island, and on the south-easterly Snake Bank extension within the 5m contour but no change at deeper sites (Figure 3.16).

The Shoal Bay channel is extremely stable - the 5m contour (Figure 3.1) is relatively unaltered, within the survey accuracies, and the mid-channel depths exhibit no change (Figure 3.16). Large-scale accretion occurs in the scoured hole off Reserve Point, but this has already been identified as a dumping ground for dredge spoils from the Shell Cut Reach channel. There is patchy erosion around McDonald Bank on the north-west and southern ends near CD, noted by both comparisons. However, much of the bank is unchanged.

In summary, the depth and contour comparisons, show that there has been a nett loss of material such that the harbour from the entrance gorge to One Tree and Reserve Points is not infilling but eroding slightly. It is still possible that the upper reaches of the harbour are gradually accreting by the addition of fine sediments consolidated by mangroves and grasses, but there is no bathymetric evidence that the lower harbour is subject to accretion. This observation accords with the work of Barwis et. al. (1977) who noted that tidal inlets along the eastern coast of the U.S.A. were only long-term sediment sinks when the inlet migrated alongshore providing new depo-centres for flood-tidal deltas. Entrance stability at Whangarei has been identified.

3.6.2.2 *Mair Bank, environs and Bream Bay*

The erosion/deposition processes identified in Bream Bay are complex with

bed level changes being strictly confined. Upper Mair Bank has shifted. Between the 1959 and 1981 surveys, its longitudinal axis has rotated by about 30^{o} to become aligned nearly east-west from a more north-east to south-west orientation. The western end of the upper Bank has moved by 400-500m northwards at CD. This is reflected by considerable accretion (as much as 1.6m) on the western and northern ends of the bank, and by deepening of up to 1.5m evident along the southern flank between the 0m and 3m contours. At the eastern end of the upper Bank, the bed levels and contours are virtually unchanged. These trends concur with the results of this writer's sounding program.

On Calliope Bank the bed level across its crest has generally lowered (Figure 3.17) with the large, central portion above CD on the 1974 chart having almost completely disappeared (Figure 3.2).

The shipping channel has moved north-eastward with accretion on the Mair Bank flank and erosion around Calliope Bank. Because of the steepness of the channel walls, depth changes exceed 4m in places and these are the largest, exhibited changes in the study region. The contour comparison shows that on the channel curve near the south of Calliope Bank, the 20m depth contour has moved by about 75m in places. Furthermore, according to the contour overlay, the accretion around Mair Bank at the 15m contour continues until just south of Home Point. The erosion on the opposite, eastern side of the channel extends further south, nearly to Busby Head. Some considerable depth changes (1.5m) occur between Home Point and Busby Head.

South of Busby Head flanking the main-channel margins, general erosion is evident with changes of up to 1.5m, over a large area of bed (Figure 3.17). Shoreward of this on the southern part of the ebb-tidal delta mostly below the 4m contour, a large zone of accretion extends some 2000m out from Bream Bay Beach. The change involves considerable volumes of sediment. Just over the accretion shown on Figure 3.17 - and its full, southerly extent is not known - about 2×10^{6} m^{3} of sediment has been deposited. This corresponds with a minimum of nearly 100,000 m^{3}/year, assuming sedimentation has occurred during the complete 22-year period between surveys. Adjacent to the accretion, Bream Bay Beach has eroded with depth changes of more than 1m. Much of central Mair Bank exhibited general stability.

The significant changes are: (i) erosion of the shipping channel and its environs near One Tree Point. This is almost certainly associated with dredging of the Shell Cut Reach Channel; (ii) the realignment of upper Mair Bank and accretion around its northern and eastern flanks. Calculations show that most of the accretion on the crest can be balanced by corresponding erosion on the seaward, southern flanks of the upper Bank between the 1m and 3m contours; and (iii) the erosion on the main channel margins in Bream Bay and the accretion on southern parts of the Mair Bank ebb-tidal delta adjacent to Bream Bay Beach. It appears that sediment has moved shoreward from the offshore

channel margins.

Using the same Navy bathymetric surveys, the DHI (1982) independently ascertained volume changes on the Mair Bank delta within chosen contour levels (reproduced in Table 3.3). As noted above, it is evident that accretion on the crest of the Bank is nearly balanced by corresponding erosion between the 1m and 3m contours. Furthermore, as expected, the accretion between the 4 and 7m contours is evident. Their analysis only included depths less than 10m and offshore erosion on the channel margins was not identified.

This writer's Figures 3.16 and 3.17 summarise the locations and the magnitudes of the bathymetry changes throughout the study region.

3.7 Beach surveys and string measurements in the inter-tidal zone

A series of 20 cross-sections across the inter-tidal sand flats between the western end of Blacksmiths Creek, along the port site and around Marsden Point (Figure 3.18) were surveyed between 19-Jun-81 and 11-Mar-83. To determine the relative stability of the inter-tidal zone, Sections 1-14 (Figures 3.19 and 3.20), surveyed on 14-Oct-81, were compared with a previous NHB survey of these sections conducted two years earlier on 9-Oct-79. Sections 15-20 (Figure 3.21) were surveyed six times during the period quoted above, and were established to investigate the effect of waves and currents on the bathymetry of the distal tip of Marsden Point at the entrance to Whangarei Harbour. The survey posts marking the dashed cross-sections 15, 16 and 17 (Figure 3.18) were lost before they could be position fixed and approximate locations are shown.

In conjunction with the beach surveys, nylon strings (approximately 5mm in diameter and attached to buried flat plates) were deployed at selected locations in the inter-tidal zone (Figure 3.18). The strings, when pulled tight vertically, allow measurement of bed level changes along their length. They serve as a safe, graduated pole requiring little or no maintenance and are not easily lost unless interfered with by the public. This survey procedure, designed by McLennan (1982),proved very successful. At the port site, strings were serviced from 7-Jun-81 to 16-Aug-81 and levels were plotted on Figures 3.22 and 3.23. On Marsden Point, they were checked mostly at weekly intervals during the 10-month period from 23-Apr-82 to 11-Feb-83, and results are shown on Figure 3.23. Concurrently, measurements of the beach levels at three pylons of the NZRC wharf were made (Figure 3.24, Pl.1 to Pl.3). Survey data reduction and plotting was performed by Mr W. King, of the NHB civil engineering staff.

3.7.1 Beach survey and string results

Comparison of the two inter-tidal surveys reveals no significant change at the port site over the two-year interval (Figures 3.19 and 3.20). There is a very slight tendency for the upper beach levels to be retreating on Profiles

1-8 with a maximum of ≈1m on Section 1. The strings (Figures 3.22 and 3.23) exhibit the same behaviour with the maximum recorded bed level difference being 0.12m of erosion. More typically, the changes were less than 0.05m. Thus, the data reveals a high level of stability on the inter-tidal sand flats through the proposed port zone.

The Marsden Point cross-sections, however, show a wider degree of variability with horizontal movement of the berm crest of up to 50m causing depth changes of approximately 1m in places (Figure 3.21). String S3.5, located on the berm, exhibited a maximum bed level difference of about 0.8m (Figure 3.24). Changes to the berm were typified by slow accretion interrupted by abrupt erosion. Further down the profile on the lower beach levels, depth changes were less than 0.4m with the strings oscillating within a 0.2m range of the initial value. Cross-sections surveyed on 6-Jul-82 (Figure 3.21) are typical of "winter" or storm beach profiles (King, 1972). The berm is more well-developed on surveys conducted at other times and exhibits the swell or "summer" condition.

The measurements at the NZRC wharf pylons, were similar to, but out of phase with, the measurements on the berm at Marsden Point (Figure 3.24). Especially at the lower beach levels (Pl.1), slow sedimentation was reversed by one erosive event which lowered the beach level by 0.9m. Upper beach levels oscillated within 0.3m of the initial value, and exhibited a maximum variation of 0.5m.

It is not within the scope or purpose of this chapter to interpret the above results but only to examine the magnitude of changes that occur in these two important regions. An interpretation of the Marsden Point data is given in Chapter 4, and a more detailed examination of the inter-tidal zone of the port site is undertaken in Chapter 8.

3.8 Sediment transport directions inferred from the bathymetry

McCave and Longhorne (1982) found that bank steepness and sediment transport directions were correlated with sediment moving over the bank crest from the gentler to the steeper slope where accretion occurs. By implication then, sediment is moving onto the north-west ends of Snake and McDonald Banks which both have low gradients. After crossing McDonald Bank, sand is spilled into the channel around McLeod Bay. Sediment transported across Snake Bank is deposited on its Shoal Bay flank where the gradient steepens.

Of interest is the implication that sediment is moving off Snake Bank between Beacon S1 and the Shell Cut Reach Front Lead (Figure 1.1) where the shipping channel is constricted between the bank and the flood tidal delta fronting Point Sinclair. The bed gradient is very steep in this region. Calliope Bank is subject to sedimentation with a north-easterly heading.

3.9 Entrance throat and port site bathymetry

Ten cross-sections, perpendicular to the mid-channel depth contours and located across the entrance gorge between Lort Point and Darch Point, were digitised for channel cross-section analyses. The cross-sections were taken from the 1981 Navy Chart (NZ5213) plotted at 1:10,000. The data were used to examine throat shapes and throat stability and were employed to locate the quay wall such that the proposed timber port should have minimum effect on the tidal inlet hydraulic regime after construction. Cross-section locations are shown in Figure 3.25 and the cross-sectional profiles are presented in Figures 3.26 and 3.27. The results of the statistical analyses giving areas etc. are listed in Table 3.4.

3.9.1 Inlet throat geometry

Fitzgerald and Fitzgerald (1977) asserted that the shape of an inlet throat is related to three controlling factors which, in order of importance are: (i) the meandering of the channel thalweg (ii) the shoreline configuration and (iii) the dominant longshore transport direction. They defined the inlet throat, or inlet gorge, as that "part of the channel which is narrowest and deepest and has the maximum hydraulic radius. The throat has the minimum cross-sectional area, which produces the maximum current velocities and the greatest potential sediment transport" (Fitzgerald and Fitzgerald, 1977, p.564). Unfortunately, at Whangarei, some of these requirements are exclusive with the deepest profile (Table 3.4) not being the narrowest, but Profile 1 on Figure 3.26 has the narrowest width and largest hydraulic radius and this will conveniently be defined as the inlet throat.

Sediment outside the entrance at Marsden Point can only approach under wave attack from the beach to the south. According to Fitzgerald and Fitzgerald, where longshore transport is important, the throat will have a gently-sloping bank where sand is entering and will be steeper on the opposite side. Examination of Profile 1 (Figure 3.26) reveals that at Whangarei the opposite is occurring implying that not enough sand enters the harbour on the south side for this longshore component to be a principal mechanism.

The channel thalweg at the throat is approximately parallel with the northern, bedrock shoreline (Figure 3.25). The thalweg meander and the shoreline configuration are then evidently coupled, with the former resulting from the latter. Thus, the shape of the inlet throat according to the assertions of Fitzgerald and Fitzgerald, primarily results from these interrelated, controlling factors.

In summary, the shape of the tidal inlet throat at Whangarei is not significantly influenced by the input of sediment from littoral drift, but is primarily determined by the configuration of the northern shoreline. This implies that the gorge is tidally-dominated.

3.9.2 Inlet throat area with relation to empirical, predictive equations

Healy (1981) found that the entrance throat area at Whangarei Harbour was greater than that predicted by the empirical relationships of Heath (1975) and O'Brien (1931). This was confirmed by this writer and it was found that the O'Brien relationship under-predicts by 26%. For a further verification, the equation of Johnson (1973) was tested and it too finds a smaller area, which was this time 31% less than the measured value. The immediate implication is that the throat should be subjected to continued sedimentation in order to reach the predicted "equilibrium", and yet the sounding program accents the great stability of the tidal inlet throat, with no evidence of accretion over a 20-year interval. Instead, the wider entrance highlights a particular characteristic of Whangarei Harbour, initially examined in the previous section, i.e. throat sheltering from wave attack by Busby Head, and the correspondingly small littoral drift.

A tidal inlet is subjected to two opposing mechanisms. Ebb-tidal flow, which is directly related to the tidal prism, tends to open an inlet, whereas wave energy, by means of littoral drift, tends to cause closure. O'Brien (1967) compared two inlets with nearly the same potential prism and, because of wave exposure, one was generally closed and the other was always open. Further to this, Johnson (1973) concludes that wave power appears to be the single-most important factor affecting the stability of tidal inlets. Thus, the bathymetric stability at the entrance of Whangarei Harbour may be attributed, at least in part, to the physcial sheltering from waves by Busby Head. The greater cross-sectional area evidently results from the unusual balance of wave attack against ebb-tidal flow.

Other than the interaction of waves and currents at the entrance throat, the anlaysis at Whangarei suggests that differences in bed erodibility immediately inside and outside the entrance may be an important second mechanism which can assist the maintenance of throat stability when the littoral drift is low. Specifically, outside the entrance near Marsden Point, the bed is loose, sandy and bedformed, whereas, inside the throat, the shell content is far greater (Chapter 2). Sand can thus be easily moved into the harbour, but not as easily swept out again. The bed differences may then offset a tidal-current asymmetry. As the volume of sediment entering under wave attack is low, it is likely that the shell lag lining the entrance throat prevents further erosion.

3.9.3 Entrance gorge stability criterion

The ten cross-sectional profiles in the entrance gorge, Figures 3.26 and 3.27, exhibit much variation in the profile shape. Particularly, as the gorge widens towards Darch Point, the maximum depth shallows considerably. Moreover, examination of the cross-section statistics in Table 3.4 reveals wide

variation in the width and hydraulic radius of the profiles. However, even with these large shape differences, the cross-sectional areas are seen to be very constant (Table 3.4) excluding Sections 9 and 10 which are not flow perpendicular for much of their length near Darch Point. The mean cross-sectional area for Sections 1-8 was 13151m^2 below CD, with a maximum deviation of only 3%. Notably, similar results are evident for the area below MSL (1.5m). Moreover, the area constancy necessarily implies a linear relationship between width and mean depth, but further to this, Figure 3.28 shows that the width of the cross-section is also strongly correlated with the maximum depth in the profile.

Thus, the data provides an effective stability criterion for the entrance gorge in Whangarei Harbour, that is, cross-sectional area should be equal to 13151m^2 below CD. As already discussed in the previous section, the stable cross-sectional area is related to the tidal prism, regional bed sediments and the littoral sediment input, and is specific to this particular entrance.

3.9.4 Positioning the quay wall of the proposed timber port

The previous results offer a highly effective method for determining the offshore extent of the timber port - the quay wall should be placed so that the cross-sectional area after port development equals 13151m^2. Discussions with the Northland Harbour Board Civil Engineering staff, revealed that likely dredge-zone specifications (which are dependent on shipping requirements) will be a basin of 13m depth and a width of 400m. The quay wall is to have a gradient of 1:1½. With these specifications, the cross-sectional area depends on the chosen location of the quay wall only.

Program CHANMN can be stepped back and forth along the cross-section in a controlled inerative fashion until a desired cross-sectional area is obtained. This operation was performed for Section 7, which runs through the centre of the port and the position of the quay wall was determined and shown on Figure 3.25 and is 520m from the beach or at the location (815,986mN, 274,953mE).

The alignment of the quay wall, can be estimated from measured flow directions in the vicinity assuming (i) that the construction will have minimum effect on the present regime and (ii) that the quay wall and flow direction should be parallel. After examination of measured flow velocities (Chapter 5) in the vicinity, the alignment was determined to be such that the normal to the front of the port should be at 25± 5° True, with the mean of 25° T plotted on Figure 3.25.

The new cross-section after port construction and dredging is shown in Figure 3.29 with the present bathymetry for Profile 7. The changes to the shape are considerable, noting especially the removal of the ridge of active sediment at 500m. It is unlikely that the constructed profile will not seek a new equilibrium. Even though the areas have been kept constant, the profile

width has been considerably reduced from 1269m to 970m. The strong correlation between width and maximum depth for the Whangarei entrance was noted in the previous section and application of Figure 3.28, suggests that the maximum depth in the profile should increase from 16m to approximately 29m. Furthermore, in order for the profile to accomplish this while maintaining constant area, it is necessary for accretion to occur somewhere else in the cross-section. The data strongly supports the contention that the depth should increase, but the location of corresponding accretion cannot be predicted with as much confidence. However, examination of the existing profiles, suggests that the accretion would most likely occur in the form of a platform at the southern or port end of the cross-sections. Thus, maintenance dredging may be required there.

The DHI, using a 2-dimensional numerical computer model, compared several different port configurations. As a result of this, discussions with the NHB Civil Engineers revealed that Case A7 (DHI, 1982) was a possible choice for the port. Design A7 places the quay wall within 40m (shoreward) of this writer's position with the normal angled at 24° True (c.f. 25°T). It is stressed that this work was undertaken independently of DHI and that this writer's results were published in an internal NHB report before the DHI investigations were completed (Black, 1982).

The agreement, which is within the DHI model accuracy limits defined by the grid size of 50m can be expected, in that numerical modelling techniques, which ensure that the flow speeds are minimally affected by the construction, rely inherently on the need to keep the cross-sectional areas constant, so that mean speed in the cross-section is unchanged. The modelling allows a better inspection of the 2-dimensional effects throughout the dredge and neighbouring zones. However, the degrees of freedom associated with the placement of the quay wall are limited to an on/offshore translation and/or a rotation, so that average effects as well as the specifics must be accounted for to ensure minimum overall changes to the speeds in the cross-section. Nevertheless, leaving the speeds unchanged after construction of the port does not guarantee that accretion or erosion will not occur in the cross-section. The profile width is also a dependent variable.

Note: Since these calculations were made, dredge zone specifications of 450m by 14m were suggested. With this requirement, repitition of previous analysis showed that the quay wall would need to be only 22m further offshore along Cross-section 7 than the position quoted in the text.

3.10 Conclusions

(i) The major zones of erosion in the inner harbour are along the shipping channel between One Tree Point and the north of Blacksmiths Creek including the Point Sinclair inter-tidal sand flats and the entire western end of Snake

Bank across to the Shoal Bay channel. This is evidently occurring in response to the new hydraulic conditions after dredging of the Shell Cut Reach channel. Patchy erosion was recorded around McDonald Bank especially on the north-west and the south to south-east flanks.

(ii) The entrance gorge and the proposed timber site exhibited great stability between 1961-1981.

(iii) Shoal Bay channel is essentially unchanged over the 20 year interval between surveys except for accretion off Reserve Point, which results from the dumping of dredge spoils there.

(iv) The inner harbour up to One Tree Point has exhibited nett erosion and is not infilling.

(v) Outside the entrance on Mair Bank and nearby, the changes are more significant with erosion along the main channel flank of Calliope Bank; along the south of Mair Bank near the 2m contour; along Bream Bay Beach; and flanking the main channel south-west to south-east of Busby Head. Accreting zones are on the western end of Mair Bank and in the distributary channel off Marsden Point; along the northern flank of Mair Bank in the main channel; and over a large area of the outer ebb-tidal delta, mainly below 4m and fronting Bream Bay Beach. Some of the depth changes are more than 4m in the main channel near the mouth of Urquhart's Bay, and are more than 2m on the crest of Mair Bank.

(vi) Beach surveys on the inter-tidal zone between the NZRC wharf and the north of Blacksmith's Creek revealed great stability over a two-year period. Beach measurements on Marsden Point, indicate typical 20-30m horizontal movements of the berm crest with corresponding bed level changes of 1m. The lower beach levels varied by less than 0.4m.

(vii) Examination of inlet throat geometry revealed a tidal dominance (as against a wave-controlled environment). The entrance throat area is greater than predicted by empirical relationships interpreted as resulting from wave sheltering of the entrance by Busby Head, which again highlights the tidal dominance of the entrance.

(viii) Although there were wide variations in the width, maximum depth and shape of entrance gorge cross-sections, the area was constant at 13151m^2 with less than 3% variation for 8 representative profiles. The maximum depth in the entrance gorge was shown to be inversely related to the cross-section width.

(ix) In view of the constancy in cross-sectional area, the offshore location of the timber port quay wall may be placed so that the cross-sectional area after construction remains unchanged and should thus be located at (815,986mN, 274,953mE). The orientation of the normal to the quay wall is suggested from measured current flow directions to be 25± 5^{o} True. Later work by the DHI on port configuration A7 proved to be within 40m and 1^{o} of the

above location, which is within the limiting grid size of 50m used in the mathematical model.

(x) The strong correlation between maximum depth and width of the cross-section, implies that the cross-section maximum depth should increase from 16m to 29m after construction. In order to maintain the constant cross-sectional area, this should be compensated for by accretion elsewhere in the profile. After examination of the existing bathymetry, it is suggested that this should occur at the southern (or port) end of the dredge zone. Maintenance dredging may therefore be required.

SEDIMENT TRANSPORT AND TIDAL INLET HYDRAULICS

Chapter 4:

BREAM BAY WAVE PROCESSES

CHAPTER 4 BREAM BAY WAVE PROCESSES

4.1 Introduction

Although most of the harbour investigations centred on the tidal hydraulics, it was necessary to specifically include wave effects in some detail at two locations, namely: (i) in northern Bream Bay because of the intermittant, high-energy wave events there and (ii) at the timber port site on the inter-tidal flats (discussed in Chapter 8) where trapping experiments showed that sediment transport was significant only under wave action.

For this Chapter, which deals with the first of these two locations, four major aims were identified: (i) to include wave-induced transport in Bream Bay sediment circulation patterns. Specifically to ascertain whether wave-induced closed or partially-closed sediment loops could exist with particular regard to recirculation of sand swept out of the main channel by the ebb currents and the possible relationship of these loops to the bathymetric changes on Mair Bank, identified in the previous Chapter; (ii) to study the degree and type of wave attack that upper Mair Bank and Marsden Point are subjected to; (iii) to examine the megaripple "finger" phenomenon noted with side-scan sonar (Chapter 2) with respect to wave refraction processes in the Bay; and (iv) to generally understand the gross behaviour of Bream Bay because of the inter-relationship of the ocean and tidal inlet characteristics.

4.2 List of Symbols

c	wave phase speed ($m.s^{-1}$)
d	water depth (m)
D	grain size (m)
E	total energy in spectrum (m^2)
f	frequency of spectral estimate (s^{-1})
f_p	frequency at peak of spectrum (s^{-1})
F	wind fetch (m)
g	gravitational acceleration (9.81 $m.s^{-2}$)
H	wave height (m)
k	wave number ($2\pi/L$) (m^{-1})
L	wavelength (m)
$S(f)$	spectral density ($m^2.s$)
T	wave period (s)
U_{10}	wind speed at 10m above sea level ($m.s^{-1}$)
$\bar{U}_o$	mean shoreward velocity at the sea bed under Stokesian waves ($m.s^{-1}$)
$\bar{x}$	non-dimensional wind fetch
β	wave ray spacing
Δ	megaripple height (m)
Λ	megaripple wavelength (m)
ω	radian frequency ($2\pi/T$) (s^{-1})
ϕ_w	wave potential ($m^4.s^{-4}$)

4.3 Background

Several studies related to foreshore erosion have been conducted in Bream Bay by the consulting firm, Tonkin and Taylor (1979a, 1979b, 1980 and 1981) for the New Zealand Electricity Department (NZED). Their series of reports

summarise much previous research by various organisations. According to Kirk (Tonkin and Taylor, 1979a, p.33).

> "The coastal system of Bream Bay consists of a large ... sand sheet, which forms the present shoreline and the prograde beach ridges and dune systems which lie behind it. ... The bay is exposed to high energy storm waves and to ocean swell from a variety of directions but wave refraction within the bay together with the protection afforded by the shore in the north by Mair Bank result in a dominant nett northward longshore drift of sand within Bream Bay ... It is probable that with respect to longshore transport of sand, the Bream Bay system is essentially a closed one characterised by a fixed quantity of sand and by internal redistributions in response to changing wave conditions. ... There is a neglible supply of sand from the rivers of Bream Bay as revealed by comparative mineralogical analysis of river and beach sands (Schofield, 1980)."

The Phase 1 report of Tonkin and Taylor (1979b) concluded that the beach has been eroding at about 1m/year in recent times and that exceptional, episodic storms were primarily responsible. Hume (1979) noted beach changes with repetitive surveys. Other research was conducted by Peek (1979) who made a comparative study of the west and east coast wave environment, and analysed records from the NHB wave rider buoy deployed in Bream Bay during 1978 and 1979.

4.4 Refraction program

A refraction program for computer solution was written by this author to investigate Bream Bay. Much of the mathematical background can be found in Black (1975) as the refraction procedure was modified from this writer's computer program which models wave-driven, nearshore circulation on beaches. A complete description of the updated refraction program is given in the publication prepared by this writer, Black and Healy (1981).

The program algorithm is based on 5 simultaneous differential equations, which describe the passage of the wave orthogonal over varying bathymetry according to the formulations of Geometric Optics Theory. The kinematics of the ray positions are described (Noda, 1972) by,

$$\frac{Dx}{Ds} = \cos\theta \qquad 4.1$$

$$\frac{Dy}{Ds} = \sin\theta$$

where ds is a small increment of arc, x,y are orthogonal coordinates and θ is the angle of the wave ray relative to the x-y axes. The rate of change of θ along the wave ray is given by,

$$\frac{D\theta}{Ds} = \frac{1}{c}\left(\sin\theta\,\frac{\partial c}{\partial x} - \cos\theta\,\frac{\partial c}{\partial y}\right) \qquad 4.2$$

where c is a wave celerity given by the dispersion relation for Airy small amplitude waves as,

$$\omega^2 = gk \tanh (kd) \qquad 4.3$$

where $\omega = 2\pi/T$, T is wave period, $k = 2\pi/L$, L is wavelength and d is the water depth. The parameter, β, describes the distance between adjacent orthogonals, and,

$$\frac{D^2\beta}{Ds^2} + p(s) \frac{D\beta}{Ds} + q(s)\beta = 0 \qquad 4.4$$

The coefficients p(s) and q(s) are given by:

$$p(s) = - \cos \theta \left[\frac{1}{c} \frac{\partial c}{\partial x}\right] - \sin \theta \left[\frac{1}{c} \frac{\partial c}{\partial y}\right] \qquad 4.5$$

$$q(s) = \sin^2 \theta \left[\frac{1}{c} \frac{\partial^2 c}{\partial x^2}\right] - 2 \sin\theta \cos \theta \left[\frac{1}{c} \frac{\partial^2 c}{\partial x \partial y}\right] + \cos^2 \theta \left[\frac{1}{c} \frac{\partial^2 c}{\partial y^2}\right]$$

Equation (4.4) can be transformed into two equivalent first order differential equations where,

$$\frac{D\beta}{Ds} = \sigma \qquad 4.6$$

$$\frac{D\sigma}{Ds} = - p(s) \sigma - q(s) \beta \qquad 4.7$$

A fourth-order Runge-Kutta procedure was employed to solve the differential equations. Offshore from the starting point, the submarine topography is assumed to be planar and initial values for all five parameters can be established. A finite difference scheme is applied to find the derivatives of the phase speed throughout the computation. Depths are input on a square grid. A choice of wave breaking criteria is offered and the program calculates the wave heights with the maximum bottom orbital velocity, to ascertain the largest, equivalent, spherical quartz grain which could be moved by the wave-induced oscillations. The user has options on the wave parameters and tide levels.

The program has been extensively applied (Mathews, 1982, Willet, 1983; Healy, 1983) and is currently being used by staff and students at the University of Waikato, and also by the DSIR for investigations of wave refraction in Lake Taupo.

4.5 Wind wave generation

This writer's FORTRAN program to determine the sea state after wind excitation is presented in Black and Healy (1981). Called WIND, the JONSWAP (Hasselman et. al., 1973) and Liu (1971) spectra and the Sverdrup-Munk-Bretschneider (Bretschneider, 1958) wave statistics are produced for user-selected wind and fetch conditions.

The JONSWAP spectrum is described by,

$$S(f) = \frac{\alpha g^2}{(2\pi)^4} f^{-5} \exp \{- \frac{5}{4} (\frac{f}{f_p})^{-4} [\gamma \exp (\frac{-(f - f_p)^2}{2\sigma^2 f_p^2}]\} \qquad 4.8$$

where S(f) is the spectral density (m^2/Hz), f is the frequency, and f_p is the peak frequency.

The coefficients are,

$$\sigma = \begin{cases} \sigma_a = 0.07 \text{ for } f < f_p \\ \sigma_b = 0.09 \text{ for } f > f_p \end{cases} \quad 4.9$$

and $\gamma = 3.3$

For the non-dimensional fetch,

$$\bar{x} = \frac{Fg}{U_{10}^2} \quad 4.10$$

where F is fetch, U_{10} is the wind speed measured 10m above the sea furface and g is the acceleration due to gravity, the non-dimensional peak frequency is given by,

$$\bar{f}_p = \frac{f_p U_{10}}{g} = 3.5 \bar{x}^{-0.33} \quad 4.11$$

Alpha is given by,

$$\alpha = 0.076 \bar{x}^{-0.22} \quad 4.12$$

The total energy in the spectrum is,

$$E = 1.6 \times 10^{-7} \frac{U_{10}^2 F}{g} \quad 4.13$$

Program JONSWAP outputs the spectrum in a suitable format for the plotting routines (Appendix III).

4.6 Previous refraction study in Bream Bay

Wave refraction has been previously undertaken by Christian (University of Auckland) for Tonkin and Taylor (1981). Modelled at scales of 1:40,000 and 1:200,000, four figures have been reproduced here (Figures 4.1 to 4.4). Typical wave periods from Bream Bay of 8s and 12s from the north-east and 12s from the east were selected after studying data from three sources: (i) a wave rider buoy deployed by the NHB and analysed by Peek (1979); (ii) observations from the Mokohinau Lighthouse; and (iii) coastal observations by NZED staff at the power station.

In his report (Tonkin and Taylor, 1981, p. 4.1), Christian states "The possibility of a south-easterly swell entering the area was also considered but because of the shelter afforded by Great Barrier Island and the general sea bed topography it was felt that this would not result in any energy concentration in the area of concern." This statement is critically discussed later in this Chapter, where it is shown that east to south-east waves of some significance do impinge on Mair Bank.

Although the rays are widely-spaced and the grid is coarse, several important features are highlighted by the refraction diagrams. In Figure 4.1, with swell from the east, only one wave ray approaches the beach between

the power station and Marsden Point and in Figure 4.2, no wave rays enter this area. Similarly, on the finer grid (Figure 4.3), some energy approaches upper Mair Bank but due to wave divergence, heights would be small.

Refraction on the ebb-tidal delta near the 10m contour bends the rays onto the crest of the delta with some strong focusing of energy off-shore from the power station. Christian suggested that there is clear evidence for a concentration of energy occurring at the Power Station site. On Figure 4.4, 8s waves respond similarly.

Thus, it would appear that the upper levels of Mair Bank and Marsden Point are offered almost total protection from east to north-east swell by Busby Head, with most energy attacking Bream Bay beach further to the south, near the NZED power station.

4.7 Detailed refraction study of the Mair Bank ebb-tidal delta

To gain insight into the sediment transport characteristics, of the ebb-tidal delta, a more detailed refraction analysis of Mair Bank was undertaken.

4.7.1 Specification of refraction parameters

The 1981 RNZN bathymetric survey of the delta was digitised from maps at 1:10,000 (NZ5213, Serials 3, 4, 5 and 6) onto a square 250m grid over an area 7.5 by 7.25km. The depths were entered onto computer disc with an input program called REFIN. For the refraction modelling, a mean tide of 1.5m above Chart Datum was used in all cases.

Initially, 12s waves were examined. Supportive evidence for the choice of this period has been presented by Dr T.W.F. Harris (Department of Physics, University of Auckland, pers. comm.) since the Tonkin and Taylor analyses. Wave spectra derived from sea level elevations recorded at Hicks Bay on a wave rider buoy, exhibited average spectral peaks at 12.5s from the east and 10.5s from the north-east.

Examination of Figure 4.3 revealed that at the edge of this writer's detailed grid, the waves have an approximately westerly heading, so that waves from 70°, 90° and 110° T were examined. However, wave directions are closer to a 90° T approach angle than to either of the test cases with angles of 70° T and 110° T. Thus, these serve as extreme upper and lower limits for the angles of incidence of ocean swell but do not include locally-generated east to south-east waves, to be discussed later.

An initial wave height of 1m was selected for the investigation. It is important to note that with the 1m starting height, linear scaling may be applied to any height shown later on the delta for a different initial wave height specification, because the program is energy conserving with no allowance for bed frictional losses. Thus, extreme sea states and design-storm waves may be examined by applying linear scaling to the given heights. The 1m wave is not extraordinary; when the NHB wave rider buoy was placed in

Bream Bay from June, 1978 to May, 1979, there was at least one occurrence of waves with a significant height larger than 1m for 10 of the 12 months, (Peek, 1979, Figure 3.03). The two exceptions were the summer months of December and January. In July 1978, a maximum significant height of 9m was measured.

In conjunction with the wave height specification, the largest, equivalent, spherical quartz grain which the bed-orbital velocities could move, is determined. Being directly proportional to the wave height, bed orbital velocities can be linearly scaled by the starting height for any initial condition. For the largest grains that the waves can move, the maximum is proportional to the bottom orbital velocity (or the wave height) to the fourth power for D<0.0005m and to the power 8/3 for D>0.0005m (Black and Healy, 1981).

4.7.2 Caustics

The locations of caustics are shown in Figure 4.5(a). At caustics the predicted wave heights become infinite when the ray spacing approaches zero. Even though caustics are sometimes interpreted as zones where large waves are guaranteed to break (Tonkin and Taylor 1981), field observations do not always support this. Instead they arise out of the mathematical solution which is modelled on the behaviour of light refraction. Thus, caustics can only be considered as singularities which highlight regions of convergence.

4.7.3 Reverse tracking

In order to ascertain directions and periods of waves capable of attacking the upper levels of Mair Bank near Marsden Point and other locations in Bream Bay, reverse ray tracking was incorporated into the computer program. This allows offshore propagating rays to be initiated at a chosen nearshore location. Notably, the starting position must be carefully selected to ensure that strong, initial divergence or convergence does not occur unless these phenomena are specifically being examined. Thus, undistinguished, flat beds away from the extreme shallows were selected for an investigation of upper Mair Bank and the power station environs.

4.8 Wave heights on the Mair Bank ebb-tidal delta

With 12s periods and an initial 1m height, the wave heights are marked along the shoreward-propogating wave rays at 1000m increments in Figure 4.5(a). Wave height reinforcement occurs over a distended region extending offshore from a location just to the south of the NZED power station. Energy is focused onto the shallower parts of the delta from the south, by the steep southern limit of the delta as already noted by the Tonkin and Taylor analysis.

Along the shoreline from north to south, Figure 4.5(a) shows that the

height is reduced in the north near the upper levels of Mair Bank by ray divergence then increases to a maximum about 500m to the south of the power station before decreasing again as the southern edge of the ebb-tidal delta is reached. Just to the south of the power station, without frictional energy losses, the height has almost doubled from its deep-water value.

4.9 Wave-induced movement of bed sediments

For the initial 1m wave height, Figure 4.5(b) shows the largest, equivalent, spherical, quartz grain which 12s waves could move on the delta. In the convergent regions, even with the moderate initial wave height of 1m, the size exceeds 0.001m which is considerably greater than typical grain sizes of less than 0.0003m found on the delta (Chapter 2) and thus typical waves would be capable of disturbing the bed sediments. The maximum bed orbital velocities attained under both the wave crest and trough are about 0.5 $m.s^{-1}$ except near caustics where predicted speeds are unrealistically higher. Common bottom-orbital velocities on other parts of the delta are 0.2 to 0.5 $m.s^{-1}$ for 1m waves.

4.10 Differences of 20^{o} in the initial wave angle

Shown on Figures 4.6 and 4.7, $\pm 20^{o}$ differences in the initial angle have a considerable effect. For 110^{o}, the waves focus strongly on a section of beach almost due west of Busby Head but still do not attack Mair Bank directly (Figure 4.6). They have a northerly direction component near the shoreline and should generate longshore currents which would move sand towards the harbour entrance.

For waves approaching from 70^{o}, some of the most northerly rays are coming off the land near their initial position and are therefore not physically realisable (Figure 4.7). The important phenomenon to note however, is the trapping of wave energy by the steep, southern flank of the ebb-tidal delta with Ray 19, for example, refracting through about 50^{o} to eventually arrive at the beach just to the south of the Power Station. The refraction process is therefore extremely important with respect to the wave energy flux which eventually impinges on the shoreline near the NZED power station with energy being trapped by the delta for waves with initial angles over the wide range of $90 \pm 20^{o}T$. The shoreline, wave-energy maximum swings about the position located 500m to the south of the Power Station, in response to changes of the initial angle around due east at the edge of the fine grid.

4.11 Reverse ray tracking with 12s waves

4.11.1 Upper Mair Bank

With 12s periods (Figure 4.8), using reverse ray tracking only one ray initiated on central Mair Bank leaves the grid with an approximate westerly heading.. The interaction of the deep main channel with the ebb-tidal delta

induces strong refraction of those rays passing obliquely across the bathymetry contours and as already shown by the forward tracking, upper Mair Bank is mostly sheltered from the east and north-east. However, waves from the south-easterly quarter are seen to converge on the position, and if it can be shown that offshore bathymetry will allow south-east waves to approach the grid, they will certainly impinge on the upper levels of the Bank with possible reinforcement occurring.

4.11.2 NZED Power Station

Reverse tracking for 12s waves from a location offshore from the power station shows that the most southerly 7 wave rays leave the grid with angles from 94^{o} to 129^{o} T (Figure 4.9). Moreover, if further bathymetry had been available in the southerly direction, it is likely that the 4 most southerly rays would also have been heading more westerly at the western grid limit (c.f. Figure 4.3). Thus, the waves which attack the power station have directions at the edge of the fine grid of about 94^{o} T and this may be attained by east to north-east deep water swell. Wave directions of 94^{o} (c.f. 90^{o} used previously) differ by just enough to swing the wave energy maximum at the beachline from 500m south of the power station, to a location more directly in front.

4.12 The significance of east to south-east waves

The analysis has shown so far that the upper levels of Mair Bank and Marsden Point are almost totally sheltered from deep ocean swells except for those coming from the south-east. However, the region is protected from south-east ocean swells by Great Barrier Island and the Coramandel Peninsula. It remains to be shown whether shorter waves with reduced periods, expected to occur under east to south-east winds, could attack the upper levels of Mair Bank.

4.12.1 Wind speed

Bioresearches (1976b) presented the probability of wind occurrence against direction for various speed ranges (Figure 4.10). The data is taken from the Marsden Point anenometer at 13m above MSL for the period from January, 1969 to August, 1973. At low speeds, west to north-west winds predominate, but storm events are mostly from the easterly sector.

Marsden Point is sometimes subjected to storms of considerable intensity. Over the 10-year period from 1969 to 1978, the number of 6-hourly observations where the wind speed exceeded 15 $m.s^{-1}$ from the easterly sector (020^{o} - 130^{o}T) varied from 9 in 1974 to a maximum of 131 in 1978 (Tonkin and Taylor, 1979b), out of 1420 possible observations each year. Over the same period at the Mokohinau Light, there were 1 to 13 observations per annum of wind speeds exceeding 25 $m.s^{-1}$ with 2 readings of 37 $m.s^{-1}$ and 2 of 41 $m.s^{-1}$ in 1978 and

all four having easterly directions.

4.12.2 Wind fetch

In a south-easterly direction from Marsden Point, the wind has an unbroken fetch of 110km to the Coramandel Peninsula along a narrow, approximately-rectangular waterway which is 20km wide between Cape Rodney and Little Barrier Island at its narrowest point. Applying the correction to fetch for a limited expanse of water (Shore Protection Manual, Figure 3.13, 1975) under the assumption that the area can be represented as a rectangular basin, the effective fetch reduces from 110km to 47km.

This reduced fetch is approximately equal to the distance to the nearest land masses in the east to south-east sector and serves as a general minimum estimate of fetch in any of these directions for the generation of local waves. Through some narrow "windows", fetches may be larger than 47km and correspondingly higher waves may occur. Furthermore, wave directions are smeared about the mean wind heading under storm conditions, and it is unlikely that the offshore islands in the east to south-east direction would totally shelter any part of Bream Bay Beach, particularly as short-period storm waves would not be expected to bend towards the islands as noted in the Tonkin and Taylor Figures for 12s waves.

4.12.3 Wave height prediction

Applying the minimum fetch of 47km, Figure 4.11 shows the JONSWAP spectra for three wind speeds of 10, 20 and 30 $m.s^{-1}$. With increased wind speed, the frequency of the peak is reduced and the total energy in the spectrum increases markedly. Table 4.1 compiles the wave heights and periods found with Program WIND for the three cases. For the more common speed of 10 $m.s^{-1}$, the significant wave height should be about 1.2m at the Mair Bank extremities. The predicted period of the waves at the spectrum peak is approximately 5s. For the extreme case of 30 $m.s^{-1}$ winds, the significant heights could attain 3.5m with periods of about 7s.

Observations during a storm on 7-Jun-83, with winds from 110° T and average speeds of 15 $m.s^{-1}$, confirmed this predicted behaviour. Waves reaching the upper levels of Mair Bank were visually estimated as being about 2m high (c.f. a predicted 1.7m on Table 4.1).

On Table 4.1, the significant period of the SMB method is less than the predicted period at the peak of the spectrum. This is not unexpected and analysis of swell waves in 10m of water by Lee and Black (1978) revealed that the significant period differed from the period at the peak of the spectrum by a factor of 0.71. Other studies quoted in the above paper found factors of 0.94, 0.75 and 0.78. Analysis by Black (1978b) suggested that the spectral peak represents the sea state better than a zero up-crossing period for applications using monochromatic waves.

4.12.4 Influence of bathymetry beyond the fine grid

Over much of the fetch, the depths are 40-50m below CD. The wave-length of a deep-water, small-amplitude Airy wave is related to the period as $L = gT^2/2\pi$ and for $T = 5s$, then $L = 39m$. Water is defined as "deep" if the ratio of depth, d, over wavelength is greater than approximately 0.25 and for the case here, $d/L \simeq 0.87$. Thus, these waves will not refract significantly or lose energy due to bed friction until impinging on the shallower water with $d<10m$, and this only occurs on the ebb-tidal delta inside the edge of the detailed grid. The direction of wave input and the wave height will therefore depend on the wind velocity only, and not on the bathymetry beyond the grid. Similarly, for 7s waves, intermediate depths occur roughly for $d<20m$ and mostly the waves will be unaffected by the refractive processes until reaching the ebb-tidal delta. Thus, the heights and periods predicted by WIND can be used as direct input into the refraction program without modification.

4.12.5 Reverse tracking from central Mair Bank using waves with 5s periods

Reverse ray tracking with 5s waves, shown in Figure 4.12, revealed that all rays with approach angles from 111° to 180° will reach the upper levels of Mair Bank which concurs with observations during a storm with winds from 110° T noted above. Mostly the rays are only slightly affected by the bathymetry, refracting usually by about 10° towards the north from their starting angles. As refraction depends on wave period, for the shorter 5s waves, the bathymetry has a much lessened effect.

The minimum angle of 111° T is approximately a critical angle such that waves with directions north of this cannot attack upper Mair Bank. There are, however, three major factors which influence estimation of a critical wave angle making 111° T an approximation. Firstly, wave refraction varies significantly with wave period which, in turn, depends on storm duration and strength. The critical angle was ascertained for 5s periods. Secondly, near Mair Bank tidal currents will transport the surface gravity waves towards the Harbour entrance during flood tides. In this case, the minimum angle of incidence may be considerably less than 111°. Thirdly, the spreading of wave directions about the mean which occurs in a stormy sea, implies that 111° cannot be used as a critical wind direction. Even for more northerly mean wind orientations, some waves should still impinge on Mair Bank. Thus, the critical angles applies mainly to short-period swell conditions and is an approximation during storms.

With regard to sediment transport, because of their oblique angle with the shoreline (Figure 4.12), storm waves from the south-east quarter, in conjunction with supporting flood currents, would aid the entry of sand into the harbour by stirring up sediment on the delta and in the marginal flood channel off Marsden Point, to be convected into the harbour by tidal currents.

Moreover, these episodic events may have a very significant influence because of their relative intensity.

4.12.6 Entrance throat tidal dominance

In Chapter 3, the morphology suggested that the entrance throat to Whangarei Harbour is tidally dominated. The analysis undertaken here has revealed that Marsden Point and upper Mair Bank are subjected to wave attack during storms but not from long-period swell waves. The ebb-tidal dominance is therefore maintained at the entrance because of the relative infrequency of storm events as against the consistency of the tidal flows which move about 150×10^6 m^3 of water, at speeds exceeding 1 $m.s^{-1}$, through the entrance four times a day.

4.13 Megaripple fingers in Bream Bay

Much of the southern, ebb-tidal delta above the 10m contour is flat and featureless, except for distinctive bands or "fingers" of megaripples identified in the side-scan sonar survey (Chapter 2). These features occur in confined, elongated sections aligned nearly perpendicular to the shoreline in an east-west distended region of approximately 500m by 4000m just south of the Marsden power station (Figure 2.8). The megaripples have average wavelengths of approximately 0.8m and exist in depressions of the bed.

Similar features have been noted by D'Olier (1979) among others. Morang and McMaster (1980, p. 83) from southwestern Rhode Island (U.S.A.) beaches, describe them as "narrow bands of megaripples ... trending perpendicular to the coastline." The features must necessarily be associated with wave orbital motion or uni-directional currents generated, for example, by tides, wind, or oceanic circulation.

4.13.1 Diver examination of megaripple fingers

Further field investigations over the features were undertaken with divers on 27-May-82, some 7 months after they were first identified with the side-scan sonar. Their continued presence suggests a long-term permanence of the features.

The bed profile was obtained by placing a 5m length of iron perpendicular to the megaripple crests and measuring the distance to the sea bed at 0.10m intervals. In Figure 4.13, the megaripples are symmetrical implying wave modification as occurs due to the reversing orbital motions, although asymmetrical, wave-generated ripples can occur under steeply shoaling or breaking waves.

The mean wavelength was,

$$\bar{\Delta} = \begin{matrix} .707 \pm 0.073\text{m crest-to-crest} \\ .700 \pm 0.055\text{m trough-to-trough} \end{matrix}$$

Mean height was $\bar{\Lambda}$ = 0.060 ± 0.010m and the ripple index was $\bar{\Delta}/\bar{\Lambda}$ = 0.09 which is about half the maximum steepness (≈0.2) of ripples but is greater than 0.06, the approximate limiting value for megaripples (Figure 4.14). Thus, they may be more correctly classified as ripples, but for uniformity the Harms et. al. classification will be maintained and, as their wavelength exceeds 0.6m, they are considered to be megaripples.

During the measurements, the sea was in a disturbed state with T = 2.5s and wave heights of H ≈ 1m were estimated. Such conditions were calculated to be insufficient to move the bed sediments for the given depth and divers confirmed this. The megaripples were aligned at 98° True (looking perpendicularly across the crests), which corresponds almost exactly with 94°, reverse angle for 12s waves from the power station (Figure 4.9). Further to this, the wavelengths of the megaripples show a general increase (Δ = 0.7 to 1.6m), as the beach is approached (Figure 2.8) which may relate to larger horizontal wave orbital diameters in the shallower water. It is therefore likely that the features are associated with typical 10-12s waves passing across the delta.

Grain sizes were found from samples taken off the crests and from the troughs by divers. Figure 4.15 shows the grain size distributions after sieving at 0.25 ϕ intervals by University of Waikato technical staff. Table 4.2 compiles the grain size statistics obtained after analysis by the moments method with this writer's programming. The troughs are much coarser and median diameters were 0.00047m (crests) and 0.00182m (troughs).

Shell material lines the troughs, and introduces a bimodality into the size distribution, with much of the weight between 0.008m and 0.002m; the typical shell sizes. The shell content can be clearly seen on underwater photographs taken against a board with 0.02m graduations (Black and Healy, 1982, Figure 32). Located outside the megaripple fields, grain size distributions at nearby Sites 51 and 53 sampled by the RNZN (Chart NZ5213, Serials 3a and 4a), exhibited low shell content and peaked distributions centred on finer median sizes of 0.00022m and 0.00018m respectively. Some winnowing has apparently occurred, whereby currents over the megaripples were sufficient to carry away the fine sands leaving an armour or lag of coarser grains.

4.13.2 Relationship of megaripple fingers to refraction on the delta

Comparison of the location of megaripple fingers (Figure 2.11) with the map of refraction wave heights, reveals that the phenomenon is almost certainly associated with the spatial wave height variations due to refraction of approximately easterly waves. Plotted on Figure 4.5(a), the megaripple zone coincides almost exactly with the strongest wave reinforcement on the delta. Furthermore with an initial offshore wave angle of 94° suggested by the reverse tracking instead of 90° as used in the Figure, the zone of

reinforcement would have swung north and even better correlation would have been obtained.

Thus, ray convergence and the associated increase in bottom orbital currents (Figure 4.5(b)), causes greater stirring of the sea bed in these regions to establish the bedforms and to cause the winnowing which is expected to occur in a higher-energy zone. The confined nature of the bands may be further associated with zones of very strong convergence within this region of overall height reinforcement.

4.14 Shoreward movement of sediment and a closed or partially-closed loop

Notably, 1m waves with 12s periods are capable of moving the typical grain sizes in Bream Bay (Figure 4.5(b)). Furthermore, the presence of the megaripples in the zone of wave reinforcement and the winnowing of finer sands testifies to this capability. Thus, the sediment transport characteristics of the ebb-tidal delta have a wave-related component. Any near-bed, shore-oriented, uni-directional current, in conjunction with the stirring action of the waves, would be expected to transport sediment towards the beach, including the sand swept out of the harbour by ebb-tidal currents. Nett tidal current resultants, or even compensatory shoreward circulation at the bed during the very common offshore winds, may be effective agents.

Notably, the latter process was judged as the cause for the dispersal and onshore movement of dredge spoil dumped in Poverty Bay (Miller, 1981; Nelson and Healy, 1982) and also partly (this writer) for the impregnation of Bream Bay with algae during the summer of 1982-83 when winds from the westerly quarter were exceptionally predominant. Coastal water temperatures were unseasonally low during this time due to the introduction of cooler, near-bed sea water to replace warmer, surface water moved offshore by the westerly winds.

Moreover, even though there is still some disagreement about the velocity magnitudes, experimental measurements and some wave theories predict a nett current in the direction of wave propogation at the sea bed under surface gravity waves alone. Longuet-Higgins (1953) theoretically derived a nett shoreward velocity, $\bar{U}_o$, at the sea bed under Stokesian waves given by,

$$\bar{U}_o = \frac{5}{4} \left(\frac{\pi H}{L}\right)^2 \frac{c}{\sinh^2 (kd)} \qquad 4.14$$

where L is the wavelength; H is the wave height; c is the phase speed; k is the wave number; and d is the water depth. Russell and Osorio (1958) compared this with experimental data and obtained good agreement. Komar (1976) noted that nett sediment drift in the direction of wave propogation is known to occur but it is uncertain whether the Longuet-Higgins theoretical development explains the cause. Nevertheless, with typical values in the above formula, (H = 1m, L = 100m, T = 10.7s, d = 10m) the current speed is 0.03 $m.s^{-1}$.

This is a small current, but let us assume that the waves disturb a layer of sand which is constantly moved forward at that speed. As an order of magnitude approximation, take the layer as being 1 grain diameter thick with D = 0.0002m, then the sand moved shoreward is $6 \times 10^{-6} m^3/m/s$ or $0.009 \ kg.m^{-1}.s^{-1}$ for density of 2650 $kg.m^{-3}$ and porosity of 0.45. This is comparable with transport rates under moderate tidal flow. It has been assumed here that even though the nett current speed of 0.03 $m.s^{-1}$ is far below the sediment threshold, the waves stir the sediment and raise it into suspension and then the nett current moves it shorewards.

Over the beach line of approximately 2,000m adjacent to the ebb-tidal delta, this process could introduce ~1,000 m^3/day onto the beach for the rate of $6 \times 10^{-6} m^3/m/s$. Even if the simple approximation is an order of magnitude too big, there is still a considerable potential for transport by this mechanism alone under large waves, as the sea bed currents are predicted to rise with the square of the wave height.

Waves of only 1m height do disturb the bed sediments, and, moreover, under wave-induced, sea-bed currents, significant quantities of sediment may be moved shoreward. Therefore, in conjunction with the ebb-tidal currents around Mair Bank in the main channel, which sweep sand out of the harbour (Figure 2.8) and a northward littoral drift along Bream Bay Beach (Kirk, quoted earlier), a closed or partially-closed, clockwise-rotating, sediment loop could exist in northern Bream Bay.

In support of this, under more usual conditions where the ebb-tidal delta is located on an unsheltered coastline subject to direct wave attack, the standard model asserts that ebb currents in the main channel move the sand offshore and waves force it back towards the beach. In Bream Bay, the geometry of the region is different but the principle is fundamentally the same. Furthermore in Chapter 5, it is shown that tidal currents near Marsden Point have a strong, northward dominance. As the littoral drift is also northward, there is a need for a constant, sediment supply which must necessarily be fed downstream from Bream Bay Beach. This implies the presence of some form of sediment loop if, as Kirk (quoted earlier) suggests with regard to longshore transport, "The Bream Bay system is essentially a closed one characterised by a fixed quantity of sand."

If the process is viewed as a circulating, impulsive loop, then the time cycle of events should be:

(i) high-wave activity moves sand shorewards from the channel flanks south of Busby Head onto shallower parts of the ebb-tidal delta particularly in the zone of wave reinforcement near the NZED power station. Corresponding beach erosion and reorientation of Mair Bank should occur; (ii) sediment is moved towards Mair Bank by longshore currents causing deposition there; (iii) beach erosion should slow as offshore sand replaces early losses under storm attack;

(iv) tidal currents in the main, ebb channel, should sweep the sand around the northern and eastern flanks of Mair Bank and back out of the harbour onto the channel margins south of Busby Head.

The process would necessarily be slow and if its initiation was abrupt, the system cannot be in equilibrium.

The comparison of 1959 and 1981 RNZN surveys (Figure 3.17), exhibited many of these aspects; particularly widespread accretion on the delta adjacent to Bream Bay Beach near the power station, with corresponding offshore erosion on the channel margins. Furthermore, accretion on the upper levels of Mair Bank and its flank along the main channel as well as erosion along the beach were noted. The event is evidently still "young" as sediment is still moving around the loop and has not yet been swept out of the harbour from Mair Bank in sufficient quantities to replenish the initial erosion south of Busby Head. Thus, the bathymetric evidence strongly supports the proposition. In this context, the changes noted along the beach in Bream Bay (Hume, 1979) are seen as part of a dynamic system in northern Bream Bay involving the beach, the upper levels of Mair Bank, ebb-tidal currents in the main channel and onshore sediment movement over the southern segment of the ebb-tidal delta.

In the loop, Mair Bank is the most likely location for sediment to be deposited. As the tidal currents are virtually unchanged over the longer term, the episodic nature of storm and wave activity can establish impulsive bursts of movement across the ebb-tidal delta. In conjunction, these factors could cause a build-up of sediment near Mair Bank, where wave influences are first over-ridden by tidal dominance.

As the cycle of events may exceed the time between bathymetric surveys (1959 to 1981), it is difficult with this data, to predict the long-term future morphology of northern Bream Bay and the shoreline. Most likely, the processes are episodic and coupled with prior events and a permanent equilibrium cannot be attained. Furthermore, the accretion on Mair Bank could be polygenetic, being related to inner harbour erosion flanking the shipping channel, realignment of the Bank itself, and an increased littoral drift in the sediment loop due to storms. The relative importance of these factors is an indicator of the advancement of the present cycle. The prediction of future trends presupposes a knowledge of the overall time scale and the time on the cycle of present events.

Nevertheless, the implications of the sediment loop are numerous. Excluding severe, abnormal wave activity in the Bay, the recorded off-shore accretion should currently help to slow or reverse the Bream Bay Beach erosion, by continued, shoreward, sediment movement and the frictional damping of waves crossing the shallower delta. Secondly, a less speculative decision can be made with regard to suitable dredge-spoil, dumping sites in Bream Bay. Furthermore, it suggests that the proposed timber port should have no measure-

able effect on the NZED power station, concurring with a previous statement by the DHI (1982). Although, by addition or subtraction of harbour sediment to or from the loop at Marsden Point, the port is seen to be related to the overall sediment transport characteristics of both the harbour and Bream Bay. Finally, the waves which are responsible for scouring the Bream Bay foreshore and threatening the NZED installation, are ironically seen to be an essential source of energy capable of bringing sand back to the beach to repair the damage done.

4.15 Interpretation of beach surveys and string results on Marsden Point

In Chapter 3, the bathymetry changes on Marsden Point, isolated by beach surveys and string measurements, were introduced and discussed with regard to bathymetric stability. In this section, their behaviour is interpreted in terms of the wind-generated waves impinging on the Point. The beach profiles are shown in Figure 3.21 and the surveys have been numbered from 1 to 6.

4.15.1 Beach profiles

The cross-sections exhibit large changes in the berm position and in the profile shape. It was suggested in Chapter 3 that survey 4 was storm scoured with a poorly-developed berm whereas the others exhibited varying degrees of correlation with the "summer" profile.

To examine this, six-hourly wind observations recorded on the anenometer at the NHB wharf were reduced to ascertain the wind climate in the period prior to the beach surveys. According to the Sverdrup-Munk-Bretschneider equations the significant wave height is a function of the wind speed raised to the fourth power, and for this analysis a wave potential ϕ_w was introduced with definition,

$$\phi_w = \Sigma U_{10}^4 \qquad 4.15$$

where U_{10} is the wind speed 10m above sea level. The anenometer on the NHB wharf is at varying heights above sea level due to the changing tide and is at 13m above MSL. The recorded speeds were used without correction for distance above sea level. A wind rose was constructed with speed measurements allotted into 8 sectors (1^o to 45^o, 46^o to 90^o, 91^o to 135^o, 136^o to 180^o, 181^o to 225^o, 226^o to 270^o, 271^o to 315^o and 316^o to 360^o True inclusive) according to the wind direction. Then for each sector a total wave potential was calculated by summing the speeds each raised to the fourth power, for all observations in each sector. The wave potential is large when the wind speeds are high and/or when there are many observations in the sector. It does not however, account for the duration of the wind which affects the ability of the sea to reach a fully-arisen state. For each of the 6 surveys, the wave potential was calculated for 30 and 15 days prior to the survey (Figure 4.16). It was found that the first and second 15-day periods, often exhibited grossly

different wind climates and the beach profiles (Figure 3.21) were better correlated with the more recent 15-day interval. The total energy directed onshore (1^{o} to 180^{o} True) and offshore (181^{o} to 360^{o} True) for the 15-day intervals is presented in Table 4.3. Major results were:

(i) the scoured "winter" Survey 4 suffered high levels of onshore activity as expected.

(ii) surveys 6 and 2 received the lowest onshore energy and exhibit similar beach cross-sections. At cross-sections 20 and 17 (Figure 3.18), Survey 6 has a wide, well-developed berm, as would be expected particularly on Bream Bay Beach (Cross-section 20).

(iii) at Cross-section 18, Surveys 6 and 2 have well-developed berms but they do not project so far to seaward as some of the others. Evidently near Cross-section 18 when the wave activity is low, the tidal influences apparently predominate and the berm may be eroded by flow passing through a channel at its toe seen on the cross-sections. Further onto Marsden Point at cross-section 17, Survey 6 once again projects furthest out to seaward at the berm, as it did at section 18.

(iv) Surveys 5 and 3 have middle order onshore wave potentials and lie between 6 and 4 at Cross-section 20.

(v) Survey 3 at Cross-section 17 was badly scoured at the berm level and no explanation is offered for this. However, Survey 3 received no energy between 46^{o} and 180^{o} True in the 15 days prior to the survey which did not occur at any of the others.

(vi) there is no apparent relationship of the profile shape to the tidal range. Surveys 6 and 2 behave similarly but have Neap and Spring ranges of 1.3m and 2.5m respectively during the day of the survey.

Thus, there is a strong correlation of the beach profile shape to the prior wind activity, even in the sheltered regions around Marsden Point as expected from the prior refraction analysis. Some lesser tidal influences were also noted, being mostly evident during periods of low-wave activity.

4.15.2 Strings on Marsden Point and the NZRC wharf pylons

Over nearly a ten-month period the strings (Figure 3.24) exhibited the following characteristics:

(i) the largest movement of the bed of 0.8m occurred at S3.5, which is on the berm at Marsden Point, concurring with the beach profile surveys.

(ii) at the lower inter-tidal zones (S3.2 and S3.3) the movement was much smaller being less than ±0.2.

(iii) S3.1 which was placed on the scoured shell lag exhibited almost total stability with maximum movement of less than 0.04m. This is exceptional behaviour as the neighbouring sandy regions exhibited large changes in bed levels. Thus, the shell lag is responsible for bed stability. There

should be no significant reduction in tidal current speeds in this area which could otherwise explain the small movements. The shell overlays sandy sediments.

(iv) The berm was subject to low, long-term sedimentation over about 70 days, which was almost totally eradicated during one easterly (110° T) storm with 15 $m.s^{-1}$ winds near Day 170. At the lower, inter-tidal levels on Marsden Point, Strings S3.2 and S3.3 concurrently exhibited nett erosion but to a lesser degree. After this event, the berm slowly accreted again over a period of another 100 days. Thus, the evidence suggests that in calm conditions Marsden Point behaves as a sediment storage reservoir. During high-energy storms, sand is swept away and must enter the harbour during flood tides.

(v) It is expected that a storm should erode the inter-tidal beach at Marsden Point as occurs on "classical" beaches (King, 1972). Alternatively however, if there is a steady reduction in wave energy as the Point is approached due to sheltering by Busby Head, the energy reduction could induce accretion to the north. This depends on the degree of sheltering and therefore on the wind direction. Opposed to this is the strengthening of tidal currents towards the harbour entrance.

There is some supportive evidence for the sheltering proposition although the assertion is definitely not proved as many other factors may be influential. The major storm which eroded the berm and lower inter-tidal levels had a steady wind direction of 110° T and Marsden Point could be eroded by direct wave attack. However, on the 3-Jul-82 a comparable storm (12 $m.s^{-1}$) caused accretion instead of erosion at the lower inter-tidal levels (S3.2) with some minor erosion of the berm. In this case, the wind was further to the north with directions of 80°- 110°T. The refraction has shown that the lower angle is definitely below critical and only limited wave activity at Marsden Point should result.

The importance of tidal currents cannot be estimated without some detailed field measurements. Notably though, during the storm which caused accretion at the lower levels, the tide range was 1.9m as against a Spring range of 2.6m when the bed was scoured. The faster tidal flows in the latter case may have been partly responsible for the scouring.

(vi) At the Pylons on the NZRC wharf (Figure 3.24) the major event was a storm with winds of 15 $m.s^{-1}$ from 060° to 090° True. The lower beach level was scoured by about 0.9m during this one storm. The upper levels, although not as badly affected, were also eroded. Notably, degradation had already started before the storm on the upper beach levels which may have resulted from concurrent accretion on Marsden Point reducing the supply of sand to the NZRC wharf during calm conditions outside the entrance. The depleted beach could have exacerbated the lower erosion during the storm by

being unable to supply sufficient quantities of sediment to maintain the lower erosion rate.

(vii) The erosion at the Pylons was not reflected by similar changes at Marsden Point, and conversely for the erosion at Marsden Point some three months later. The wind strengths were similar in both cases but the angle differed by about 45^{o}. This demonstrates the sensitivity of sediment transport to the local wind direction. At 060 - 090^{o} T the Pylons were open to attack from waves generated over the small fetch across Calliope Bank. These small waves of about 0.3m high (field observations on 7-Jun-83 during similar conditions) were responsible for the erosion, in conjunction with the strong tidal currents in the area.

(viii) After the erosion, the lower beach level at the Pylons slowly accreted for 180 days at an average rate of 0.0025 m/day. This is equivalent to a layer of about 8 grain diameters thick added each day.

(ix) Accretion at the lower level near 150 days was preceeded by a storm with winds of 13 $m.s^{-1}$ from 030^{o} True. This is almost straight onshore which had the effect of taking sand from the upper level, Pl.2, and depositing it further down the profile.

4.16 Conclusions

(i) A detailed wave refraction study of the Mair Bank ebb-tidal delta was carried out. These results specifically showed strong focusing of wave energy near the NZED power station with energy being diverted away from the southern flank of the ebb-tidal delta by the increasing depths there. The Mair Bank upper levels are mostly protected from east to north-east swell waves by Busby Head.

(ii) Wave heights over the ebb-tidal delta are shown for 12s waves. These can be scaled for any design-wave, initial height.

(iii) It was found that typical 12s waves of 1m height were capable of moving sandy sediment over the delta and flanking channel margins.

(iv) The locations of finger megaripples on the ebb-tidal delta were shown to coincide with the area of strongest wave height reinforcement caused by refraction. With other supportive evidence, surface waves are acknowledged as being responsible for the generation of the megaripples.

(v) A wave-induced current at the sea bed is shown to be capable of moving significant volumes of sediment shoreward across the ebb-tidal delta. In conjunction with the ebb-tidal currents in the main channel and the littoral drift along Bream Bay Beach, this shoreward movement completes a closed or partially-closed sediment loop in northern Bream Bay. This loop is reflected in bathymetric survey comparisons which exhibited offshore erosion flanking the channel south of Busby Head and accretion shorewards. Furthermore, within the loop, the crest of Mair Bank and its northern flank

along the main channel have both accreted. The sand, however, has not yet been swept out of the harbour by ebb-tidal currents in sufficient quantities to infill the original erosion. Thus the system is currently unstable and evidently episodic. Within this loop, erosion in Bream Bay should slow or be reversed as adjacent deposition on the ebb-tidal delta continues to move shoreward.

(vi) Waves focusing on the power station had offshore directions of 94^{o} T near the edge of the ebb-tidal delta.

(vii) Reverse ray tracking showed that south-easterly waves focus on upper Mair Bank and Marsden Point. Application of semi-empirical wave generation theories revealed that 1-2m waves over a fetch of 47km from the east to south-east could be generated by 10-15 $m.s^{-1}$ winds. Marsden Point is therefore subjected to occassional storm wave attack. Tidal dominance in the entrance throat of the Harbour is maintained because of the relative infrequency of these events.

(viii) Strong correlations were noted between the shape of beach profiles on Marsden Point and the onshore-wind, wave-generating potential, defined as the sum of the six-hourly onshore wind speeds raised to the fourth power during a 15-day interval prior to beach surveys.

(ix) With respect to string surveys at the tip of Marsden Point, several interactive mechanisms were identified which are responsible for the transport of sand. Due to the rapid changes in coast orientation, wind and wave directions which influence each location vary widely. It appears that a strong wind from 060^{o} to 090^{o} T will cause erosion near the NZRC wharf, while the more northerly wind from 030^{o} T may induce accretion at the lower beach levels there. Once the wind direction exceeds 100^{o} T, the NZRC wharf is not as greatly affected by wind or wave action whereas the distal tip of Marsden Point is then attacked by waves. Beyond 135^{o} T, the fetches are too small to be of significance, and the winds have little effect at either site, until the angle exceeds about 315^{o} T. Most sediment would enter the harbour with wave assistance during east to south-east storms.

(x) Accretion on Marsden Point was observed to occur slowly over a long period of the order of 100 days, while erosion happens suddenly, usually during one storm episode.

SEDIMENT TRANSPORT AND TIDAL INLET HYDRAULICS

Chapter 5:

TIDAL FLOW VELOCITIES AND IMPLICATIONS FOR SEDIMENT TRANSPORT

CHAPTER 5 TIDAL FLOW VELOCITIES AND IMPLICATIONS FOR SEDIMENT TRANSPORT

5.1 Introduction

This chapter examines the tidal hydraulics of lower Whangarei Harbour and northern Bream Bay. The aim was firstly to determine the water flow velocities and then to correlate these with the sediment transport characteristics of the tidal inlet. This goal was approached by undertaking (i) drogue tracking; (ii) circumferential velocity measurements and a morphological examination of the major ebb-tidal and flood-tidal deltas, Mair and Snake Banks. These bound the proposed port site and the dredge zone intersects the path between them. (iii) an examination of continuous-recording current meter data with respect to the relationship of tidal range to water speed. (iv) vertical velocity profile measurements. These are discussed with particular reference to residual velocities over the tidal cycle and their influence on the tidal inlet sediment transport characteristics.

In all cases, the data is later applied to verification of the 1- and 2-dimensional models in Chapters 9 and 10. This Chapter satisfies an initial aim of ascertaining the present hydraulic regime.

5.2 List of symbols

a	intercept of best fit curves
b	gradient of best fit curves
ΔH	tidal range (m)
q	velocities are raised to this power for residual calculations
T	tidal-cycle period (s)
$T_{\frac{1}{2}}$	half tidal-cycle period (s)
T_{ebb}	time velocity is above threshold during the ebb (s)
T_{flood}	time velocity is above threshold during the flood (s)
u_z	speed at z metres above the sea bed ($m.s^{-1}$)
u_1	speed at 1m above the sea bed ($m.s^{-1}$)
u_{1cr}	sediment threshold speed at 1m above the bed ($m.s^{-1}$)
u_{cr}	mean vertically-averaged speed at the sediment threshold ($m.s^{-1}$)
u_r	x-component of residual velocity ($m.s^{-1}$)
u_*	friction velocity ($m.s^{-1}$)
v_r	y-component of residual velocity ($m.s^{-1}$)
V_r	residual velocity ($m.s^{-1}$)
V_2	Allen's (1980) Symmetry Index
V_{max}	maximum, half tidal-cycle speed recorded at Aanderaa meters ($m.s^{-1}$)
z	distance above the sea bed (m)
z_o	roughness length (m)
θ_{pot}	direction of sediment transport potential
θ_{ebb}	direction of ebb residual
θ_{flood}	direction of flood residual
θ_r	direction of residual
ϕ_{pot}	sediment transport potential

5.3 Drogues

Drogue experiments were conducted to ascertain current strengths and flow patterns in the study region. Drogue paths and speeds offer insights into loose sediment pathways, pollutant dispersion, possible drift patterns of oil spills, flows which may affect ship manoeuvring and even nutrient dispersal

for the abundant shell life in Whangarei Harbour. They may also indicate ebb and/or flood dominance which reflect harbour sediment transport characteristics. Moreover, although the investigation deals primarily with sediment transport and the impact of the proposed timber port on the sediment budget, potential dispersal patterns of material added to the harbour water near the port site were specifically investigated. Ebb and flood experiments were designed to derive harbour flushing characteristics, by examining the paths of drogues over complete tidal cycles after separate releases at both low and high tide.

Experiments were of three types: (i) general path tracking in channels and over the banks (1-6 hours duration); (ii) channel tracking (1-6 hours duration); and (iii) whole, tidal cycle tracking (9-11 hours duration). These investigations offer a considerably more comprehensive coverage than prior drogue experiments undertaken by Bioresearches (1976) to examine harbour flushing characteristics with regard to pollutants entering the water near the NZRC wharf.

5.3.1 Drogue design

The drogues (Figure 5.1) were made to be disposable yet reliable. A wooden frame was covered with heavy plastic sheeting to form the vanes of size 0.60m by 0.65m. This was lashed to a post with a float attached and the depth below the surface of the drogue vane was adjusted by changing the length of the rope attachment. The unit was held vertical with weights tied below.

5.3.2 Computer programming

Drogue positions were fixed either using a sextant or using Motorola Electronic Distance Measuring (EDM) Equipment and the general methods of computer reduction of position fixing data are given in Appendix 1. For drogue tracking, the programming creates files containing reduced positions in Mt Eden Geodetic Metric Coordinates, the fix times, and the drogue and experiment numbers for identification. From this, drogue speeds and travel directions are determined. The speed was defined as the distance between two consecutive fixes divided by the time taken to travel there. This assumes a straight line path between fixes and must necessarily be the minimum speed of the drogue. The error here depends on the distance between fixes and the path curvature. In most cases when fixes were made often, the deviation should be small. The direction is defined to be along the straight line joining the two fixes, and the velocity event is assumed to occur midway between them.

This information is written onto a new file for (i) plotting by the Map Plotting Routine (Appendix 2); (ii) bar plotting to show the speed distributions; and (iii) general statistical analysis (Appendix 3). A small program neatly formats the data for production of computer printed tables. The programs involved were TRISPN, SEXFIX, MAPLT, GRDCON, BARPLT, DRGSTS, DRGPOS, DRGTAB, LETPLT and LINEAR. The complete package allows rapid handling of

positional data and provides mapped output at any scale with bathymetry and selected landmarks included if desired. The statistical analysis provides mean speeds, medians, maxima and minima etc.

5.3.3 Experimental procedures

5.3.3.1 *General tracking*

A series of drogue experiments were conducted from May to July 1981. Up to 4 drogues were tracked during each of 9 experiments and a total of 31 paths were produced. Working from either the University of Waikato jet boat or the University's 4m dinghy, the drogues were positioned with a sextant by shooting two consecutive fixes after the vessel was brought alongside the drogue. Due to boat movement under the influence of tide and wind, the fixing method introduces an experimental error of an unknown magnitude. The results were fastidiously screened by examination of both the positions and associated speeds and any fixes that were considered to be unsatisfactory were omitted. It is estimated that maximum deviations remaining in the data are less than 50m.

The drogues were suspended high in the water column to enable them to drift across the banks, and the centre of the vanes was 0.9m below the surface. At this position, depending on the water depth, the measured speed should lie between the mean vertically-averaged speed and the surface current with the former typically being 10-15% smaller.

The tides and winds were recorded at the gauges on the NHB Marsden Point wharf. Statistics, including release times, experiment durations and tidal ranges for the 9 experiments are listed in Table 5.1. On most days the tide range was just less than the mean range of 1.9m, except Experiment 8 which was on an ebb release off Blacksmith's Creek on a high Spring range of 2.4m. The drogues were released usually 1-1½ hours after the previous slack water to span the period of peak flow. Figures 5.2 and 5.3 show the composite of the ebb and flood paths respectively. Figures 5.4 and 5.5 show the corresponding drogue speeds. Note that plotted positions are joined by straight lines which do not necessarily represent the drogue paths between fixes.

5.3.3.2 *Channel tracking*

Drogue tracking in the harbour channels was undertaken by NHB Civil Engineers, M. Hunter and M. Beazley under the direction of the DHI for numerical model calibration. The data was computer reduced and analysed by this author using the Drogue Tracking Reduction Package discussed earlier in this chapter. The results were examined by this writer and published in Black et. al. (1982).

Drogues were released on Spring tides only, during 8 days in October and December, 1981, and January, 1982. Up to 6 drogues were tracked each day in

the region extending from One Tree Point in the west to Busby Head in the east. Position fixing was performed with Motorola EDM equipment.

5.3.3.3 *Complete tidal cycle tracking*

Hung with the middle of the vanes 1.3m below the surface and tracked with Motorola EDM from the NHB oil spill vessel "Maunu", drogues were set free across the entrance throat between the NHB wharf and Passage Island. When the drogues drifted into "new" zones, the shore transponder station was moved by support personnel working from the NHB jet boat. During the flood experiment only, one channel of the Motorola EDM malfunctioned, so the drogue positions were fixed with a sextant and lengths from the operating channel were used to verify the reduced fixes. The drogue paths were plotted and the distance from each sextant fix to the transponder station was measured off the plot and compared with the Motorola output. In most cases, the sextant fixes were accurate but some displayed deviations which were corrected by moving the fix onto the arc of the Motorola station in the direction of drogue travel. Of the corrections made, most were less than 50m.

Wind and tides were obtained from the two gauges on the NHB wharf. For the ebb experiments commenced at 09:50 on 8-Feb-82, wind speeds were light and variable with a predominant south to south-east direction and the tidal curve is shown in Figure 5.6. For the flood experiments commenced at 07:00 on 1-Mar-82, the tidal range was 2.21m on both the ebb and flood (mean Spring) and the wind speed was approximately 7 $m.s^{-1}$ from 020^{o} True for much of the day. Two drogues disappeared (believed to be due to human intervention) during the first stages of the flood experiment and their paths have not been included on the Figure.

Figure 5.7 from Table 5.2 shows the ebb case paths and Figure 5.8 shows the corresponding drogue speeds. During this experiment, the seaward extent of the ebb jet into Bream Bay was position fixed and results are shown on Figure 5.9. The flood case paths are presented on Figure 5.10 from Table 5.3. Flood case speeds are not given because the fixes were not as close together for this experiment. Due to path curvature, speeds calculated assuming a straight line path may have been misleading.

5.3.4 Drogue results

5.3.4.1 *General tracking results*

Flow out of Parua Bay (Expt. 4, Figure 5.3) turns east across the deep channel south of Reserve Point. Depending on the proximity of the drogue to the land, it will either move into Shoal Bay or into the channel north of McDonald Bank towards Munro Bay. The water speeds in this region (Figure 5.4) were small at approximately 0.4 $m.s^{-1}$ on a tidal range of 1.60m.

Drogues released off Munro Point (Expt 7, Figure 5.4) moved at low speeds

(0.2-0.3 $m.s^{-1}$) across the north-east corner of McDonald Bank and into the channel between the Bank and the shore. A drogue which passed close to the pine-covered point to the north of McLeod's Bay (marked on Figure 5.2) was caught in a lee eddy behind the point and moved into the beach to run aground there. The water speeds were below the sandy sediment threshold.

Drogues released on the ebb between One Tree Point and Reserve Point (Expts 2 and 5, Figures 5.2 and 5.4) revealed that:

(i) from the shallow, western end of Snake Bank, the flow heads north of east towards Shoal Bay and not south of east to the shipping channel.

(ii) in Shoal Bay, there is a gradual change of flow direction enforced by the gently-sloping, south-west face of McDonald Bank. Speeds were about 0.5 $m.s^{-1}$.

(iii) from Shoal Bay, the drogues moved close to Darch Point with speeds of 0.4 to 0.7 $m.s^{-1}$.

(iv) ebb flow down the northern side of the shipping channel headed across the sub-tidal, ebb spit which extends south-east from Snake Bank and moved into the deeper channel beyond.

(v) flow from the south side of the harbour converges on a course past the end of the NHB and NZRC wharves.

Ebb flow out of the harbour entrance gorge (Expts 5 and 8, Figures 5.2 and 5.4) exhibited the following major characteristics:

(i) from the south side of the entrance gorge near the NZRC wharf, when upper Mair Bank was submerged, the drogues moved down the marginal flood channel separating Marsden Point and Mair Bank or diverged when travelling across the upper Bank.

(ii) flow from the northern side of the gorge is likely to reach Bream Bay via the main channel.

(iii) just before high tide, a drogue travelled across Calliope Bank, but this is the only evidence of flow in this direction.

(iv) speeds are low on the Mair Bank ebb-tidal delta to the south of the upper Bank levels, being less than 0.2 $m.s^{-1}$.

(v) in the main channel and on upper Mair Bank, flow is much faster with speeds often in excess of 0.7 $m.s^{-1}$.

Flood drogue releases (Expts. 1, 3 and 9, Figures 5.3 and 5.5) revealed that:

(i) flood flow from central Mair Bank (when the bank was exposed) entered the harbour either in the marginal flood channel or via the small channel at the eastern end of the Bank.

(ii) in the entrance gorge, the paths are apparently wind-affected and the results are inconclusive.

Small-scale phenomena that were noted include:

(i) eddies behind Darch and Home Points on the ebb. After entering these,

the drogues moved against the main stream to be caught in a closed loop.

(ii) a flood eddy in the main channel near the mouth of the marginal flood channel off Marsden Point.

(iii) a flood eddy later identified on the southern end of Calliope Bank north of Home Point. After several trips around a closed loop, the drogue was retrieved.

The eddy off Marsden Point could play an important role with regard to sediment deposition. Flood flow leaves the shallow confinement of the marginal channel at high speed. It expands suddenly in the main channel and would be expected to exhibit some vertical recirculation as is demonstrated with the 2-dimensional model in Figure 10.10. The horizontal eddy circulation noted with the drogue, is probably established by the interaction of the two perpendicularly-aligned horizontal flows: one in the main channel and the other coming off the Bank. In both cases, the eddy circulation should behave as a sediment trap (Ferentinos and Collins, 1980) and may be partly responsible for deposition found in this region (Chapter 2). This would restrict the passage into the Harbour of suspended sand swept off the ebb delta during the flood.

5.3.4.2 *Channel tracking results*

Outside the harbour entrance the major results were:

(i) flow off Home Point was the fastest recorded in the study region at 1.75 $m.s^{-1}$. This is in accord with previous drogue tracking by Bioresearches (1976b).

(ii) the main channel exhibited faster ebb than flood speeds and both were significantly greater than the sediment threshold. Channel ebb dominance complies with standard descriptions of similar features nominated as main ebb channels for their ebb dominant currents (Boothroyd, 1978). However, this result is not in agreement with Bioresearches (1976b, Table 1) who report faster flood currents along this channel.

(iii) the marginal channels off Marsden Point and behind Calliope Bank both exhibited faster flood currents.

(iv) behind Calliope Bank, the speeds averaged 0.16 $m.s^{-1}$ on the ebb and 0.29 $m.s^{-1}$ on the flood. Two-thirds of the measured flood speeds were less than 0.3 $m.s^{-1}$, the approximate sediment threshold speed.

(v) in the marginal flood channel off Marsden Point, both the ebb and flood speeds exceeded the sediment threshold.

(vi) on central Mair Bank, the ebb speeds were very low. Flow leaving the marginal flood channel diverged and slowed considerably there. This suggests that water reaching this region on the ebb will not travel far, and will most probably re-enter the harbour on the next flood tide.

(vii) to enter the harbour from central Mair Bank, drogues near Bream Bay Beach, moved into the marginal flood channel off Marsden Point, whereas

those further offshore - more than approximately 500m offshore - headed around the eastern end of Mair Bank and into the main channel.

(viii) flood flow favoured the northern side of the channel near the entrance throat and the ebb favoured the southern side.

The regional mean speeds are listed in Table 5.4.

Inside the entrance throat the major results were:

(i) the main shipping channel south of Snake Bank, exhibited higher water speeds on both the flood and ebb than the channel through Shoal Bay with mean speeds being up to 50% greater in the shipping channel.

(ii) average speeds in both channels were faster on the ebb than the flood.

(iii) as previously noted, ebb flow was sub-parallel with the south-east, sub-tidal extension of Snake Bank.

(iv) the tendency for ebb flow to converge past the NHB wharf was verified.

(v) it was crudely estimated that on the flood, water to the north of a location about 250-300m south west of Passage Island will move into Shoal Bay with the remainder following the shipping channel.

(vi) flow behaviour past Reserve Point was verified, with ebbing drogues near the land moving north of McDonald Bank and those further south heading down Shoal Bay.

(vii) it took approximately 2 hours for flow to travel from Reserve Point to the entrance throat and 2¼ hours to return.

The regional mean speeds are listed in Table 5.4.

5.3.4.3 *Complete tidal cycle results*

Ebb case: After travelling only from the face of the NHB wharf to be trapped in the pylons of the NZRC wharf, the most southerly drogue was replaced with another dropped in the mouth of the marginal flood channel off Marsden Point (Drogue 3, Figure 5.7). It drifted to Bream Bay via the channel but only reached a maximum southerly extent to the west of Busby Head. It was released early in the tide, but its speeds for much of the half-tidal cycle were extremely low on the seaward flank of upper Mair Bank (Figure 5.8), confirming previous drogue experiments. It is evident that central Mair Bank "stagnates" after the bank is exposed by the falling tide.

