Moveable Bed Physical Modelling of Ebb Delta Growth Characteristics

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Glossary
In any discussion on coastal engineering an agreement of the meaning of terms is necessary. Provided here are definitions of terms used in this report. Although the terms came from many sources, the Shore Protection Manual was of particular value.

AHD - Australian Height Datum

Artificial Bypassing - A mechanical action where sediment is transport past a tidal inlet prior to it infilling the channel.

Bar - A submerged or emerged embankment of sand, gravel, or other unconsolidated material built on the sea floor in shallow water by waves and currents.

Channel - A natural or artificial waterway of perceptible extent which either periodically or continuously contains moving water, or which forms a connecting link between two bodies of water.

CMBPM - Coastal Movable Bed Physical Model.

Coastline - The line that forms the boundary between the land and the water.

Cross-shore transport - The transport of sediment onshore and offshore via current.

Current - A flow of water.

Current, Ebb - The tidal current away from shore or down a tidal stream. Usually associated with the decrease in the height of the tide.

Current, Flood - The tidal current toward shore or up a tidal stream. Usually associated in the increase in the height of the tide.

Current, Tidal - The alternating horizontal movement of water associated with the rise and fall of the tide caused by the astronomical tide producing forces.

Dimensional analysis - Using all the known physical variables in a system in conjunction with known relationships to derive

Downdrift - The directing of predominant movement of littoral materials.

Ebb Delta - A mass of sand that accumulates on the ocean side of a tidal inlet as a result of ebb tidal flow.

EPA - Environmental Protection Agency

Equilibrium Beach Profile - Defined by Dean(1977), \[ z = Ax^{2/3}, \] \( z \) = depth below MWL, \( x \) = distance offshore.

Fall Velocity - The speed that a sediment particle falls in a fluid

Flood Delta - A mass of sand that accumulates on the protected side of a tidal inlet as a result of flood tidal flow.

GCCM - Griffith Centre for Coastal Management

Geomorphology - That branch of both physiography and geology which deals with the form of the earth, the general configuration of it's surface, and the changes that take place in the evolution of landform.

High-energy wave event - Significant waves over 2.5 metres measured continuously over several wave records.

Inlet - A short, narrow waterway connecting a bay, lagoon, or similar body of water with a large parent body of water.

Inlet gorge - Generally the deepest region of the inlet channel.

Jonswap Spectrum Waves - A random train based on a wave energy spectrum defined from wave measurements in the North Sea.

Laboratory effect - Errors in modelling associated with model configuration and equipment accuracy.
Littoral drift - The sedimentary material moved in the littoral zone under the influence of waves and currents.

Littoral Transport - movement of littoral drift in the nearshore zone by waves and currents. Movement can be perpendicular (onshore-offshore) or parallel (longshore) to the shore.

Littoral zone - zone of water extending from the shoreline to just beyond the seaward most breakers.

Longshore transport - littoral transport in the direction parallel to the shore.

Longshore transport - The transport of sediment alongshore via the action of wave generated currents.

Monochromatic Waves - Constant train of waves of the same height and period.

Nearshore zone - region where the forces of the sea react against the land.

Offshore - The comparatively flat zone of variable width, extending from the breaker zone to the seaward edge of the continental shelf. The direction seaward from the shore.

P vs A - The relationship between the tidal prism and the minimum throat cross-sectional area of a tidal inlet.

Prototype - The real world structure/process that is under investigation. In the case of this study a tidal inlet like the Tweed River Entrance.

R.L. - Reduced Level

Sand bypassing - Hydraulic or mechanical movement of sand from the accreting updrift side to the eroding downdrift side of an inlet. Natural or man made.

Scale effect - Errors in modelling because prototype physical properties cannot be reproduced at model scale.

Scour - Removal of underwater material by waves and currents, especially at the base or toe of a shore structure.

Seas - Waves caused by wind at the place and time of observation.

Setup, Wave - Superelevation of the water surface over normal surge elevation due to onshore mass transport of the water by wave action alone.

Shoal: Verb - To become shallow gradually.

Shoal or Delta: Noun - A detached elevation of the sea bottom, comprised of any material except rock or coral, which may endanger surface navigation.

Significant wave height - Average of the highest one-third waves in the record.

Spit - A small point of land or a narrow shoal projecting into a body of water from the shore.

Surge - The name applied to wave motion with a period intermediate between that of the ordinary wind wave and that of the tide.

Swash - The rush of water up onto the beach face following the breaking of a wave.