After the tide change, the drogue entered the deep, main channel by passing around the eastern (Home Point) end of Mair Bank. It was finally recovered 1500m short of Reserve Point in Shoal Bay, having travelled up-harbour beyond its release point by several kilometres. A second drogue (No. 6) released at mid-tide on the central zone of the Mair Bank ebb-tidal delta, verified the latter stages of this behaviour. It remained just seaward of upper Mair Bank until the tide changed, after which it was swept around the

the eastern end of the Bank to be later retrieved in Shoal Bay.

Prior drogue experiments have shown that flow out of the extreme southern side of the entrance throat (Figure 5.2; Black et. al. 1982, Figure 4) may travel down the marginal flood channel. Thus, pollutants entering the water during the ebb near the NHB and NZRC wharves on the south side of the harbour, may re-enter the harbour on the next half cycle by passing down the flood channel, slowing on central Mair Bank and returning up-harbour during the next flood tide. The location of the streamline in the entrance throat such that flow to its south will pass down the distributary channel rather than travel around the north of Mair Bank in the main channel has a dependence on the state of the tide with relation to the exposure of Mair Bank and has not been precisely determined. However, the drogue paths suggest that it may be just shoreward of the northern face of the NZRC wharf; note particularly, Figure 4 in Black et. al. (1982) when drogues were released 1½ hours after high tide.

The extreme reduction in ebb speeds beyond the upper Bank and the mapped flow directions are in gross disagreement with results published by Bioresearches (1976) who show that across central Mair Bank from the marginal flood channel, the drogues move south-west to unite with flow in the main ebb channel. This behaviour was not confirmed with any experiment (Figures 5.2 and 5.7; Black et. al., 1982, Figure 4). Furthermore, Bioresearches (1976b, Figure 3) show the drogue that travels down the marginal flood channel as moving into Bream Bay to be lost from the harbour system, and this was not confirmed.

Three drogues released near the centre of the entrance throat (Nos. 2, 4 and 5) moved out to Bream Bay in the main channel and after clearing Busby Head, turned east towards Bream Head. When the tide started to flood, they drifted back to the west and it appeared as though they would be re-enter the harbour. Instead, they turned to the south and moved away from the harbour entrance, drifting into an opposing wind. Their continued southerly motion, well into the flood half cycle, made it evident that they would continue to drift to the south, having been lost from the harbour tidal system. A southerly-directed flow in Bream Bay, known as the North Auckland current, was previously described by Tonkin and Taylor (1979a) and is evidently a component of the Continental Shelf circulation along the east coast of the North Auckland peninsula. It evidently effectively aids the flushing of Whangarei Harbour by carrying away water passing beyond Busby Head during the ebb.

The flow patterns identified with the drogues could be verified during this experiment because the extension of the ebb jet into Bream Bay was marked by a distinctive boundary between the sea water and the harbour water laden with suspended, colloidal matter. Called a brown plume for convenience, its boundaries were fixed with EDM equipment on several occasions during the day with the results shown in Figure 5.9. Flood flow travelling west from Bream

Head to Busby Head, forced the brown plume towards Bream Bay Beach mostly near Busby Head. It was less affected further to the south in the region where the drogues exhibited a southerly drift against both the wind and the expected flood currents.

Therefore, the Bream Bay circulation due to tidal and ocean currents is evidently described by three major features: (i) much of the flood flow entering the harbour passes close to the land heading west between Bream and Busby Heads; (ii) ebb currents jet south to south-east and are supported by the North Auckland current; (iii) the strong influence of the ebb jet, maintaining southerly flow in the Bay after the low tide at Marsden Point, should force early flood flow to travel across the ebb-tidal delta shorewards of the jet.

The first two of these should represent resultant circulation directions as well. It is very likely that resultant tidal flow over the delta shoreward of the jet has a flood dominance, although there is no information available to determine how far this may continue to the south of central Mair Bank.

With regard to the sediment transport loop discussed in Chapter 4, the flood movement of the brown plume onto the ebb-tidal delta, should reflect similarly-induced, shoreward sediment movement especially if waves were present to stir the bed sediments. Shoreward, flood tidal currents alone were about 0.35 $m.s^{-1}$ (Figure 5.8) which is perhaps a little less than threshold for unidirectional currents depending on flow depth and bed roughness (Chapter 7).

Flood case: Under the influence of the 7 $m.s^{-1}$ wind from 020^{o} True drogues 1 and 2, released on the south side of the entrance in front of the NHB wharf, (Figure 5.10) were forced towards the nearby shallow inter-tidal zones. Although not normal, this behaviour exemplifies the influence wind would have on the dispersal of oil spills which would be forced onto the inter-tidal zones near the NHB wharf during north winds and flooding tides.

Drogues which moved around the north side of Snake Bank in the Shoal Bay channel completed an anti-clockwise loop around the Bank by returning down-harbour in the shipping channel. In this case, the role of the wind cannot be isolated.

Drogue 4 moved a distance of nearly 8 km up-harbour which compares with a maximum displacement of 12 km reported by Bioresearches (1976b). Had the drogue stayed further north near the shipping channel in the faster current flows, it may have travelled further up-harbour. Once again, the wind influences may have been important.

All drogues were retrieved outside the entrance throat. Two drogues (Nos. 1 and 2) travelled beyond Busby Head into Bream Bay to link with the southerly current identified in the previous experiment. However, they were not released at low tide. No. 1 was dropped on the Snake Bank flood ramp about 2½ hours after slack water and No. 2 was released at high tide. Of the two drogues which were not interfered with throughout the whole cycle, one

grounded on Calliope Bank while the other was caught in an eddy to the north of Home Point. It would be expected that both of these would have been returned to the harbour with the next flood tide.

The results show an ebb resultant. However, the wind influences during the experiment were seen to be significant and difficult to isolate. Millar (1980) arrived at a Spring tide flushing time of 2.2 cycles using the Tidal Prism method and times of 24 and 120 days (winter and summer) using the Fraction of Fresh Water method. The drogue paths suggest that the shorter time of 2.2 tidal cycles is a more realistic flushing time for the lower harbour.

5.4 The two major deltas - Mair and Snake Banks

There are two major deltas in the study region. Mair Bank is an ebb-tidal delta at the entrance to Marsden Point which separates the open ocean from the main ebb-tidal channel. For sediment to enter the harbour, it must pass around or over upper Mair Bank. Snake Bank is a flood-tidal delta located immediately up-harbour of the narrow, entrance throat and strong ebb and flood-tidal currents pass through adjacent channels. These banks play an important role in the inlet dynamics. Bank morphology and sediment-storage capacity, as well as regional, sediment transport characteristics and local, tidal flows all exhibit strong, interactive behaviour.

As such, an investigation of the tidal flows around Snake and upper Mair Banks (including the tip of Marsden Point) was undertaken. The results were verified by examination of large-scale bedforms noted on aerial photographs and by making comparisons with standard models of tidal deltas (Hayes and Kana, 1976). A similar investigation of Calliope and McDonald Banks (ebb and flood-tidal deltas, respectively) was not made, as these are not subjected to such high, wave and current energy levels. Furthermore, the banks chosen for the study bound the proposed timber port site, and the dredge zone intersects the straight line between them. Sediment exchange between the two deltas is expected to occur.

5.4.1 Experimental technique

With an Ott impellor-type current meter, speeds were measured at 0.2m and 0.5m above the sea bed for 50s at each level. Direction was fixed by monocular sighting along the alignment of a streamer tied to the support rod from which the meter was suspended. Water depths were read directly off the rod. Measurements were made for approximately 7 hours, moving between sites in an outboard-powered, small, aluminium boat. Tides were obtained from the Marsden Point wharf gauge and wind velocities from the anenometer on the same wharf at 13m above MSL. Site positions were fixed by sextant, reduced with SEXFIX (Appendix 1), and mapped with the computer package described in Appendices 2 and 3.

5.4.2 Experimental conditions and analysis summary

To ensure exclusion of wind- and wave-induced currents, observations around Mair Bank were made on a windless day of neglible wave activity on 24-Jun-81. The Marsden Point anenometer registered calm conditions. The tidal ranges (Table 5.5) correspond nearly with a mean range for Marsden Point. Thus, according to the Aanderaa analysis presented later in this Chapter, during high Spring tides the speeds may increase by about 50% from those reported. The data are largely summarised in Figures 5.11 to 5.16 which show the measured water speeds and site locations. The Mt Eden Geodetic Metric Coordinates of the sites are listed in Table 5.6. A schematic diagram accompanying an aerial photograph (Figure 5.17) summarises the sediment transport directions inferred from large-scale bedforms and the analysis of field measurements. Two other aerial photographs of the Bank are presented in Figures 5.18 and 5.19. A schematised representation of the dominant sediment transport directions around upper Mair Bank is shown in Figure 5.20.

Around Snake Bank, currents were measured at 5 shallow sites (Figure 5.21) at Sounding Posts SP1H to SP5H. Mt Eden Geodetic Metric Coordinates of the sounding posts are given in Table 1.1. Speeds were also recorded on the intertidal zone fronting Point Sinclair. All velocities are given in Table 5.7 which will be discussed in some detail. The tidal ranges were 1.48 (ebb) and 1.56 (flood) on the 14-Jul-81, then 1.62 (ebb) on 16-Jul-81. These are Neap ranges. Wind speeds on the two days were commonly about 4 $m.s^{-1}$ from 100^{o} True and 5 $m.s^{-1}$ from 180^{o} True on the 14th and 16th respectively. Schematic diagrams accompanying the aerial photographs (Figures 5.22 and 5.23) summarise the sediment transport directions inferred from large-scale bedforms and the analysis of field measurements.

5.4.3 Sediment threshold speed at 0.2m and 0.5m above the bed

Most of the velocity measurements were made at 0.2m and 0.5m above the sea bed. The sediment threshold at this level can be estimated using a logarithmic velocity profile equation if the bed roughness, z_o, and the threshold speed at the more commonly measured level of 1m above the bed, u_{1cr}, is known. Data recorded in the harbour and described in Chapters 6 and 7 can be employed for this determination. For sandy beds, $z_o \simeq 0.0025$m and the sediment threshold speed at 1m was found to be approximately $u_{1cr} = 0.32$ $m.s^{-1}$. The logarithmic Karman Prandtl equation is,

$$u = 5.75\, u_* \log_{10} (z/z_o) \qquad 5.1$$

where the velocity u is measured z metres above the sea bed and u_* is the friction velocity. Manipulation yields,

$$u = u_{1cr} \left[(1 - \log_{10} (z)/\log_{10} (z_o)\right] \qquad 5.2$$

and with the above values, the threshold speed at 0.2m is 0.23 $m.s^{-1}$ and at 0.5m is 0.28 $m.s^{-1}$. This is applied for all the following measurements in shallow water around Mair and Snake Banks without accounting for variations in bed roughness. The result is more sensitive to the choice of u_{1cr} than to z_o. However, it is shown in Chapter 7 that the critical speed at 1m also depends on z_o, and thus, without detailed measurements of bed roughness, the above threshold speeds can only be approximations.

5.4.4 Physical setting and investigation of Mair Bank morphology

At the upper levels, the major axis of Mair Bank is aligned east-west being flanked to the north and east by the main tidal channel and to the south by a large, subtidal delta extending into Bream Bay about 5km to the 10m contour. Beyond this, it drops away more suddenly to over 30m. The bank is separated from the distal end of Marsden Point by a marginal flood channel 150m wide at Chart Datum with a depth of 1.6m at its deepest section (NZ5213, 1981) These channels are typically subjected to dominant flood currents due to (i) the direction of tidal current approach relative to the channel orientation; (ii) tidal current asymmetry associated with ebb tidal dominance in the main channel; and (iii) the support of wave-generated currents (Boothroyd, 1978).

Along the southern edge of the upper Bank levels, a ridge of sediment which could be loosely described as the "swash bar" can be seen in Figure 5.19 and is composed mainly of a shell lag. Mair Bank is unusual in that its major axis and the swash bar are almost perpendicular with the ocean coastline, due to the interaction of the ebb jet and the sheltering afforded by Bream Head. The orientation is more normally parallel with the coast when waves oppose the ebb jet and force extruded sand back towards the beach.

At the northern end of the marginal flood channel, a channel-margin, linear bar (equivalent to a levee) can be seen in Figure 5.17. This is associated with the interaction of the ebb currents leaving the entrance throat and the flood currents in the marginal flood channel. At the southern end of the flood channel, a terminal lobe is evident. The presence of this feature suggests that the ebb currents have sufficient energy to carry sand along the marginal flood channel for deposition there.

The runnel separating the beach and offshore bar acts as a channel which, because of its proximity to the beach, should be strongly flood dominated. Here and at Marsden Point, the bed is sandy. Megaripples are found in the inter-tidal zone and on the flood channel margin off Marsden Point. To the north, a large area of scoured shell lag with an ebb-dominant orientation is found (Figure 5.17). This shelly area was shown in Chapter 3 to be very stable (S3.1, Figure 3.24) and is evidently under-supplied with sandy sediments.

5.4.5 Velocity measurements around Mair Bank

At the northern end of the marginal flood channel (SP1M in Figure 5.16), ebb speeds attain 0.6 $m.s^{-1}$ and flood and ebb speeds exceed the sediment transport threshold for approximately 208 and 188 minutes respectively (Figure 5.11). This is more than half the tidal cycle and as the bed in this area was identified as loose, megarippled sand (Chapter 2; Black et. al. 1982; Black and Healy, 1982) significant quantities of sediment are being moved through this region by tidal currents.

In the deepest part of the marginal channel about half-way along its confined length at SP2M, fast ebb and flood speeds, reaching maxima of 0.62 and 0.72 $m.s^{-1}$ respectively, were measured just below the water surface (Figure 5.12). These speeds are both far in excess of the sediment threshold and maintain the channel in this vicinity. The ebb flows are fast enough to carry sand seawards for deposition on the terminal lobe, identified on the aerial photographs.

In mid-channel, the bed is composed of shell, largely Tuatua (Papies subtriangulatum) and Cockles (Chione stuchburyi), with interstitial sand (Black et. al., 1981, Site 29). Loose bedformed sand flanks the channel margins (Figure 2.7). In the period from 1959 to 1981, the flood channel has apparently undergone accretion of more than 1m in places (Figure 3.17), and approximately 0.4m at SP2M. Mair Bank has moved to the north and west encroaching on the flood channel. The shell may play an important role in this, not only by slowing erosion, but also, by assisting sediment accretion. Diver observations at other locations in the harbour revealed that loose shell retards erosion and also encourages accretion by trapping sand grains which fall between the coarse shells. The tidal currents then have more difficulty reinitiating sediment transport, and sedimentation progresses.

Located in the partially-blind, flood channel, shorewards of the marginal flood channel, the Beach site 120m shorewards of SP2M is strongly, flood dominated to the extent that ebb speeds never exceed the sediment threshold speed (Figure 5.13). However, flood velocities are above threshold for much of the half-tidal cycle (Figure 5.13) and visibly proved by water clouded with matter in suspension observed from the boat during the flood tide only. Thus, both the tidal sediment transport and the littoral drift (Chapter 4) are directed towards the harbour along Bream Bay Beach at this site.

High levels of turbulence and "clouds" of suspended sediment were seen from the boat at the Marsden Point site, and measured water speeds on Figure 5.14 always exceeded the sediment threshold. This is a very active location where high flows work the bed into cuspate megaripples seen in the inter-tidal zone at low tide. The amplitudes of these features oscillate in phase with the Spring-Neap tidal cycle. Thus, they are constructed during each half cycle. Over bedforms, the vertical velocity profile varies with the horizontal

measurement position relative to the bedform crest (Raudkivi, 1963). This is reflected by the wide variations in the velocity gradient measured at this site (Figure 5.14).

The shallow site in the "middle" of Mair Bank is exposed at low tide and is subjected to high water speeds far exceeding the sandy sediment threshold (Figure 5.15). The bed is a scoured, shell lag which was not visibly moving at peak flow. The threshold speed for the initiation of movement of the shell must therefore be greater than 0.81 $m.s^{-1}$ at 0.5m above the bed. For a roughness length of 0.001m using the logarithmic Karman-Prandtl profile, this corresponds with a 1m speed of 0.90 $m.s^{-1}$. Thus, the shell is extremely stable.

The speeds and velocity asymmetry at this and at the previous Marsden Point site are almost identical, yet one is sand-covered and the other is shell lagged. The difference depends on the availability of sand in transport, and implies that the middle of Mair Bank is not an active sediment pathway. Notably though, bed undulations which could possibly be described as "shell megaripples" were identified on the Bank. These caused surface waves to develop at the measured Froude Numbers of 0.2.

5.4.6 Residual velocities around Mair Bank

The residual velocity is discussed in detail later in this Chapter when velocity profile measurements are reviewed. In summary, the residual is a nett current velocity over a tidal cycle at any location. It specifically represents degrees of ebb and flood dominance and, with some caution depending on the magnitudes of the water speeds, the residual reflects nett sediment transport through a region under the action of tidal currents only. It is suggested that the reader may care to examine Section 5.8 before continuing here.

In relation to the sparseness of the data, approximate residuals around Mair Bank were obtained as follows. Over each half cycle, speeds at 100, 150, 200 and 250 mins before and after high tide were found by linear interpolation of the measured speeds. To form the residual these were averaged, then ebb and flood average speeds were added as vectors with directions given by the mean of the measured angles with each half cycle treated separately.

Plotted in Figure 5.16, the residuals present a clockwise-rotating loop resulting from the interaction of the flow with the regional geometry. As the measurement sites move out of the shadow of Marsden Point, the dominant flood vectors turn through nearly 180^{o} to have ebb orientation over the shell lag on the upper platform of Mair Bank. This loop is of smaller scale than the previously-deduced Bream Bay sediment loop based on tidal discharge and wave interactions crossing the southern segment of the ebb-tidal delta (Chapter 4).

The smaller loop concurs with (i) interpretation of aerial photographs; (ii) the standard model of an ebb-tidal delta; (iii) depictment of ebb-directed sand wave orientation near SP1M in Figure 2.7; and (iv) the orientation of Marsden Point which is a prograded spit. Although no specific details will be discussed here, residual velocities at 1m above the bed at three velocity profile measurement sites around Mair Bank offer further supportive evidence. Shown in Figure 5.42, as well as identifying the flood dominance in the marginal flood channel and the ebb residual across the northern side of the Bank, strong flood dominance is evident in the small channel at the eastern end of the Bank. Predominant flood flow through this channel has been already indicated by drogues which entered the main channel via this route.

Thus, the tidal residual velocities are well supported by other evidence. As such, these residuals are summarised in a schematic diagram in Figure 5.20, and reflect nett sediment transport directions.

5.4.6.1 *Sediment transport implications*

To cope with the strong, flood-directed tidal velocity asymmetry in conjunction with a northward littoral drift, a downstream supply of sediment is required to maintain beach equilibrium at the site 120m shoreward of SP2M. Maximum flood speeds at this location of approximately 0.8 $m.s^{-1}$ at 1m above the bed (found with the Karman Prandtl equation from the 0.5m speed), correspond with a high, tidal sediment transport rate of 0.06 $kg.m^{-1}.s^{-1}$ (Figure 7.33) and is evidenced by the clouds of sediment stirred up during the flood. If the current acts for an average of 2 hours per tidal cycle, the sediment flux converts to $208/m^3/m/year$ for a porosity of 0.45 and sediment density of 2650 $kg.m^{-3}$. Over an approximate 100m width of bed this corresponds with an input of $20,000m^3/year$. This does not include the extra input due to wave-generated, littoral transport, but is based on tidal currents only. Also, no allowance has been made for the influence of tidal range on the water speeds. The ranges were nearly equal to the Marsden Point mean range of 1.9m during measurement and therefore annual rates could be slightly larger than the figure quoted.

As presented in Table 3.3, the DHI determined the volume changes to the Bank between the 1959 and 1981 RNZN surveys and although there was considerable accretion above the 1m contour this was almost entirely balanced by corresponding erosion between the 1m and 3m contours. Independent analysis by this writer summarised in Figure 3.17 shows the regions where this is occurring. The DHI found that there is nett accretion above 3m of only 910 $m^3/year$. This volume could easily be supplied by tidal currents alone, according to the above result. A considerable flux of sediment is therefore passing right through the region and as there is no noted accretion inside the harbour up to One Tree Point (Figure 3.16), excess sediment is evidently being swept around or over

Mair Bank by the ebb currents and this is shown by the residuals. The data do not imply that no sand is entering the harbour. Particularly during storms, sand lifted into suspension has a potential to enter the harbour with supporting flood currents. However, the evidence shows that much of the sand on Marsden Point is swept away by ebb currents.

The ebb residuals on Marsden Point and to the north of Mair Bank, are then responsible for the transportation of sand causing deposition noted along the main channel flanks to the north and east of Mair Bank (Figure 3.17). With the northward movement of the western end of Mair Bank (Figure 3.2), excess sediment spilled off the Bank near the northern end of the marginal flood channel could be swept around the Bank by ebb currents with the noted accretion resulting. Moreover, the constriction of the channel is apparently being compensated for by erosion on the opposite, Calliope Bank side (Figure 3.17). There are several major results of this analysis. Primarily:

(i) The tidally-dominated segment around northern Mair Bank of the large Bream Bay sediment loop implied in Chapter 4, is well demonstrated by these data.

(ii) The ebb-dominated inter-tidal zones on upper Mair Bank and on Marsden Point are both shell lagged denoting an imbalance of sediment supply. Thus, the shell beds prevent erosion of these regions.

(iii) The vector tidal velocity residuals, aerial photographs, and the model of a standard ebb-tidal delta provide concordant descriptions of the sediment transport patterns in the Mair Bank/Marsden Point region, summarised in Figure 5.20.

5.4.7 Physical setting and investigation of Snake Bank morphology

A morphologic model of a standard flood-tidal delta (Hayes and Kana, 1976) is shown in Figure 5.24 and Snake Bank exhibits most of the features. As summarised by Boothroyd (1978), the major components are:

(i) "flood ramps, the seaward-dipping surfaces dominated by flood tidal currents with major sediment movements accomplished by the migration of sand waves landward of the ramp". On Snake Bank, the flood ramp and the landward migrating sand waves can be seen on the aerial photographs (Figures 5.22 and 5.23).

(ii) "flood channels, sub-tidal continuations of the flood ramp that separate the main inter-tidal flats of the delta and are dominated by flood-tidal currents". No flood channels are found on Snake Bank. However, a flood channel disects the inter-tidal sand flats near Point Sinclair (Figure 5.21).

(iii) "an ebb shield, the topographically-high, landward margin of the tidal delta that protects portions from modification by ebb tidal currents". The ebb shield is plainly evident and weaves across Snake Bank in snake-like fashion and is responsible for the Bank's name. Millar (1980) reported that the snake is composed of disarticulated cockle shell. The location of the

ebb shield is offset towards the harbour entrance. Up-harbour of the shield, ebb currents should be dominant. Accordingly much of the flood-tidal delta is subjected to stronger ebb currents. This was also inferred from the sea bed gradients in Chapter 3. Ebb-directed sand waves seen on the aerial photographs up-harbour of the ebb shield offer further, supportive evidence.

(iv) "ebb spits, formed chiefly by ebb tidal currents with some interaction by flood currents". These spits were noted with the photography and side-scan sonar survey and, because of their importance to the Harbour sediment transport characteristics, diver verification of their presence was undertaken (Chapter 2). One extends in a south-easterly direction off Snake Bank beside the shipping channel and heads towards Marsden Point. The second separates the southern flank of the Shoal Bay channel and the Snake Bank flood ramp.

(v) "spillover lobes, linguoid-barlike features formed mainly as a result of ebb-tidal current flow over low areas of the ebb shield". No spillover lobes were identified on Snake Bank.

The main feature not classified by the standard model is the channel between the ebb shield and the flood-directed sand waves, named by this writer on the aerial photographs as a "flood current, spillover channel". It should be flood dominated and may have been established by flow converging at the ebb shield causing current acceleration and scour in the channel.

5.4.8 Velocity measurements around Snake Bank

During the Neap tides when measurements were made, the velocities at the shallow sites around Snake Bank were below threshold for much of the time. At SP1H, SP2H and SP3H (Table 5.7), speeds exceeded the threshold of 0.28 $m.s^{-1}$ at 0.5m above the bed for a short time during the ebb only. SP5H was always below threshold. Thus, bed sediment will only be disturbed at this site by faster currents which would occur at Spring tides, or by the action of locally-generated surface waves. The bed was rippled denoting prior activity but covered with algae. Marine growth can stabilise the sediments (de Boer, 1981) and as such, the periods of inactivity during the Neap tides may affect the transport rate during Spring tides. At SP4H (Table 5.7) the threshold was clearly exceeded during the flood but was only just attained on the ebb. The bed here was composed of loose sand with ripples at an oblique angle to the ebb current. The ripples were apparently generated during the flood, and the ebb speeds were not sufficient to modify them.

Many shallow regions around Snake Bank during Neap tides, are seen to be either stable or subject to one-way sand transport (either ebb or flood-directed). These sites are much more susceptible to wave-induced sediment transport than the deep channels. Because of the small wind fetches in the harbour, only short-period, low amplitude waves are generated (Chapter 8) and

these only interact with the bottom in shallow water. It is likely that much of the sediment movement in some of the less active zones is therefore wave, rather than tidally induced, as was found to occur on the inter-tidal zone at the proposed port site (Chapter 8).

5.4.9 Dominant transport directions around Snake Bank

The data was too sparse to accurately calculate residual water speeds over the tidal cycle. Instead, dominance was assigned to the ebb or flood direction according to which half cycle attained the highest speeds. The direction was found from the average of the measurements during the dominant half cycle. No magnitude was assigned. The directions are listed in Table 5.8 and plotted on Figure 5.21.

The results comply with the inferences from the aerial photographs made in conjunction with an examination of the standard model of a flood-tidal delta of Hayes and Kana. Of particular note, are the vectors on the bank at SP2H and SP5H, which are nearly oppositely aligned. Accordingly there needs to be a neutral zone between these, which could be recognised by deposition. Indeed, this is the ebb-shield separating the ebb and flood-directed sand waves evident on the aerial photographs.

The essential features of the sediment transport patterns around Snake Bank are : (i) Sand entering the harbour at the throat is moving onto Snake Bank by passing up the flood ramp. (ii) This is bounded to the north and south by bedformed, ebb-tidal spits. (iii) Beyond the flood ramp, sediment is directed up-harbour by flood dominant currents around the north-east flank of the Bank. This region was identified as active during the side-scan sonar survey and divers have verified its presence (Chapter 2). The zone is linked with sediment that enters the harbour up the Snake Bank flood ramp and according to the drogue paths, this transport initiates on the northern side of the entrance gorge. (iv) Sand is moving onto the slow-gradient, western flank midway between One Tree and Reserve Points. (v) Much of the crest of the Bank, up-harbour of the ebb shield, exhibits ebb-directed sediment transport.

Finally, at the entrance to the blind flood channel leading onto the sand flats at the Point Sinclair Front Lead, the expected flood dominance is evident (Figure 5.21).

5.5 Full tidal cycle, velocity profile measurements

Over nearly a four-year period, the vertical velocity profile was measured at intervals throughout the tidal cycle at some 58 locations. This very detailed coveraged allowed further specification of the harbour hydraulic conditions and is later unified with the sediment transport investigations in Chapter 7, to ascertain the fluxes of sediment at these representative locations around the harbour. In the following sections, the profile data is

examined for flow magnitudes, ebb and flood dominance, harbour hydraulic characteristics, and to establish a general specification of regional flow speeds.

5.5.1 Methods

Impellor revolutions, directions to the nearest 10^{o}, temperature and salinity were recorded with a Braystoke multiparameter, MKII, BFM010 meter. A built-in sounder provided distance of the meter from the sea bed. The instrument was winched between levels and the data was mostly recorded either at 1, 3, 5, 7m etc. from the sea bed or at 0.5, 1, 2, 4, 6, 9m etc.

At Sites A to N, an average speed was obtained at each level over 50s. However, due to the effects of turbulence (see Chapter 6), later measurements, this time performed by the writer, were made for 100-150s at each level to improve the average and smooth out the turbulent oscillations more completely. Dyer (1973) suggests averaging for a time equivalent to that taken for the water to flow a distance equal to 10 times the depth. The Whangarei measurements roughly confirm with this "rule of thumb", although the data still exhibits severe fluctuations in some specific locations.

Vessels used were the University of Waikato jet boat and the NHB oil spill vessel, "Maunu".

5.5.2 Computer programming

The large recorded volume of velocity, temperature and salinity data necessitated computer manipulation. The major velocity analysis program written for this study was CUR4, which primarily finds the bed frictional losses, bed shear stresses etc. (Chapter 6), but it was also utilised in this Chapter for determination of mean speeds, and for its general statistical and file manipulating capabilities. Other programs written for the study to reduce the profile data allowed; (i) input of raw data; (ii) format manipulation in preparation for entry into the 8-column packages (Appendix 3); (iii) specific table preparation and graph plotting; and (iv) residual speed determination. The programs used for the velocity analysis in this Chapter were BRAINP, BFILES, BRADIF, BRAVEL, CUR4, LINSTS, VELPLT, MEANS and RESULT.

5.5.3 Choice of sites

Figure 5.25 shows the locations of the Braystoke sites and speeds at some of these were measured during two separate tidal cycles. The measurement program was undertaken by several different organisations including a consulting firm (Sites A and B), but mostly by the NHB with direction from this writer, the DHI and Dr T. Healy (University of Waikato). This writer was directing or assisting during field measurements at 46 sites.

Site locations were chosen by this writer for certain characteristics: non-cohesive sand beds (S-series), shell beds, suspected ebb or flood dominance

and timber port proximity. At several locations, sediment trapping was undertaken simultaneously and this is discussed in Chapter 7. The A, B, C, and D series were required by the DHI for model calibration.

Table 5.9 compiles relevant site characteristics and specifies locations, depths, temperatures, salinities, and tidal ranges.

5.5.4 Vertical profiles and the hydraulic regime in Whangarei Harbour

Typical vertical profile velocity measurements recorded throughout the tidal cycle are shown in Figure 5.26. At all levels the speeds oscillate virtually in phase with the seim-diurnal, tidal-cycle period. They exhibit an expected increase in velocity as the surface is approached, caused by frictional retardation at the bed.

In Figure 5.27, the distributions of measured velocities at 1m above the bed for all sites combined typify the speeds in Whangarei Harbour and to some degree define the hydraulic character of the estuary. The ebb distribution is more peaked than the flood, in partial response to a shorter ebb, half-cycle period. The mean ebb and flood velocities at 1m were 0.42 and 0.38 $m.s^{-1}$ and the maxima were 0.99 and 0.90 $m.s^{-1}$ respectively. Notably, the means suffer from sampling errors due to biased selection of site locations, unequal sampling intervals over the tidal cycle, and the experimental technique where speeds were sometimes not recorded near slack water.

The velocity-time profiles for the 1m speeds are shown for all sites on Figures 5.28 to 5.31. Superimposed on the curves having semi-diurnal periods, are oscillations with much shorter periods of the order of ½-2 hour duration. These may be caused by seiching within the estuary or established by shelf oscillations outside the entrance (Chapter 9).

The ebb curves are typically steeper and of shorter duration than the flood curves. Furthermore, peak speed occurs closer to high than to low tide during both the ebb and flood for most sites. These characteristics are due to the harbour morphology and to the interaction of the sea slope which accelerates the flow, and the bed friction which causes retardation. A balance is struck on the deeper side of mid-tide during both half cycles.

Most sites at Whangarei exhibit this behaviour with the exception of Site 2 during the ebb and Sites (C, D, I, M, A1, B1) and (D2, D3) during the flood. At the exceptional Site 2, only 3 measurements were made during the ebb and so possibly the peak flow was missed. During the flood, the anomalous sites in the first group are located near the NZRC and NHB wharves and the fastest flow occurs just after low tide. In the entrance throat, the ebb flow continues past the time of low water and into the flooding stage of the tidal cycle. The flooding, open-ocean water sneaks around the edge of the ebb jet and enters the inlet throat via the marginal flood channel then along the southern shoreline. Thus, high speeds occur during the early flooding

portion of the cycle when the flow is constricted to the channel margin. Boothroyd (1978) reported that this behaviour is typical of tidal inlets. Furthermore, Dahm (1983) in Tauranga Harbour noted the same behaviour with early flooding occurring on the southern side of the inlet throat while the main, ebb jet continued to flow.

Notably the water speed does not necessarily pass through zero at the time of high or low tide (Figures 5.28 to 5.31) and is seen to lag behind the change of tide by 40-60 minutes in some cases. This phenomenon is due to (i) inertial flow continuing after the tide change (e.g. Site 9 on the ebb) and (ii) local variations in sea slope established by basin morphology. Site K is an excellent example: speeds measured after two different high tides were still flooding 45 and 55 minutes later. Verified by the 2-dimensional model (Chapter 10), this behaviour is related to a pattern of flooding on the northern side of the inlet throat at Marsden Point after the southern side has started to ebb.

5.5.5 Temperature and salinity

Further to the velocity measurements made at the Braystoke sites, temperature and salinities were also recorded throughout the tidal cycle. The high and low tide values for each site are given in Table 5.9. As the high tide temperature generally corresponds with water from Bream Bay, and low tide is associated with temperatures in the upper harbour, the results show that in summer the upper harbour temperature is elevated above ocean levels by solar heating with the opposite occurring in winter. The estuary is thus more directly affected by immediate weather conditions than the deeper ocean. Further temperature measurements made using Aanderaa current meters and presented by the DHI (1982, Figure 3.2a to d), varied with tidal cycle, diurnal and seasonal periods, and with another component related to the persistance of weather conditions. Table 5.9 reveals that ocean winter temperatures are 12-14^{o}C while summer temperatures are about 20^{o}C.

The estuary is mostly well-mixed, with salinities at all sites, including those in Bream Bay, varying between 33.4-34.6‰ at high tide, and 31.3-34.5‰ at low tide (Table 5.9). Millar (1980) measured similar salinities. From the narrow range at high tide, it is evident that ocean water disperses throughout the study region up to One Tree Point - again denoting low flushing times. The wider variation at low tide and the generally lower salinities reveal that some fresh water is entering the upper harbour, especially in winter when the lowest salinities were recorded.

Horizontal salinity gradients are small with a maximum high tide to low tide variation of 2.4‰ measured at Site 3 in the main shipping channel near S2. For a half cycle water displacement of 10 km, this corresponds with a salinity gradient of 2.4×10^{-4} ‰.m^{-1}. However, after heavy rainfall, gradients near fresh water inflows such as Blacksmith's Creek may be more

significant.

Examination of the hydrodynamic equation (Chapter 9) reveals that, in estuaries open at one end with the highest salinity usually recorded at the ocean mouth, the density gradient term is always positive throughout the tidal cycle, so will tend to reduce flood friction coefficients and increase ebb values. The longitudinal density gradient was incorporated into the 1-dimensional model and it was shown to have a small, measurable effect.

5.6 Aanderaa current meters

Sternberg (1979), in Western Port Bay, Australia, found a distinct non-linear relationship between the maximum speed at 1m above the sea bed and the ratio of tidal range, ΔH, over the half tidal period, $T_{\frac{1}{2}}$. A relationship of this kind is extremely useful for prediction of speeds at a site over a range of tidal conditions when measurements were only made on one occasion. This can also be applied to the determination of annual sediment transport rates. A similar relationship in Whangarei Harbour was sought using speeds measured with four continuous-recording Aanderaa current meters deployed in the period from October 23 to December 18, 1981.

The meters were sited by the NHB under direction from the DHI to monitor the shipping channel near One Tree Point (OTP), the Shoal Bay channel (SHOAL), the port site off Blacksmith's Creek (BLACK), and the southern side of the entrance gorge between the NHB and ZNRC wharves (NHB). Locations are shown in Figure 5.25. Magnetic tapes from the current meters were reduced and plotted by the DHI (1982, Figures 3.2a to d). The current meters were placed near mid-depth (DHI, 1982, Table 3.2) so the measured velocities approximate the mean vertically-averaged velocity. For the following analysis, the maximum velocity attained during each half tidal cycle was taken from the Figures presented by the DHI. Thus, maximum flow speeds only are examined. Tidal information was read directly from the original charts recorded on the Marsden Point gauge.

5.6.1 Velocity magnitudes

During the two-month recording period, maximum speeds at the Aanderaa sites averaged from 0.51 to 0.86 $m.s^{-1}$ (Table 5.10). The speeds ranged from approximately 0.46 to 1.21 $m.s^{-1}$ during the ebb and 0.38 to 1.07 $m.s^{-1}$ during the flood. Thus, high Spring tides induce large increases in maximum water speeds at all sites with changes exceeding 100% of the Neap maximum. Mostly, the water speeds exceed the sediment threshold for some of the tidal cycle. As the bedload transport rate shows correlation with the excess speed above threshold raised to the third power (Chapter 7), the velocity variations from Spring to Neap tides would correspond with at least an order of magnitude increase in the potential transport rate.

The comparison of mean ebb and flood maxima only (not the tidal residual speeds), shows that BLACK and NHB are strongly ebb-dominated, while OTP and SHOAL are nearer neutral but apparently ebb-dominated.

5.6.2 Relationship of tidal range to current speed

In Figures 5.32 to 5.39, the half cycle maximum speeds, V_{max}, attained at the four Aanderaa meters over the two-month measurement period, were plotted against ΔH and $\Delta H/T_{\frac{1}{2}}$ for ebb and flood separately. Best fit linear curves against ΔH had a significantly high correlation coefficient (shown on the Figures) and the linear relationship can be accepted. However, the correlation did not improve against $\Delta H/T_{\frac{1}{2}}$. The duration of flow during a half tidal cycle should be inversely related to the maximum water speed attained for a given tidal prism. This is the justification for forming the ratio of $\Delta H/T_{\frac{1}{2}}$. However, $T_{\frac{1}{2}}$ does not always represent the duration of ebb and flood flow at every site. The latter depends on site location as well as wind and atmospheric conditions. As a result, the half tidal cycle period is not a dominant factor and the maximum speeds are as accurately predicted from the tidal range alone.

The data was then fitted to four curves with only the range as the independent variable i.e. (i) linear through the origin, $V_{max} = b.\Delta H$; (ii) linear $V_{max} = b.\Delta H + a$; (iii) power, $V_{max} = a(\Delta H)^b$; and (iv) exponential, $V_{max} = a.e^b.\Delta H$. In all cases a and b are unknown constants. Table 5.11 compiles the results and the exponential curve exhibited the worst fit and was rejected. The linear and power curves which both have two degrees of freedom (a and b), have very similar correlation coefficients which are significant in both cases. The type (ii) best fit linear equations were plotted on the same axes in Figure 5.40 and maximum water speeds in the vicinities of the four Aanderaa meters can be estimated by entering with the tidal range.

It is noted that two of the four types of curve tested do not pass through the origin. This is felt to be unimportant with regard to representation of a physical process which never exhibits tidal ranges less than about 1.0m. However, application is considerably simplified with Type (i).

5.7 Profile velocities scaled to a standard, tidal range

The Type (i) equation can be used to scale velocities according to the tidal range without regard to site location.

If $u = b.\Delta H$, where b is an unknown constant, then,

$$u_2 = u_1 . \Delta H_2/\Delta H_1 \qquad 5.3$$

where u_2 is the velocity scaled to the range ΔH_2 and u_1 was measured when the tidal range was ΔH_1. As the constant b is not included, the relationship does not depend on site location.

However, the best fit linear curves did not pass through the origin (Table 5.11) so this assumption is subject to some error which will increase as the difference between the actual and selected tidal ranges widens. As seen in Table 5.11, the intercept was mostly small, and the errors should be considerably less than those which arise if tidal range is not accounted for. Furthermore, even though the linear relationship was only shown to exist between the maximum speed attained near mid-depth and the tidal range, for successful application it is necessary to scale all of the velocity time curve in this fashion. Examination of results below shows this to be a reasonable assumption.

With this procedure, all velocity measurements at sites throughout the harbour can be scaled to a standard tidal range for direct comparisons. This was undertaken for the maximum of the mean vertically-averaged velocities ($\bar{u}_{max}$) and 1m velocities (u_{1max}) and results are compiled in Table 5.12 for a standard Spring tidal range of 2.2m. The effectiveness of the method, can be gauged by comparing scaled maxima at sites that were visited twice or at neighbouring locations and the results are adequate, with mostly a much improved speed correlation after the scaling.

The fast ebb sites, defined as having $u_{1max} > 0.6$ m.s^{-1} and $\Delta H = 2.2$m, were all in the main shipping channel and include all sites in the channel except 3, B, E, B1 and A2. Site 3 is protected from the ebb currents by an upstream, sub-tidal spit and the other 4 sites are all tucked closely into the southern shore of the harbour near the NHB wharf. Speeds would be slower there simply because of the shallower depths. Site A2 however, is an exception being offshore from faster Sites A and A1 and possibly, peak ebb flow was not measured at A2. Site S5, found on the megarippled, 5m contour in Shoal Bay, was the only site not in the main channel which recorded speeds faster than 0.6 m.s^{-1}.

Fastest flood sites are those on the north side of the inlet throat (9, B3 and B4); those to the north and in the middle of the shipping channel (A4, K, S1, S2, S3, S4, A2, 2, 3, C2 and C3); one in the Marsden Point marginal flood channel (5); and one in the main channel north of Mair Bank (6). All of these ebb and flood sites have the potential to transport sediment.

There is a considerable difference in the locations of the fast ebb and flood sites. At the throat, strong flood occurs on the northern side and fast ebb to the south. Near cross-section A, the ebb is fast right across the channel but the flood is only fast towards the northern side. In the shipping channel near One Tree Point, all the C series sites during the ebb are fast including those in 5m of water near the channel margins, whereas fast flood is confined to the middle of the channel.

Notably, very high speeds were recorded on the megarippled, sub-tidal spit extending off the south-east of Snake Bank at Sites S1-S4, K and A4.

These are some of the largest measured speeds with velocities at 1m exceeding 1 $m.s^{-1}$ at S1 and K. There is therefore, a considerable potential for the transport of sand along this spit.

The speeds through the main channel (C series) are considerably faster than their counterparts in Shoal Bay (D series). The respective averages of the maximum 1m speeds across each section are 0.67 and 0.43 $m.s^{-1}$ for the ebb and 0.54 and 0.40 $m.s^{-1}$ for the flood. These are 56% and 35% increases. This difference in speed can be inferred from the bathymetry of the two channels - the hydraulic radius is considerably smaller in the Shoal Bay channel (Chapter 3) - and was noted with previous drogue experiments and with Aanderaa meters.

At the proposed port site, the maximum ebb currents are stronger than the flood. The A series shows a cross-channel, horizontal velocity gradient with $\bar{u}_{max}$ increasing from 0.75 to 0.95 $m.s^{-1}$ towards the north. Site F, which is at the north of the dredge zone, peaks at 1.06 $m.s^{-1}$ for the mean Spring tidal range.

5.8 Residual velocities

Under the action of reversing tidal currents, sediment transport rates are constantly adjusting to the governing water speeds. Thus, the residual or resultant velocity integrated over the tidal cycle which accumulates these velocity variations, may reflect the nett movement of sediment. As a second example, the residual velocity strongly influences the rate of pollutant dispersal, and harbour flushing characteristics. The residual velocity is therefore an effective, descriptive parameter reflecting aspects of estuarine behaviour. Specifically, the residual ascertains degrees of ebb and flood dominance, which may be examined with respect to regional sediment transport resultants.

5.8.1 Definition

In integral form, the residual is related to the area under the velocity-time curve, and is defined as,

$$\tilde{V}_r = \frac{1}{t_2 - t_1} \int_{t_1}^{t_2} \tilde{v}.dt \qquad 5.4$$

where the velocities are treated as vectors. This is the definition of an average velocity which can only be considered a residual when integration is undertaken over one complete cycle i.e. $t_2 - t_1 = T$. In a discrete form, and treating orthogonal components separately,

x-velocity: $$u_r = \frac{1}{N} \sum_{i=i}^{N} (v_i - v_{cr})^q \cos \theta_i \text{ for } v_i > v_{cr} \quad 5.5$$

y-velocity: $$v_r = \frac{1}{N} \sum_{i=i}^{N} (v_i - v_{cr})^q \sin \theta_i \text{ for } v_i > v_{cr} \quad 5.6$$

magnitude: $$V_r = (u_r^2 + v_r^2)^{1/2q} \quad 5.7$$

direction: $$\theta_r = \text{atan } (v_r/u_v) \quad 5.8$$

where v_i is the ith speed and θ_i is its direction, in a right-handed, horizontal orthogonal axis system. v_{cr} is a critical water speed which can be equal to zero and q is a chosen power. At this stage, if q = 1 and $v_{cr} = 0$, the equations define the average velocity over the period, $T = N\Delta t$, and Δt is the time increment between N discrete velocities.

If summed over the tidal cycle, the residual represents the Eulerian resultant flux of velocity past the measurement point. In recilinear flow, the residual can be zero when the sum of the upstream speeds during a half cycle equals the sum of the downstream speeds, irrespective of the individual velocity magnitudes. The residual thus measures the degree of velocity asymmetry at the location. Positive or negative residuals are associated with flood or ebb dominance.

The precise meaning of the equation for a power of q other than 1 is not so straightforward, but in specific terms, it represents the velocity found from the qth root of the area under the velocity-time curve with speed raised to the qth power. The reason for introducing q was to represent a process, such as the sediment transport potential at the measurement site, which is a non-linear function of water speed. In the algebraically-simple Gadd et. al. (1978) formulation, the sediment transport rate is proportional to the cube of the excess velocity above the sediment threshold. The residual calculated with q = 3 and v_{cr} set to the threshold velocity represents this better than the linear formulations.

A residual distance was defined as the product of the residual velocity and the time that the speed was above threshold. This can be a better indicator of the nett movement of sediment, for example, than the residual velocity which may be large but short-lived. The total work done on the bed depends on the period over which the residual velocity acts. When the threshold is zero, residual velocities may be directly compared, as the time period is then always one tidal cycle.

5.8.2 Calculation

As very detailed velocity profile measurements were made in Whangarei

Harbour, a computer program called MEANS was specially written to derive accurate residual velocities. The program uses velocity profiles previously corrected for non-simultaneity (Chapter 6), and operates as follows:

(i) residuals can be found using either mean vertically-averaged velocities or velocities at a chosen level above the sea bed. In the former case, the program calculates the mean speed from the profile using a parabolic interpolation (described in Chapter 6) between measured levels. Flow orientation is resolved to be the mean of the measured directions in the vertical profile.

(ii) it has been shown that tidal amplitude and flow velocity are linearly related, and MEANS applies this relationship to remove the effects of tidal asymmetry between the ebb and flood half cycles.

(iii) even though a complete 13-hour measurement program was undertaken at harbour sites, the first records were often made midway through a half cycle, and the other half was recorded at the end of the day. In order to ascertain residuals with these measurements, after the previous correction for tidal asymmetry, velocities were alloted a time relative to the previous slack water for sorting into an ascending time sequence in separate ebb and flood groups.

(iv) the parabolic interpolation procedure was applied to find both the speed and direction at 1-second intervals throughout the tidal cycle.

(v) from these velocities, residuals were found according to the equations presented above, with user-selected sediment threshold speed and power of q.

5.8.3 Residuals in Whangarei Harbour

As velocities were not always measured near slack water and because, for this study, the interest lies primarily with the transport of sediment, residuals were obtained only for those periods when the speeds exceeded a sediment threshold. Using the sediment threshold results discussed in detail in Chapter 7, approximate general sediment thresholds of $u_{1cr} = 0.33$ m.s^{-1} at 1m above the bed and $\bar{u}_{cr} = 0.40$ m.s^{-1} for the mean vertically-averaged velocity were selected for application to the profile data.

Figures 5.41 and 5.42 show the residual mean and 1m velocities at all sites using q = 1 and the above thresholds. The mean velocity residuals and those at 1m above the bed exhibit essentially the same characteristics. The residuals at 1m above the bottom for Spring and Neap tidal range and for both q = 1 and q = 3 are listed in Table 5.13 and 5.14. The residual distances obtained from the 1m speeds and the times above threshold with q = 1 are shown in Figure 5.43. Site identification and location are shown in Figure 5.25.

5.8.4 The relationship of the residual to the sediment transport rates

A sediment transport potential ϕ_{pot}, was defined to represent the

potential flux of sediment past each measurement site using the residual velocities. These are not exact sediment volumes, which depend on grain size, density, and sediment availability, but instead they allow comparison of relative potential fluxes past each of the sites for both Spring and Neap tide conditions.

From the available data, there are at least four possible ways to define the transport potential. To demonstrate the importance of this, with regard to interpretation of the residual velocity, an example is cited. Site B linear residuals with q= 1 in Table 5.13 are employed for cases (i) and (ii) below and the Site B q= 3 data in Table 5.14, for the other two cases. Ebb and flood velocities are almost rectilinear, so the signs are taken only as positive for the flood and negative for the ebb in the example.

(i) Velocity to the first power. Defining ϕ_{pot} as the product of the residual velocity and the time above threshold then,

$$\text{Neap } \phi_{pot} = -0.36 \times 61 = -21.96$$
$$\text{Spring } \phi_{pot} = -0.43 \times 199 + 0.38 \times 215 = -3.87$$

and evidently more nett sand movement occurs during the Neap than Spring tides. This is an invalid solution arising because of the threshold imposed when the residuals were determined. This corresponds with the residual distance.

(ii) Excess velocity to the first power. Defining ϕ_{pot} as the product of the excess velocity above threshold times the period of threshold exceedence then,

$$\text{Neap } \phi_{pot} = -(0.36 - 0.33) \times 61 = -1.83$$
$$\text{Spring } \phi_{pot} = -(0.43 - 0.33) \times 199 + (0.38 - 0.33) \times 215 = -9.15$$

In this formulation, the Spring tide potential is 5 times the Neap result.

(iii) Velocity cubed. Defining ϕ_{pot} as the product of the cube of the residual velocity times the period of threshold exceedence, then,

$$\text{Neap } \phi_{pot} = -(0.36)^3 \times 61 = -2.85$$
$$\text{Spring } \phi_{pot} = -(0.44)^3 \times 199 + (0.38)^3 \times 215 = -5.14$$

The Spring rate is 1.8 times the Neap rate.

(iv) Excess velocity cubed. Defining ϕ_{pot} as the product of the cube of excess velocity above threshold times the period of threshold exceedence, then,

$$\text{Neap } \phi_{pot} = -(0.36 - 0.33)^3 \times 61 = 0.002$$
$$\text{Spring } \phi_{pot} = -(0.44 - 0.33)^3 \times 199 + (0.38 - 0.33)^3 \times 215 = 0.238$$

The Spring rate is 119 times the Neap rate.

There is considerable variation and therefore care has to be exercised with the interpretation of the residual velocities with regard to sediment transport. The difficulty arises mainly because the transport rates are non-linearly related to water speed and because the bed is dormant for speeds above zero. It is shown in Chapter 7, using the formulation of Gadd et. al.

(1978), that a statistically significant correlation between the excess velocity cubed and the sediment transport rate was evident at Whangarei. Thus, the linear residual velocities do not accurately reflect relative sediment transport rates and therefore, for determination of the sediment transport potential, the Type (iv) method was employed using the q = 3 data with $u_{lcr} = 0.33\ m.s^{-1}$. The definition of the sediment transport potential during the ebb is then,

$$\phi_{ebb} = (u_{lebb} - u_{lcr})^3\ T_{ebb} \qquad 5.9$$

and for the flood,

$$\phi_{flood} = (u_{lflood} - u_{lcr})^3\ T_{flood} \qquad 5.10$$

where speeds are taken as positive. u_{lflood} and u_{lebb} are the residual velocities with q = 3 (Table 5.14). T_{ebb} and T_{flood} are respective ebb and flood times that speeds exceed the threshold. If the potentials are rectilinear then,

$$\phi_{pot} = -\phi_{ebb} + \phi_{flood} \qquad 5.11$$

When resolved into x-y components, the transport potential is defined as,

$$\phi_{pot} = (\phi_x^2 + \phi_y^2)^{\frac{1}{2}} \qquad 5.12$$

where,

$$\phi_x = \phi_{ebb} \cos \phi_{ebb} + \phi_{flood} \cos\ \theta_{flood}$$

$$\phi_y = \phi_{ebb} \sin \theta_{ebb} + \phi_{flood} \sin\ \theta_{flood}$$

and θ_{ebb} and θ_{flood} are respective ebb and flood residual velocity directions. The direction of the tidal cycle sediment transport potential is then defined as,

$$\theta_{pot} = (\text{atan}\ (\phi_y/\phi_x) \qquad 5.13$$

The transport potentials are included in Table 5.14. In less extreme cases than Site B, Spring tides are shown to transport about 10 times more sand than Neaps. It is stressed that although this represents potential sediment transport better than the linear residuals, it still takes no account of sediment availabilitites.

5.8.5 Description and implications of residual velocities in Whangarei Harbour

Examining Figures 5.41-5.43, beyond the harbour entrance, Mair Bank has already been discussed. The main channel off Home Point at Site 7, shows strong ebb dominance as would be expected due to flow convergence on the outside of the curved channel. A slight flood dominance is evident upstream on the northern side of the main channel between Mair and Calliope Banks. Higher ebb speeds were recorded but flood velocities exceeded the threshold for a longer period (Table 5.13, Site 6) and a flood-dominated residual

resulted. Conversely, the sediment transport potential at this site has ebb orientation because this parameter is dominated by the water speed rather than the time above threshold. Therefore, a potential for ebb-directed sediment transport exists at this site even though the linear residual has flood orientation.

In the entrance throat, ebb flow dominates the southern shore. Furthermore, as the resultant distance is also large the ebb dominance is maintained for a long time period above the sediment threshold. On the northern side of the entrance throat, the flood is dominant (Site B4) but because the ebb and flood currents are obliquely aligned, the vector residual is directed towards the centre of the channel. Opposingly-dominant currents on opposite sides of the channel just outside the entrance throat, were inferred after examination of the channel cross-section bathymetry in Chapter 3. Furthermore, drogue tracking supported this conclusion. Ebb dominance out of the southern side of the entrance throat reinforces the Mair Bank circulation derived earlier where ebb currents were shown to sweep sand around its northern flank along the main channel margin.

Along the southern shore, passing through the proposed port site, ebb dominance gradually diminishes until near neutrality is attained at Site B. Diver observations of the bed and side-scan sonar (Figures 2.7 and 2.8) revealed total shell lag near the NHB wharf which is gradually replaced by megaripple bedforms up-harbour near Site E. The deposition therefore occurs because the zone has tidal current neutrality, showing increasing sand concentrations upstream as the neutrality becomes more pronounced.

At the port site in the main channel, ebb dominance is evident except at Site A2 which is nearly neutral and has a flood directed sand transport potential. Along the line of the contours, up-harbour from A2 a small patch of megaripples surrounded by shell lag was identified using side-scan sonar (Figure 2.7, Fixes 103-104). No explanation for their occurrence within a scoured shell zone was offered. It is now evident from this data, that they may have formed in a neutral zone between two ebb-directed streams. Strong ebb convergence past the NHB wharf and a tendency for the ebb to pass across the ebb-tidal spit to the north were identified using drogues. Flood paths were more typically up the centre of the channel (Black, et. al. 1982, Figures 8 and 9).

On the sub-tidal spit extending off Snake Bank at the northern edge of the dredge zone, small residuals reflect near neutrality. Furthermore, Site F, which lies slightly to the south of the main bed of megaripples, is ebb dominant and Site K, which lies north of centre, is strongly flood dominant. Notably, some of the fastest currents at 1m above the bed were recorded on this ridge, and currents have the potential to move large fluxes of sediment. However, deposition occurs and must result because the nett currents are nearly

neutral. The ridge separates ebb and flood dominant zones. This is a major result with several implications for the proposed port.

Sediment deposition may occur for two main reasons: (i) The zone is an active sediment pathway i.e. sand is moving between two locations with a marked uni-directional component. An upstream sediment supply of sufficient magnitude is required to maintain the flow. Marsden Point is an example. (ii) The zone is neutral and deposition only occurs because sand entering the neutral zone oscillates back and fourth but nett transport is very small.

These two types of sandy region, have distinctly different impacts on their environments and thus it is important to make the distinction. Interference with an active pathway could have a significant influence on downstream erodible areas. On the other hand, a neutral zone has a much more limited influence as nett transport is grossly restricted. The residuals show that the sandy ridge at the north of the dredge zone is neutral and should thus be expected to have a limited influence on its environment.

There is some contrary evidence in this particular case. The feature is morphologically an ebb-tidal spit extending from Snake Bank. Accordingly, the nett sediment movement across some of its width should be in an ebb (out-harbour) direction, moving along the pathway which extends from Snake Bank past the front of the NZRC wharf and onto Marsden Point as delineated by the side-scan sonar and underwater video (Figure 2.7). With respect to this, the side-scan sonar differentiated a distinct division on the ridge with megaripples on the southern flank and sand waves to the north. This corresponds with the Hayes and Kana (1976) model for a flood-tidal delta which is acknowledged to be megarippled on the ebb spits whereas sediment movement up the flood ramp is accomplished by sand waves moving into the harbour. Sand waves are also known to occur in strongly asymmetrical flow (Table 2.1) and Site K on the flood side is very strongly flood dominated. These then suggest that, although the ridge has overall neutrality, there is interaction between the ebb and flood sides, and there is likely to be nett ebb sediment movement on the southern flank accomplished by a slow, out-harbour progression of the megaripple bedforms.

The ebb flux is fed, at least in part, by flow up the flood ramp. As this initiates on the northern side of the entrance and as the ebb spit terminates on the southern side, interference with the ridge by dredging could cause downstream erosion near the 15m contour in front of the NZRC wharf. Determination of the magnitude of this impact warranted considerably more study of sediment transport on the ridge. Accordingly, a decision on the effect of the dredging is postponed until after this is examined in Chapter 7. Furthermore, the 2-dimensional hydrodynamic and sediment transport models are later employed to predict behaviour of the region by simulating port construction and dredging, and final recommendations are then made in Chapter 11.

The residuals in Shoal Bay at Cross-section D, reveal both an interesting and important phenomenon. The channel is seen to be flood dominated on the southern side, passing through neutral near the 5m contour to the north and becoming strongly ebb-dominated over McDonald Bank. The flood dominance to the south has been previously shown to occur with Ott Meter measurements, and megaripples and sand waves are found there. As transport to this region was shown to initiate on the flood ramp to the north of the entrance throat, a change of bathymetry on the flood ramp should manifest in scour along the north-eastern flank of Snake Bank. The sediment transport potential at D4 is fairly small (1.54 c.f. a maximum of 24.02 at Site K on the flood ramp) and thus, the changes should occur slowly. Sediment passing up the flood ramp has already been shown to feed the ebb-tidal spit to its south and there is a second ebb spit (Figure 2.6) to its north and oriented towards Darch Point. Thus, the ramp feeds at least three major regions and all of these would be influenced by changes to bathymetry.

The out-harbour vectors on McDonald Bank show that the flood-tidal delta is in fact ebb dominated. The ebb residuals on the Bank are very asymmetric but the magnitude of the speeds are small and the sediment transport potentials at D1 and D2 are extremely low and only exceed threshold during Spring tides. Fine, stable sand, impregnated with marine animal burrows, was sighted using underwater video in nearby regions (Figure 2.4) and this is in agreement with the low sediment transport potentials. Drogue speeds nearby were also small (Figure 5.2).

At Site 4, south of Reserve Point, nett flow is ebb-directed. The sediment transport potential is very low, and the magnitudes of the water speeds were only just above threshold. Sea bed photographs revealed a very stable bed (Black, et. al., 1981, Photo Site 3).

The results suggest several general trends:

(i) deposition in the harbour occurs primarily in neutral zones. This implies a paucity of sand in Whangarei Harbour and shell beds are therefore primarily responsible for stability and not an abundant, loose, sandy, sediment supply.

(ii) some deposition occurs in the flood-dominated zones in the shipping channel near One Tree Point and on the north of Snake Bank. However, in the strongly asymmetrical flood currents exhibiting a high sediment transport potential, the supply is evidently not sufficient to maintain the imbalance and scoured shell lag occurs over much of the Snake Bank flood ramp (Figure 2.7).

(iii) the only noted deposition in an ebb-dominant zone, other than on the bank crests where speeds are mostly below threshold, was along the ebb shield to the inter-tidal sand flat near One Tree Point. This must, at least in part, be maintained by sediment forced off the inter-tidal zone by flood

currents, and this behaviour has been identified in other tidal inlets (Boothroyd, 1978).

(iv) the scoured channel floors are shown to result from high measured water speeds and a shortage of sediment in supply. This, then, is the explanation for the high level of bathymetric stability in Whangarei Harbour. Regular inputs of large quantities of sediment through the entrance throat do not occur (Chapter 4), and tidal currents which are "constant" in the long-term have established a bathymetric stability in the inlet which is not easily disturbed by influences from outside the throat.

5.9 Symmetry Index

Allen's (1980) Velocity Symmetry Index is defined as,

$$V_2 = \frac{U_{fl,\,max} - U_{ebb,\,max}}{U_{fl,\,max} + U_{ebb,\,max}} \qquad 5.14$$

where $U_{fl,max}$ and $U_{ebb,max}$ are the magnitudes of the maximum ebb and flood velocities over a tidal cycle. V_2 measures the tidal velocity asymmetry and is positive for flood and negative for ebb dominance. This parameter was derived for all sites using the 1m speeds and is included in Table 5.12.

V_2 is clearly a simple function to produce, but the question arises as to whether it adequately represents any physical process. Unlike V_2, the residual accounts for both velocity and time asymmetry. For this reason, V_2 was compared with the linear residuals in Figure 5.44. There is some scatter and the best fit linear curve has the form,

$$V_r = 1.13\ V_2 + 0.02 \qquad 5.15$$

with correlation coefficient of 0.85. This is significant at the $\alpha = 0.05$ level (Freund, 1974) and the linear relationship can be accepted. Important deviations occur in the upper right and lower left rectangles (2nd and 4th quadrants) in Figure 5.44, where the residual and V_2 predict oppositely-directed dominance. The linearity of this relationship would decrease with increased variations in the shape of the ebb and flood velocity-time curves. The use of V_2 should therefore be tempered with some prior knowledge of the tidal velocity curve. However, for the Whangarei data, the significance of the correlation coefficient shows that the parameter does represent tidal asymmetry.

5.10 Conclusions

(i) a detailed drogue tracking program was undertaken. Mostly, the results are summarised in point form in Sections 5.3.4.1 and 5.3.4.2.

(ii) full tidal cycle drogue experiments showed that flow passing out of the main channel near Busby Head does not return on the next flood tide.

(iii) flow from the marginal flood channel separating Mair Bank and Marsden Point, "stagnates" over central Mair Bank and water is returned to the harbour during the next flood tide. It is flow from the extreme, southern margin at the entrance throat which takes this course.

(iv) Bream Bay ocean and tidal current circulations are characterised by three major features (Section 5.3.4.3): (a) a resultant flood flow which heads west along the land from Bream to Busby Heads; (b) ebb currents which jet south to south-east and are supported by the North Auckland Current; and (c) early flood flow passing across the ebb-tidal delta. In the most part, the circulation supports the large sediment loop proposed in Chapter 4 which circumnavigates the Mair Bank ebb-tidal delta.

(v) lower harbour flushing times are evidently very short, being only a small number of tidal cycles.

(vi) both the Mair Bank ebb-tidal delta and the Snake Bank flood-tidal delta were found to conform with behaviour expected of standard tidal deltas proposed by Hayes and Kana (1976). In conjunction with this, an examination of large-scale bedforms noted in aerial photographs and shallow water velocity measurements, the sediment transport directions around the Banks were identified. Sediment is moving northwards along Bream Bay Beach near Marsden Point under the influence of both tidal and wave induced currents. On Marsden Point, the residual movement is directed to the east along the main channel where it is swept either over or around upper Mair Bank. The seaward flanks of the upper Bank are flood dominated.

(vii) In conjunction with realignment of the upper western levels of Mair Bank (Figure 3.2), the circulation cited in (vi) is responsible for the transport of sediment deposited on the northern flanks of the Bank along the main channel.

(xiii) sediment transport fluxes along the beach near Marsden Point under tidal flow alone are shown to exceed the annual rate of accretion on the upper Bank between the 1959 and 1981 RNZN bathymetric surveys. Therefore, much sediment passes through this region and Marsden Point is an active sediment pathway.

(ix) shell lag on the crest of Mair Bank is found to be stable in currents of 0.9 $m.s^{-1}$ at 1m above the bed, which is far in excess of the sandy, sediment threshold. This explains the noted stability of the eastern end of upper Mair Bank, when the flood-dominated, sandy, western end has moved considerably.

(x) ebb-dominated zones on Marsden Point and on upper Mair Bank are typically shell-lagged, thus they are not part of an active sediment pathway, and the shell evidently prevents erosion.

(xi) sand entering the harbour on the northern side of the entrance throat up the Snake Bank flood ramp is found to be feeding three major regions, i.e. the two ebb-tidal spits bounding the flood ramp, and the flood-dominated

active region on the north-east side of Snake Bank in Shoal Bay. Interference with the flood ramp bathymetry by dredging, would influence all of these regions.

(xii) the ebb spit extending south-east from Snake Bank and passing across the northern side of the dredge zone is found to have overall neutrality and separates flood and ebb-dominated zones to its north and south respectively. Nett sediment transport on its southern flank is directed out of the harbour and accomplished by the slow advance of megaripple bedforms.

(xiii) the ebb-shield on Snake Bank (formed of disarticulated cockle shell) separates ebb and flood dominated zones, each of which are characterised by sand waves advancing in the corresponding directions.

(xiv) the blind flood-channel and, by implication, the adjacent regions on the inter-tidal zone fronting Point Sinclair, are flood dominated.

(xv) the sea bed in shallow regions around Snake Bank were found to be often inactive during Neap tides, when tidal currents are below the sediment threshold.

(xvi) full-tidal cycle velocity profile measurements were conducted at sites throughout the harbour. Average speeds at 1m above the bed for all measurements were 0.42 and 0.38 $m.s^{-1}$ for the ebb and flood respectively, while the maxima were 0.99 and 0.90 $m.s^{-1}$.

(xvii) most sites are found to reach maximum speed closer to high than to low tide during both the ebb and flood half cycles. Exceptions occurred along the southern shore of the throat due to early flooding along the throat channel margin while the ebb continued to jet out of the entrance in deeper water offshore.

(xviii) continued flood or ebb flow after the time of high or low tide was recorded at many sites. Inertial flow and basin morphology were nominated as the causes.

(xix) due to solar radiation, the tidal inlet was warmer in summer and colder in winter than the waters of Bream Bay.

(xx) the lower harbour is mostly well-mixed.

(xxi) maximum water speeds during half tidal cycles increase by as much as 100% from Neap to Spring tides.

(xxii) it is shown that maximum mean water speed and the tidal range are linearly related. This was applied to remove the effects of tidal asymmetry, at each velocity measurement site and allows direct comparison of water speeds at all sites throughout the harbour.

(xxiii) fastest speeds during ebb tidal flow were found to occur in the main shipping channel. Fastest flood currents favour the north side of the inlet throat, the north and middle of the shipping channel and the marginal flood channel off Marsden Point. Flow through the main shipping channel is notably faster than through Shoal Bay.

(xxiv) residual velocities and a defined sediment transport potential were derived for each Braystoke site. The residuals are summarised in Figures 5.41 to 5.43. The major aspects noted are: (a) ebb dominance on the southern side of the entrance throat and flood dominance to the north; (b) overall neutrality on the sub-tidal spit extending off Snake Bank across the northern side of the dredge zone; (c) along the southern shore near the NHB wharf ebb dominance passing through neutral near Blacksmith's Creek and becoming flood dominant towards Point Sinclair; (d) ebb dominance on both margins of the main channel near One Tree Point and flood dominance near mid-channel; (e) flood dominance on the south-west side of the Shoal Bay Channel changing to ebb dominance on the opposite, McDonald Bank side.

(xxv) The reader is referred to the summary of general results at the end of Section 5.8.5.

(xxvi) Allen's (1980) Symmetry Index is shown to be linearly related to the residual velocity which measures tidal asymmetry.

SEDIMENT TRANSPORT AND TIDAL INLET HYDRAULICS

Chapter 6:

BED FRICTION

CHAPTER 6: BED FRICTION

6.1 Introduction

More than 600 vertical profiles were measured and, in this Chapter, these are examined to determine the frictional energy losses at the bed. In addition, a series of experiments over megaripples were conducted to ascertain the sediment transport rates under tidal flow. This Chapter examines the frictional energy losses over megaripples, with this data, by isolating the skin (or effective) friction and the form drag components of the friction velocity. This analysis was performed, in part, for application to sediment transport and develops the framework of the procedure to determine sediment transport rates over megaripples presented in Chapter 7.

6.2 List of symbols

c friction factor ($\bar{u}/u_*$)
C Chezy's C ($m^{\frac{1}{2}}.s^{-1}$)
C_D drag coefficient
C_1 Drag Coefficient with speed measured 1m above the bed
C_{DS} Drag Statistic
d water depth
D_{35} grain size for which 35% of distribution is finer by weight (m)
D_{50} grain size for which 50% of distribution is finer by weight (m)
D_{65} grain size for which 65% of distribution is finer by weight (m)
D_{90} grain size for which 90% of distribution is finer by weight (m)
f Drach-Weisbach friction coefficient
g gravitational acceleration (9.81 $m.s^{-2}$)
k Nikuradse's equivalent sand roughness
n number of velocity measurements in the vertical profile
r friction parameter, correlation coefficient representing the goodness of fit of the measured speeds to the logarithmic profile
R hydraulic radius (m)
Re Reynold's Number
s_i standard deviation of sample distribution
S sea surface slope
t time (s)
u velocity measured z metres above the seabed ($m.s^{-1}$)
u_1 velocity measured at 1m above the seabed ($m.s^{-1}$)
$\bar{u}$ mean vertically-averaged velocity ($m.s^{-1}$)
u_{z_i} velocity measured z_i metres above the sea bed ($m.s^{-1}$)
$\dot{u}$ acceleration ($m.s^{-2}$)
u_* (total) friction velocity ($m.s^{-1}$)
u_*' skin friction velocity ($m.s^{-1}$)
u_*'' form drag friction velocity ($m.s^{-1}$)
$\tilde{u}_*$ friction velocity after correction for flow acceleration ($m.s^{-1}$)
U_* free stream velocity at the outer edge of the boundary layer ($m.s^{-1}$)
$\bar{x}_i$ mean of sample distribution
z distance above the sea bed (m), z-statistic
z_o roughness length (m)
$\tilde{z}_o$ roughness length after correction for flow acceleration (m)
α statistical significance level
δ boundary layer thickness (m)
Δ bedform height (m)
ε_o vertical eddy viscosity ($m^2.s^{-1}$)
γ constant for flow acceleration correction (0.04)
κ von Karman's Universal constant
Λ bedform wavelength (m), constant for acceleration correction (m)
ν fluid viscosity ($m^2.s^{-1}$)

ρ fluid density (1025 $kg.m^{-3}$)
ρ_s sediment density (2650 $kg.m^{-3}$)
τ_o bed shear stress ($N.m^{-2}$)
τ_{DS} bed shear stress obtained without curve fitting ($N.m^{-2}$)
θ Shields Entrainment Function
(') association with skin friction
(") association with form drag

6.3 Fully-rough turbulent velocity profiles

In turbulent flow along a wall, the turbulence is directly affected by the wall. This occurs through the action of forces resulting from the flow around the roughness elements (Hinze, 1959). The work done by the flow on the bed and the conversion of flow kinetic energy to turbulent eddies due to the bed friction is reflected in the vertical velocity gradient. The mean velocity changes from zero at the bed to the free stream speed at the outer edge of the boundary layer.

In free turbulence, at sufficiently large Reynolds numbers, the effect of viscosity on the over-all behaviour of the flow can be neglected (Hinze, 1959). Instead, the transfer of momentum in both the vertical and horizontal directions is accomplished by the action of turbulent eddies, a phenomenon known as turbulent diffusion. This process allows salt or sediment as well as momentum to be diffused throughout the water column, and is responsible for the well-mixed nature of Whangarei Harbour. Away from the boundary, the turbulence is characterised by large eddies elongated in the flow direction (limited in the vertical by the water depth), while the scale of the larger eddies is determined by the distance from the boundary (Hinze, 1959). Eddy sizes relate to estuary width, depth of flow, current velocity, bottom roughness and density stratification (Dyer, 1973).

Bowden and Proudman (1949) found that longitudinal turbulent fluctuations in the Mersey Narrows (U.K.) were of two types. Fluctuations of the order of a few seconds period were superimposed on fluctuations with periods of 30 seconds to several minutes. The amplitude of the horizontal fluctuations was about 10% of the mean current. Dyer (1973) interpreted this by associating the short-period turbulence with bottom friction and the longer period oscillations with horizontal eddies or internal waves. Heathershaw and Simpson (1978) found that the vertical and horizontal velocity components have quasi-Normal probability density functions.

Bowden and Fairburn (1956) found in Red Wharf Bay (U.K.) that at heights of 0.5 to 1.75m above the bottom, the spectra of horizontal and vertical velocities were wide with respective peaks at 75 and 7 seconds.

Measurements of the velocity gradient (e.g. Hama, 1954, p. 336) show that the boundary layer is divided into two parts: an inner region up to about 15% of the boundary layer width from the bottom and an outer region extending from there to the outer edge of the boundary layer. In the inner layer most of the

turbulence energy produced is converted directly into heat by turbulence dissipation (Hinze, 1959). However, part of this turbulence energy is transported by turbulent diffusion toward the outer region of the boundary layer.

Throughout the inner layer, if the assumption is made that the shear stress is constant and equal to the bed shear stress then the vertical profile equation may be obtained as follows. The shear stress, τ_o, is proportional to the velocity gradient,

$$\tau_o \propto \frac{du}{dz} \qquad 6.1$$

where z is distance above the sea bed defined positively upward and u is the corresponding flow velocity. Introducing a vertical eddy viscosity, ε_o as a proportionality constant and fluid density, ρ, then,

$$\tau_o = \rho \varepsilon_o \frac{du}{dz} \qquad 6.2$$

Defining the friction velocity, u_*, as,

$$u_* = (\tau_o/\rho)^{\frac{1}{2}} \qquad 6.3$$

then,

$$u_*^2 = \varepsilon_o \frac{du}{dz} \qquad 6.4$$

A second assumption is made here, that in the layer of constant stress, the eddy viscosity varies linearly with the distance from the bed i.e. the size of the large eddies, which are responsible for the eddy viscosity, varies in proportion to the distance from the boundary.

A linear eddy viscosity was measured by Klebanoff and reported in Hinze (1959, Figure 7.17). This same data was fitted to a polynominal with 4 free coefficients by Smith and McLean (1977) who obtained the normalised eddy viscosity,

$$\frac{\varepsilon_o}{\kappa u_* \delta} = \left(\frac{z}{\delta}\right) - 1.3 \left(\frac{z}{\delta}\right)^2 + 16.9 \left(\frac{z}{\delta}\right)^3 + 25.2 \left(\frac{z}{\delta}\right)^4 \qquad 6.5$$

in the range $\frac{z}{\delta} < 0.3$ and δ is the boundary layer thickness. For small $\frac{z}{\delta}$ the coefficient is linear. Therefore, to the first order,

$$\varepsilon_o = \kappa u_* z \qquad 6.6$$

where κ is von Karman's constant. With (6.4) and (6.6),

$$u_*^2 = \kappa u_* z \frac{du}{dz} \qquad 6.7$$

Integrating this with respect to z (noting that stress, and thus u_*, is constant and shows no dependence on z) gives the logarithmic distribution,

$$\frac{u}{u_*} = \frac{1}{\kappa} \ln z + B \qquad 6.8$$

where B is a constant of integration. If the velocity at the outer edge of the boundary layer of thickness $z = \delta$, is $u = U_\infty$ then,

$$B = \frac{U_\infty}{u_*} - \frac{1}{\kappa} \ln \delta \qquad 6.9$$

and

$$\frac{U_\infty - u}{u_*} = - \frac{1}{\kappa} \ln \left(\frac{z}{\delta}\right) \qquad 6.10$$

which is known as a velocity defect law.

Alternatively, the Karman Prandtl (KP) equation is obtained by introducing the level $z=z_o$, where the speed $u=0$ and,

$$B = \frac{1}{\kappa} \ln \left(\frac{1}{z_o}\right) \qquad 6.11$$

Then,

$$\frac{u}{u_*} = \frac{1}{\kappa} \ln \left(\frac{z}{z_o}\right) \qquad 6.12$$

z_o is a notional zero velocity height called the roughness length.

Both of these equations describe the inner boundary layer. In the outer layer, the velocity gradient is known to increase (Figure 6.1). Hama (1953) presented an empirical velocity defect law to describe the outer layer,

$$\frac{U_\infty - u}{u_*} = 9.6 \left(1 - \frac{z}{\delta}\right)^2 \qquad 6.13$$

Alternatively, the constant in the logarithmic law may be adjusted to approximate the change of profile gradient producing a relationship, such as the one presented in Dailey and Harleman (1966, p. 236),

$$\frac{U_\infty - u}{u_*} = - 8.6 \log_{10} \left(\frac{z}{\delta}\right) \qquad 6.14$$

This will be described as the Universal Velocity Defect Law (UVDL) throughout the thesis. Figure 6.2 shows the UVDL and Hama relationships plotted together and it is clear that over the range $0.15<z/\delta<1.0$, they are similar.

It is common practice for ocean and estuarine boundary layer flow to be described by the Karman Prandtl equation near the sea bed. Sternberg (1968) found over beds roughened by ripples or gravel that the logarithmic profile in the inner layer was evident 62% to 100% of the time, and averaging 82%. However, the UVDL is not so commonly used. The outer layer is further away from the bed, implying a potential reduction in correlation of the profile gradient to the bed shear stress. As stated by Hinze (1959, p. 504), the turbulence in the outer region "obtains its energy mainly by diffusion from the inner region. A consequence of this is that the turbulence is determined by the conditions in the inner region far upstream i.e. by the distribution of the wall shear stress far upstream". Hinze states metaphorically that the outer region has a long "memory" in sharp contrast to the quick recovery from disturbances which occurs in the inner region. The conclusion to be made here, is that the outer region will reflect the bed shear stress best if the upstream

bed is similar to the position where measurements are taken.

If fluid flows off a bank into a deep channel, for example, the outer boundary layer will not necessarily relate to the bed shear stress at a measurement site in the channel, due to sheltering by the bank of the lower levels. Furthermore, the outer layer reflects average bed conditions whereas the inner layer indicates more immediate roughness elements. This concurs with discussion in Smith and McLean (1977) who state that at great distances from the boundary, the velocity field is related to the overall boundary shear stress, that is, to the boundary stress averaged over a large region of the bed.

At Whangarei, speeds measured in the outer and inner boundary layers were fitted to the UVDL and KP respectively.

6.4 Non-simultaneity correction

The velocities were measured with a Braystoke meter described in the previous chapter. The meter was winched between levels and on most occasions, speeds were measured consecutively for 100s at each level, so that nearly 15 minutes were required to record a complete profile. Accordingly, the data had to be corrected for non-simultaneity in the vertical. This was accomplished in the author's FORTRAN program called ACCLN, which fits a parabola to three consecutive measurements at each level. The parabola is fitted to the measurement of interest and the one recorded earlier and later at the same level. The time between measurements was normally half an hour.

The general parabolic equation is,

$$u = At^2 + Bt + C \qquad 6.15$$

where speed u occurs at time t. The three unknown constants A,B,C are found from the three speed measurements u_1, u_2, u_3 recorded at times t_1, t_2, t_3 respectively as,

$$B = \frac{1}{t_3 - t_1}\left[\frac{(u_1 - u_2)(t_2 + t_3)}{(t_1 - t_2)} - \frac{(u_2 - u_3)(t_1 + t_2)}{t_2 - t_3}\right]$$

$$A = \frac{1}{t_1 + t_2}\left[\frac{(u_1 - u_2}{t_1 - t_2} - B\right] \qquad 6.16$$

$$C = u_1 - At_1^2 - Bt_1$$

This particular solution is not unique and there are other permutations which would supply a solution to the general equation. By interpolation of the parabole, fitted to each level, instantaneous velocity profiles relative to the time of the first measurement near the bed, were determined. All Braystoke velocity profiles were standardised in this fashion.

The importance of this correction is shown in Figure 6.3. At Site S5, the friction velocity was determined and compared with the velocity at 1m. The

correlation of the two parameters was markedly improved after application of the non-simultaneity correction to the profile. The correlation coefficient rose from 0.37 without the correction, to 0.67 after the correction.

6.5 Von Karman's Universal constant

The value of von Karman's Universal constant is approximately $\kappa = 0.4$. This was reinforced recently by Soulsby and Dyer (1981) who found von Karman's constant equal to 0.40, being closer to the laboratory values than the "now widely used atmospheric value of 0.35", (Soulsby and Dyer, 1981, p. 8067). They did, however, find evidence supporting a slow decrease in von Karman's constant with increasing boundary-layer thickness to roughness length ratio.

Einstein and Chien (1955) showed that κ was reduced by high suspended sediment concentrations (see Raudkivi, 1976, Figure 7.37). Under suspended sediment concentrations of 1 g/l Vanoni (1975) also showed that κ decreases. Measured suspended loads at Whangarei (Chapter 7) were much less than the Vanoni concentrations and thus $\kappa = 0.04$ was adhered to throughout the thesis.

6.6 Criterion for fully rough flow

The equations derived to describe the boundary layer profile assume that the flow is fully rough. That is, the roughness elements project through the viscous, laminar sublayer into the main body of turbulent flow. In this state, the viscosity of the fluid becomes unimportant and the Reynold's Number, which measures the ratio of inertial to viscous forces, becomes large.

In estuarine conditions, Sternberg (1968) found that the occurrence of rough flow is related to the geometry of the bed, with the more complex beds becoming fully rough at higher Reynolds numbers than the more simple roughness patterns. Over rippled and gravel beds, the flow became fully rough when the Reynold's Number exceeded 1.5×10^5. As,

$$Re = u_1 . z_1 / \nu \qquad 6.16a$$

and with a fluid viscosity of sea water at 15^oC of $\nu = 1.2 \times 10^{-6}$ $m^2.s^{-1}$ and $z_1 = 1m$, then for fully rough flow, $u_1 = 0.18$ $m.s^{-1}$.

To ensure that the flow was fully rough, statistics compiled in this Chapter, unless stated otherwise, correspond with profiles having u_1 greater than 0.3 $m.s^{-1}$. This has a margin for error above the Sternberg value, and is also a natural cut-off being approximately the entrainment threshold speed for sandy sediment (Chapter 7). As the main interest of this thesis is to study sediment transport rates, transitional profiles were not examined. Under fully rough conditions, the boundary layer thickness is assumed to be equal to the water depth.

6.7 Determination of the bed shear stress

The major computer program which ascertains the bed shear stress, friction velocity, drag coefficient etc. is the author's velocity profile analysis program, called CUR4. The KP and UVDL can be written as,

$$u = 5.75\, u_* \log_{10} z - 5.75\, u_* \log_{10} z_o \qquad \text{KP} \qquad 6.17$$

$$u = 8.6\ u_* \log_{10} z + U_\infty - 8.6\, u_* \log_{10}\delta \qquad \text{UVDL} \qquad 6.18$$

A semi-log plot of velocity versus the log of the distance from the sea bed, should be linear with gradients,

$$g_{KP} = 5.75u_* \qquad \text{KP} \qquad 6.19$$

$$g_{UVDL} = 8.6u_* \qquad \text{UVDL} \qquad 6.20$$

and intercepts,

$$I_{KP} = -\ 5.75u_* \log_{10} z_o \qquad \text{KP} \qquad 6.21$$

$$I_{UVDL} = U_\infty - 8.6u_* \log_{10}\delta \qquad \text{UVDL} \qquad 6.22$$

The program finds u_*, z_o and U_∞ by fitting the measured data by linear regression to the above equations. For each case, a linear correlation coefficient is derived to describe the goodness of fit of the velocities to the logarithmic profile.

The roughness length is defined as the height above the sea bed where the velocity is zero. If this does not correspond with bed level, then the vertical axis should be shifted accordingly and the KP equation then has the form,

$$\frac{u}{u_*} = 5.75 \log_{10} \left(\frac{z + z_o}{z_o}\right) \qquad 6.23$$

In CUR4, z_o is found using an iterative procedure requiring successive approximations until the difference between the new and old z_o value is less than some pre-defined number, say 10^{-5}. As z_o at Whangarei is typically about 0.0015m (thus, $z \gg z_o$), the correction made by redefining the origin was not important, relative to the errors in the meter distance above the bed obtained from the sounder. Although the computer program will find z_o by both methods, the simpler form of the KP equation was used throughout as was done by Dyer (1980), Sternberg, (1972), for example.

Ludwick (1974) fitted velocities measured in the outer layer to the Hama equation. To assess the applicability of this to Whangarei, outer boundary layer data from several locations were fitted to both the Hama and UVDL equations. The comparison of the correlation coefficients revealed a small (and certainly not significant) improvement of the fit with the UVDL. A small difference only could be expected after examination of the similarity of the two curves shown on Figure 6.2. As it is more easily manipulated algebraically, the UVDL was then selected to model the outer boundary layer.

The velocity gradients in the inner and outer boundary layers from equations (6.19) and (6.20) are theoretically $5.75u_*$ and $8.6u_*$ respectively. Under the constant shear stress assumption, this corresponds with a gradient ratio of 1:1.5. A verification of this for estuarine flow, was undertaken by comparing the velocity gradients in the inner and outer boundary layers.

Of all the data, the most complete and detailed measurements were taken over megaripples during the S-series. These measurements spanned the inner and outer boundary layers with at least 3 speeds recorded in each. Using data from S1, S2, S3 and S5 (S4 was too shallow), the velocity gradient in both regions was obtained by fitting speeds measured within 20% of the depth from the bed to the Karman Prandtl equation and speeds at more than 15% of the depth from the bed to the UVDL. Depending on the tide level, this meant that normally the inner layer included measurements at 0.5, 1.0 and 2.0m and the outer layer included those at 2.0, 4.0 and 6.0m. No fit was undertaken if fewer than 3 velocities were measured in either layer. For a comparison, the entire profile was fitted to both the KP and UVDL.

The results of this analysis are shown in Table 6.1. If the gradients vary in the predicted ratio, the calculated friction velocity from the outer layer should equal that from the inner layer. For the four sites studied, excellent agreement occurs with the overall average friction velocity in both layers being 0.028 $m.s^{-1}$. Alternatively, when the full profiles were fitted to the KP, a considerably higher mean friction velocity of 0.034 $m.s^{-1}$ resulted and after fitting all levels to the UVDL, the friction velocity was much reduced to 0.023 $m.s^{-1}$. The important implications are (i) that the theory is appropriate to estuarine conditions and (ii) that a distinction between the inner and outer layers must be made when curve fitting to obtain the bed shear stress. Incorrect choice of the appropriate velocity profile will result in large errors in u_* and when the bed stress is applied to determination of sediment transport rates, these errors are magnified by the non-linear nature of bedload equations.

Although the average gradients over the tidal cycle were in the correct ratio, the correlation of inner and outer level gradients for each individual profile was found to be fairly poor. Much of this scatter can be attributed to turbulence with periods approximating the measurement period of 100s. The averaging helped to reduce this. The results show that transient effects may be masking a phenomenon which was predicted by measurements made in the more controlled laboratory environment.

As a result of this analysis, all profiles throughout the harbour were fitted to either the KP or UVDL depending on whether measurements were made in the lower 20% or upper 85% of the boundary layer thickness. When a friction velocity was obtained in both layers, it was averaged and bed shear stress, Drag Coefficients etc. were found from this average value. The correlation

coefficient presented was the average of the two correlation coefficients obtained in each layer. This is not an entirely satisfactory estimator of the goodness of fit of the whole profile, however it offers some useful information. Data was only fitted to either layer if at least 3 measurements were made there, and when this was not satisfied a fit was made to the KP or UVDL only, with the latter being more common.

6.8 Effects of tidal accelerations

Some recent work by Soulsby and Dyer (1981) which was published after this study was initiated, was not incorporated into the analysis program, CUR4, but it will be discussed here for its relevance to the Whangarei anlaysis. Soulsby and Dyer derived and field tested a correction to be made to the shape of the velocity profile under accelerating tidal flow conditions. The Karman Prandtl equation becomes,

$$\frac{u}{u*} = \frac{1}{\kappa}\left[\ell n\left(\frac{z}{z_o}\right) - \frac{z - z_o}{\gamma\Lambda}\right] \qquad 6.24$$

where γ and Λ have to be found. The authors suggest that γ can be considered constant at $\gamma \simeq 0.04$. The other function is given by,

$$\Lambda = \frac{z}{\gamma} + \frac{\kappa z u}{2\dot{u}}\frac{\partial u}{\partial z}\left[1 + \left(1 + \frac{8z\dot{u}}{\kappa\gamma u^2}\right)^{\frac{1}{2}}\right] \qquad 6.25$$

where $\dot{u}$ is the flow acceleration and all other symbols have been previously defined. It is evident that when the acceleration is zero, this function approaches infinity and the velocity profile (eqn. 6.24) has the form of the standard Karman Prandtl equation without correction for acceleration.

To establish the magnitude of this correction, a case example for one of the most rapidly accelerating sites was examined.

Time of Day	$u_{0.5}$	$u_{1.0}$	$u_{2.0}$
9:45	-	.45	-
10:00	.33	.55	.71
10:15	-	.67	-

Velocities ($m.s^{-1}$) measured at Site S3 on 9-Feb-82

At 10:00 on 9-Feb-82, the tidal acceleration was $\dot{u} = 0.000122\ m.s^{-2}$ at 1m above the bottom. Insertion of these values into equation (6.25) produces $\Lambda = 299m$.

The friction velocity obtained from the profile without consideration of acceleration, $\tilde{u}_*$, is related to the true friction velocity, u_*, as,

$$\tilde{u}_* = u_*\left(1 - \frac{z_m}{\gamma\Lambda}\right) \qquad 6.26$$

where $z_m = (z_1.z_2)^{\frac{1}{2}}$ and z_2 and z_1 are the upper and lower limits above the bed of the measurements of velocity. With $z_1 = 0.5$, $z_2 = 2.0$ then,

$$u_* = 1.09\ \tilde{u}_* \qquad 6.27$$

Under these strenuous conditions, there is apparently a 9% under-estimate of the friction velocity. For decelerating conditions the friction velocity is over-estimated when acceleration is not accounted for.

Similarly the uncorrected roughness length, $\tilde{z}_o$, is related to the true value z_o as,

$$\tilde{z}_o = z_o \exp\left[\frac{\ell n(z_m/z_o) - 1}{1 - \frac{\gamma\Lambda}{z_m}}\right] \qquad 6.28$$

Taking a typical case with $z_o = 0.005$m then,

$$z_o = 1.48\ \tilde{z}_o \qquad 6.29$$

and there is a factor of 1.48 to be applied.

The deviation in friction velocity is fairly small at 9% but the 48% correction to z_o may be important. The example cited here is for accelerating flow where u_1 reached 0.83 $m.s^{-1}$ in just under 3 hours.

The results of Soulsby and Dyer (1981) are evidently important and some analysis is later undertaken to test if the effect was measurable at Whangarei.

6.9 The Drag Coefficient

The Drag Coefficient arises out of a measured proportionality between the shear stress and the square of the velocity, known as the Quadratic Stress Law. Introducing a proportionality coefficient, then,

$$\tau_o = C_D \rho u^2 \qquad 6.30$$

where C_D is a drag coefficient.

If the speed is measured at a representative distance above the bed, say $u=u_1$, (Sternberg, 1968) then,

$$\tau_o = C_1 \rho u_1^2 \qquad 6.31$$

or in terms of the friction velocity,

$$C_1 = (u_*/u_1)^2 \qquad 6.32$$

6.10 Friction factors

Several friction parameters are commonly used to describe bed friction (Raudkivi, 1976). Those encountered in this chapter are:

(i) the friction factor formed by the ratio of mean flow velocity to friction velocity,

$$c = \bar{u}/u_* \qquad 6.33$$

(ii) The function of the friction factor called the Darcy-Weisbach coefficient,

$$f = 8/c^2$$

(iii) Chezy's C which is a proportionality coefficient between mean speed, $\bar{u}$, and the product of flow surface slope, S, and the hydraulic radius, R, i.e.,

$$\bar{u} = C(R.S)^{\frac{1}{2}} \qquad 6.35$$

and in 2-dimensional flow where,

$$u_* = (gSR)^{\frac{1}{2}} \qquad 6.36$$

then,

$$C = c(g)^{\frac{1}{2}} \qquad 6.37$$

Chezy's C is not dimensionless but has units $m^{\frac{1}{2}}.s^{-1}$. If the boundary flow can be described by the KP equation which has mean speed at 0.37d, then substitution yields,

$$C = 5.75(g)^{\frac{1}{2}}. \log_{10} (0.37d/z_o) \qquad 6.38$$

(iv) The roughness length, z_o, has previously been introduced as the level at which the flow velocity goes to zero. In fully rough flow this is related to Nikuradse's equivalent sand roughness, k, as,

$$z_o = k/30.2 \qquad 6.39$$

although, as stated by Raudkivi, the value of 30.2 varies with the type of surface roughness. However, equation (6.39) will be applied as defined for all bed types thus defining k when z_o is determined from the velocity profile. After substitution into (6.38) and simplification, Chezy's C becomes then,

$$C = 18 \log_{10} (11.05d/k) \qquad 6.40$$

(v) The friction parameter is a non-dimensional form of Chezy's C,

$$r = g.C^{-2} = c^{-2} \qquad 6.41$$

(vi) The Drag Coefficient C_1 has been discussed separately but with the KP, it is related to Nikuradse's equivalent sand roughness as,

$$k = 30.2/(10^{1/\ 5.75\ (C_1)^{\frac{1}{2}}}) \qquad 6.42$$

and to the roughness length as,

$$z_o = 10^{-1/(5.75\ (C_1)^{\frac{1}{2}})} \qquad 6.43$$

6.11 Vertical profile types

The vertical velocity profile is later shown to be logarithmic much of the time, but deviations from this shape occurred and fell into categories similar to those described by Ludwick (1974) for Chesapeake Bay (USA). Some extreme examples of the variations noted at Whangarei are shown in Figure 6.4 and notably for some of these $u_1 < 0.3$ m.s^{-1} and therefore the profile would not be considered as representative of fully rough flow.

Profile D is logarithmic with speed changing most rapidly near the bed. Profile A occurs at the end of the flood tide and mid-column flow has ceased while the upper and lower levels continue to flood. This sort of variation near slack water can be due to drifting eddies which accelerate the flow temporarily throughout the water column but not for the entire vertical profile

measurement period (see also B and F). Profile C exhibits a linear increase in speed above 0.5m. The flow was accelerating at the start of the ebb tide. Profile E is logarithmic in the outer boundary layer (above 2m from the bed) but is disjointed in the inner boundary layer, and either the 1 and 2m speeds or the 0.5 and 1m speeds are exceptional. Fitting a logarithmic profile to the inner and outer boundary layers would produce significantly different friction velocities in each layer. Profile I, measured in fast speeds (>1 m.s^{-1}) in the top two thirds of a water column over 24m deep, is irregular, even though speed shows a general increase with distance from the bottom. The most extreme case of turbulent interference was recorded on the north-west side of Passage Island at Site B4 during the ebb (Profile G). The downstream island was causing the flow to decelerate suddenly and large-scale turbulence resulted.

6.12 Bed friction in Whangarei Harbour

More than 600 profiles, measured over various beds of megaripples, shell, sponge and algae (Table 5.9 and Black et. al. 1981), were fitted to the logarithmic equations. After deleting some which did not have enough measurements in both layers, and those having speeds at 1m of less than 0.3 m.s^{-1}, the friction velocities etc., were obtained from 423 remaining profiles. This data allowed (i) determination of average frictional losses over the area which was later modelled numerically, and (ii) the more specific determination of friction coefficients over particular beds.

For each Braystoke site, the maximum, minimum, mean, median and standard deviation of the 1m speeds, friction velocities, Drag Coefficients, roughness lengths, and correlation coefficients representing the goodness of fit of the measured speeds to the logarithmic profile, are listed in Table 6.2. The median values are included because it is a more stable statistic than the mean and possibly represents the physical processes better for distributions which vary over several orders or magnitude such as that of the Drag Coefficient and roughness length. With a log-normal distribution, the large values dominate the mean on an arithmetic scale (which is a justification for using the phi scale in grain size analysis), and a similar scale could be introduced to describe the Drag Coefficient. However, the median is an alternative, well-behaved statistic. It is the counterpart of the D_{50} grain size and will be denoted as $C_{1\ 50}$. The mean was still retained.

6.12.1 General results

Table 6.3 compiles the statistics for all sites combined. The mean speed at 1m for all sites was 0.5 m.s^{-1}; mean friction velocity was 0.026 m.s^{-1}; and mean and median Drag Coefficients were 5.28×10^{-3} and 3.03×10^{-3} respectively. $C_{1\ 50}$ compares favourably with the mean value of $C_1 = 3.1 \times 10^{-3}$ obtained by

Sternberg (1972) in Puget Sound.

Mean and median z_o values for all sites are 0.0087 and 0.0007m respectively. These bound the general roughness lengths for estuaries of 0.005m over rippled sand and of 0.001m for irregular gravel suggested by Heathershaw (1981). The mean value is very large considering the channels at Whangarei are typically shell-lagged. Thus, the median value should be more representative of the sea bed roughness.

Applying Millar's (1980) planimeter results, the harbour mean depth found by dividing the harbour volume by the surface area is 3.77m at mid-tide and 3.72m at a high spring tide. These are nearly equal because when the depth increases as the tide rises, so does the harbour surface area. Using either of these depths the median z_o corresponds with a Chezy's C of 59 $m^{\frac{1}{2}}.s^{-1}$ (equations 6.39 and 6.40). Mean z_o corresponds with Chezy's C of 40 $m^{\frac{1}{2}}.s^{-1}$; a very high frictional resistance.

Smith-McNaught (1976) obtained an average C=71 using the sea surface slope at peak flow found from the Port Whangarei and Marsden Point tide gauges. In Chapter 9, similar calculations suggest that the Smith-McNaught value is too large, and Chezy's C corresponds more nearly with C=60 $m^{\frac{1}{2}}.s^{-1}$ found from the median value of z_o.

Chezy's C of 60 converts to a friction parameter, r, of 2.7×10^{-3} which compares favourably with $r = 3.3 \times 10^{-3}$ (C=55 $m^{\frac{1}{2}}.s^{-1}$) recommended by Ramming and Kowalik (1980) for numerical modelling of estuaries. Mean r found by various researchers and compiled by Ramming and Kowalik (see their Table (p.17) for details) were 2.4, 2.6, 9.3 and 1.8. The second value, being the closest to the Whangarei result, was obtained by Grace (1936) in the Bristol Channel.

With equation (6.39), conversion of z_o to k gives a Nikuradse equivalent sand roughness of k=0.02m (median) and 0.26m (mean). Both of these are much greater than the mean sand grain size of approximately 0.00025m (Chapter 2). Thus, the bed roughness is associated with larger features.

6.12.2 The Drag Coefficient and the relationship of u_1 to u_*

Sternberg (1968) found that the Drag Coefficient was not very sensitive to bed type over rocks and gravel (maximum heights of 10 cm) and rippled sand (2 cm high). The mean value of C_1 varied by less than a factor of 2 i.e. 2.3×10^{-3} to 4.0×10^{-3}. In Chesapeake Bay (USA) Ludwick (1975) did not confirm this result but found instead that C_1 ranged through unacceptably wide limits and varied over 4 orders of magnitude. Ludwick also noted that over a movable bed in time varying flow, C_1 changed continuously with boundary shear stress.

Formed of the ratio u_*/u_1 the Drag Coefficient at Whangarei is shown explicity and implicitly on Figures 6.5 and 6.6 where the C_1 distribution is presented on the former and the friction velocity is plotted against the 1m

speed on the latter.

The Drag Coefficient is distributed over 4-5 orders of magnitude which compares with the similar spread noted by Ludwick (1975). Much of this is accounted for by the variations in form drag over the wide range of bed types in Whangarei Harbour including sand bedforms, shell, gravel and marine flora. The Drag Coefficient is therefore not constant, but varies widely for different bed types as well as changing throughout the tidal cycle at each location. The suggestion of Sternberg (1972) that C_1 can be considered constant at 3.1×10^{-3} has not been confirmed. Thus, as u_* and u_1 do not vary in the same ratio (Figure 6.6), a very important implication is the need to determine which parameters best represent the sediment transport rate i.e. the speed at 1m, the bed shear stress or a combination of the two. This question is considered in detail in Chapter 7.

Plotted on the log axis (Figure 6.5), the C_1 distribution appears to be nearly symmetrical and thus could be log-Normal. However, a Chi-squared Goodness of Fit test revealed that at the $\alpha = 0.05$ level of significance the data is not log-Normally distributed, having $\chi^2 = 292$ with 39 degrees of freedom which is considerably greater than the Tabled value.

Figures 6.7 and 6.8 show the Drag Coefficient against the friction velocity and the velocity at 1m. In the latter case, there is very little correlation, although there is a general decrease in C_1 with increasing speed at 1m and, as u_1 is in the denominator of the Drag Coefficient definition, the inverse relationship may be expected. This important result shows that, with all data combined, for a given speed at 1m the Drag Coefficient cannot be predicted. Best fit linear curves to the flood and ebb data for speeds greater than 0.3 $m.s^{-1}$ had correlation coefficients of only 0.16 and 0.05 respectively.

On the contrary, the Drag Coefficient and friction velocity are better correlated. The best fit power curve of C_1 against $|u_*|$ had a correlation coefficient of 0.94 with N=423. Figure 6.7 exhibits a fan-like increase in C_1 as u_* increases both positively and negatively. Thus, for a given friction velocity, a more reasonable estimate of C_1 could be obtained, although for the common value of $u_* \approx 0.03$ $m.s^{-1}$, C_1 still varies over more than 2 orders of magnitude.

As shown later, the friction velocity is related to bed geometry which makes it strongly site dependent. Furthermore, from equation (6.43),

$$C_1 = \left[5.75 \log_{10} (z_o)\right]^{-2} \qquad 6.45$$

and thus C_1 depends on the bed roughness length which is also site specific, and implies a relationship between C_1 and u_*. On the other hand, the bed roughness only depends on u_1 for speeds above the entrainment threshold. If the bed is composed of mobile, loose sand, the geometry will alter to some degree in accordance with water speed and the correlation between u_1 and C_1

should be better than over an immobile shell lag, for example. However, as the data includes all bed types low correlation between u_1 and C_1 could be expected to occur.

6.12.3 Negative friction velocity

Many of the profiles have a negative u_* (Figure 6.6). This is indicative of a profile having larger speeds near the bed than at the surface. It occurs more commonly at low u_1 but is not restricted to this range: one "reverse" profile has u_1=0.67 $m.s^{-1}$. The phenomenon could be attributed to density gradients, turbulence, and/or sea bed bathymetry.

The sites which most commonly exhibit reverse behaviour were A to M (excluding B). No salinities were recorded at these locations, so it is not known whether horizontal salinity gradients were exceptional. However, sites A, C, D, E, I, M, 7, 9 and C1 all had more than one reverse profile and with the exception of 7 and C1, these are all along the southern side of the harbour near the two wharves. Dintinct horizontal density stratification has been sighted occasionally in this region near Blacksmith's Creek after heavy rain. The fresher discoloured water and the more saline sea water, can be easily distinguished.

The other factor common to the sites along the southern side of the harbour was short velocity measurement times of 50s used at sites A to N. Turbulence may be playing a role with the number of reverse profiles merely reflecting the statistical probability that large-scale turbulent oscillations will exceed the expected logarithmic change in speed between levels. Drifting eddies may over-ride the logarithmic velocity gradient and this will be especially evident using short velocity measurement times.

Taking a simplified example, with typical values d=10m, u_*=0.03 $m.s^{-1}$, U_∞=1.0 $m.s^{-1}$, then at 2m above the bed (near the bottom of the outer layer), the UVDL gives u_2=0.82 $m.s^{-1}$. With a 10% reduction of the upper speed and a corresponding increase of the lower speed the profile could reverse. As quoted earlier, turbulent fluctuations in the Mersey Narrows averaged approximately 10% of the mean current. With periods of several minutes in the horizontal, similar behaviour at Whnagarei would be sufficient to cause profile reversal.

6.12.4 Correlation of water speeds to the logarithmic profile

The J-shaped distribution of the correlation coefficient representing the goodness of fit of the measured profile to the logarithmic shape is shown on Figure 6.9. The sample correlation coefficient, r, may be tested for significance with a Student's-t test (Freund, 1974). The null hypothesis that the population correlation coefficient is zero i.e. the profile exhibits no correlation with the logarithmic shape, can be rejected if,

$$t = \frac{r\sqrt{n-2}}{\sqrt{1-r^2}} \qquad 6.46$$

exceeds a Tabled value. The number of data points in the fit, n, must be three or more. By manipulation of the above formula, Table 6.4 was prepared giving the correlation that must be exceeded to reject the null hypothesis at the α =0.05 and α =0.10 levels of significance.

By comparison of the correlation coefficients for each profile with Table 6.4, the percentage of profiles which exhibited a logarithmic shape was found and compiled in Table 6.5. There is variation from site to site of 0% to 85% and overall, at the α=0.05 level of significance, an average of 43% of the ebb profiles, 26% of the flood profiles and 34% for both combined exhibited the logarithmic shape. This is a small percentage being much less than 82% obtained by Sternberg (1968). The poorer correlation at Whangarei results because:

(i) the outer boundary layer is more susceptible to turbulent fluxes which could not be totally accounted for when the profile was measured sequentially. Sternberg used an array of meters near the bed which measured speeds simultaneously.

(ii) Sternberg applied a different test for of goodness of fit; a test he described as "somewhat arbitrary". He fitted six velocity measurements (each averaged over a 10-min. period) to a straight line and defined a logarithmic profile as occurring when 4 of the 6 measurements were within 10% of the best fit curve. To show how this complies with the correlation coefficient an example is introduced using data from the outer boundary layer at 20:00 hrs, Site S4.

distance above bed (m)	2.0	4.0	6.0
measured speed ($m.s^{-1}$)	.47	.62	.64
best fit speed ($m.s^{-1}$)	.48	.59	.66

When fitted on a semi-logarithmic scale the velocities from the best fit curve are all within 10% of the measured speeds. This is three out of three which is better than 4 out of 6 and the profile would be logarithmic under Sternberg's criterion. The correlation coefficient was 0.96 and for α =0.05 the profile would be rejected, and for α =0.10 it would be just considered logarithmic, even though it passed Sternberg's test easily. It is unfortunate that Sternberg did not use a more rigorous approach. He sensibly chose the 10% limit because it was the approximate precision of the current meters. Considering the errors in measuring the speed which depends on instrument response in time-varying currents, as well as the duration of the averaging period and accounting for the errors associated with the measurement of distance above the bed the significance test, at the α =0.5 level is perhaps too stringent.

To be more compatible with Sternberg's criterion, the S-series sites were examined again for a logarithmic nature at an α =0.10 level of significance and

Table 6.5 shows that an average of 56% of the profiles were then logarithmic as against 20% at the 0.05 level.

Figure 6.10 presents an interesting result showing that for 423 profiles, the friction velocity is linearly related to the correlation coefficient. The correlation coefficient for this relationship was 0.69 compared to 0.01 for speed at 1m against r. The friction velocity varies widely as r approaches 1, but when the correlation to the logarithmic profile diminishes the friction velocity also exhibits a reduction. It would not be expected that a curve fitting procedure could be used when the data does not adhere to the theoretical curve, but as a result of the scatter between u_1 and r, the goodness of fit cannot be predicted from water speed alone.

6.12.5 Ebb and flood tides

Ludwick (1975) noted that the profile exhibited larger departures from the logarithmic during the flooding than ebbing stage of the tide. He found also that at the $\alpha = 0.05$ level of significance the Drag Coefficient on the ebb was greater than on the flood. He states (1975, p. 25), "for reasons unknown more energy is dissipated near the bed in friction or turbulence under ebb flow than under flood flow.". Knight (1981, p. 317) also found that "in general the flood resistance coefficients were slightly less than the ebb values" in a tidal reach of the Conwy estuary.

Similarly at Whangarei for all sites averaged, the median Drag Coefficients (Table 6.3) were 3.69×10^{-3} and 2.69×10^{-3} during the ebb and flood respectively implying higher bed friction during the ebb. Furthermore, the corresponding median correlation coefficients were 0.94 and 0.87, respectively and thus, deviations from the logarithmic profile were greater during the flooding than the ebbing tide.

Notably, for all sites, the average water speed at 1m was also greater during the ebb (Table 6.3). To examine whether the behaviour of the Drag Coefficient is related simply to water speed differences a sign test was employed to attempt to isolate causation in a process that is strongly site dependent. Thus, the number of sites where faster ebb speeds and larger ebb Drag Coefficients coincided were counted. This was undertaken for both the mean and median Drag Coefficients, with a result of 19 in 35 for the former and 18 in 35 for the latter. As this is not significantly different from the random expectation value of 1 in 2, no measurable relationship exists between speed and C_1 arising out of velocity asymmetry at each site.

With large samples, for the difference between the means to be significant, the z-statistic,

$$z = \frac{\bar{x}_1 - \bar{x}_2}{\sqrt{\frac{s_1^2}{n_1} + \frac{s_2^2}{n_2}}} \qquad 6.47$$

must exceed 1.96 at the $\alpha = 0.05$ level of significance (Freund, 1974). x_i are the means and s_i are the standard deviations and n_i are the number of values in the samples. Application of this revealed that the mean Drag Coefficients were not significantly different (z=0.46), but that the apparent difference can be accounted for by the large standard deviations of the data. However, when the correlation coefficient was tested with the z-statistic, r was found to be significantly greater (z=5.59) during the ebb than during the flood. Thus, there was a measurably improved correlation to the logarithmic profile during the ebb phase.

6.12.6 Accelerating and decelerating phases

The data were broken into a further two components (with $u_1 > 0.3$ m.s^{-1}) i.e. the accelerating and decelerating phases. Shown on Table 6.3, both the mean and median Drag Coefficients are greater during flow acceleration. This is at variance with the result of Soulsby and Dyer (1981) which predicts an underestimate of u_* and thus C_1 during the accelerating phase. A significance test revealed that the mean C_1 values were not different (z=1.23) for $\alpha = 0.05$. There were also no significant differences between the means of the correlation coefficients representing the goodness of fit to the logarithmic profile.

Greater turbulence levels during deceleration have been measured by Anwar and Atkins (1980). Also Gordon (1975) measured Reynolds stress and turbulent kinetic energy in a natural tidal boundary layer and found that for the same mean flow both these quantities increased by at least a factor of two, when the longitudinal pressure gradient changed from favourable to adverse. McCave (1973) was reported in Gordon to have measured a similar hysteresis in the relationship of friction velocity to current speed when the former was obtained from the velocity profile. These results have not been substantiated at Whangarei.

The correction for non-simultaneity which was applied to the data causes u_* to decrease during the accelerating phase. The effectiveness of this was demonstrated in Figure 6.3 where the accelerating data approached the best fit line from above (and conversely for deceleration) until the scatter was greatly reduced. To observe the behaviour cited by Gordon, the friction velocities would have to pass and eventually lie below the best fit line during acceleration. The accuracy of the non-simultaneity correction depends on the estimate of the acceleration obtained from three half-hourly spaced data points, which are themselves affected by turbulence. The correction is possibly not sufficiently accurate to allow identification of varying turbulence levels during the accelerating and decelerating phases of the tide.

Further analysis revealed that of 38 reverse profiles, 18 occurred on accelerating conditions and 20 during decelerating: a difference which is not significant. If turbulence is responsible for reverse profiles, then once

again, no marked increase in turbulence levels was identified during deceleration.

6.12.7 Sand and shell beds

The sites which were sand floored and those with a shell-lag bed were examined separately (Table 6.3). $C_{1\ 50}$ was 3.82×10^{-3} over sand and 2.52×10^{-3} over shell. This reflects greater obstacle form drag resistance on sand due to the bedforms which protrude into the flow more than the shells. Mean Drag Coefficients were higher at 5.46×10^{-3} and 4.94×10^{-3} respectively. Median values of z_o of 0.0015m and 0.0003m correspond with k=0.05m sand and k=0.01m (shell) and both are intuitively sensible. The S-series sites over megarippled sand had a median z_o of 0.0022m when averaged. This compares with z_o = 0.005m suggested by Heathershaw (1981) for sand beds.

With the z-statistic, the mean Drag Coefficients over sand and shell were not found to be significantly different (z=0.77). However, a 2-way median test (Freund, 1974) showed that with one degree of freedom, the medians were significantly different. This variation corresponds with a five-fold increase in median z_o on sand above the shell value.

6.13 The Drag Statistic

The analysis in this chapter has shown that the Drag Coefficient is not constant but varies over several orders of magnitude. Thus it is not possible to accurately predict the friction velocity from the 1m speed using an assumed Drag Coefficient. However, the Drag Coefficient may be found knowing two speeds and their distances above the bed. Analysis was performed to determine the accuracy of this procedure with measured profiles.

Taking the KP equation,

$$u = u_* \ 5.75 \ \log_{10} \ (z/z_o) \qquad 6.48$$

Then for two speed measurements u_{z1} and u_{z2} at z_1 and z_2 metres above the sea bed,

$$u_{z1} = u_* \ 5.75 \ \log_{10} \ (z_1/z_o) \qquad 6.49$$

$$u_{z2} = u_* \ 5.75 \ \log_{10} \ (z_2/z_o) \qquad 6.50$$

Subtraction of these and simplification produces,

$$\frac{u_{z2} - u_{z1}}{5.75 \ \log_{10} \ (\frac{z_2}{z_1})} = u_* \qquad 6.51$$

The Drag Coefficient is formed after division by the one metre speed and squaring thus,

$$C_1 = (\frac{u_*}{u_1})^2 = \left[\frac{u_{z2} - u_{z1}}{5.75 \ u_1 \ \log_{10} \ (\frac{z_2}{z_1})}\right]^2 \qquad 6.52$$

As such three velocities must be measured. However, if z_1 is chosen as 1m, and denoting the function as C_{DS}, the Drag Statistic, then,

$$C_{DS} = \left[\frac{u_{z_2} - u_1}{5.75\ u_1 \log_{10}(z_2)}\right]^2 \qquad 6.53$$

and only 2 speeds are required. This is then only valid if u_{z2} and u_1 are measured in the inner boundary layer. By a similar procedure with the UVDL,

$$C_{DS} = \left[\frac{u_{z_2} - u_1}{8.6\ u_1 \log_{10} z_2}\right]^2 \qquad 6.54$$

and this requires that u_{z2} and u_1 be measured in the outer boundary layer. As u_1 has to be either in the outer or inner layer only one of the two equations is valid, for any particular series of measurements. The grace of these equations is in the removal of the difficult parameters to measure; the boundary layer thickness and free stream velocity for the UVDL and the roughness length for the KP.

To distinguish this procedure from the Drag Coefficient found in all cases by curve fitting to the velocity profile, the parameter here was denoted as the "Drag Statistic", C_{DS}.

Using 3 velocity measurements, i.e. the measurement at the top and bottom of each layer as well as u_1, the Drag Statistic was found using Whangarei data with equations (6.53) and (6.54) and compared with the Drag Coefficient on Figure 6.11. The correlation coefficient linearly relating the two was very high at 0.96. Therefore, a good estimate of the Drag Coefficient can be obtained from this method without having to curve fit the data.

The boundary shear stress can be also found in this manner. Using equation (6.51) in the inner layer,

$$\tau_{DS} = \rho u_*^2 = \rho \left[\frac{u_{z2} - u_{z1}}{5.75 \log_{10}(\frac{z_2}{z_1})}\right]^2 \qquad 6.55$$

This equation was given by Sternberg (1972) although the squared factor was neglected due apparently to a typographical error. A similar equation can be obtained from the UVDL and in S.I. units with $\rho = 1025\ kg.m^{-3}$.

$$\tau_{DS} = a \left[\frac{u_{z_2} - u_{z_1}}{\log_{10}(\frac{z_2}{z_i})}\right]^2 \qquad 6.56$$

where $a = 31.0\ kg.m^{-3}$ in the inner layer and $a = 13.9\ kg.m^{-3}$ in the outer layer.

6.13.1 Application of the Drag Statistic

Figure 6.12 shows the Drag Statistic plotted against the ratio of the measured speed at z metres above the bed and the 1m speed. This figure applies if all speeds were measured in the outer boundary layer and strictly this limits the water depth to about 6m. Nevertheless, as velocities are often

measured in the outer boundary layer this figure was generated to suit this experimental procedure. A first estimate of C_1 can be obtained from the Figure but the full formula given in the text would supply an improved result, especially if u_1 as well as a speed near 20% of the depth and one near the surface were measured, which would allow application of equations (6.54) and (6.56).

6.14 Bed friction and sediment transport experiments

Six full-tidal cycle experiments were conducted over megarippled beds, at locations of significance to the harbour sediment circulation and are described in much greater detail in Chapter 7. Titled the S-series (S1 to S6), measurements were made of water velocities and the bedload and suspended load. During the first two experiments (S1 and S2), the sea floor was observed on an underwater camera. During slack water and throughout the tide, divers measured the distance between adjacent megaripple crests, the lengths of the slipface and stossface, and the megaripple height over at least 6 consecutive features. The crest positions were marked with stakes at slack water, both for identification of the feature and for subsequent measurements of megaripple advance under tidal flow. Bedforms were photographed against a graduated board and these pictures were used to confirm the megaripple height measurements.

At this stage, the data compiled here and in Chaper 7 will be applied to (i) bed friction losses; (ii) the correlation of this with the bedform geometry; and (iii) the applicability to tidal flow of various friction factors. The results are related to sediment transport in the next Chapter. General site statistics and locations are given in Table 5.9 and Figure 5.25.

6.14.1 General results

Figures 6.13-6.17 present the velocities at 5 of the S-series sites after correction for non-simultaneity. (Data from S6 was rejected due to wave interference.) There is a clear increase in speed as the surface is approached. At all sites excepting S5 in Shoal Bay during the ebb, maximum speed at any level is nearer high than low water on both the flood and ebb. This is more evident on Figure 6.18 which shows the mean vertically-averaged speeds. The flood accelerates slowly reaching a maximum 2-2½ hours before high tide. This behaviour has been discussed in Chapter 5. At all sites, the direction of flow oscillates about the mean value (Figures 6.13-6.17) and S3 on 9-Feb-82, exhibits particularly large deviations in the early stages of the flood.

All data were fitted to the logarithmic profiles and correlation coefficients are shown in Figure 6.19. More than half the correlations are significant (Table 6.5), but there are deviations which appear to be random. The profile is not always logarithmic and estimates of u_* found by curve fitting must include these deviations.

Figures 6.20 (a,b) show the full-tidal cycle friction velocities and the speeds at 1m. The friction velocity varies over a wider relative range than the speed at 1m. For example, the friction velocity at S3 is about 5 times the value at S2. The explanation for this is later shown to lie with the changing form drag and sympathetic Drag Coefficient which are indicative of bed geometry.

The short-term oscillatory nature of the friction velocity, compares with similar variations described by Dyer (1970) as "anomalously high and low" and assessed to be associated with the position relative to the megaripple crest at which the profile was measured. At Whangarei, measurements were made in both the inner and outer layers and the long "memory" of the outer layer ensures that the measurement position relative to the bedform is not as critical. Thus, peak u_*, which in all cases is very distinctive, is possibly related to the time when the megaripples are turning around. In the early tidal stages, the megaripples have their steep faces pointed into the flow and offer more resistance. Until the unstable megaripple configuration is re-oriented, the energy losses should be greater which would inflate the early friction velocities. In nearly all cases (ebb and flood), the peak u_* (Figure 6.20) occurs when the 1m speed is about 0.5 $m.s^{-1}$.

When the maximum speed attained at each of the S-series sites is plotted against the median Drag Coefficient, a linear proportional increase in the two variables is exhibited (Figure 6.21). Thus, the bed roughness (and C_1) over loose sediments increases with the speed at 1m. Furthermore, the Figure may be used to estimate C_1 from the maximum water speed. Grain sizes which describe the bed sediments are given in Table 7.2.

6.14.2 The relationship of the Drag Coefficient to the bedform geometry

Dyer (1980) following Lettau (1969) suggests that the areal concentration of features is related to the flow roughness length as, $z_o = h.s/2S$, where h is the effective obstacle height, s is the obstacle's cross-sectional area seen by the flow per unit horizontal area, S. For two-dimensional megaripples,

$$z_o = \Delta^2/2\Lambda \qquad 6.57$$

and inserting this into the KP equation produces,

$$C_1 = [5.75 \log_{10} (2\Lambda/\Delta^2)]^{-2} \qquad 6.58$$

Using the bedform statistics from Table 7.2, the expected value of C_1 was calculated with equation (6.58) above and compared with the median and mean values of C_1 obtained from u_1 and u_* with the latter found by curve fitting the velocity profiles. Shown in Table 6.6, there is a general agreement.

Several approximations have been made: (i) the side-scan sonar survey revealed that the bed is not truly 2-dimensional but that the megaripples have sinuous crests at the experimental sites; (ii) the analysis in Chapter 2,

showed that the bedform wavelengths exhibited a Normal probability distribution and the choice of a mean value to represent this density function is a simplification of the real process; and (iii) the Drag Coefficient (Figure 6.22) and the bedform geometry vary over the tidal cycle. If the bedform wavelength were to be kept constant, a correction to the bedform height, which would account for the difference between measured and calculated C_1, would be less than 20% at the 3 most reliable sites (S3, S4 and S5). Thus, with these considerations, the general correlation between the Drag Coefficient found from the velocity profile and the bedform geometry is notable. It is therefore apparent that the Drag Coefficient cannot be approximated by a constant value because it varies in response to changing bed geometry and areal concentration. Furthermore, the overall agreement using average bedform geometry and Drag Coefficients supports the suggestion of Smith and McLean (1977) that the velocity profile in the outer layer (speeds were measured in both the inner and outer layers at Whangarei) reflects the average shear stress over a large region of the bed.

At Whangarei, typical magnitudes over megaripples were $C_1 = 0.004$ to 0.007. Notably, this is considerably less than Ludwick's (1975) results which showed C_1 being in the range of 0.021-0.028 for "duned" beds with sand of 0.28mm size. In view of the noted correlation of the Whangarei data to the bed geometry and to the mean C_1 of 0.0031 found by Sternberg (1972), Ludwick's result appears to be the exception, but no information was supplied with respect to megaripple steepness in Ludwick's paper. His result corresponds with an exceptionally high value of $z_o = 0.012$m.

6.15 Form drag and skin friction

As proposed by Einstein and Barbarossa (1952) shear stress may be partitioned into three components due to (i) skin friction; (ii) form drag; and (iii) energy taken from the flow when sediment is suspended. If the third component is assumed to be small or included in the skin friction as an effective component then, according to Einstein and Barbarossa, the following relationships can be written,

$$u_*^2 = u_*'^2 + u_*''^2 \qquad 6.59$$

$$1/c^2 = 1/c'^2 + 1/c''^2 \qquad 6.60$$

$$f = f' + f'' \qquad 6.61$$

where (') denotes skin friction and ('') refers to form drag. The phrase "total friction velocity" shall describe the friction velocity including both components.

Skin friction is related to the grain or surface roughness. Grains project into the flow and energy is lost when water passes these roughness elements. The form drag arises out of the energy losses induced by turbulent

flow past an obstacle (such as a bedform) and does no work on the bed. The skin friction component is associated with the transport of sediment. However, turbulence resulting from form drag over the bedforms can stir up sediment which is then converted away by the flow. Moreover, turbulence levels and vertical flow velocities are especially high at the point of flow reattachment as seen on the underwater video in Whangarei Harbour (Chapter 7) and measured by Raudkivi (1963). Thus, even though the form drag does no work on the sediment grains, the presence of large bed irregularities is linked with the skin friction and the ability of the flow to move sediment.

For sediment transport calculations, normally one attempts to de-couple these two components and ultimately, an empirical constant in the transport equation is adjusted to define an "effective" stress (Yalin, 1963; Engelund and Hansen, 1967; etc.). Furthermore, an "effective" roughness is often used to described the bed roughness found to best represent the energy lost by the flow while sediment is being transported. For example, Vittal et. al. (1973) conclude that the effective shear stress for bedload transport is approximately equal to the grain shear stress, but under high suspended load conditions, the effective shear stress is much larger. The effective and grain or surface roughness should not be confused.

In non-steady tidal flow, the problem is further complicated by the changing, loose-sediment bed geometry as the tide advances. Being related to current strengths, the bed geometry alters with the current speeds. Moreover, the bedform profile changes at a slower rate than the flow conditions because of the greater "inertia" of the former (Allen and Friend, 1976). Fully-developed megaripples are not usually produced by non-steady flows.

Much of the literature considers the form drag and skin friction components for steady-state flow and bed equilibrium conditions (Graf, 1971; Vanoni, 1975; Simons and Senturk, 1977). The problem for non-steady tidal flow has not been as well researched and it is hoped that this and the next Chapter will offer some new insights. Isolation of the two components (skin friction and form drag) is broached in this Chapter and the results are used to predict transport rates in Chapter 7.

The specific aim of the following sections is to find form drag and skin friction formulations which are valid over megaripples under tidal flow. To make this assessment, after examination of various formulae of both kinds, separately-determined form drag and skin friction factors are summed according to equation (6.60) and compared with friction factors obtained experimentally by curve fitting the measured vertical velocity profiles.

6.15.1 Einstein and Barbarossa procedure

Einstein and Barbarossa (1952) made the first significant contribution to the subject. Using data from rivers, they presented a curve relating the mean

vertically-averaged speed to the skin friction component of the Shield's Entrainment Function, θ. The skin friction and form drag components are found by successive interations and the method is adequately described in Yalin (1977). However, calculations with typcial Whangarei data, produced either unrealistic values for the form drag or no solution whatever.

To circumvent the iteration procedure, the Einstein-Barbarossa curve was plotted in a site specific format (Figure 6.23). To produce this diagram, values of the curve relating $\bar{u}/u_*''$ to $(\theta')^{-1}$ were taken from Figure 8.3 in Yalin (1977).

$$\theta' = \frac{\rho u_*'^2}{(\rho_s - \rho)\, g D_{35}} \qquad 6.62$$

and ρ is the water density, ρ_s is the sediment density and D_{35} is the grain size for which 35% of the distribution is finer by weight. By entering 6.23 with the mean speed and the friction velocity, the intersection of the two curves should supply the magnitudes of u_*' and u_*''.

In many cases with Whangarei data, it was found that the $\bar{u}$ and u_* curves did not intersect and no solution was possible especially near the beginning or end of the tide. Over bedforms which are not fully developed or relict from the previous half-tidal cycle, near the beginning of the tide the form drag component is higher than would occur if the bedforms and the water speeds were in equilibrium. In the latter case, small speeds should result in smaller bed features. Furthermore, the Einstein-Barbarossa relationship which relates mean speed to friction velocity assumes that the bed friction and the mean water speed have attained an equilibrium as occurs in steady-state, uni-directional flow in rivers and channels. This does not necessarily occur under non-steady tidal flow where accelerating mean speeds through a region are not in equilibrium with the bed friction losses.

6.15.2 Form drag

6.15.2.1 *Alam and Kennedy procedure*

Alam and Kennedy (1969) presented a family of curves relating the hydraulic radius and D_{50} grain size to the form drag Darcy-Weisbach friction coefficient. Yalin (1977) notes correctly that the analysis is only adequate for antidunes. Insertion of some typcial Whangarei data into their relationships produced unrealistic solutions e.g. they state that dune wavelengths can be found as,

$$\Lambda = \frac{2\pi}{g}\, \bar{u}^2 \qquad 6.63$$

and with $\bar{u} = 1.0\ m.s^{-1}$ then $\Lambda = 0.6m$ which is almost an order of magnitude less than a measured value of 4.6m.

6.15.2.2 *Engelund and Hansen procedure*

Engelund and Hansen (1967) employed Carnot's equation for sudden flow expansion to derive the form drag Darcy-Weisbach friction coefficient,

$$f'' = \frac{\Delta^2}{\Lambda d} \qquad 6.64$$

The Darcy-Weisbach coefficients are related then as,

$$f = f' + \frac{\Delta^2}{\Lambda d} \qquad 6.65$$

For typical values, $\Delta = 0.25$, $\Lambda = 4.0$ and $d = 10$, then,

$$c'' = \sqrt{\frac{8}{f''}} = 71.6 \qquad 6.66$$

which is unrealistically high.

6.15.3.2 *Vanoni and Hwang procedure*

Vanoni and Hwang (1967) conducted tests in a flume with an erodible bed of ripples and megaripples to isolate the surface and form drag friction factors. Over a two-dimensional bed, the Darcy-Weisbach form drag friction coefficient was found to be best represented as,

$$1/\sqrt{f''} = 3.3 \log_{10} (d\Lambda/\Delta^2) - 2.3 \qquad 6.67$$

where the gradient and intercept of this line are empirical.

Pillai (1979) performed a similar analysis over fixed, roughness elements and obtained,

$$1/\sqrt{f''} = 3.52 \log_{10} (d\Lambda/\Delta^2) - 2.46 \qquad 6.68$$

Both these equations, with typical values, produce a realistic f". With the data previously inserted into Carnot's equation, c"=19.7 using the Vanoni and Hwang formulation.

6.15.2.4 *Comparison of the form drag formulations*

Figure 6.24 compares the Engelund and Hansen, Vanoni and Hwang, and Pillai formulations and over a narrow range all three give very similar answers. The Engelund and Hansen equation, which was derived from considerations of the expansion loss past the bedform crest, is very similar to the equations obtained by empirical means over the range $20<\Lambda d/\Delta^2<70$, which suggests that the Carnot theory is valied at least over the quoted range. When the abscissa increases, for example in greater depths, the friction coefficient decreases rapidly. Although intuitively correct (the flow expansion becomes relatively less important as the depth increases or as the megaripple steepness decreases), the friction coefficient changes unrealistically fast, as previously shown for a depth of 10m. The other two are very similar. The Vanoni and Hwang equation included experiments over natural beds, and it represents the compilation of a wider data base. In summary, of the equations examined to describe the form

drag energy losses in tidal flows over megaripples, it is evident that the Vanoni and Hwang formulation is the only one which gives realistic estimates of the friction coefficient. However, this does not mean that the equation is definitely valid and vertification can only be undertaken after determination of a representative skin friction formulation.

6.15.3 Skin friction

There are two major aims of this analysis. One is to isolate the form drag and skin friction components and the other is to apply the results to sediment transport. Thus, in this section, both the grain roughness and the effective roughness are examined together and the distinction is noted here.

6.15.3.1 *Engelund and Hansen and the Kishi and Kuroki procedures*

Engelund and Hansen (1967) showed that the effective stress at the bed could be found from the Shield's entrainment function as,

$$\theta' = 0.06 + 0.4\theta^2 \qquad 6.69$$

This is used by Engelund and Hansen to develop a total load sediment transport equation discussed in Chapter 7.

Kishi and Kuroki (1967) obtained a similar relationship,

$$\theta' = 0.21\theta^{\frac{3}{2}} \qquad 6.70$$

Both of these derive the skin friction as a proportion of the total friction velocity. There is an implicit assumption that the form drag energy losses depend only on the total friction velocity for particular grain sizes and densities. Thus, the expansion loss must be the same fraction of the total energy loss for all flows over non-cohesive beds. This assertion is expressed by the Engelund and Hansen (1967) "Similarity Principle".

On Figure 6.25, the two equations are compared and over the range of $0.1<\theta<0.5$, they are similar. The Engelund and Hansen formulation approaches a critical sediment threshold of 0.06, as θ becomes small, and above $\theta = 0.5$, which corresponds with transition to the upper flow regime, the equation is not valid.

6.15.3.2 *Manning-Strickler equation*

A method often used to find the skin friction velocity employs the Manning-Strickler equation which relates the mean speed to the skin friction velocity as,

$$c' = \bar{u}/u_*' = 7.66\ (d/k')^{\frac{1}{6}} \qquad 6.71$$

where k' is usually taken to be D_{65}, the grain size for which 65% of the distribution is finer by weight.

Raudkivi (1976) in his Figure 6.2 presents the friction coefficient against flow speed over a sand bed. The skin friction was obtained with a

Manning-Strickler type equation but the effective surface roughness was found to be given by a grain size of D=0.001m when the actual geometric mean diameter was 0.000152m. Thus, the effective roughness is evidently under-estimated by the Manning-Strickler formula unless compensation is made using a larger grain size. Raudkivi's figure shows the friction coefficient rising steeply above that due to the surface roughness as ripples grow and the form drag becomes important. With increasing speed, as the ripple index decreases, the friction coefficient drops and again approaches a value associated with grain roughness only on a transition plane bed.

Sediment transport formulae (Vanoni, 1975) commonly use Manning-Strickler type equations to find the skin friction factor. For example, Laursen (1958),

$$c' = \bar{u}/u_*' = 7.62\ (d/D_{50})^{\frac{1}{6}} \qquad 6.72$$

Meyer-Peter and Muller (1948),

$$c' = 8.30\ (d/D_{90})^{\frac{1}{6}} \qquad 6.73$$

Vittal, Raju and Garde (1973),

$$c' = 7.66\ (d/D_{65})^{\frac{1}{6}} \qquad 6.74$$

These all produce very similar results depending on the shape of the grain size distribution.

6.15.3.3 *Karman Prandtl equation*

The Karman Prandtl equation relates the friction velocity to the mean velocity as,

$$\frac{\bar{u}}{u_*} = 5.75 \log_{10} \left(\frac{0.37\ d}{k\ /30.2}\right) \qquad 6.75$$

where the mean velocity occurs at 0.37d, and k is a bed roughness which has been shown earlier to depend at least on bed geometry. Engelund and Hansen (1967) reintroduced the equation of Einstein (1950) for calculating side-wall correction to isolate the skin friction velocity as,

$$\frac{\bar{u}}{u_*'} = 5.75 \log_{10} \left(\frac{0.37d'}{k'/30.2}\right) \qquad 6.76$$

which has the same form as the previous equation, except for the interpretation of d' and k'. In the iteration procedure of Einstein and Barbarossa examined earlier, d' is that part of the total depth assessed to be associated with skin friction energy losses and is thus only equal to the total water depth when the form drag is zero over a plane bed. However, Engelund and Hansen associated d' with the boundary layer width, or total water depth in fully-developed flow. This assumption relies on the correct choice of k' which, over megaripples, would necessarily be less than the effective roughness including the form drag. Engelund and Hansen (1967) selected $k' = 2.5\ D_{50}$ and this was later revised and reported in Raudkivi (1976) as $k' = 2D_{65}$. Thus,

$$\frac{\bar{u}}{u_*} = 5.75 \log_{10} \left(\frac{0.37d}{2D_{65}/30.2}\right) \qquad 6.77$$

The important aspect to note here is that the equation is to be employed over megarippled, as well as plane beds.

On the assumption that the above equation is valid, this writer here introduces a similar equation using the velocity at 1m instead of the mean speed and then,

$$\frac{u_1}{u_*'} = 5.75 \log_{10} \left(\frac{30.2}{2.D_{65}}\right) \qquad 6.78$$

6.15.3.4 *Comparison of skin friction formulations*

To examine the suitability of the previous equations to the marine environment, the S-series data were employed for comparisons. Skin friction velocity was calculated (i) using the Engelund and Hansen equation (6.69) with the friction velocity obtained by curve fitting the profiles and with a sediment density of 2650 $kg.m^{-3}$ and D_{50} grain size from Table 7.2 (EH); (ii) using the Manning-Strickler formula (6.71) with the mean speed and the D_{65} grain size (Table 7.2) for each profile (MS); and (iii) using the KP equation with the 1m speed and the D_{65} grain size (KP). Notably, the last two require no curve fitting or knowledge of the friction velocity.

The first two were compared on Figure 6.26 and the correlation is poor. Thus, the mean water speed and the total friction velocity are not well correlated. This restates the previous result obtained when the Einstein and Barbarossa division of skin friction and form drag were examined. In that case, poor correlation between the mean speed and the friction velocity was partly responsible for the inadequacy of the method. In support of this Figure 6.27 plots the friction factor, $c = \bar{u}/u_*$, and variation throughout the tidal cycle is clearly evident. Notably, the trend seen on Figure 6.26 results from the algebraic formulation of the Engelund and Hansen equation, as the behaviour shown is mirrored by the plot of the equation in Figure 6.25.

At low water speeds, the threshold of 0.06 in the Engelund and Hansen equation flattens the curve. It should be noted that an "effective" friction velocity is being considered and so the behaviour below the sediment threshold is not important. For Whangarei grain sizes, the value of $\theta = 0.06$ corresponds closely with the critical skin friction velocity obtained from the Yalin sediment threshold curve (Miller et. al., 1977).

On Figure 6.28, the KP and MS equations are compared. The former depends on the 1m speed and the latter on the mean water speed. An alternative comparison with the KP equation using the mean speed rather than u_1 (6.77) degenerates into a comparison of two equations with different functional forms but the same free parameters. Algebaraic manipulation produces,

$$\frac{u_*'\ (MS)}{u_*'\ (KP)} = 0.75\ (\frac{D_{65}}{d})^{\frac{1}{6}}.\ \log_{10}\ (\frac{5.74d}{D_{65}}) \qquad 6.79$$

and for relevant depths, this ratio is less than 1.0 and varies from 0.83 at 1m depth to 0.63 in 30m for D_{65} = 0.0003m. If a roughness of $2D_{65}$ is introduced into the MS equation the ratio is still less than 1.0. Similarly, the graphical comparison (Figure 6.28) shows that the KP equation using the 1m speed arrives at a greater u_*'. The correlation between the two methods is visually fairly good and is considerably better than for the previous comparison. The scatter arises out of poor correlation between the mean vertically-averaged water speed and the speed at 1m. Assuming that mean speed occurs at 0.38 of the water depth, then the relationship of these two is at least a function of the water depth.

Under tidal flow, where depths may be very large (e.g. 10-35m), a near-bed speed should represent the sediment transport rate better than the mean speed in the water column. Sternberg (1972), Gadd et. al. (1978) and Heathershaw (1981) all predicted transport rates in tidal inlets using the near-bed 1m speed. Thus, a procedure using the 1m speed for estimating the skin friction and the effective bed stress is preferred. However, it is evident from the comparisons made that each of the three methods examined, if applied to sediment transport predictions, would systematically and randomly derive different transport rates. This is particularly evident in Figure 6.29 which compares the skin friction values obtained from the Karman Prandtl and Engelund and Hansen formulations.

6.15.4 Unification of the skin friction and form drag

Method 1: Using average bedform heights and wavelengths and the mean depth (Table 7.2), the form drag friction factor, c", was calculated for four of the S-series sites with the Vanoni and Hwang equation (6.67). For the same sites, the skin friction factor, c', was found with both the KP and MS equations (6.77 and 6.71 respectively) using the mean depth and D_{65} grain sizes (Table 7.2) in both cases. The total friction factor, c, for each site could then be found by summing the components according to equation 6.60 as,

$$1/c^2 = 1/c'^2 + 1/c''^2 \qquad 6.80$$

These results are compiled in Table 6.7.

Method 2: The friction factors at these sites, determined by curve fitting the measured speeds to the logarithmic profile to find u_* and by calculation of the mean flow speed for each profile, are shown in Figure 6.27 against the time after high tide. From these, for each site, the median friction factor was obtained (Table 6.7), for comparison with the previous data.

There is excellent agreement between the two methods of obtaining c, especially with the Manning-Strickler formulation in combination with the Vanoni

Hwang equation which exhibits no more than a 3% deviation from the median friction factor. With the Karman-Prandtl and Vanoni and Hwang equations, the deviations are larger but they are very consistent being 6-10% less than the median c obtained from the velocity profile and mean speed. Notably, the friction factors were derived by two entirely independent methods i.e. (i) from the bed parameters (grain size and megaripple geometry) and (ii) from the measured velocity profile and mean speed. This consistency is therefore very encouraging considering that several assumptions were made including: (i) the bed geometry measured at slack water is representative of the bed during the tide; (ii) a mean water depth taken as the depth at mid-tide is sufficient; (iii) the megaripples are two-dimensional; and (iv) the D_{65} grain size represents the bed grain roughness adequately.

Further examination revealed that the friction factor predicted using the Karman Prandtl and Vanoni and Hwang equations corresponds most nearly with the 39th percentile of the friction factors at each site such that 61% of the c values were greater than the predicted values. As bed roughness increases with a decrease in c, this corresponds with a slightly rougher bed than what is represented by the median friction factor and predicted with the Manning-Strickler equation.

6.16 Conclusions

(i) mean friction velocity at the measured sites in Whangarei Harbour was found to be 0.026 $m.s^{-1}$.

(ii) mean and median Drag Coefficients were 5.28×10^{-3} and 3.03×10^{-3} respectively.

(iii) median z_o for the Harbour was found to be 0.0007m which corresponds with typical roughness over irregular gravel or shell. This concurs with sea floor photography and side-scan sonar evidence that the channels are shell-lagged.

(iv) using the mean harbour depth, median z_o corresponds with a Chezy's C of 59 $m^{\frac{1}{2}}.s^{-1}$.

(v) the Drag Coefficient was found to vary over 4-5 orders of magnitude changing with bed geometry at different locations and throughout the tidal cycle at each site.

(vi) as u_1 and u_* do not form a consistent ratio, it is important to ascertain which of these speeds can best be applied to determine sediment transport rates.

(vii) over all bed types the Drag Coefficient cannot be predicted from the 1m speed without some prior indication of the bed geometry. However, over megarippled, mobile beds, a linear relationship was noted between the Drag Coefficient and 1m speed which may be applied to determination of C, from u_1.

(viii) at the $\alpha = 0.05$ level of significance, only 34% of the profiles

were found to be logarithmic. Using a less stringent test, at the $\alpha = 0.10$ level of significance, the S-series profiles were found to be logarithmic 56% of the time, as against 20% for $\alpha = 0.05$.

(ix) the magnitude of the friction velocity, but not the 1m speed, was found to be related to the goodness of fit of the measured speeds to the logarithmic profile. The goodness of fit cannot be predicted from the 1m speed alone.

(x) larger ebb than flood Drag Coefficients were measured, but a z-statistic test revealed that the differences were not significant. However, there was a measurably improve correlation of the measured speeds to the logarithmic profile during the ebb phase of the tidal cycle.

(xii) no significant differences between Drag Coefficients determined during accelerating and decelerating phases of the tidal cycle were noted.

(xiii) median z_o values of 0.0015m and 0.0003m were obtained over sand and over shell beds respectively. At the S-series over megaripples the average of the median z_o values was 0.0022m. These differences in z_o over sand and shell were significant at the $\alpha = 0.05$ level.

(xiv) median Drag Coefficients over sand and shell beds were 3.82×10^{-3} and 2.52×10^{-3} respectively.

(xv) a function denoted as the Drag Statistic, C_{DS}, was derived and found to represent the Drag Coefficient obtained by curve fitting the velocity profile to obtain u_*. Application of this function was enabled by equations (6.53), (6,54) and (6.56) and Figure 6.12.

(xvi) using an equation presented by Dyer (1980) which derives the roughness length from the bedform height and wavelength, the Drag Coefficient was obtained. Good agreement between it and the Drag Coefficient found from the velocity profile was noted. Thus, the roughness length over megaripples may be approximated by equation (6.57).

(xvii) the Einstein and Barbarossa (1952) division of form drag and skin friction was found to be unworkable over megaripples under tidal flow. This occurs primarily because the bed and the flow do not attain an equilibrium state under non-steady tidal flow.

(xviii) an equation to derive the skin friction velocity using the 1m speed in the Karman Prandtl equation was introduced (6.78). Comparisons with the Manning-Strickler formula revealed that the former derives larger skin friction velocities but that the relationship between the two is reasonably linear. However a considerable amount of scatter occurred between the skin friction derived from the 1m speed and that derived from the friction velocity with the Engelund and Hansen formulation. This has ramifications with regard to sediment transport determination.

(xix) form drag friction factors were determined using the Vanoni and Hwang (1967) equation (6.67) from bedform geometry and mean depth. Skin

friction factors were obtained with the Manning-Strickler equation (6.71) and the Karman Prandtl equation (6.77) using mean speeds and the D_{65} grain size. After the components were summed appropriately, with the Manning-Strickler equation the resulting friction factor was within 3% of the median friction factor obtained from the measured velocity profiles at 4 independent locations. Thus, the Vanoni and Hwang equation (6.67), evidently represents the form drag energy losses under tidal flow over megarippled beds and the Manning-Strickler equation (6.67) represents the skin friction. The Karman-Prandtl equation (6.78) predicts a consistently higher skin friction velocity than the Manning-Strickler formula. In combination with the Vanoni and Hwang equation derived friction factors corresponded with a rougher bed than what is represented by median c, being associated with the 39th percentile of the measured friction factors. With regard to sediment transport rates, it is unknown which of these - Manning-Strickler or Karman Prandtl - best represents the effective skin friction. This is examined in the next Chapter.

SEDIMENT TRANSPORT AND TIDAL INLET HYDRAULICS

Chapter 7:

SEDIMENT TRANSPORT UNDER TIDAL FLOW OVER BEDFORMS

CHAPTER 7: SEDIMENT TRANSPORT UNDER TIDAL FLOW OVER BEDFORMS

7.1 Introduction

Transport of bed sediments by uni-directional currents has been often studied (Einstein, Bagnold, Yalin, Engelund and Hansen etc.). Nevertheless, there is still debate in the literature about the best method to predict sediment fluxes from the hydraulic parameters. Moreover, under non-steady reversing tidal flow over a bedformed sea bed, the transport rates are less well understood. Complying with this, Heathershaw (1981) noted that no transport equations have been developed for use in the sea under oscillatory tidal currents, but that most are based on steady flow concepts. Partly, this has resulted from an experimental data deficiency. Both the necessary experimental techniques and the adversity of the underwater environment make accurate data collection extremely difficult.

At Whangarei, a series of 6 experiments, titled the S-series, were established to make bedload, suspended load and megaripple advance measurements. Due to their scope, the experiments required coordinated assistance from the Northland Harbour Board, the Royal New Zealand Navy and the University of Waikato who jointly participated in the field data collection. The results were applied to determination of sediment transport rates under tidal flows. Sediment threshold as well as the bedload and suspended load were examined. Some major results were published (Black and Healy, 1982b) in a paper prepared from an early draught of this and the last Chapter.

7.2 List of Symbols

a	function in Yalin equation, reference level for suspended sediment measurements
c''	form drag friction factor ($\bar{u}/u''$)
C	Chezy's C ($m^{\frac{1}{2}}.s^{-1}$), concentration of suspended sediment ($kg.m^{-3}$)
C_a	concentration of suspended sediment at reference level ($kg.m^{-3}$)
C_1	Drag Coefficient
d	water depth (m)
D	grain size (m)
D_{50}	median grain size by weight (m)
D_{65}	grain size for which 65% of the distribution is finer by weight (m)
D_{50f}	median fall velocity grain size (m)
f_c	friction coefficient in Engelund and Hansen bedload equation
$f(x)$	Normal probability density function
g	gravitational acceleration (9.81 $m.s^{-2}$)
ΔH	tidal range (m)
K	coefficient in Sternberg's bedload equation, Engel and Lau Coefficient
P	porosity
q_m	bedload mass transport rate ($kg.m^{-1}.s^{-1}$)
q_{ss}	suspended load mass transport rate ($kg.m^{-1}.s^{-1}$)
R	correction coefficient for sediment threshold speed determination
s	standard deviation
S	sea surface slope
$T_{\frac{1}{2}}$	half tidal cycle period
u	water velocity ($m.s^{-1}$)
u_1	water velocity at 1m above the bed ($m.s^{-1}$)
$\bar{u}$	mean vertically-averaged velocity ($m.s^{-1}$)

u_* (total) friction velocity ($m.s^{-1}$)
u_*' skin friction velocity ($m.s^{-1}$)
u_*'' form drag friction velocity ($m.s^{-1}$)
u_{1cr} sediment threshold velocity at 1m above the bed ($m.s^{-1}$)
u_{cr} sediment threshold mean vertically-averaged velocity ($m.s^{-1}$)
u_{*cr} sediment threshold friction velocity ($m.s^{-1}$)
u_{*cr}' sediment threshold skin friction velocity ($m.s^{-1}$)
u_{*cr}'' sediment threshold form drag friction velocity ($m.s^{-1}$)
U_w megaripple advance rate ($m.s^{-1}$)
x power of fall velocity in Rouse number modification
y distance above sea bed (m)
z distance above the sea bed, Rouse Number
z_o roughness length (m)
β constant for modification of Rouse Number
Δ bedform height (m)
κ von Karman's Universal constant (0.4)
Λ bedform wavelength (m)
ρ fluid density (1025 $kg.m^{-3}$)
ρ_s sediment density (2650 $kg.m^{-3}$)
τ_* dimensionless bed shear stress
θ Shield's Entrainment Function
θ'_{cr} critical (skin friction) Shield's Entrainment Function
ϕ dimensionless sediment transport rate
ϕ_m total load mass transport rate ($kg.m^{-1}.s^{-1}$)
(') association with skin friction
(") association with form drag

7.3 Experimental sites

Six sediment transport experiments were conducted over megaripples, identified previously during the side-scan sonar survey (Chapter 2), at positions shown in Figure 5.25. For their proximity to the timber port, S1 to S4 examined the sub-tidal spit extending from the south-east end of Snake Bank and passing across the northern side of the proposed dredge zone. S5 was in Shoal Bay on megaripples encompassing the 5m contour on the southern side of the channel and S6 was on Marsden Point but due to wave interference, the data was rejected.

7.4 Experimental techniques

Bedload movement was obtained with two sediment traps of the VUV type (Graf, 1971, p. 363) and emptied every 1-2 hours. The trap (Figure 7.1) was designed to be lowered by rope from the surface but experimentation revealed that a diver was needed to correct trap alignment at the bed and to stabilise it in high flow on the way to the surface. Moreover, to avoid sand being lost when the trap was raised, a door was fitted to the mouth which could be closed by a diver before retrieval. The traps have a rectangular mouth of size 0.26m wide by 0.12m high.

Of the six sediment transport experiments, three days of trapped bedload data were rejected (i) for wave interference; or (ii) because only one bedload trap was serviced. On the remaining three days (S3, S4 and S5), two traps were deployed simultaneously. Experimental accuracy increases with the number of traps deployed. However, diver bottom time established

a limit of two. The sediment caught was dried, weighed and then averaged. Accepting Graf's (1971) assessment of work by Novak that VUV traps are 70% efficient, the trapped weights were scaled accordingly to compensate for systematic deficiencies. More than 100% weight variations between the two traps occurred which is due, at least in part, to the position of the trap on the bedform. A trap placed in the trough of an approaching megaripple would not be expected to catch large quantities of sediment which is moving opposite to the general flow in a back eddy.

To verify the trap data, measurements of megaripple advance were made. During slack water, steel stakes were hammered by divers into the crests of 6 consecutive megaripples and crest advance was measured at the next two quiescent periods and during the tide. Underwater photographs of the bedforms and direct diver measurements of the megaripples provided bedform geometry, i.e. bedform heights and wavelengths and the lengths of the slip and stoss faces.

Suspended load was sampled regularly at 0.1, 0.3 and 1.0m above the bed by pumping samples to the surface from nozzles mounted on a steel tripod frame. Samples of about 2 litres were passed through a 70 micron filter. Trapped material was washed to remove all fine matter which was not of interest to this study. The sieving was performed by the New Zealand Refining Company Laboratory staff at Marsden Point under the direction of Mr Alan Stringer.

An underwater video camera mounted on a steel frame (Figures 7.3 and 7.4) on the sea floor, monitored the sediment threshold, ripple advance rates and the behaviour of the sediment under tidal flow. Of the six experiments, the video was available for use during the first two only (S1 and S2).

During these, the Royal New Zealand Navy Diving Tender "Manawanui" (Figures 7.4 and 7.5) served as a work base for the divers and the underwater video monitors. The vessel was large (26m long) with the draught being a significant percentage of the flow depth. To prevent flow interference by the larger ship, the shallower draught NHB oil spill vessel "Maunu" (Figure 7.6), was anchored beside the sea bed experimental area instead. Maunu, which had previously been equipped to support the current meter and the suspended sampler, served as the major operations vessel for the measurement of all parameters including the water speeds.

7.5 Bed photographs

Ninety underwater photographs were shot of the megaripples during the S-series of experiments. A selection of these at three sites (S3, S4 and S5) is presented in Figures 7.7 to 7.9 and a megaripple field near the entrance is shown in Figure 7.10. Each were taken against a steel backboard with a 0.02m square grid for scale.

S3 has relatively high shell content (Figure 7.7). The shell is mostly broken due to sand abrasion and collisions during its own transport. The shell is clean, without algal growth, thus signifying bed mobility. Figure 7.7 shows a megaripple crest, slipface and trough. The crest is rippled with streaks of shell lining the ripple troughs. Ripple wavelengths are about 0.10m and heights are 0.01-0.02m. The slipface of the megaripple is coated with broken shell fragments. Because of the shells' different hydraulic behaviour, the smaller sand grains fall between the shells as material slides down the slipface. After examination of other photographs of the same feature, the height of the megaripple shown in Figure 7.7 was found to be 0.26m with a slipface length of 0.56m. The angle of repose was determined to be 30^{o} which is within the range of expected angles after sediment transport of $30\text{-}35^{o}$ (Allen, 1982). In the megaripple trough, the shell concentration increases. Note the seemingly random orientation and packing of the shells in the left-most ripple trough with both convex up and down placement. Flow during the previous half cycle was directed from right to left of the photographs.

In Figure 7.8, S4 exhibits similar characteristics but the shell content is lower. In the top photographs, the ripples are aligned perpendicular to the larger megaripple, indicating a transverse flow along the megaripple trough. At the base of the megaripple slipface, superficial broken shell is noted. The bedforms shown have respective heights of 0.14 and 0.22m with angles of repose of 28^{o} to 32^{o}. Ripples on the bedform crest have heights of 0.045-0.055m and are 0.28m between crests. This corresponds with Yalin's (1977) definition of a ripple as having a wavelength approximately 1000 times the grain size which is $D_{50} = 0.00032$m at this site (Table 7.2).

The megaripple troughs (Figure 7.8) are again characterised by greater shell concentrations. The shell placement is apparently random with separated pelecypods lying with a wide range of orientations. Flow was from right to left, but flow separation at the crests establishes reverse eddies, and the shell orientation may depend on these oppositely-directed velocities. The clustering in the ripple troughs suggests that the shells could also have been placed here during the passage of a larger bedform such as megaripple. The ripples have wavelengths of 0.20-0.22m and heights of 0.015m.

At S5 (Figure 7.9), the finer sediments are identified by steeper angles of repose of $32\text{-}33^{o}$. The megaripples are approximately 0.16m high and the stakes near the crests were placed earlier for measurement of megaripple advance.

Relevant measurements taken from the 90 bedform photographs were summarised in Table 7.1. There appears to be a reduction in megaripple height at peak flow at S3. This may relate to the shape of the tidal friction velocity curve which exhibited an early peak flow followed by a

trough during peak flow (Figure 6.20).

7.6 Grain sizes

Sediment samples were taken by divers off the troughs and crests of the bedforms during each experiment. These, and a sample from the bedload traps serviced after peak flow, were sieved at 0.25 phi intervals by University of Waikato technical staff. The carbonates are often dissolved before a sieve analysis (e.g. Millar, 1980). However, as it has a significant influence on the sediment transport fluxes, the shell content was retained. Grain size analyses were performed with the author's "Grain Size Analysis by the Moments Method" computer program package. Results are in Tables 7.2 and 7.3 and Figures 7.11 to 7.14. The samples from the sediment traps were felt to be most representative of the material in motion, and a more detailed breakdown of their grain sizes is given in Table 7.2. Grain sizes in the traps varied in the range $0.00024 < D_{50}(m) < 0.00034$ which correlates best with megaripple crest sediments (Table 7.3).

As noted with the underwater photographs, the troughs are generally coarser and less well sorted than the crests. S3 is an exception to this, exhibiting a more-peaked, finer sediment distribution in the trough. An explanation for this may lie with the analysis method. The change of variable from mm to phi units has the effect of reducing the weighting of the coarse tail of the distribution. To examine this, the grain size analysis by the Moments Method was performed again but on an arithmetic (mm) scale instead of phi units. The trough then proved to be coarser than the crest and less well sorted. The differences are small but in this particular case, the phi logarithmic scale evidently does not represent the physical processes as well as the arithmetic scale. The percentile sizes do not depend as strongly on the distribution scale and these are preferred for application to sediment transport calculations.

S5 (Figure 7.14) exhibits a high shell content in the troughs where the sample was taken. This inhibited scour at the point of flow reattachment and the megaripples had an unusual profile of sharp ridges over a nearly-planar bed. Further into the megaripple field where the sediment transport experiment was conducted, the shell content dropped and the bedform cross-sectional profile became more "normal". The sediment caught in the trap at S5 was fine and mostly shell free with the maximum grain size being 2mm.

In summary, even though there is shell present at the sites, the grain sizes in the sediment traps are representative of sandy sediments. Accordingly, the influence of the shell content at these sites is expected to be minimal.

7.7 Underwater video observations during sediment transport

Several hours of video tapes showing the seabed during all stages of the tide are summarised here. A previous description of the processes occurring during accelerating tidal flow and observed on underwater video is given by Dyer (1980) and many of the same observations were made at Whangarei.

Initially, small shell fragments and organic debris begin intermittent random movements. Dyer suggests that this may indicate the onset of flow separation in the ripple lee. The sandy sediment first moves in bursts which increase in frequency as the water velocity increases. The duration of these bursts is about 1-2 seconds, supported by Dyer's 1 second estimate. At higher flow speeds a "suspension carpet" of sand grains evolves and thickens with each turbulent increase in speed.

The behaviour of flow over ripples is shown on a schematic diagram in Figure 7.15. Flow reattachment is evidenced by high levels of turbulence with grains whisking suddenly both down and upstream from the reattachment point which also oscillates horizontally. Flow is directed downwards and tends to scour the bed. The bedforms advance with a positive and a negative sediment flux, and thus the crest movement does not measure the nett sediment transport rate. This was noted by Engel and Lau (1980) who suggest a correction which depends on the bedform geometry and the resulting position of flow reattachment.

Measurements of turbulence and temporal mean shear on the ripple form by Raudkivi (1963) revealed that, at the point of flow reattachment, the shear was zero and increased to a maximum over the crest, while turbulence intensity was a maximum at the reattachment point and decreased downstream. Raudkivi (1976) suggests that sediment is moved by the combined action of the turbulence agitation and mean shear stress. He states that turbulence affects entrainment as: (i) the particle may be moved by the drag exerted by a passing eddy; (ii) the eddy may lower the local pressure and the particle may be ejected from the bed by hydrostatic pressure; and (iii) entrained particles may be put into suspension rather than moved along the bed. The effects of these mechanisms were evident at Whangarei.

At T= 191s in Figure 7.15, a second ripple was formed at the point of flow reattachment and the birth of the bedform was witnessed. This suggests that flow separation is necessary for the generation and maintenance of these features. Both birth and death processes were evident (Allen, 1982, p. 279), such that individuals died away and their place was taken by similar but new forms. At T= 220s, both ripples were suddenly buried under a suspension carpet of sand arriving with a turbulent burst.

Flow was directed along the troughs with fast helical vortex motion

being traced by the fine shell and sand particles. Due to crest curvature, the inclincation of the flow to the crest normal was mostly non-parallel, and the cross-current component establishes the vortices noted. This flow was very fast, being comparable with the speeds across the crest. Allen (1968) reported the presence of these vortices over plaster cast models of rippled forms.

The ripples advanced at rates of a few centimetres per minute (which is much faster than speeds reported by Dyer (1980) of a few centimetres per hour. The wavelengths were about 0.16m with heights of 0.02, which gives a ripple index of 0.125 and is not atypical.

The behaviour of a pipi shell (Amphidesma australe) was watched during the accelerating tide at S1. The disarticulated bivalve was gradually exposed by erosive currents over a period of 25 minutes. After which, it oriented itself with the beak pointing upstream (Nagle, 1967, reported in Allen, 1982). In currents of 0.58 $m.s^{-1}$ at 1m the exposed shell zig-zagged backwards as oscillations of the flow direction attacked the shell of its flatter sides. A stubby gastropod, the mudflat top shell (Zediloma subrostrata), sat upright and induced erosion at its leading edge and deposition in its lee. It moved when the currents reached about 0.6 $m.s^{-1}$. At speeds of 0.5 $m.s^{-1}$ smaller whole pelecypod shells were sighted moving across the crests and into the troughs, where they were buried by the advancing ripple. The relative stability of the pelecypods and gastropods is not known.

The shells will not move unless they are exposed by erosive losses of surrounding sand. Thus, if the transport rate is neutral with sand losses equalling gains, the shells, even when only partially buried, will be unmoved by high current flows. After total exposure, pelecypod halves are likely to "skate" across the mobile, sandy surface with the convex side up, almost in hovercraft fashion, reflecting the Bernoulli pressure differential on the top and under surfaces of the shell. On reaching the bedform troughs, they are often buried by the advancing bedform. The gastropods being more spherical, tend to tumble across the bed.

7.8 Sand/shell interaction

The video and diver observations revealed two important processes on a bed composed of sandy and shelly sediments.

(i) if the sand is more abundant than the shell and megaripples are formed, then the shell tends to accumulate in the trough and is not easily eroded again. By this process the advancing megaripple lays a continuous bed of shell in front of itself. Reversing flow continues to separate the shell and sand components until the fine, shell-free sand moves back and forth over the shell bed. Figure 7.10 shows the sea bed at a site of this type near the entrance throat.

(ii) where the shell is more abundant and the formation of bedforms is retarded, the smaller sand grains fall between the shells which leaves a shell layer covering the sandy sediment. Any vibration of the shell by the tidal flows assists this separating process. Thus, sediment settling out of suspension is prevented from eroding again by a protective shell layer.

In Chapter 5, the stability of Whangarei Harbour is shown to be strongly dependent on the balance of shelly and sandy material. Furthermore, if the bed is thought of as being composed of pockets of erodible sandy sediment alternating with non-erodible shells then the volumes of sediment transported under tidal flow are related to the percentage of the total area covered by sand. This concept is expanded upon numerically with the sediment transport model in Chapter 11.

7.9 Sediment threshold

The sediment threshold over bed roughness elements decreases with increasing roughness length (Heathershaw, 1981). However a convenient method to accurately predict this change has not been presented to the knowledge of this author. Under steady uni-directional flow over a plane bed of non-cohesive uniform grains, prediction of threshold has been made possible after considerable laboratory data collection e.g. Miller et. al. (1977). However, as stated by Dyer (1980, p. 198), "It is not possible to use laboratory curves for threshold of movement and sediment transport formulae in areas of rippled sand without considerable care". This is due to the complications introduced by the distribution of grain sizes, turbulent fluctuations of flow, as well as the effects of bed geometry in the estuarine environment. Accordingly, the concept of threshold is often greatly simplified and the speeds are approximated for the sake of application to field cases.

In the following sections the effect of form drag over megaripples on the sediment threshold speed in Whangarei Harbour is examined. A formula is derived which accounts for the form drag and skin friction components and the predictions are compared with measured thresholds.

7.9.1 Threshold definition

In turbulent tidal flow, the experimental technique of measuring water speeds over an extended time interval to find the average in the period, necessitates the introduction of a corresponding threshold motion. Maximum instantaneous flow speeds, which may move sediment, can be much larger than the mean time-averaged velocity. Heathershaw and Simpson (1978) found that the horizontal water speeds oscillate about the mean with pseudo-Normal distribution. Thus, within any averaged velocity period at speeds near threshold, there will be bursts of quiet and activity on the bed. As it is often not possible to measure instantaneous speeds with many current meters,

so it is not possible to utilise an instantaneous threshold speed. A convenient threshold thus needs to be defined and is introduced here.

The Normal distribution is symetrical and thus the mean is the speed which is exceeded 50% of the time. If the mean lies at the instantaneous sediment threshold speed then the bed should be in motion 50% of the time (depending to some degree on the periods of the turbulent fluctuations and the correlation of successive bed observations). The sediment threshold may then be defined as the average speed measured over a period when the bed was in motion 50% of the time. This simple, but well-founded definition can be applied to all averaged velocities.

Many other thresholds have been defined all of which were not applicable to the experiments at Whangarei. Grass (1970) defined a threshold considering the overlap of the critical stress distribution associated with the placement of the grains on the bed and with the distribution of shear stresses due to fluid flow. This sophisticated procedure accounts for both the turbulent variations and the grain size distribution but could not be applied to the field measurements at Whangarei. Sternberg (1971) preferred the simpler definition that threshold was a state of "general sediment movement". Dyer (1980) interpreted the threshold as coinciding with the intercept of the regression line of sediment transport against water speed or friction velocity. This is a useful procedure and by its nature the turbulent fluctuations are smoothed out.

7.9.2 Plane bed

Miller et. al. (1977) found that over a plane bed of quartz-density uniform grains, the critical speed at 1m above the bed in water of 20°C is given by,

$$u_{1cr} = 122.6\ D^{0.29} \qquad 7.1$$

where D has cm units and the speed is in $cm.s^{-1}$. Insertion of a typical grain size of 0.0003m gives $u_{1cr} = 0.44\ m.s^{-1}$. This is an upper limit as it does not include the effects of form drag over bed features.

The Yalin (1977) modified Shields threshold curve plots the Shields Entrainment Function against a non-dimensional grain size called the Yalin Parameter, and critical threshold friction velocity for D = 0.0003m is $u'_{*cr} = 0.014\ m.s^{-1}$ at 20°C. This is also applicable to a plane bed and is thus a lower limit because it does not account for the form drag.

7.9.3 Rippled and megarippled beds

Heathershaw (1981) calculated critical velocities at 1m using the Karman Prandtl equation and the Yalin threshold curve. For a roughness length of 0.001m and a grain size of 0.0002m, the threshold was found to be $u_{1cr} = 0.23\ m.s^{-1}$ which is much less than a corresponding plane bed value.