Swash channel - A secondary channel passing through or shoreward of an inlet or river bar.

Swell - Wind generated waves that have travelled out of their generating area. Swell characteristically exhibits a more regular and longer period and has flatter crests than waves within their fetch.

Tidal Inlet - A location where an embayment or river joins the open ocean.

Tidal inlet - A natural inlet maintained by tidal flow.

Tidal prism - The total amount of water that flows into a harbour or estuary or out again with movement of the tide, excluding any freshwater flow.

Tidal range - The difference in height between consecutive high and low (or higher high and lower low) waters.
Undistorted model - A physical model which is geometrically a scale replication of a prototype.

Updrift - The direction opposite that of the predominant movement of littoral materials.

W/D - The width depth relationship in a tidal inlet entrance

Wave - A ridge, deformation, or undulation of the surface or a liquid.

Wave height - The vertical distance between a crest and the proceeding trough.

Wind setup - The vertical rise in the still water level on the leeward side of a body of water caused by wind stresses on the surface of the water.
1 INTRODUCTION

Tidal inlets along the Australian coastline and throughout the world present a challenge to coastal managers in managing the navigational and beach amenity requirements of the particular coastal environment. Tidal inlets also represent a challenge to estuarine water quality managers as the features of a specific inlet (natural or constructed) can influence the tidal flushing characteristics of enclosed waters.

Tidal inlet processes are complex due to the inter-related action of waves and tidal currents and their interaction with mobile bed sediments. Ebb-deltas (or entrance bars) are the central geomorphological and hydrodynamic feature controlling the estuary/adjacent beach interactions. Past research on tidal inlets has confirmed the pivotal role of ebb deltas in the tidal inlet system however understanding of their development processes and specific response characteristics have been restricted primarily to qualitative understanding.

The purpose of this research is to investigate key parameters affecting the growth of ebb tidal deltas and their interaction with adjacent beach forms using a scale physical model. The development of a detailed understanding of characteristics such as growth rates, plan form and ruling depths in the Coastal Movable Bed Physical Model (CMBPM) was undertaken as a initial step towards the development of frameworks for appropriate management of entrance channel movement, navigation and open coast sediment budgets.

The primary requirement of the scaled physical model was to simulate a tidal inlet system with waves, tidal flows and moveable sediment to provide an experimental test-bed for the assessment of key physical processes affecting the ebb-delta characteristics at the model scale. A secondary objective was to generate a physical model data set for comparison with numerical model simulations as a link between the physical model and prototype tidal inlets.

2 LITERATURE REVIEW

Appendix D includes an annotated bibliography of all research documented during this study and represents over 200 research sources on tidal inlet processes and physical modelling methodologies. This dataset was put together for the benefit of future researchers in this area and coastal managers who may wish to investigate specific inlet problems.

2.1 Moveable Bed Physical Modelling

An international literature review was carried out on the state of art in Coastal Moveable Bed Physical Modelling (CMBPM) in order to provide the best foundation for the research.

The starting point and primary foundation of the CMBPM study was “Physical Models and Laboratory Techniques in Coastal Engineering”, by Steven Hughes (1993). This text provided a summary of physical modelling methodologies and modelling experiences that were of great benefit to the project.

A limited number of moveable bed physical model studies specifically on tidal inlets have been undertaken however the following studies all provided input to the final selection of scaling criteria and modelling methodology. The most significant of these
studies were undertaken by the US Army Corps of Engineers under their Coastal Inlet Research Project (CIRP).

**Delft (1974)** – Gold Coast Seaway Design Model  
**Wang et al. (1996)** – Inlet Movable Bed Physical model” and “Criteria for Beach Response”  
**Seabergh and Smith (1999)** – Physical Inlet Model  
**Vithana et al. (2000)** “spit formation in simulated river mouth”  
**Deguchi et al. (2000)** “study on flushing of river mouth sand bar”  
**Seabergh and Sager (1979)** “Masonboro Inlet Physical Model study

The intention of this modelling exercise was to generate sand transport along a model beach at a known rate in order to feed the tidal inlet and compare with ebb delta formation characteristics. Kamphuis and Kooistra (1990) carried out a number of model longshore transport tests and from them defined a relationship for model longshore transport for given wave conditions, beach slope, orientation and sediment characteristics. Hence a major task of this project was to reproduce the tests of Kamphuis and Kooistra (1990).

As stated by William Kamphuis movable bed physical modelling is an ‘art’ rather than a science so the model design and testing methodology evolved during the project to achieve the desired outcomes.