However, Heathershaw included the form drag in the roughness length but incorrectly neglected form drag in the friction velocity by applying the Yalin threshold curve for determination of critical friction velocity which accounts for skin friction only. An improved method was derived by this writer to determine the critical velocity which utilises results in Chapter 6.

Firstly, assume that for a given grain size, the critical skin friction velocity u'_{*cr}, can be found from the Yalin threshold curve. The form drag friction velocity can be obtained from the Vanoni and Hwang equation (6.67) over 2-dimensional roughness elements.

$$\frac{c''}{\sqrt{8}} = 3.3 \log_{10} \left(\frac{d\Lambda}{\Delta^2}\right) - 2.3 \qquad 7.2$$

where,

$$c'' = \bar{u}/u''_* \quad \text{or} \quad \bar{u} = c''\ u_*'' \qquad 7.3$$

The analysis by Lettau (1969) referenced in Dyer (1980) and somewhat validated at Whangarei, revealed that over 2-dimensional bedforms,

$$z_o = \frac{\Delta^2}{2\Lambda} \qquad 7.4$$

Thus, insertion into the Vanoni and Hwang equation produces,

$$\frac{c''}{\sqrt{8}} = 3.3 \log_{10} \left(\frac{d}{2z_o}\right) - 2.3 \qquad 7.5$$

The mean speed in the water column is related to u_* and z_o by the Karman Prandtl equation as,

$$\bar{u} = u_*\ 5.75 \log_{10} \left(\frac{0.37d}{z_o}\right) \qquad 7.6$$

Then with equation (7.3),

$$u''_* = \frac{u_*}{c''} 5.75 \log_{10} \left(\frac{0.37d}{z_o}\right) \qquad 7.7$$

The form drag and skin friction velocities can be summed to give the total friction velocity as,

$$u_*^2 = u_*'^2 + u_*''^2 \qquad 7.8$$

By insertion of equation (7.7) and after manipulation, at threshold,

$$u_{*cr} = \frac{u'_{*cr}}{\left\{1 - \left[\frac{5.75}{c''} \log_{10} \left(\frac{0.37d}{z_o}\right)\right]^2\right\}^{1/2}} \qquad 7.9$$

with c" given by equation (7.5) above.

The friction velocity is related to u_1 at threshold as,

$$u_{1cr} = 5.75\ u_{*cr} \log_{10} (1/z_o) \qquad 7.10$$

In Table 7.4, the critical friction velocities and speeds at 1m for 5m

and 10m depths were compiled for several selected grain sizes and roughness lengths using equations (7.9) and (7.10). For rough beds composed of fine material, the critical velocity at 1m is only 0.22 $m.s^{-1}$ (Table 7.4), compared with the plane bed value of Miller et. al. (1977) of 0.32 $m.s^{-1}$. However, critical u_1 is greater using the method proposed here than obtained by Heathershaw (1981). Notably, the relationship is not very sensitive to depth and varies little for the typical depths of 5m to 10m.

7.9.4 Video examination of the sea bed for verification of the theory

The sea bed was viewed during the initiation and cessation of sediment transport with an underwater video camera to find the threshold speeds. The films were analysed in a fashion similar to that undertaken by Dyer (1980). At 5-second intervals, a yes/no decision was made as to whether sandy sediment was moving anywhere in the frame of the camera (approximately a bed area of 0.12m by 0.16m). From 12 consecutive answers, a probability of movement was calculated for the one-minute period. Grain sizes were obtained from the sieved samples taken from the bedload traps (Table 7.2).

7.9.4.1 *Calculated S-series thresholds*

Threshold speeds calculated with equations (7.9) and (7.10) for the S-series sites are compiled in Table 7.5. Critical skin friction was found from the Yalin Threshold curve using D_{50}, sediment density of 2650 $kg.m^{-3}$, and a fluid viscosity dependent on the water temperature (Table 7.2). The roughness length was obtained by three methods i.e. from the bedform geometry (Table 7.2) with equation (7.4) and from the median and mean values both obtained by curve fitting the velocity profiles (Table 6.2). Critical velocity at 1m was determined separately with the three z_o values and then averaged to get an overall mean (Table 7.5). In all cases, the threshold corresponding with z_o from the bedforms, lies between the two estimates using mean and median z_o, and the latter is always the greater.

7.9.4.2 *Measured thresholds at S1 and S2*

Figures 7.16 to 7.18 show accelerating and decelerating cases of the probability of movement together with the speeds at 1m. Initially, below 0.25 $m.s^{-1}$ no movement was seen. Then, as previously reported by Dyer (1980), shell fragments move randomly, appearing to wobble in the flow. These fragments were not considered for the threshold which corresponds with the initial motion of sand grains only.

Initial motion at S1 (Figure 7.16) - such that sand grains were moving at least 50% of the time - occurred when the 1m speed was between 0.3 and 0.35 $m.s^{-1}$, on the accelerating phase. During deceleration (Figure 7.17), the probability first falls below 0.5 when the speed is 0.34 $m.s^{-1}$, but rises above 0.5 again until the speed is less than or equal to 0.30 $m.s^{-1}$.

Table 7.5 shows the predicted threshold speed at this site as averaging 0.35 $m.s^{-1}$, spanning the range from 0.32 $m.s^{-1}$ to 0.39 $m.s^{-1}$, and with z_o from the bedform geometry, $u_{1cr} = 0.33$ $m.s^{-1}$. The latter closely approximates the measured threshold.

In Figure 7.18, at Site S2, speeds were not measured at the time of threshold. However, interpolation of the trend of the measured data, gives a threshold speed $u_{1cr} = 0.36$ $m.s^{-1}$. No bedform geometry was measured at S2 but using the mean and median roughness lengths obtained from the velocity profiles, the critical speed is predicted to lie in the range of 0.32-0.45 $m.s^{-1}$ with a mean of 0.39 $m.s^{-1}$. Although this is a wide range, and thus not conclusive, the predicted threshold does lie within it. Moreover, the higher predicted threshold at this site compared with S1 is reflected by a higher measured range. Accordingly, the data supports the theoretical development but best agreement was obtained when detailed bed geometry measurements were available.

Beyond the threshold speed, the bed still moves intermittently when speeds lie between 0.3 and 0.45 $m.s^{-1}$. Constant bed movement occurs when the speed exceeds 0.45 - 0.50 $m.s^{-1}$ and this is graphically shown in Figure 7.19, during a period of steady-state flow at S1. The speed of $u_1 = 0.5$ $m.s^{-1}$ was found to correspond with peak friction velocity over the tidal cycle (Figure 6.20). The evidence here supports the suggestion that this is related to a flattening of the bed under the heavy sediment loads at these speeds as already noted by the measurements of bedform heights from underwater photographs at S3.

7.9.5 Examination of Sternberg's data for verification of the theory

Sternberg (1971) published sediment thresholds obtained with an underwater camera together with the bed geometry and grain sizes. A comparison of the previous theory with Sternberg's results was therefore possible. The threshold speeds were calculated using equations (7.9) and (7.10) with critical skin friction velocity taken from the Yalin Threshold Curve. The prediction of threshold (Table 7.6) falls neatly between the two Sternberg estimates of threshold firstly defined as a "state of general sediment movement" and secondly obtained by extrapolation of ripple migration rates to zero. This offers good independent confirmation of the new procedure.

7.9.6 Application of the method

The equation relating the total friction velocity to the skin friction velocity is a function of the non-dimensional ratio of flow depth over the roughness length only. Thus, Figure 7.20 was prepared to enable simpler application of the method. Entry with the ratio d/z_o, gives the correction R, to be applied where,

$$u_{*cr} = u'_{*cr} \cdot R \qquad 7.11$$

and u'_{*cr} is obtained from the Yalin Threshold Curve. The critical 1m speed is found with u_{*cr} and z_o in equation (7.10).

7.9.7 General harbour threshold speeds

In Chapter 6, mean and median z_o for Whangarei Harbour (Table 6.3) were found to be 0.0087m and 0.0007m respectively. In view of this, a general harbour sediment threshold speed was selected to be $u_{1cr} = 0.33$ m.s^{-1} which corresponds with a roughness length lying between the above values and a grain size of 0.0003m (Table 7.4). For the typical 5-10m depths where most active sediment transport occurs in Whangarei Harbour, the threshold mean vertically-averaged speed was selected to be $\bar{u}_{cr} = 0.40$ m.s^{-1}. This has a depth dependence and is not determined by z_o and D_{50} only, and may be a poor approximation in some locations.

7.9.8 The Karman Prandtl equation

An equation to derive the skin friction velocity using the Karman Prandtl equation was introduced in Chapter 6,

$$u_1 = 5.75\ u'_*\ \log_{10}\ (\frac{30.2}{k}) \qquad 7.12$$

where $k = 2D_{65}$. Taking $k = 2.5D_{50}$ (which is a small change depending on the shape of the grain size distribution), the critical 1m speed can be found by insertion of the skin friction velocity from the Yalin curve. Compiled in Table 7.4, this relatively simple formula approximates the theoretical threshold in the middle of the range of given roughness lengths, but falls off slowly as the grain size increases. Agreement for typical Whangarei grain sizes ($D = 0.0003$m) supports bedload transport results presented later.

7.9.9 Diver observations of sediment threshold

At 2 sites in the harbour, divers observed the sea bed at near threshold speeds. The experiments were difficult to conduct due to problems of decompression time at one of the deeper sites and due to the difficulties of recording observations quickly and often. The video film, which can be viewed in more congenial conditions, is a more accurate procedure to ascertain sediment threshold speeds. Nevertheless, some useful information was gleaned from these experiments.

7.9.9.1 *Procedure*

Divers were stationed at the sea bed on lines attached to the boat moorings. Observations were recorded on a slate by one diver and independently, a second diver relayed predetermined signals by pulling a rope from the attending vessel, "Maunu". Their independence allowed a cross-check of the results. Current speeds were measured with the Braystoke meter almost

continuously, and recorded every 50 seconds.

7.9.9.2 *Site T1, Figure 5.25, 11-Mar-82*

Located in 18m of water between the NHB and NZRC wharves, the bed at this site was not bedformed and was composed of an apparently non-cohesive sand and shell conglomeration, with shell covering approximately 30% of the surface area. The shells were partially-buried, clean, loose to the touch and generally whole, with typical diameters of 0.012m. A hole dug into the bed by divers about 0.20m deep revealed an internal setting similar to the surface with shell embedded in what appeared to be a random fashion. The lack of bedforms in this terrain suggests that the shell content was sufficient to inhibit their formation.

The site was situated adjacent to an up-harbour, sandy, bedformed region. After the ebb tide, sand was swept down to cover the experimental site with a rippled bed. Apparently the previous flood-tide has transported this sand up-harbour exposing the more consolidated shell and sand mixture beneath. The edge of the loose sand belt evidently oscillates with a tidal cycle period.

Figure 7.21 shows the velocity curve measured at 1m above the bed with a summary of relevant diver observations. Notably, the two divers report a different threshold. It is not possible to state when the bottom was moving 50% of the time, but the sandy sediment threshold evidently lies between 0.42 and 0.48 $m.s^{-1}$ and the bed was definitely moving often at the latter speed. No grain size analysis was performed, but for a typical size of 0.0003m over a plane bed, the Miller et. al. plane bed relationship predicts the threshold speed of 0.44 $m.s^{-1}$. From Table 7.4 the threshold speed for $z_o = 0.0001m$, (no bedforms were present) is 0.46 $m.s^{-1}$. Both of these threshold estimates fall within the range of the field measurements.

Clearly, as also illustrated by the threshold equations just derived, the threshold speed is grossly affected by the bed geometry with the threshold at this site being much greater than over the megaripples observed with the video. Not including the reduction of sea bed surface area covered by erodible material, the presence of the shell, by inhibiting the bedform formation, increases the threshold speed. This should seemingly cause a reduction of the sediment transport rate. However, this notion is more complex. With a reduction in roughness length, the energy lost by the flow will decrease and thus, for a given mean speed and flow depth, u_1 increases with decreasing z_o. In other words, the flattening of the bed may raise the threshold but the 1m speed rises as well.

In estuaries, the mean speed is governed mostly by the accelerations due to sea slope balanced against frictional energy losses. At peak flow, an equilibrium is attained (Chapter 9) and,

$$\frac{\partial u}{\partial t} = 0 \qquad 7.13$$

then,

$$S \approx \frac{\bar{u}^2}{C^2 d} \qquad 7.14$$

where S is the sea slope and C is Chezy's C. For a given tidal condition, where the sea slope depends on tidal range, a local increase in C, i.e. lower frictional retardation, will result in higher water speeds. As noted by Dyer (1980, p. 198), "The tidal dynamics are in balance with the fact that the sediment is moving and that bedforms are present ... Thus, should the bed become stable" - such as that which results from the presence of the shell - " the local current pattern would change".

In summary, a reduction in z_o should increase the mean speed as well as allowing u_1 to increase. Among other important implications, this means that velocity excess, and not the velocity alone should inherently reflect the sediment transport rates.

7.9.9.3 *Site T2, Figure 5.25, 5-Apr-82*

Unfortunately, field failure of the current meter sounder meant that no distances of the velocity measurements above the sea floor could be obtained, so a threshold speed cannot be reported here. However, some interesting general observations were made:

(i) this site near the entrance throat, changed from megarippled, to a sand/shell mixture, to a total, aged shell cover in a cross-channel direction over the very small distance of approximately 20m. Notably, Dahm (1983) observed similar sharp boundaries between bed types in Tauranga Harbour. Underwater photographs (Figure 7.22) show the sand/shell and aged shell beds. At the former site the shell covers about 70% of the sea floor. Most of it is broken compared with the total cover composed mainly of whole shells, no more than 5m away. The distinct boundary between these two types can be seen in the bottom photograph (Figure 7.22). The boundary locates the northern edge of the active sub-tidal spit running from Snake Bank (Chapter 2).

(ii) divers worked separately, and the diver on the megarippled floor reported initial sediment movement before the diver on the flat bed of sand/shell, as would be expected from the previous analysis due to the increase in z_o over bedforms.

(iii) maximum speed recorded was 0.9 $m.s^{-1}$ at approximately 6m above the bed and the shell region was completely stable. With the Karman Prandtl equation this converts to $u_1 = 0.75$ for $z_o = 0.0001m$ and is much less than a speed of 0.9 $m.s^{-1}$ at 1m above the bed recorded over stable shell on Mair Bank (Chapter 5). When picked up by a diver, the shell fell to the sea bed

and oscillated in the flow until a stable orientation was obtained which seemed to depend, in part, on the orientation relative to its neighbours. As can be seen in Figure 7.22, the shells are both concave and convex up.

(iv) at peak flow a large pelecypod half (approximately 0.04m across) was placed concave up on the sandy bed. The shell was stable in the midst of rapid sandy sediment transport. By flow convergence, erosion around its edges allowed the shell to become partially buried and more stable.

7.9.10 Shell threshold

The shells are undoubtedly less prone to erosion than the sand grains. However, their threshold speed depends on a great variety of shell shapes and sizes and a separate threshold is required for each of these. Furthermore, their stability depends on clast packing, which increases the permutations considerably. The experiments at Whangarei revealed that the shell beds did not move at high speeds, but little can be said about the exact bed threshold speed.

Mehta et. al. (1980) measured the fall velocity of some disarticulated common bivalve shells. Chione cancellata (the Chione stuchburyi is found commonly in Whangarei Harbour) was found to have a mean fall velocity of 0.24 $m.s^{-1}$. This compares with 0.032 $m.s^{-1}$ for sand with D = 0.0003m (reported in Raudkivi, 1976). The shell stability is thus ensured by its considerably larger fall velocity. Mehta and Christensen (1980) used divers to determine the shell threshold in natural harbour entrance gorges. They arrived at a friction velocity of 0.046 $m.s^{-1}$ for the initiation of movement of shells with diameters greater than approximately 0.005m. With a shell density of 2790 $kg.m^{-3}$ (Mehta et. al. 1980), and for D = 0.005m, the Yalin threshold curve predicts a critical friction velocity of $u'_{*cr} = 0.059$ $m.s^{-1}$ for a plane bed. This is 28% greater than the critical value of Mehta and Christensen.

In Chapter 6, the median roughness length over shell beds for Whangarei Harbour was reported as 0.00045m (Table 6.3). Using the Karman-Prandtl equation, critical speeds are then $u_{1cr} = 1.18$ and 0.92 $m.s^{-1}$ for $u'_{*cr} = 0.059$ and 0.046 $m.s^{-1}$ respectively. Moreover, $\bar{u}_{cr} = 1.51$ and 1.18 $m.s^{-1}$ for the same u'_{*cr} values and a mid-channel depth of 26m. The shell clearly has a very high threshold velocity. Although no bed observations were made, a speed of 1.44 $m.s^{-1}$ was measured at mid-depth in the entrance gorge on 11-Nov-81 during a Spring tide with d = 26m. This speed lies between the two estimates of critical velocity and thus, the bed is only disturbed during high Spring tides, if at all. Speeds over shell on Mair Bank of 0.9 $m.s^{-1}$ were measured and are approximately equal to the lower estimate of the critical 1m velocity of 0.92 $m.s^{-1}$. However, turbulent velocity fluctuations should exceed the measured mean 1m speed and if the lower threshold is

correct, there should have been some spasmodic movement of the bed which was not noted.

The mean of the two threshold estimates is 1.05 $m.s^{-1}$ at 1m from the bed, with the qualification that bed roughness variations due to shell type and packing and biological cohesion will affect this figure. Nevertheless, this may be the best estimate of the threshold over shell in Whangarei Harbour. The fastest 1m speed measured in the harbour was 0.99 $m.s^{-1}$ which, after correction for tidal range, corresponded with $u_{1max} = 1.13$ $m.s^{-1}$ for $\Delta H = 2.2$m (Table 5.12). This suggests that the shell exhibits general stability throughout the harbour, but this is again qualified as above. Sea floor observations and measurements over the shell beds at maximum Spring tides need to be performed to verify the analysis. It is clear, nevertheless, that the shells are highly stable.

7.10 Bedload and total load transport formulations

Many bedload equations have been proposed in the literature, most of which were derived in uni-directional currents. Many of these were examined by this author but ultimately, four were chosen for careful scrutiny and comparison with the measured bedload and suspended load rates at Whangarei.

7.10.1 Sternberg's modified Bagnold equation

Sternberg (1972) modified the Bagnold (1963) bedload equation which relates the bedload transport rate to the stream power. The latter is the product of the mean velocity and the bed shear stress. Sternberg arrived at the equation,

$$q_m = \frac{K\rho\rho_s u_*^3}{g(\rho_s - \rho)} \qquad 7.15$$

where K is an empirical constant to be found, and q_m is the dry mass transport rate per unit width of bed with units of $kg.m^{-1}.s^{-1}$.

By examination of the Guy et. al. (1966) flume results, in conjunction with some field data, K was found to be a logarithmic function of the excess shear stress and fitted to a power equation by this writer,

$$K = 10^{0.299\tau_*^{-2.3}} \qquad 7.16$$

for D = 0.00028m. The excess shear stress is,

$$\tau_* = \frac{u_*^2}{u_{*cr}^2} - 1 \qquad 7.17$$

Bagnold (1963) was reported by Sternberg as suggesting that K reaches a constant value which is a function of the relative roughness d/D. Sternberg's equation was incorporated into the author's YALIN series of computer programs.

7.10.2 Gadd et. al.'s modified Bagnold equation

Gadd et. al. (1978) modified the same equation. Introducing the Drag Coefficient to relate u_* to u_1 in the same manner as suggested by Sternberg, they produced,

$$q_m = \frac{\rho_s \rho}{g(\rho_s - \rho)} K C_1^{3/2} u_1^3 \qquad 7.18$$

After comparison with the Guy et. al. (1966) data, discrepencies led them to introduce an excess velocity, and with all the other coefficients bundled together into one constant (which should be at least a function of the grain size) they arrived at,

$$q_m = \beta (u_1 - u_{1cr})^3 \qquad 7.19$$

From the Guy et. al. flume data they found $\beta = 7.22$ kg.m^{-4}.s^2. for $D_{50} = 0.00019$m and $\beta = 1.73$ kg.m^{-4}.s^2 for $D_{50} = 0.00045$m. This equation was applied to a field case by the authors.

7.10.3 Engelund and Hansen Total Load formulation

The Engelund and Hansen (1967) equation arises out of the relationship between the total friction velocity and the effective component (eqn. 6.69) as discussed in Chapter 6. The Dimensionless Transport Rate is related to the Shield's Entrainment Function by the equation,

$$f_c.\phi = 0.1\theta^{5/2} \qquad 7.20$$

where the Dimensionless Transport Rate is,

$$\phi = \frac{q_m}{\rho_s} \left(\frac{\rho}{g(\rho_s - \rho)D^3}\right)^{1/2} \qquad 7.21$$

$D = D_{50f}$, the equivalent size having a fall velocity of D_{50} and given by the authors on their Table 2.2.1. The friction coefficient is given by,

$$f_c = \frac{2u_*^2}{\bar{u}^2} \qquad 7.22$$

The formulation was tested by comparison with the Guy et. al. data. The authors state that it is valid for "duned" beds and for grain sizes greater than 0.00015m, and it includes the suspended load. After some manipulation, the Engelund and Hansen equation can be written in the same form as the others,

$$q_m = \frac{0.05\ \rho_s}{\left[g\left(\frac{\rho_s}{\rho} - 1\right)\right]^2} \cdot \frac{\bar{u}^2.u_*^3}{D_{50f}} \qquad 7.23$$

The equation was selected for comparison because of its applicability to "duned" beds, and was included in the YALIN programs.

7.10.4 Yalin Bedload Equation for Plane Beds

The Yalin (1963) equation was derived by considering the average lift forces exerted on the particles at the bed. In Yalin's theory, particles are assumed to move over the bed by saltation and any increase in transport is brought about by an increase in particle path length and not necessarily an increase in the number of particles moving. Notably, the equation is valid over a plane bed only.

The transport rate per unit bed width is given by,

$$q_m = 0.635\ \rho_s D\ u_*'\ \left[\tau_* - \frac{1}{a}\ \ell n\ (1 + \tau_* a)\right] \qquad 7.24$$

$$a = \frac{2.45\sqrt{\theta'_{cr}}}{(\rho_s/\rho)^{0.4}},\quad \theta'_{cr} = \frac{\rho u_{*cr}'^{2}}{g(\rho_s - \rho)D}$$

where the superscript (') is included to highlight the applicability of the equation to plane beds and $D = D_{50}$ was applied for the following comparisons, in the YALIN series.

7.11 Determination of representative velocities

The traps were generally deployed for 1-2 hours (Table 7.7) so a representative speed in the period had to be found. To do this, the area under the curves of velocity versus time was ascertained numerically by finding the area under continuously-updated parabolae (equation 6.15) fitted to three values of velocity; the one nearest the time of interest and the one before and after. Because bedload equations are non-linear in speed and vary with the third power, in the equations described the computer was programmed to find the area under the function raised to a power q. The representative speed was the qth root of the area then divided by the elapsed time. $q = 3$ was used, and comparisons later will show that the refinement was significant with respect to a linear ($q = 1$) procedure. By a similar method, the Engelund and Hansen predicted transport rate, was found from the area under the function $u^{-2}\ u_*^3$. The detailed computer program which performs these operations as well as calculating transport rates and several non-dimensional variables, was called DOMNCE.

It was necessary to introduce a critical velocity because often the traps were placed just after slack water in readiness for the next tide, and the total time that the trap was submerged included periods when the bed was dormant. The transport rates and representative velocities are more correctly estimated from the time that the bed was in motion. Critical skin friction velocity was found from the Yalin threshold curve, while u_{*cr} and u_{lcr} were obtained in the fashion just described in this chapter and listed in Table 7.5.

7.12 Bedload transport results

7.12.1 Engelund and Hansen

Figure 7.23 shows the rate predicted by the Engelund and Hansen equation (7.23) against the measured rate determined from the bedload traps, and, in general, the equation overestimates. This would be expected for a total load equation especially at the fastest site (S3), where suspended loads, which have not been included, are likely to be significant. For some cases though, there were greater catches in the bedload traps alone than predicted by the total load equation. This deviation is apparently systematic occurring mostly at the shallowest site S4 where d = 3.4m.

If the Engelund and Hansen equation is applicable to tidal flow, then this result implies that the estimate of the shear stress from the profile is too small at S4. This was not evident from the previous analysis of friction coefficients at this site where strong correlation between the bedform geometry measured by divers and the friction coefficients found from the velocity profile was noted. The Engelund and Hansen equation relies on a partition of effective stress, which did not compare well over tidal megaripples with the value derived from the Karman Prandtl equation or from the Manning-Strickler equation (Figures 6.29 and 6.26). As has been shown, due to bedform hysteresis, the bed geometry and the flow mean speed are not necessarily related. The Engelund and Hansen equation suggests that they are.

7.12.2 Yalin and Sternberg

The measured bedload catches plotted against the total friction velocity were compared with the Yalin (7.24) and Sternberg (7.15) equations in Figure 7.24. The Yalin equation is only applicable to plane beds and as the friction velocity includes the form drag, which does no effective work on the sediment, the equation overestimates the transport rate in all cases. The Sternberg formula, which was partly derived from rippled beds, approximates the Whangarei data better, but there is still a considerable scatter, especially for the cases of high Drag Coefficient at S3. Sternberg's equation is reported to account for variations in the Drag Coefficient (i.e. form drag changes) within the formulation of the constant K which increases with excess shear stress. But the equation grossly overesimates the transport rate at high Drag Coefficient (approx. 6.5×10^{-3}) and generally underestimates for $C_1 = 4.0 \times 10^{-3}$.

7.12.3 Gadd et. al.

The Gadd et. al. modified Bagnold equation was examined by plotting the transport rate against the cube of the excess 1m velocity in Figure 7.5. Threshold speeds were obtained from Table 7.5. The best fit linear curve

has a correlation coefficient of 0.8 with 18 data points and this is highly significant at the $\alpha = 0.05$ level. The best fit equation was,

$$q_m = 0.123(u_1 - u_{1cr})^3 + 0.00045 \qquad 7.25$$

or,

$$q_m = 0.145(u_1 - u_{1cr})^3 \qquad 7.26$$

The constants are not non-dimensional and SI units have been used throughout.

As found by Gadd et. al. and now at Whangarei there is a relationship between the two parameters. However, the gradient of the curve at Whangarei of $\beta = 0.145$ kg.m^{-4}.s^2 is more than an order of magnitude less than that found by Gadd et. al. from the flume data of Guy et. al. Their larger value could arise from the assumption that the mean speed in the flume in water of depth 0.61m was equivalent to the 1m speed. They also selected very low thresholds of 0.15 to 0.20 m.s^{-1} which related again to the mean flow velocity which occurs at 0.38 of the depth i.e. at approximately 0.23m above the bed. When Heathershaw (1981) compared several bedload equations, the Gadd et. al. formulation exceeded the predictions of the others by more than an order of magnitude in some cases and was nearly always greater. It is believed that they have overestimated the proportionality constant and that it should be closer to 0.145 kg.m^{-4}.s^2 for grain sizes of $D_{50} \approx 0.0003$m. The equation (7.26) is easy to apply for a first estimate of sediment transport rates under tidal flow but care must be taken to estimate the threshold speed which has been shown to depend on the size of the bedforms.

7.12.4 A new bedload procedure

Because of bedform hysteresis and site dependent form drag components, bedload equations which use total bed shear stress are susceptible to inaccuracies in tidal flow because there can be no way of guaranteeing that the effective shear can be represented by the total friction velocity which often has variable form drag components. It is felt that a better approach is to attempt to isolate the skin friction (Vittal et. al. 1973) and revert to a bedload equation for plane beds such as the Yalin curve. To this end, the results of Chapter 6 were applied.

For two reasons it was preferred that the formulation used the 1m speed because: (i) this speed is easily measured and commonly used in estuaries, and (ii) the alternative mean vertically-averaged speed occurs at a position in the water column which is a function of the depth as well as the bed roughness. Intuitively, in an estuary this is not satisfactory when the sediment transport rate would seemingly be governed by a speed near the bed.

Thus, the effective skin friction velocity was estimated with equation (6.78) from the 1m speed as,

$$u_*' = \frac{u_1}{5.75 \log_{10}(30.2/2D_{65})} \qquad 7.27$$

The selection of the bed roughness which fitted the data best was found to be $k = 2D_{65}$ after numerical experiments with the D_{90} size. This complies with results of Engelund and Hansen cited in Chapter 6. Numerical experiments were also conducted with the Manning-Strickler equation but the correlation was poorer than with the KP.

Figure 7.26 shows the Yalin Bedload Equation with the measured bedload transport rate over megaripples against the skin friction velocity, u'_*, found from the 1m speed with equation (7.27). There is good correlation of the measured transport rates to the Yalin equation with no apparently systematic deviations related to ebb or flood, or decelerating or accelerating flow (Figure 7.26).

In Chapter 6, the skin friction equation (7.27) applied in combination with the Vanoni and Hwang formulation for form drag estimation, consistently predicted friction factors found to be related to the measured values obtained by curve fitting the vertical velocity profile. The equation is now shown to be representative of the effective stress responsible for bedload sediment transport. The formulation relies on the 1m speed to isolate the effective friction velocity and has no explicit dependence on bed geometry. However, with an increase in bed roughness, the 1m speed is reduced by the form drag losses and thus it implicitly contains the expected dependence.

The application of this procedure has advantages in that the rate of transport depends on easily measured quantities, reducing fundamentally to a determination of the speed at 1m and the D_{50} and D_{65} grain sizes to specify the bedload rate.

The correlation to the Yalin equation shows that in oscillatory tidal flow over the short term (1-2 hours), the bedload transport rates approximate their counterparts under uni-directional currents. The remaining scatter in the data could be due to:

(i) bedload trap deficiencies and too few sampling locations on the megaripples.

(ii) turbulence at near-bed levels which are coupled with the effective stress. However, this should appear as a systematic deviation which was not noted.

(iii) experimental error, related to long-term turbulence and the position of the current meter relative to the bedform crest when speeds were measured.

Finally, to examine the influence of the power of $q = 3$, representative friction velocities were obtained for $q = 1$, and measured bedload rates were compared with the Yalin equation in Figure 7.27. No comparative goodness of fit was performed, but the third power procedure (Figure 7.26) looks to be better correlated with the Yalin curve.

7.13 Megaripple advance rates

The experiments with staked megaripple crests were designed to measure nett bedload movement over a tidal cycle. For this reason, the crests were staked and generally measurements were taken at slack water only. In reversing flow, it is difficult to estimate the bedload transport rate over a half cycle because, when the megaripples turn around, the crest movement is often far in excess of the movement of the centre of mass of the feature and the latter is required to specify the transport rate. A complete set of measurements was taken during the tide (i.e. after the crests had turned around) and then again at slack water when the features still faced the same way on one occasion. Only this data is used here to verify the bedload transport rates.

Due to reverse flow in the bedform troughs - seen on the underwater video - megaripples advance with a positive and a negative sediment flux. Engel and Lau (1981) presented a method to compensate for this which relies on the determination of the point of flow reattachment.

The equation is,

$$q_m = \rho_s \ (1 - P).K. \ \frac{\Delta}{4}. \ U_w \qquad 7.28$$

where the porosity, P, of the sediment, was assumed to be 0.45. This is greater than 0.4 which is often used (Komar, 1976) in response to the very loose packing of sand in active megaripples noted by divers and has been used throughout the thesis. K is a constant which depends on ripple steepness (Δ/Λ). Their Figure 3 (Engel and Lau, 1981) presents K for a family of D_{50}/Δ curves and against the ripple steepness. Table 7.8 gives the value of K for the S-series using bedform heights and lengths compiled in Table 7.2. These average values were compounded from the underwater photography and diver bed geometry measurements (Table 7.9). U_w is the speed of the bedform and the formula (7.28) assumes that the bedforms have a triangular cross-section.

At S4, over the period from 10.25 to 12.00, six megaripples moved an average of 0.3m (Table 7.9) when the 1m speed averaged 0.53 $m.s^{-1}$. When plotted in Figure 7.26, the megaripple advance rate compares favourably with both the Yalin bedload equation and the bedload trapping results, supporting the new method.

7.14 Nett megaripple advance over the tidal cycle

At S3-S5 a complete set of measurements of the bedform geometry and advance was taken at 3 successive slack waters (Table 7.9). Further verification of the new method was undertaken by:

(i) using the Engel and Lau equation, the mass transport rate per metre of bed per tidal cycle was found from the average nett advance distance for the six megaripples at each site.

(ii) using the measured 1m speeds, a predicted nett mass rate was found with this writer's program MEANS (Chapter 5) by applying the new bedload method with the Yalin bedload equation and the skin friction formulation (7.27). The threshold speed was obtained from the Yalin Threshold Curve. Velocity scaling for tidal asymmetry was not necessary.

Table 7.6 compares the results and the very good agreement strongly supports the validity of the new bedload procedure. The quality and implications of this result are worth stressing. The two methods of comparison utilised entirely independent procedures for solution. The first employed megaripple advance rates measured by divers and the second relies on the measurement of the 1m speed and the D_{50} and D_{65} grain sizes only. The excellent agreement arises after the summation of transport rates over 60s intervals for the entire tidal cycle with a possibility of a large error accruement.

At S5, the small over-estimate may result from the fact that ebb and flood currents were not oppositely aligned. Due to the destruction and creation of the megaripples in this case, nett transport determination over a complete tidal cycle cannot be accurately ascertained with the megaripple advance experimental procedure. For example, in the worst case with currents 90^{o} apart, after reconstruction the new perpendicular megaripples cannot be considered as the same features with different orientation. At S3 and S4, there were no significant deviations between the calculated and measured transport rates over the tidal cycle.

7.15 Suspended load

Suspended load concentrations have often been described by the Rouse (1937) formula,

$$\frac{C}{C_a} = \left[\frac{d - y}{y} \cdot \frac{a}{d - a}\right]^z \qquad 7.29$$

where C is the sediment concentration ($kg.m^{-3}$) at y metres above the sea bed; C_a is the reference concentration at "a" metres; and d is the total water depth. The Rouse Number, z, is defined as,

$$z = \frac{w}{\kappa u_*} \qquad 7.30$$

where w is the fall velocity. Raudkivi (1976) suggests that for small values of z, the formula gives "surprisingly" good results but for larger Rouse numbers there are deviations. Measured Rouse numbers are mostly less than the value calculated by equation (7.30) (Fukuoka, Figure 9.1, 1978), signifying a more uniform sediment distribution than that predicted by the Rouse formula. The formula could not be applied at Whangarei without modification to z. Allen (1982) suggests that the deviation occurs because free falling velocity is used to calculate the exponent, whereas during turbulent transport

the grains are accompanied by others and are moving relative to a violently, agitated fluid. The effective falling velocity is then reduced, often substantially, from the value for a solitary particle in a still, unbounded fluid.

It was necessary to select a reference height from one of the three measurement levels; 0.1, 0.3 and 1.0m. The uppermost level was preferred because in megarippled regions, the definition of the zero bed level becomes important when sampling heights are comparable to the megaripple height. Also at low levels, the suspended load varies more markedly with distance from the crest, and practical difficulties prevented samples being taken along the length of the bedforms. The stand remained in place for each tidal cycle and was turned at slack water only. Thus, the upper level was chosen as a reference.

Taking the logarithm of both sides of equation (7.29) gives,

$$\log_{10}(C) = z_m \log_{10}\left[\frac{d - y}{y} \cdot \frac{a}{d - a}\right] + \log_{10}(C_a) \qquad 7.31$$

and this is a linear equation with gradient z_m and intercept $\log_{10}(C_a)$.

Using the data shown in Table 7.10, the concentrations were fitted by linear regression to obtain the best fit Rouse Number, z_m. As no grain size analyses were performed, to find z, D_{35} from the samples out of the bedload traps was selected to represent the material in suspension using a corresponding fall velcoity from the Table on p. 10 of Raudkivi (1976). In all cases z_m was less than the value of z found with equation (7.30), and was generally within the range of scatter in Fukuoka (1978).

The Rouse Number is often modified and Raudkivi (1976) suggests two methods: (i) inclusion of a constant, β, into the denominator as,

$$z = \frac{w}{\beta \kappa u_*} \qquad 7.32$$

or (ii) raising the fall velocity to a power, x, as,

$$z = \frac{w^x}{\kappa u_*} \qquad 7.33$$

Incorrect selection of the D_{35} grain size to represent the suspended material will be compensated for by the modification of the fall velocity in the Rouse Number.

Both methods were examined and the values of x and β obtained at Whangarei are listed in Table 7.10. At the three sites, x was more constant, and average values were found to be,

$$\bar{x} \begin{cases} = 1.31 \text{ at S3 for N = 3} \\ = 1.29 \text{ at S4 for N = 3} \\ = 1.32 \text{ at S5 for N = 3} \end{cases} \qquad 7.34$$

with overall mean of $\bar{x} = 1.31$. Notably, the overall mean of β was $\bar{\beta} = 3.27$.

The suspended transport rate as mass per unit width of bed per unit time is equal to the integral over the depth of the concentration of suspended material multiplied by the water speed. To evaluate this, the speeds at 0.1 and 0.3m were found from the gradient of the velocity profile on a semi-log plot and measured concentrations were introduced. In the upper levels, where speeds had been measured, concentrations were derived from the Rouse formula with Rouse Number defined by equation (7.33) and $x = 1.31$ in all cases. The selection of a constant x was meant to smooth the fluctuations of sediment load which occurred during the tidal cycle due to turbulent velocity bursts and changes of the stand position relative to the bedform crest. It was not possible during the tide to check stand inclination, or changes of the level of the nozzles off the bottom due to the passage of bedforms. It is thus expected that some variation of x should occur and the mean value would best represent average conditions throughout the tide. The integral was evaluated from 0.12m above the bed (the height of the sediment trap mouth) up to the surface by numerical integration, assuming linear changes between measured levels.

When plotted against $\bar{u}$ and u_* (Figures 7.28 and 7.29), the calculated suspended load sediment transport rate showed scatter. However, against u_1 the correlation was good. Shown in Figure 7.30, an empirical best fit power equation was obtained which related the suspended load to the 1m speed as,

$$q_{ss} = (2.03u_1)^{7.75} \times 10^{-3} \qquad 7.35$$

where q_{ss} is the vertically-integrated dry mass transport rate with units $kg.m^{-1}.s^{-1}$ and u_1 has units of $m.s^{-1}$. The correlation coefficient was 0.94. This equation was obtained with 8 data points and did not include the two points furthest from the curve in Figure 7.30. Although it is always difficult to justify the deletion of data, these points were felt to arise from a lack of experimental control and could be associated with the passing of a megaripple crest or turbulence. One of the anomalous points arises out of an abnormally low 1m speed measurement relative to the measurements at the other levels. The rest of the data shows strong correlation. Applying error bars to the sample correlation of 0.94, with the Fisher Z transformation (Freund, 1974, p. 428), the population correlation coefficient lies in the range $0.70<r<0.99$ at the $\alpha = 0.05$ level of significance.

The high power dependence of 7.75 compares with a suspended transport rate proportional to the friction velocity to the seventh power found by Dyer (1980). The correlation of the depth-integrated transport rate to the 1m speed arises out of a similar correlation of the suspended load concentrations at 1m above the bed to the 1m velocity. With the latter shown in Figure 7.31, the trends are the same as before and the best fit curve, excluding 2 data points, was,

$$C_1 = 56.5\ (u_1)^{6.3} \times 10^{-3} \qquad 7.36$$

with correlation coefficient of 0.88.

7.16 Total load

On the strength of the high correlation of suspended load to the 1m speeds, which includes data from three sites with different grain sizes, the suspended loads for the periods when the traps were on the bed were derived for calculation of total load. Using each measured velocity at 1m throughout the tidal cycle, the suspended sediment transport rate was determined with equation (7.35). Then the suspended load transport rates during the bedload trapping periods were found numerically in the manner described for determining representative velocities using program DOMNCE. These were added to the bedload trap rates to find the total load.

The comparison with the Engelund and Hansen total load equation shown in Figure 7.32 is now adequate. Data from the fast site with high Drag Coefficient (S3) and from the slower site (S5) shows good correlation. However, at the shallower site (S4), the measured rates are far in excess of the predicted rate being greater by a factor of about 3. Raju et. al. (1981) found that the Engelund and Hansen equation underestimated the total load by an average factor of about 2 for similar hydraulic conditions in uni-directional flow.

At Whangarei, the deviation is associated with wide variations of friction velocity during the tidal cycle and with the determination of bed shear stress by the profile method. The plotted point which deviates the most corresponds with an anomolously low friction velocity at about 3 hrs after high tide (Figure 6.20). The form of this figure shows a strong resemblance to flood case (a) of Figure 6 in Dyer (1970), which is typified by a steep peak ($u_* > 0.06$ m.s^{-1}) then a sudden drop to an extremely low value of less than 0.02 m.s^{-1}. Irrespective of whether these variations (i) result from measurement position relative to the megaripple crest; (ii) are associated with megaripple reversal; (iii) result from a reduction in roughness length with high sediment loads; or (iv) in some cases may even be errors introduced by turbulence, they make it more difficult to determine sediment transport with the bed shear stress obtained from the profile curvature. The correlation of the 1m speed to the transport rates is pleasing because the former is an easier parameter to determine than either the bed shear stress or the mean velocity.

The new bedload procedure (7.24 with 7.27) could have wide general application as the Yalin Equation has a specific grain size and density dependence. The constant of 2.03 in the suspended load equation (7.35) is at least a function of the grain size and would be expected to vary with the grain diameter. It includes the range of $0.00024 < D_{50}m < 0.00034$ which is

typical of many estuaries, and the empirical curve spans the speed range of $0.47<u_1<0.78$.

In summary, Figure 7.33 shows the bedload, suspended load and total load equations against the 1m speed for $D_{50} = 0.0003$m, $D_{65} = 0.00035$m and $u'_{*cr} = 0.014$ m.s^{-1}. The suspended load equation is plotted for speeds exceeding 0.5 m.s^{-1}.

7.17 Determination of the distribution of tidal range

For application to estimates of annual sediment transport fluxes, the distributions of tidal ranges and periods were sought. Tidal statistics taken from the Marsden Point gauge and recorded in the NHB Pilot's Log Book were examined for the period June 1 to December 31, 1982. A summary of a statistical analysis of this data is given in Table 7.11. Histograms of the flood and ebb half tidal periods, $T_{\frac{1}{2}}$, were plotted in Figures 7.34 and 7.35. A shorter ebb half period averaging 6 hrs 9 mins compared to 6 hrs 16 mins for the flood was identified. Notably, this does not equal the periods reported in Chapter 1 after Millar (1980) but the sum of the two times corresponds with the expected semi-diurnal tidal period of 12 hrs 25 mins.

Tidal range (ΔH) distributions in Figure 7.36 and 7.37, have the expected mean of 1.87m. Maximum and minimum ranges over the 7-month period were 2.80m and 1.17m. The data includes the mid-Spring equinox and the Winter and Summer solstices.

The flood and ebb distributions of the ratio $\Delta H/T_{\frac{1}{2}}$ (Figures 7.38 and 7.39), reflect the dominant period distribution exhibiting a lower flood than ebb mean.

With ebb and flood data combined, the half period centres very strongly between 6 and 6½ hours (Figure 7.40). The tidal range distribution (Figure 7.41) is skewed to the right with the mean being greater than the median so that smaller ranges are apparently more common than the larger ones. This probably results from using 7 months of data and not a complete year. Most common ranges were 1.6 - 2.0m.

The tidal range cumulative probability curve was compared with both the Normal and Weibull probability density functions in Figure 7.41 and the data distribution is better represented by the Normal function. The best fit 2-parameter Weibull distribution was ascertained using the method outlined in Black (1978a). The Normal distribution with standard density function is given by,

$$f(x) = \frac{1}{\sqrt{2\pi}} e^{-x^2/2} \qquad 7.37$$

where,

$$x = \frac{\Delta H - \bar{\Delta H}}{s} \qquad 7.38$$

and for the tidal ranges at Marsden Point ΔH = 1.87m, s = 0.347m.

7.18 Annual sediment transport rates

By utilising the sediment transport equations derived in this Chapter, the distribution of tidal ranges just reported and the bed facies information from Chapter 2, the annual sediment transport fluxes at each site may be obtained.

7.18.1 Procedure

This writer's Program MEANS uses a parabolic interpolation of water velocities to ascertain the total load transport over the tidal cycle. The interpolation procedure has already been described (Chapter 6). The bedload transport rate was obtained using the new procedure. The suspended load was found from the 1m speed according to equation 7.35. Ebb, flood and nett transport rates were derived.

The grain sizes used were D_{50} = 0.0003m and D_{65} = 0.00035m for all sites. These sizes are typical of the loose sand found on the south-east extension of Snake Bank north of the NHB wharf and was universally applied because mostly no grain size analyses were performed with sediment from the Braystoke measurement sites. Moreover, many of the locations are shell covered or have a high shell content and this factor is more significant than small grain size variations. The treatment of the shell cover is discussed later.

The nett sediment transport fluxes over the tidal cycle in kg/m of bed/tidal cycle were found for a low Neap range of 1.4m, a mean tidal range of 1.9m and a high Spring range of 2.4m with MEANS, using the procedure of linear velocity scaling according to tidal range (previously described in Chapter 5) to ascertain velocities in each case from the measured speeds. These results are shown in Table 7.12, and reveal that the high Spring rates are about 50-100 times the low Neap rates. Clearly, the tidal range is a very important factor.

For determination of annual fluxes, a mean rate, weighted with the probability that each tidal range will occur, was derived (Program SNDWGT). A span of tidal ranges from 1.1m to 2.7m was divided into 100 increments and the probabilities were found from the Normal distribution just derived.

The transport rate for each increment was found by extrapolating or interpolating using the three known rates with ranges 1.4m, 1.9m and 2.4m. Transport rate, ϕ_m, is not linearly related to tidal range, but is better approximated by a power curve of the form,

$$\phi_m = a(\Delta H)^b \qquad 7.39$$

Taking the logarithm of both sides gives,

$$\log_{10}\phi_m = \log_{10} a + b \log_{10} \Delta H \qquad 7.40$$

which is now a linear relationship. The gradient and intercept of this straight line was found from two of the three known transport rates for ΔH = 1.1m to 1.9m or for ΔH = 1.9m to 2.7m, and rates were found by interpolating or extrapolating along this linear curve. When multiplied by the number of tidal cycles in a year, the weighted mean gives the annual sediment transport fluxes.

Both the weighted means in kg/m/tidal cycle and the annual fluxes in m^3/m/year are given in Table 7.12. The sediment was assumed to have quartz density of 2650 $kg.m^{-3}$ and porosity of P = 0.45. The rates shown correspond with sand beds at every site and volumes include the pore spaces.

7.18.2 Results

Annual nett sediment transport rates vary up to a maximum of 402 m^3/m/year at Site K, but are more typically 20-50 m^3/m/year, with several near-neutral or low energy sites having nett rates of less than 10 m^3/m/year (Table 7.12).

7.18.2.1 *Entrance throat*

The B-series across the entrance throat and Sites I, M, 9, C and D on the southern side of the entrance throat exhibit an important trend. In all cases, except Site B4 on the northern side of the entrance throat, the nett sediment transport is directed out of the harbour (Table 7.12). In the presence of these dominant ebb currents, the estuary is not receiving large inputs of sediment which reaffirms the previous discussion in Chapter 5 stating that the shell lags are caused by an under supply of sandy sediment entering Whangarei Harbour. Without other factors, such as external storm events and differences in bed facies on either side of the entrance throat, the evidence shows that the harbour floor should be scoured by erosive ebb-tidal flows. Annual sediment fluxes on the southern side of the entrance are typically 20-30 m^3/m/year. Over the southern half of the throat of 360m width, this corresponds with 7200-10,800 m^3/year of sediment leaving the harbour. However, the bed is not all sandy, and much of the sea floor under the strongest currents is shell-lagged in response to the tidal current asymmetry noted here and in fact the entrance is stable (Chapter 2). These results show that it is not possible to estimate sediment fluxes in estuaries, especially near the entrance throat, from the water speed alone. Without some detailed knowledge of the sea floor sediments, determinations which do not correctly account for sea bed shell content, are guaranteed to derive incorrect result.

7.18.2.2 *Proposed port and dredge zone*

On the south side of the harbour at the proposed port zone (Figure 7.42) Sites A, B and E, all exhibit very low rates of 11, 2 and 4 m^3/m/year respectively due to low current speeds and near neutrality at B and E. Thus, excluding the effect of the changed hydraulics in the region, the port should

have a minimal impact at these sites.

Offshore from this, Site A2 is nearly neutral with a rate of 9.17 m^3/m/year directed up-harbour. Site A3 exhibits a strong ebb dominance with a rate of 136 m^3/m/year. However, throughout much of this region the bed is shell lagged, and the actual transport rates would be much less than this, and again the port should have little impact.

The northern side of the dredge zone (Case A7, DHI, 1982) cuts across an active sub-tidal spit. With configuration A7, the depths along this ridge will increase to 13m in the dredge zone from 7.5m at the western end, 5.5m at the shallowest part of the ridge and 9m at the eastern end. The change to the bathymetry is considerable.

As previously noted, this ridge of sediment is subject to ebb dominant tidal flows on its southern side - Sites F, S2 and S3 exhibit ebb-directed transport rates of 58, 93 and 27 m^3/m/year respectively. Over an active width of approximately 70m (Figure 2.7) this corresponds with a nett rate of 4150 m^3/year directed out of the harbour. On the northern side, the transport is flood-directed at Sites S1 and K with rates of 114 and 402 m^3/year. K is much further up-harbour and more onto the flood ramp. Thus, using the S1 rate only, over a width of about 90m (Figure 2.7) this corresponds with a nett up-harbour transport rate of 10260 m^3/year. Both of these quantities are very small with respect to maintenance dredging and arise because of the near neutrality of the region, and because of the narrow restricted nature of the active zone.

If it is assumed that (i) flows are unchanged after dredging; and (ii) the dredge zone acts as a sediment sink catching all sand passing along the active ridge, then the rate of accretion may be approximated by summing the present ebb and flood tide sediment fluxes. Shown in Table 7.12, in this case, the ebb dominant sites would introduce sediment at the rate of 71 (F), 402 (S2) and 63 (S3) m^3/m/year, which corresponds with an annual rate of 12500 m^3/year. The flood site S1 exhibits a total flux of 268 m^3/m/year which converts to 24,120 m^3/year across the flood dominated width of the ridge. In total the maximum flux of sediment entering the dredge zone would be 36,620 m^3/year. All of this sediment arrives along the active ridge on the northern side, and the result assumes that all sand entering the dredge zone remains there and is not swept away by tidal currents. This flux is therefore an approximate upper limit of the sedimentation rate in the dredge zone. The actual rate of accretion can be only estimated after numerical modelling to determine the water speeds and 2-dimensional sediment fluxes including both the gains and losses in the region.

The upper limit of accretion derived here is very similar to figures produced by the DHI (1982). As they arrived at a rate by originally assuming that the harbour was covered with sandy sediment, and as their calculations

derive the rates by examining the volume of sediment entering the dredge zone from the main shipping channel with ebb dominant currents, the agreement is coincidental.

7.18.2.3 *Shipping channel near One Tree Point and Shoal Bay*

In the main shipping channel towards One Tree Point, cross-section C shows highest rates (Table 7.12) between C1 and C2 with an ebb resultant and is associated with the ebb shield and megaripples near the 5m contour (Chapter 2). Over a width of 100m, approximately 4450 m^3/year is evidently moving in the ebb direction on the southern side of the channel. Site C3 is near neutral or flood-directed and in the latter case approximately 4000 m^3/year (over a 100m width) could be moving up harbour and should be associated with the megaripples that floor the Shell Cut Reach Channel (Chapter 2).

There is no definite information on the bed type at C4 on the northern side of the channel or at downstream Site 2, which exhibit ebb-directed fluxes of 28 and 63 m^3/m/year respectively.

Over the D-cross-section in Shoal Bay, the rates at D1, D2 and D3 are very low due to a balance of the ebb and flood currents at D3 and to the very low water speeds at D1 and D2. Megarippled Site D4, carries 23 m^3/m/year of sand over a width of about 150m. The total quantity moving up the north side of Snake Bank is then approximately 3450 m^3/year.

7.19 Conclusions

(i) higher shell content was found to generally occur in megaripple troughs with finer, sandier sediments on the crests. Slipface angles of repose were found to be 28^o - 33^o.

(ii) a reduction in bedform height near peak flow noted in bed photographs, was associated with corresponding decreases in friction velocity, during periods of high sediment transport.

(iii) video examination of flow over ripples is summarised in Figure 7.15. Scour and turbulence at the point of flow reattachment, reverse flow and longitudinal helical flow in the ripple troughs were noted.

(iv) a sediment threshold speed definition was introduced such that threshold occurs when the bed is moving 50% of the time.

(v) A threshold equation (7.10) was derived to determine the sediment threshold speed at 1m over megaripples. It employs the Vanoni and Hwang (1967) formulation to find the form drag friction velocity. Verification was made using underwater video inspection of the sea bed at Whangarei to ascertain the sediment thresholds, and by comparison with data published by Sternberg (1971). The method requires determination of the skin friction velocity from the Yalin Threshold curve, total friction velocity using (7.9) with (7.5), and 1m threshold speed using (7.10).

(vi) Over a planar sandy bed with interspersed, embedded shell, the sediment threshold speed was found to be about 0.45 $m.s^{-1}$ which was much higher than over megarippled sediments being about 0.33 $m.s^{-1}$. The difference is caused by form drag energy losses over megaripples.

(vii) due to the wide variety of shell shapes and sizes, a general shell threshold speed is difficult to determine. However, an approximate threshold speed over shell for Whangarei Harbour of 1.05 $m.s^{-1}$ at 1m above the bed was derived. This is much higher than over sandy bed sediments.

(viii) the Sternberg (1972) bedload equation was shown to be a poor estimator of the sediment transport rates over megaripples, due to variations in the Drag Coefficient.

(ix) significant correlation was identified between the bedload sediment transport rates and the cube of the velocity excess at 1m. This formulation was previously presented by Gadd et. al. (1978). The proportionality constant of 0.145 $kg/m^{-4}.s^2$, at Whangarei, however, was found to be nearly an order of magnitude less than that found by Gadd et. al.

(x) a new procedure to determine sediment transport rates over megaripples in tidal flow was presented and verified with field measurements. Skin friction velocity is obtained with equation (7.27), and this is utilised in the Yalin Plane Bed Sediment Transport equation (7.24). The application of this method, has advantages in that the rate of transport depends on easily measured quantities, fundamentally reducing to a determination of the speed at 1m and the D_{50} and D_{65} grain size to specify the bedload rate.

(xi) megaripple advance rates measured throughout the tidal cycle provided excellent confirmation of the new bedload transport procedure.

(xii) a suspended load sediment transport equation was empirically derived which relates the transport rates to the 1m speed and is given in (7.35).

(xiii) using the bedload trap measurements and deriving suspended load with the empirical formula, the total load was determined for comparison with the Engelund and Hansen equation. At two of three sites the comparison was adequate, but at a shallower third site there were systematic deviations. The Engelund and Hansen equation relies on the determination of friction velocity and mean speed, and the former varies widely throughout the tidal cycle under non-steady tidal flow making application of the equation difficult.

(xiv) the bedload, suspended and total load sediment transport rates for a typical Whangarei grain size were plotted in Figure 7.33 against the 1m speed.

(xv) the annual tidal range distribution in Whangarei Harbour was described by a Normal probability density function with mean of 1.87m and standard deviation of 0.347m.

(xvi) annual sediment transport rates at the Braystoke sites were determined, and exhibited typical nett rates of 20-50 $m^3/m/year$ with a maximum of

402 m^3/m/year at Site K. At the entrance throat, ebb dominance on the southern side predicts a loss of sediment from the harbour at Marsden Point, but this is retarded by bed shell cover. At the proposed port site, the dredge zone disects an active ridge of sandy sediment at its northern side. The maximum accretion rates in the dredge zone were calculated to be 36,620 m^3/year. Along the southern side of the channel near One Tree Point, an ebb-directed transport rate of 4450 m^3/year was determined. In Shoal Bay, along the north-east of Snake Bank, some 3450 m^3/year was found to be moving up-harbour.

SEDIMENT TRANSPORT AND TIDAL INLET HYDRAULICS

Chapter 8:

THE INTER-TIDAL ZONE
AT THE PORT SITE

CHAPTER 8: THE INTER-TIDAL ZONE AT THE PORT SITE

8.1 Introduction

The proposed timber port at Marsden Point will span the sandy inter-tidal zone near the present NHB wharf and reclamation will totally obstruct longshore transport of sediment. Bathymetric stability of the region was identified over a two-year interval between two bathymetric surveys described in Chapter 3. This dynamic equilibrium is examined here to determine the sediment fluxes moving across the flats and particularly to ascertain the nett sediment transport directions. These aspects determine the future impact of the port on downstream locations along the inter-tidal zone. As most research is conducted in higher-energy regions, such as open beaches, sediment fluxes and wave climate had to be measured on site.

8.2 List of symbols

E	total energy in the spectrum (m^2)
f	frequency of a spectral estimate (s^{-1})
f_p	frequency at the peak of the spectrum (s^{-1})
H	wave height (m)
$\bar{H}_z$	zero up-crossing mean wave height (m)
$H_{z,1/3}$	zero up-crossing significant wave height (m)
$H_{z,1/10}$	average height of the largest tenth of the zero up-crossing waves (m)
$H_{z,max}$	highest zero up-crossing wave (m)
m_n	spectral moments
N	number of data points in a time series, number of waves in a record
Q_p	Goda's Spectral Peakedness parameter
T	record duration, wave period (s)
$\bar{T}_z$	mean zero up-crossing wave period (s)
$T_{Hz,1/3}$	significant wave period (s)
$T_{z,1/3}$	average period of the largest third of the zero up-crossing periods (s)
T_p	period at the peak of the spectrum (s)
$S(f)$	spectral energy density ($m^2.s$)
$u_{0.2}$	water velocity at 0.2m above the sea bed ($m.s^{-1}$)
u_1	water velocity at 1m above the sea bed ($m.s^{-1}$)
z_o	roughness length (m)

8.3 The inter-tidal flats

Aerial photographs (Figures 8.1-8.3) show the inter-tidal sand flats at the site of the proposed timber port. The shelf is typified by a nearly horizontal upper sand platform about 120m wide and 1.6m above CD. This falls away beyond a sand-waved region (marked on Figure 8.1) at 1.2-1.5m above CD to a lesser gradient lower platform before dropping suddenly into the main channel (see Cross-sections 5 and 6, Figure 3.19). The shelf is about 280m wide from the beach to Chart Datum.

The beach is narrow (approx. 10m) and steep, fronting a scoured shore-line which has been protected by the placement of tree trunks to prevent erosion of the gravel road servicing the houses along the foreshore. The road and sand flat are aligned nearly east-west, with an offshore normal at 22°

True (approximately equal to the Magnetic North bearing of 18° True).

In the vicinity of the boat ramp seen on Figures 8.1 and 8.2, the inter-tidal cross-section (Figure 3.19, Profile 4) exhibits a long, rectilinear slope with no evidence of platform formation. This region is scoured with virtually no beach and a rock wall has been placed along the foreshore. Healy (1981b) notes porewater pressure due to groundwater seepage in this area. Pipis (Paphies australis) are found at the northern end of the cross-section. Further towards Marsden Point to the east of the NHB wharf, the sandy sediments are less well-packed suggesting greater mobility.

At the port site, slow-moving sand waves with various orientations occur in confined zones. Small (Δ = 0.02m, Λ = 0.10m) ripples with symmetrical cross-sections, generated by surface waves, are widely distributed.

Schematic diagrams accompanying the aerial photographs show complex sediment transport patterns inferred from the large-scale bedform orientations. The region, centrally located at the proposed port site where most intensive field investigations were undertaken (Figure 8.2), appears to be subject to onshore sand transport at the seaward edge of the upper platform with long-shore divergence both to the east and west as the shoreline is approached. On the lower platform, the sand waves appear to be moving at 45° to the onshore normal and out of the north-east. Later measurements of current speeds suggest that this is opposed to an ebb-dominant current direction which implies sediment should have an out-harbour resultant. Examination of the aerial photograph published by the DHI (1982, Plate 1), at a low Spring tide shows short-crested sand waves moving shorewards at 45°, but out of the north west which correlates better with the water speed measurements. However, these sand waves are short-crested due to the presence of two perpendicular co-existant sand wave fields: one out of the north-west and the other out of the north-east.

The sediment transport pattern is localised and complex and each region needs to be examined individually. The upper platform was selected for closer examination and the transport patterns in the lower zone have not been further qualified.

8.4 Tidal current velocity measurements

On two calm days (July 17 and 23, 1981), current speeds were measured at 13 locations (Figure 8.4) with an Ott impellor-type meter supported on a steel rod. The experimental procedure is more-fully described in Chapter 5. The wind speeds during the two recording days, were less than 3 $m.s^{-1}$, so wave acitivity and wind-induced currents were neglible. On the 17th the tide range of 1.7m was less than the harbour mean average. On the 23rd, the tide range was 2.2m which is an average Spring.

The measured water speeds are shown in Figure 8.5 from Table 8.1 and most

were made on the 23rd during the Spring tide. All flood measurements are below threshold. Thus at the sites considered (which offer a comprehensive overview), sand is not being transported up-harbour under tidal currents alone even during Spring tides.

Sediment threshold exceedence mainly occurs along the NHB wharf from Pylon 14 to 24 during the ebb. The inter-tidal zone east of the NHB wharf is further above CD than the western side (compare Cross-sections 4 and 2 in Figure 3.19). As a result, flow constriction induces velocity acceleration and a directional shift around the obstacle towards the north from the east, and higher speeds occur.

During the field measurements in the shallow water, mobile bedforms were sighted from the boat at the high-speed locations on the ebb only. These verify that the threshold was being exceeded. However, the rates of bedload transport are still low. The maximum measured speed was 0.33 $m.s^{-1}$ at 0.2m.

For a rippled bed with Δ = 0.03m and Λ = 0.20m,

$$z_o = \frac{\Delta^2}{2\Lambda} \qquad 8.1$$

then z_o = 0.002m. The maximum speed at 1m can be found from the Karman-Prandtl equation with z_o and the speed at 0.2m as,

$$u_1 = u_{0.2} \frac{\log_{10} (1/z_o)}{\log_{10} (^{0.2}/_{z_o})} = 0.45 \text{ m.s}^{-1} \qquad 8.2$$

With this speed from Figure 7.33, the transport rate should be 0.0003 $kg.m^{-1}.s^{-1}$. If the current acts for a maximum of 1 hour per tidal cycle, then the transport rate per annum is 0.5 m^3/m/year where a porosity of 0.45, sediment density of 2650 $kg.m^{-3}$ and tidal period of 12.42 hrs were introduced. Thus, the tidal flow transport rates are very small even in the fastest currents.

The aerial photographs taken by this writer (Figures 8.1 to 8.3) reveal that sediment is apparently moving up-harbour between the NHB and NZRC wharves. This has been previously suggested by Millar (1980) and Healy (1981), implied from sediment texture and mineralogy. From the velocity measurements and beach surveys, the up-harbour sediment advance is evidently slowed or stopped by the ebb-dominant currents near the NHB wharf causing the bed level differences. Figure 8.4 shows the largest measured velocity at each site and depicts tidal ebb dominance. If this is the case, then the timber port should have no detrimental effect on the NZRC wharf and environs at the inter-tidal level as it too would act as a barrier to the up-harbour transport of sediment and accretion on its down-harbour side should occur.

It is noted that the influence of the port projection into the main current stream and the dredging are not included here as this discussion only deals with the transport of sediment along the inter-tidal zone. Furthermore, the resultant up-harbour between the wharves has not been verified with field measurements but the implications are generally supported by the bedload

trapping experiments on the upper platform west of the NHB wharf to be described later.

The second important implication of the velocity measurements is that sediment cannot be moving up-harbour due to tidal currents alone because: (i) the resultant is oppositely directed, and (ii) the speeds are mostly below the sediment threshold.

8.5 Sediment trap calibration experiments

Initial sediment trapping experiments, were established to examine the suitability of various types of sediment trap to the inter-tidal environment. Primarily, the comparison was made between VUV, (Figure 7.1) and embedded (circular and rectangular) traps. The effective widths of these are derived in Appendix 6.

The layout of the first experiment is shown in Figure 8.6. At Site B0, six VUV traps were placed on the circumference of a 10m-diameter circle around a rectangular embedded bin. The bin had length, width and depth of 0.63m, 0.49m and 0.13m respectively, thus the effective width (Appendix 6) was,

$$\bar{L} = \frac{2}{\pi} (0.63 + 0.49) = 0.71 \qquad 8.3$$

A further four traps were placed on an approximate on/offshore transect (locations B1 to B4 on Figure 8.6). B1, B2 and B3 were circular with effective widths of 0.20m, 0.20m and 0.22m respectively. B4 was placed in the loosely-packed sediments of the sand wave zone, and was rectangular with dimensions of 0.67m by 0.40m and effective width of 0.68m. The traps were placed when the tidal flat was exposed at low tide at 19:00 on 5-Jun-81 then serviced at the two successive low tides at 04:00 and 14:00 on 6-Jun-81.

The results are shown on the schematic diagram in Figure 8.7. Comparing the mean of all VUV trap rates with the rate in the central rectangular bin, reveals that the VUV traps predicted a higher transport rate for the first experiment (1.99 to 0.72 kg/m/tidal cycle) and lower for the second experiment (7.61 to 11.29 kg/m/tidal cycle).

Due to the occurrence of favoured directions, the experimental errors associated with all sediment trapping operations which are sometimes accredited with causing powers of 2-10 variations, and the necessary corrections which were applied for the effective widths, the deviations are felt to be small within the limits of the experimental procedure. However, if the small deviations between the two types of trap are significant, they could be due to (i) turbulence under the waves and the reversing nature of the currents which tend to wash sand out of the VUV traps, particularly if the waves approach from behind and/or (ii) the process of placement of the embedded trap which had the effect of loosening surrounding sediment. The sand has a

natural cohesiveness on some parts of the tidal flat, and the loose sand is more easily washed into the trap.

As would be expected, Trap B5, placed on the active sand wave, was catching more sand than any of the others. B1, B2 and B3 caught similar amounts to the VUV traps (Figure 8.7). In view of these calibration experiments, the VUV traps were employed for sediment trapping experiments in the inter-tidal zone.

8.6 A new sediment trap

A sediment trap was designed (Figure 8.8) for suitability to the inter-tidal environment with the following characteristics:

(i) the trap was embedded in the sand to minimise its effects on the flow conditions.

(ii) it was circular offering a symmetrical effective width to the flow from all directions.

(iii) it was segmented with four equally-sized compartments to provide directional differentiation.

(iv) for easier servicing, each compartment was lined with a sail cloth bag which could be removed, weighed and emptied. The bag was attached to the trap with VELCRO, which is an adhesive, nylon material with reusable bonds. To prevent sand loss a rubber flap was glued to the trap which fitted over the outer edge of the bag. The remainder of the cloth insert was held with plastic clips fitted over the central vanes of the trap. The steel trap could be left untouched and thus it was not necessary to disturb surrounding sand after each servicing.

(v) small vanes projected above the bed level to improve compartment isolation, and obstruct the passage of sand saltating over the upstream comparment and settling in the downstream side of the trap.

(vi) holes were drilled in the base of the trap to prevent it from floating during the rising tide and to allow water to drain on the ebb.

(vii) the compartments were "deep" to ensure that once sand was caught it would not be entrained again by wave-induced turbulence.

(viii) a heavy lid was constructed which could be removed or placed over the trap at high tide if half cycle sediment transport characteristics were required.

The unit is only suitable for the inter-tidal zone where it can be easily placed and serviced at low water.

8.7 Field sediment trapping program

Sediment trapping was undertaken during two periods: June 5, to 13, 1981 (June series), and July 21 to August 19, 1981 (August series). To differentiate nett transport directions, arrays of VUV traps were deployed at several locations. During the June series, VUV traps were emptied at every

tidal cycle i.e. approximately every 12 hours when the tidal flat was exposed. During the August series, the VUV traps were placed with neighbouring "new" sediment traps, and all types were emptied every second tidal cycle only (i.e. daily) except in some specifically noted instances.

Positions were fixed with a sextant and defined in an x-y grid established relative to the line of power poles along Papich Road. They were reduced to x-y coordinates and converted to the Mt Eden Geodetic Metric Grid on a Tl-59 programmable calculator, using programs written for the study. The trap locations are shown in Figure 8.6 with the June series denoted by locations A, A2A, A2B, A3A and the August series located at Fl-F5. The latter series are named after the New Trap numbers with Fl corresponding with New Trap 1 etc. During the August series, the VUV traps were placed: 1 (288^{o} T) at Fl;2 (108^{o}T) and 3 (288^{o}T) at F3; and 4 (108^{o}T) and 5 (288^{o}T) at F4. The angles are trap normals pointing into the mouth and orientations were found with a monocular. The site locations are shown on the aerial schematics (Figures 8.1 to 8.3). Fl lies shoreward of the sand wave zone; F2 and F3 appear to be more active; F4 is on the scoured cross-section 4 (Figure 3.19) just east and offshore of the boat ramp, and F5 lies in a more shelly region near live Pipi beds.

The results of the June series are presented schematically in Figures 8.9 to 8.11 for locations Al, A2 and A3 respectively. The August results are given in Table 8.2. All of the data has been scaled by the effective width (Appendix 6) and for the VUV traps the 70% efficiency correction (Chapter 7) has also been applied. Notably, the sediment was damp when weighed in all cases, so the masses are greater than the dry weights by approximately 20%.

8.8 New trap calibrations

Using the August series data, after correction for the effective widths, the VUV traps were found to be catching an average of 2-5 times more sand than the new traps with wide variations over a range of 2 to 18 times. This is an unacceptable deviation and yet when plastic embedded traps were compared with the VUV-type in the initial calibrations, the results were acceptable. The major difference between these and the new trap is the vane projection designed to improve compartment isolation. It is possible that the flow interference caused by these vanes is the major cause of the fault.

Secondly, embedded traps do not rise or fall with bed level oscillations. After a period of scour, the lip of the trap may be above the general bed level and sediment will pass around the trap. To allow for this, traps should be initially deployed with the lip approximately 0.01m below surrounding bed level, and must be dug in again after periods of scour, but low catches may occur during the scouring period.

Clearly more calibration needs to be undertaken, and this was planned for the field program when the wave height probe, discussed later in this Chapter, was installed. Delays in the probe establishment made it impossible to perform further tests with the new sediment traps and they are reported here to be apparently catching 2-5 times too little sand. However, it is later shown that they offer good nett sediment transport directional information.

8.9 Sediment trapping results

The trapped rates vary over at least two orders of magnitude from 1-100 kg/m/tidal cycle with the maximum catch being 411 kg/m/tidal cycle occurring on 12-13th of June at Trap 6 (Figures 8.9 to 8.11). These are very large variations which cannot be due to changing tidal currents alone. Furthermore, on some occasions, no sand was caught in the traps which supports the assertion that tidal currents are mostly below threshold. The order of magnitude rates are equivalent to 0.5-50 m^3/m/year for a tidal period of 12.42 hrs, a porosity of 0.45 and density of 2650 $kg.m^{-3}$.

It is noted that the effective widths employed so far assume all transport directions are equally likely. Thus, rates quoted are estimates of total directions in a year. If an obstruction such as the temporary NZRC barge off-loading ramp, was placed across the sand flat, the build up on either side should be at half the rate quoted i.e. from 0.25-25 m^3/m/year, because no sediment passes across the boundary. This ignores the changes to local hydrodynamics due to the imposition of the structure. Notably, the lower figure corresponds with the bedload transport rate found from the maximum recorded tidal speed, but the quantities here are not being moved by tidal currents alone.

8.9.1 Wind, wave and sediment transport correlation

Figures 8.12 and 8.13 show the wind velocities during the trapping periods plotted with average catches. There is a strong correlation of the wind speed to the sediment transport rates, and a weaker correlation to the wind direction. This implies that the transport rates are associated either with (i) wind-induced water currents forced by surface stress or; (ii) wind-generated surface, gravity waves.

The 2-dimensional modelling in Chapter 10 shows that the wind will effect the water speeds but the influence falls far short of the 2-3 order of magnitude variations noted in the transport rates. Thus, and this was confirmed by field observations, the increase in transport rate is primarily due to the stirring action of the bed by short-period, surface gravity waves. The harbour is strictly fetch-limited and wind generated waves are only small, yet they still have the energy to cause the considerably higher transport levels. Notably, during an easterly storm on 6-Aug-81, the sediment traps

were completely filled in a tidal cycle (Figure 8.13), which corresponds with a sediment transport rate of about 0.2 $kg.m^{-1}.s^{-1}$ and is equivalent to fast tidal currents of 0.67 $m.s^{-1}$ at 1m above the bed (Figure 7.33).

As noted in Chapter 4, westerly winds are the most common, while the biggest storms with the strongest gusts come from the easterly sector (Figure 4.10). Because of the alignment of the coastline, the sand flat is mostly sheltered from west to south winds. Of the remaining directions, the wind rose reveals an annual wind resultant with an up-harbour component.

This compares with general observations of Millar (1980, p. 177), who stated, "Wind waves inside the harbour entrance are also important in moving sand. During storms from the east-northeast, wind waves up to 0.60m in height break diagonally to the shore between Marsden Point and One Tree Point. On the flood tide considerable amounts of sand move up-harbour along the intertidal flats; however during the ebb tide sand transport along the intertidal flats was not observed."

This interesting observation implicates a tidally-asymmetrical process where current direction is an important parameter. Uni-directional currents will transport waves across large distances and are partly responsible for the entry into the harbour of low swell observed at the port site during heavy sea conditions outside. The process in estuaries is made more complex by refraction of waves on a current exhibiting transverse velocity gradients. In channelised flow, where currents are usually strongest in mid-stream as in the shipping channel to One Tree Point,waves approaching an opposing current are bent towards the fastest flow region. With supporting currents (flood tide at Marsden Point), waves are refracted towards the channel margins and this may explain the phenomenon noted by Millar.

The nett up-harbour sand transport thus depends non-linearly on the frequency of some wind directions, the wind strength and the stage of the tide. It is therefore not possible to make precise calculations without a more detailed knowledge of the wave climate, but the largest storms are infrequent. If a wind frequency of occurrence weighting is applied to the sediment transport range of 0.5-50 $m^3/m/year$, the lower figure would be favoured due to the predominance of lighter winds. The average annual movement is thus estimated as being in the smaller range of 2-10 $m^3/m/year$.

As an approximation, over the upper platform of 120m width, the build-up beside the port is expected to be at a rate of about 120-600 $m^3/year$.

8.9.2 Resultant transport

If the transport rates are treated as vectors and assuming all sand passes along the normal to the trap mouth, the resultants may be found. Symmetrical arrays offer an unbiased estimate and four and two trap arrays were mostly deployed for this reason. In the former case, four traps were placed at 90^{o} increments with one pointing offshore, to resolve longshore

and onshore components. Similarly the new traps were placed with one compartment facing offshore. Two VUV traps in a rectilinear arrangement have no orthogonal components. The resultants were obtained from the transport rates halved as the effective width employs a factor of 2 correction to allow for the 2 blind quadrants of the traps (Appendix 6).

During the June series (Figures 8.9 to 8.11), the resultants of the 4-trap arrays are mostly directed out of the harbour. This is in agreement with the expected transport direction due to the tidal currents. Location A3 exhibited more variable behaviour but had a strong onshore resultant overall.

At the two-trap locations, high-energy events were directed up harbour, while during low-catch periods, the opposite occurred. This is an important result which gives the first indication of how the sand may move up-harbour against the dominant ebb currents. Apparently, the tidal dominance is only significant during near calm conditions and as tidal speeds are mostly below threshold, this is a feasible result.

The resultants during the August series are shown in Table 8.3, and behaviour is variable. However, when daily resultants during the June and August series were summed as vectors at each site for the complete trapping periods the overall resultants for the latter period (Figure 8.6) from Table 8.4 exhibit commendable similarity with the sediment transport directions inferred from the large-scale bedforms on the aerial photographs (Figures 8.1 to 8.3). This behaviour was particularly evident for the August series which has a much longer sampling period and should be more representative of average conditions reflected by the bedforms. The new traps, although they apparently underestimate the magnitudes, are evidently specifying the nett sediment transport directions.

From Table 8.4, the August resultants averaged over the trapping period and adjusting artificially for the low new trap catches are seen to be of the order of 1 m^3/m/year, which is a small nett rate. This compares with total movement estimated to be 2-10 m^3/m/year.

8.10 Discussion

The analysis has shown that the sediment transport processes in the inter-tidal zone are governed by both surface waves and currents, with the former being the dominant influence. A strongly 2-dimensional, horizontal, transport pattern was resolved, with vectors at some locations being directed out of the harbour and shoreward at others. Therefore, the introduction of sediment to the harbour along this zone is not simply occurring as a "river of sand" flowing from Marsden Point into the upper reaches of the harbour and slowly infilling the inlet.

Furthermore, the quantities are small relative to the potential transporting capabilities of the nearby channels. In Chapter 7, it was shown that

the scouring rate in the channels was of the order of 100 m^3/m/year as against a much lower resultant rate of 1 m^3/m/year on the sand flat. Thus, even if sediment did move up harbour at this location, this does not imply that the harbour is infilling. Any small, nett-ebb imbalance in the current strengths upstream would be sufficient to offset this input.

Current meter measurements further towards One Tree Point revealed flood dominance on the sand flats near Point Sinclair and ebb dominance flanking this along the main channel margin. At low tide, a well-developed, separating ebb shield is evident. If it is assumed, and this is a strongly supported proposition (Chapters 5 and 10), that the inter-tidal zone in this region has a flood-directed sediment transport, then material forced off the tidal flats by flood currents may be returned down-harbour along the ebb shield with no nett infilling necessarily resulting. In support of this, bathymetric comparisons (Figure 3.16) revealed a nett loss of material from the harbour in the last 20 years.

With regard to the port site, the resultant transport rates measured in these experiments suggests that the sediment movement on the flat is small and that the port would have a limited effect on its environment caused by changes to sand transport on the upper platform.

8.11 Wave height recording

In view of the strong correlation between the sediment transport rates on the tidal flat and the wave heights resulting from the local winds, a wave height recording system was established. It was designed by this writer to be left unattended so that wave climates could be monitored for extended periods. With this requirement and as the sea surface elevation is sampled at small time intervals (2-8Hz), voluminous amounts of data were to be recorded. Thus, the system allows sea level measurements to be entered directly into the computer via a number of electronic stages without any data key punching.

This necessitated a high degree of sophistication in the electronic and computer stages and problems with the establishment of the electrical components of the probe meant that only the initial data was recorded and the detailed sediment transport experiments to be undertaken on the sand flat were not completed. However, the system was established and, at the time of writing was recording wave heights at Marsden Point in Whangarei Harbour. The instrumentation is described in this Chapter and results reported deal with wave generation by wind in a confined estuary subjected to strong tidal flows.

8.12 SUBTLE

SUBTLE (System Utilising Both Tape and Logger Equipment) is the interface between the analogue recording instrument and the computer. The system is composed of an analogue to digital converter, a solid-state memory unit,

a timing unit, and a frequency to voltage converter (Figure 8.14). It is fully self-contained and capable of taking any analogue output (voltage, current or resistance) from a field recording instrument and passing it onto the NHB PDP 11/23 computer in digital format. It is not essential that SUBTLE be used to read voltages from the wave probe. It could alternatively be linked with other recording instruments.

The system flow diagram is shown in Figure 8.15. The recording instrument may be linked either directly with the Tasman Mini Data Logger or via a Timing Unit for special applications. Both were manufactured by Solid State Equipment Ltd., N.Z. The Data Logger digitises the analogue input in the range of 0 to 255 using an 8-bit byte. The input is stored sequentially in its 32,000 byte memory. The instrument is capable of linking directly with a computer terminal RS232 interface, or with a cassette tape recorder. In the former case to view the data the logger will drive a standard computer terminal with no computer programming required.

In the latter case, the data is tone encoded using frequency shift keying (fsk); a 2400Hz tone represents a logical zero and a 1200Hz tone represents a logical 1.

As employed at Whangarei, the Data Logger is unmodified from its factory shelf specifications.

For the wave height recording, Solid State Ltd were asked to design and build a Timing Unit with the following capabilities:

(i) Intermittant input was required so that wave heights could be recorded for a short time at regular intervals. Specifically the logger was to come to life every 1-12 hours with 1 hour graduations to be set by the instrument user.

(ii) The standard scan rate of the instrument was too slow for wave height recording so scan rates of 0.5, 1, 2, 4 and 8 Hz were requested.

(iii) The instrument was to record a fixed number of data points before shutting down until the next scanning period. Two settings of 1024 and 2048 data points were required. These are multiples of two for subsequent Fast Fourier Analysis of the sea levels.

(iv) A separator code to identify the beginning of different scanning periods was needed. The company introduced a 255 followed by a 0, between data blocks, which is a physically unlikely combination.

The Timing Unit was set in the field to scan every 6 hours until 1024 points had been recorded at a frequency of 4Hz. With 32K of memory this allowed the instrument to be left unattended for just over 1 week. Both the Data Logger and Timing Unit are powered by a 9-volt rechargeable battery which was easily capable of maintaining the power requirements for the one-week period.

To give the Data Logger continuous residence in the field, data was

recorded at weekly intervals onto cassette tape. This necessitated construction of a frequency to voltage converter which was built from a circuit by Jordon and Schaare (1980). The converter has a built-in cassette tape recorder and the circuitry converts fsk tones to high and low voltages (±5 volts) suitable for computer recognition.

In the single user environment on the NHB 11/23, it was not necessary to start the tape recorder from the computer by holding a handshake line high, so the circuitry allowing remote start of transmission in the original design was not included. The system operates by running this writer's computer program designed to receive the data. Then the tape recorder is turned on manually. The circuit diagram for the frequency to voltage interface is given in Appendix 4.

As the incoming data in the range of 0 to 255, is viewed by the computer as a series of valid ASCII characters, some specialist programming was required to over-ride the computer's response to the treatment of control characters.

Furthermore, at the NHB installation, the Baud rate is not programmable so it was necessary to allow data to enter from a specific remote line while program control was maintained from the console terminal. The computer program (TAPINP) to perform these functions was written in FORTRAN for the Digital PDP 11/23 and is listed in Appendix 5.

A second program was written to allow formatted reproduction of the data at the line printer. Program PRBPRT takes the input file stored on disc, breaks it into scanning periods using the separator code, and dates each segment with user input entered at the keyboard. SUBTLE is complete at this stage.

8.13 Wave height probe specifications

The wave height probe is a capacitance-type which measures sea levels directly, as the water surface oscillates up and down its length (Figure 8.16). The instrument is a Robertshaw Capacitance Transmitter, Model 167 with a 6m long capacitance probe (740-B1-A236), and was used by this writer for wave height measurements on the south shore of Oahu, Hawaii (Black, 1978b).

The transmitter operates on 18VDC, and is powered in the field by two 12-volt car batteries regulated to an 18V fixed supply. The sensor is 6m long to allow for 3m tidal ranges at the deployment site with some higher or lower water levels occurring in the crests and troughs of the waves. This 6m length was calibrated over a usable span of 4.8m for field operations.

The probe is a 5mm steel rod with a Teflon insulating coating. It senses the immersion level from the varying capacitance between itself and a second capacitor plate running parallel with the probe and exposed to the fluid. The sensor outputs a varying current in the range from 4-20mA. The instrument requires a fixed load of 300 ohms, (which depends on the instrument voltage)

and therefore at 20mA, this gives an output maximum of 6.0 volts. The Data Logger digitises in the range of 0-2.55V so instrument sensing was done across one of two bridging parallel resistors, which scaled the voltage in the range of 0.5l-2.5l volts (for 4-20mA) over the 4.8m physical length of the probe. This gave a resolution limited by the logger of 0.024m, i.e. 4.8/200m where the Data Logger resolution is 0.0l volts.

8.13.1 Calibration tests

The linearity, long and short term stability and general probe operation were carefully tested in laboratory conditions before field deployment. A calibration tube was designed in collaboration with the NHB senior work shop engineer, Mr Frank Hicks. A 6m-tall, 0.15m-diameter, plastic sewerage pipe was fitted with suitable plumbing to allow adjustment of the water level. An adjoining parallel, clear-plastic tube of 0.25m diameter was backed with a steel tape measure for level readings. The probe was inserted into the tube and supported by a steel frame at the top. The calibration tube was established at NHB Works Headquarters and the instrument was serviced from an overhead platform.

Initial tests were required to select the second capacitor plate. The surface area of the second plate affected the magnitude of the probe's capacitance. A 5mm stainless steel rod was chosen for its surface area and strength which suited the field deployment needs discussed later. The probe was held to this rod with a series of rubber O-rings along its length and which had no measurable effect on probe operation.

Examination with an oscilloscope, revealed a small ripple on the probe output voltage which was only important because of the "instantaneous" sampling of voltage by the Data Logger. The amplitude of the ripple was seemingly dependent on radio interference and was reduced by placing a 0.70mF capacitor across the output selected by incrementing upwards until the ripple was reduced to about 0.005 volts peak to peak i.e. 50% of the limiting logger resolution.

8.13.1.1 *Linearity*

Figure 8.17 shows a probe calibration curve over a 4.8m span. The response is excellent with the voltages deviating by less than 1% of the full span over its entire length. The best fit curve has the expected gradient of 1.00 and correlation coefficient of 1.00. The calibration curve assuming linear changes between measured levels was incorporated into the computer program which converts voltages to sea levels, and thus the system's limiting accuracy would be due to Logger resolution rather than probe linearity. The probe accuracy is about 0.05m over the full length but is much better than this over a working length of about 0.5m during wave conditions.

8.13.1.2 *Stability*

With the capacitor across the output, the short-term stability is good. "Long" term stability in the laboratory was tested by leaving the probe at constant water levels for a 3 day period and no variation in voltage was evident. It remained constant at 1.65 volts throughout. An earlier test, over a 5-day period revealed maximum voltage changes of ±0.01 volts, which is 1% of the total span.

8.13.1.3 *Temperature sensitivity*

Sensitivity to water temperature (not atmospheric heating or cooling of the instrument transmitter) was tested by comparing voltages with water at 16°C and at 49°C. A full calibration revealed small differences of less than 0.03 volts throughout. This is a considerably wider temperature range than the probe would be subjected to in field conditions where water temperatures are between about 11 and 23°C from winter to summer.

8.13.1.4 *Salinity sensitivity*

When household salt (primarily NaCl) was added to the water, the voltage rose by 0.05 volts or 8% of the 0.65V reading. However, further additions of salt had no effect on the voltage. Thus, the probe needs to be calibrated on site and cannot be shifted from fresh to salt water without recalibration, but differences in salinity during the tidal cycle at Marsden Point of less than 4 parts per 1000 are not expected to affect the probe readings.

8.14 Field deployment

The probe was placed on a frame and two-pile structure designed by NHB Civil Engineer, Mr Mike Beazley, in collaboration with this writer (Figure 8.18). To reduce wave reflections, the piles were placed along the line of principal wave attack i.e. nearly north-south. The structure was topped with a platform supporting a weather-proof steel box holding the sensing electronics, including the Data Logger, Timing Unit, and two car batteries.

To further reduce wave reflections from the structure, the probe was supported from a steel frame projecting 2m out in front of the pile onto which it was bolted. The 0.005m stainless steel rod was threaded to allow application of stretching tension against the frame to reduce vibration under wave attack. All wave probe parts were electrically insulated from the steel frame. Further calibration tests with the probe on the frame raised and lowered by a mobile crane revealed the same degree of linearity as found in the calibration tube. During the period of field deployment, instrument management was taken over by Mr Beazley and Mr W. King with this writer acting in a consulting capacity. The field deployment location selected to monitor conditions at the proposed port site, is shown in Figure 8.19.

8.15 Filmed calibration

A Super 8 movie film was taken of waves passing the probe while simultaneously recording with the data logger. The filmed sea level was digitised on a Movie Editor which allowed examination of single frames. A graduated pole was placed near the probe for scale. Comparison of the two records revealed that the capacitor - used to smooth the output voltage - was causing some reduction in wave amplitude. Moreover, some of the very high frequency waves ($T \simeq 1s$) were being missed if amplitudes were large.

The probe was deployed with two configurations: (i) during the first three weeks of data collection using 2 parallel capacitors totalling 69.8mF and then (ii) using one smaller capacitor of 22.4mF. The filmed calibration showed that heights were under-estimated by approximately 50% with 2 capacitors and by 20% with one. The height statistics tabulated in the following sections have all be corrected using these calibration factors.

8.16 Future deployment

Future deployment of the probe would be made with two modifications:

(i) the probe and the stainless steel rod would be separated by a small gap of approximately 15mm, to facilitate easier cleaning and to reduce algal growth. The algae flourished along the probe's length where it touched the stainless rod and tended to retain sea water, particularly in the cold, damp, night airs, and some non-linear behaviour resulted.

(ii) the capacitor would not be used again. The influence of radio interference was found to be a second order error relative to the dampening of wave amplitudes with a capacitor across the output.

8.17 Field settings

The instrument was set to read 1024 data points at 4 points/second every 6 hours. The sampling frequency of 0.25 Hz was sufficient to prevent aliasing of the spectrum in the low-energy, short-period wave climate and the record duration was long enough to resolve the typical 3s wave periods after smoothing of the spectrum. Subsequent analysis revealed a poorly-resolved, longer wave with energy in the lowest frequency bands. The instrumentation provides a setting of 2Hz for these cases. However, the experiments were concerned with locally-generated harbour waves and a faster digitising interval was utilised. Accordingly, there is still some research which could be conducted to examine long waves in Whangarei Harbour. Their amplitude is often low but these waves will affect shipping because of their relatively large energy associated with the long wavelength.

8.18 Computer analysis

Sea level elevations were analysed using this writer's time series

analysis computer program (Black, 1977) written for a previous study of wave transformation over a coral reef (Black, 1978b). The program provides several options including: (i) Fast Fourier Analysis; (ii) Zero up-crossing analysis; (iii) Zero Up-crossing Spectrum (described later); (iv) fitting of the Weibull distribution to the spectrum, wave heights or periods; and (v) many general statistics such as height and period ratios. Several line printer plots are available including the distribution of the sea level elevations, the wave height and period distributions, the relationship of height to period; the Fourier Spectrum and the Zero Up-crossing Spectrum.

8.19 Zero up-crossing analysis

A zero up-crossing occurs when the sea level passes from below to above the mean level of the record. The zero up-crossing wave period is defined as the time between two consecutive zero up-crossings. The wave height is equal to the maximum minus the minimum sea level elevation between zero up-crossings. To accurately provide the time of the crossing with digital data, the program linearly interpolates between the defining sea levels. For the heights, parabolae are separately fitted to the highest and lowest three data points and the maximum and minimum occurs where the gradients of the parabolae are zero (Black, 1978b).

The significant wave height, $H_{Z,\frac{1}{3}}$, is defined as the average of the highest one third of the zero up-crossing waves. The significant period, $T_{HZ,\frac{1}{3}}$, is the average of the periods of these waves. This should not be confused with the average of the largest one third of the periods, $T_{Z,\frac{1}{3}}$, which takes no account of wave height.

8.20 Probe reliability and accuracy in field conditions

The probe transmitter functioned without requiring any maintenance for a ten-month deployment period from January until October, 1983 when field experiments were terminated. Considering the weather exposure of the instrument, the reliability is excellent.

The deployment site was close to the NHB tide gauge on the Marsden Point wharf which allowed direct instrument calibration for each 4-minute measurement period. It was found that algal growth on the probe influenced the voltage output. A weekly cleaning,when the logger is emptied, is sufficient to prevent excess fouling but, due to diver availability, this was not always possible. Accordingly, the mean level for each record was compared with the tide gauge and those which deviated from the probe calibration curve were not used. Mostly, this occurred near low tide at night. Being more often submerged, the bottom of the probe - particularly in the cold and damp night air - was more susceptible to this problem. Less than 10% of the total number of records were neglected for this reason and the others exhibited

good linear behaviour.

8.21 Results

Tables 8.5 and 8.6 provide the wave height and period statistics, tide and wind information for a 6-week measurement program in the period from 18-Aug-83 to 3-Dec-83. The wind data was obtained from the nearby anenometer on the NHB wharf situated at 13m above Mean Sea Level. During the wave measurement period, the anenometer was rotated by 90^{o} from the True Bearing while the wharf was being used for off-loading, so 90^{o} was added to all directions taken from the wind charts.

Relatively calm conditions prevail with wave heights being less than 0.04m and 0.05m which are the scaled sizes associated with minimum logger resolution. This correlates with the measured wind strengths which were often less than 5 $m.s^{-1}$ and mostly less than 10 $m.s^{-1}$ during the recording period. In the short fetches of the sheltered harbour, the sea state responds quickly to increased wind strengths and rough seas are spent on the harbour beaches soon after the wind abates. There is a clear correlation of the wave height to the concurrent wind strength.

The largest waves were measured under fresher winds of 14 $m.s^{-1}$ from 330^{o} T on 15-Sep-83 at 5:52. The significant height was 0.36m and maximum height was 0.54m and the significant wave period was 2.6s. The tide was ebbing and nearly low (0.9m above CD). The sea level time series is shown in Figure 8.20 and the short-period waves are clearly evident. The record exhibits some amplitude modulation at a beat period of about 70-90s. Furthermore, in view of the changing level of the troughs, there is apparently a long-period, low wave present in the record and this is later confirmed by spectral analysis.

Figures 8.21 and 8.22 show the sea levels 6 hours before and after the peak of the storm. There are some dramatic differences in the character of the sea state. The time series are now typified by longer waves with periods of about 12s and the leading faces are steep being indicative of a shoaling wave. During the recording period after the storm, the wind was from the west (280^{o} T) and the tide was flooding. Waves had to approach the probe opposed to the tidal flows causing them to shoal (Figure 8.22). Further to this, the current has apparently caused a redistribution of the wave energy to lower frequencies.

Surface gravity waves will interact with tidal currents and several factors play an influential role. Longuet-Higgins and Stewart (1961) showed that the wave system cannot propogate when the mean current exactly opposes the energy propogating speed of the wave system. In deep water, the group speed is given by,

$$c_g = gT/4\pi \qquad 8.4$$

Currents in the One Tree Point channel during Spring flood tides may attain, 1.2 $m.s^{-1}$ and therefore waves with periods less than 1.5s should be dissipated. The process in a tidal inlet is further complicated by refraction of waves subject to transverse current velocity gradients. In channelised tidal flow where currents are fastest in mid-stream, supporting flow bends the crests towards the channel banks and opposing currents cause energy to be concentrated in mid-channel. An analytical solution and numerical procedure to model this phenomenon is presented by Noda (1974). However, it is still not clear which mechanism is responsible for the shift of wave energy to lower frequencies noted above.

A second interesting case of changing wave conditions with the state of the tide occurs on the 1-Sep-83 (Table 8.6). At high tide (11:48) when the banks were submerged and the fetch was greater, the wave heights were much larger ($H_{z,\frac{1}{3}}$=0.23m) than at the lower tide 6 hours before and after. The wind was approximately from the north and waves had to pass across the Shoal Bay channel and the shipping channel as well as some of the shallow portions of Snake and McDonald Banks to reach the probe. Changes to tide level and current direction are seen to be of crucial importance on this route.

It is already evident at Whangarei that the wave generation process in a confined estuary subject to moderate tidal currents is extremely complex. Prediction of heights cannot be easily undertaken and no substitute can be made for in situ measurements, at least to allow for "calibration" of the region in question.

Statistics for the 10 largest wave events have been taken from Tables 8.5 and 8.6 and compiled in Table 8.7 for comparison. They correspond with wind speeds exceeding 8 $m.s^{-1}$. The wind directions mostly coincide with the widest "open window" from One Tree Point to Darch Point or the second window through the entrance from 80° to 110° T (Figure 8.19). Some of the angles are just outside this range but as the wind direction oscillates about its mean value and as the tidal flows are capable of transporting the waves this is not unlikely.

A good example is the case on 20-Aug-83 at 7:48 (Table 8.7). The wind direction was 50° T which is between the two windows and coming from Reotahi Bay. However, the current was near maximum ebb at this time and could have transported waves towards the throat from further upstream. To demonstrate this a simplified example is offered. If the wave celerity is estimated as $c = gT/2\pi$ then for 2s waves, $c \simeq 3.1$ $m.s^{-1}$. The fetch from Munro Bay to the southern side of the harbour is about 4400m at high tide. The 2s waves would take approximately 1400s to travel this distance. In this time, with typical currents of 1 $m.s^{-1}$ flowing almost perpendicular to the course, the waves would be transported down-harbour a distance of about 1400m. This distance is large enough for waves generated across McDonald Bank with a

wind from 50°T to reach the wave probe on an ebb tide.

Table 8.8 compiles the wave heights etc. for the cases of highest wind speed - exceeding 7 $m.s^{-1}$ and excluding the 10 major events. Even under windy conditions small waves occur when the wind blows off the land near the probe e.g. 26-Aug-83 at 11:53 where wind speed was 8 $m.s^{-1}$ from 170°T and $H_{z,\frac{1}{3}} = 0.05m$.

On 12-Sep-83 at 11:48, the wind speed was 8 $m.s^{-1}$ from 40°T and $H_{z,\frac{1}{3}}$ was only 0.04m. This compares with the major event when winds of the same strength from 50°T were recorded and $H_{z,\frac{1}{3}}$ was much larger at 0.17m. In the small-wave case, the tide was full and currents would not be expected to transport waves generated across the longer fetch from Munro Bay as detailed above. Again, this highlights the importance of wave/current interaction in the harbour.

The most unexpected result in Table 8.8 is the small wave heights which occur under high winds from 320-340°T. Snake Bank and the raised ebb shield which weaves across it are situated on this bearing. Evidently, it offers some shelter from this direction, particularly at low tide. However, it is noted that two of the major wave events were from this general direction and the results are inconclusive.

8.22 Comparison with empirical wave generating relationships

The measured wave heights were compared with three semi-empirical wave generating relationships. These are the Liu (1971), JONSWAP (Hasselmann et. al., 1973) and Sverdrup, Munk and Bretschneider (Bretschneider, 1958) relationships incorporated into this writer's computer program called WIND (Black and Healy, 1981).

For the very small fetches in the harbour, the Liu spectrum grossly over-estimated the wave heights and was not further examined. The other two are compared with the measured significant wave height and period on Table 8.9 and the measured fetches are shown in Figure 8.19. In general there are some serious discrepencies with the empirical equations mostly over-estimating by about a factor of two. The only reasonable prediction occurs on 15-Sep-83 at 5:52. This also coincides with the smallest fetch measured between the probe and the ebb shield on Snake Bank and, in view of the inconclusive results in the previous section, this may not be acceptable.

Figure 8.23 shows the Fourier spectrum for this case. The Fourier analysis was undertaken with 1024 data points and 512 spectral estimates were returned. These were smoothed by band averaging with no overlapping (Black, 1977). Using 16 spectral estimates, the smoothed spectrum has 32 degrees of freedom. The low frequency energy in the spectrum seen in Figure 8.23 is not associated with local wind waves. Moreover, it is not a linear trend in the record due to changing tide level during the recording period,

as this is filtered before spectral analysis using a linear detrend described in Black (1977).

When compared with the JONSWAP spectrum (Figure 8.23), the prediction of peak frequency is reasonable. However, the JONSWAP spectrum is too steep having an f^{-5} tail. In the actual case, energy is smeared across a wider frequency range.

It is apparent, that in a confined waterway such as Whangarei Harbour, an "effective" fetch must be defined for each wind direction, tidal stage and channel flow velocity before an accurate prediction can be made.

8.23 Statistical analysis

A statistical analysis of wave parameters was described by this writer in Black (1978b). The work was undertaken on a coral reef on the south shore of Oahu, Hawaii which is subject to long, low ocean swell often generated several thousand kilometres from the experimental site. In Whangarei Harbour, the environment is totally different. Waves are generated over a very short fetch with rapidly changing bathymetry while strongly interacting with channelised tidal currents exhibiting large lateral gradients. It is of extreme interest to ascertain if the wave statistics and other experimental results over the coral reef are applicable to the estuarine environment.

8.23.1 Height, period and spectrum

Table 8.10 compiles the ratios of wave heights and periods for five selected records chosen for the variable character of the sea level time series. These are compared with the Hawaii results. Some of the ratios exhibit scatter which cannot be rationalised without further data. However, some are quite stable and most notable of these is the significant period over the average of the largest one third averaging 0.77. The comparison with the Hawaii data is favourable. Whangarei Harbour exhibits best correlation with the reef environment rather than the swell conditions offshore. Notably, there is a distinct difference in the frequency of the spectrum peak for the two environments but this evidently does not always influence the height and period ratios.

Over the coral reef, the wave height and period distributions were described by the Weibull 2-dimensional probability density function given by,

$$f(X) = \alpha\beta X^{\beta-1} e^{-\alpha X\beta} \qquad 8.5$$

where X may be height or period. α is a measure of the mean of the distribution and β describes the peakedness. Best fit curves to the Weibull distribution at Whangarei using the method presented in Appendix 3, supported the coral reef results. The correlation coefficients representing the goodness of fit in the 5 height and period cases always exceeded 0.92.

As the density function is known, the height and period ratios may be

predicted knowing the peakedness, β, only. The exact solutions for the ratios are derived in Black (1978b). Figure 8.24 shows two examples, ($X_{z,\frac{1}{3}}/X_z$ and $X_{z,\frac{1}{10}}/\bar{X}_z$) plotted against β with the Whangarei field data included. There is clearly good correspondence and given any of the ratios, the complete probability density function may be ascertained or alternatively, the density function will provide the height or period statistics. Unfortunately, with the limited data base, prediction of the density function peakedness for particular wind and tide conditions is not possible. It is important to note, however, that the heights are not Rayleigh distributed which is a special case of the Weibull distribution with $\beta = 2$.

Generally, the period distribution is less peaked than the heights. This trend is supported by results of Bretschneider (1959) who found that the period squared was approximately Rayleigh distributed which corresponds with $\beta = 1$ compared with an average of 1.58 at Whangarei and 1.67 on the coral reef. The zero up-crossing wave periods may universally exhibit a Weibull distribution with peakedness averaging approximately 1.6, although there is scatter in the individual cases.

Table 8.11 lists the skewness and kurtosis of the distribution of instantaneous sea levels for the 5 selected records each containing 1024 data points. They all exhibit positive skewness having tails to the right similar to sea level elevations for shoaling waves which typically have long shallow troughs and narrow high crests. Once again, the average at Whangarei of 0.94 corresponds with the reef environment rather than the deeper swell conditions offshore.

The mean frequency of the spectrum is defined as,

$$f_{m01} = 1/T_{m01} = m_1/m_0 \qquad 8.6$$

where m_0 and m_1 are the zeroth and first spectral moment. The mean period is,

$$T_{m02} = (m_0/m_2)^{\frac{1}{2}} \qquad 8.7$$

where m_2 is the second spectral moment. These periods relate the spectrum to the zero up-crossing statistics in the ratios shown in Table 8.11. There is some considerable scatter in both but $T_{Hz,\frac{1}{3}}/T_{m01}$ has an average of 1.49 which is almost equal to the Hawaii reef value of 1.47.

Goda's (1970) spectral peakedness, Q_p, is defined as,

$$Q_p = \frac{2}{m_0^2} \int_0^\infty f\, S(f)^2\, df \qquad 8.8$$

and it may be shown (Black, 1978b) that for a spectrum with the shape of the Weibull distribution,

$$Q_p = \beta/2 \qquad 8.9$$

The well-known Bretschneider spectrum is Weibull distributed with $\beta = 4$, and thus $Q_p = 2$. Compiled in Table 8.11, the values of Q_p for the five cases

at Whangarei suggest that the spectrum is much less peaked than the Bretschneider spectrum with β varying from 0.74 to 2.44. Some of the low values occur because of the high energy found in low frequency ranges and these multiple peaked spectra are not well described by any of the well-known empirical relationships. Only the spectrum shown in Figure 8.23 for the 15-Sep-83 at 5:52 has a well defined peak in the frequency range most suited to the time series digitising interval and record duration. This spectrum has $\beta = 2.44$ and the normalised Weibull-shaped empirical spectrum for $\beta = 2$, is then,

$$\frac{S(f)}{S(f_p)} = \left[\exp(\frac{3}{2})\right] \quad (f/f_p)^{-3} \quad \exp\left[-\frac{3}{2}\,(f/f_p)^{-2}\right] \qquad 8.10$$

where $S(f)$ is the spectral density and f_p is the peak frequency. This corresponds with a design spectrum presented by Lee and Black (1978) for shallow water ($\simeq$ 2m) on the offshore side of the coral reef (Figure 8.25). As noted graphically, this is wider than the JONSWAP spectrum and has an f^{-3} tail.

8.24 Zero up-crossing spectrum

A procedure to generate a spectrum from the zero up-crossing waves was introduced in Black (1978b). Called the zero-up-crossing spectrum (ZUS), the method has some advantages over the traditional procedure. The Fourier spectrum is composed of multiple sine waves fitted to the record to recreate the time series. Interpretation of these components as monochromatic superimposed wave trains can be misleading especially when waves are shoaling in shallow water.

The ZUS is defined as,

$$S(f) = g(f) \cdot \frac{1}{8} H^2_{rms}\ (f) \qquad 8.11$$

where $g(f)$ is the probability that the frequency f will occur in the record and H_{rms} is the zero up-crossing RMS wave height. For the discrete case,

$$S(f) = \frac{\sum_{i=i}^{m} H_i^2(f)}{8.N\Delta f} \qquad 8.12$$

where Δf is the frequency interval and includes waves with frequencies from $f - \Delta f/2$ to $f + \Delta f/2$, N is the number of waves in the record; m is the number of waves each of zero up-crossing height H_i and with frequencies that lie in the interval defined above. The energy in a band is large if many waves fall in that band and/or if the amplitude of those waves is large. Thus, the ZUS has a direct correspondence with the individual waves in the record.

Over the coral reef, the ZUS was shown to approximate the Fourier spectrum but mostly it allotted energy into a narrower frequency range.

This occurred because the zero up-crossing procedure isolates the dominant frequency in the record and will always neglect low-amplitude long-period oscillations.

The ZUS was constructed for the highest wave event at Whangarei and plotted with the Fourier spectrum in Figure 8.26. As expected, the low frequency component is not evident in the ZUS. However, there is good correlation at the peak with the estimates of peak frequency and amplitude being essentially equal. The definition of the ZUS is evidently applicable to the estuarine environment.

8.25 Conclusions

(i) A field investigation of the upper platform of the inter-tidal zone at the proposed port site was undertaken to quantify the sediment transport fluxes there. An initial examination of the large scale bedforms evident on aerial photographs revealed 2-dimensional transport patterns and sediment was not simply "flowing" up-harbour along the inter-tidal zone. Sediment trapping experiments offered good confirmatory evidence of this. Nett flux directions are summarised in Figures 8.1 to 8.3 and Figure 8.6.

(ii) Flow velocity measurements revealed that the inter-tidal zone is ebb dominant but mostly currents are below the sediment threshold. Bedload trapping experiments showed that sediment is only transported in significant quantities when surface gravity waves stir the bed sediments.

(iii) Sediment trap catches in storm conditions were as large as 411 kg/m/tidal cycle. After an approximate storm frequency weighting was applied, the annual transport rates on the upper platform were estimated as being in the range from 2-10/m^3/m/year and resultant rates were only 1 m^3/m/year. It is therefore evident that the proposed timber port will have very little influence on its environment arising out of changes to sediment transport patterns on the upper platform.

(iv) A computerised data acquisition system was established. Named SUBTLE, the system enables data recording, storage and subsequent transfer onto computer without key punching. It may be interfaced with an analogue or digital field recording instrument and left unmonitored for extended periods. The system is composed of an Analogue to Digital converter with a solid state memory unit, a timing unit, and a frequency to voltage converter and is summarised in Figure 8.15.

(v) A wave height recorder was established at Marsden Point. The wave height probe was of the capacitance type which makes direct measurements of sea level in response to its immersion in the fluid. The data acquisition system, SUBTLE, was employed in this application.

(vi) Over a 6-week recording period, the wave heights at Marsden Point were often small. Over the short fetches in the sheltered harbour, the local

waves respond quickly to wind changes. The maximum significant height recorded was 0.36m occurring during a storm with winds of 14 $m.s^{-1}$ from 330^{o} T.

(vii) Semi-empirical wave prediction formulae were found to be inadequate in the harbour environment due to wave/current interactions and the changing geometry of the inlet with tidal stage. The JONSWAP and Bretschneider spectra were too peaked as energy was smeared across a wider frequency range in the inlet. An empirical normalised spectrum with the shape of the Weibull distribution is presented as equation (8.9), to describe the waves on the outer edge of the tidal flat.

(vii) A statistical analysis of wave height and period ratios showed some good correlation with a similar study over a coral reef on the south shore of Oahu (Black, 1978b). The estuary resembled the reef more than an offshore location subjected primarily to swell conditions.

(ix) The Weibull distribution described the wave height and period distributions with correlation coefficients exceeding 0.92 in 5 test cases. The period distribution was less peaked than the heights with β averaging 1.58 compared to 2.39.

(x) The distribution of sea level elevations was positively skewed which is typical of shoaling waves.

(xi) The product of the significant wave period and the mean frequency of the spectrum averaged 1.47 corresponding with 1.49 obtained in the coral reef study.

(xii) The Zero Up-crossing Spectrum (ZUS) (Black, 1978b) was found to be applicable to the estuarine environment.

SEDIMENT TRANSPORT AND TIDAL INLET HYDRAULICS

Chapter 9:

ONE-DIMENSIONAL, NUMERICAL HYDRODYNAMIC MODEL

CHAPTER 9: ONE-DIMENSIONAL, NUMERICAL, HYDRODYNAMIC MODEL

9.1 Introduction

As emphasised previously, one of the major themes of this thesis is to assess the impact of a port development at Marsden Point. In modern research, computer modelling has evolved as a valuable tool in such applications (DHI, 1982). They are capable of generating information that may take months or even years to measure in the field and can be used to predict estuarine hydrodynamics after port construction, for example. There is, therefore, a growing acceptance of these models as an indicator of coastal and estuarine behaviour.

Accordingly, a one-dimensional, hydrodynamic numerical model was developed to simulate the behaviour of tidal flows, and applied to Whangarei Harbour. The results presented in this Chapter assert the usefulness and reliability of such models, concurring with similar reports from many scientific communities throughout the world (Hansen, 1956; Leendertse, 1967; Heaps, 1969; Abbott et. al., 1973; Tee, 1976; Falconer, 1980; and Ramming and Kowalik, 1980).

Models may describe density gradients, convective and local accelerations, wind stress, bed friction, sea slopes and additional inflows or discharges. Furthermore, the hydrodynamic equation may be indefinitely extended to include aspects that were not felt to be of primary importance at Whangarei e.g. barometric pressures, vertical salinity gradients etc. Modelling in one dimension has limitations as tidal flow is a three-dimensional process. However, in locations where an application is not concerned with variations across the flow, but with a description along the length of a channel, the 1-d model can be effectively applied, to ascertain mean flow velocities. A further important application at Whangarei arose out of computer memory limitations. The harbour was schematised in one dimension and sea levels from the 1-d model generated open boundary conditions for a 2-d model(described in Chapter 10) of a smaller region where greater detail was required.

9.2 List of symbols

A	cross-sectional area (m^2)
A_{CD}	cross-sectional area below Chart Datum (m^2)
b	cross-section width (m)
b_{CD}	cross-section width at Chart Datum (m)
b_{HW}	cross-section width at high water (m)
C	Chezy's C ($m^{\frac{1}{2}}.s^{-1}$)
C_{den}	longitudinal density gradient coefficient (s^{-2})
d	water depth below a horizontal datum (CD) (m)
$\bar{d}$	mean cross-section depth (m)
d_{eff}	effective depth (m)
d_{max}	maximum depth at model cross-sections (m)
g	gravitational acceleration (9.81 $m.s^{-2}$)
G	gradient of sloping harbour side walls
h	distance between velocity and sea level nodes (m)
j	subscript denoting location in space
k	sea bed roughness (m)

l	superscript denoting time
q	additional inflow ($m^3.s^{-1}$); inflow per unit length ($m^2.s^{-1}$)
Q	average discharge over a half tidal cycle ($m^3.s^{-2}$)
r	friction function
R	friction function; hydraulic radius (m)
t	time (s)
T	model time step (s)
$T_{½}$	half tidal cycle period (s)
$\underline{u}$	water velocity ($m.s^{-1}$)
u	mean flow velocity through a cross-section ($m.s^{-1}$)
U_{max}	maximum flow velocity during a half tidal cycle ($m.s^{-1}$)
W_z	wind speed at z metres above the surface ($m.s^{-1}$)
W_{10}	wind speed at 10m above the surface ($m.s^{-1}$)
x	longitudinal channel coordinate (positive in the flood direction)
z_o	roughness length (m)
α	velocity distribution coefficient
γ_{10}	coefficient of aerodynamic resistance
ρ	fluid density (1025 $kg.m^{-3}$)
ρ_a	air density (1.29 $kg.m^{-3}$)
Ω	tidal prism (m^3)
τ_x	surface wind stress ($N.m^{-2}$)
ξ	sea surface elevation above a horizontal datum (CD) (m)
ξ_{HW}	sea surface elevation above CD at high tide (m)

9.3 The hydrodynamic equations

The one-dimensional hydrodynamic equation for tidal flow is,

$$\frac{\partial u}{\partial t} + \frac{u\partial u}{\partial x} = - g \frac{\partial \xi}{\partial x} - \frac{g|u|u}{C^2R} + \frac{\tau_x}{\rho R} - \frac{gR}{2\rho}\frac{\partial \rho}{\partial x} \qquad 9.1$$

where $u(x,t)$ is the doubly-averaged mean velocity in the cross-section at x and at time t, ξ is the sea level above a horizontal datum, ρ is the fluid density, g is the gravitational acceleration, C is Chezy's C, R is the hydraulic radius, and τ_x is the surface stress induced by the wind. The normal practice of defining the up-harbour or flood direction as positive will be adhered to.

The left-hand side of the equation gives the acceleration force, both local and convective, and the right-hand side gives the forces driving or retarding the flow due to the slope of the water surface, the bed frictional resistance, the wind and longitudinal density gradients. Not included are terms for barometric pressure differences and Coriolis forces. The latter acts at right angles to the flow and cannot be represented in one-dimension.

The equation for conservation of volume or the continuity equation for a one-dimensional region is,

$$\frac{\partial (Au)}{\partial x} + b\frac{\partial \xi}{\partial t} - q = 0 \qquad 9.2$$

where A is the cross-sectional area, b is the stream storage width, and q is a supplementary inflow per unit length due to flow convergence or branching. Notably, q may include discharge. However, as no extra term has been included in the momentum balance equation as presented, the inflow or discharge must have zero longitudinal velocity.

Excluding density gradients, these equations are presented in Dronkers (1969), with the hydraulic radius replacing the total water depth. The density gradient term is as presented by Knight (1981) but again the hydraulic radius replaces the total water depth.

There are several possible refinements which may be important depending on the region to be modelled.

(i) the convective acceleration is a momentum term which arises out of the derivative of the velocity squared. In a one-dimensional model the velocity has been averaged twice, i.e. averaged in the vertical and then throughout the cross-section and is the overall mean speed. In tidal inlets, the velocity varies across the section, being greatest near the deepest location and thus, the non-linear convective term may need modification. A velocity distribution coefficient (Abbott, 1979, p. 41) can be applied which is defined as,

$$\alpha = \frac{1}{u^2 A} \int_A u^2 dA = \frac{A}{Q^2} \int_A u^2 dA \qquad 9.3$$

where the discharge, Q, is Q = uA.

When written in terms of discharges, and ignoring wind stress and density gradients, the momentum balance equation is then,

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\alpha \frac{Q^2}{A}\right) = - gA \frac{\partial \xi}{\partial x} - \frac{g|Q|Q}{C^2 AR} \qquad 9.4$$

The velocity distribution coefficient relies on a good approximation to the integral of the square of the velocity and in field cases, this is not always possible, and an approximate constant is introduced.

(ii) A second refinement also relates to the shape of the cross-section. Discharges are not necessarily equal when the cross-sectional areas are of the same magnitude. The discharge is also proportional to the mean depth or hydraulic radius in the section, as the flow retardation due to bed friction is strongly dependent on the water depth. For example at Whangarei the Shoal Bay and Shipping channels have similar areas (Chapter 3) but due to the greater depth in the shipping channel it would be expected to carry more flow.

Ramming (1971) (reported in Ramming and Kowalik, 1980) presented a method to model this in one-dimension which divides the cross-section into a series of unequal-width vertical strips. The discharge through each of these, which varies with depth according to the channel morphology, is calculated and total cross-section discharge is the sum of the components. Ramming found much improved correlation with the tidal curve in the River Eider, when this refinement was incorporated into the model. Without the refinement he was not able to reproduce the steep part of the tidal curve during the flood, because the complex geometry of the channel distorted the friction and non-linear convective terms. This occurs particularly in very shallow locations

(Ramming and Kowalik, 1980).

A simplification is possible by considering the cross-section to be composed of only two parts: an "effective" and a "reservoir" region. In typical channels, most of the flow passes through the deepest effective sections. On the shallow flanking platforms, velocities are low and these act mainly as storage reservoirs. Some schematisation may be performed to incorporate the two compartments into the model which relies mainly on adjustment of the mean depth in the resistance term. In the model to be described, an approximation to the hydraulic radius in a simulated estuary with sloping side walls is employed to introduce a channel shape dependence into the bed friction term.

9.4 Orders of magnitude

In Whangarei Harbour the characteristic depths, lengths and velocities are 10m, 1000m and 1 $m.s^{-1}$ respectively. The orders of magnitude of the terms in the momentum balance equation are then,

$$\frac{\partial u}{\partial t} = \frac{\Delta u}{\Delta t} = \frac{1}{3 \times 60 \times 60} = 1 \times 10^{-4}$$

$$u\frac{\partial u}{\partial x} = u\left(\frac{\Delta u}{\Delta x}\right) = \frac{1 \times 0.1}{1000} = 1 \times 10^{-4} \qquad 9.5$$

$$g\frac{\partial \xi}{\partial x} = 10 \times 10^{-5} = 1 \times 10^{-4}$$

where the sea slope is typically 10^{-5} (Smith-McNaught, 1976).

$$\frac{g|u|u}{C^2R} = \frac{10 \times 1 \times 1}{60^2 \times 10} = 3 \times 10^{-4} \qquad 9.6$$

Thus, the convective momentum and frictional retardation terms are of importance to the harbour hydrodynamics and both were included in the model. The effect of the longitudinal density gradients associated with salinity variations is,

$$\frac{gd}{2\rho}\frac{\partial \rho}{\partial x} = \frac{10 \times 10}{2 \times 1000} \times \frac{1}{1000} = 5 \times 10^{-5} \qquad 9.7$$

which is an order of magnitude less than the frictional retardation.

Winter salinity between Port Whangarei and Marsden Point decreases from 34‰ at the ocean end by about 8‰ (Smith-McNaught, 1976; Millar, 1980). This is a distance of 16,000m so over the characteristic length, the change is 0.5‰ /1000m. Thus the density change of 1 $kg.m^{-3}$/1000m may be a factor of 2-3 too high and the density gradient term lies in the range (1-5) x 10^{-5}. In a tidal inlet with the highest salinity normally at the ocean end, the salinity gradient is directed down-harbour so that it accelerates the flood and slows the ebb currents.

9.5 Implicit or explicit?

The hydrodynamic and continuity equations can be solved numerically with either an explicit or implicit solution. The explicit-scheme requires less computation for solution per time step, but the time steps are limited by a stability criterion. In the implicit case, the solution may be unconditionally stable but consideration of the physcial process to be modelled usually leads to a time step limitation as well, and a trade-off between computation time and the minimum time step may result.

Furthermore, both schemes have been applied successfully to field applications (Dronkers, 1969; McDowell and O'Connor, 1977). Dronkers found good agreement within the accuracy of the field observations between the modelled tides and discharges with both bypes of solution. Thus, in view of the reported successful applications, this writer decided that on the limited-memory, NHB PDP 11/23 computer an explicit scheme was more suitable.

The purpose of the modelling was not to make comparative assertions about the relative merits of the two types of approach. The writer was fortunate, however, that the Danish Hydraulic Institute also modelled Whangarei Harbour with their 2-d System 21, mixed-implicit model and comparisons could be made with this writer's 2-d model, by examination of their relative abilities to reproduce the measured water speeds. This is discussed in Chapter 10, and although the implicit methods have been preferred by many researchers, the modelling at Whangarei shows excellent correlation with the DHI model and field measurements using explicit solutions.

9.6 An explicit scheme

Dronkers (1969, p. 46) presented an explicit scheme which he described as the "well-known finite difference scheme found in Hansen (1956)". The finite difference representation with density gradients included becomes,

$$\frac{u_{2j}^{2\ell+1} - u_{2j}^{2\ell-1}}{2T} = \frac{-u_{2j}^{2\ell+1}}{4h}\left(u_{2j+2}^{2\ell-1} - u_{2j-2}^{2\ell-1}\right)$$

$$- \frac{g}{2h}\left(\xi_{2j+1}^{2\ell} - \xi_{2j-1}^{2\ell}\right)$$

$$- \frac{g\,|u_{2j}^{2\ell-1}|\,u_{2j}^{2\ell+1}}{(C_{2j}^{2\ell})^2 R_{2j}^{2\ell}}$$

$$- \frac{g}{2\rho}\frac{\partial \rho}{\partial x} \cdot R_{2j}^{2\ell}$$

$$- \frac{\tau_x}{\rho R_{2j}^{2\ell}} \qquad 9.8$$

and the continuity equation is,

$$\frac{\xi^{2\ell+2}_{2j+1} - \xi^{2\ell}_{2j+1}}{2T} = \frac{1}{h\,(b^{2\ell}_{2j+2} + b^{2\ell}_{2j})} \left[u^{2\ell+1}_{2j+2} A^{2\ell}_{2j+2} - u^{2\ell+1}_{2j} A^{2\ell}_{2j} \right]$$

$$+ \frac{1}{4h} \left[\frac{q_{2j+2}}{b^{2\ell}_{2j+2}} + \frac{q_{2j}}{b^{2\ell}_{2j}} \right] \qquad 9.9$$

The subscript, j, denotes locations in space and superscript, ℓ, denotes time levels. The grid defining the notation is shown in Figure 9.1. In space, the depths and velocities are located at alternative nodes. In time, all velocities are found from the momentum equation (9.8). Then at the second half-time step, sea levels are calculated with the continuity equation (9.9). This leap-frog process is continued throughout the modelling period.

Where possible, the velocities in equation (9.8) correspond with the current time step. Only the partial derivative in the convective acceleration term and the modulus of speed in the frictional resistance term, use speeds from the prior time step. The scheme then, may be described as semi-explicit. However, as the solution for velocity and sea level requires no iteration and may be obtained explicity it shall be described simply as "explicit".

The introduction of updated velocities has an important repercussion, (Ramming and Kowalik, 1980). For the case of flow subject only to a forcing sea surface gradient and a retarding friction loss, the momentum equation simplifies to become,

$$u^{\ell+1} = u^{\ell} \left(1 - \frac{gT|u|}{C^2 R}\right) - gT \frac{\partial \xi}{\partial x} \qquad 9.10$$

which shows that numerical stability is only preserved if,

$$1 - \frac{gT|u|}{C^2 R} > 0 \qquad 9.11$$

In shallow water as $R \to 0$, this may be violated. However, with the friction term in an implicit form,

$$u^{\ell+1} - u^{\ell} = \frac{-gT\, u^{\ell+1} |u|}{C^2 R} - gT \frac{\partial \xi}{\partial x} \qquad 9.12$$

then,

$$u^{\ell+1} = \left[u^{\ell} - gT \frac{\partial \xi}{\partial x} \right] / \left[1 + \frac{gT|u|}{C^2 R} \right] \qquad 9.13$$

and numerical stability is always guaranteed. Behaviour in very shallow water as $R \to 0$ is discussed in Section 9.12.2. Collection of updated velocity terms in equation (9.8) produces,

$$u_{2j}^{2\ell+1} = \{u_{2j}^{2\ell-1} - \frac{gT}{h}(\xi_{2j+1}^{2\ell} - \xi_{2j-1}^{2\ell}) - \frac{-gT}{\rho}\frac{\partial\rho}{\partial x} \cdot R_{2j}^{2\ell} + \frac{2T\tau_x}{\rho R_{2j}^{2\ell}}\} /$$

$$\{\frac{2gT|u_{2j}^{2\ell-1}|}{(C_{2j}^{2\ell})^2 R_{2j}^{2\ell}} + \frac{T}{2h}(u_{2j+2}^{2\ell-1} - u_{2j-2}^{2\ell-1}) + 1\} \qquad 9.14$$

In the continuity equation (9.9), the cross-section width is defined at the velocity nodes, and not at the sea level nodes as used by Dronkers (1969). This was made necessary after introduction of variable cross-section width to improve the representation of the profile shape. The cross-section width at the sea level nodes is found from the average of the bounding widths at the velocity nodes. The plan shape of the modelled estuary therefore varies linearly between velocity nodes.

The inflow, q, has no time dependence and is constant throughout the modelling period at each velocity node. This was enforced by computer memory limitations but introduction of q was a time series would not be difficult to program. As given in the continuity finite-difference equation (9.9), q is a volume inflow per unit time ($m^3.s^{-1}$) into any of the sections and is not an inflow per unit length. This made it easier for the modeller to describe the input data, by removing the dependence on the distance between nodes.

The inflow or discharge occurs at the velocity nodes and is assumed to disperse equally over the region between bounding sea levels with the velocity node at the centre. Thus, the change in sea level in the continuity equation depends on the section width, b, in a compartment which has surface area, 2hb. The changes at central sea level nodes is defined as the average of the changes due to inflow or discharge in bounding compartments.

9.7 Hydraulic radius, cross-section width and area

In a tidal inlet, as the tide rises the cross-sectional area and width both increase, due to the flooding of sand banks and inter-tidal flats. It would be possible to model this flooding process using actual cross-section morphology input for the computer to assess the relevant parameters with tidal stage. Alternatively, the flooding may be approximated by using an average cross-section width then assuming that the tidal inlet is vertically-walled and width remains constant throughout the tide. A compromise between these options was taken. The cross-section was schematised as being rectangular below CD then having sloping walls above this level as shown in Figure 9.2.

The modeller is required to ascertain the cross-section width at CD, the area below CD, and to enter the gradient of the sloping walls for each cross-section. The gradient is defined as,

$$G = \frac{b_{HW} - b_{CD}}{2.\xi_{HW}} \qquad 9.15$$

where b_{HW} and b_{CD} are the cross-section widths at high tide and at CD respectively, and ξ_{HW} is the high tide level above CD.

The width at any tidal stage is,

$$b = b_{CD} + 2G\xi \qquad 9.16$$

and the cross-sectional area is,

$$A = A_{CD} + b_{CD}\xi + G\xi^2 \qquad 9.17$$

The mean depth in the cross-section is,

$$\bar{d} = A/b \qquad 9.18$$

If G is set to zero, the tidal inlet has vertical walls so that width is constant and area increases as expected. This procedure improves the representation of the tidal inlet over the vertical-walled type, giving both the cross-section width and area a time dependence.

In a tidal inlet where the width is much greater than the depth, the hydraulic radius is very nearly equal to the mean depth. Accordingly, the hydraulic radius in the momentum balance equation was approximated with the mean depth, given by equation (9.18). The sea level at a velocity node is defined as,

$$\xi^{2\ell}_{2j} = \frac{1}{2}\left(\xi^{2\ell}_{2j+1} + \xi^{2\ell}_{2j-1}\right) \qquad 9.19$$

Then the width of the section is,

$$b^{2\ell}_{2j} = b_{CD2j} + 2G_{2j}\,\xi^{2\ell}_{2j} \qquad 9.20$$

and the cross-sectional area is,

$$A^{2\ell}_{2j} = A_{CD2j} + b_{CD2j}\,\xi^{2\ell}_{2j} + G_{2j}\,(\xi^{2\ell}_{2j})^2 \qquad 9.21$$

The hydraulic radius is then,

$$R^{2\ell}_{2j} = A^{2\ell}_{2j}/b^{2\ell}_{2j} \qquad 9.22$$

Because of the sea level changes during the tide, the cross-sectional width and area and the hydraulic radius are all functions of time. For correct representation of the stream width (Abbott, 1979), the area and sea level should be related such that,

$$\frac{\partial A}{\partial t} = b\frac{\partial \xi}{\partial t} \qquad 9.23$$

with equation (9.21),

$$\frac{\partial A}{\partial t} = b_{CD}\,\frac{\partial \xi}{\partial t} + 2G\xi\frac{\partial \xi}{\partial t} \qquad 9.24$$

and,

$$b = \frac{\partial A}{\partial t} / \frac{\partial \xi}{\partial t} = b_{CD} + 2G\xi \qquad 9.25$$

as given by equation (9.20). b is often called a storage width.

Determination of the sea level at velocity nodes by averaging bounding values is very similar to a method suggested by Leendertse (1967) to help prevent short waves accumulating at the shortest wavelengths of 2h. Energy which physically cascades from large to small scales, by non-linear interactions, cannot be transferred to scales smaller than twice the grid size. For this reason oscillations accumulate and after some time may completely swamp the numerical solution (Falconer, 1980). In this writer's model, no artificial-viscosity term to smooth these non-linear oscillations was included in the finite difference scheme and yet instabilities were not noted. Leendertse (1967, p. 34) states that "feedback can be eliminated if the finite-difference expression for the non-constant parameter is chosen in such a manner that it does not represent waves with wavelength of 2h." He suggested approximating the continuity equation,

$$\frac{\partial (d + \xi)u}{\partial x} + \frac{\partial \xi}{\partial t} = 0 \qquad 9.26$$

where d is depth below a datum, as,

$$\frac{1}{4h} \left\{ \left[d_{2j+2} + \frac{1}{2} (\xi_{2j+3} + \xi_{2j+1}) \right] u_{2j+2} \right. \qquad 9.27$$

$$\left. - \left[d_{2j} + \frac{1}{2} (\xi_{2j+1} + \xi_{2j-1}) \right] u_{2j} \right\} + \frac{\partial \xi}{\partial t} = 0$$

By utilising mean sea levels to find the cross-sectional area, this procedure has been effectively incorporated into the finite difference equations.

9.8 Stability criterion

The stability criterion for the scheme was derived by Dronkers (1969) and was found to be,

$$\frac{h}{T} > \left(\frac{gA}{b}\right)^{\frac{1}{2}} \left(1 + \frac{1}{2} b^*\right)^{-\frac{1}{2}} \qquad 9.28$$

where,

$$b^* = \frac{2gT \left| (u^{2\ell-1}_{2j}) \right|}{C^2 R^{2\ell}_{2j}} + (u^{2\ell-1}_{2j+2} + u^{2\ell-1}_{2j-2}) \frac{T}{2h} \qquad 9.29$$

The friction influences the stability in a favourable sense (see later). The minimum time step depends on the largest depth to be modelled and without the second brackets the stability is approximated by the well-known Courant, Friedrich and Lewy condition.

$$T < \frac{h}{\sqrt{g(d + \xi)}} \qquad 9.30$$

where $d + \xi$ is the total water depth. For a fine grid in deep water, the time step may have to be quite small.

9.9 The friction coefficient

Two formulations are commonly used for the representation of bed friction, that is, Chezy's C or Manning n. The two are related by the depth raised to the 1/6th power and both coefficients are used in numerical models (McDowell and O'Connor, 1977). As in Chapter 6, the Chezy's C formulation is given by,

$$C = 18 \log_{10} (11d/k) \qquad 9.31$$

where k is a boundary roughness and is related to the roughness length, z_o, as,

$$z_o = k/30.2 \qquad 9.32$$

The water depth, d, is given by the hydraulic radius.

9.10 Density gradient

As no measurements of salinity were made for specific tides, a simplified longitudinal density structure in the estuary was employed. No attempt was made to approximate the gradient with a finite difference equation, using densities given at each node. Instead a constant longitudinal gradient and an average value for the density throughout the harbour were employed. The use of a constant density at every node in conjunction with a density gradient, is necessarily an approximation. However, the variations in hydraulic radius dominate the density gradient term, and the approximation is adequate. For,

$$I = \frac{g}{2\rho} \frac{\partial \rho}{\partial x} R \qquad 9.33$$

and grouping all coefficients together as a constant,

$$I = C_{den.} R \qquad 9.34$$

Thus, the specification of $C_{den.}$ will define the density gradients, and as shown earlier for Whangarei, $C_{den.} \simeq (1 - 5) \times 10^{-6}$.

9.11 Wind stress

The wind stress at the sea surface may be calculated as,

$$\tau_x = \rho_a \gamma_z |W_z| W_z \qquad 9.35$$

where ρ_a is the air density, γ_z is the coefficient of aerodynamic resistance at a height z above the sea, and W_z is the wind velocity at the same level. The determination of γ_z is as difficult as the determination of the Drag Coefficient for tidal flow discussed in Chapter 6. However, Ramming and Kowalik (1980) showed that the coefficient of aerodynamic resistance has a linear dependence on wind speed and for the speed measured at 10m above the sea,

$$\gamma_{10} = (0.9 + 0.08 W_{10}) \times 10^{-3} \qquad 9.36$$

Typical air and fluid densities are 1.29 $kg.m^{-3}$ and 1025 $kg.m^{-3}$ respectively. Notably, in one dimension, the wind may only be modelled as positive or negative along the harbour axis.

9.12 Computer solution

A computer program named LINMOD was written in FORTRAN to solve the finite-difference equations explicitly and incorporating terms for local and convective acceleration, sea slope, bed friction losses, wind stresses and salinity gradients.

9.12.1 Boundary specifications

For Whangarei Harbour, the boundary conditions were the sea level elevations at the seaward entrance and, being a closed tidal inlet, a zero speed at the upper harbour boundary was imposed. The model allows input of digitised tidal records or a program-generated sinusoidal tide with selected amplitude, mean tide level and period at the open boundary.

In the former case, a parabolic interpolation is applied to the digitised tidal record (which is normally input at 15 or 30 minute intervals) to ascertain the tide levels at the shorter time steps imposed on the model by the stability criterion.

For the general parabola with equation,

$$\xi = at^2 + bt + c \qquad 9.37$$

the constant may be found, given any three points on the curve (t_1, ξ_1), (t_2, ξ_2), (t_3, ξ_3) as in Chapter 6. However, for data which is equally spaced in time, a simpler expression can be obtained by shifting the time axis so that $t_1 = 0$. If the digitising interval is Δt, then $t_2 = \Delta t$ and $t_3 = 2\Delta t$ and,

$$a = \frac{1}{(\Delta t)^2}\left[\frac{\xi_1}{2} - \xi_2 + \frac{\xi_3}{2}\right]$$

$$b = \frac{1}{2\Delta t}\left[-3\xi_1 + 4\xi_2 - \xi_3\right] \qquad 9.38$$

$$c = \xi_1$$

Continuously-updated parabolae are fitted throughout the modelling period to the three tide levels nearest the time of interest, and interpolated sea levels are obtained from these.

Using a sine wave digitised at 15-minute intervals for test input data, initial application of a linear interpolation between the digitised data caused spurious velocity oscillations. The parabolic interpolation smoothed these but did not completely eliminate them (Figure 9.3(a)). Numerical experimentation revealed that the two decimal place accuracy of the input data was also responsible and when the tide was input with three decimal places and with the parabolic interpolation, the osciallations disappeared

(Figure 9.3(b)). Thus, small variations in velocity of the order shown on Figure 9.3(a) may be due to the accuracy of the tidal record.

The staggered scheme simplifies the numerical treatment of the open boundary. With sea level at the first node input as the boundary conditions, the only term which needs special treatment is the velocity differential in the convective acceleration. As the seaward speed is not available, a forward difference,

$$(\frac{\partial u}{\partial x})_{2j} = \frac{u^{2\ell-1}_{2j+2} - u^{2\ell-1}_{2j}}{2h} \qquad 9.39$$

is used.

The last node requires no special treatment (except to ensure that it is assigned zero velocity) unless there is a discharge or inflow and a corretion must be applied to the continuity equation as the surface area of the last compartment is half that of other cells. The up-harbour half beyond the cross-section is not accessible when $u = 0$ is imposed at the velocity node.

9.12.2 Effective depth

The finite difference equation (9.14) contains two terms with the total water depth in the denominator i.e. the frictional resistance and the wind stress. In shallow water these terms become very large and model behaviour is uncertain. Furthermore, in shallow water, the representation of frictional retardation of flow with a squared velocity law is not necessarily adequate. In the 2-d model during the flooding and uncovering of sand flats, the high frictional resistance slowed the water speeds so much that late drying and early flooding lagged unrealistically behind the surrounding sea levels. Numerical investigations were undertaken with the 2-dimensional model and as the results were then incorporated into LINMOD they will be discussed here.

Figure 9.4, reproduced from Ramming and Kowalik (1980, p. 239) shows the depth dependence of the frictional resistance,

$$R = r/H \qquad 9.40$$

where H is the total water depth (corresponding with hydraulic radius), and $r = g/C^2$. For their Figure, the choice of $r = 0.003$ corresponds with $C = 57$ $m^{\frac{1}{2}}.s^{-1}$. This function is equivalent to the frictional retardation term in the momentum balance equation (9.14), without the velocity dependence. In this standard form (Type A, Figure 9.4), the frictional resistance rises sharply for depths less than 0.5m. This causes water speeds to rapidly approach zero and as drying continues on tidal flats in the 2-d model until the depth is less than 0.05m, a considerable sea level lag relative to adjacent channels was occurring.

After experimentation with several types of correction suggested by Ramming and Kowalik, the simplest was opted for because it prevented the

friction term from increasing sharply in shallow water and it kept the computation time down. An effective depth was defined as,

$$d_{eff} = d + \text{const} \qquad 9.41$$

and the constant was selected to be 1m and later revised in Chapter 10 to be 0.5m. Shown in Figure 9.5, this type of correction has a reducing influence with increasing depth; Cases A and B curves are asympotic.

After application in the 2-d model, water levels on the banks remained more nearly in phase with the surrounding channel tides and for uniformity, the same correction was introduced into LINMOD. This increase of hydraulic radius in the frictional resistance term needs to be compensated for by a slightly smaller Chezy's C which typically changed from 63 to 60 $m^{\frac{1}{2}}.s^{-1}$ during 1-d model calibrations.

Ramming and Kowalik (1980) applied the effective depth correction, Cases A and C (Figure 9.4), to a model of the German Bight and found better correlation with the Case C type. Their published Figure shows that the shape of the tidal curve is indeed better approximated after the correction. Comparisons of predicted and actual tides in Whangarei Harbour showed no noticeable change with an effective depth in the 1-d model after Chezy's C was correspondingly altered. This results mainly because the harbour cells do not completely dry out during the tide and very shallow depths do not occur.

It is evident that the alteration to the frictional resistance has no rigorous mathematical foundation but is merely a numerical procedure designed to improve the modelling over shallow water. The improved behaviour of the 2-d model and the quality of the predictions by the 1-d model (see later) suggest that the effective depth is of importance in some instances, and has little effect in other cases.

9.13 A rectangular channel closed at one end

A rectangular channel of constant depth and closed at one end was modelled as a test rig for LINMOD.

9.13.1 Expected behaviour

Dyer (1973) interprets tidal amplitude changes in an up-harbour direction as due to two influences, i.e. friction which dampens the tidal wave as opposed to tidal flow convergence which causes reinforcement. In a channel closed at one end with low friction, the amplitude increases in the upstream direction due to the "pressure" of the up-harbour boundary. Water builds up on the flooding tide, establishing an unfavourable gradient necessary to reduce flow speeds to zero. Conversely, at the open end, the basin may continue to drain until the rising tide stems the jetting flow at the entrance. Thus, the ebb will have a longer duration, and lower ebb tides will occur in the upper reaches.

9.13.2 Modelled behaviour

Figure 9.6 shows the predicted sea levels at the open and closed ends of a 14,000 long rectangular channel with 4.5m depth and for a tidal range of 2m and $C = 60\ m^{\frac{1}{2}}.s^{-1}$. The initially sinusoidal, open-boundary tide becomes skewed with an amplitude reinforcement and an extension of the ebb duration as expected. The water speeds are shown in Figure 9.7 and the ebb is longer and slower than the more sharp-crested flood curves. The water speeds increase towards the open end as the prism correspondingly increases. For a much longer channel of 40,000m length but the same depth and Chezy's C (Figure 9.8), the tidal curve distortion is more pronounced.

Overall, the general features of the sea level and velocity curves in the rectangular channel correspond with the expected behaviour.

The time between high water at the two ends of the long channel is about 76 minutes and, in this case which depends on the depth and frictional retardation, the high tide propogates at about 9 $m.s^{-1}$. Notably, in Whangarei Harbour, a lag of 33 mins occurs between two gauges 16 km apart, which corresponds with a similar propogation rate of 8.1 $m.s^{-1}$, suggesting that the mean depth of 4.5m and $C = 60\ m^{\frac{1}{2}}.s^{-1}$ are approximately representative of Whangarei Harbour.

9.14 Schematisation of Whangarei Harbour

For successful application in LINMOD, Whangarei Harbour must be represented as a straight channel of variable width and depth. Prior to the modelling, it was not known if the complex circulation patterns could thus be adequately specified in one-dimension, while maintaining the gross behaviour of the tidal inlet. Notably, there is a distinction between the harbour being one-dimensional and its response to 1-d schematisation and with some imagination, it may be seen as a channel running primarily east-west. Perhaps surprisingly (even to this writer) the 1-d representation of the harbour proved to be very successful.

9.14.1 General procedures

Harbour schematisation has two major constraints:

(i) it is essential that the low, mean and high tide harbour surface areas are accurately specified. These determine the tidal prism.

(ii) the cross-sectional areas which determine the mean depths should sum to give the harbour volume when multiplied by the node spacing.

It is not essential that the cross-sections be drawn at precise spacings. The schematisation requires mainly that constraints (i) and (ii) are upheld. Thus, the cross-sections are drawn perpendicular to the bed contours where possible, in a central lcoation within each cell that best represents the average of that particular zone. The cross-section width at CD and the width at high tide are measured and the corresponding surface areas are the product of the width and the node spacing. Noting that the node spacing is

fixed, it is only the cross-section width which can be adjusted to ensure accurate specification of both the surface area and mean depth for a given cross-sectional area. Accordingly, the schematisation must accommodate the simultaneous specification of the dependent parameters and some "trial and error" is often necessary to obtain the best result.

9.14.2 Whangarei Harbour

Depths were taken from the 1981 RNZN bathymetry survey (NZ5213) of the region from Mair Bank to west of One Tree Point. Further up-harbour beyond the recent survey, and in Parua Bay, the 1974 RNZN Chart was used. A cell width of 1400m (700m between sea level and velocity nodes) was chosen after consideration of the scale of the important bathymetric and morphological features of the Harbour. Where possible, cross-sections were spaced along the harbour thalweg, but in the lower harbour where flow is channelled along both sides of Snake Bank, this was not feasible (Figure 9.9).

Harbour surface areas were obtained by overlaying a grid and counting the number of enclosed squares. Numerical integration of charted depths allowed determination of cross-sectional areas and mean depths. With the high tide and CD cross-section widths, the gradients of the sloping walls were obtained with equation (9.15).

The Harbour was divided into five regions (Figure 9.9) with final schematisation shown in Figure 9.10:

(i) Mair Bank and environs from Frenchman Island to the entrance throat. This region was difficult to schematise because the flow turns through 90^{o} from Frenchman Island to the entrance throat. Thus, within a 1400m compartment, the velocities over Mair Bank may be at right angles to those in the main channel. Some liberal schematisation was required and, after determining the total surface area, using three cross-sections 1400m apart, the widths were artificially adjusted to balance the constraints. The approximate locations of the cross-sections are shown in Figure 9.9.

(ii) the entrance throat to One Tree Point. These cross-sections widen considerably at high tide due to the flooding of Snake and McDonald Banks (Figure 9.10) and the sloping wall represention allowed a much improved representation in this case. The One Tree Point section is constricted to the north by Reserve Point and shows little tidal variation (Figure 9.10).

(iii) Parua Bay. Section 9 (Figure 9.10) is much longer than the physical cross-section width and this was necessary to ensure that the Parua Bay surface area was fully represented.

(iv) One Tree Point to Limestone Island. The expansive tidal flats between Parua Bay and Limestone Island are clearly represented (Figure 9.10).

(v) upper reaches. Beyond Limestone Island, the Harbour branches into the Portland Reach to the south and Port Whangarei and the Town Basin to the north. With some detailed current speed measurements to evaluate the relative

volumes flowing north and south, this could have been modelled. However, an alternative procedure was designed whereby the north and south flows were summed to make one schematised cross-section. From Limestone Island, cross-sections were drawn both north and south and corresponding zones were added together. Notably, the velocities in the upper reaches would not be accurate but the tidal prism through the Limestone Island cross-section should be properly represented.

When the entire harbour was schematised, the total surface area and mean depths were compared with these parameters for Whangarei Harbour determined from the 1974 Navy Chart by Smith-McNaught (1976) (Figure 9.11) and Millar (1980) with a planimeter. Small final adjustments were made by addition or subtraction of area in the upper reaches as this zone has not been recently sounded and the extent of high tide intrusion into the mangroves was not accurately known.

Plotted in Figure 9.11 with the Smith-McNaught planimeter data, the model surface areas offer a good fit to the three data points at CD, MSL and at 3m above CD. However, the harbour surface area, according to the planimeter results, does not increase linearly, but appears to increase more sharply between CD and MSL while the banks are being flooded.

9.15 Harbour simulations

9.15.1 Modelled tides

Three days were selected for modelling: Spring tides on November 12 and 14 and a Neap tide on December 3, 1981. The Springs were chosen because vertical profiles at four locations across the entrance throat were measured throughout the tidal cycle during these two days (B series, Chapter 5). The Neap tide was chosen by the DHI for the 2-dimensional modelling and comparisons were made with their results.

9.15.2 Friction

For the modelling of the harbour a decision was taken to use a constant friction coefficient (C or z_o) throughout because: (i) of the wide variability of measured frictional resistance in the lower harbour and Mair Bank, (ii) of the changes of bed roughness due to the variability of bed facies (Chapter 2) within each cross-section, (iii) no measurements were made in the upper harbour to quantify the friction coefficients.

9.15.3 Stability and the time step

The time step was chosen according to the stability criterion and for d_{max} = 9.4m, h = 700m the approximate Courant number is,

$$T < \frac{h}{\sqrt{g d_{max}}} = 73s \qquad 9.42$$

For the Dronkers solution (equation 9.28),

$$T < \frac{h}{gd}\,(1 + \tfrac{1}{2}b^*)^{\frac{1}{2}} \qquad 9.43$$

and for typical values,

$$(1 + \tfrac{1}{2}b^*)^{\frac{1}{2}} = 1.02\text{-}1.05 \qquad 9.44$$

which has little effect but friction does slightly improve the stability.

With numerical experimentation, it was found that the scheme was stable up to a limiting time step of 95s but that total instability occurred at 98s, which is longer than expected. Velocities at the deepest cross-section are compared in Figure 9.12 for time steps of 95s and 30s and it is evident that there is an insignificant difference in the results. Nevertheless, as program solution was rapid, being less than 5 minutes for a complete simulation, the shorter step of T = 30s was selected to ensure stability.

9.15.4 Initial conditions

In all cases, a cold start was made i.e. all sea levels are initially set to the open boundary level and all velocities are set to zero. Depending on the bed friction coefficient, and the water depth, spurious oscillations caused by the initial condition dissipate quickly within ½ - 1½ tidal cycles (See Figures 9.6 and 9.7, for example).

9.15.5 Tidal records

There were two permanent tide gauges in Whangari Harbour located 16,000m apart at Marsden Point and Port Whangarei on respective NHB wharves. The 1-d model calibration was undertaken with sea levels from these two permanent instruments. The Marsden Point gauge is located between schematised sections 4 and 5, and thus sections 1-4 outside the entrance were excluded. With the lower harbour sea levels as the open boundary condition, model predictions were compared with the upper harbour tidal records corrected for the 0.23m lower level of CD at Port Whangarei relative to Marsden Point (Chapter 1).

9.15.6 Calibration

Initial calibrations were conducted with a constant Chezy's C throughout the harbour and with this formulation C does not vary with water depth. Incrementation of the friction coefficient revealed that $C = 60\ m^{\frac{1}{2}}.s^{-1}$ produced the best comparisons with the Port Whangarei sea elevations (Figure 9.13). Further calibrations were then undertaken with a constant roughness length, and Chezy's C was found from z_o with equation (9.31). In this case, C changes with tidal variations in the water depth. By incrementation of z_o, the best results were obtained with $z_o = 0.001m$ (Figure 9.14).

Both the roughness length and Chezy's C show agreement with the measured friction coefficients in the lower harbour. In Chapter 6, mean and median roughness lengths for all current profiles with $u_1 > 0.3\ m.s^{-1}$, were found to

be 0.0087m and 0.0007m respectively (Table 6.3). These bound the model roughness length of 0.001m.

A harbour Chezy's C of 59 $m^{\frac{1}{2}}s^{-1}$ was determined in Chapter 6 using the measured median z_o and the mean harbour depth from Millar's (1980) planimeter results and compares with C = 60 in the model.

These friction coefficients are not unusual. Ramming and Kowalik (1980), for example, recommended C = 55 for use in numerical models. Also z_o = 0.001m was suggested by Heathershaw (1981) as being representative of irregular, gravel beds. Due to the prevalence of shell lag predominantly lining the channel floors in the lower harbour (Chapter 2), the association of friction with a gravel roughness could be expected.

This compares with roughness lengths of z_o = 0.003m (Table 7.5) over megarippled sandy beds with greater energy losses occurring due to form drag in the lee of the bedform crests. This roughness length converts to a Chezy's C of 48 for a mean harbour depth of 3.7m (Chapter 6), which compares with C = 45 for tidal inlets suggested by Bruun (1966, p. 107, referenced by Healy, 1981). As noted, the higher Chezy's C at Whangarei corresponds with the abundance of shell lag and the limited extent of the megarippled sand cover.

9.15.7 Verification

For verificiation, tides on 12-Nov-81 and 3-Dec-81 were modelled with z_o = 0.001m (Figures 9.15 and 9.16). The comparison on 3-Dec-81 revealed a slight tendency for the model sea levels to be lower than those of the Port Whangarei tide gauge throughout most of the tidal cycle. This cannot be corrected for with a change of the friction coefficient as both the low and high tide amplitudes would be dampened by a friction increase.

9.15.7.1 *Density gradients*

The wind on 3-Dec-81 during the modelling period was "calm" at the Marsden Point anenometer (Table 9.1), and could not be a relevant factor. However, longitudinal density gradients associated with down-harbour salinity increases will accelerate the flood and decelerate the ebb. Thus, a constant density gradient (equation 9.34) was introduced and, with $C_{den} = -5 \times 10^{-6}$, excellent agreement with the Port Whangarei tidal record resulted (Figure 9.17).

Even though the density gradient may be nearly an order of magnitude less than the sea slope and frictional retardation, it still plays a significant role and should be included in the complete hydrodynamic equation. This agrees with findings of Knight (1981, p. 320) who stated that "the influence of the longitudinal density gradient needs to be included in any resistance study". As this term is nearly always directed up-harbour (with the exclusion of fresh water inflow in the lower reaches) greater friction coefficients on the ebb than the flood result. This was found to occur in the field measurements in the lower harbour (Chapter 6) and longitudinal density gradients are evidently

partly responsible.

9.15.7.2 *Wind influences*

On both the 12th and 14th of November, there are discrepencies near high tide. These suggest either the presence of a wind blowing in the ebb direction or an inflow of fresh water in the upper reaches. The wind velocities for the modelling period are given in Table 9.1 and speeds are high particularly on the 14th. These high winds from the north would be expected to cause a reduction of the high tide level on the 14th at Port Whangarei in the north-south oriented channel. However, the harbour is mostly oriented east-west and this makes it difficult to simulate wind conditions which can only be modelled as positive or negative along the axis of the schematised harbour.

Nevertheless, a wind of $-10\ m.s^{-1}$ (directed out of the harbour), in conjunction with a longitudinal density gradient of $C_{den} = -5 \times 10^{-6}$, were modelled for the 12th and 14th tides. The comparison has improved (Figures 9.18 and 9.19),but the wind strengths were selected on a "best fit" basis made necessary by the changes of harbour orientation, and do not correspond precisely with the actual records.

9.15.7.3 *Fresh water inflows*

The third factor to be considered is the fresh water inflow at the upper harbour. Shown in Table 9.2, in the previous 24 hours at Marsden Point, there was no rainfall on December 3, 1mm on the 12th and a heavy rainfall of 32mm on the 14th. This can be compared with an annual rainfall of 1530mm (Smith-McNaught, 1976) which averages to 4.2mm in a 24-hour period.

For a tidal cycle period of 12.42 hrs, the 24 hour rainfall on the 14th, converts to 16.6mm per tidal cycle. Smith-McNaught estimated the evapo-transpiration losses over the catchment area to be 1.0mm per tidal cycle. The evaporation from the free water surface and the wetted mud and sand areas below H.W. were 1.4mm per tidal cycle. Using their figures for the total areas of the catchments of $301 \times 10^6\ m^2$ land and $99 \times 10^6\ m^2$ water, then the input to the harbour during this event was,

$$\begin{aligned} (301 \times 10^6) \times 0.0156 &= 4.7 \times 10^6 \\ (99 \times 10^6) \times 0.0152 &= \underline{1.5 \times 10^6} \\ \text{Total:} \quad & 7.2 \times 10^6\ m^3/\text{tidal cycle} \end{aligned} \qquad 9.45$$

This corresponds with a continuous average of 161 cumecs, and is nearly 5% of the mean harbour tidal prism. A fresh water input of this magnitude will not only assist the ebb currents due to the volume added to the harbour, but it will also establish a higher salinity and density gradient.

The rainfall was simulated on the 14th by the addition of 100 cumecs to the closed end of the harbour and 5 cumecs to each of the other 12 sections to make a total of 160 cumecs. With this included, the final model simulation for the tide of the 14th is shown in Figure 9.20 and the agreement with the

Port Whangarei tide is pleasing. Notably, this simulation uses $z_o = 0.001m$, a density gradient with $C_{den} = -5 \times 10^{-6}$, a wind condition of -10 $m.s^{-1}$ and fresh water inflow as detailed above. Evidently, the salinity, wind and inflow each play a small but important role.

9.15.8 Comparisons of three simulations

In all three cases, the bed roughness of $z_o = 0.001m$ and a density gradient of $C_{den} = -5 \times 10^{-6}$ were used. A wind of -10 $m.s^{-1}$ was applied on the 12th and 14th, and inflow included on the 14th only. The model is seen to be reproducing the Port Whangarei tides from the Marsden Point data with a high degree of accuracy (Figures 9.17, 9.18 and 9.20).

The most significant deviation still present in the comparison occurs for all three days after mid-tide on the ebb half cycle. The tide drops more rapidly in the prototype than the model and then slows near low tide. The over-riding influence at Whangarei which causes the former is considered to be the harbour morphology. Sea levels drop more rapidly in the second half of the ebb because the shallower storage regions are considerably reduced after the banks become exposed. Flow is mainly restricted to the deeper conveying channels and the tides fall more rapidly. This is reflected in the harbour surface area determination by Smith-McNaught (1976) shown in Figure 9.11(a), where area decreases most rapidly between MSL and CD. Because of the linearly-sloping side wall representation, the model cannot accurately reproduce this behaviour. This then, is not a failing of the numerical scheme, but is instead due to poor representation of the bathymetry.

McDowell and O'Connor (1977) associated the flattening of an ebb tidal curve near low tide, with slow draining of water into the channels from flanking banks. As the tide level drops rapidly in the channels, water seeps from the pore spaces in the banks and causes the ebb, sea-level, fall rate to slow.

9.16 Examination of the terms in the hydrodynamic equation

This section examines the relative influences of the terms in the hydrodynamic equation responsible for the good model verification. Inspection of equation (9.1) reveals that the local flow acceleration arises out of the interaction of the five terms on the right hand side of the following equation,

$$\underset{(1)}{\frac{\partial u}{\partial t}} = \underset{(2)}{-\frac{u\partial u}{\partial x}} \underset{(3)}{-\frac{g\partial \xi}{\partial x}} \underset{(4)}{-\frac{g|u|u}{C^2 R}} + \underset{(5)}{\frac{\tau x}{\rho R}} - \frac{gR}{2\rho}\frac{\partial \rho}{\partial x} \qquad 9.46$$

The effect of the convective acceleration term can be isolated by expansion of the continuity equation (9.2), excluding additional inflows,

$$\frac{\partial uA}{\partial x} + \frac{b\partial \xi}{\partial t} = 0 \qquad 9.47$$

then,

$$\frac{u\partial A}{\partial x} + \frac{A\partial u}{\partial x} + \frac{b\partial \xi}{\partial t} = 0 \qquad 9.48$$

and,

$$\frac{u\partial u}{\partial x} = -\frac{u^2}{A}\frac{\partial A}{\partial x} - \frac{ub}{A}\frac{\partial \xi}{\partial x} \qquad 9.49$$

Introduction of this into the momentum balance equation, with density and wind stress terms omitted, gives,

$$\frac{\partial u}{\partial t} = -\frac{g\partial \xi}{\partial x} - u\left(\frac{g|u|}{C^2 R} - \frac{u}{A}\frac{\partial A}{\partial x} - \frac{b}{A}\frac{\partial \xi}{\partial t}\right) \qquad 9.50$$

and the total resistance to motion is,

(i) (ii) (iii)

$$\frac{g|u|}{C^2 R} - \frac{u}{A}\frac{\partial A}{\partial x} - \frac{b}{A}\frac{\partial \xi}{\partial t} \qquad 9.51$$

The signs of these terms are:

	(i)	(ii)	(iii)
Flood	+	+	-
Ebb	+	-	+

Thus, a reduction of cross-sectional area in the upstream direction will accelerate the ebb and decelerate the flood. The change of sea level with time will have the opposite effect. However, in an estuary closed at one end, the sea level term is more important than the change of area. For example, in a schematised estuary of constant depth and linearly decreasing width, Term (iii) can be shown analytically to be approximately a factor of 2 times more effective. This can be reinforced by noting that overall $|u|$ decreases in the up-harbour direction from the entrance to the closed end where $u = 0$. Averaged over the basin, the velocity gradient will always be negative during the flood and positive during the ebb. The convective acceleration will then support the flood and retard the ebb overall, even though variations due to changing cross-sectional area may reverse this at some specific locations.

As expected, the bed friction will always oppose the flow. Furthermore, as already noted, if maximum salinity occurs at the entrance throat, the density gradient supports the flood. The wind stress assists the current travelling in the same direction as the wind.

The five terms in the momentum balance equation (9.46) were derived and are presented in Figures 9.21 and 9.22 showing Section 8 (12-Nov-81) at One Tree Point and Section 5 (3-Dec-81) at the open sea boundary at Marsden Point (Figure 9.9).

The sea slope is the dominant forcing agent during the Spring tide at One Tree Point (Figure 9.21). It attains a maximum of about 2×10^{-5} and this occurs at high and low tide, not at mid-tide. Moreover, the sea surface gradient is greater during the ebb tide than the flood - a compensation for

the positive convective acceleration term which supports the flood currents, as is expected to occur in a closed estuary. At peak speeds, the convective acceleration is seen to have a considerable influence being about 30% of the bed frictional retardation. The bed friction always opposes the currents and is mostly in phase with the current flows being dominated by the squared water velocity. The sea slope leads the bed friction term throughout the tide, attaining a maximum near the time when bed friction goes to zero, and establishing unfavourable gradients during flow deceleration in the later stages of each half cycle.

The longitudinal density gradient is directed up-harbour and is about 20% of the friction term at maximum. The density term is nearly constant but in phase with the tidal sea levels being greatest at high tide. For this particular cross-section, the wind stress term is approximately equal to, but 180^{o} out of phase with, the density term. As they are directly and inversely related to depth, the relative influence of the density and wind terms varies markedly from shallow to deep water.

On December 3, (Figure 9.22) during Neap tides, water speeds through the entrance cross-section are similar to the previous case at One Tree Point but the depth is greater in the entrance throat. Accordingly, the influence of the longitudinal density gradient is more pronounced and is comparable with other terms during the flood tide. The sea slope curve is grossly asymmetrical being much greater during the ebb tide because of the support during the flood that is offered by the density gradient and convective acceleration.

9.16.1 Average sea slopes between Marsden Point and Port Whangarei

To introduce more detail into the sea level comparisons, the average sea slopes between Marsden Point and Port Whangarei are compared with the actual records for the 3 modelled tides in Figures 9.23 to 9.25. On December 3, the model is adequately predicting the actual slope even to the extent of detailing some of the small oscillations (Figure 9.23). This is an important result offering very strong support for the model and the choice of parameters to describe Whangarei Harbour. On the two windy Spring tide days (Figures 9.24 and 9.25) the curve is very well represented by the model for much of the tide, except for the large sea gradients near peak ebb. No tests were made to determine the cause of this, and it may be related to wind influences or possibly more importantly to bathymetry schematisation which is more critical on a Spring tide.

There are some wide variations in the sea slope from one tide to the next. The measured values vary from a maximum of 2.7×10^{-5} during the ebb on the 14th to a maximum of only 0.7×10^{-5} during the flood on the 3-Dec-81.

9.16.2 Simplified expression for determination of bed roughness

At peak flow, when the local flow acceleration is zero, a simplified equation, keeping only the sea slope and bed friction terms, is often used to

ascertain the bed friction coefficients. In the reduced form at peak flow ($u = U_{max}$),

$$C = \frac{U_{max}}{\sqrt{R\frac{\partial \xi}{\partial x}}} \qquad 9.52$$

Smith-McNaught (1976) applied this equation to Whangarei Harbour after acknowledging the non-linear influence of the tidal flats and their effect on the simplified form of the equation. They obtained very large Chezy's C values (i.e. low friction resistance) with an ebb and flood average of $\bar{C} = 71\ m^{\frac{1}{2}}.s^{-1}$. Greater sea slopes during the ebb were identified (concurring with this analysis), but due to a disparity in peak speeds, the frictional resistance ,they obtained with equation (9.52) above, was smaller during the ebb.

With respect to the prior analysis, where the longitudinal density gradient and the convective acceleration were shown to be important, the above simplification is not adequate in Whangarei Harbour and all resistance to flow cannot be attributed solely to the bed roughness. With all terms included, the model shows that Chezy's C is more nearly $C = 60\ m^{\frac{1}{2}}.s^{-1}$.

9.17 Flow velocities

Program LINMOD derives flow velocities averaged throughout the cross-section at each velocity node. Some of these are shown in Figures 9.26 to 9.28 at three cross-sections: the open sea boundary (Section 5); One Tree Point (Section 8); and up-harbour of Limestone Island (Section 16).

Three major aspects of behaviour noted in the field velocity measurements (Chapter 5) were reproduced by the model. Firstly, the flood exhibits a longer duration and a slower maximum speed. This is opposite to behaviour in the rectangular channel. Secondly, maximum speeds are attained nearer to high than to low tide during both the ebb and flood half cycles. These inter-related aspects are due, in part, to the interaction of the sea slope and the bed friction, such that at high tide the frictional resistance is less than at low tide and maximum speed occurs nearer high tide. The early acceleration of the ebb currents sustains the flow, allowing the tidal inlet to ebb more quickly. However, this is not the only factor, and if it were, the rectangular channel would exhibit similar behaviour.

Examination of Figure 9.22 which shows the influence of the various terms in the hydrodynamic equation at the entrance throat on December 3, reveals that the interaction of all terms establishes a very large sea gradient at high tide (the opposite occurs in the rectangular channel) which allows the ebb to accelerate early and the exhibited behaviour results. During the flood (Figure 9.22), the sum of the forcing terms is more constant throughout the tide and flow speeds change more slowly. The convective acceleration term evidently plays an important role in estuarine dynamics. When viewed in terms of the harbour morphology, the greater cross-section width and corresponding surface

area at high tide means that a much larger volume of water attempts to leave the estuary after full tide and early rapid flow acceleration necessarily results.

Thirdly, oscillations with periods of ½-2 hrs are evident on the velocity-time curves. Similar behaviour was commonly noted during Braystoke velocity measurements in the Harbour. The model shows that they propogate upstream from the entrance gorge evidenced by their appearance on up-harbour velocity-time curves at later times (Figures 9.26 to 9.28). The rate of propogation (approximately 8 $m.s^{-1}$) depends on mean water depth and bed friction which governs the speed of long waves in shallow water.

The mechanism which causes the perturbations at the open boundary is not definitely known but they may either be caused by seiching within the harbour or trapped long-period continental shelf waves outside the entrance. Examination of the tidal and wind records for Marsden Point revealed that the amplitude of the irregularities is related to wind and wave conditions outside the entrance. In several East Coast harbours (not including Whangarei), Heath (1979) associated similar oscillations having periods of 2.4-2.7 hours with edge wave excitation on the continental shelf. Shorter waves with periods of 1.7 hours were associated with resonant seiching within the harbour. The periods of the largest oscillations on Figures 9.26 to 9.28 are typcially 1.4-1.5 hrs. Nevertheless, this does not exclude continental shelf waves as these depend on shelf depth in the vicinity of the Harbour Entrance.

At the open boundary, which passes through the proposed port site, maximum water speeds were 0.8 $m.s^{-1}$ (flood) and 0.91 $m.s^{-1}$ (ebb) averaged throughout the cross-section during the high Spring tides on the 14th. Further east, in the narrowest part of the entrance throat, speeds would be higher. On Neap tides, the flood and ebb speeds attain a reduced maximum of approximately 0.5 $m.s^{-1}$.

9.18 Tidal prism

LINMOD keeps a running total of discharge through each cross-section to output the half cycle tidal prism when the flow reverses. The tidal prism for the open boundary and One Tree Point cross-sections are included in Figures 9.26 to 9.28. Prisms through the entrance throat were listed in Table 9.3 after correction for the surface area between the open boundary and the entrance throat ($3 - 5 \times 10^6$ m^3).

9.18.1 Procedures for verification of tidal prisms

Verification of the tidal prisms was performed using results from three sources.

(i) the tidal prism can be found from the Whangarei Harbour volumes below selected contours shown in Figure 9.11(b) (Smith-McNaught, 1976). The tide

level above CD for this Figure was reported as the average of the Marsden Point and Port Whangarei gauges. Although not stated specifically by Smith-McNaught, this would suggest that no correction was made for the differences of the CD levels at the two gauges. Accordingly, no correction was made for the prism calculations. The tide levels and corresponding harbour volumes for the modelling periods are shown in Table 9.4 and prisms from these are listed in Table 9.3.

(ii) During the 12th and 14th, water speeds were measured at irregular intervals, throughout the tidal cycles across the entrance throat (B series, Chapter 5). To obtain prisms from these, the data was processed in three stages: (a) after correction of water speeds for non-simultaneity of measurement in the vertical (Chapter 6), a parabolic interpolation was employed to find the mean vertically-averaged velocity for each profile with program CUR4; (b) mean speeds at each site time-averaged over the half tidal cycle, were obtained numerically from the mean vertically-averaged velocities; and (c) a third averaging was then applied to find the discharge across the entrance throat over each half cycle using program CHANMN. This program employs the measured speeds at known locations to linearly interpolate for speeds, u_i, along a chosen cross-section at increments of width $\Delta w = 1m$. These speeds, weighted with a corresponding water depth d_i, obtained by linear interpolation of the cross-section depths, sum to provide the total discharge, Q. That is,

$$Q = \sum_{i=i}^{N} u_i d_i \, \Delta w \qquad 9.53$$

where N is the cross-section width for 1m-wide increments. The prism, Ω, is the discharge multiplied by the half cycle flow duration, $T_{\frac{1}{2}}$,

$$\Omega = QT_{\frac{1}{2}} \qquad 9.54$$

and mean speed, $\bar{u}$, is,

$$\bar{u} = Q/A \qquad 9.55$$

For the determination, a Mean Sea Level of 1.5m above CD was imposed.

Notably, if the velocities during the B-series had been measured at closer time intervals, the averaging could have been undertaken such that the discharges across the gorge were found at successive time increments throughout each half tidal cycle and summed to provide the prism. This method allows for changes in depth throughout the tide.

The B-series sites were not on a straight line across the throat and the best fit cross-section is not flow-perpendicular. A representative, flow-perpendicular, cross-section was selected from entrance throat cross-sections discussed in Chapter 3 and of these, Profile 3 (Figure 3.26), best represents the B-series locations. This profile has been reproduced in Figure 9.29 and site locations were placed to correspond with the depths below CD of the actual

measurement sites (Table 5.9). Table 9.5 gives the mean speeds after Stage (b) of the reduction, having been averaged vertically and averaged over the half tidal cycle. With the 12th data, plotted on the throat cross-sections the expected ebb dominance on the southern side of the profile near Marsden Point and flood dominance to the north near Passage Island are evident. Interestingly, the average of the ebb and flood half cycle speeds exhibit a very pronounced correlation of this average speed to the absolute water depth below CD. Prisms determined from these speeds with CHANMN are compiled in Table 9.3.

(iii) The DHI 2-dimensional model of the harbour produced a discharge-time curve for the entrance throat on the 14th (DHI, 1982, Figure 5.18). Numerical integration of this curve to find the prism was performed by this writer. As their boundary was east of the harbour throat, a small correction of -4×10^6 m^3 to each calculated prism was made.

9.18.2 Results of tidal prism verification

The comparison of all determined prisms is shown in Table 9.3. There is good agreement within 4% in all cases excepting the current speed measurements. This discrepency occurs mainly because of insufficient data measured in the field. In some cases, only three profiles were recorded during the half tidal cycle and with only four sites across the throat, these were not sufficiently descriptive. However, with the exception of these, the comparison, which similarly confirms the accuracy of the DHI model, shows that the prism is being represented. In conjunction with previous test comparisons of the sea levels, the analysis shows that LINMOD is adequately modelling tidal current and sea level behaviour in Whangarei Harbour.

9.19 Conclusions

(i) A one-dimensional numerical hydrodynamic model was formulated and successfully applied to a rectangular channel test rig and to the prediction of sea levels and velocities in Whangarei Harbour. The model solves the hydrodynamic equation with a (semi-)explicit finite difference scheme.

(ii) The model includes terms for convective acceleration, the longitudinal density gradient, wind and additional inflows. Each of these were found to play a significant role.

(iii) The estuary was schematised as having rectilinear sloping walls to simulate the flooding and drying of inter-tidal zones. This allowed an adequate representation of Whangarei Harbour tidal behaviour verified by the sea level and velocity predictions.

(iv) An effective depth was defined whereby a constant was added to all depths in the frictional resistance term to prevent unrealistic retardation of flow in very shallow water. This correction had little effect on the harbour

simulations as cross-sections did not completely dry out during the tidal cycle.

(v) Friction coefficients for the harbour were found to be $C = 60\ m^{\frac{1}{2}}.s^{-1}$ and $z_o = 0.001m$ in agreement with field measurements. These are associated with an irregular, gravel bed which predominantly lines the channel floors at Whangarei.

(vi) The model simulated three behavioural aspects noted in the field measurements: (a) a relatively-fast, short-duration ebb flow; (b) the distortion of the velocity-time curve so that maximum speed occurs nearer high than low tide on both the ebb and flood; and (c) velocity oscillations with periods of ½-2 hrs arising from tide curve perturbations of indefinite origin.

(vii) In a rectangular channel of constant depth and closed at one end, the modelled ebb currents were slower and had a longer duration than the flood due mainly to the asymmetric boundary conditions. In Whangarei Harbour the opposite occurs and was associated with the morphology of the harbour channels and banks and the interaction of bed friction with the sea level.

This has important implications with respect to the genesis of an estuary and its subsequent stability. When the surface area remains constant throughout the tide as in the rectangular channel the dominant flood currents induce landward sediment transport. The estuary then begins to fill with sediment, allowing inter-tidal zones to develop. The velocity asymmetry is successively reduced until ebb dominance prevails thereby preventing sedimentation. Accordingly, an equilibrium may be attained with ebb and flood dominance being in balance with the morphology of the inter-tidal zones and channels. The modelling shows that this mechanism is a very influential factor.

(viii) A reduction of cross-sectional area in the up-harbour direction will accelerate the ebb currents. However, in a channel closed at one end, the upstream condition that $u = 0$ causes the convective acceleration term to support the flood currents overall.

(ix) The maximum sea slopes in Whangarei Harbour are significantly greater on the ebb than the flood and relate to higher ebb speeds.

SEDIMENT TRANSPORT AND TIDAL INLET HYDRAULICS

Chapter 10:

TWO-DIMENSIONAL, NUMERICAL, HYDRODYNAMIC MODEL

10.1 Introduction

Numerical modelling is very effective in coastal engineering applications, because it allows prediction of future events. This cannot be done accurately or with confidence using field measurements made before dredging or construction. As noted in the Introduction to Chapter 9, numerical models have accordingly been well utilised in recent coastal engineering investigations. In this Chapter, 2DD, a 2-dimensional, hydrodynamic, numerical, tidal flow model developed by this writer is described, tested and applied to the proposed timber port construction at Marsden Point.

The model, includes terms to simulate acceleration of flow (both local and convective), the bed friction, the Coriolis force, the wind stress and the horizontal eddy viscosity. Density gradients are not included primarily because field data were not available to describe the horizontal density variations.

A semi-explicit, finite-difference scheme was selected and then modified to solve the hydrodynamic equations. The scheme is fast and is shown to be as accurate as a mixed-implicit model with comparisons using test rigs and a difficult field application in Whangarei Harbour.

10.2 List of Symbols

A_H	horizontal eddy viscosity coefficient ($m^2.s^{-1}$)
d	depth below a horizontal datum (m)
D_U	depth below datum at the U velocity nodes (m)
D_V	depth below datum at the V velocity nodes (m)
$\tilde{D}_r$	tidal cycle residual distance vector (m)
f	Coriolis parameter (s^{-1})
g	gravitational acceleration (9.81 $m.s^{-2}$)
h	distance between velocity and sea level nodes (half cell width) (m)
H_U	total water depth at the U velocity nodes (m)
H_V	total water depth at the V velocity nodes (m)
j	subscript denoting x location in space
k	subscript denoting y location in space
l	superscript denoting location in time
m	number of cells in the x direction
n	number of cells in the y direction
t	time (s)
T	half time step (s)
T_s	period of one sidereal day (86,164s)
u	velocity in the x direction ($m.s^{-1}$)
U	mean vertically-averaged velocity in x direction ($m.s^{-1}$)
$\tilde{u}_{cr}$	critical threshold velocity ($m.s^{-1}$)
v	velocity in the y direction ($m.s^{-1}$)
V	mean vertically-averaged velocity in y direction ($m.s^{-1}$)
w	vertical velocity ($m.s^{-1}$)
V_0, V_1, V_2, etc	velocities (defined in Figure 10.5) ($m.s^{-1}$)
W_x	wind speed in the x-direction
W_y	wind speed in the y-direction
x	horizontal coordinate
y	horizontal coordinate
z	vertical coordinate
z_o	roughness length (m)
α	correction factor

γ	wind resistance coefficient
κ	von Karman's constant (0.4)
ρ	fluid density ($kg.m^{-3}$)
ρ_a	air density ($kg.m^{-3}$)
τ_s	wind shear stress ($N.m^{-2}$)
τ_{sx}	wind shear stress in x direction ($N.m^{-2}$)
τ_{sy}	wind shear stress in y direction ($N.m^{-2}$)
ξ	sea level above a horizontal datum (CD) (m)

10.3 The hydrodynamic equations

The two-dimensional hydrodynamic equations for nearly horizontal flow are (Leendertse, 1967),

$$\frac{\partial U}{\partial t} + U\frac{\partial U}{\partial x} + V\frac{\partial U}{\partial y} - fV = -g\frac{\partial \xi}{\partial x} - \frac{gU(U^2 + V^2)^{\frac{1}{2}}}{C^2(d+\xi)} + \frac{\rho_a \gamma |W| W_x}{\rho (d+\xi)} + A_H\left(\frac{\partial^2 U}{\partial x^2} + \frac{\partial^2 V}{\partial y^2}\right)$$

$$\frac{\partial V}{\partial t} + U\frac{\partial V}{\partial x} + V\frac{\partial V}{\partial y} + fU = -g\frac{\partial \xi}{\partial y} - \frac{gV(U^2 + V^2)^{\frac{1}{2}}}{C^2(d+\xi)} + \frac{\rho_a \gamma |W| W_y}{\rho (d+\xi)} + A_H\left(\frac{\partial^2 U}{\partial x^2} + \frac{\partial^2 V}{\partial y^2}\right) \qquad 10.1$$

where the x and y directed horizontal orthogonal velocities (U,V) are vertically-averaged, f is the Coriolis parameter (s^{-1}), g is the gravitational acceleration ($m.s^{-2}$), ξ is the sea level above a horizontal datum (m), d is the depth below the datum (m), ρ is the water density ($kg.m^{-3}$), W is the wind speed at 10m above sea level ($m.s^{-1}$), W_x and W_y are the respective x and y components of the wind speed ($m.s^{-1}$), γ is the wind resistance coefficient, ρ_a is the density of air ($kg.m^{-3}$), C is Chezy's C ($m^{\frac{1}{2}}.s^{-1}$) and A_H is the horizontal eddy viscosity coefficient ($m^2.s^{-1}$)

These equations can be obtained by integrating the three-dimensional Reynolds equations over the depth, with the condition that vertical accelerations are neglible compared to gravity and thus the flow is nearly horizontal.

Falconer (1980) suggested a correction factor to the non-linear convective acceleration terms due to the non-uniform vertical velocity distribution. For a logarithmic vertical velocity profile, the factor is,

$$\alpha = 1 + \frac{g}{C^2\kappa^2} \qquad 10.2$$

but for typical values at Whangarei of $C = 60\ m^{\frac{1}{2}}.s^{-1}$, $\kappa = 0.4$ we find $\alpha = 1.02$, i.e. the correction is neglible.

The momentum equation is very similar to the 1-d case introduced in the last chapter but includes (i) the Coriolis force which acts at right angles to the flow direction and diverts flow to the left in the southern hemisphere; and (ii) the horizontal eddy viscosity term which simulates the cascade of

energy transfer by non-linear interactions to smaller scales to be finally dissipated by viscous friction. The second order partial derivatives have the effect of smoothing the velocity field by an averaging process where the central speed is adjusted according to the magnitude of its neighbours. The term has no effect in uniform flow.

Notably, no term was included to model the density gradients. This is not because they do not play a role, but because insufficient field salinity measurements meant that the salinity throughout the tidal cycle could not be specified on a square grid. According to analysis in Chapter 9, the salinity plays a relatively small role, although its influence was noted. However, its exclusion in the one-dimensional model is compensated for by a change in sea surface slope necessary to drive the tidal flows during each half tidal cycle.

The continuity equation in 2-dimensions is,

$$\frac{\partial \xi}{\partial t} + \frac{\partial}{\partial x}(d+\xi)U + \frac{\partial}{\partial y}(d+\xi)V = 0 \qquad 10.3$$

Within a region, any gain or loss of fluid will be balanced by a corresponding change in sea level.

10.4 Finite difference scheme

Many finite difference schemes for the modelling of 2-dimensional hydrodynamic flow have been published in the last 15-20 years e.g. Hansen, 1956; Leendertse, 1967; Heaps, 1969; Williams and Hinwood, 1976; Tee, 1976; Snyder et. al. 1979; Falconer, 1980; Benqué et. al., 1982 etc. The methods of solution of the hydrodynamic equations cited in these papers include explicit, implicit, and mixed-implicit finite difference equations.

Due to the computer memory and speed limitations and in view of the success of the procedure in one dimension, an explicit scheme very similar to that presented in Chapter 9 was chosen for the 2-d modelling at Whangarei. The notation for the scheme is shown in Figure 10.1.

After algebraic manipulation finite difference approximations to the momentum balance equations (10.1) are,

$$U_{j,k}^{\ell+1} = \{2T\,(f\,V_x^{\ell-1} - V_x^{\ell-1}\frac{(U_{j,k+1}^{\ell-1} - U_{j,k-1}^{\ell-1})}{4h}$$

$$- \frac{g}{2h}(\xi_{j+1,k}^{\ell} - \xi_{j,k}^{\ell}) + A_H\,U_G^{\ell-1}$$

$$+ \frac{\rho_a \gamma |W|\,W_x}{\rho\,H_{Uj,k}^{\ell}}) + U_{j,k}^{\ell-1}\}/$$

$$\{1 + \frac{2gT\left[(V_x^{\ell-1})^2 + (U_{j,k}^{\ell-1})^2\right]^{\frac{1}{2}}}{C_{j,k}^2\,H_{U\,j,k}^{\ell}} + \frac{2T}{4h}(U_{j+1,k} - U_{j-1,k})\} \qquad 10.4$$

where $V_x^{\ell-1} = (V_{j+1,k}^{\ell-1} + V_{j,k}^{\ell-1} + V_{j,k-1}^{\ell-1} + V_{j+1,k-1}^{\ell-1})/4$

$$H_{Uj,k}^{\ell} = D_{Uj,k} + (\xi_{j,k}^{\ell} + \xi_{j+1,k}^{\ell})/2$$

$$U_G^{\ell-1} = \frac{U_{j+1,k}^{\ell-1} + U_{j-1,k}^{\ell-1} + U_{j,k+1}^{\ell-1} + U_{j,k-1}^{\ell-1} - 4U_{j,k}^{\ell-1}}{4h^2}$$

$$C_{j,k} = 18 \log_{10} \left(\frac{11.H_{Uj,k}^{\ell}}{30.2.z_o}\right) \qquad 10.5$$

and z_o is the roughness length.

$$V_{j,k}^{\ell+1} = \{2T(-f\, U_x^{\ell-1} - U_x^{\ell-1} \frac{(V_{j+1,k}^{\ell-1} - V_{j-1,k}^{\ell-1})}{4h}$$

$$- \frac{g}{2h} (\xi_{j,k+1}^{\ell} - \xi_{j,k-1}^{\ell}) + A_H V_G^{\ell-1}$$

$$+ \frac{\rho_a \gamma |W| W_y}{\rho H_{Vj,k}^{\ell}}) + V_{j,k}^{\ell-1}\} /$$

$$\{1 + \frac{2gT[(U_x^{\ell-1})^2 + (V_{j,k}^{\ell-1})^2]}{C_{j,k}^2 \cdot H_{Vj,k}^{\ell}} + \frac{2T}{4h} (V_{j,k+1} - V_{j,k-1}) \qquad 10.6$$

where $U_x^{\ell-1} = (U_{j,k}^{\ell-1} + U_{j,k+1}^{\ell-1} + U_{j-1,\,k+1}^{\ell-1} + U_{j-1,k}^{\ell-1})/4.$

$$H_{Vj,k}^{\ell} = D_{Vj,k} + (\xi_{j,k+1}^{\ell} + \xi_{j,k}^{\ell})/2$$

$$V_G^{\ell-1} = \frac{V_{j+1,k}^{\ell-1} + V_{j-1,k}^{\ell-1} + V_{j,k+1}^{\ell-1} + V_{j,k-1}^{\ell-1} - 4V_{j,k}^{\ell-1}}{4h^2}$$

$$C_{j,k} = 18 \log_{10} \left(\frac{11.H_{Vj,k}^{\ell}}{30.2\ z_o}\right) \qquad 10.7$$

The continuity equation is,

$$\xi_{j,k}^{\ell+2} = \xi_{j,k}^{\ell} + \frac{T}{h}\{-[D_{Uj,k} + \tfrac{1}{2}(\xi_{j,k}^{\ell} + \xi_{j+1,k}^{\ell})]\, U_{j,k}^{\ell+1}$$

$$+[D_{Uj-1,k} + \tfrac{1}{2}(\xi_{j,k}^{\ell} + \xi_{j-1,k}^{\ell})]\, U_{j-1,k}^{\ell+1}$$

$$- [D_{Vj,k} + \tfrac{1}{2}(\xi_{j,k}^{\ell} + \xi_{j,k+1}^{\ell})]\, V_{j,k}^{\ell+1}$$

$$+[D_{Vj,k-1} + \tfrac{1}{2}(\xi_{j,k}^{\ell} + \xi_{j,k\ 1}^{\ell})]\, V_{j,k-1}^{\ell+1}\} \qquad 10.8$$

Where possible the velocity in the frictional resistance and convective momentum terms are shifted in time to the current time step. It is noted, however, that Tee (1976, p. 607) found that the explicit and semi-explicit formulations of the friction terms are the "same and equal to the analytical value and shifts in frequency due to the finite difference forms of the equations are small and can be neglected." The semi-explicit formulation was maintained, nevertheless, mainly to ensure numerical stability in very shallow water over the inter-tidal flats (Section 9.12.2).

The velocity in the Coriolis term is derived from the four surrounding speeds. The order of the approximation can be improved according to Fischer (1965) and discussed in Ramming and Kowalik (1980), by using the average of the updated and old water speeds in the Coriolis term, that is,

$$\text{Coriolis acceleration} = f\left(\frac{U^{\ell+1} + U^{\ell-1}}{2}\right) \qquad 10.9$$

By rearrangement of the equations, the explicit nature can be maintained (Ramming and Kowalik, 1980). However, the computation time rises because the solution is obtained in two half steps, and so this was not included in the model. The average of the four surrounding speeds is also used in the frictional retardation term for the transverse velocity.

The equations are solved by a leap-frog procedure where the speeds (U,V) are found at the first half-time step and then the sea levels are obtained at the next half step.

As presented, the sea levels and speeds on the right hand side of the continuity equation (10.8) are at different time levels. It is possible to solve the continuity equation in two half time steps (Lax and Wendroff, 1960) by finding $\xi^{\ell+1}$ from ξ^{ℓ} for the entire model region then repeating to find $\xi^{\ell+2}$ from ξ^{ℓ} with $\xi^{\ell+1}$ used in the velocity terms. Numerical tests with the double time step procedure showed small, neglible differences in the solution (Runs 166 and 167), and the computation time was reduced without it.

10.4.1 Bathymetry representation

The grid (Figure 10.1) is staggered and is the same as that employed by Leendertse (1967) except for the depth below datum locations. Normally the depth is represented at the corners of the cells (see also Falconer, 1980) and then the depth at the mid-point of any side is obtained by averaging bounding values. In this case, each depth is involved in four averages to derive four depths at adjacent velocity nodes. Over rapidly changing topography, bathymetry schematisation was difficult to undertake. An adjustment to one depth affected 3 others and on the 250m minimum grid size imposed by the limited computer memory, juggling to accurately represent all four surrounding nodes proved to be impossible in the vicinity of the steep harbour gorge. It was not possible to halve the grid size due to the memory

requirements so the depths were made independent of each other by shifting them to the centre of the cell sides at the same location as the water speeds.

On an m x n grid, the number of intersections of the corners is m.n whereas, when depths are placed on the mid-points of the cell sides, for an m x n grid, the number of faces is,

$$n(m-1) + m(n-1) = 2mn - m - n \qquad 10.10$$

As m and n become large this approaches 2mn. Thus, nearly twice as many depths must be input and accordingly a better representation of the bathymetry can be produced. However, no extra output is gained, i.e. the number of speeds and sea levels determined is fixed. Thus, the memory requirements are only marginally increased relative to reducing the grid size to achieve a similar effect.

Markedly improved results were obtained after this modification was added to the program, and schematisation was a much simpler proposition. The depth along a side was set to the average of all depths along that side and being independent of its neighbours, a much more accurate schematisation was possible.

Each node has depths below Chart Datum $D_{Uj,k}$ and $D_{Vj,k}$ to correspond with the locations of the U and V speeds. Total water depth at a velocity node is then,

$$H_{Uj,k} = D_{Uj,k} + \tfrac{1}{2}(\xi_{j+1,k} + \xi_{j,k}) \qquad 10.11$$

$$H_{Vj,k} = D_{Vj,k} + \tfrac{1}{2}(\xi_{j,k+1} + \xi_{j,k}) \qquad 10.12$$

By using depths at cell mid-points, the model allows one or more sides of a cell to be above sea level and obstructing flow, while circulation continues through the others. This procedure was employed several times and meant that an entire cell did not have to be closed to represent narrow, distended subaerial features such as the shell ridge on Snake Bank. Instead, only one wall had to be established as a permanent barrier and on the limited, minimum grid size, much better overall representation could thus be obtained.

10.5 Accuracy of the scheme

To examine the accuracy of the explicit method, comparisons will be made with the Danish Hydraulic Institute System 21 Model (Abbott et. al. 1973) which uses a mixed-implicit scheme similar to that of Leendertse (1967) but employs a Double Sweep alternating direction (DS) method of solution. The DHI numerically modelled Whangarei Harbour for the NHB during the course of this research making it possible to compare the two models when applied to the same field case.

10.6 Cell velocity

In the square cells, U and V speeds are respectively situated on opposite

walls of the cell (Figure 10.1). The velocity at the cell mid-point, corresponding with the location of the sea level node, is derived using the separate average of the U and V velocities on opposite cell walls added vertorially.

10.7 Stability criterion

In two-dimensions, the stability criterion for the explicit scheme can be approximated by the well-known Courant, Friedrich and Lewy condition (Ramming and Kowalik, 1980),

$$\frac{h}{T} \leq \sqrt{2g\ (d+\xi)} \qquad 10.13$$

10.8 Zig-zagging and land/sea boundaries

If a cell is designated as a "land" cell then all four speeds on the cell sides are set to zero. The nearest non-zero speed is half a cell width off-shore and flow is parallel with the boundary. In a dry cell in the inter-tidal zone or on a sand bank, all four velocities are also set to zero.

It was stated by Ramming and Kowalik (1980) that boundaries of this kind need no special treatment other than to ensure that the speeds are zero on the land. Under conditions where the sea bed is gently sloping and comes slowly out of the water so that the velocity gradient is correspondingly slow, numerical tests showed that this is correct. However, for a sudden discontinuity in depth, such as the introduction of a groyne into the flow, or at Marsden Point near the entrance gorge where sea bed slopes are steep and speeds may change from 1.0 $m.s^{-1}$ to zero in half a cell width, it is not possible to leave the scheme unmodified. It was found that a poor approximation in these situations for the velocity gradient in the convective momenta terms results in a phenomenon known as "zig-zagging".

Two examples of zig-zagging are shown in Figure 10.2 from Tee (1976) who employed an explicit finite difference scheme and Figure 10.3 showing output from the DHI S21 model of Whangarei Harbour using a 30s time step on a 150m grid. In the latter case, maximum depths were about 35m and the Courant Number,

$$Cr = \frac{T}{h}\sqrt{2gd} \qquad 10.14$$

was therefore about 5.

Zig-zagging was described by Abbott (1979, p. 260) who stated that it may be eliminated by raising the accuracy of the convective momenta terms, from a second order accurate scheme. Falconer (1980, p. 32) presented a three-level scheme to allow time centering of the convective momenta terms to reduce numerical stability particularly "in harbours and estuaries where a narrow entrance exists".

At Whangarei it was found that neither time centering or the order of

the scheme was the primary cause of zig-zagging but that it was due to a poor approximation for the convective momenta terms at land/sea boundaries where the depth increased rapidly.

10.8.1 Treatment of zig-zagging

Figure 10.4 shows flow ebbing from Whangarei Harbour and, near the entrance gorge, severe zig-zagging is evident. With numerical tests, the specific cells responsible for this were isolated and found to occur where velocity gradients were steep at land/sea boundaries in the deep, steep-walled gorge. In these regions, in actuality, the velocities change quickly very close to the land but the transverse gradient is slower further offshore. The linearisation of the velocity gradient in the centred convective momenta terms with zero speed on the land provides a poor approximation in these cases and zig-zagging results. After experimentation with various finite difference schemes it was found that a sufficiently accurate approximation for the derivatives can be obtained in these instances by offsetting the derivative away from the land (forwards or backwards) when a zero speed is encountered (see Figure 10.5 for an example). Thus,

$$\frac{\partial u}{\partial y} = \frac{U_{j,k} - U_{j,k-1}}{2h} \quad \text{for } U_{j,k+1} = 0 \qquad 10.15$$

$$\frac{\partial u}{\partial x} = \frac{U_{j,k} - U_{j-1,k}}{2h} \quad \text{for } U_{j+1,k} = 0 \qquad 10.16$$

$$\frac{\partial u}{\partial y} = \frac{U_{j,k+1} - U_{j,k}}{2h} \quad \text{for } U_{j,k-1} = 0 \qquad 10.17$$

$$\frac{\partial u}{\partial x} = \frac{U_{j+1,k} - U_{j,k}}{2h} \quad \text{for } U_{j-1,k} = 0 \qquad 10.18$$

and similarly for the V speed derivatives.

This procedure completely cured the zig-zagging in the narrow, deep entrance to Whangarei Harbour where flow is strongly convergent. As only three cells were used to represent the entrance where cross-channel depths changed from zero to 30m in 1½ cell widths, the elimination of zig-zagging was a major breakthrough. Successful modelling in such a difficult environment was then possible.

There are some bathymetric configurations - particularly over wide intertidal zones adjacent to deep channels - where the offset derivative may offer a considerably worse approximation than the centred derivative. Accordingly, 2DD was established with three options, - The convective momenta finite difference term can be (i) always centred; (ii) always offset if a zero speed is encountered; or (iii) treated individually. In case (iii), if there are

only limited troublesome regions, the modeller may force the program to offset only those derivatives which encounter a symbolic depth of -99.9m entered by the modeller into the depths grid at selected nodes.

10.9 The eddy diffusion term

The eddy diffusion term diffuses velocity throughout the model by adjusting a central velocity at each node according to the relative magnitudes for four surrounding velocities. (Abbott (1979, p. 204) describes this term as a "centred dissipative interface".) Considering Figure 10.6, including only the eddy diffusion, the central velocity, V_o, is found as,

$$V_o = V_o + (V_1 + V_2 + V_3 + V_4 - 4V_o) \times (A_H/4h^2) \qquad 10.19$$

where A_H is the horizontal eddy diffusion coefficient.

10.9.1 Land/sea boundaries

When V_3 is on land (Figure 10.6) and if all offshore water speeds are approximately equal then,

$$V_o = V_o + (V_3 - V_o) \times (A_H/4h^2) \qquad 10.20$$

If the land speed V_3 is set equal to zero as is normally done (Ramming and Kowalik, 1980), the eddy diffusion term reduces offshore speeds by absorbing energy from the flow at the land/sea boundaries. This establishes a horizontal boundary layer which, because of the finite cell widths, has a thickness of the same order as the grid spacing. In some cases, this may be a considerable distance and can be physically untenable.

Tee (1976), for example, noted this problem on a grid with spacing of 2.8km and compensated for it by situating the rigid boundary inside the coastline. His method, however, has limited precision and, as it requires a sophisticated "guess" for application, it is considered to be unsatisfactory.

10.9.2 Percentage slip

A new notion of percentage slip at the land/sea boundaries was introduced into 2DD. Using a simple, numerically-fast procedure, the land speed V_3 was set to some percentage, s, of the offshore speed as,

$$V_3 = s.V_o/100 \qquad 10.21$$

where the percentage slip is such that $0<s<100$. The boundary is nonslip with $s = 0$ and free slip with $s = 100$. Any percentage of slip may be modelled and the value of s is selected by the modeller at the computer keyboard. Notably, s only affects the diffusion term at the land/sea boundaries.

10.10 Open boundaries

The staggered grid introduces an asymmetry at the boundaries depicted in Figure 10.1. Left and bottom boundaries are similar, as are top and right

boundaries. The programming has to treat these individually. Accordingly, a system of symbolic integers was established to nominate specific cell types. Open-sea boundaries are identified by the modeller with codes of 4-7 inclusive which are associated with the grid sizes in an anti-clockwise direction starting from the bottom of the grid (Figure 10.1).

The program allows the modeller to specify either the velocities or sea levels at the open boundaries.

10.10.1 Bottom boundaries, Type 4

For a specified sea level, the U speed in each open boundary cell is set equal to its counterpart at the next internal node within the grid. Notably, this speed is parallel with the boundary and setting it equal to the nearest internal value means that the grid refreshes the boundary conditions instead of the boundaries refreshing the grid (Abbott, 1979). This, of course, only applies to the velocities. The sea levels ultimately determine the behaviour of the model.

Using this procedure, inward and outward flow can behave differently as the model speeds depend on either where the flow is going or where the flow has been respectively. In actuality, for inwardly-directed flow the boundary condition is less stable. Other alternatives were tried, including calculating the speed with a simplified momentum balance equation, and instability resulted. These boundary speeds are only used in the convective momentum and diffusion terms at the next internal node and the small errors from the boundary condition are not felt to be significant. This procedure is much improved over that of Tee (1976) who assumed that the flow was always parallel to the land at the boundary. This is often incorrect and is definitely not the case at Marsden Point.

The V speed is found with a simplified version of the momentum balance equation including all terms except convective momentum and eddy diffusion.

For a specified flow speed perpendicular to the boundary (V), the sea level and U speed are set equal to the values at the next internal node.

10.10.2 Right and top boundaries. Types 5 and 6

At these boundaries, both the U and V speeds are set equal to the value of their counterparts at the next internal node for a specified sea level boundary condition.

If the perpendicular flow velocity is given, then sea level is found from the continuity equation, and the other speed (parallel to the boundary) is obtained from the simplified momentum balance equation noted above, without convective momentum or eddy diffusion.

10.10.3 Left boundaries. Type 7

These are similar to Type 4. The parallel speed (V in this case) is set

equal to the next internal node, and the U speed is found from a simplified form of the momentum balance equation for a given sea level condition. If perpendicular speed is specified then sea level and parallel speed (V) are both set equal to the next internal node.

10.11 Dry, wet and inter-tidal cells

As noted, a system of integers is used to nominate the cell types. With these, the modeller must not only describe the boundary cells but also nominate the land cells which are permanently dry (Type 0) and those cells in which sea levels and speeds are to be computed (Type 1).

2DD also keeps an internal record of the cells which are Type 1 but have been exposed by the falling tide. Their code is set to 3 in this case. Figure 10.7 shows the computer output at the beginning of a modelling run at low tide with the codes showing the land (0), and water (1) and the inter-tidal zones (3). Particularly evident are Snake and McDonald Banks being Type 3 at low tide.

10.12 Bed friction

The Chezy's C formulation was employed to describe the bed friction. This requires a roughness length to be input for evaluation in the formula (Chapter 6),

$$C = 18.\log_{10}(11(d+\xi)/30.2z_o) \qquad 10.22$$

The equation has a depth dependence with must be evaluated at each U and V node for every iteration. The logarithm is a slow computer function and computation times were noticeably increased with the above formula employed instead of constant C which was initially tested.

To reduce computation time an approximation was introduced. Using the first term of the series expansion for natural logarithms (Thomas, 1960), it was found that C may be approximated by,

$$C = 18\left[\frac{y-1}{y+1}\right] - \text{const} \qquad 10.23$$

where $y = 11(d+\xi)/30.2$, $\text{const} = 18.\log_{10}(z_o)$

Figure 10.8 shows the comparison between the exact solution and the approximation and the differences are small over the range of interest. The maximum deviation is only 6% up to 30m while in the more common range of depths less than 15m, the deviations are much smaller. Notably, as the approximation only treats the water depth, the equation is valid for all z_o. This simpler formula is handled quickly by the computer.

10.13 Wind stresses

The wind stress term is given by,

$$\tau_s = \rho_a \gamma |W| W \qquad 10.24$$

where W is the wind speed, ρ_a is the density of air, γ is a drag or wind resistance coefficient given in Chapter 9 (equation 9.36). As formulated γ has a linear dependence on the absolute magnitude of velocity. In two orthogonal directions, the components are,

$$\tau_{sx} = \rho_a \gamma |W| W_x \qquad 10.25$$

and,

$$\tau_{sy} = \rho_a \gamma |W| W_y \qquad 10.26$$

(see also Falconer, 1980), and W_x, W_y are the components of the wind in the x and y directions.

10.14 Coriolis parameter

The Coriolis parameter, f, depends on Latitude, ϕ, as,

$$f = 2(2\pi/T_s) \sin \phi \qquad 10.27$$

where T_s is the period of one sidereal day and is $T_s = 86{,}164$s. For Whangarei at Latitude $-35^{o}94'$, the Coriolis parameter is $f = -8.53 \times 10^{-5}$. The Coriolis force, which arises from the rotation of the earth, diverts flow to the left in the Southern Hemisphere.

10.15 Flooding and drying in the inter-tidal zone

Whangarei Harbour, like most meso-scale tidal range inlets, has large inter-tidal zones. For accurate modelling it was necessary to incorporate flooding and drying of these flats into the numerical procedure.

A zero depth is numerically difficult to handle, so an arbitrary minimum depth (chosen as 0.05m for 2DD) was defined as a drying limit. For water depths less than 0.05m the cell side is considered to be above the water level, no flow can pass across it and speed is set to zero. The four sides of the cell are treated separately which allows much smoother flooding.

For presentation of results, an entire cell is considered to be dry if the mean of the four side depths below Chart Datum, plus the central sea level is less than 0.05m. If,

$$D_{j,k} = \frac{1}{4} (D_{U\,j,k} + D_{Vj,k} + D_{U\,j-1,k} + D_{Vj,k-1}) + \xi_{j,k} < 0.05\text{m} \qquad 10.28$$

the cell is represented as being dry even though, within the program, flow may be occurring across one or more of the sides.

The major difficulty encountered with the flooding and drying process was the high frictional energy losses at very small depths which retarded the flow to such an extent that the sea level in the cell lagged unrealistically behind its deeper neighbours (Chapter 9). An "effective" depth was defined for use in the frictional retardation term in the hydrodynamic equation only, whereby a constant of 0.5m was added to the actual water depth in all cases.

10.16 Complementary programming

A model of this type requires considerable supplementary software for data manipulation and plotting. This section briefly describes the supporting software for 2DD.

10.16.1 2DBORD, 2DBOR2

These two programs handle operations on the boundaries particularly when changing from a coarse to a fine grid and sea levels for the latter case are to be found from the former. 2DBOR2 interpolates for gradients along the boundaries using a fitted parabola.

10.16.2 2DPLT

Velocity arrow plots at chosen model times are drawn to a selected map scale and a separate velocity arrow length scale. There are many examples of the output; see Figure 10.37.

10.16.3 2DPLT2

This program produces x-y plots of sea level or water speed against time for selected locations in the grid. During calibration and verification of 2DD measured velocities were compared with the model output using this program. Locations of the measured sites mostly do not correspond with the cell mid-points and, over steeply-sloping topography with large, horizontal, velocity gradients, the comparisons were difficult to make. It is possible to plot more than one velocity-time scale from nodes bounding the measurement site (e.g. DHI, 1982) but this does not allow accurate comparison particularly on coarser grids. Accordingly, for 2DD a linear interpolation was introduced into 2DPLT2 to find speed at any chosen location on the grid nominated by decimal x,y coordinates related to the cell numbers. The interpolation uses the four nearest, surrounding speeds (V_1, V_2, V_3, V_4) at linear distances (d_1, d_2, d_3, d_4) respectively from the chosen location. Then,

$$V = \left(\frac{V_1}{d_1} + \frac{V_2}{d_2} + \frac{V_3}{d_3} + \frac{V_4}{d_4}\right)\left(\frac{1}{1/d_1 + 1/d_2 + 1/d_3 + 1/d_4}\right) \qquad 10.29$$

Several curves can still be placed on the same graph.

10.16.4 2DFLUX

The discharges through chosen cross-sections are calculated and a disc file is constructed for plotting with LINPLT (Appendix 3). The discharges are found by summing the products of perpendicular flow velocity and the water depth along the cross-section.

10.16.5 RESOUT

This program was written to calculate the vector residual velocity over a tidal cycle or between two selected times. A threshold velocity can be

imposed. A resultant distance is also determined which is the product of the residual velocity multiplied by the time above threshold. All output disc files are in a format suitable for plotting with 2DPLT. Ebb and flood velocities may be individually scaled to a selected common tidal range to reduce the effect of semi-diurnal tidal amplitude asymmetry or to scale for Neap and Spring tides.

10.16.6 PERCNT

Often it is necessary to compare flow conditions for two bathymetric configurations, e.g. with and without the timber port or sometimes one input parameter may be changed to note its influence as comparing calm and windy conditions. For these situations, PERCNT derives the comparative percentage changes in water speeds and sea levels and the differences in flow directions for line printer output if requested. Onto disc, it creates a file containing the vector difference in water velocities which can either be arrow plotted using 2DPLT or graphed in 2DPLT2 to note the variations over the tidal cycle. Moreover, the percentage change in water speed may be plotted as an integer number at each node in 2DPLT (Figure 10.72).

10.17 Test rigs

A series of test rigs were modelled to ascertain the power of 2DD and some very pleasing results were obtained. The rigs were chosen for comparison with the DHI Model S21 published by Abbott (1979). All applicable rigs presented by Abbott were tested.

10.17.1 Steady-state flow past a rectangular step

Sudden divergence as flow leaves a confined channel is shown in Figure 10.9. Parameters for this run were depth, 5m; $A_H = 5\ m^2.s^{-1}$; step length, 2000m; Chezy's C, 60 $m^{\frac{1}{2}}.s^{-1}$; and a tide level difference maintained at 0.10m at the open boundaries to drive the circulation.

Maximum speeds in the narrow channel are 0.8 $m.s^{-1}$. Beyond the step, flow diverges and decelerates in response to redistribution of velocity across the width. A small, recirculating eddy has developed in the lee of the step.

In Figure 10.10, the physical dimensions have been reduced and the lee face of the step is now 400m long. Sea levels were lowered to a difference of 0.02m at the open ends and all other parameters were held constant. A free slip boundary was used.

On the smaller physical scale, a strong eddy has developed and near the wall beyond the step, reverse flow attains 0.26 $m.s^{-1}$ compared with oppositely-directed currents of 0.90 $m.s^{-1}$ in the unrestricted flow. The physical scale of the modelled step and the flow speeds are typical of the extension into flow of the proposed timber port at Marsden Point, and the model shows the circulation expected to occur. The lee eddy should cause a build-up of sand

along the side walls under reversing tidal flow.

Specific speeds and directions cannot be compared with S21 because there is no information about the parameters used in the latter for frictional retardation, eddy diffusion etc. or about the physical size of the step. However, the general comparison of 2DD with S21 (Figure 10.10) is excellent.

The 2DD numerical open boundary procedure, previously described, is seen in this Figure to be behaving very well. There is no recognisable discontinuity of the recirculating flow at the open boundary with this difficult test rig.

10.17.2 Convective acceleration

The convective acceleration terms representing the influence of horizontal velocity gradients and the propogation of vorticity, are responsible for the eddy circulation. With these suppressed (Figure 10.11), the recirculation previously evident beyond the rectangular step completely disappears (c.f. S21). Flow along the wall without the convective terms is directed upwards at about 0.3 $m.s^{-1}$ (Figure 10.11) but with the terms included, flow is oppositely-directed with a magnitude of 0.26 $m.s^{-1}$ (Figure 10.10). Furthermore, in the narrow part of the channel, the velocities are nearly 20% greater (0.90 to 0.74 $m.s^{-1}$) with the convective terms included. Considering this in one-dimension, the reduction of speed downstream due to the widening of the channel, allows the convective momentum term to support an increase in speed in the main flow upstream. Thus, convective momentum has a considerable influence and it is evident that the improved treatment of these terms of land/sea boundaries was an important result. It would not have been possible to model Whangarei Harbour by suppressing the terms.

Moreover, Tee (1976) using numerical model tests in the Minas Basin, Bay of Fundy concluded that Eulerian residual currents over a tidal cycle are established by convective momentum. For reversing tidal flow, the modelling over the rectangular step supports this. The current is directed into the base of the step during both the ebb and flood phases with the convective momenta terms included.

10.17.3 The eddy diffusion

10.17.3.1 *Zero diffusion*

Figure 10.12 shows flow past the rectangular step with the eddy diffusion coefficient set to zero. There are some small instabilities particularly evident near the top boundary where the centre node has gained velocity at the expense of its right-hand neighbour. Without diffusion, the redistribution of momentum is not being smoothly apportioned. As the diffusion of momentum is a natural physical process, it would not be expected that the model would adequately operate without this term included. The comparison with the DHI

model using $A_H = 0$ shows that the mixed-implicit scheme is not affected as much as 2DD by the exclusion of eddy diffusion. This is also the case with Falconer's (1980) mixed-implicit model which uses no horizontal eddy diffusion.

10.17.3.2 *High diffusion*

Figure 10.13 shows the same rig with a very high eddy diffusion coefficient of 100 $m^2.s^{-1}$. In this case, the diffusion of velocity is so strong that the eddy circulation has completely disappeared. Similar damping of recirculation occurs with S21.

10.17.3.3 *"Correct" diffusion*

Ramming and Kowalik (1980) suggest that a good approximation for A_H can be obtained with a formula which depends on the model time step and grid spacing.

$$A_H \simeq 0.01\ h^2/T \qquad 10.30$$

Introduction of this into equation (10.20) gives,

$$V_o = V_o\ (1 - 0.01/4T) \qquad 10.31$$

for the land speed $V_3 = 0$. As the grid spacing is not included, the boundary layer thickness will be equal to several cell widths, and as noted previously, this may not be physically plausible. However, an eddy diffusion formulation of this kind is necessary within the grid away from the boundaries, otherwise on coarse grids, diffusion would be neglible.

For the grid size and time step in the rectangular step test rig, equation (10.30) gives $A_H \simeq 6$ and most tests were conducted with $A_H = 5\ m^2.s^{-1}$. As has been seen in test rigs so far, this evidently provides a suitable estimate of A_H.

10.17.4 Percentage slip tests

With $A_H = 5\ m^2.s^{-1}$ in all cases, the rectangular step was modelled with a non-slip (Figure 10.14), free-slip (Figure 10.15) and a 50% slip (Figure 10.16) boundary. Along the straight wall and in the unrestricted flow, the non-slip boundary causes a considerable reduction in flow speed. Moreover, the recirculation behind the step is also grossly reduced, while for the free slip case (Figure 10.15), reverse flows was fast. As speeds in the main flow are reduced by damping at the boundary, the eddy diffusion behaves as a friction coefficient at the side walls which can have a large effect noting that flow speeds were reduced by about 30% in the approach channel in the non-slip case (Figure 10.14). Thus, the choice of eddy diffusion which best models the internal nodes of the grid may not be suitable at the boundaries without compensating by allowing a percentage of slip to occur at the boundaries.

10.17.5 Steady-state flow around a groyne

Steady-state flow around a groyne is shown in Figure 10.17. The open boundaries were held fixed with level differences of 0.10m and the steady state developed after approximately 600 interations from a cold start. Depths were constant at 5.0m and Chezy's C was fixed at 60 $m^{\frac{1}{2}}.s^{-1}$, corresponding with a roughness length of $z_o = 0.0008m$. The horizontal eddy viscosity coefficient was set to 5 $m^2.s^{-1}$. The groyne is 2000m long and the width of the channel is 4250m.

The model shows flow convergence around the end of the groyne due to the reduction in cross-sectional width with the groyne in place. This convergence is a well-known phenomenon which can be responsible for eroding a channel at the toe of the groyne. In its lee, a recirculating eddy has developed due to the shear of the horizontal velocity gradient from maximum speeds in the main steam of about 0.7 $m.s^{-1}$ to much reduced values in the shelter of the groyne. As seen with the rectangular step, the strength of this recirculation depends primarily on the physical dimensions of the structure in relation to the speed of the water currents. Along the straight wall on the right-hand side, a boundary layer has developed with smaller speeds near the wall due to the horizontal diffusion term along the non-slip boundary.

These features are all shown on the S21 output except, along the straight wall on S21, no boundary layer is evident. S21 has used a free slip boundary condition, for reasons which are not stated by Abbott.

There is no zig-zagging evident in either case. In 2DD, the convective momenta partial differentials were offset whenever a zero speed was encountered i.e. away from the land at land/sea boundaries. In Figure 10.18 for comparison, the convective acceleration terms were always centred in space and zig-zagging upstream of the groyne has developed. This is compared with a similar output from S21.

10.17.6 Discussion

These test rigs have highlighted two major aspects:

(i) the power of 2DD to simulate hydrodynamic flows.

(ii) the similarity of results from 2DD (explicit scheme) and S21 (mixed-implicit scheme). No important differences between the two models were noted.

10.18 Modelling of Whangarei Harbour

2DD was applied to the modelling of Whangarei Harbour. This gave the model an important field test for verification of its behaviour and estimation of its power to model estuarine flow. As stated by Falconer (1980, p. 37), "The most appropriate method of verifying the results of such a model is to compare them with prototype velocity measurements for some particular

harbour or estuary." However, often these measurements are not available due to the "general difficulties and expenses in obtaining prototype velocities". The field test of 2DD was made possible by the considerable amount of field data that had been collected in Whangarei Harbour, allowing detailed model calibration and verification.

Whangarei Harbour is a difficult estuary to model being typified by fast-flowing currents in a narrow, deep entrance which widens suddenly into a shallow expanse of channels and inter-tidal banks. Large flood-tidal deltas which are exposed at low tide flank two major channels passing through the model region. The topography is variable and sea bed gradients are steep. Accordingly, Whangarei Harbour provides an excellent test of 2DD and numerical instabilities or an inaccurate finite difference solution should become evident in this difficult environment.

10.18.1 Grid size

Accurate bathymetry may be the single, most-important aspect of hydrodynamic modelling. Accordingly, the scale of the area to be modelled imposes a strict upper limit on the bathymetry digitising interval beyond which a faithful representation of the shape of the harbour cannot be attained. Always contrary to this, computer memory and time limitations impose a minimum limit on the digitising interval.

Harbour modelling was undertaken on the NHB PDP 11/23 computer. On this machine, after the monitors, handlers and program listing were stored in core, the remaining available memory was less than 4,500 real words of 4 bytes each. This is a severe limitation and, even though much attention was paid to the efficient use of core, the limits were still difficult to stay within.

To cover the research area from west of One Tree Point including Parua Bay to the Frenchman Island and central Mair Bank, the minimum possible grid size was calculated to be 500m. In places, this spanned the full width of the major shipping channel so good bathymetric representation was not possible. Some runs were made with a grid of this size but the calibrations were mostly unsuccessful due to the poor schematisation.

Eventually, most of the modelling was concentrated in a region bounding the timber port from the entrance throat to One Tree Point. The grid size was 250m and with bathymetry represented on the cell mid-points this is an "effective" cell width of between 125m and 250m relative to other hydrodynamic models (DHI, 1982; Falconer, 1980; etc).

10.18.1.1 *Bathymetry*

The RNZN Hydrographic Office recently undertook a complete bathymetric survey of lower Whangarei Harbour and the Mair Bank vicinity (Chapter 3). With these depths, the harbour was digitised on the 500m and 250m square

grids over the regions shown in Figure 10.19.

Figure 10.20 provides a clear overlay of the bathymetry at the scale of 1:40,000 for the finer grid. Model output is later plotted to the same scale and the overlay can be used for bathymetry specification. Locations of velocity measurement sites are shown in Figure 5.25.

For the coarser 500m grid, Parua Bay was schematised by keeping the surface area and depths as nearly correct as possible. The entrance gorge which is about 730m wide at its narrowest point, had to be widened to 1000m (2 cells) and Passage Island could not be represented. Other bathymetry was input as the mean of the depths along the sides of the cells.

For the smaller 250m cells, the alignment of the grid was mainly chosen for the best placement at the timber port site and entrance throat. The entrance throat was represented with three cells totalling a width of 750m which is a close approximation to the actual case. The major concern in the throat was the representation of Passage Island. It was possible to schematically include Passage Island on the 250m cell width by opening or closing sides of two cells on the northern side of the entrance as shown in Figure 10.21. After numerical tests with these two combinations a better representation of the entrance throat speeds (B-series) was obtained with the cells closed.

Darch Point was represented using one permanently-dry cell wall projecting into the flow and having a depth of -5m (see Figure 10.36, for example). Similarly, the ridge on Snake Bank was represented by single cell walls out of the water.

The alignment of the grid did not favour the schematisation of bathymetry near One Tree Point. Within one cell on the open boundary at One Tree Point, bathymetry changed from dry land, to a nearly horizontal platform above CD passing into a channel with depths of 7m. After tests, best representation was obtained with this cell closed. Further down harbour along the ebb shield separating the sand flats and main channel, similar depth variations occurred within one cell width. Two cells were deepened on one side to be more representative of the channel depths, rather than those of the bank.

In all other cases, the depths were input as the average on the cell walls. The final bathymetry is given in Table 10.1 for the finer grid.

10.18.2 Modelled days

In accordance with the modelling in Chapter 9, and for comparison with the DHI (1982) S21 output, the high Spring tide on 14-Nov-81 and the Neap tide on 3-Dec-81 were modelled.

10.18.3 Boundary sea levels

The one-dimensional model, LINMOD, was shown in Chapter 9 to be

successfully predicting the harbour tidal behaviour. With this model, tidal levels are known at successive cross-sections throughout the inlet. Thus, it becomes feasible to generate boundary sea levels for 2DD with LINMOD.

Use of 1-d and 2-d models in conjunction, which has not been reported elsewhere as far as this writer is aware, can be a very effective procedure. LINMOD has small memory requirements and if it can be shown to be reproducing harbour tidal behaviour then, with the generated boundaries, areas of most concern may be modelled in detail in two dimensions. In view of the results in Chapter 9, this method was applied to find all the boundary sea levels for the Whangarei Harbour simulations.

Using the Marsden Point tide as input at the open boundary in the 1-dimensional model, the up-harbour mean sea levels were obtained with $z_o = 0.001$m (Chapter 9). No density gradients, wind or additional inflows were included, as these were not being represented in the 2-d model. A half cycle prior to the modelling period was used to allow cold start perturbations to dissipate.

The major disadvantage of this procedure is that transverse sea gradients are not predicted by the 1-d model and in a tidal inlet the tide levels across an approximately flow perpendicular boundary are rarely equal. There are two ways to handle this deficiency:

(i) move the boundaries away from the area of interest to allow boundary errors to dissipate. This is a common modelling practice used at open boundaries particularly when the sea slopes are too small to be noted on neighbouring tide gauges.

(ii) estimate transverse boundary gradients and perform successive runs altering the slopes until a good calibration agreement is reached.

Eventually, a combination of these two options was employed. On the finer grid, the very difficult boundary at the entrance, which is subject to large transverse gradients, could not be managed. Accordingly, the grid was extended as a rectangular channel of 5 extra cells long and 3 cells wide each having constant depth (17.5m) so that the channel cross-sectional area was 13151 m^2 below CD. This was found to be the stable area in the harbour gorge (Chapter 3). The length of the extension (1250m) was chosen to be approximately equal to the cell width in the one-dimensional model of 1400m. One extra node was then added to the one-dimensional schematisation (area of 13151 m^2) for generation of up-harbour open boundary levels. In the 2-d model, the transverse gradient at the down-harbour boundary was set to zero and the rectangular channel merely acted as a velocity "ramp" allowing speeds to stablise by the time they reached the area of interest. The model calibrations verify that this commonly-employed procedure is quite acceptable.

At the One Tree Point to Reserve Point boundary (up-harbour), memory limitations prevented a grid extension. Moreoever, the gradient could not be

set to zero because Snake Bank effectively separates two almost isolated channels at the boundary. Thus, it was necessary to estimate the transverse slopes. It would have been much simpler, more acceptable and more accurate if a grid extension could have been accommodated as at the down-harbour boundary, and this writer would have preferred an extension. However, this could only have been managed if the 250m grid size had been increased.

The modeller was given the option to adjust the boundary gradient but not to adjust the mean tide level across the boundary. Accordingly, the gradient was approximated as being linear from One Tree Point to Reserve Point and pivoting about the mid-point of the boundary so that a change of gradient had the effect of lifting one end and causing a corresponding fall at the other. Thus, the mean level always remained unchanged. The severe restrictions of linear slope and constant mean level meant that the modeller had very limited influence.

The final tides and gradients for the fine grid calibration and verification runs are tabulated in Tables 10.2 and 10.3. Each model run took approximately 12 hours to simulate one tidal cycle so it was not possible to allow for a half cycle running in period. The gradients at the down-harbour boundary are set to zero. At the up-harbour boundary, the gradients were firstly approximated with output from the coarse 500m grid and then adjusted with successive runs. The orders of magnitude are certainly correct. Some of the detailed variations are not important and testify, to some degree, to the time when this writer ran out of patience, as small "trial and error" changes represent a considerable amount of total time.

The variation in mean sea level is the primary sea slope driving the flow. It is believed that if the model had been allowed to find its own transverse gradients at the up-harbour boundary an even better representation of the flow speeds over those reported later would have been obtained.

10.18.4 Stability and the limiting time step

The Courant, Friedrich and Lewy condition (equation 10.13) gives the limiting time step. For the grid spacing of 250m ($h = 125$m) and with maximum depth in the grid at high tide of about 32m, the limiting half time step is 5s. Numerical experimentation found that this was approximately correct, but to ensure stability a 3s half time step was used. Each full time step, which updates both the velocity and sea level, corresponds with a real time increment of 6s. A full time step of $2T = 12$s was employed in the coarse grid.

10.18.5 Calibration and verification procedures

The DHI model was calibrated against four Aanderaa continuous-recording current meters and the A, B, C and D series of Braystoke current meter speeds (Chapter 5). The Aanderaa meters were deployed near mid-depth and velocities can be directly compared with the mean flow speeds of the models.

To scale the Braystoke measurements to correspond with velocities during the modelled tidal cycle, the DHI compared measured mean flow velocities at each of the Braystoke sites with nearby, simultaneous Aanderaa records and found approximately linear relationships (DHI, 1982, Figures 3.3). A linear scaling factor was applied to the Aanderaa speeds to represent the Braystoke velocities.

Analysis in Chapter 7 reveals that this linearity is established by a linear correlation shown to occur between flow speed and tidal range at any site. However, because this correlation is site specific, some of the DHI figures exhibit a considerable amount of scatter. Thus, there may be instances where the model is correct and the field data may be subject to a poor scale factor. Moreover, with this method, the "measured" velocity profiles are simply linearly-scaled replicas of the nearby Aanderaa meter speeds. In an estuary, the shape of the velocity-time curve is known to vary across the channels and therefore the models may be accurately reproducing the details of the velocity-time curves, but the "measured" data will not necessarily reflect this.

The correlation of flow speed to the tidal range noted at Whangarei suggests a possibly improved method of scaling the Braystoke data. Maximum ebb and flood mean vertically-averaged velocities recorded at each Braystoke site (Table 5.12) were linearly-scaled according to the corresponding tidal ranges during the modelling periods. In most cases, the results are very similar to those obtained by the DHI but there are occasional notable differences.

For relative evaluation of the models, the 2DD velocities at the Aanderaa and Braystoke sites were traced onto the DHI calibration and verification Figures for their coarse grid with 150m spacing (DHI, 1982, Figures 5.6 and 5.10). The maximum measured mean speeds scaled according to the tidal range are included on the Figures.

The DHI also compared speeds at peak flow through cross-sections at the Braystoke sites. This was not undertaken for 2DD because the interpolation method programmed into 2DPLT2 provides predicted speeds at any chosen location and the cross-section comparisons add no new information. However, to further examine model behaviour, maximum mean vertically-averaged flow velocities at peak ebb and flood derived from the measured vertical profiles at all pertinent Braystoke sites (Table 5.12) were scaled to the appropriate tidal ranges and compared with model speeds.

Finally, drogue paths (Black, et. al. 1982), were compared with model flow directions.

10.18.6 Bed friction

Throughout the model, as is common practice, a constant roughness length

was applied. With the considerable amount of information about the friction coefficients given in Chapter 6, it would have been possible to use individually selected friction coefficients at each node for the Harbour model, or at least to distinguish between megarippled sand and shell. However, computer memory limitations meant that the roughness length for each node could not be stored and no tests with variable friction were undertaken.

The frictional resistance was found to be best represented with $z_o = 0.0007$m. This is less than 0.001m used to generate the open boundary levels with LINMOD and the reduction corresponds with an energy loss in the 2-d case at the boundaries due to eddy diffusion. This value of z_o is equal to median z_o for Whangarei Harbour obtained from the analysis of vertical velocity profiles in Chapter 6.

10.18.7 Eddy diffusion

After experimentation, the eddy diffusion coefficient was set to 5 $m^2.s^{-1}$. This is a factor of 10 less than A_H given by the Ramming and Kowalik formulation (equation 10.30). An eddy diffusion coefficient of 50 (80% slip) unrealistically smooths the horizontal velocity gradients. Moreover, non-slip boundary with A = 50, also causes gross damping of the tidal flows. Use of a low eddy diffusion means that numerical stability is not being maintained by strong damping.

To ascertain the best choice for percentage slip at the boundary, flow speeds were examined in the narrow channel at the entrance throat where its influence is most marked. After conducting tests with 0%, 50% and 100% slip, the boundary was set to be non-slipping for all model runs. However, this decision was made with respect to the magnitude of A_H. Large values of this parameter may need to be compensated for at the boundaries by allowing a percentage of slip to occur.

10.18.8 Convective acceleration

In all model runs, offset (forward or backward) finite difference approximations were used to find the convective acceleration partial derivatives at the land/sea boundaries. No zig-zagging was evident but occasional temporary instabilities occured on the sand flats adjacent to deep channels.

10.18.9 Calibration and verification results

The comparison of measured mean velocities at 20 Braystoke sites is shown in Figures 10.22 to 10.29 for both the calibration and verification runs. The comparisons are excellent with the output from 2DD being equivalent to that of S21. Particularly pleasing is the improved representation of velocity at the port site (A-series, Figure 10.22). This important region is being well simulated by 2DD while S21 is apparently over-estimating the flood currents at the northern end (Site A4, Figure 10.22(4)). 2DD is

behaving poorly on the southern side of the One Tree Point cross-section (C-series, Figures 10.24(1) and 10.28(1)) and this is caused by schematisation of the ebb shield in this region.

One of the most unexpected results is the adequate prediction of speeds across the entrance throat (B-series, Figures 10.23 and 10.27). In view of the fact that the entrance had to be represented by only 3 cells, the good comparison is perhaps unlikely. It shows that the non-slip boundary condition in this region is the correct choice.

Examining the currents measured by the Aanderaa meters, Figures 10.30 to 10.33 which are not subject to any scaling, 2DD is accurately predicting most speeds including those at the Shoal Bay meter during the ebb calibration period (Figure 10.31), which is being grossly under-predicted by S21. Poorest representation occurs at the One Tree Point meter during the flood (Figures 10.31 and 10.33) and, again, this site is located on the southern side of the shipping channel where some bathymetry schematisation had to be undertaken.

The comparison of maximum speeds at all Braystoke sites during peak flood and ebb is presented in Table 10.4 and most deviations are less than 15% of the measured speeds with many being much less than this. This is good correlation in view of the potential errors in the scaling of measured velocities for both Spring and Neap tide conditions. There are a few deviant sites noting particularly Site K on the flood ramp which is apparently being under-estimated during the flood. However, on the northern side of the flood ramp at Site S5, speeds are being accurately predicted.

In Figures 10.34 and 10.35, the velocity directions at peak ebb and flood are compared with drogue paths. The agreement is again excellent with most model velocity vectors being within a few degrees of the tracked drogue directions.

10.18.9.1 *Discussion and implications*

As previously noted, Whangarei Harbour is a difficult estuary to model. The bathymetry is typified by deep, steep-walled channels flanking sub-aerial banks. Flow in the entrance gorge is strongly convergent during the ebb and divergent during the flood. Depths increase rapidly from an 8-9m platform near the port site to more than 30m in mid-gorge less than 500m away. Thus, this successful application of the model in Whangarei Harbour is an important verification of 2DD in a difficult environment.

The comparison with S21 reveals that the explicit scheme behaves as well as the mixed-implicit (DS) model, as was also found to occur with the test rigs. Moreover, in the field case, 2DD was disadvantaged by the coarser grid spacing and could only improve with better representation of the bathymetry.

The results confirm the validity of employing 1- and 2-d models in conjunction for the generation of open boundary levels.

10.18.10 Full tidal cycle velocities

Spring and Neap tide velocities for the model region are shown in Figures 10.36 and 10.37 for complete tidal cycles at 1-hourly intervals. Bathymetry is provided on the clear overlay in Figure 10.20.

The following discussion provides further evidence supporting the model's capabilities while describing the general model results. Several aspects noted in the field measurements are predicted by the model.

(i) Most evident throughout the tidal cycle is the varying exposure of Snake and McDonald Bank as the tide rises and falls. The harbour fringes, flanked by inter-tidal sand flats, are also seen to oscillate with the changing tide levels.

(ii) Strongest currents are restricted to the channels.

(iii) In the main shipping channel, flow constriction near the Shell Cut Reach Front Lead causes currents to converge and faster speeds result during both the ebb and flood.

(iv) Flows are markedly faster in the shipping channel than in the Shoal Bay channel as previously noted with drogue tracking and current meter measurements.

(v) The strongest flood currents occur on the northern side of the entrance throat while the ebb favours a more southerly course.

(vi) The model predicts sub-parallel flow across the sub-tidal spit extending south-east of Snake Bank as noted by drogue tracking.

(vii) Near slack water in the entrance (Figures 10.38 and 10.39), there is a phase difference between the northern, southern and central regions. After the flood tide, the deep central region continues to flow for about 26 minutes after the southern side commences to ebb (Figure 10.38). After the ebb at low water, the southern and northern sides flood early while the ebb continues to jet out of the deeper central region of the entrance for approximately 20 minutes (Figure 10.39). This behaviour was also noted with S21 which predicted phase lags of 22 and 29 minutes after the flood and ebb respectively (DHI, 1982, Figure 5.16).

(viii) The shape of the velocity-time curve on the flood at the southern side of the entrance (Figure 10.23(1)) is unusual with maximum speed occurring shortly after low tide instead of nearer high tide as at most other locations. This was observed in the field data and discussed in detail in Chapter 5. It is seen here to be related to early flooding on the southern shore, as suspected.

(ix) Near slack water at high tide, the plan view (Figure 10.36g) shows that ebbing commences in several regions while inertia sustains flow in the

two main channels. Ebb flow is seen in the McLeods Bay channel behind McDonald Bank, along the sand flats near Point Sinclair and at the port site and along the southern side of the entrance throat. Continuation of strong flood flow is particularly evident on the flood ramp to Snake Bank. Site K in this region was found to be subject to similar behaviour with the flood continuing some 45 minutes after high tide. The model provides an overview which shows that this is related to favoured flood flow on the northern side of the entrance throat and to inertial flow in the channels.

10.18.11 Residuals

Tidal cycle residual velocities represent the Eulerian nett circulation over the tidal cycle at any location. They are formed by summing the vector velocities over one cycle to accumulate the nett or resultant velocity. A complete discussion of the interpretation of linear residuals with regard to sediment transport is given in Chapter 5. In summary, as the transport of sand is a non-linear function of flow velocity, above a critical threshold, the linear residual velocity does not always provide an accurate estimate of relative fluxes of sediment. However, the residual velocities reflect the directions of nett sediment transport and depict inter-related regions and closed or partially-closed circulation loops. These results can be most helpful with the selection of dredge spoil dumping sites or specification of changes to bathymetry induced by "downstream" dredging.

10.18.11.1 *Tidal asymmetry*

Semi-diurnal tidal amplitude asymmetry, which commonly occurs around New Zealand, may introduce a false dominance into the residual vectors (Chapter 5). Accordingly, all residuals presented were scaled linearly according to tidal range using the procedure described in Chaper 5. Ebb and flood velocities were separately scaled to a common range.

Although this removes the influence of tidal amplitude asymmetry, there are always variations in the shape of the daily sea-level curves and in the corresponding velocity-time profiles. Therefore, some differences in the results for the modelled Spring and Neap tides can be expected.

10.18.11.2 *Comparison with field data*

Residuals were derived for all Braystoke measurement sites in Chapter 5 and were presented in Figures 5.41 to 5.43. Comparison of these with the model results presented in Figures 10.40 and 10.41 for Spring and Neap tides shows excellent agreement. Namely: (i) the flood dominance on the Snake Bank flood ramp; (ii) the flood-directed transport along the north and north-east of Snake Bank; (iii) the ebb vectors at the south of the entrance throat and flood vectors at the north; (iv) neutrality along the sand ridge extending south-east off Snake Bank; (v) flood-directed residuals on the sand flat

fronting Point Sinclair up to One Tree Point; (vi) ebb vectors on the ebb shield at One Tree Point; (vii) flood vectors in the shipping channel off One Tree Point; (viii) ebb vectors near the Shell Cut Reach Front Lead on the northern side of the shipping channel; (ix) ebb vectors across Snake Bank upstream of the ebb shield and flood vectors downstream; (x) ebb-directed transport on McDonald Bank; (xi) near neutrality upstream of the NHB wharf; (xii) ebb-directed residuals along the inter-tidal zone at the port site.

In Chapter 5, a considerable amount of supplementary evidence such as bathymetric interpretation, standard tidal delta models, drogue paths, bed sediments, side-scan sonar depictment of bedforms, direct diving observations etc. was provided to support the validity and interpretation of the residuals derived from the field measurements. Here, it is shown that the model predicts the same results, further supporting the validity of the conclusions. Moreover, this very important result confirms that 2DD is an effective tool capable of providing the residual flow vectors.

10.18.11.3 *The influence of tidal current residuals in Whangarei Harbour*

Neutral zones: The side-scan sonar map (Figure 2.7) has been reduced to a scale of 1:40,000 for an overlay (Figure 10.42). When placed over the maps of model residuals, it is evident that sedimentation occurs in all neutral zones. Primary examples of this in regions of high current flow, are the ridge of sediment extending off Snake Bank and the large sand waves off Reotahi Bay where current neutrality is depicted by the Neap tide (Figure 10.41) results. On the sediment ridge extending towards the NZRC wharf from Snake Bank, the model shows high current flows (Figures 10.36) yet neutrality is seen to guarantee ridge stability. The model further demonstrates that the sand waves off Reotahi Bay occur for two reasons: (i) current neutrality and (ii) high current speeds which are responsible for their magnitude. These are the largest bedforms found in the harbour.

There are other examples of neutrality and sedimentation coupling in less active zones. The model isolates a distended neutral region along a band of megaripples which encompass the 5m contour in Shoal Bay off Darch Point. A second, and interesting, case is the small cluster of megaripples on the 5m contour west of the NHB wharf, which coincides precisely with a limited neutral pocket seen in Figure 10.40.

It is evident that in a well-mixed estuary or tidal inlet, subject primarily to moderate tidal currents (Figure 5.27), flow neutrality will be reflected by sediment deposition. This occurs in Whangarei Harbour which receives low inputs of sandy sediment (Chapters 4 and 5) and is more likely to occur when loose sediment is more abundant.

The converse, however, is not necessarily correct, i.e. sediment

deposition does not guarantee tidal current neutrality.

Ebb and flood regions: As noted in Chapter 5, the scoured shell lags are mostly ebb dominant zones, e.g. Shoal Bay on the McDonald Bank side, the proposed port site, the north side of the shipping channel up to the Shell Cut Reach Front Lead, and McLeod Bay. An exception to this is the flood ramp to Snake Bank which is flood dominant and scoured, and is associated with strong asymmetric flow.

Loops and pathways: Whangarei Harbour is typified by several closed or partially-closed recirculating loops. These have been schematised in Figure 10.43. One is centred on the inter-tidal flats at Point Sinclair and encompasses the ebb dominant flow in the shipping channel near the Shell Cut Reach Front Lead. Three loops include the Snake Bank flood ramp. Divergence of flow at the top of the ramp to the south into the shipping channel and north into the Shoal Bay channel account for two of these. A larger loop incorporates ebb-directed flow across McDonald Bank and around McLeod Bay, with flood-directed vectors up the flood ramp and along the north-east side of Snake Bank.

These loops allow sediment to circulate within the model region without any need for further input through the boundaries. Thus, the circulation may be considered as being composed of two parts, namely, a throughput component crossing the boundaries and a recirculating component moving within the model boundaries.

The model confirms that the Snake Bank flood ramp is a sediment pathway. This means that sediment passes through this region with a strong uni-directional component to feed other locations in the lower harbour. Accordingly, interference with this ramp by dredging would be expected to have consequences at all upstream zones as defined by the pertinent loops in Figure 10.43.

10.18.12 Residual distances indicating the sediment transport potential

The linear residual velocities with no threshold speed imposed mostly reflect the nett transport directions. These are not the best indicators of the potential of the flow to transport sediment (Chapter 5). A more useful parameter is the residual distance vector derived from the excess velocity, i.e.,

$$\tilde{D}_r = \int_0^T (\tilde{u} - \tilde{u}_{cr}) \cdot dt \qquad \text{for } u > u_{cr} \qquad 10.32$$

where u_{cr} is the sediment threshold velocity. The vector $\tilde{D}_r$ is thus formed by vectorially averaging the excess velocity over the tidal cycle and multiplying by the time that the velocity is above threshold. This provides a better estimate of the total sediment fluxes that could be moved over the tidal cycle.

A general sediment threshold speed of 0.4 $m.s^{-1}$ was selected for the

mean vertically-averaged model velocities in accordance with the results in Chapter 7. This value, is a loose approximation which takes no account of flow depth or bed roughness. Nevertheless, it allows specification of approximate residual distances for the periods that the flow velocity is above threshold.

Shown in Figures 10.44 and 10.45 for the Spring and Neap tides respectively, and in Figure 10.46 for the Neap tide scaled to the Spring range of 2.6m, the residual distance above threshold is quite different from the residual velocity. The most dominant feature is a continuous ebb-directed pathway running from One Tree Point across the shipping channel, through the proposed dredge zone and out of the harbour on the southern side of the throat. In view of this extraordinarily strong potential for uni-directional sediment transport along this route, it is not surprising that the bed is armoured. Without a very large supply of sandy sediment available at the up-harbour boundary off One Tree Point to feed this pathway, preferential transport of sandy material will lead to a concentration of shell on the bed.

A difference in bed facies on either side of the Shoal Bay channel was clearly depicted by the side-scan sonar and video camera. The flood route at the north-east of Snake Bank was typically sandy and bedformed while the more stable bed on the ebb route was scoured, gravelly and "aged". Again, the reasons for this are evident. The upstream supply for the ebb pathway is not sufficient to maintain the sandy bed. This Shoal Bay route has limited transporting potential during the Neap tides (Figure 10.45).

On Snake and McDonald Banks, the transport potential is very low and, in some cases, is below the chosen threshold even during the Spring tides. Accordingly, these banks should be very stable and this was shown by the comparison of RNZN bathymetry surveys in Chapter 3. During the Neap tides, sediment transport is restricted entirely to the two major channels. In accordance with this, measured currents in shallow water around Snake Bank (Chapter 5), were also found to be below threshold during Neap tides. It was concluded then that sediment transport occurs primarily during Spring tides and under local wind-generated wave action.

The sediment threshold is not exceeded along the sand flat at the port site and this concurs with tidal current measurements described in Chapter 8. Once again, transport in this region was found to occur primarily under wave attack.

10.18.13 Wind influences

Several model runs were made to examine the influence of wind on the tidal behaviour of Whangarei Harbour. The typical 10 $m.s^{-1}$ storm wind from 110°T (Chapter 4) was modelled and some test runs were made with 10 $m.s^{-1}$

winds from 0^{o} and 225^{o} relative to the grid, i.e. 101^{o} T and 236^{o} T.

To correctly model these winds the up-harbour open boundary sea level should be generated in LINMOD with the wind included. This is difficult because the wind can only be represented along the direction of the schematised estuary and the model area exhibits 2-dimensional behaviour. Nevertheless, for the storm wind from 110^{o} T the boundaries were generated in LINMOD after including a flood-directed 10 $m.s^{-1}$ wind because the wind direction is approximately along the axis of the 1-dimensional harbour. For the other two cases, the boundaries were not changed. With the wind from 0^{o} relative to the grid (101^{o} T) the vector velocity differences for the unperturbed and windy case at peak flood and ebb are shown in Figures 10.47 and 10.48. As expected, when the boundaries are not changed, the shear stress at the surface, accelerates the flood currents and decelerates the ebb. Largest changes occur along the inter-tidal zones and over the shallow banks. Retardation of flow during the ebb is more pronounced in the shipping channel than in the Shoal Bay channel (Figure 10.48).

For the approximately south-west wind (236^{o} T) more complex changes occur. During the flood (Figure 10.49), flow is accelerated and turned across the shallow banks. Snake Bank exhibits vector velocity differences of up to 0.15 $m.s^{-1}$ and this change of flow direction causes speeds in the Shoal Bay channel to decrease. However, the velocity magnitudes are not greatly altered as the wind mostly affects the direction of flow. Similar behaviour occurs during the ebb (Figure 10.50), with flow across Snake Bank drawing more water through the boundary at One Tree Point and retarding currents in the Shoal Bay channel.

Figures 10.51 and 10.52 show the modelled simulation of a 10 $m.s^{-1}$ wind from 110^{o} T using updated open boundaries. The results are distinctly different from those previously discussed using unaltered boundaries for the similarly-directed wind from 101^{o} T (Figures 10.47 and 10.48). The flood currents were increased in the channels in the former case. For the updated boundaries, channel velocities decrease to compensate for stronger up-harbour flow over the shallow banks (Figure 10.51). Furthermore, during the ebb (Figure 10.52), with the wind retarding flow off the banks, increased velocities occur in the channels. This is the correct general principle. However, because of the need for one-dimensional schematisation of the wind for generation of boundary levels, the precise choice of a schematised wind strength is difficult to make. It appears that 10 $m.s^{-1}$ winds in LINMOD are too large as the changes to flow velocities in the channels are possibly too substantial.

The residual velocities for the case of 110^{o} T winds, are shown in Figure 10.53 and there is one very important result. The inter-tidal flats at the port site have become neutral instead of ebb dominant. Thus, the

wind stresses, by changing the harbour circulation patterns, could allow sediment to move up-harbour along the inter-tidal flats in an otherwise ebb dominant zone (Chapter 8). This is without the supporting influence of surface gravity waves which are known to enter the inlet during easterly storms.

10.18.14 The Coriolis force

The influence of the Coriolis term in Whangarei Harbour was examined by comparing the unperturbed condition ($f = 0$) with the case including the Coriolis force. During the flood (Figure 10.54) the Coriolis force diverts the flow to the left as is expected to occur in the Southern Hemisphere. Considering the gross circulation, flow on the flood ramp to Snake Bank is diverted into the shipping channel and water is diverted across the upper levels of Snake Bank from the Shoal Bay channel to the shipping channel. This draws water into the northern side of the entrance throat and there is a corresponding decrease in flood currents on the southern side. In each channel, flood flow is faster to the left i.e. on the Snake Bank side of the Shoal Bay channel and the One Tree Point side of the shipping channel.

During the ebb (Figure 10.55) diversion of flow to the left over Snake Bank draws water into the shipping channel at One Tree Point, and reduces the ebb currents into the Shoal Bay channel off Reserve Point. Flow is also diverted from the shipping channel onto the Snake Bank flood ramp over the sediment ridge and is seen to be partially responsible for the sub-parallel flow recorded in this region. This flow diversion causes a general decrease in currents in the Shoal Bay channel and an increase in the shipping channel. There is also a general increase of ebb currents on the northern side of the throat with a very small decrease to the south.

The percentage changes in water speed shown in Figure 10.56 and 10.57, summarise the gross changes. The Coriolis force assists the currents in the shipping channel curing both the ebb and flood and causes a general reduction of ebb currents in Shoal Bay. It reinforces the flood dominance on the Snake Bank flood ramp and along the north-east flank of Snake Bank in Shoal Bay. Flood currents around Snake Bank are about 4% faster and ebb currents are reduced by about 7% while on the flood ramp ebb currents are up to 21% slower with the Coriolis force included and flood currents are 2-3% faster.

Figure 10.58 shows the tidal cycle residual velocities with the Coriolis acceleration neglected. It is evident that there are some changes in the details of the circulation (compared with Figure 10.40). However, the gross circulation which typifies the model region is still clearly defined. Thus, the Coriolis acceleration offers some support to the residual circulation but it is not the primary mechanism responsible for nett tidal cycle currents.

10.18.15 The influence of the convective acceleration

The rectangular step test rig, showed that the convective momenta terms are primarily responsible for eddy circulation. Moreover, it was evident that the recirculation was the cause of residual currents over the tidal cycle for this test case.

Tee (1976) introduces his paper with a detailed list of models which ignore the convective momenta terms and while arguing that this is not a valid approximation, Tee states, "strong tidal currents and complicated geometry can produce strong inertial effects that dominate the generation of the residual current" (Tee, 1976, p. 604). Using numerical tests to support this statement by modelling the Bay of Fundy, Tee found that residual currents were grossly diminished from the order of 0.5 $m.s^{-1}$ to only a few centimetres per second when the convective momenta terms were neglected. Similar tests were performed with 2DD to examine the influence of convection in Whangarei Harbour and to verify Tee's results.

Boundary sea levels were generated in LINMOD with the convective momentum term neglected using the Neap tide (3-Dec-81) for the down-harbour open boundary condition. Then with no change to bathymetry, the harbour was modelled in 2 dimensions with 2DD. All terms were retained except the convective momentum and the Coriolis acceleration. The influence of the Coriolis force has already been examined.

Shown in Figures 10.59 and 10.60, the maximum flood and ebb current patterns are not markedly different compared with Figures 10.37(b) and 10.37(i) with the convective terms included. However, the residual velocities are considerably altered (Figure 10.61). The complex residual circulation pattern which typifies Whangarei Harbour is no longer evident. Moreover, the remaining residuals, being nearly all flood-directed are most likely due to variations in the shape of the tidal curve between the ebb and flood cycles and are not general residual velocities for lower Whangarei Harbour.

Thus, it is apparent that the convective momenta terms are responsible for residual circulation. These describe the horizontal velocity gradients and the advection of inertia.

10.18.16 The influence of the harbour outline and bathymetry

The question raised in this section is whether the bathymetry of the harbour outline is responsible for the velocity gradients and the resulting residual flow pattern. Accordingly, the bathymetry was flattened with the harbour set to 5m deep except in the entrance throat which was set to 10m. Boundary conditions were generated using the verification (3-Dec-81) tide in LINMOD by introducing the same schematisation into the 1-dimensional harbour representation. Transverse gradients were set to zero at both open boundaries.

Flow velocities are shown in Figures 10.62 and 10.63 for peak flood and

ebb. The more-informative residual velocities are presented in Figure 10.64. Even after removing the bathymetric influence, there are still many features of the residuals which mimic those in Figures 10.40 and 10.41 with the modern bathymetry. Amongst these are the flood vectors on the flood ramp and around Snake Bank, the ebb currents off One Tree Point, the ebb currents around McLeod Bay, the ebb vectors at the port site and there is even a sign of the neutral zone which indicates the neutral sediment ridge off the south-east of Snake Bank. The expected dominance is noted in the entrance throat.

A quantitative statement cannot be made partly because no calibration can be performed and the circulation is affected by the choice of transverse gradient at the up-harbour boundary. However, it is evident that the residual circulation is established to a great degree, by the shape of the harbour outline alone. This, in turn, would encourage the formation of banks and channels in particular regions. Accordingly, an insight into the genesis of the estuary has been gained with this modelling.

10.19 The proposed timber port

Modelling was undertaken to examine the impact of the proposed timber port at Marsden Point. The model predicts changes to flow speeds expected to occur after the port construction.

This work duplicates comprehensive analysis already undertaken by the DHI to determine a suitable layout for the timber port. As such, no attempt was made to redesign the port but only to test some of the DHI analysis while generating complete sets of flow velocities for application to the sediment transport model in the next Chapter.

10.19.1 Case A7

Discussion with the NHB senior civil engineer, Mr M. Hunter, revealed that port configuration Case A7 (Figure 10.65) was a likely choice for the port layout although no final decision had been made when this work was undertaken. Case A7, which was only examined in a physical model, is examined in this and the next Chapter. The layout is essentially identical with Case A3 examined in the mathematical model (DHI, 1982) and consists of a 450m wide and 13m deep dredge basin fronting a 540m long quay with the port being adjacent to the present NHB wharf.

10.19.2 Modelled port

On a discrete grid, the port plan has to be approximated and the shape of the modelled port is shown on a clear overlay at 1:40,000 in Figure 10.20.

10.19.3 Model runs

The port was simulated for Spring (14-Nov-81) and Neap (3-Dec-81) tidal conditions with the open boundary sea levels being (i) unchanged from those

runs without the port; and (ii) generated in LINMOD with the port included in the one-dimensional schematisation.

The port is located in Cross-section 6 (Figure 9.10) and reduces the cross-section width from 1400m to 1320m, while the gradient representing the flooding of the inter-tidal zone is reduced from 100 to 36. The cross-sectional area remains unchanged. One model run was made with a 14m-deep dredge zone (rather than 13m) and, in this case, the cross-sectional area was increased by 450 m^2 in the one-dimensional schematisation, while the reduced width and gradient were unchanged from the shallower 13m case. All runs are summarised in Table 10.5.

10.19.4 Comparison of results with and without updated boundary levels

The percentage changes in absolute water speed at peak flow due to the construction are shown in Figures 10.66 and 10.67 for the unaltered boundary sea levels (Case (i) and in Figures 10.68 and 10.69 for the updated boundary levels (Case (ii)). There are differences between the two cases. Overall, speeds are slightly slower (1-2%) with the updated boundaries. As the port and dredge zone spans the approach to the shipping channel and leaves most of the Shoal Bay route on the flood ramp unchanged, it would be expected that the discharge in the shipping channel should increase and correspondingly decrease in Shoal Bay. This is most evident for the updated boundary cases (see Figures 10.68 and 10.69). Accordingly, only these will be discussed further.

10.19.5 Port impact

With the inclusion of the timber port and dredge zone (13m deep), predicted water velocities throughout the tidal cycle at 1-hourly intervals are shown in Figures 10.70 and 10.71 for Spring and Neap tides respectively. Strong currents are evident on the northern side of the dredge zone but slower speeds occur near the quay wall. The horizontal velocity gradient perpendicular to the quay wall is approximately 0.1 $m.s^{-1}$/100m during Spring tides. This would influence ship manoeuvring in the dredge basin but is not as large as the current gradient in the entrance throat near the NZRC wharf which is approximately 0.3 $m.s^{-1}$/100m.

There is a speed reduction in the dredge basin of up to 10% during the flood and a larger decrease of up to 26% during the ebb near the quay wall (Figures 10.72 and 10.73). The simulated quay wall alignment is $11^{o}T$ as against approximately $25^{o}T$ for Case A7. With the western end of the quay wall projecting further offshore, the speed reduction may be slightly less than 26%. Nevertheless, there is a general pronounced decrease throughout the dredge zone.

Immediately downstream of the dredge zone, there are local increases in

flow velocity due to the reduced frictional retardation of currents passing through the deeper water.

Slight velocity increases are evident in the shipping channel and marginal decreases occur in Shoal Bay. Common changes are about 3%. There are some differences between the Spring and Neap cases with a greater percentage reduction in Shoal Bay during the ebb (Figures 10.69 and 10.73) in the Neap simulation.

At the entrance throat near the NZRC wharf, there is an increase in flood and decrease in ebb speeds on the southern shore with changes being of the order of 5-10%. These should negate some of the ebb dominance known to occur on the southern side of the throat. To the north of the throat, the flood has weakened and the ebb has strengthened with changes being mostly less than 5%.

The vector velocity differences between the perturbed condition (with the port) and the original bathymetry are shown in Figures 10.74 to 10.77. The concept in these figures is a little more difficult to understand and changes must be viewed in conjunction with the flow directions at the time. For the flood where flow is from right to left in the dredge basin, oppositely-directed vectors (left to right) show a reduction in speed.

During the Spring ebb tide (Figure 10.75), currents are reduced by 0.28 $m.s^{-1}$ along the quay wall. This is significant with respect to potential sediment transport capabilities, although it is noted again that the quay wall is 14^{o} out of alignment and slightly faster speeds should occur with the western end projecting further offshore. Maximum speed change during the flood in the dredge basin is 0,18 $m.s^{-1}$.

Around the inter-tidal zones, velocity differences are actually very small, even though the percentage changes can be sometimes very large (Figures 10.72 and 10.73).

The vector velocity changes for the Neap tide are much smaller. Maximum change along the quay wall during the ebb is about 0.16 $m.s^{-1}$ (Figure 10.77) and largest change during the flood is 0.11 $m.s^{-1}$ (Figure 10.76).

10.19.6 Comparison with DHI results

The DHI presented contours of the percentage changes to flow speed for Case A3 (which is very similar to A7) and these are reproduced in Figures 10.78 and 10.79, for the flood and ebb respectively. The comparison with Figures 10.72 and 10.73 from 2DD shows similar results noting that the DHI have apparently only included contours for up to 10% changes. Velocity differences in the immediate shelter of the new port, which are necessarily greater than 10% are not evident. Nevertheless, the general trends and in many cases the actual percentage changes compare well.

There is one important exception. In the vicinity of the NZRC wharf,

S21 shows an increase in the ebb speed while 2DD shows a decrease. No immediate explanation can be offered for the results of the DHI model.

There are three factors which support the 2DD assertions:

(i) The DHI noted that the orientation of the southern flank of the drogue zone between the NHB and NZRC wharves has an influence on the resulting flow speeds at the NZRC wharf. For the orientation shown in Figure 10.65, it acts as a submarine barrier which "deflects ebb flow away from the jetties" (DHI, 1982, p. 110). This alignment is very similar to that used in 2DD as it coincides almost precisely with the angle of the bathymetry grid (11^{o}T normal).

(ii) strong flow which converges on the NHB wharf is totally obstructed by Case A7 and field measurements (Chapter 5) show that the present current speeds in this region are considerable.

(iii) deepening on the northern side of the dredge zone allows much easier diversion of sub-parallel flow which has a natural tendency to pass across what was a shallow ridge to enter the deep gorge west of Passage Island.

All of these factors, suggest a decrease in currents near the NZRC wharf with the A7 configuration during the ebb half tidal cycle.

10.19.7 Dredge basin with 14m depths

With the deeper 14m dredge basin, the flow velocities (Figures 10.80 to 10.83) are reduced marginally below those for the 13m depths (Figures 10.72 and 10.73). The changes are usually within 2-3% with slightly greater differences during the ebb.

The changed impact with a 14m-dredge zone is not likely to be of significance. However, it is evident that the port influence is altered. Any large deviations from 13m may need more detailed examination with possibly a change to the port layout and location of the quay wall to minimise the port impact by adjusting the cross-sectional area.

10.19.8 Residuals with the proposed port

The residual velocities for the port simulations are shown in Figures 10.84 and 10.85. The residual distances for speeds above 0.4 $m.s^{-1}$, are presented in Figures 10.86 and 10.87. The differences between these and the residuals for the present bathymetry are small except in the immediate vicinity of the port and dredge zone where some important changes are noted.

At the north-east corner of the dredge zone, over what was formerly a neutral sediment ridge, the vector residuals have a slight ebb dominance for the Spring tide case (Figure 10.84). For the Neap tide simulation, (Figure 10.85) the change is not so large but the flood residual velocity has diminished. Moreover, the residual distance representing the sediment

transport potential shows a distinct ebb dominance (Figures 10.86 and 10.87).

In the lee of the port on the south side of the entrance throat, the ebb-directed residual velocities have been reduced in magnitude and even show a slight flood dominance for about 750m towards the entrance during the Neap tide (Figure 10.85). The transport potentials are small also. There is a reduced flood dominance on the northern side of the entrance.

In the dredge zone there is a reduction in ebb-directed residual velocities and distances.

10.19.9 Sediment transport implications

The influence of these velocity changes after port construction with respect to the sediment budget in the inlet are modelled and discussed in detail in the next chapter. However, general results from the analysis so far suggest several general conclusions. Notably, no estimate of the rate or magnitude of changes discussed below is offered at this stage.

The reduction in ebb and increase in flood currents should induce sedimentation on the southern side of the entrance. Considering this was a stable region subject to grossly asymmetrical ebb-dominant currents, under the new hydraulic regime with low transport potentials, sedimentation should be expected to occur. Moreover, in conjunction with the increased ebb dominance to the north, some realignment of the entrance throat may result with shoaling on the southern side being balanced by scouring at the north.

In the dredge zone, the reduction of speed suggests that sedimentation should occur there. In fundamental terms, the bed may try to attain its previous equilibrium depth of 8-9m below CD. However, this region is still subject to an ebb-dominant transport potential, and sedimentation, if it is to occur, will require a sufficient sediment supply. So far there is no evidence to suggest that this sand is available on the pathway between One Tree Point and the entrance gorge.

Changes to residual flow vectors at the northern side of the dredge zone, show that the ridge of sediment may not be able to re-establish itself in an ebb dominant zone. Accordingly, maintenance dredging requirements could be considerably reduced, because of natural scouring in response to the altered bathymetry.

The small speed changes in Shoal Bay channel should have a very limited influence, with neglible bathymetric changes resulting due to this factor alone.

In the shipping channel, the small speed increases may cause limited amounts of erosion, similar to what has evidently occurred since the Shell Cut Reach channel was dredged (depicted in Figure 3.16 which compares the bathymetry in 1961 with 1981).

These changes are not expected to be of significance to the overall

harbour circulation or morphology. Most important differences occur in the immediate vicinity of the dredge zone, as noted above.

10.20 Conclusions

(i) 2DD is a numerical hydrodynamic model which has been shown to be capable of reproducing currents and sea levels in a tidal inlet. The model solves the hydrodynamic equations in 2-dimensions using a (semi-) explicit finite differences scheme and the accuracy of this procedure is attested to by comparison with (a) test rigs; (b) a field case in an estuary of extremely variable bathymetry typified by a narrow deep entrance throat which widens into an expanse of banks and channels; and (c) the well-renowned System 21 DS model of the Danish Hydraulic Institute which is reputed to have had over 50 field tests in harbours around the world (R. Warren, DHI, pers. comm.). Most 2-dimensional models do not receive such detailed comparative scrutiny when financial restrictions can limit field data collection. Moreover, two different 2-dimensional models are rarely employed to simulate the same estuary. In this writer's opinion, the tests made on 2DD are conclusive and assert its competency when modelling hydrodynamic flows.

(ii) A combination of a semi-explicit scheme presented by Dronkers (1969) and an explicit scheme in Ramming and Kowalik (1980) were utilised. Most important improvements made were (a) a percentage of slip at the boundaries to enable reduction of the thickness of an untenably-large horizontal boundary layer which develops at land/sea boundaries on coarse grids with high values of the horizontal eddy coefficient; (b) the isolation of the cause and the prevention of zig-zagging established by a poor approximation for the convective momenta terms at land/sea boundaries over steep bathymetry where the velocity gradient changes sharply; (d) the introduction of effective depth (after Ramming and Kowalik (1980)) for application to inter-tidal zones where bed friction unrealistically retards flow; (d) a new schematisation of the bathymetry which eliminates the need to average surrounding values to find the depth on the cell walls at the velocity nodes and offers approximately twice the resolution without necessitating significantly greater computer memory requirements; (f) flooding and drying of inter-tidal zones; and (g) an approximation to the Chezy's C function to reduce computation time and which deviates by only a maximum of 6% up to 35m depths from the exact solution.

(iii) The horizontal eddy viscosity was found to be an essential function responsible for the smoothed redistribution of momentum throughout the grid. Similarly, the convective momenta terms were responsible for flow recirculation and the modelling of vorticity established by horizontal velocity gradients.

(iv) The roughness length which best represented the bed friction in

Whangarei Harbour was found to be 0.007m which is equal to the median value found after curve fitting more than 400 measured velocity profiles to a logarithmic boundary layer equation (Chapter 6).

(v) Several features of the harbour flow pattern were noted including fast flow in the channels, and sub-parallel flow across the sub-tidal ridge extending from Snake Bank.

(vi) In the entrance throat, both the flood and ebb currents commence earlier on the southern side after the low and high tides respectively. The phase lag is about 25 minutes. Similarly, inertia assists continuance of flow in the deep channels near slack water after the currents have turned over the banks.

(vii) Tidal cycle residual velocities and excess distances were related to the sediment transport patterns of the harbour. It was deduced that sedimentation occurs in all neutral zones even under high flow conditions which exceed 1 $m.s^{-1}$ above the bed on the neutral ridge extending south-east of Snake Bank. Scoured shell lags are mostly ebb dominant zones, with the exception of the flood ramp to Snake Bank which is flood dominant with scouring being associated with strong asymmetric flow.

(viii) Whangarei Harbour is typified by several closed or partially-closed recirculating loops summarised in Figure 10.43.

(ix) Sediment transport potentials reveal a continuous ebb-directed pathway running from One Tree Point across the shipping channel, through the proposed dredge zone and out of the harbour on the southern side of the throat. As there is no complementary flood-directed route, shell armouring results due to preferential scouring of sandy sediments. Similarly, an ebb directed route in the Shoal Bay Channel is floored by a shelly lag. Alternatively, the flood route along the north-east of Snake Bank in Shoal Bay is sandy being supported by sediment embarking on the pathway near the entrance throat and on the Snake Bank flood ramp.

The banks are subjected to low currents and are correspondingly stable.

(x) Shear stress at the sea surface under wind during the typical easterly storm from 110°T causes greater up-harbour flow over the banks which is compensated for by a decrease in channel speeds during the flood and an increase during the ebb. Under these conditions, known ebb dominance on the inter-tidal zone at the port site can be neutralised by the additional wind stresses during the tidal cycle. With supporting surface gravity waves, sediment could then move into the inlet along the inter-tidal zone during easterly storms.

(xi) The Coriolis force supports the gross residual circulation in Whangarei Harbour including flood flow on the Snake Bank flood ramp and along the north-east side of the Bank in Shoal Bay, as well as the ebb dominance in the shipping channel and the sub-parallel flow over the neutral ridge

extending off Snake Bank. However, it is not the primary mechanism responsible for the residual circulation.

(xii) Residual circulation in Whangarei Harbour was due to the convective momentum and without the momenta terms included in the hydrodynamic model the residual circulation disappeared. Further tests which maintained the harbour outline but introduced constant depths, showed that the residual circulation is established, to a great degree, by the shape of the harbour alone. Thus, banks and channels may form in particular locations in response to the shape of the harbour outline and this offers some considerable insight into the genesis of a tidal inlet and its expected morphology.

(xiii) Modelling was undertaken to examine the impact of the proposed timber port (Case A7) at Marsden Point. During Spring tides there are speed decreases of up to 19% (flood) and 26% (ebb). Immediately downstream of the dredge zone there are local increases in flow velocity. Slight speed increases are evident in the shipping channel and marginal decreases occur in Shoal Bay with common changes being about 3%. The port causes a reduction to ebb speeds and an increase to flood speeds on the southern side of the throat with the converse occurring on the northern side.

(xiv) The residual velocities show that after port construction, the neutral segment of the sediment ridge which is removed by the proposed dredging becomes subject to an ebb dominance. Accordingly, the natural processes may sweep some or all of the sediment away from this region, thus preventing re-establishment of the ridge and reducing maintenance dredging costs.

(xv) The magnitude of changes expected after port construction are provided in the next Chapter. However, some general assertions are provided by the 2-dimensional hydrodynamic model with respect to the changed sedimentation patterns after port construction. If there is an available supply, sedimentation would be expected to occur at the western end of the dredge zone. Ebb directed transport along the sub-tidal spit from Snake Bank should cause accretion at the north of the dredge zone. On the north-eastern corner, the residuals show that sedimentation may be limited by an ebb-directed residual over what was formerely a neutral ridge (see above). Some sedimentation should occur on the southern side of the entrance throat near the NZRC wharf.

The small velocity differences in Shoal Bay should have a very limited influence with neglible bathymetric changes resulting due to this factor alone. In the shipping channel, the small speed increases may cause limited amounts of erosion, similar to what has occurred since the Shell Cut Reach Channel was dredged. The changes are not expected to be of significance to the overall harbour circulation or morphology. Most important differences occur in the immediate vicinity of the dredge zone.

SEDIMENT TRANSPORT AND TIDAL INLET HYDRAULICS

Chapter 11:

TWO-DIMENSIONAL, NUMERICAL, SEDIMENT TRANSPORT MODEL

CHAPTER 11: TWO-DIMENSIONAL, NUMERICAL SEDIMENT TRANSPORT MODEL

11.1 Introduction

The numerical sediment transport model (SAND2d) to be described in this Chapter is the first of its kind to this writer's knowledge. Previous developments have been hindered by two major constraints. Firstly, a complete 2-dimensional numerical hydrodynamic model is required to generate speeds for application in the sediment transport model, and such hydrodynamic models have not been readily available in estuarine applications. Secondly, the state of the art of sediment transport determination in tidal inlets is such that there has been some considerable uncertainty in the equations to be applied in the transport model. It was the successful verification of 2DD (Chapter 10) and the development of a new procedure to determine bedload transport under tidal flow in conjunction with an empirical suspended load equation (Chapter 7), which made possible the development of this sediment transport model.

The construction and application of SAND2d unified much of the research so far described in this thesis, including of course, the transport equations and the 2-dimensional model. Furthermore, the data collection program to describe the nature of the sea floor, together with the vertical velocity profile and the bed friction analyses, provided essential information for the application stage.

SAND2d estimates sedimentation rates, maintenance dredging requirements and the sedimentary impact of construction in a tidal inlet. It will also predict the nature of the sea bed in a tidal inlet using a numerical iterative technique designed for this purpose. In a coupled mode, the model may be applied to prediction of equilibrium bathymetry. This is the numerical equivalent of a loose bed physical model.

11.2 List of symbols

A	Availability, percentage of sea bed area covered by sand
c	friction factor
d	water depth (m)
D_U	depth below datum on x cell wall (m)
D_V	depth below datum on y cell wall (m)
D_{50}	median grain size by weight (m)
D_{65}	grain size such that 65% of distribution is finer (m)
D_{50f}	equivalent median grain size having the fall velocity of D_{50} (m)
E	Erodibility, percentage of transport rate reduced due to bed cohesiveness
g	gravitational acceleration (9.81 $m.s^{-2}$)
h	half cell width (m)
j	x direction subscript
k	y direction subscript
H_U	total water depth on x cell wall (m)
H_V	total water depth on y cell wall (m)
M_U	mass discharge passing through the x cell wall (kg)
M_V	mass discharge passing through the y cell wall (kg)
P	Porosity (0.45)

q sediment flux per unit bed width per unit time ($kg.m^{-1}.s^{-1}$)
q_f sediment flux scaled by Erodibility and Availability
q_U sediment flux per unit bed per unit time passing through the x cell wall ($kg.m^{-1}.s^{-1}$)
q_V sediment flux per unit bed per unit time passing through the y cell wall ($kg.m^{-1}.s^{-1}$)
S_U speed on the x cell wall ($m.s^{-1}$)
S_V speed on the y cell wall ($m.s^{-1}$)
u flow velocity at z metres above the bed ($m.s^{-1}$)
u_1 flow velocity at 1 metre above the bed ($m.s^{-1}$)
$\bar{u}$ mean vertically-averaged flow velocity ($m.s^{-1}$)
u_* friction velocity ($m.s^{-1}$)
u_*' skin friction velocity ($m.s^{-1}$)
V velocity at cell mid-point output by 2DD ($m.s^{-1}$)
z_o roughness length (m)
ΔH tidal range (m)
ΔM total mass lost from a cell over the tidal cycle (kg)
Δt time increment between samples on the v-t curve from 2DD (s)
ΔV time volume lost from a cell over the tidal cycle (m^3)
Δz change of bed level in a cell over the tidal cycle (m)
ρ_s fluid density ($kg.m^{-3}$)
ρ sediment density ($kg.m^{-3}$)
θ_U flow orientation at x cell wall
θ_V flow orientation at y cell wall
ξ sea level at cell centre from 2DD.

11.3 General operation

SAND2d utilises the tidal cycle water velocities determined by the 2-dimensional hydrodynamic tidal flow model to derive sediment fluxes passing between neighbouring square cells throughout the model region. The program applies a conservation of matter algorithm to ensure volume losses are balanced by gains in one of the two orthogonal neighbouring cells. Summation of total sediment flux throughout the tidal cycle, provides overall bed level changes. These may be scaled for annual sedimentation rates and predictions of maintenance dredging requirements.

Wave influences are not directly included in the model. However, the sediment inputs at open boundaries are established by the modeller and high sediment fluxes at an entrance throat subjected to wave attack may be accounted for.

11.4 Erodibility and Availability

In an estuary, much of the harbour floor is not covered by loose, sandy sediments but by various mixtures of shell, sand, marine biomass, cobbles, rocks etc. This type of material behaves quite differently from loose sandy sediments and is mostly much more stable and less prone to transport under tidal flow. For example, the underwater experiments described in Chapter 7 which were established to examine the stability of the shell lags in Whangarei Harbour, revealed that they are virtually immobile at all times and move only during high Spring tides if at all. The threshold velocity at 1m was estimated as being approximately 1.05 $m.s^{-1}$. No velocities as fast as this were measured

in the harbour, as shown in Figure 5.27 which presents a histogram of the distribution of recorded 1m speeds at all vertical profile measurement sites.

The stability of these regions is further evidenced by the erosion patterns in relation to the shell cover in the harbour. Figure 2.9 shows the contours of the D_{50} grain size from the RNZN bed sediment sample sites. Comparison of this with the erosion pattern in Figure 3.16 reveals a very marked correspondence between stability and percentage shell cover.

Accordingly, the model initially needs to treat the estuary as being composed of two distinctly different bed types with respect to their behaviour under tidal flow. The first of these is essentially immobile and can be treated as such. The bed is composed of the materials listed above, but is primarily shell in Whangarei Harbour. The second type is erodible and is composed of sandy sediments with typical grain sizes in the harbour of about 0.0003m.

Sediment transport equations are applicable to the second of these - a loose, non-cohesive bed of sand. In Whangarei Harbour however, beds of this kind are strictly limited (Figures 2.7 and 2.9). The sandy zones are often interspersed in small pockets between non-erodible shelly regions. Moreover, the major sandy zones in high current flows are predominantly restricted to narrow, distended spits of loose sediment (Figure 2.7).

For a concise representation of Whangarei Harbour, it is therefore evident that this complex sea bed must be described numerically to represent the intermediate types between fully erodible and immobile. Accordingly, after consideration of the relevant physical processes, two new parameters were introduced to provide for a schematisation of the nature of the sea floor. Called the Erodibility (E) and Availability (A) - two integer numbers in the range from 0 to 100 - these parameters respectively represent a percentage of the transport rate that would occur if the sediment was sandy and (i) non-cohesive and (ii) covering 100% of the sea floor.

If the sediment transport flux over megarippled sand is q, then the flux over a cohesive bed (described by E) with interspersed shell (described by A) is,

$$q_f = \frac{q.E.A}{100 \times 100} \qquad 11.1$$

Thus, the Erodibility parameter is set to a percentage representing the reduction in transport rate in each cell due to sediment cohesiveness over clays, or due to impregnation by marine organisms (de Boer, 1981) etc. An alternative to the Erodibility would be to utilise different transport equations applicable to the material in question. This was not necessary at Whangarei where sediment cohesiveness in the high current flows was considered to be second order with respect to the influence of the shell cover.

The Availability is set to the percentage of sea floor area which is

covered by sandy sediments and distinguishes between stable and erodible sediment.

11.5 Numerical scheme

There are many different numerical schemes which could be employed and during the design of SAND2d several were examined. The most important realisation after tests of these was that some were entirely unstable due to preferential, one-way, sediment transport with sand passing predominantly from cells with higher sand content into their less sandy neighbours. The sediment is constantly lost in the direction of minimum resistance established by the sediment Availability rather than by tidal current residuals over the tidal cycle. This only occurs when the bed is not 100% sandy.

After further consideration, it was noted that numerical stability can be ensured by modifying the scheme. An Availability is derived at the cell walls from the average of the Availabilities in the two pertinent adjacent cells. With this modification, sediment may pass freely in both directions through each cell wall, thereby preventing unnatural deterioration of the sandy regions. This is an important result which enabled the extension of the model from an all-sand bed to include non-erodible shells and gravels.

This scheme is stable but is not entirely satisfactory for the same reasons that the depth schematisation was modified in 2DD. As for the depths in the early 2DD scheme, each Availability is involved in 4 averages (one for each cell wall) and often rapid changes in bed facies cannot be accommodated by only one value of A. Thus, the more sophisticated numerical scheme shown in Figure 11.1 was designed. This offers a much improved representation of the bed facies. Moreover, the sediment transport fluxes are calculated individually on each cell wall, instead of using an average at the cell centre, and the accuracy of the calculations is greatly improved.

The cells are physically in the same locations as for 2DD. The water depth below datum is defined at the mid-points of the cell sides. The Erodibility and Availability are both defined in the same locations as the depth. Sea level above datum and the vector flow velocity are known at the centre of the cells from 2DD.

Sediment transport fluxes are determined on the mid-point of each cell side. To make this calculation, the speed, S, at this location is defined as the average of the bounding velocity magnitudes, i.e.,

$$S_{Uj,k} = \tfrac{1}{2}(V_{j,k} + V_{j+1,k})$$

$$S_{Vj,k} = \tfrac{1}{2}(V_{j,k} + V_{j,k+1}) \qquad 11.2$$

The flow orientation ($\theta_{Uj,k}$, $\theta_{Vj,k}$) is similarly defined as the average of bounding angles of flow at adjacent cell centres. Total water depth at the

cell sides is,

$$H_{Uj,k} = D_{Uj,k} + \tfrac{1}{2}\,(\xi_{j,k} + \xi_{j+1,k})$$

$$H_{Vj,k} = D_{Vj,k} + \tfrac{1}{2}\,(\xi_{j,k} + \xi_{j,k+1}) \qquad 11.3$$

11.6 Volume fluxes and the sedimentation rate

The sediment transport equations will provide dry mass of sediment moved per unit time per unit bed width, while the discharges across the cell walls are proportional to the "effective" wall lengths relative to the direction of flow propogation. Given that the sediment transport rate along the length of any wall is constant and equal to the value calculated at the wall mid-point (q_U or q_V), then across the cell wall of length 2h in a time interval Δt we have,

$$M_{Uj,k} = q_{Uj,k} \cos(\theta_{Uj,k}).2h.\Delta t$$

$$M_{Vj,k} = q_{Vj,k} \sin(\theta_{Vj,k}).2h.\Delta t \qquad 11.4$$

where $M_{Uj,k}$ and $M_{Vj,k}$ are x and y components of the total mass flux passing respectively through U and V sides of the cell. The total loss of mass from the cell summed over the tidal cycle of duration T is thus,

$$\Delta M_{j,k} = \sum_T (M_{Uj,k} + M_{Vj,k} - M_{Uj-1,k} - M_{Vj,k-1}) \qquad 11.5$$

For a sediment with density ρ_s and Porosity P over the entire cell this corresponds with a volume loss ΔV in the cell including the pore spaces of

$$\Delta V_{j,k} = \frac{\Delta M_{j,k}}{\rho_s(1-P)} \qquad 11.6$$

and the change of bed level in the cell is

$$\Delta Z_{j,k} = \Delta V_{j,k}/(2h)^2$$

$$= \frac{\Delta M_{j,k}}{\rho_s(1-P)(2h)^2} \qquad 11.7$$

SAND2d utilises flow patterns over the model region output by 2DD at selected time intervals Δt (usually ½ hr) to derive the nett mass fluxes across each cell wall over the complete tidal cycle or during a chosen segment of the cycle. It is initially assumed that the bed is 100% sandy and non-cohesive. Flux through each cell wall is then converted to a bed level change in the cell scaled by the values of A and E.

11.7 Tidal asymmetry

The detrimental influence of half cycle tidal asymmetry may optionally be removed from the velocity-time profile by applying linear scaling for tidal range as described in Chapter 7. All velocities are scaled as,

$$V_n = V_o \Delta H_n / \Delta H_o \qquad 11.8$$

where V_o is the model velocity for tidal range ΔH_o and V_n is the velocity scaled to the range ΔH_n.

11.8 Determination of annual sedimentation rates

Each model run predicts the bed level changes for one tidal cycle and these are converted to annual sedimentation rates using extra software written by this author in FORTRAN and included in the complementary programming package for the model. The method of solution is summarised here and is very similar to the procedure applied to the field measurements in Section 7.18.1 and is performed in four steps:

(i) using the flow pattern from 2DD, sediment transport fluxes were found for Spring and Neap tidal conditions with SAND2d.

(ii) velocities for the Mean tide were obtained twice by linearly scaling both the Spring and Neap flow patterns according to tidal range (equation 11.8)

(iii) sediment fluxes were determined in SAND2d with both sets of flow pattern and then averaged to reduce the detrimental influences of tidal curve distortions arising from factors not being modelled, as well as compensating for scaling errors.

(iv) annual fluxes were found by weighting sediment fluxes according to the frequency of occurrence of each tidal range in the interval $1.1m<\Delta H<2.7m$. The ranges were shown to be approximately Normally distributed in Chapter 7, and sediment fluxes were found by linearly interpolating between known values on a logarithmic scale as described in Section 7.18.1.

These operations are performed in every cell throughout the model region by program 2DSNWT.

11.9 Impervious cell sides

2DD allows the modeller to set the depth on any cell side so that it remains permanently out of the water and totally obstructing flow while circulation continues through the others. This provides for representation of narrow-distended features within the grid. In program SAND2d, no sediment passes across one of these impervious boundaries.

11.10 Sediment transport equations: the "new" and the Engelund and Hansen methods:

Modelling was undertaken with two alternative sets of total load equations.

(i) The bedload fluxes were found using the procedure developed in Chapter 7, to be referred to now as the "new" method. Using the 1m speed, the bedload is obtained with the Yalin bedload equation for plane beds (equation 7.24). The skin friction velocity u_*', is,

$$u_*' = u_1/5.75 \log_{10} (30.2/2D_{65}) \qquad 11.9$$

where u_1 is the speed at 1m above the bed, u_*' is the skin friction velocity, and D_{65} is the grain size for which 65% of the grain size distribution is finer by weight. The logarithm is dimensionless and includes the level z = 1m above the bed.

The suspended load component is obtained with the empirical equation developed at Whangarei (equation 7.35),

$$q_{ss} = (2.03u_1)^{7.75} \times 0.001 \qquad 11.10$$

where q_{ss} is the vertically-integrated dry mass rate of transport per unit bed width. This equation is empirical and was derived over megaripples for grain sizes $0.00024<D_{50}(m)<0.00034$ and a velocity range at 1m of $0.47<u_1(m.s^{-1})<0.78$. Neglible sediment catches were made below 0.47 $m.s^{-1}$ and accordingly, the program assigns the suspended load transport rate to be zero below these speeds. This may include a small error which is neglible relative to the transport rates which occur at higher speeds when flux increases with the speed raised to the power 7.75. The bedload is still calculated below 0.47 $m.s^{-1}$.

In order to obtain u_1 from the mean vertically-averaged velocity output of 2DD, three assumptions were made:

(i) the flow is fully rough.

(ii) the profile is logarithmic.

(iii) the profile is described by the Karman Prandtl equation (6.12) throughout the entire water column.

As the program has no need to utilise low speeds below the sediment threshold, assumption (i) is valid according to the results in Chapter 6.

Sternberg (1978) found that the profile was logarithmic in the lowest 2m for 85% of the time. Detailed in Chapter 6, it was found at Whangarei that, including the outer boundary layer, the profile was logarithmic 56% of the time for fully rough flow at the α = 0.10 level of significance. Under a less stringent test a higher percentage than this may have resulted as obtained by Sternberg. Thus, the second assumption is mostly supported by field evidence.

The third assumption was not validated at Whangarei as a difference in boundary layer velocity gradient was measured in the inner and outer layers. The gradients were in the ratio of 5.75:8.6 (Table 6.1), as predicted by laboratory measurements and incorporated into the coefficients of the logarithmic Karman Prandtl and Universal Velocity Defect boundary layer equations. Accordingly, the inner boundary layer which occupies the lowest 20% of the flow depth may be described by the Karman Prandtl (KP) equation (Chapter 6),

$$u = 2.5\, u_* \ln(z/z_o) \quad \text{for} \quad z<0.2d \qquad 11.11$$

where u is velocity at z metres above the bed, z_o is the roughness length and u_* is the friction velocity. The outer boundary layer which occupies the highest 80% of the flow depth may be described by the Universal Velocity Defect Law (UVDL)

$$u = U_\infty + 3.74 \ln z/d \quad \text{for } z>0.2d \tag{11.12}$$

U_∞ is the velocity at the surface and d is the flow depth.

By assuming that the KP equation is valid throughout the water column some error is introduced into the determination of the 1m velocity. The following analytical solution determines the magnitude of this error based on the assumption that the vertical profile is better described with the two boundary layer equations presented above in accordance with field measurements.

Both equations must be valid at the confluence of the two zones, i.e. $z = 0.2d$, thus,

$$\begin{aligned} 2.5u_* \ln\left(\frac{0.2d}{z_o}\right) &= U_\infty + 3.74 \ln(0.2d/d) \\ &= U_\infty - 6.01\, u_* \end{aligned} \tag{11.13}$$

or

$$U_\infty = 2.5u_* \ln\left(\frac{2.22d}{z_o}\right) \tag{11.14}$$

To find the mean vertically-averaged velocity, the KP is integrated throughout the inner layer and the UVDL throughout the outer layer. The roughness length represents the notional height above the bed where speed is zero and so the inner layer is integrated in the range $z_o < z < 0.2d$. Thus,

$$\bar{u} = \frac{1}{d}\int_{z_o}^{0.2d} 2.5u_* \ln(z/z_o)\, dz + \frac{1}{d}\int_{0.2d}^{d} [U_\infty + 3.74\, u_* \ln(z/d)]\, dz \tag{11.15}$$

The first integral is then,

$$I_1 = \frac{2.5u_*}{d}\int_{z_o}^{0.2d} (\ln z - \ln z_o)\, dz \tag{11.16}$$

$$= \frac{2.5u_*}{d}\left[z \ln z - z - z \ln z_o\right]_{z_o}^{0.2d}$$

$$= \frac{2.5u_*}{d}\left[z \ln\left(\frac{0.368z}{z_o}\right)\right]_{z_o}^{0.2d}$$

$$= \frac{2.5u_*}{d}\left[0.2d \ln\left(\frac{0.368 \times 0.2d}{z_o}\right) - z_o \ln(0.368)\right] \tag{11.17}$$

Now $z_o << d$ and the second term may be ignored. It is then evident that integration over the full depth would provide the expected solution whereby

mean speed occurs at 0.368 of the depth from the bed. With further simplification,

$$I_1 = 0.5u_* \ln \left(\frac{0.74d}{z_o}\right) \qquad 11.18$$

The second integral is,

$$I_2 = \frac{1}{d} \int_{0.2d}^{d} \left[U_\infty + 3.74u_* \ln (z/d)\right] dz \qquad 11.19$$

$$= \left[dU_\infty\right]_{0.2d}^{d} + \frac{3.74u_*}{d} \left[z \ln \left(\frac{0.368z}{d}\right)\right]_{0.2d}^{d}$$

$$= 0.8U_\infty - 1.78u_* \qquad 11.20$$

Substitution of equation (11.14) for U_∞ gives,

$$I_2 = 0.8 \times 2.5\ u_*\ \ln \left(\frac{2.22d}{z_o}\right) - 1.78\ u_*$$

$$= 2u_*\ \ln\left(\frac{0.91d}{z_o}\right) \qquad 11.21$$

Thus, the mean velocity is,

$$\bar{u} = I_1 + I_2 \qquad 11.22$$

$$= 0.5u_*\ \ln\left(\frac{0.74d}{z_o}\right) + 2u_*\ \ln\left(\frac{0.91d}{z_o}\right)$$

$$= 0.5u_*\ \ln\left[\left(\frac{0.87d}{z_o}\right)^5\right]$$

$$= 2.5u_*\ \ln \left(\frac{0.87d}{z_o}\right) \qquad 11.23$$

This may be compared with the solution, assuming the Karman Prandtl equation is valid throughout the water column,

$$\bar{u} = 2.5u_*\ \ln \left(\frac{0.37d}{z_o}\right) \qquad 11.24$$

and it is apparent that the change in boundary layer gradient can be accounted for by using an identical formulation but with a change to the level where the mean speed occurs. This does not imply that mean speed actually occurs at 0.87 of the flow depth from the bottom. Indeed, in 10m of water the mean speed occurs at approximately 0.55 of the flow depth from the bottom for $z_o = 0.001$ and $U_\infty = 1.0\ m.s^{-1}$.

To derive the 1m velocity from the mean vertically-averaged velocity, there are two solutions which depend on the flow depth. If the depth is greater than 5m, the 1m speed occurs in the inner boundary layer and thus,

$$u_1 = 2.5u_*\ \ln(1/z_o) \qquad 11.25$$

Division by equation (11.23) for the mean speed to remove the dependence on

friction velocity gives,

$$u_1 = \bar{u} \frac{\ell n(1/z_o)}{\ell n(\frac{0.87d}{z_o})} \qquad 11.26$$

If the flow depth is less than 5m, then the 1m speed occurs in the outer boundary layer and then,

$$u_1 = U_\infty - 3.74\, u_* \ell n(d) \qquad 11.27$$

Substitution of equation (11.14) for U_∞ gives,

$$u_1 = 2.5u_* \left[\ell n(\frac{2.22d}{z_o}) - 1.49\, \ell n(d)\right]$$

$$= 2.5u_* \ell n(\frac{2.22d^{-0.49}}{z_o}) \qquad 11.28$$

Again dividing by equation (11.23) for the mean speed produces,

$$u_1 = \bar{u} \frac{\ell n(\frac{2.22d^{-0.49}}{z_o})}{\ell n(\frac{0.87d}{z_o})} \qquad 11.29$$

11.29

Equations (11.26) and (11.29) relating u_1 to $\bar{u}$ compare with the solution assuming the KP is valid throughout the depth and denoting the 1m velocity as u_{1old},

$$u_{1old} = \bar{u} \frac{\ell n(1/z_o)}{\ell n(\frac{0.37d}{z_o})} \qquad 11.30$$

Accordingly, the difference between solutions may be obtained from,

$$u_1 = u_{1old}\, \ell n\, (\frac{0.37d}{z_o}) \;/\; \ell n(\frac{0.87d}{z_o}) \quad \text{for } d>5m \qquad 11.31$$

and,

$$u_1 = u_{1old} \cdot \frac{\ell n(\frac{2.22d^{-0.49}}{z_o})}{\ell n(\frac{0.87d}{z_o})} \cdot \frac{\ell n\, (\frac{0.37d}{z_o})}{\ell n(1/z_o)} \quad \text{for } d < 5m \qquad 11.32$$

With these last two equations, Table 11.1 shows the effect of assuming that the Karman Prandtl equation is valid throughout the water column compared with the solution which accounts for a change of boundary layer gradient. For a roughness length of $z_o = 0.001m$ (applied later to the modelling), it is apparent that the 1m velocities are over-estimated by a maximum of 11% if the change of boundary layer gradient is ignored. This occurs for typical estuary depths of 5m to 10m. In shallower depths the error is much less, and is only 3% in 1m of water.

Accordingly, the Karman Prandtl equation alone will produce a conservative over-estimate of the sediment transport rates. The deviation is considered

to be acceptable as final maintenance dredging quantities are quoted with ±50% error bars. It is noted, however, that after most analysis was completed this analytical solution was derived in response to questions raised by Prof. R. Hosking, Mathematics Department, University of Waikato, with respect to this writer's acknowledgement of the measured change of boundary layer gradient. If further field measurements relating u_1 and $\bar{u}$ provide confirmation, future applications of the sediment transport model may utilise equations (11.26 and 11.29) which account for the change of vertical profile gradient. For the modelling reported here, equation (11.30) was applied to derive u_1 from $\bar{u}$.

(ii) The Engelund and Hansen (1967) formula for total load transport (eqn 7.23) was also examined. This equation utilises the mean velocity so there is no need to make any velocity adjustments. It does, however, include the friction velocity incorporating both the form drag and the skin friction components. It is necessary to assume that the friction velocity is some function of the mean water speed. Taking the linear proportionality given by the friction factor, c,

$$u_* = \bar{u}/c \qquad 11.33$$

The Engulend and Hansen equation becomes,

$$q = \frac{0.05\rho_s}{\left[g\left(\frac{\rho_s - \rho}{\rho}\right)\right]^2} \frac{\bar{u}^5}{c^3 D_{50f}} \qquad 11.34$$

where ρ_s is the sediment density; ρ is the fluid density' g is the gravitational acceleration; and D_{50f} is the equivalent median grain size having the fall velocity of D_{50}.

This equation has no depth or roughness dependence. As such, in deep channels where speeds are fastest, sediment transport rates rise sharply due to the deepening of the channel alone. Alternatively then, using the Karman Prandtl equation to introduce the expected depth and roughness length dependence,

$$u_* = \bar{u}/5.75 \log_{10} (0.38d/z_o) \qquad 11.35$$

so that,

$$q = \frac{0.05\rho_s}{\left[g\left(\frac{\rho_s - \rho}{\rho}\right)\right]^2} \frac{\bar{u}^5}{D_{50f}\left[5.75 \log_{10}(0.38/z_o)\right]^3} \qquad 11.36$$

11.11 Grain sizes and the critical friction velocity

Grain sizes were determined with samples from bedload traps placed in megaripples on the sub-tidal spit extending south-east from Snake Bank (Table 7.2). In accordance with this, grain sizes of $D_{50} = 0.0003$m and

$D_{65} = 0.00035$m were used throughout the model region. With a sediment density of 2650 $kg.m^{-3}$, the D_{50} size corresponds with a critical skin friction velocity from the Yalin Curve (Miller et. al. 1977) of $u_*' = 0.014$ $m.s^{-1}$. This grain size is greater than the mean size $M_z = 0.0025$m obtained by Millar (1980) after dissolution of the carbonate content. Use of a smaller grain size and critical friction velocity would produce dredging quantities approximately 20% larger.

Throughout the thesis, Porosity of $P = 0.45$ has been utilised to reflect the loose packing of sediment on the active megaripples. Although universally applied, this value may be too high for the modelling of some locations and P may fall to about 0.40 so that 0.45 corresponds to a possible 12% over-estimate of the sediment volumes. Fluid density was taken to be 1025 $kg.m^{-3}$.

11.12 Roughness length

Both sets of transport equations include the bed roughness length z_o and thus a specification of this parameter must be made but unfortunately, it is a function of both the flow strength and the bed type. Over loose sediment the roughness length increases with higher flow speeds as megaripples become steeper. Figure 6.21 shows the Drag Coefficient increasing with the flow velocity for the S-series sites. However, it was also noted (Chapters 6 and 7) that, contrary to this, the bed is flattened by high sediment loads. Moreover, for faster speeds or in low sandy sediment supply locations, the bed in Whangarei Harbour is typically shell lagged. In these cases, reduced form drag losses result in low z_o even in high flow. Measurements in Whangarei Harbour revealed that z_o varies over the order of magnitude range (0.0005m, 0.005m). Median z_o was 0.0007m and was associated with the shell beds. Over megaripples, z_o averaged approximately 0.002-0.003m. Clearly some difficulty will be encountered when attempting to specify z_o is every cell so its influence is examined here. In the transport equations, z_o is needed to convert $\bar{u}$ to u_1 and to provide u_* from $\bar{u}$. Values of the conversion factors (equations 11.30 and 11.35) have been compiled in Table 11.2 for a range of z_o and some typical depths.

For the conversion to u_1, there is a strong dependence on flow depth such that the factor varies from 0.77 to 1.04 for depths of 20m and 2m respectively. It is evident however, that the conversion is not very sensitive to the choice of z_o, varying only between 0.8 and 0.85 at constant depth. Thus, if a middle value of z_o is chosen there is only a 3-4% error.

Nevertheless, some tests were still conducted in the model with a variable roughness length determined by linearly interpolating on a log scale between 0.0005 and 0.005m in proportion to the percentage sand cover. That is,

$$z_o = 10^{\{(\log_{10}0.005 - \log_{10}0.0005) \times A/100. + \log_{10}0.0005\}} \qquad 11.17$$

Greater scour occurred in the shelly regions with this formulation because, for a given mean speed and flow depth, u_1 is larger for low z_o and the scouring potential is correspondingly increased.

A formulation of this kind makes an effective attempt to model the change of roughness length which occurs as a sandy bed becomes scoured. It does not however, correctly model low speed flow over sandy beds which are typically roughened by small ripples. This formulation ascribes a roughness length of 0.005m in these cases which is too large. Thus, an overall mean z_o may be a better approximation than the linearly interpolated value, especially in view of the analysis above which shows that z_o is not a critical parameter for the determination of u_1.

To find u_* for the Engelund and Hansen equation, Table 11.2 shows that both z_o and the depth are important. They form a ratio and both vary typically over about 1 order of magnitude. The variation in the friction factor over the range of z_o values and for constant depth is from 16.6 to 23.3 i.e. more than 30% difference. As the friction factor is cubed in the transport equation, the results will therefore be sensitive to the choice of z_o. Moreover, contrary to the previous method, the sediment transport rate rises with increasing z_o (decreasing c). The rise reflects a greater friction velocity, even though much of this increased energy loss is simply due to form drag and not a rise in effective shear stress. Table 6.7 lists friction factors at the S-series sites and median values are from 17-21 although c varies during the tidal cycle (Figure 6.27). The range coincides with those in Table 11.2 for $z_o = 0.001 - 0.003$m in 10m of water.

11.13 Comparison of the "new" and the Engelund and Hansen methods

It was seen in Chapter 7, after comparisons with field measurements and the "new" method, that the Engelund and Hansen equation "adequately" predicted the total load at 2 out of 3 megarippled sites in Whangarei Harbour. At one shallow site, the equation under-predicted the transport rates. Their formulation was examined in SAND2d by modelling Whangarei Harbour floored by 100% sand at all nodes with a constant roughness length, $z_o = 0.003$m chosen to correspond with the sandy sea floor.

For the two methods, erosion and deposition rates were considerably different with the Engelund and Hansen equation predicting 2-5 times more sediment movement (implying the same increase in maintenance dredging costs). Further examination revealed that the Engelund and Hansen formula nearly always predicted higher transport rates except in shallow depth ($\simeq$2m) at fast flow speeds ($\simeq$1 $m.s^{-1}$). However, during the field experiments, when the friction velocity was determined from the measured vertical profile, no such deviation was noted.

The use of the Karman Prandtl formula to ascertain the friction velocity

may be responsible for the differences. To test this, the transport rates predicted by the "new" method using the mean speed and Karman Prandtl equation to find u_1 were compared with the measured sediment transport data discussed in Chapter 7, and an acceptable correlation was obtained. Thus, it is apparent that the sensitivity of the Engelund and Hansen equation to roughness length makes the application of their method very difficult. The formulation provides reasonable agreement with field measurements when the mean speed and friction velocity are measured but it is not considered to be suitable for 2-dimensional model applications. Therefore the "new" method was adopted for all modelling results described in this Chapter.

11.14 Iterative mode

As the bathymetry in Whangarei Harbour is known to be stable, the sediment transport dynamic equilibrium must be primarily related to sandy sediment availability and to bed erodibility. Furthermore, as the lag-shell beds were found to be immobile at all times except possibly during peak Spring tides, the transport rate is then mostly governed by the availability of sandy sediment in any cell or the percentage sand cover. Accordingly, as the depth remains constant, there is no need to model the Harbour stability by examining the usual relationship between flow velocity and water depth. Instead, the stability is modelled by adjusting the percentage of sand cover throughout the harbour.

To successfully describe this, the sediment transport model needs to be more sophisticated than simply a predictor of sediment movement over a sandy harbour floor. Already, two parameters, the Availability and Erodibility, have been introduced to provide for schematisation of the nature of the sea bed. To further expand on this, an iterative technique was designed where the model itself seeks the stable harbour configuration by adjusting the percentage sand cover separately in each cell until a dynamic sediment transport equilibrium is attained.

An initial condition is established where the harbour is set to a 100% sand cover except for the open boundaries. The boundaries are separately set by the modeller to the actual percentage sand cover in each cell and these determine the final solution and remain unchanged throughout the iteration procedure. The model derives the bed level changes in all cells for one tidal cycle. Each cell is then examined individually and, to accelerate the convergence, only Availabilities on cell walls with outwardly-directed sediment flux are adjusted: The Availability A is raised by 1% in an accreting cell, or reduced by 1% if the cell has scoured. This either assists or retards the flow of sediment from the cell respectively. The iteration continues for several hundred tidal cycles until the sediment transport equilibrium state is attained. At this stage, for the given bathymetry

and flow pattern, the harbour has stabilised due to the interaction of flow velocity and sediment availability. Notably, this stable configuration may include the effects of sediment input through the entrance throat due to littoral drift or direct wave attack on the ebb-tidal delta if good sediment inflow data at the throat is available. The modeller can include this input when the boundary conditions are established.

In regions of low transport rates, the iterative process is sensitive to small bathymetry schematisation errors. Sometimes also, the sediment threshold is only exceeded during one half-hour time interval and small errors in the velocity magnitude can cause larger deviations in the sediment fluxes. Accordingly, some instability on the sand banks was found to occur. To prevent this, the modeller may select a critical minimum flux through any side wall. The Availability is only adjusted on walls which exceed this.

11.14.1 Uniqueness

The iterative solution must be unique for given boundary conditions and cannot depend on the initial nature of the harbour bed. The verify this, tests with different initial conditions (50% to 100% cover throughout) but the same boundaries produced almost identical Availability (±2%). These would have been closer if alterations smaller than 1% were allowed. The model gets to a stage where a 1% change is too great and it oscillates between two configurations. Larger artificial differences may occur on the sand banks or in sheltered regions where the velocities are often below threshold, as the final solution remains unchanged from the initial condition in these regions.

As the solution is "unique" for the boundary conditions and given flow pattern, the stable configuration depends only on the input of sediment through the chosen boundaries. The flow pattern is governed by the bathymetry.

11.15 Coupled mode

Numerical experiments were conducted by coupling the sediment transport and hydrodynamic numerical models. When unified, the models simulate the growth of a tidal inlet, starting from a simple schematised configuration evolving to a developed estuary with formed banks and channels in a state of dynamic sediment transport equilibrium.

Initially, 2DD provides tidal cycle flow patterns for the schematised shape. SAND2d is then applied to derive nett bed level changes in each cell for the given flow conditions. Depths are adjusted accordingly. Then using 2DD again, the new flow pattern is ascertained for application in SAND2d and further depth adjustment. This process continues indefinitely until a sediment transport dynamic equilibrium is attained. The convergence is

accelerated by applying a user-selected scale factor to the predicted bed level changes after each iteration which is mostly chosen to be as large as possible without causing instabilities in the hydrodynamic model to develop over unnatural, rapidly-changing bathymetry.

11.16 Complementary programming

Several complementary programs were required for quick and easy data manipulation and presentation. These are briefly described in this section. (Program 2DSNWT which derives annual sedimentation rates has been previously discussed in Section 11.8).

(i) SNDGRD

This program manipulates the input file of depths and harbour geometry used by 2DD. It provides an output file suitable for SAND2d including bathymetry as well as user-selected values of Availability and Erodibility.

(ii) 2DDSAN

When the hydrodynamic model and sediment transport model are coupled to construct a stable bathymetry, this program adjusts the depths according to the amount of scour or accretion which has occurred during the previous model run. A user-selected linear scale factor is applied to the depth adjustments.

(iii) 2DPLT

This program was described in Chapter 10 for its graphics plotting capabilities utilised by 2DD. In addition, the program will provide a plan map of sedimentation rates, a map showing accreting and eroding zones signified by a plus or minus sign, or mapped values of the percentage of sand cover.

11.17 Test rigs

In the design of test rigs, one of the most important bathymetric configurations in Whangarei Harbour was considered, - i.e. a narrow, distended ridge of loose sediment flanked by a scoured shelly lag. These have been shown to exist in high current flows with low sediment supply, because of overall tidal current neutrality.

Consider the sandy zone with 100% cover oriented at 45^{o} to the grid and surrounded by scoured regions with 0% cover, as depicted in Figure 11.2. The depth is constant and the currents are of equal strength in all cells. Reversing tidal flow is modelled by introducing a constant rectilinear current acting for equal times in both directions. Accordingly, the flow is neutral everywhere; and the sandy zone should always remain stable irrespective of its orientation relative to the grid or the angle of current attack.

11.17.1 Unstable scheme

Initial tests confirmed that schemes which do not allow free passage

of sediment in both directions through the cell wall are unstable. With Availability independently specified for each cell, no sand leaves those cells with A = 0 but sediment passes freely from the cells with A = 100. As shown in Figures 11.3 (a to c), after 20 iterations, the sandy zone is reduced to an 80% sand cover. Eventually, with unchanging boundary conditions, the sand is almost entirely dissipated, leaving only a trace (less than 4%) of the original 100% cover.

11.17.2 Stable scheme

With the Availability defined on the cell walls as the average of adjacent cells, under reversing tidal flow the sandy zone was found to be entirely stable for all current orientations.

11.17.3 Round-off error

After each iteration, the Availability A is adjusted according to whether the mass change in the cell is greater than or less than zero. In the test condition, round-off error leaves a residual after one tidal cycle of about 10^{-5} mass units when approximately 50 mass units of total sediment transport occurs. This very small error causes the sandy zone to deteriorate as shown in Figures 11.4 a and b; after 400 iterations, a most unlikely but stable configuration has evolved. Accordingly, the scheme needs to adjust A if changes are greater than a small number larger than the round-off error.

11.17.4 Uni-directional flow

As expected, the 45^{o} sandy zone is gradually swept away by perpendicular, uni-directional currents ($\theta = 135^{o}$) as presented in Figure 11.5 a to c. Due to the boundary conditions imposed on this test rig, after 200 iterations a stable configuration evolves, with sediment washed from the sandy zone being trapped in the upper left corner of the grid.

Longitudinal flow ($\theta = 45^{o}$) along the sand zone, established a sediment flux along its length with no deterioration because sand is introduced through the boundary node to replenish the supply. Again, this is the expected behaviour.

The final test rig shows the influence of a sandy zone on the right-hand boundary under a uni-directional current flowing into the rig from the right ($\theta = 180^{o}$). After 40 tidal cycles (Figure 11.6a), the model region is being floored with sandy sediment entering through the boundary. After 120 iterations, (Figure 11.6(b)) the entire test rig is sandy except for the left-hand boundary which is fixed at zero. The scheme allows sediment to pass out of the boundary even though A = 0%.

11.18 The genesis of a tidal inlet

The coupled hydrodynamic and sediment transport 2-dimensional models

were employed to simulate the genesis of a schematised tidal inlet. The initial configuration is shown in Figure 11.7. The inlet has two entrances, one of which is offset and protected by a land mass on the southern side projecting out to sea. The barriers are assumed to be immobile and impermeable and both entrance throats are 1000m wide at the narrowest point. The configuration is similar in plan to Tauranga Harbour on New Zealand's east coast, with an offset entrance at Mt Maunganui (Figure 11.8).

The sea floor is 100% sand and non-cohesive with $D_{50} = 0.0003$m and was given a rectilinear gradient with average depths at the centre of the cells increasing from 9.3m below datam at the seaward boundary to inter-tidal zones of -1.5m at the up-harbour end (Figure 11.7). The tides at the open boundary were assumed sinusoidal with a range of 3m and mean tide at 1.5m above datum. Thus, the tidal flats in the inner harbour should be completely flooded at about mid-tide. The tidal prism is approximately 180×10^6 m^3, or 90×10^6 m^3 through each entrance and the surface area in the inner harbour at high tide is 68×10^6 m^2.

11.18.1 Initial flow pattern

For the initial configuration, the flow pattern at peak flood and ebb (output from 2DD) is shown in Figures 11.9 and 11.10 respectively. Flow is seen to converge upstream of the entrance but due to inertial influences, the dominant feature is the jet beyond both entrances during the ebb and flood; its influence is still marked some 3-5 cells from the entrance, that is, 1500-2500m. Flow jets almost perpendicular to the bed contours at the symmetrical entrance but the offset configuration rotates the jetting stream to become more north-south oriented.

Residual velocities are shown in Figure 11.11 and the pattern is complex with several recirculating cells formed due to the geometry of the inlet. The jetting of flow through the entrances causes ebb dominance to occur outside the throat and flood dominance inside. Adjacent to this, in the shelter of the land masses, the residual flow is opposed to the dominant jet direction. For example, on the northern side outside the offset entrance, flow near the land is flood dominant.

Together with the residuals, Figure 11.11 shows the circulation pattern in Tauranga Harbour after Davies-Colley and Healy (1978). Subsequent investigations by Dahm (1983) confirmed much of this, although Dahm's conclusions (Figures 11.12 and 11.13) differ particularly in the vicinity of the Western Channel inside the entrance throat (Figure 11.12). Nevertheless, the modelling shows overall similarities with the prior field investigations noting that the configuration is schematised and the bathymetry is planar in the model. Accordingly, and as was found in Chapter 10, there is evidence to suggest that the harbour outline plays a dominant role in determining the

overall flow patterns. This result is not surprising and is fundamental to the success of physical, loose-bed modelling to derive expected bed changes and stable bathymetry after man-made construction of a port or breakwater, for example. In these cases, the physical model is allowed to stabilise with immobile structures included. What is being examined here and next is the numerical equivalent of this procedure.

11.18.2 Evolution in the coupled mode

The tidal cycle nett erosion/deposition pattern derived with the sediment transport model for the initial configuration and corresponding flow pattern is shown in Figure 11.14. Flood and ebb-tidal deltas begin to form immediately, being initiated by sediment scoured out of the entrance throat.

After four coupled iterations, Figure 11.15 shows the bathymetry which has developed. Most evident at this stage is the deepening of the entrance throat by about 1-1.5m. There is some shallowing immediately inside and outside the entrance. The residual flow pattern (Figure 11.15) is similar to that with the earlier bathymetry but nett vectors are mostly reduced in magnitude. The erosion/deposition pattern is also similar (Figure 11.17) with scour continuing in the entrance throat and accretion on both sides. The marginal channels are seen to be scouring and are more well-defined.

After 8 coupled iterations, there have been some dramatic changes to the bathymetry (Figure 11.18). The symmetrical entrance throat is 11.0m deep (from 5.3m initially). The offset throat has also deepened substantially. Ebb and flood tidal deltas are well defined and appear like "circular islands". The depths shown in the bathymetry Figures are the average of the depths on the four cell walls. Although not evident in Figure 11.18, some individual cell walls have accreted to above the low tide level presenting an exposed inter-tidal delta at low tide inside both entrances. Depths have shallowed by more than 3m in places.

Marginal channels inside and outside the throat are clearly defined. For instance, at the offset entrance, the depth inside the throat beside the southern land boundary is 5.1m which decreases to 2.7m in the adjacent up-harbour cell.

On the inter-tidal zone at the up-harbour boundary, new sedimentation is flanked by adjacent scour which could signify the early stages of a channel carrying flow around the outside edge of the harbour. Similar channels with offshore deposition are common in tidal inlets e.g. McLeod Bay Channel, Calliope Bay Channel.

Peak flood and ebb flow for this bathymetry are shown in Figures 11.19 and 11.20. The strength of the ebb and flood jet is markedly diminished by some 40-50%. Moreover, the deltas adjacent to the throat particularly at the symmetrical entrance cause flow to diverge away from the very shallow

regions. In response to this, a coupled iteraction between the bathymetry and flow was identified during the modelling whereby, the deposition rate was successively diminished after each iteration due to flow divergence away from the banks and generally slower current speeds.

The residual vectors are considerably reduced in magnitude after 8 coupled iterations (Figure 11.21). At the symmetrical entrance, the flood delta diverts flood flow to the north (Figure 11.19) into deeper water after passing through the entrance (Figure 11.18). At the shallowest cell (1500m landward from the throat), the residual currents are oppositely-directed on its down and up-harbour sides. Evident in many estuaries including Whangarei Harbour, this circulation is responsible for the formation of the ebb shield protecting ebb dominant up-harbour portions of the flood-tidal delta from flood-directed currents. However, the flow pattern over the shallowest regions is quite different at the offset entrance. The shallowest part of the delta is flood dominated, and nett circulation is evidently related to strong residual flow coming from the northern marginal channel.

The general pattern of change observed with these simulations may be summarised as:

(i) convergent flow in the entrance is fast and begins to scour the throat. Initially sediment is deposited in adjacent cells.

(ii) as the scour in the throat lessens, adjacent cells begin to erode and the ebb and flood tidal deltas move away from the entrance.

(iii) as the throat deepens, currents are reduced and the initially dominant ebb and flood jets become less important, causing a resulting change in residual flow patterns with an overall decrease in residual vectors.

(iv) flow is diverted by the deltas and enters the deep throat through flanking channels which become more scoured as the banks build up.

(v) this further reduces the influence of the ebb and flood jets and the channels become more important. The process is self-propogating with the accretion on the banks constricting flow and allowing the channels to deepen and spread throughout the estuary. The estuary builds outwards from the region of highest current flow.

(vi) the ebb shield found on many flood-tidal deltas is also a stable feature. As it builds to form an impermeable barrier, the ebb and flood dominance on both sides becomes more pronounced allowing further accretion to occur if sediment is available and currents exceed the sediment threshold.

With the selected grain size and critical friction velocity, the bathymetry was relatively stable after 8 iterations. Nett level changes in the cells were a maximum of 2-3 units compared to initial changes of over 500 units. With a reduced grain size and corresponding threshold, further erosion/deposition could be induced (Figure 11.22), but a limited amount of new information would be gained by continuing with this schematised

configuration.

However, there is considerable potential for this procedure. One of the most important applications could be the simulation of stable bed levels expected to evolve after man-made changes to bathymetry, particularly after port construction. The flow and bathymetry are coupled such that the flow patterns adjust to the bed level changes and vice versa. As identified in the simulated case, the erosion/deposition rates asymptotically approached zero as the new equilibrium evolves.

Secondly, the implications of the coupling of the two models with respect to the genesis and morphology of a tidal inlet have been demonstrated. Research is planned for other configurations with the inclusion on a more powerful computer of (i) a variable grain size to simulate the typical reduction in sediment coarseness up-harbour and (ii) shell influences.

11.19 Whangarei Harbour simulations

The sediment transport model was written to determine the impact on the sediment budget of the proposed timber port at Marsden Point. With the options on percentage cover of sandy sediment and the iterative technique, this could be undertaken using three broad approaches. These are firstly discussed in detail for the present bathymetry. The impact of the port is examined subsequently.

11.19.1 Modelled days

In accordance with the 2-dimensional modelling in Chapter 10, flow patterns in Whangarei Harbour for the Spring tide (14-Nov-81) and Neap tide (3-Dec-81) were utilised in this Chapter. From Table 10.5, these are Runs 219, Spring, present bathymetry; 220, Spring, including the port; 221, Neap, present bathymetry; 222, Neap, including the port.

11.19.2 Roughness length

In all the following simulations, the roughness length was taken as $z_o = 0.001$m irrespective of percentage sand cover. Accordingly, over sandy regions in high current flows, the low value means that sediment fluxes may be slightly over-estimated.

11.19.3 Bathymetry

A clear overlay detailing harbour bathymetry is provided in Figure 10.20.

11.19.4 Determination of Availability

Numerical representation of the Availability and Erodibility requires collation of information from three major sources. The primary technique used here is underwater photography which allows visual estimation of the area

of bed covered by sand and shell. Divers are instructed to photograph typical bed conditions, and use multiple photographs where facies change.

The RNZN undertook a detailed bottom sampling program in the model region and this provided secondary information. The grain size samples, however, suffer from 3 major deficiencies. Firstly, one spot sample may not represent the overall facies within the cell. Secondly, dredged samples include sub-surface sediments which are not of interest. Thirdly, the weight distribution after a textural analysis is not necessarily related to the percentage of surface sand cover.

To utilise the samples, a boundary grain size must be defined to separate sand from shell. To assess the most applicable size for Whangarei Harbour, the weight distribution of the bed samples were examined in conjunction with the underwater photographs at the same locations. It was found that at the very sandy sites, virtually all of the weight was at sizes finer than 1 phi. At shelly sites, most weight was at coarser sizes. Furthermore, when the estimated percentage sand cover in the bed photographs was compared with the percentage of weight exceeding 1 phi at some of the intermediate sites, the correlation was acceptable. Notably, in a typical classification (e.g. Raudkivi, 1976, Table 2.1), the 1 phi boundary separates coarse and medium sand, but the table has no relevance to the application here. The contours of the percentage of weight exceeding 1 phi are shown in Figure 2.10 for all sediment sample sites and derived from values compiled in Table 11.3

The third technique employed was side-scan sonar. As noted, there are often distinct changes in the sea bed over small distances and the side-scan sonograph depicts these clearly in some cases. Further to this, the instrument will isolate bedformed regions where most sediment transport occurs.

With the above information and with supplementary data recorded during diving expeditions, the Availability in Whangarei Harbour was defined on every cell wall. For this application, no attempt was made to quantify the Erodibility or cohesiveness of the sediment. Instead, the nature of the sea bed was described with the Availability alone. In the high currents flows, cohesiveness due to clays was not felt to be of great significance. However, some locations noted later were suspected of being cohesive when sandy regions with fast currents were devoid of bedforms, and the bed was assigned an artificially low Availability.

11.19.4.1 *Boundary conditions for the iterative mode*

In the iterative mode, only the open boundary conditions (E and A) must be specified. However, for a given flow pattern and bathymetry, the final solution is entirely governed by the boundary conditions and so an accurate specification of the bed facies was sought with a supplementary, detailed underwater photography survey. The photograph sites, mapped in Figure 11.23,

spanned the One Tree Point to Reserve Point (up-harbour) and the Marsden Point to Lort Point (down-harbour) boundaries. Photographs were taken at approximately 150m intervals which allowed some overlap on a 250m grid model. The locations of the sites along the cross-channel profiles are shown in Figures 11.24 and 11.25 with a brief summary of the bed facies.

At the down-harbour boundary, the cross-section is deep and scoured towards Marsden Point (Figure 11.24). The bed is mainly shelly except near the NZRC wharf where total sand cover was noted. At the up-harbour boundary (Figure 11.25), a scoured channel off One Tree Point is separated from the shipping channel by an ebb-tidal spit. The topography rises nearly to Chart Datum on Snake Bank then drops away again into a deep hole off Reserve Point. Megarippled sand was found on the northern flank of the shipping channel. A fine, flat bed of sand with a clay-like consistency to touch was noted on Snake Bank. Ripples were found on the northern Shoal Bay flank of Snake Bank and aged, algae-coated shell lines the deep hole off Reserve Point.

The final specification of the boundary Availability is shown in Figure 11.26. As previously noted, a sandy bed which was not rippled was suspected of being cohesive. This occurred on the crest of Snake Bank where virtually total sand cover was assigned an Availability of 80% and in the channel off One Tree Point, which was ultimately assigned an artificially low Availability of 40%.

11.19.4.2 *Wave influences at the entrance throat*

As the boundary conditions are set by the modeller in the Iterative Mode, wave-induced sediment input at the throat may be accounted for by increasing the percentage of sand cover at the open boundary if good field data is available. However, Whangarei Harbour is strongly tidally dominated (Chapters 3, 4, 5 and 7) and thus wave influences were not included. Good comparisons using the Iterative mode later confirm the validity of this assumption.

11.19.4.3 *Optimisation*

As it seeks stable bathymetry, the iterative technique may be employed as an optimising routine. By iterating a selected number of times, the digitised Availabilities determined from the field data may be subsequently optimised to provide a more stable sediment transport equilibrium in the harbour. Notably, the boundary Availabilities always remain unchanged. The field data was comprehensive but there was not always information about the bed in every cell and some error may occur and could easily be as much as 10%. Accordingly, optimisation was undertaken by iterating 10 times thus allowing a maximum change of 10% on any cell wall.

This was undertaken using the Spring tide simulation and the Neap tide simulation scaled to a Spring range. The resulting Availabilities were then averaged and final values are presented in Figure 11.26. For presentation purposes, the values shown are at the cell mid-points and correspond to an average of the separate Availabilities on the 4 cell walls utilised by the sediment transport model. Accordingly, the individual Availabilities are presented in Table 11.4 and these may be compared with the sea bed data from which they were derived in Chapter 2 and with the P(1ϕ) percentages in Table 11.3.

11.19.5 Total sand cover

Preliminary numerical investigations were made assuming that the harbour floor is covered entirely by a non-cohesive bed of sandy sediments. These results may not be simply interpreted as providing erosion/deposition rates in the inlet because the sea bed facies are not correctly described. As the flow pattern varies throughout the estuary, the sedimentation rate in any location depends on the horizontal rate of change of sediment transport, which in turn strongly depends on the relative locations of shelly and sandy regions. Sedimentation will always occur in decelerating flow, such as at the edge of the dredge zone, if there is an unlimited supply of material. However, if patchy shell lag reduces the input, deposition does not have to result as the decelerating currents may still have sufficient potential to remove the influx of sediment even through this potential is less than in the adjacent upstream area. Thus, it is not obvious how the magnitude of sedimentation rates predicted with a sandy floor will relate to the prototype with shelly beds included. Nevertheless, the results will reflect an upper limit approximating maximum rates of bed level change.

One mechanism, noted during the field data collection program, is not being modelled and could influence the results. With the correct sand/shell ratios, the shell may sit loosely on the sea bed overlaying a sandy floor. In strong tidal flow, sand grains may then fall between the shells which also prevent a subsequent initiation of sand erosion and sedimentation progresses accordingly. This mechanism is not well understood and is therefore not modelled.

The model simulations with 100% sand cover serve as generalised calibrations. As the shell lag results from preferential transport of sand, the channels should scour if the shelly sediment was removed. Moreover, the known sandy areas, such as the sand banks and neutral zones, should accrete under conditions of grossly increased sediment supply.

11.19.5.1 *Results*

Figure 11.27 shows the annual volume changes in each cell in 1000's of

cubic metres and scaled for annual rates using the methods previously described. The volumes include the pore spaces. Figure 11.28 shows the corresponding annual bed level changes in centimetres (m x 100) in each cell.

The model provides evidence that the shell is responsible for stability. Considerable changes to bed level occur in this all-sand case with scour in the channels and accretion in the neutral zones. The regions of lag shell known to exist along the shipping channel and at the port site are all predicted to erode with most bed level changes averaging about 0.10m per year, and corresponding with volume changes over the 250m-square cells of about 5,000 m^3/year.

Along this route, nett tidal cycle fluxes of sediment are very large particularly during Spring tides and far exceed the sedimentation rates. As an example, the perpendicular flux through three cells totalling a 750m width in the port site was examined. The cross-section is shown in Figure 11.27, and the tidal cycle fluxes have nett magnitudes in ebb direction of,

425,000 m^3/year	for Spring tides of 2.6m range
27,500 m^3/year	for Mean tides of 1.9m range
5,200 m^3/year	for Neap tides of 1.6m range

Being a nett discharge, the sandy sediment must be supplied from up-harbour. However, as noted in Chapter 10, there is no equivalent flood-directed pathway capable of transporting large quantities of sediment up-harbour from the entrance throat. Accordingly, the harbour cannot maintain this discharge and a scoured shell lag has resulted along the shipping channel.

Near One Tree Point there is considerable scour on the southern flank of the channel with sediment being deposited on the sandy inter-tidal flats. This region was subject to some error in the prediction of water speeds due to bathymetry schematisation on the 250m grid (Chapter 10) so there is some uncertainty in the sediment transport prediction.

Large-scale sedimentation on the sandy neutral ridge extending south-east from Snake Bank is clearly defined (Figures 11.27 and 11.28). Moreover, in the weaker current flows of Shoal Bay, limited scour occurs along the length of the channel in compliance with underwater video observations of a lag surface. Accretion is predicted to occur on the distended sandy zone discovered on the north-east side of Snake Bank encompassing the 5m contour and heading towards Darch Point and Reotahi Bay. Bed level changes are about 0.01m/year. Off Reotahi Bay, the expected sedimentation occurs but is displaced westward by ½-1 cell width from its actual position. During bathymetry schematisation for 2DD, due to the finite cell sizes, Darch Point had to project by nearly ½ a cell width too far to the west in the model. This forces more discharge across the shallow sediment ridge separating the

flood ramp from the ebb dominant currents which sweep out of Shoal Bay and past Darch Point. The accreting zone has therefore moved westward a corresponding distance.

The two flood-tidal deltas exhibit mostly zero changes of bed level in the presence of the extra sediment supply because of low speeds there. The banks have apparently reached an equilibrium which would only change slowly under the present hydraulic regime irrespective of the availability of sediment. The only major change to Snake Bank occurs on the entrance side of the ebb shield where a bed level change of 0.81 m/year is predicted. Extra sediment would be expected to reinforce the ebb shield increasing the size of the inter-tidal zone on the flood side. This typically occurs in estuaries with a larger wave-induced sediment input. For example, Rangaunu Harbour on Northland's east coast, has a very well developed ebb shield with a nearby entrance open to ocean wave attack.

In the model, the entrance throat to Whangarei Harbour is predicted to scour in mid-channel with the sandy bed. This confirms the suggestion in Chapter 3 that the shell alone prevents further widening of the throat. On the southern side of the throat near the NZRC wharf, bed level changes are small with only 0.01m/year of erosion predicted.

The simulation has offered a good generalised confirmation of numerical model behaviour. The predictions of accretion in known sandy areas and scouring in the channels validate both hydrodynamic models as well as the sediment transport model, which are all involved in the determinations. However, the harbour is known to be stable, so the predicted sedimentation/ erosion rates are necessarily incorrect. It is therefore essential that an improved representation of the sea floor facies must be accommodated.

11.19.6 Model calibration

Figures 11.29 and 11.30 show the annual volume and bed level changes with the bed Availabilities properly represented (Figure 11.26). There are some localised areas which are not entirely stable and poor schematisation of the bathymetry from 2DD is responsible for the most outstanding of these off One Tree Point. However, the harbour is now mostly stable and the calibration of the model is therefore completed. The overall results are very encouraging.

11.19.7 A stable harbour found using the Iterative mode

Using the boundary conditions obtained from the underwater photography survey, the stable harbour was obtained by iteration. Figure 11.31 shows the Availabilities derived by iteration for the Spring tide simulation with a critical nett flux cut-off through any cell wall of 10 kg/m/tidal cycle. The cut-off prevents further adjustment to the Availability on any wall if

the flux multiplied by the Availability (i.e. q.A/100) is less than the cut-off and is necessary because of round-off errors and small modelling and bathymetry schematisation errors. Figure 11.32 shows the Availabilities for the Spring tide simulation with a much lower, mostly-ineffectual cut-off of 0.4 kg/m/tidal cycle. In Figure 11.33, the Availabilities are shown for the Neap tide simulation scaled to a Spring range and using the lower cut-off of 0.4 kg/m/tidal cycle.

The results vary in the details but there is a strong resemblance in all cases to the measured Availabilities (Figure 11.26). Using the bathymetry overlay (Figure 10.20), the channels are seen to be typified by a low sand content as expected. The port site is shell lagged with covers of about 20%. The banks are much sandier. In the Neap simulation, the ebb-tidal spits from Snake Bank, both along the southern side of Shoal Bay and extending south-east across the dredge zone, are evidenced by cells with a higher sand content. The best agreement with the field measurements occurs for the Neap tide simulation scaled to a Spring range where in many regions the prediction and measured result are within 10-20% of one another. However, due to the uncertainty in the measured Availabilities it cannot be definitely stated which simulation provides the best solution. However, it is emphasised that general agreement results when many factors could cause deviations. This solution is reached by summing fluxes throughout the tidal cycle with the possibility of a large error accruement. The velocity-time curve is only sampled at ½ hour intervals, the roughness length and grain size were constant throughout the model region, the grid size was farily coarse at 250m, and the operation of the Iterative Mode depends on the 1-dimensional and 2-dimensional hydrodynamic models as well as the sediment transport model, and yet there is clearly a good general correspondence.

It is apparent that a prediction of the nature of the sea bed in an inlet may be made using the numerical models alone. The major field data input required is only the bathymetry and the nature of the sea bed facies at the open boundaries.

11.20 The impact of the proposed timber port

The proposed timber port is expected to have an impact primarily on its immediate environment (Chapter 10). The magnitude of this impact is summarised in the following sections by maintenance dredging requirements and erosion rates.

11.20.1 Total sand cover

Assuming the harbour floor is entirely sandy, the annual bed level changes and volume changes with the port included are shown in Figures 11.34 and 11.35. Accretion occurs along the northern and western flanks of the

dredge zone primarily on the batter slopes. Maximum volume changes occur in the north and west corner in two adjacent cells - 22,000 and 16,000 m^3/year. The total accretion in the dredge zone is 47,000 m^3 including the pore spaces.

On the 1:10 batter slopes, sediment moving as bedload would be expected to cause infilling by travelling downhill under the action of gravity (DHI, 1982). Using the method of Fredsoe (1978), the DHI estimated that this would cause infilling at the rate of 18,000 m^3/year (corrected by this writer for pore spaces with P = 0.45). In this case, the total sedimentation may be as much as 65,000 m^3/year. This lies below, but near the middle of the range of 40,000 to 120,000 m^3 obtained by the DHI assuming an all sand bed. It is evident that for a sandy bed, the DHI estimate of sedimentation and the analysis of this writer are compatible.

This writer's model predicts scour in the central, southern and eastern segments of the dredge zone. The region where the sediment ridge is currently located is also subject to some scouring. As proposed in Chapter 10, the changed hydraulics prevent its re-establishment. In order for the ridge to reform, flow patterns need to approximate the present regime. This may occur if the ridge can creep forward from the north-west corner of the dredge zone simultaneously altering flow conditions as it progresses. However, there is no assurance at this stage that there is enough sediment available in the prototype to allow this, and the rate of progression will be a function of the supply.

At the NZRC wharf for the all sand configuration, the port has no significant effect on the erosion/deposition patterns with 1000 m^3 of erosion predicted both with and without the port (Figures 11.27 and 11.34). More notable is the scour in the centre of the entrance throat and accretion to the north after construction. However, mid-gorge is currently shell lagged and the scour predicted should be almost totally retarded by the lag gravel.

The shipping channel towards One Tree Point is subject to greater scour and balancing accretion.

11.20.2 The stable harbour

This section examines the erosion/deposition rates for the stable harbour configuration using the proper representation of the bed facies verified by the calibration discussed earlier. Accordingly, the results presented here, by accounting for shell cover, are considered to be the most reliable sedimentation rates expected after port construction.

As a worst case and to provide approximate upper limits on the dredging quantities, it is assumed that the dredging will expose a total cover of non-cohesive sand (Figure 11.35). If shell is uncovered or the bed is found to be cohesive, the dredging quantities will be reduced. All other regions are left unchanged from the Availabilities in the present configuration.

11.20.2.1 *Sedimentation rates*

Figures 11.37 and 11.38 show the annual volume and bed level changes after construction. The only notable accretion occurs in the northern side where 16,000 and 3,000 m^3/year are deposited in two adjacent cells. Much of this is due to sediment passing in the ebb direction along the ebb-tidal spit from Snake Bank where some upstream scour of 9,000 m^3/year is evident. Notably, in this scouring cell considerable deposition of 22,000 m^3/year was predicted in the all-sand case. The western margin of the dredge zone is still subject to accretion; diminished, however, from 7,000 to 1,000 m^3/year with proper bed representation. Most of the dredge zone is expected to erode or stabilise if cohesive or shelly sediments are exposed.

Thus, as suggested after examination of field current measurements, the primary source of sedimentation is the ebb-tidal spit from Snake Bank, and deposition in large quantities should not occur in mid-channel on the western flank of the dredge zone.

Contrary to this, there should be some infill on the batter slopes reducing the gradient due to: (i) Gravitational effects moving sediment downhill. (ii) Retarded flow at the bed due to sheltering of the near-bed levels by the batter slopes similar to the reverse eddy circulation modelled in Figure 10.10. These influences should be localised.

In summary, the model shows that approximately 20,000 m^3/year of sedimentation could occur in the dredge zone. In view of the necessary approximations made, the maintenance dredging volumes should lie in the range of 10,000 to 30,000 m^3/year including the pore spaces for $P = 0.45$ and for a grain size of 0.0003m. This figure would be reduced if the bed in the dredge zone is floored by immobile material or increased by about 20% for a smaller grain size of $D_{50} = 0.00025$m. As sedimentation is somewhat restricted, the gravitational influence on the batter slopes should cause early, additional sedimentation of no more than 5,000 m^3/year until the slopes flatten and stabilise. This estimate utilises the DHI determination of 18,000 m^3/year in the all-sand case.

11.20.2.2 *Marsden Point and the NZRC wharf*

Comparison of the volume changes (Figures 11.29 and 11.37) at the NZRC wharf reveals only minor changes to sedimentation patterns and the port will have a very limited impact on Marsden Point.

11.20.2.3 *The shipping channel and Shoal Bay*

In the shipping channel after port construction, the changes to the erosion/deposition patterns are small and vary locally. Generally there is an increase in sediment movement. In Shoal Bay, the port has no notable influence.

11.20.3 The Iterative Mode

With unchanged boundary conditions, Iteration produced three similar stable configurations after port construction as shown in Figures 11.39 to 11.41. These coincide with the same three cases of tidal range and cut-off flux as detailed earlier for the case without the port.

In the dredge basin, the percentage sand cover has increased in all cells. However, the cover is still mostly less than 100%. Considering this as representing the percentage efficiency of the current necessary to remove all sediment input, then the iteration procedure shows that the new bathymetry could become stable simply by adjusting the percentage of sand cover rather than altering the bathymetry. Whether the estuary makes this adjustment depends on the availability of both types of sediment. However, by extending this notion, it is apparent that only cells predicted to have approximately 100% sand cover can accrete. The others, which do not utilise the full potential of the current will scour if the bed is sandy.

There is one 100% cell on the central north side of the basin with the Spring tide simulation (Figures 11.39 and 11.40) and one on the central western side and a 94% cell in the north-west corner with the Neap tide (scaled to a Spring tide) simulation (Figure 11.41). Thus, the iterative technique provides further evidence that most of the dredge zone will not be subject to accretion and only limited deposition will occur on the northern and western flanks of the dredge zone.

11.21 Discussion

Whangarei Harbour has proved to be unusual in one very important aspect. It is an inlet with a very low sandy sediment supply. The entrance is tucked in behind Busby Head sheltered from most wave attack on what is already a low energy coastline compared with its west coast counterpart . As has been noted many times during the field program, shell lag lines the channels in the high current flows, and importantly, it prevents further deepening of the channels. Erosive current flows are amply sufficient to move sandy sediment but they are mostly unable to dislodge the aged, algae-encrusted shell.

With respect to the timber port the implications are now apparent. Artificial deepening of the channel by dredging accomplishes the scouring of shells which the currents cannot attain. But, unlike other sandy inlets where waves and currents are expected to induce total infilling of the deeper channel as the inlet strives to attain the former equilibrium, in Whangarei Harbour firstly, the sediment is not available and, secondly the channel may still not be sufficiently deep for the inflow of sediment to ever exceed the potential discharge. In simple terms, in Whangarei Harbour it may be possible to dig a hole without expecting it to fill up again!

Strong support for this is offered by the dredging of the Shell Cut Reach Channel in Whangarei Harbour and only 3 kms upstream from the timber port. Dredged to 8.1m from a minimum of about 4.3m in 1969 (NZ5213), repetitive soundings by the NHB Pilot's Office have revealed no appreciable change in depth since the dredging some 14 years ago. In this time span the channel has not needed any maintenance dredging.

The model has simulated this low supply situation and is predicting similar behaviour further downstream at the timber port site. The only significant accretion occurs beside the only known supply of sandy sediment - the Snake Bank ebb-tidal spit. Thus, excluding localised influences on the batter slopes, dredge basin accretion is expected to be limited.

11.22 Conclusions

(i) A 2-dimensional sediment transport numerical model was designed to extend the 2-dimensional hydrodynamic model, enabling determination of the sediment transport budget in a tidal inlet. The model behaviour was examined using several test rigs and was calibrated and applied to sediment transport in Whangarei Harbour, particularly to study the impact of the proposed timber port at Marsden Point. The results of the calibration and other behavioural aspects revealed that the model is performing its design function.

(ii) The numerical scheme introduced is similar to the scheme designed for 2DD. Sediment fluxes are calculated on the mid-points of the cell walls. Tests revealed that this scheme was considerably more accurate and reliable than a scheme which derives fluxes at the centre of each cell.

(iii) Two parameters to describe the nature of the sea floor were introduced. These are the Availability and Erodibility which represent the percentage of sea floor covered by sandy sediment and the cohesiveness of the sediments respectively. The Availability provided for much better representation of the sea floor facies in Whangarei Harbour. The alternative is to assume the harbour is floored entirely by non-cohesive sandy sediments, but field investigations do not support this.

(iv) Sediment transport fluxes are derived using the "new" procedure presented and field tested in Chapter 7. The skin friction velocity is obtained from the 1m speed and the D_{65} grain size,for application in the Yalin Bedload Equation for plane beds. The suspended load is found with an empirical formula derived over megaripples in Whangarei Harbour which relates the vertically-integrated mass transport flux to the 1m speed. To apply this, 1m speed must be obtained from the mean vertically-averaged speed derived by the flow model. The Karman Prandlt equation was applied and utilising field sediment transport measurements, the method was found to be acceptable.

(v) The Engelund and Hansen equation, although it was mostly adequate when mean speed and friction velocity were measured, was found to be

difficult to apply due to a strong dependence on roughness length.

(vi) Some highly informative numerical experiments were conducted by coupling the sediment transport and hydrodynamic numerical models. When unified, the models simulate the growth of a tidal inlet starting from a simple schematised configuration and evolving to a developed estuary, with formed banks and channels in a state of dynamic sediment transport equilibrium. Introducing a schematised harbour having a symmetrical and an offset entrance, the models predicted the formation of the ebb and flood tidal deltas, scouring in the throat, and marginal channels at the entrances. At the symmetrical throat, an ebb shield developed and was found to be stable.

(vii) The schematisation was similar in plan to Tauranga Harbour and tidal cycle residual circulation in the schematised inlet with rectilinear bathymetry exhibited many features of nett flow deduced by Davies-Colley and Healy (1978) and Dahm (1983) after studies in the prototype. This supported prior evidence in Chapter 10 that the harbour outline plays a dominant role in determining the residual circulation and suggests a possibility of using the hydrodynamic and sediment transport numerical models to predict equilibrium bathymetry after port construction. This is the numerical equivalent of a physical loose-bed model.

(viii) Simulations of Whangarei Harbour with an all sand bed revealed that the shell is responsible for stability, and prevents further deepening of the shipping channel. Sedimentation was predicted to occur on the known narrow distended sandy regions (a) extending south-east of Snake Bank and (b) along the north-east flank of Snake Bank in Shoal Bay. The flood-tidal deltas were mostly unaffected by the increase in sediment supply and have apparently reached a maximum level of accretion for the given flow conditions.

(ix) The model was successfully calibrated after estimation of the Availability throughout the inlet using underwater photography, dextural analysis of bed sediment samples, and side-scan sonar. A 10 iteration optimisation was performed.

(x) An Iterative technique was designed, whereby the model itself seeks the stable harbour configuration by adjusting the Availabilities until a dynamic sediment transport equilibrium is attained. The results compared very favourably with the actual values. Thus, a prediction of the nature of the sea bed facies in an inlet may be made using the numerical models alone. Moreover, the procedure may be used to determine minimum quantities of dumped material necessary to establish a dynamic sediment equilibrium over large areas of sea bed.

(xi) If the harbour was entirely floored with non-cohesive sandy sediment, sedimentation after port construction could be as great as 65,000 m^3/year. This generally agrees with an estimate by the DHI (1982). However, when the shell cover is accounted for, the sedimentation rates are

much less. Maintenance dredging requirements were determined to be in the range of 10,000 m^3/year to 30,000 m^3/year including the pore spaces with another 5000 m^3/year of localised deposition on the batter slopes. This assumes that the dredging will reveal non-cohesive sandy sediments in the dredge basin. The quantities will be reduced if the bed is shelly or cohesive. Most sedimentation is concentrated on the northern side of the dredge basin adjacent to the ebb-tidal spit from Snake Bank.

(xii) The port is expected to have a very limited influence on Marsden Point. There is increased sediment movement in the shipping channel to One Tree Point and some small changes to the channel should occur being of the same magnitude as changes noted after dredging of the Shell Cut Reach Channel. In Shoal Bay, the port has no notable influence.

SEDIMENT TRANSPORT AND TIDAL INLET HYDRAULICS

Chapter 12:

CONCLUSIONS

CHAPTER 12: CONCLUSIONS

The major study aims have been achieved, i.e. to assess the present sediment transport characteristics of the tidal inlet to Whangarei Harbour, to undertake research into tidally-induced sediment transport, and if possible to extend the investigation to predict the impact of the construction of the proposed timber port at Marsden Point. This Chapter provides a summary with general conclusions of the research undertaken to satisfy these study aims. Detailed conclusions are summaried in point form at the end of each Chapter and the reader is advised to consider these in conjunction with the following.

The study area primarily extends from One Tree Point in the west to northern Bream Bay. Centred in this zone, is the deep (31m) and narrow (730m) entrance throat which separates two flood-tidal deltas - Snake and McDonald Banks - from two ebb-tidal deltas - Calliope and Mair Banks. Snake Bank, in the decelerating, divergent flood jet, isolates the shipping channel to One Tree Point from the slower currents in the shallower Shoal Bay Channel. Two narrow, distended ebb-tidal spits flanking a scoured flood ramp project seaward from the high point of the Bank at a well-developed ebb shield composed primarily of disarticulated cockle shell. Sub-aerial Mair Bank has east-west alignment and separates the deep, main channel from the sub-tidal expanse of the majority of the ebb delta to its south. McDonald and Calliope Banks are subject to much lower tidal and wave energy levels.

Sea bed investigations by SCUBA observation, underwater photography, side-scan sonar and sediment textural analysis revealed that the channel floors are typically armoured with a lag deposit of shells, and cobbles, occasionally interspersed with patches of sandy sediments. The shipping channel, the Shoal Bay channel and the entrance throat are all of this kind. The lag shelly gravels in low energy zones with limited suspended sediment loads are encrusted with algae or anenomes - Shoal Bay, Snake Bank flood ramp. In the entrance, the shell is cleaner, abraded by sediment impact under stronger current flows.

Sandy regions are strictly limited inside the harbour except in low current regions on the banks and inter-tidal flats. A narrow, distended ebb-tidal spit extends south-east from Snake Bank. Another is found on the north-east flank of Snake Bank and along the 5m contour on the southern side of Shoal Bay. The zone of largest sand waves is found in high energy flow off Reotahi Bay, with wavelengths up to 50m and heights of 1.6m.

Outside the entrance, a major megaripple bedformed zone of active sediment transport flanks the main channel where ebb currents diverge south of Busby Head. Large sand waves, often backed by megaripples, indicate a dominant ebb jet which sweeps sand out of the harbour. Inter-tidal Mair

Bank is shelly at its eastern end and sandy to the west. The bank is distinguished from the prograded sandy spit of Marsden Point by a narrow, marginal flood channel floored by shell in the strongest currents and by sand at its northern and southern ends.

Bathymetry comparisons revealed neglible changes to the banks in the inner harbour and entrance throat during the 20 year interval between surveys. Limited scouring (≈0.3m) flanking the shipping channel to One Tree Point apparently indicates the minor impact of increased flow velocity since the dredging of the Shell Cut Reach Channel. Depicted by a 30-40^{o} rotation of the major axis, Mair Bank has accreted off Marsden Point on the upper levels, but this is almost totally balanced by scour between the 1m and 3m contours. Large-scale sedimentation on the swash platform of the southern parts of the delta offshore from the NZED power station is mirrored by degradation further offshore on the flanks of the main ebb channel.

Dominant circulation in the harbour is described by three facets. Overwhelming ebb currents, responsible for scour along the shipping channel, pass from One Tree Point through the proposed port zone and out of the entrance on the southern side. To its north, the Snake Bank flood ramp links the flood currents which dominate the northern side of the entrance to the ebb shield on the bank, the flanking ebb-tidal spits, and to the controlling flood currents around the north-east of Snake Bank in Shoal Bay. The Shoal Bay channel is controlled by an ebb-directed stream. Within the inlet, zones of tidal cycle current neutrality are reflected by deposition i.e. the two ebb spits off Snake Bank, Blacksmith's Creek region, off Reotahi Bay. Some of these are subjected to fast currents (≈1 $m.s^{-1}$).

The scoured lag gravels on Mair Bank at the eastern end are swept clean of sand by ebb currents and not directly linked with the flood-directed pathway of sandy sediment along Bream Bay Beach and through the marginal flood channel. In plan, the residual currents from a clockwise-rotating closed or partially closed loop passing through Marsden Point.

A much larger loop in Bream Bay includes: (i) tidal current induced sediment transport around the north and east of Mair Bank in the shipping channel; (ii) inferred wave-induced transport shorewards from the south of Busby Head across the swash platform of southern Mair Bank and (iii) long-shore return of sediment from the power station environs to Marsden Point.

The inertia of the ebb jet expanding beyond Busby Head establishes residual south to south-east circulation in Bream Bay which is evidently aided by the North Auckland Current. Controlling flood flow passes along the land from Bream to Busby Head and also across the northern and central shallow portions of the ebb-tidal delta.

Protection of the throat by Mair Bank and Busby Head, ensures entrance throat tidal dominance even though waves of some significance from the east

to south-east sector do occasionally impinge on the region. The tidal dominance is denoted by an unusually wide entrance throat prevented from further scour by coarse shelly lag.

Due to current inertia in the deeper, central regions of the throat the southern margin floods while the ebb continues offshore. This is linked with early flood flow through the marginal channel off Marsden Point. Similarly, the southern side ebbs earlier than the central regions.

The inlet is typified by fastest currents nearer high than low tide during both half cycles, caused by the expansion of surface area as the tide rises and the inter-tidal zones become submerged.

Bed friction in Whangarei Harbour is primarily associated with lag shell beds with Chezy's C being $C = 60\ m^{\frac{1}{2}}.s^{-1}$. The roughness length has a median value of 0.0007m. Over megaripples the roughness length is greater, being in the range 0.002-0.003m and reflecting a greater form drag component. The median Drag Coefficient was 3.03×10^{-3} but the parameter varies over 4-5 orders of magnitude in Whangarei Harbour in partial response to changing sea bed facies - varying from sand, to gravel, to marine biomass etc.

The form drag components account for a large portion of the range of the Drag Coefficient. Over non-cohesive sand, the Drag Coefficient is better behaved and an empirical, roughly-linear relationship between C_1 and the velocity at 1m was obtained.

The Universal Velocity Defect Law and the Karman Prandtl Equation were applied in the outer and inner boundary layers respectively when a predicted difference in boundary velocity gradient in the two regions was measured. Over megaripples, 50% of the profiles were logarithmic at the $\alpha = 0.10$ level of significance.

The equation (6.57), which relates the roughness length to the bed geometry,

$$z_o = \Delta^2/2\Lambda \qquad 12.1$$

presented by Dyer (1980) after Lettau (1969) was found to be approximately valid over megaripples.

Research was concentrated over megarippled beds to isolate the effective bed shear stresses for application to the transport of sandy, non-cohesive sediment. Several theories and empirical relationships were examined to uncouple the form drag and skin friction components of the friction velocity. This was mostly unsuccessful - including the Einstein Barbarossa division - until excellent predictions were made using the Vanoni and Hwang (1967) equation (6.67) for form drag,

$$c''/\sqrt{8} = 3.3 \log_{10} (d\Lambda/\Delta^2) - 2.3 \qquad 12.2$$

and the Manning-Strickler equation for skin friction,

$$c' = 7.66(d/D_{65})^{\frac{1}{6}} \qquad 12.3$$

when predicted values were combined as,

$$1/c^2 = 1/c''^2 + 1/c'^2 \qquad 12.4$$

no more than a 3% deviation from the measured median friction factor occurred.

Utilising the Karman Prandtl equation, a second equation was examined to derive the effective skin friction component,

$$c' = 5.75 \log_{10} \left(\frac{0.37d}{2D_{65}/30.2}\right) \qquad 12.5$$

and when combined with the Vanoni and Hwang equation, a rougher bed than that suggested by the median friction factor was predicted. However, the results were consistent being 6-10% less than median z_o at four different megarippled sites.

With the ability to isolate the skin friction and form drag, a new method to predict the sediment threshold speed over megaripples was introduced. The critical friction velocity, which includes both form drag and skin friction components, is found (eqn. 7.9) as,

$$u_{*cr} = \frac{u'_{*cr}}{\left\{1 - \left[\frac{5.75}{c''} \log_{10} \left(\frac{0.37d}{z_o}\right)\right]^2\right\}^{\frac{1}{2}}} \qquad 12.6$$

The critical velocity at 1m is then,

$$u_{1cr} = 5.75\, u_{*cr} \log_{10} (1/z_o) \qquad 12.7$$

The roughness length may be obtained with equation 12.1 from the bedform geometry, c" is obtained with the Vanoni and Hwang equation (12.2) and u'_{*cr} from the Yalin Threshold curve for plane beds. This theoretical derivation was verified using underwater video observations of threshold in Whangarei Harbour as well as data published by Sternberg (1971). A graphical procedure for simpler application is described in Section 7.9.6.

Both the above skin friction equations include the mean flow speed and in deep, estuarine flow, a near-bed speed may be a better predictor of sediment transport rates. Accordingly, a new, but simpler equation to find the skin friction velocity was introduced which includes the 1m speed,

$$u'_* = u_1/\left[5.75 \log_{10} (30.2/2D_{65})\right] \qquad 12.8$$

By utilising results from several sediment transport experiments conducted over megaripples, this equation was found to be representative of the effective component of the friction velocity which is responsible for the transport of sandy sediment under tidal flow. In conjunction with the Yalin equation for plane beds (7.24), and utilising the above equation with

the measured 1m velocity to find u_*', a new procedure to find the bedload transport rates was introduced.

Further to the bedload trapping experiments, a detailed verification of the method was undertaken using simultaneously measured megaripple advance data. At three sites, the nett tidal cycle megaripple advance, which includes an upstream and downstream component with bedform destruction occurring at the change of tide, was compared with the predicted nett flux derived by utilising only the half-hourly velocity measurements at 1m above the bed and the D_{50} and D_{65} grain sizes. Agreement between the two entirely independent procedures was extremely good. This confirmation is potentially subject to a large error accruement due to experimental error over the complete tidal cycle, and accordingly, it offers very strong support for the new method. As the bedload equation includes grain size, density and a critical stress, it may have quite general applications.

A significant correlation between the excess velocity at 1m and the bedload transport rate according to the formula of Gadd et. al. (1978) was also obtained. However, the empirical constant of proportionality at Whangarei was almost an order of magnitude less than that found by Gadd et. al. The equation may be used as a first estimate of bedload transport rates although the constant depends at least on the grain size which here was typically 0.0003m. The mass flux per unit time per unit bed width is,

$$q_m = 0.145\ (u_1 - u_{1cr})^3 \qquad 12.9$$

Considerable care must be exercised when determining the threshold velocity for this equation and the method previously described is recommended.

Utilising data from the sediment transport experiments, an empirical suspended sediment transport equation was derived relating the vertically-integrated mass transport rate per unit time per unit bed width to the 1m velocity (eqn. 7.35),

$$q_{ss} = (2.03\ u_1)^{7.75} \times 0.001 \qquad 12.10$$

In combination with the measured bedload rates, total load transport rates were produced and comparisons with the Engelund and Hansen formulation were adequate at 2 out of 3 measurement sites. At the shallow third site, the Engelund Hansen equation systematically under-predicted the sediment fluxes.

The distribution of tidal ranges in Whangarei Harbour at Marsden Point is approximately Normal with mean of 1.87m and standard deviation of 0.347m. Furthermore, there is a linear relationship between tidal range and flow velocity. In conjunction, these results make it possible to scale velocity profile measurements according to the tidal range for determination of annual sediment transport rates. Utilising this and the new sediment transport equation, typical tidal cycle nett transport rates in Whangarei Harbour for an

all-sand bed were found to be 20-50 m^3/m/year with a maximum at the highly-asymmetric flood-dominated site on the Snake Bank flood ramp of 402 m^3/year. With some accounting for the shell content maximum sedimentation rates at the proposed port and including the pore spaces were estimated to be 36,620 m^3/year occurring predominantly on the north-western regions of the dredge zone. This estimate is later refined using 2-dimensional model results. Approximately 4500 m^3/year is evidently passing in the ebb direction along the ebb shield off One Tree Point. The nett flux moving up the north-east side of Snake Bank in a flood direction is approximately 3500 m^3/year.

On the upper platform of the inter-tidal zone at the port site, the nett sediment transport patterns are complex and 2-dimensional. Sediment fluxes are neglible except under attack from surface gravity waves either locally generated in the harbour or from low, longer-period swells passing through the throat during storms. The latter are much reduced in amplitude by refraction and diffraction. Nett fluxes are estimated to be about 1 m^3/m/year with up-harbour and down-harbour components being in the range of 2-10 m^3/m/year. Accordingly, the proposed timber port will have a very small impact on its environment due to changed sediment transport patterns on the upper platform of the inter-tidal zone.

Measurements of wave heights with a capacitance-type recorder revealed generally low wave conditions with a maximum significant height of 0.36m and significant period of 2.6s. The spectrum consistently exhibited a low frequency peak associated with longer period oscillations than those of the local waves.

Three numerical models were developed for the study. These are one- and two-dimensional hydrodynamic models and a 2-dimensional sediment transport model. The hydrodynamic models employ an explicit, finite-difference scheme for solution of the hydrodynamic equations. The one-dimensional model includes terms for acceleration both local and convective, sea surface slopes, bed friction, surface wind stresses,and density gradients. The two-dimensional case excludes density gradients, but introduces Coriolis force and horizontal eddy viscosity.

With a 1-d schematisation of Whangarei Harbour, a good prediction of the Port Whangarei tidal curve was made using the down-harbour sea levels at Marsden Point, verifying model behaviour. Accordingly, the one-dimensional model was employed to generate open boundary levels for the 2-d case. This procedure, which has not been reported elsewhere to this writer's knowledge, can be very effective. The one-dimensional model has small computer memory requirements. With the generated sea levels at the open boundaries, areas of most concern may then be modelled in detail in two dimensions.

The 2-dimensional model behaviour was verified using test rigs and by application to tidal hydraulic modelling in Whangarei Harbour. Further to this, it was compared with a mixed-implicit model called S21 (Abbott, et. al. 1973), which was utilised by the DHI (1982) under contract to the Northland Harbour Board for simulation of Whangarei Harbour. The comparisons (explicit and mixed-implicit Alternating Direction Double Sweep) had equivalent accuracy, and applicability.

The 2-dimensional model incorporated several new features designed by this writer. These included: (i) a percentage of slip at the boundaries to enable reduction of the thickness of an untenably-large boundary layer which develops at land/sea boundaries on coarse grids; (ii) the isolation of the cause and the prevention of zig-zagging established by a poor approximation for the convective momenta terms at land/sea boundaries over steep bathymetry where the velocity gradient changes sharply; (iii) a new schematisation of the bathymetry which eliminates the need to average surrounding values to find the depth, and offers approximately twice the resolution without necessitating significantly larger computer memory requirements; and (iv) an approximation to the Chezy's C function to reduce computation time.

Other features incorporated in the model were the flooding and drying of inter-tidal zones, and the introduction of an effective depth (after Ramming and Kowalik, 1980). The model has general applicability to tidal inlets and allows either flow speeds or boundary sea levels to be specified at the open boundaries.

When applied to Whangarei Harbour, dominant sediment transport pathways, neutral zones of deposition and scoured shell lags, conforming with field observations, were isolated. Possibly, the two most important determinations with respect to the proposed timber port were (i) the neutrality of the sandy sediment ridge discovered during sea bed investigations to the north of the dredge zone and; (ii) the scoured lag shell beds associated with a strong ebb-directed residual in the shipping channel and at the port site.

Ebb dominant zones were found to be shell lagged; supporting a "one-way" transport process by helping to balance channel ebb dominance such that sediment entering the harbour on the flood routes is not as easily removed again by the ebb currents. Predominant transport occurs on the channel flanks at about the 5m contour where the erosive potential transport rates balance with the availability of the sandy sediment. Of course, this implies that tidal flow measurements or hydrodynamic model predictions may not be directly interpreted as providing sediment transport rates without complementary information about the bed type and the wave energy at the throat.

A second natural balance of the sediment availability and the current flows was demonstrated with the one-dimensional hydrodynamic model. In a

schematised rectangular basin with no inter-tidal zones the flood currents were faster and of shorter duration than the ebb. However, in Whangarei Harbour the opposite occurs, and is caused primarily by the harbour morphology and the expansion of surface area with the rising tide. The modelling showed that as the inter-tidal zones evolve, the estuary becomes successively more ebb dominated. This reduces the potential for harbour deposition. Thus, for given wave and current conditions outside the throat responsible for the input of sediment, and with ebb tidal dominance within the harbour, these deposition and erosion processes may balance accordingly.

By independent examination of the terms in the hydrodynamic equation, the convective momentum was found to be primarily responsible for residual circulation in the harbour. The horizontal diffusion operation redistributes flow velocity and smooths the eddy circulation. The Coriolis force mostly supports the residual flow.

Numerical experiments suggested that the harbour outline, rather than inlet bathymetry, is a dominant influence with respect to the residual circulation. Thus, banks and channels may form in given locations as a result of the shape of the harbour outline.

The sediment transport model developed here is the first of its kind to this writer's knowledge. By utilising the complete tidal cycle flow patterns from 2DD, the model derives the sediment budget throughout the estuary by coupling a conservation of matter algorithm to balance tidally transported sediment fluxes into and out of model cells. Both the bedload and suspended load fluxes are included. Bed facies may be schematised to represent the sediment transport potential at the sea floor, governed by the Availability of sandy sediment. As an extension to this, an iterative technique was developed, which will determine the bed sediment availability knowing only the boundary conditions for the given flow pattern. In Whangarei Harbour, the model needed to provide these options when lag shell plays a significant role in the sediment budget.

In a coupled mode, the hydrodynamic and sediment transport models interact to produce an evolved inlet in a state of dynamic sediment equilibrium starting from an initially schematised configuration. The models together predicted the formation of ebb and flood tidal deltas, marginal channels and deep scouring in the confined entrance throat. Moreover, a stable ebb shield developed on the flood-tidal delta. These are all essential geomorphic features of a typical tidal inlet on New Zealand's east coast. This procedure is the numerical equivalent of a loose-bed physical model and has considerable application for the prediction of stable equilibrium bathymetry after man-made changes due to port construction, training walls etc.

In Whangarei Harbour, the sediment transport model was calibrated by

utilising field observations of bed facies and then applied to estimation of the impact of the proposed port at Marsden Point. Due to the shell lags in the shipping channel, the port is expected to experience limited accretion after construction. Maintenance dredging requirements were estimated to be in the range of 10,000 to 30,000 m^3/year which is an upper limit that depends on the bed facies uncovered after dredging. The quantities should be reduced if lag shell beds develop. Most sedimentation will occur on the north and north-west side of the basin adjacent to the ebb-tidal spit extending from Snake Bank.

The numerical modelling showed that what is currently a neutral ridge of sedimentation on the north-east corner of the dredge zone, becomes ebb dominant after dredging and much less prone to accretion. Interestingly, this suggests a possibility of specifically designing the shape of the dredge basin to enforce erosion in known depositing areas. Using the hydrodynamic and sediment transport models in conjunction, this would be possible.

SEDIMENT TRANSPORT AND TIDAL INLET HYDRAULICS

APPENDICES

APPENDIX 1 POSITION FIXING

Throughout the study, most position fixing was performed either with a sextant, or with NHB Motorola Electronic Distance Measuring equipment. As so much of the work undertaken required an ability to place objects in space, some specialty computer programs were written to make the operations manageable. The mapping aspects are discussed in Appendix 2 and the reduction of position fixes is detailed here.

A1.1 Sextant

When operating a sextant your position lies on a circle, the locus of which is defined by the coordinates of the two fix marks, and the angle subtended by this chord at the point on the circumference where the sextant angle was shot. Your position is obtained from the intersection of the two circles defined by the sextant fixes.

The general equation for a circle is,

$$(x - a)^2 + (y - b)^2 = r^2 \qquad \text{A1.1}$$

where the circle has centre coordinates of (x = a, y = b) and radius, r. With the sextant angle, α, and the coordinates of the landmarks, (x_1, y_1) and (x_2, y_2), after making the constructions shown on Figure A1.1, the radius of the circle is obtained from the half length of the chord OA and the right triangle OBC, so that,

$$r = \frac{\sqrt{(x_1 - x_2)^2 + (y_1 - y_2)^2}}{2 \sin \alpha} \qquad \text{A1.2}$$

Note that the angle subtended by a chord at the centre of a circle is twice the angle subtended at the circumference. From triangle OBC, the coordinates of the centre of the circle are,

$$a = x_1 + r \cos \phi \qquad \text{A1.3}$$
$$b = y_1 + r \sin \phi$$

where

$$\phi = \psi - (90 - \alpha) \qquad \text{A1.4}$$

and ψ is obtained from the landmark positions and the gradient of the line OA, as,

$$\psi = \text{atan} \left[(y_2 - y_1)/(x_2 - x_1)\right] \qquad \text{A1.5}$$

From equations A1.2 and A1.3, the circle is defined by the known parameters. With two fixes, two circles are defined, and their intersection remains to be found.

If the general equation, (A1.1) is expanded and terms grouped then,

$$x^2 - 2xa + y^2 - 2yb = c \qquad \text{A1.6}$$

where

$$c = r_1^2 - a^2 - b^2$$

for the first circle of radius, r_1, and centre (a, b).

Similarly

$$x^2 - 2xd + y^2 - 2ye = f \qquad \text{A1.7}$$

where

$$f = r_2^2 - d^2 - e^2$$

for the second circle of radius, r_2, and centre (d, e)

Subtracting A1.6 from A1.7 and rearranging gives,

$$y = M - Nx \qquad \text{A1.8}$$

where

$$M = \frac{f - c}{2\,(b - e)}, \quad N = \frac{a - d}{b - e}$$

By substitution back into equation A1.6,

$$x^2 - 2xa + (M - Nx)^2 - 2\,(M - Nx)\,b = c \qquad \text{A1.9}$$

and after rearrangement,

$$x^2 + Hx + G = 0 \qquad \text{A1.10}$$

where

$$H = \frac{(2\,Nb - 2\,MN - 2a)}{1 + N^2}, \quad G = \frac{(M^2 - 2\,Mb - c)}{1 + N^2}$$

This is a general quadratic equation with solutions,

$$x = -\frac{H \pm \sqrt{H^2 - 4G}}{2} \qquad \text{A1.11}$$

The coordinates of the two points of intersection using A1.8 are,

$$\left(x = \frac{-H + \sqrt{H^2 - 4G}}{2}, \quad y = \frac{2M + NH - N\sqrt{H^2 - 4G}}{2}\right) \qquad \text{A1.12}$$

and

$$\left(x = \frac{-H - \sqrt{H^2 - 4G}}{2}, \quad y = \frac{2M + NH + N\sqrt{H^2 - 4G}}{2}\right)$$

A computer program called SEXFIX was written to perform these manipulations. The program accepts landmark identification and sextant angles to produce positions in the Mt Eden Geodetic Metric Coordinate Grid. To simplify program operation, some 64 landmarks in the study region with known coordinates have been numbered and their coordinates stored on computer disc memory. The operator does not have to enter the coordinates of each landmark (which is usually 16 digits long) but may optionally enter the number of the landmark only, and its coordinates are retrieved from the disc for use in the computer program. The numbering sequence is given in Table 1.1. There are two circles, (each reflected about the chord through the landmarks), defined by each sextant angle, so the operator is requested to enter the left hand landmark first (as viewed through the sextant) to remove this ambiguity. As

there are still two intersection points, both are printed and the operator selects the correct one. The program allows for the construction of an output disc file containing the reduced coordinates, fix identification and the time of the fix for later manipulation and plotting.

Calibration of the Kelvin Hughes surveyor's sextant (with 5 x magnification) was obtained by comparing sextant angles with (i) angles shot with a theodolite and (ii) angles obtained by triangulation using the known coordinates of the fix marks and the position from which the fixes were taken. Shown on Table A1.1, the deviations were usually less than 2' except for those involving Passage Island, which was found to be incorrectly placed in the Metric Grid. After calculating positions from pairs of fixes on Table A1.1 using SEXFIX, it was found that under good fixing conditions, where care can be taken with the shots (not for example, on a moving vessel), the sextant accuracy is better than 6m using angles less than about 25^{o} and ±3m for larger angles.

A1.2 Electronic Distance Measuring Equipment

Motorola EDM equipment outputs the distance from your position to two shore transponder stations placed at known coordinates. A computer program was written, called TRISPN, to reduce this information and create positions in the Mt Eden Geodetic Metric Grid.

In any x - y coordinate system (Figure A1.2), if the shore stations are located at (x_1, y_1) and (x_2, y_2) then the boat position (x_f, y_f) can be obtained as follows. After construction of the right angled triangle ABD, the angle BAD is given by,

$$\phi = \text{atan} \left[(y_2 - y_1)/(x_2 - x_1)\right] \qquad \text{A1.13}$$

The lengths BC and AC are known (given by the EDM equipment) and the length of AB is then,

$$\ell_3 = \sqrt{(x_2 - x_1)^2 + (y_2 - y_1)^2} \qquad \text{A1.14}$$

From the cosine rule, the subtended angle θ is,

$$\theta = \cos \left[(\ell_3^2 + \ell_1^2 - \ell_2^2)/(2\ell_1 \ell_3)\right] \qquad \text{A1.15}$$

From the right-angled triangle ACE, the coordinates of C are given by,

$$x_f = x_1 + \ell_1 \cos (\theta + \phi) \qquad \text{A1.16}$$
$$y_f = y_1 + \ell_1 \sin (\theta + \phi)$$

TRISPN takes the Motorola lengths and fix times to produce an output disc file of identical format to that generated by the sextant position fixing program. In both cases, the file is ready for introduction into further analysis programs and for plotting. With their streamlined input subroutines, SEXFIX and TRISPN made it possible to handle the large volume of position fixing data generated throughout the study.

Mr Bill Haines, NHB electronics technician, performed calibration of the Motorola instrument and found it to be accurate to 1m under static conditions.

A1.3 Coordinate conversions

Maps of the Whangarei region are usually projected in one of three grid systems i.e. (1) Geographic Latitude and Longitude coordinates, (2) North Island National Yard Grid, or (3) Mt Eden Circuit Geodetic Metric Coordinates. These three grids are not parallel and conversion of coordinates from one to another is not a simple matter. After discussions with the NHB surveyor, Mr E.N. King, it was decided that the best system to use for standardisation of all mapping in this thesis was the Metrid grid, because coordinates are easier to manipulate and because much of the current NHB work was already using this system. Unfortunately, no metric maps of Northland had yet been produced (1981) by the Department of Lands and Survey and moreover to convert from National Yards to Metrics, the coordinates had to be sent to the Lands and Survey computer in Auckland for reduction.

To overcome these difficulties it was decided to program the NHB computer to perform the coordinate conversions. The work was undertaken in collaboration with Mr R. Adams of the Lands and Survey Department, Whangarei who supplied the mathematical derivations required for the solutions. The computer was programmed to convert between the three systems from any one to the other two.

The shape of the Earth that has been adopted for topographical surveys in New Zealand is the "international spheroid" with fundamental elements,

major semi-axis, $a = 6\ 378\ 399.058\text{m}$ A1.17

compression, $(a - b)/a = 1/297$ A1.18

eccentricity, $e^2 = (a^2 - b^2)/a^2 = 0.00672\ 26700\ 22333$ A1.19

2^{nd} eccentricity, $e'^2 = (a^2 - b^2)/b^2 = 0.00676\ 81701\ 97224$ A1.20

The meridinal distance, m, from the equator to latitude, ϕ, is computed from the series formula,

$$\begin{aligned} m = a(1-e^2)\ \{&(1 + \tfrac{3}{4}\,e^2 + \tfrac{45}{64}\,e^4 + \tfrac{175}{256}\,e^6 + \tfrac{11025}{16384}\,e^8)\ \phi \\ &- \tfrac{1}{2}\ (\tfrac{3}{4}\,e^2 + \tfrac{15}{16}\,e^4 + \tfrac{525}{512}\,e^6 + \tfrac{2205}{2048}\,e^8\ ..)\ \sin 2\phi \\ &+ \tfrac{1}{4}\ (\tfrac{15}{64}\,e^4 + \tfrac{105}{256}\,e^6 \quad \tfrac{2205}{4096}\,e^8 + ..)\ \sin 4\phi \\ &- \tfrac{1}{6}\ (\tfrac{35}{512}\,e^6 + \tfrac{315}{2048}\,e^8 + ..)\ \sin 6\phi \\ &+ \tfrac{1}{8}(\tfrac{315}{16384}\,e^8 + ..)\ \sin 8\phi \\ &+ ..\}, \end{aligned} \qquad \text{A1.21}$$

and with a and e given above,

$$
\begin{aligned}
m = {} & 6367665.53987\ \phi \\
& - 16107.02620 \sin 2\phi \\
& + \quad 16.97624 \sin 4\phi \\
& - \quad\ 0.02227 \sin 6\phi \\
& + \ \ldots\ldots
\end{aligned}
\qquad \text{Al.22}
$$

where ϕ is in radians.

The original of the Mt Eden Geodetic Metric Grid is at,

$$
\begin{aligned}
&\text{Longitude} \quad 174^{\circ}\ 45'\ 51".6217 \\
&\text{Latitude} \quad\ 36^{\circ}\ 52'\ 47".5150
\end{aligned}
\qquad \text{Al.23}
$$

and this origin is falsely defined as (700,000 mN, 300,000 mE).
The origin of the National Yards Grid is at

$$
\begin{aligned}
&\text{Longitude } \lambda_o \quad 175^{\circ}\ 30' \\
&\text{Latitude } \phi_o \quad\ 39^{\circ}\ 00'
\end{aligned}
\qquad \text{Al.24}
$$

and this origin is falsely defined as (400,000 yds N, 300,000 yds E). The meridinal distance from the equator to the origin of the Metric Grid from equation (Al.22) is m_o (metres) = 4083252.388m and the National Yards origin m_o (yards) = 4318584.281 m.

After some initial steps the conversion from both National Yards and Metres into Latitude and Longitude is identical. Firstly from National Yards, shift the origin and convert to metres as,

$$N_1 = (N - 400{,}000) \times 0.9144, \quad E_1 = (E - 300{,}000) \times 0.9144, \qquad \text{Al.25}$$

From this position the meridinal distance to the equator relative to the Yards Grid origin is $m = m_o$ (yards) $- N_1$. The Metric coordinates are converted as,

$$N_1 = (N - 700{,}000)/0.999901718, \quad E_1 = (E - 300{,}000)/0.999901718 \qquad \text{Al.26}$$

The meridinal distance from the equator relative to the Metric Grid origin is found as,

$$m = m_o \text{ (metres)} - N_1$$

From this stage the conversion is identical. The first approximation to the Latitude is obtained from the implicit formula by iteration,

$$\phi_1 = \frac{1}{A_o}\left(\frac{m}{a} + A_2 \sin 2\phi_1 - A_4 \sin 4\phi_1 + A_6 \sin 6\phi_1\right) \qquad \text{Al.27}$$

and the constants are,

$$
\begin{aligned}
A_0 &= 0.9983172081 \\
A_2 &= 2.52525 \times 10^{-3} \\
A_4 &= 2.6615 \times 10^{-6} \\
A_6 &= 3.5 \times 10^{-8}
\end{aligned}
\qquad \text{Al.28}
$$

The radius of curvature in the meridian is,

$$\rho = \frac{a(1 - e^2)}{(1 - e^2 \sin^2 \phi_1)^{3/2}} \qquad \text{Al.29}$$

and the radius of curvature perpendicular to the meridian is,

$$\gamma = \frac{a}{(1 - e^2 \sin^2 \phi_1)^{1/2}} \qquad \text{Al.30}$$

Two other functions are defined as,

$$G = -\frac{\tan \phi_1}{2\rho\gamma}, \quad I = \frac{\tan \phi_1 \ (5 + 3 \tan^2 \phi \ )}{24 \ \rho\gamma^3} \qquad \text{Al.31, Al.32}$$

and the Latitude can now be found in radians as,

$$\phi = \phi_1 + E_1^2 . \ G + E_1^4 . \ I \qquad \text{Al.33}$$

The Longitude is obtained from the functions

$$F = \frac{1}{\gamma \cos \phi_1} \qquad \text{Al.34}$$

$$H = \frac{1 + 2 \tan^2 \phi_1 + e'^2 \cos^2 \phi_1}{6 \ \gamma^3 \cos \phi_1} \qquad \text{Al.35}$$

$$J = \frac{5 + 28 \tan^2 \phi_1 + 24 \tan^4 \phi_1}{120 \ \gamma^5 \cos \phi_1} \qquad \text{Al.36}$$

and the Longitude is

$$\lambda = \lambda_o + E_1 F \ + E_1^3 H + E_1^5 J \qquad \text{Al.37}$$

where the Longitude at the origin, λ_o, is given by equation Al.23 for Metres and equation Al.24 for Yards conversions. The solution is in radians.

The conversion from Latitude and Longitude (ϕ,λ) into Metres or Yards is accomplished as follows. All of the following functions are calculated,

$$\gamma = \frac{a}{(1 - e^2 \sin^2 \phi)^{1/2}}$$

$$\begin{aligned} f &= \gamma \cos \phi \\ g &= -\tfrac{1}{2} \gamma \sin \phi \cos \phi \\ h &= \tfrac{1}{6} \gamma \cos^3 \phi (1 - \tan^2 \phi + e'^2 \cos^2 \phi) \\ i &= -\tfrac{1}{24} \gamma \ \sin \phi \cos^3 \phi \ (5 - \tan^2 \phi + 9 \ e'^2 \cos^2 \phi) \\ j &= \tfrac{1}{120} \gamma \cos \phi \ (2 \cos 2\phi + 3 \cos 4 \ \phi) \end{aligned} \qquad \text{Al.38}$$

The meridinal distance, m_b, is obtained with equation (Al.22) from the Latitude. The meridinal distance from the grid origin is,

$$m = m_o - m \qquad \text{Al.39}$$

The longitude difference is,

$$\Delta\lambda \ = \lambda - \lambda_o \qquad \text{Al.40}$$

and both λ_o and m_o are chosen according to whether the conversion is to be into Metrics or Yards. Then,

$$\begin{aligned} N_{,} &= m + \Delta\lambda^2 \ g + \Delta\lambda^4 i \\ E_{,} &= \Delta\lambda f \quad \Delta\lambda^3 \ h + \Delta\lambda^5 \ j \end{aligned} \qquad \text{Al.41}$$

and for Metrics

$$\begin{aligned} N &= 0.999901718 \ N_{,} + 700{,}000\text{m} \\ E &= 0.999901718 \ E_{,} + 300{,}000\text{m} \end{aligned} \qquad \text{Al.42}$$

or Yards,

$$N = N_1/0.9144 + 400{,}000 \text{ yds}$$
$$E = E_1/0.9144 + 300{,}000 \text{ yds} \qquad \text{Al.43}$$

The computer program called GRDCON which solved these equations was written with double precision variables which have 16 significant digits to reduce round-off errors. It allows the user to input coordinates at the keyboard or for mapping applications discussed in the next Appendix, it will read data from computer disc storage, make the conversion and write the output back onto the disc in a six-column format (allowing 2 columns for the x and y coordinates in each of the grids). As data is stored in all three coordinate systems, mapping programs can accurately reproduce topography in any of the three projections.

The program was tested for accuracy by converting test coordinates produced by the Department of Lands and Survey computer and supplied by Mr R. Adams. Table Al.2 shows the accuracy with maximum deviations being 0.01m which is no greater than the significance of the input data.

APPENDIX 2 MAPPING

Mapping of information was an integral part of the study, so the establishment of an efficient computer mapping system was essential. A Map Plotting and Digitising Package was written in Fortran IV for the NHB PDP 11/23 computer and the peripheral HP1123C eight-colour plotter. It has the following abilities:

A2.1 Map Digitising

Topographic maps are laid on the plotter, and digitised by following contours, shorelines, or positioning specific features with an optical glass fibre sight manipulated by the operator with controls on the plotter control panel. The computer program, MAPDIG, assesses the coordinates of the optical sight, and produces a disc file for permanent storage, of the coordinates in the coordinate system of the map; usually National Yards or Latitude and Longitude. The program incorporates a rotation-of-the-axes algorithm for data realignment when maps are placed on the plotter at an angle to the base line. Separate scales in the perpendicular x and y directions are maintained to allow for possible map distortions previously incurred during reproduction on copying machines. Map sections can be automatically added to existing files from within the program and, during operation, a data point deletion procedure is offered for correction of digitising errors.

For successful mapping, some pen control from within the file is needed, specifically to ensure that the pen is lifted before moving to isolated features such as islands. A code is maintained throughout the mapping (and also within a separate graph plotting package) which enforces this. Whenever the plotter encounters a -999.99 within the file, it moves to the coordinates of the following data point with the pen up, then continues moving through the file with the pen down. All isolated features must be preceeded by this code and the map digitising program allows the operator to enter it at any stage during the program operation.

A2.2 Joining of sections

Because of the A3 size limitation of the plotter, maps must be digitised in "pages", and some file manipulation is often required to link these pages into a neat record. Program MPJOIN was written to undertake these operations.

A2.3 Coordinate Conversion

The output file from MAPDIG is formatted for direct input into the Coordinate Conversion program GRDCON, referenced in Appendix 1. This creates a new disc file in 6-column format with the coordinates of each data point in all three systems: Mt Eden Circuit Geodetic Metric Grid, North Island National Yards Grid and Latitude and Longitude. From this, the first ever Mt Eden

Circuit Metric Grid maps of a Northland region were constructed.

A2.4 Map Plotting

Program MAPLT, accurately and repetitively reproduces the coordinate files in any of the three grid systems. For maintenance of generality, maps are constructed in sections by sequentially adding data from chosen files stored on the computer disc. Simply, one may include any or all of the available files, including those containing information such as sounding line locations, photography sites, EDM equipment tracking information etc.

Maps of a selected section of the harbour designated by assigning coordinates to the lower left corner of the map can be drawn with choice of scale, physical size up to the limiting 350mm by 250mm plotter size, position on the page which is often useful for insetting, colours and line patterns.

Four distinct types of plot are accommodated and each must be differentiated by a code in the disc file preceeding the data for each type. Normal mapping operations use -999.99, N. for lifting the pen, and N is a number from 0 to 8 which allows the user to change line patterns. Symbols (i.e. squares, circles etc.) can be plotted to mark features like beacons. Symbol files are preceeded by the code -777.77, N. where N is a number from 0 to 8 representing the location of the label, relative to the symbol. Arrow plots are obtained by preceeding the data with the code -666.66 and arrows are drawn at an operator selected scale with the base of the arrow at given coordinates and with an angle previously written onto disc file. Bathymetric maps with depths written so that their base is at the location of the measurement can be constructed and the file type is identified by the preceeding -888.88 code. In all cases the file types can be mixed and a change is accomplished by ending the section with the positive value of the preceeding code.

A2.5 Bathymetric chart digitising and the sounding line package

The digitising program allows entry of depths on a bathymetric chart as well as contours. The digitiser locates the depth which is entered by the operator at the keyboard. The most common application of this was for the comparison of cross-sections.

To be able to produce cross-sections from any of the 3 map projections and reproduce these in a chosen length unit the operator(i) digitises using MAPDIG (ii) converts the coordinates with GRDCON and then (iii)marries the original depths file with any of the 3 pairs of coordinates using MAPSND. This program establishes a file of depths with distance from an origin as well as maintaining the depth coordinates. It is suitable for manipulation by the sounding line reduction package (Black, 1983).

A2.6 Labelling

Place names, general labels, figure captions, numerals etc. are added to the figures by application of Program LETPLT. Labels can be entered from the VT100 terminal keyboard, or previously stored on computer disc if they are long and detailed. The program offers (i) variable character slant (ii) variable label orientation from 0 to 360^{0} (iii)user selected character height in millimetres (iv) colour selection and (v) the necessary ability to place the label anywhere on a page. This program was used extensively, not only for labelling maps, but for generally labelling all diagrams in this thesis.

A2.7 Summary

The mapping package was used with great success throughout this thesis for diagram production, and a particularly pleasing example of the type of maps that can be produced is shown on Figure 4.12. The programming is available - "KPB Package" (Black, 1983b) - from the author.

APPENDIX 3 8-COLUMN FORMAT

No relevant software was initially present on the NHB computer so it was necessary to establish a computerised data reduction system. This meant that a considerable amount of software had to be written, which clearly had its time-consuming disadvantages, but it had the important benefit, of allowing the system to be almost entirely interactive. That is, output files from one program were made suitable for input into a whole series of other programs. This potential for creating an interactive system is usually not present, because of the preceeding generations of programmers who have each independently added a pound of flesh to the system.

The most important implication of this was the early decision to construct as many disc files with the same format as possible. To this end, an 8-column format (structured as 3F8.4,5E11.4) was introduced. It allowed up to eight variables (either related or independent) to be listed in columns. Several general and specific programs were written to read these columns and perform, either independent or comparative plotting and analysis. Two major examples of this which were used consistently throughout the thesis were:

A3.1 Program LINEAR

Which takes an 8-column file and determines general statistical information: mean, standard deviation, maximum and minimum etc. Standard definitions were applied.

Mean

$$\bar{x} = \frac{1}{N} \sum_{i=1}^{N} x_i \qquad \text{A3.1}$$

Standard deviation

$$s = \sqrt{\frac{1}{N-1} \sum_{i=1}^{N} (x_i - \bar{x})^2} = \left[\frac{\sum_{i=1}^{N} x^2 - \left(\sum_{i=1}^{N} x \right)^2 \Big/ N}{N - 1} \right]^{\frac{1}{2}} \qquad \text{A3.2}$$

Coefficient of variation

$$V = (s/\bar{x}) \cdot 100 \qquad \text{A3.3}$$

The program will also plot a histogram of a column of data, sort the data into numerical order then print the result for inspection, or fit a column to the Weibull distribution. The probability of exceedence of the Weibull distribution is given by

$$\frac{n}{N} = e^{-\alpha x^{\beta x}} \qquad \text{where n values exceed x out of a total of N.} \qquad \text{A3.4}$$

The data is sorted into ascending order to find n and x. The best fit is performed by linear regression to the linearised version of (A3.4), given by,

$$\ell n\,[\ell n\,(N/n)] = \ell n\alpha + \beta \ell nx \qquad \text{A3.5}$$

to ascertain the defining cooefficients α,β.

Two columns can also be fitted by linear regression to any of a linear curve, a linear curve through the origin, a power curve or an exponential curve. The linear equation is

$$y = bx + a \qquad \text{A3.6}$$

and the gradient, b is,

$$b = \frac{\Sigma xy - \Sigma x\,\Sigma y/N}{\Sigma x^2 - (\Sigma x)^2/N} \qquad \text{A3.7}$$

and intercept, a is,

$$a = (\Sigma y - b\Sigma x)/N \qquad \text{A3.8}$$

where the sums are over the range i = 1,N.

The correlation coefficient is: $r = b.s_x/s_y$ A3.9

where s_x, s_y are standard deviations of the x and y data respectively.

The power curve, $y = ax^b$ A3.10

is linearised to become, $\ell ny = \ell na + b\,\ell nx$ A3.11

The exponential curve, $y = ae^{bx}$ A3.12

is linearised to become, $\ell ny = \ell na + bx$ A3.13

The program will also plot two chosen columns against one another on the line printer.

A3.2 Program LINPLT

Which produces x-y plots on either two linear or two log axes or one linear and one log axis, on the HP7221C plotter. Any two columns from the 8-column file can be chosen. Options also include a selection of scale on both axes, distances between axis tick marks, axis labels, physical plot sizes up to the maximum on the plotter of 350 by 250mm, a best fit, linear, power or exponential curve on the plot with gradient, intercept and correlation coefficient written across the top and the ability to place the plot anywhere on the page. Within this framework, several curves can be plotted on the same axes with a choice of three types or graph i.e. (1) just symbols plotted at the data points, (2) lines joining symbols or (3) just lines with no symbols. One of 32 symbol types can be selected and a name to identify the curve can be entered at the keyboard for reproduction on the plot.

Further control is enabled by means of coded information within the files. A -999.99 placed within the data file will force the pen to be lifted and moved to the next data point. A -9.9 for the x or y coordinate is a null value and no response is taken by the program except to ignore this point and read the next value on the file to continue normally.

Many of the graphs presented in this thesis were plotted with LINPLT and labels were entered with LETPLT described in the previous Appendix.

A3.3 Program BARPLT

Which will take a column of data and plot it as a histogram with a cumulative distribution curve. Options include pen colour, plot size and position on the page.

APPENDIX 4: CIRCUIT OF FREQUENCY TO VOLTAGE INTERFACE

To enable transfer of digital information from tape recordings of data from the Tasman Mini Data Logger onto the NHB PDP 11/23 computer, a frequency to voltage converter was constructed from the circuit published by Jordan and Schaare (1980). The data is tone encoded by the Logger using frequency shift keying; a 2400 Hz tone represents a logical zero and a 1200Hz tone represents a logical 1. The frequency to voltage unit converts these to high and low voltages of ±5 volts for computer recognition at a standard RS232 computer input interface. The circuit is presented here in Figure A4.1 and does not include a handshake line for remote control of tape recorder operation.

APPENDIX 5: COMPUTER PROGRAM FOR A PDP 11/23 TO ACCEPT TASMAN LOGGER DATA

Specialist programming to over-ride the computer's response to control characters had to be written for input of digital data from the data acquisition system, SUBTLE. Moreover, in the single user environment on the NHB PDP 11/23, data is input at one port of the RS232 interface at 300 Baud in compliance with the tape recorder speed, while program control is maintained at the console terminal. The FORTRAN program to perform these functions is listed here.

```
C-----------------------------------------------------------------
C
C               TAPINP.FOR
C               TASMAN MINI DATA LOGGER TAPE INTERFACE PROGRAM.
C               FOR A DIGITAL PDP 11/23 WITH AN RT-11
C               OPERATING SYSTEM.
C
C       PROGRAMMER : KERRY BLACK
C                    c/o 6 Alison St.,
C                    Portarlington. 3223.
C                    Victoria.
C                    Australia.
C
C-----------------------------------------------------------------
        LOGICAL*1 ESC
        DIMENSION ICHAR(20)
        DIMENSION INFO(4)
        DATA ESC/27/
        TYPE 12, ESC,ESC,ESC,ESC
12      FORMAT(X,A1,'[2J',A1,'[4;3H',A1,
        1 '#3*** TAPE RECORDER INPUT ***'/
        2 3X,A1,'#4*** TAPE RECORDER INPUT ***'//)
        TYPE 10
10      FORMAT(//' THIS PROGRAM ACCEPTS TASMAN MINI LOGGER DATA'/
        1' VIA THE MAGNETIC TAPE INTERFACE'//
        1' PLEASE COMPLETE THE FOLLOWING INSTRUCTIONS'/
        1' (1) SET THE SWITCH ON COMPUTER TO 300 BAUD'/
        3' (2) CONNECT THE INTERFACE TO THE TERMINAL COMPUTER LINE'/
        4'     USED FOR THE PLOTTER. '/
        5' (3) POSITION TAPE AT START OF RECORD'/
        2' (4) THEN PRESS  RETURN  ON THIS TERMINAL'//)
        ACCEPT 20, ICH
20      FORMAT(A1)
        CALL OPENNF(1)
        ILUN=3
        J=MTATCH(ILUN,)
        I=MTGET(ILUN,INFO)
        INFO(2)=128
        INFO(1)=194+4096
        I=MTSET(ILUN,INFO)
        TYPE 30
30      FORMAT(//' NOW START THE TAPE RECORDER'//)
C
        IOLD=0
        M=0
        K=0
61      L=0
60      L=L+1
        ICNT=0
77      I=MTIN(ILUN,ICHAR(L))
        IF(I.EQ.0) GO TO 78
        IF(I.EQ.1.AND.M.EQ.0) GO TO 77
        ICNT=ICNT+1
        IF(ICNT.LT.1000) GO TO 77
82      WRITE(1,59) (ICHAR(I),I=1,L)
        STOP ' END OF DATA TRANSMISSION'
C
78      IF(K.EQ.0.AND.ICHAR(L).NE.0) GO TO 61
        M=1
        INEW=ICHAR(L)
        IF(INEW.EQ.IOLD) GO TO 80
        IOLD=INEW
        K=1
        GO TO 81
80      K=K+1
        IF(K.EQ.10.AND.INEW.EQ.72) GO TO 900
C
81      IF(L.LT.20) GO TO 60
        WRITE(1,59) (ICHAR(I),I=1,L)
        TYPE 59, (ICHAR(I),I=1,L)
        L=0
        GO TO 60
C
59      FORMAT(X,20I4)
900     WRITE(1,59) (ICHAR(I),I=1,L)
        STOP 'PROGRAM ENCOUNTERED 72 TEN CONSECUTIVE TIMES'
        END
C
        SUBROUTINE OPENNF(LUN)
        DIMENSION INAM(10)
        INAM(10)=0
        TYPE 10
10      FORMAT(/' NAME OF NEW FILE TO BE CREATED (OUTPUT FILE)'/
        1' LESS THAN 19 CHARACTERS'/
        2' (THE LOGGER MEMORY WILL BE TRANSCRIBED USING 20I4 FORMAT)')
        ACCEPT 20, (INAM(I),I=1,9)
20      FORMAT(9A2)
        OPEN(UNIT=LUN,NAME=INAM,TYPE='NEW',CARRIAGECONTROL='FORTRAN')
        RETURN
        END
```

APPENDIX 6 SEDIMENT TRAP ANGLE OF ATTACK AND EFFECTIVE WIDTH

When a non-symmetrical sediment trap is placed in currents with variable flow directions, the effective width of the trap responds to the angle of attack of the water flow. Furthermore, to compare relative catches in differently-shaped traps (e.g. embedded circular or rectangular), the weights must be normalised by the effective trap width to obtain the mass moving per unit bed width. This appendix derives the effective widths of traps for application to sediment trapping experiments in the inter-tidal zone at the proposed port site discussed in Chapter 8.

A6.1 Derivations

Considering Figure A6.1, for one wall of a rectangular trap of length, b, and attacked by a current approaching with angle, ψ, defined clockwise from the wall normal, the effective trap width is,

$$L = b. \sin \psi \qquad \text{A6.1}$$

from the right triangle ODC.

Under normal field conditions, no information about the angle ψ, is available and the effective width cannot be found without an additional assumption. However, if all directions of approach are equally likely, the mean effective width, $\bar{L}$, may be found by integrating over any quadrant as,

$$\bar{L} = \frac{2}{\pi} \int_0^{\pi/2} b \sin \psi \, d\psi \qquad \text{A6.2}$$

and thus,

$$\bar{L} = \frac{2b}{\pi} \qquad \text{A6.3}$$

which is a function of the side length only.

VUV traps are enclosed on 3 of 4 sides, and have an open mouth of 0.26m width and 0.12m height. In low current speeds the water velocities are not sufficient to align the trap with flow directional oscillations and it remains in place throughout the tidal cycle. Therefore, the effective width correction should be applied.

Sediment can only enter the VUV trap from two of the four quadrants, and if all sediment transport directions are equally as likely, the trapped mass must equal 50% of the actual mass of sand moved during the tidal cycle. The effective width which scales the masses for total rates over the trapping period is thus,

$$\bar{L} = \frac{1}{2} \left(\frac{2b}{\pi}\right) = \frac{b}{\pi} \qquad \text{A6.4}$$

and $\bar{L}$ = 0.083m for b = 0.26m.

The sediment transport rate (kg/m/tidal cycle) is then,

$$Q = \frac{\text{mass caught}}{0.083 \text{ (no. of tidal cycles)}} \qquad \text{A6.5}$$

(Further to the correction for effective width, the VUV trapped weights were scaled in Chapter 8 to compensate for the 70% trap efficiency reported in Chapter 7).

For an embedded rectangular receptable with sides of length a and b shown in Figure A6.1, the average effective width is,

$$\bar{L} = \frac{1}{4}\left[\frac{a}{\pi} + \frac{a}{\pi} + \frac{b}{\pi} + \frac{b}{\pi}\right] = \frac{1}{2}\left(\frac{a + b}{\pi}\right) \qquad \text{A6.6}$$

In this case, each side is scaled to catch 100% of the total sediment being transported and 4 sides will then collect 400%. The effective width for this receptable is thus,

$$\bar{L} = 2\left(\frac{a + b}{\pi}\right) \qquad \text{A6.7}$$

For a = b, the square receptacle catches 4 times the sand of the one-sided VUV trap.

A circular trap with four, isolated compartments of equal volume and surface area, as shown on Figure A6.2 will have overall effective width equal to the diameter, 2r. However, if all directions are equally favoured, each compartment should necessarily contain equal quantities of sediment and will thus be catching at one quarter of the actual sediment transport rate then.

$$\bar{L} = \frac{2r}{4} \qquad \text{A6.8}$$

and $\bar{L}$ = 0.06m for a diameter of 0.24m.

As a brief check if the circular trap is approximated by a rectangular receptacle ABCD with sides of length 0.17m (Figure A6.2), then for each side (equation A6.4),

$$\bar{L} = \frac{0.17}{\pi} = 0.054\text{m} \qquad \text{A6.9}$$

which approximately agrees with the previous determination of 0.06m and differs due to the poor approximation to the circle.

SEDIMENT TRANSPORT AND TIDAL INLET HYDRAULICS

REFERENCES

REFERENCES

ABBOTT, M.B. (1979). *Computational Hydraulics. Elements of free surface flows.* Pitman Pub. 326pp.

ABBOTT, M.B., DAMSGAARD, A. and RODENHUIS, G.S. (1973). System 21, Jupiter, a design system for two-dimensional nearly horizontal flows. *Journal of Hydraulic Research.* 11:1-28.

ALAM, M.Z. and KENNEDY, J.F. (1969). Friction factors for flat bed flows in sand-bed channels. *Journal of the Hydraulic Div.* (ASCE). 95 (HY4).

ALLEN, J.R.L. (1968). *Current Ripples (Their relation to patterns of water and sediment motion).* North-Holland. Amsterdam. 443pp.

ALLEN, J.R.L. (1980). Sand waves: A model of origin and internal structure. *Sedimentary Geology.* 26(4):281-328.

ALLEN, J.R.L. (1982). *Sedimentary structures: A character and physical basis.* Vol 1. Elsevier Pub. 594pp.

ALLEN, J.R.L. and FRIEND, P.F. (1976). Relaxation time of dunes in decelerating aqueous flows. *Journal of the Geological Society,* Vol. 132, Part 1 : 17-26.

ANWAR, H.O. and ATKINS, R. (1980). Turbulence measurements in simulated tidal flow. *Journal of the Hydraulics Division.* (ASCE). 106 (HY8): 1273-1289.

BAGNOLD, R.A. (1963). Mechanics of marine sedimentation. In : M.N. Hill (Ed). *The Sea, 3.* Wiley-Interscience, New York. pp.507-582.

BARWIS, J.H., PERRY, F.C., and La GARDE, V.E. (1977). Computer-aided photo studies of Inlet Stability. *Coastal Sediments '77. 5th Symposium.* ASCE. pp.1057-1070.

BENQUÉ, J.P., CUNGE, J.A., FEUILLET, J., HAUGUEL, A., HOLLY, F.M. (1982). New method for tidal current computation. *Journal of the waterway.* (ASCE) WW3 : 396-417.

BLACK, K.P. (1975). *Rip currents.* B.Sc. (Hons) thesis. Univ. of Melbourne. 187pp.

BLACK, K.P. (1977). *User's guide to programs written for J.K.K. Look Laboratory wave transformation project.* University of Hawaii. Misc. Rep. No. 17. U Hawaii - Look Lab-77-17. 224pp.

BLACK, K.P. (1978a). Simple procedures to apply the Weibull distribution to ocean wave data. *LOOK LAB/HAWAII.* Vol. 8. No. 1. p.1-17.

BLACK, K.P. (1978b). *Wave transformation over a coral reef.* M.S.C. thesis. Univ. of Melbourne. 240pp.

BLACK, K.P. and HEALY, T.R. (1981). *Computer programs for wave analysis, wind wave generation, wave refraction diagrams, and fast Fourier analysis.* Occasional Rep. No. 6, Dept of Earth Sciences, Uni. of Waikato. 76pp.

BLACK, K., HEALY, T., VENUS, G., STOAKES, J., and HUNTER, M. (1981). Marsden Point Forestry Port Investigation. *Sea Floor Photographic Survey.* Northland Harbour Board 149pp.

BLACK, K.P. (1982). Positioning the quay wall. Northland Harbour Board Internal Report. 10pp.

BLACK, K.P. and HEALY, T.R. (1982). Sediment transport investigations in a New Zealand tidal inlet. *Proc. 18th Int. Conf. on Coastal Eng.* ASCE. Cape Town.

BLACK, K.P., BEAZLEY, M.J., and HUNTER, M., (1982). *Channel Drogue Tracking.* Marsden Point Forestry Port Investigation. Northland Harbour Board. 73pp.

BLACK, K.P. (1983). *Computerised sounding reduction.* Northland Harbour Board. 25pp.

BLACK, K.P. (1983b). THE KPB PACKAGE. Hydrodynamic data reduction for application to ocean engineering. K.P. Black, C/- 6 Alison St., Portarlington, Victoria, Australia.

BIORESEARCHES (1976). Aspects of the ecology of the areas surrounding the oil refinery at Marsden Point. 190pp.

BIORESEARCHES (1976b). Surface water currents in the Marsden Point region, a report for the N.Z. Refining Company. 7pp.

BIORESEARCHES (1978). The monitoring of marine habitats in the vicinity of Marsden Point oil refinery. Report No. 1. 47pp.

BIORESEARCHES (1979). Ibid Report No. 2. 50pp.

BIORESEARCHES (1980). The ecological impact of the proposed forestry wharf at Marsden Point, Northland. 79pp.

BOOTHROYD, J.C. (1978). Mesotidal Inlets and Estuaries. Chapter 6 in R.A. Davies (ed): *Coastal Sedimentary Environments.* Springer-Verlag. New York. pp.287-360.

BOOTHROYD, J.C. and HUBBARD, D.K. (1975). Genesis of Bedforms in mesotidal estuaries. In Cronin, L.E. (ed): *Estuarine Research, Vol. 2. Geology and Engineering.* Academic Press. New York. pp.217-234.

BOWDEN, K.F. and FAIRBAIRN, L.A. (1956). Measurements of turbulent fluctuations and Reynolds stresses in a tidal current. *Proc. Roy. Soc. of Lond.* 237:422-438.

BOWDEN, K.F. and PROUDMAN, J. (1949). Observations of the turbulent fluctuations of a tidal current. *Proc. Roy. Soc.* A199:311-327.

BRETSCHNEIDER, C.L. (1958). Revisions in wave forecasting: Deep and shallow water. *Proc. 6th Conf. Coastal Eng.* (ASCE).

BRETSCHNEIDER, C.L. (1959). Wave variability and wave spectra for wind generated gravity waves. *U.S. Army Corp. of Engineers.* Tech. Memo 118.

BRUUN, P. (1966). *Tidal inlets and littoral drift.* Oslo Pub.

DAHM, J. (1983). *The geomorphic development, bathymetric stability and sediment dynamics of Tauranga Harbour.* M.Sc. thesis. University of Waikato. 230pp.

DAILEY, J.W. and HARLEMAN, D.R.F. (1966). *Fluid Dynamics.* Addison-Wesley, Reading, Mass., 454pp.

DAVIES-COLLEY, R.J. and HEALY, T.R. (1978). Sediment transport near the Tauranga Entrance to Tauranga Harbour. *N.Z. Journal of Marine and Freshwater Research.* 12(3):237-43.

DE ALTERIS, J.T. and BYRNE, R.J. (1975). The recent history of Wachapreague Inlet, Virginia. In Cronin, L.E. (ed). *Estuarine Research.* Vol. 2: 167-182. Academic Press. New York.

DE BOER, P.L. (1981). Mechanical effects of micro-organisms on inter-tidal bedform migration. *Sedimentology.* 28:129-132.

DE LANGE, W.P. and KAMP, P.J.J. (1980). Computer programs for sediment data analysis and empirical determinations of littoral drift. *Occasional Rep. 5.* Earth Sciences. Univ. of Waikato.

DHI (1982). *Proposed forestry terminal hydraulic model studies.* Danish Hydraulic Institute for the Northland Harbour Board. 132pp.

D'OLIER, B. (1979). Side-scan sonar and reflection seismic profiling. Chapter 4 in R.K. Dyer (ed): *Estuarine Hydrography and Sedimentation.* Cambridge Univ. Press. pp.57-70.

DRONKERS, J.J. (1969). Tidal computations for rivers coastal areas and seas. *Journal of the Hydraulic Div.* (ASCE). HY1 (6341): 29-77.

DYER, K.R. (1970). Current velocity profiles in a tidal channel. *Geophysical Journal of the Royal Astronomical Society.* 22: 153-161.

DYER, K.R. (1973). *Estuaries: a physical introduction.* John Wiley and Sons. 140pp.

DYER, K.R. (1980). Velocity profiles over a rippled bed and the threshold of the movement of sand. *Estuarine and Coastal Marine Science,* 10: 181-199.

EINSTEIN, H.A. (1950). The bed load function for sediment transportation in open channels. *U.S. Dept of Agriculture, Soil Conservation Serv. Tech Bull.* 1026.

EINSTEIN, H.A. and BARBAROSSA, N.L. (1952). River channel roughness. *Transactions ASCE.* 117:1121-1146.

EINSTEIN, H.A. and CHIEN, N. (1955). Effects of heavy sediment concentration near the bed on the velocity and sediment distribution. University of California Institute of Engineering Research and the U.S. Army Corps of Engineers. Missouri River Division, *MRD Sediment Series 8.* 76pp.

E.I.R. (1981). Environmental Impact Report for the Northland Harbour Board with supplementary appendices. Northland Forestry Port Marsden Point.

ENGEL, P. and LAU, Y.L. (1980). Computation of Bed Load Using Bathymetric Data. *Journal of the Hydraulics Div.* (ASCE). 106 (HY3): 369-380.

ENGEL, P. and LAU, Y.L. (1981). Bed load discharge coefficient. *Journal of the Hydraulic Div.* (ASCE). (HY11): 1445-1454.

ENGELUND, F. and HANSEN, E. (1967). A monograph on sediment transport in alluvial streams. *Technisk Vorlag,* Copenhagen. 62pp.

FALCONER, R.A. (1980). Numberical modelling of tidal circulation in harbours. *Journal of the waterway port coastal and ocean division.* (ASCE) WW1:31-48.

FERENTINOS, G. and COLLINS, M. (1980). Effects of shoreline irregularities on a rectilinear tidal current and their significance in sedimentation processes. *Journal of sedimentary petrology.* Vol. 50, No. 4, 1081-1094.

FERRAR, H.T. (1925). The geology of the Whangarei Bay of Islands Subdivision. *N.Z. G.S. Bulletin* 27.

FISCHER, G. (1965). On a finite-difference scheme for solving the non-linear primitive equations for a baratropic fluid with applications to the boundary current problem. *Tellus,* Vol. 17, No. 4.

FITZGERALD, D.M. and FITZGERALD, S.A. (1977). Factors influencing tidal inlet throat geometry. *Coastal Sediments '77. 5th Symposium of the Waterway, Port, Coastal and Ocean Div.* (ASCE). 563-581.

FOLK, R.L. (1968). *Petrology of sedimentary rocks.* Hemphills, Austin. 170pp.

FREDSOE, J. (1978). Sedimentation of river navigation channels. *Journal of the Hydraulics Div.* (ASCE). 104(HY2): 223-236.

FREUND, J.E. (1974). *Modern elementary statistics.* Fourth edition. Prentice/Hall International Editions. 532pp.

FUKUOKA, S. (1978). Interaction between turbulent fluid and suspended sediments. Chapter 9 in H.W. Shen and H. Kikkawa (ed.) *Application of Stochastic Processes in Sediment Transport.* Water Resources Publications. Col. USA.

GADD, P.E., LAVELLE, J.W., and SWIFT, D.J.P. (1978). Estimates of sand transport on the New York shelf using near-bottom current-meter observations. *Journal of Sedimentary Petrology,* 48: 239-252.

GODA, Y. (1970). Numerical experiments on wave statistics with spectral simulation. *Report of the Port and Harbour Res. Inst.* Vol. 9, No. 3. pp.3-57.

GORDON, C.M. (1975). Sediment entrainment and suspension in a turbulent tidal flow. *Marine Geology,* 18:M57-M64.

GRACE, S. (1936). Friction in the tidal currents of the Bristol Channel. *Geoph. Suppl. Man. Nat. R. Astr. Soc.* V. 4.

GRAF, W.H. (1971). *Hydraulics of sediment transport.* McGraw-Hill Pub. 513pp.

GRASS, A.J. (1970). Initial instability of fine bed sand. *Journal of the Hydraulics Division (ASCE).* HY3: 619-631.

GUY, H.P., SIMONS, D.B. and RICHARDSON, E.V. (1966). Summary of alluvial channel data from flume experiments 1955-1961. *U.S. Geol. Surv. Prof. Pap.* 4621 .92pp.

HAMA, F.R. (1953). Discussion: The frictional resistance of flat plates in zero pressure gradient, L. Landweber. *Society of Naval Architecture and Marine Engineering.* 61: 27-28.

HAMA, F.R. (1954). Boundary-layer characteristics for smooth and rough surfaces. *Soc. Naval Architects and Marine Engineers.* Vol. 62: 333-358.

HANSEN, W. (1956). Theorie zur Errechnung des Wasserstandes und der strömungen in randmeeren nebst anwendungen. *Tellus,* Vol. 8, No. 3.

HARMS, J.C., SOUTHARD, J.B., SPEARING, D.R. and WALKER, R.G. (1975). Depositional environments as interpreted from primary sedimentary structures and stratification sequences. *Lecture Notes, SEPM Short Course* No. 2. Dallas, Texas. 161pp.

HASSELMAN, K. and 15 co-authors. (1973). Measurements of wind wave growth and swell decay during the Joint North Sea Wave Project (JONSWAP). *German Hydrographic Inst.* Hamburg. 95pp.

HAYES, M.O. and KANA, T.W. (1976). Terrigenous clastic depositional environments. *Tech. Rep. No. 11-CRD*, Dept of Geology. Univ. of Sth Carolina 302pp.

HEALY, T.R. (1981). Sediments and hydraulics in the vicinity of the proposed timber port at Marsden Point. At contribution to the Environmental Impact Report for the Northland Harbour Board. Appendix 3. Northland Forestry Port. Marsden Point.

HEALY, T.R. (1981b). Sedimentation aspects In *Assessment of the sedimentation, hydrological, and biological impacts of the proposed temporary offloading ramp at Marsden Point.* Bioresearches Report for Badger-Chiyoda Ltd. Auckland. p.1-42.

HEALY, T.R. (1983). Assessment of stability and natural hazard risk of the Whakatane Spit and inlet in relation to proposed marina development and Environmental Impact assessment of the sediment and hydrological aspects of the marina development. Coastal Investigations report for Whakatane District Council. 73pp.

HEAPS, N.S. (1969). A two-dimensional numerical sea model. *Proc. Roy. Soc.* A 365,93.

HEATH, R.A. (1975). Stability of some New Zealand coastal inlets. *N.Z. Journal of Marine and Fisheries Research (JMFR)* 9:449-457.

HEATH, R.A. (1979). Edge waves on the New Zealand East Coast. *Marine Geodesy.* 2(4):337-350.

HEATHERSHAW, A.D. (1981). Comparisons of measured and predicted sediment transport rates in tidal currents. *Marine Geology*, 42:75-104.

HEATHERSHAW, A.D. and SIMPSON, J.H. (1978). The sampling variability of the Reynolds stress and its relation to boundary shear stress and drag coefficient measurements. *Estuarine and Coastal Marine Science*, 6:263-274.

HINZE, J.O. (1959). *Turbulence, an introduction to its mechanism and theory*, McGraw-Hill, New York. 586pp.

HUME, T.M. (1979). Changes in Ocean Beach and Bream Bay. Beach and nearshore profiles between August 1976 and July 1978. *N.Z. Ministry of Works and Development.*

JOHNSON, J.W. (1973). Characteristics and behaviour of Pacific Coast tidal inlets. *Journal of the Waterway.* (ASCE). WW3:325-339.

JORDON, R.B. and SCHAARE, P.N. (1980). Tasman-Apple Interface Users Manual. Ruakura Agricultural Research Centre, Ministry of Agriculture and Fisheries Interim Report.

KING, C.A.M. (1972). *Beaches and coasts.* 2nd ed. St Martin's Press. New York. 570pp.

KISHI and KUROKI (1967). In Shen H.W. (ed): *Sedimentation.* Symposium to honour H.A. Einstein, Colorado State University.

KNIGHT, R.J. (1977). *Sediments, bedforms and hydraulics of a macrotidal environment, Cobequid Bay (Bay of Fundy) Nova Scotia.* Thesis, McMaster University, Hamilton, Ont. 693pp.

KNIGHT, D.W. (1981). Some field measurements concerned with the behaviour of resistance coefficients in a tidal channel. *Estuarine, Coastal and Shelf Science.* 12:303-322.

KOMAR, P.D. (1976). *Beach processes and sedimentation.* Prentice-Hall. New Jersey. 429pp.

LAURSEN, E.M. (1958). The total sediment load of streams. *Journal of the Hydraulic Div.* (ASCE). 54(HY1):1-36.

LAX, P. and WENDROFF, B. (1960). Systems of Conservation Laws. *Comm. Pure Appl. Math.*, 13:217-237.

LEE, T.T. and BLACK, K.P. (1978). The energy spectra of surf waves on a coral reef. *Proc. of 16th Int. Conf. on Coastal Engineering.* (ASCE). pp.588-608.

LEENDERTSE, J.J. (1967). Apsects of a computational model for long water wave propogation. *Rand Memorandum.* RH-5299-PR, Santa Monica, California.

LETTAU, H. (1969). Note on aerodynamic roughness-parameter estimation on the basis of roughness-element description. *Journal of Applied Meteorology.* 8:828-832.

LIU, P.C. (1971). Normalised and equilibrium spectra of wind waves in Lake Michigan. *Journal of Physical Oceanography,* 1:249-257.

LONGUET-HIGGINS, M.S. (1953). Mass transport in water waves. *Phil. Trans. Roy. Soc. (London).* Series A, 903:535-581.

LUDWICK, J.C. (1974). Tidal currents and zig-zag sand shoals in a wide estuary entrance. *Geol. Soc. of Am. Bull.* 85:717-726.

LUDWICK, J.C. (1975). Variations in the boundary-drag coefficient in the tidal entrance to Chesapeake Bay, Virginia. *Marine Geology,* Vol. 19: 19-28.

MATHEWS, R.S. (1982). *Physical and chemical characteristics of the Waitara Estuary and Coastal embayment with particular reference to effluent disposal.* M.Sc. thesis. Univ. of Waikato.

MCCAVE, I.N. (1973). Some boundary-layer characteristics of tidal currents bearing sand in suspension. *Mem. Soc. R. Sci. 6th Series. Liege*, 6:107-126.

MCCAVE, I.N. and LANGHORNE, D.N. (1982). Sand waves and sediment transport around the end of a tidal sand bank. *Sedimentology*. 29:95-110.

MCDOWELL, D.M. and O'CONNOR, B.A. (1977). *Hydraulic behaviour of estuaries*. Macmillan Press. London. 292pp.

MCLENNAN, N.R. (1982). *Fitzroy Beach processes and quantitative estimation of littoral drift*. M.Sc. thesis. Univ. of Waikato.

MEHTA, A.J. and CHRISTENSEN, B.A. (1976). Incipient sediment motion in entrances with shell beds. *Rivers 76 - Symposium on Inland Waterways for Navigation, flood control and water diversions*, Vol. II, Fort Collins Col. pp.960-977.

MEHTA, A.J., LEE, J. and CHRISTENSEN, B.A. (1980). Fall velocity of shells as coastal sediment. *Journal of the Hydraulics Division*. (ASCE). HY11:1727-1744.

MEYER-PETER, E. and MULLER, R. (1948). Formulas for Bed-load Transport. *Proc. 3rd Meet. Int. Ass. Hydr. Res.* Stockholm pp.39-64.

MILLAR, A.S. (1980). *Hydrology and surficial sediments of Whangarei Harbour*. M.Sc. Thesis, Univ. of Waikato, 212pp.

MILLER, K.R. (1981). *Surficial sediments and sediment transport in Poverty Bay*. M.Sc. thesis. University of Waikato.

MILLER, M.C., MCCAVE, I.N. and KOMAR, P.D. (1977). Threshold of sediment motion under unidirectional currents. *Sedimentology*. 24:507-527.

MORANG, A. and MCMASTER, R.L. (1980). Nearshore bedform patterns along Rhode Island from side-scan sonar surveys. *Jour. of Sedimentary Petrology*. Vol. 50, 3:831-840.

NELSON, C.S. and HEALY, T.R. (1982). Side-scan sonar surveys of the sea floor of Northern Poverty Bay in the vicinity of the Port of Gisborne. *Occasional Report No. 9*. Earth Sciences, University of Waikato.

NODA, E.K. (1972). Wave induced circulation and longshore current patterns in the coastal zone. *Tetra Tech*. Rep. No. TC - 149-3. Calif. 120pp.

O'BRIEN, M.P. (1931). Estuary tidal prism related to entrance areas. *Civil Engineering*. (ASCE) Vol. 1. No. 8:738-739.

O'BRIEN, M.P. (1967). Equilibrium flow areas of tidal inlets on sandy coasts. *Proc. 10th Conf. on Coastal Engineering*. (ASCE). pp.676-686.

PEEK, S.M. (1979). *A comparative study of beach morphodynamics on the east and west coasts of the North Island, New Zealand.* M.A. Thesis (Geography). University of Auckland. 112pp.

PILLAI, C.R.S. (1979). Effective depth in channels having bed undulations. *Journal of the Hydraulics Division.* (ASCE). 105 (HY1):67-81.

RAJU, R.G.R., GARDE, J.J., and BHARDWAJ, R.C. (1981). Total load transport in alluvial channels. *Journal of the Hydraulics Div.* (ASCE). 107 (HY2):179-191.

RAMMING, H.-G. (1971). Hydrodynamical-Numerical investigations and horizontal dispersion of Seston in the Elbe River. *Memoires de la Societé Royale des Sciences de Liege.* TOME V1.

RAMMING, H.-G. and KOWALIK Z. (1980). *Numerical modelling of marine hydrodynamics. Application to dynamic physical processes.* Elsevier Scientific Pub. Co. 368pp.

RAUDKIVI, A.J. (1963). Study of sediment ripple formation. *Journal of the Hydraulic Div.* (ASCE). HY6:15-33.

RAUDKIVI, A.J. (1976). *Loose boundary hydraulics.* Second edition. Pergamon Press. 397pp.

REINSON, G.E. (1979). Longitudinal and transverse bedforms on a large tidal delta, Gulf of St. Lawrence, Canada. *Marine Geology,* 31:279-296.

ROGERS, N.C. (1959). *Hydraulic Model Study,* Northland Harbour Board.

ROUSE, H. (1937). Modern conception of the mechanisms of turbulence. *Trans. A.S.C.E.,* 102:463-505.

RUSSELL, R.C.H. and OSORIO, J.D.C. (1958). An experimental investigation of drift profiles in a closed channel. *Proc. 6th Conf. Coast. Eng.* (ASCE), pp.171-183.

SHORE PROTECTION MANUAL, (1977). *U.S. Army Corps of Engineers.* Coastal Engineering Research Centre.

SIMONS, D.B. and SENTURK, F. (1977). *Sediment transport technology.* Water Resources Publications. Fort Collins. 807pp.

SMITH, J.D. and McLEAN, S.R. (1977). Spatially averaged flow over a wavy surface. *Journal of Geophysical Research,* 82:1735-1746.

SMITH-MCNAUGHT, E.T. (1976). *Whangarei Harbour dredging study.* Northland Harbour Board. 96pp.

SNYDER, R.L. SIDJABAT, M. and FILLOUX, J.H. (1979). A study of tides, setup and bottom friction in a shallow semi-enclosed basin. Part II: Tidal model and comparison with data. *Journal of Physical Oceanography.* Vol. 9: 170-188.

SOULSBY, R.L. and DYER, K.R. (1981). The form of near-bed velocity profile in a tidally accelerating flow. *Journal of Geophysical Research.* 86: 8067-8074.

STERNBERG, R.W. (1968). Friction factors in tidal channels with differing bed roughness. *Marine Geology,* 6: 243-260.

STERNBERG, R.W. (1971). Measurements of incipient motion of sediment particles in the marine environment. *Marine Geology,* 10:113-119.

STERNBERG, R.W. (1972). Predicting initial motion and bedload transport of sediment particles in the shallow marine environment. In D.J.P. Swift, D.B. Duane and O.H. Pilkey (eds): *Shelf Sediment Transport.* Dowden, Hutchinson and Ross. pp.61-82.

STERNBERG, R.W. (1979). Bottom-current measurements and circulation in Western Port, Victoria. *Marine Geology,* 30: 65-83.

TEE, K.T. (1976). Tide-induced residual current, a 2-D nonlinear numerical tidal model. *Journal of Marine Research.* 34:603-628.

THOMAS, G.B. (1960). *Calculus and analytic geometry.* Addison-Wesley Pub. World Student Series.

TONKIN and TAYLOR, (1979a). N.Z. Electricity Report on Foreshore Erosion. Marsden Power Stations,Bream Bay.

TONKIN and TAYLOR,(1979b). *Phase 1 Report.* N.Z. Electricity Marsden Power Stations Coastal Erosion Investigations.

TONKIN and TAYLOR,(1980). *Foreshore protection trial.* N.Z. Electricity Marsden Power Stations Coastal Erosion Investigation.

TONKIN and TAYLOR, (1981). *Wave studies.* N.Z. Electricity Marsden Power Stations Coastal Erosion Investigations.

VANONI, V.A. (1975). *Sedimentation Engineering.* ASCE Task Committee. Manual on Sedimentation. Hydraulic Division. New York. 745pp.

VANONI, V.A. and HWANG, L.S. (1967). Relation between bedforms and friction in streams. *Journal of the Hydraulics Div.* (ASCE). 93 (HY3): 121-144.

VITTAL, N., RANGA, R.K.G., and GARDE, R.J. (1973). Sediment transport relations using concept of effective shear stress. *International Assoc. for Hydraulic Research. International Symposium on River Mechanics,* 1:489-499.

WILLET, R.N. (1982). *Erosion-accretion and sedimentology of Aotea Harbour, with reference to the erosion of Pourewa Point.* M.Sc. thesis. University of Waikato.

WILLIAMS, B.J. and HINWOOD, J.B. (1976). Two-dimensional mathematical water quality model. *Journal of the Environmental Engineering Div.* (ASCE). 102 (EE1):149-163.

YALIN, M.S. (1963). An expression for bed-load transportation. *Journal of the Hydraulics Div.* (ASCE). 89(HY3):221-250.

YALIN, M.S. (1977). *Mechanics of sediment transport.* Second edition. Pergamon Press.

YALIN, M.S. and KARAHAN, E. (1979). Steepness of sedimentary dunes. *Journal of the hydraulics div.* (ASCE). 105(HY4):381-392.