A number of other research experiences were used in the development of the model and are listed below:

**Chesnutt (1975)** – Laboratory effects in coastal movable bed models, particularly beach evolution.  
**Kamphuis (1975)** – “Coastal Mobile Bed Model – Does it Work?”  
**Hughes and Fowler (1990)** – Movable bed scaling  
**Dean (1977)** “equilibrium beach profiles”  
**Hughes (2000)** “current induced scour – P vs A”  
**McNair (GITI) (1976)** “Model material evaluation”  
**Yalin and Price (1975)** “Time growth of tidal dunes in a physical model”

### 2.2 Tidal Inlet Relationships

Tidal Inlet relationships generated from field observations and measurements are an important tool in increasing understanding in tidal inlet processes and in assisting the development of the physical modelling methodologies. For this reason internationally recognised tidal inlet relationships were collated as part of this project. Given the benefit of this information for coastal managers these relationships have been included in this document as a draft summary report of tidal inlet relationships located in Appendix B.

### 3 PHYSICAL MODEL DESIGN METHODOLOGY

Because of the purpose of this study, to investigate ebb delta development, a movable bed physical model was required that could simulate the movement of sediment resulting from current and wave interaction at a tidal inlet.

Tidal inlets are an extremely complex interaction of tidal currents and wave forcing which impact and are impacted on by localised bathymetry.
There are two types of CMBPM they are (1) Prototype models which are used to investigate specific processes and (2) Idealised models which are used to investigate complex systems with a simplified system.

(1) Prototype models primarily focus on specific sites and processes and use distortion of parameters in order to calibrate with usually limited prototype data. This method gives qualitative results, based on the assumption that because the model gives the same end condition as the prototype it will respond in a similar manner to testing conditions. When using this modelling method great care is required in interpreting results because the model does not reproduce all prototype physical processes. However these models provide confidence in assessing management alternatives for complex coastal systems.

(2) Idealised models are important tools in understanding complex systems and are often used to form empirical relationships and calibrate numerical models. It is important to note however that idealised models are simplified physical systems not a representation of the prototype situation so again results must be carefully interpreted.

Given the objective of this study an idealised model methodology was accepted over a prototype scale model because of the lack of adequate prototype data and the constraints on the modelling facility.

The initial model design methodology is defined in detail in Appendix A. The design process following the literature review was to assess the model facilities available, including modelling area, wave generation limitations, pump flow rate limitations and available model sediment. When all these factors were considered a Dimensional analysis was carried out on all the physical parameters involved with a prototype tidal inlet (hydrodynamics and sediment transport). This methodology enabled the project to choose scaling criteria based on the facilities, sediment and physical limitations to achieve the best outcome.

3.1 Dimensional Analysis

Dimensional analysis (detailed in Appendix A) showed that it is simply not possible to reproduce prototype inlet processes in a movable bed physical model much smaller than the prototype, due primarily to the limitations of model sediment and the practical limitations on the size of modelling facility. The largest shed at QGHL is approximately 30 m wide and 100 m long. Even if the project had this entire area the largest scale inlet possible would be 1:40. Given that the prototype sand has a median grain size of 0.3 mm if it were to be scaled down geometrically the model sediment would have to have a median grain size of 0.007 mm which would no longer be a non-cohesive sediment. For this reason a critical part of the project was the selection of model sediment.

3.2 Sediment Selection

Prototype Sediment

Prototype sand was chosen as the sand from the ebb shoal at the Tweed River Inlet, because dredging of the entrance at the time of the project allowed easy access to samples of the material. Several samples from the Tweed entrance ebb shoal, adjacent swash zone and adjacent beach face indicated the significant difference in median sediment size due to the natural sorting action of tidal inlets. Even in the Ebb shoal itself values of d50 = 0.27 to 0.32 were measured. Although the project was focused on an idealised inlet, it was appropriate to use a prototype sand that represented the high energy coastlines of South East Queensland.
The following initial properties were measured of the material:

- $d_{50} = 0.266$ mm
- Fall Velocity, $\omega = 0.051$ ms$^{-1}$
- Critical bed shear velocity under unidirectional flow, $V_{crit} = 0.181$ ms$^{-1}$
- Submerged angle of repose $\alpha = 35.5^\circ$

(please refer to testing method below)

From literature it was defined that sand was the best model sediment for the objectives of the modelling project. Lightweight sediment would more readily be moved under the action of waves and currents however sand was chosen as recommended by past research. The following reasons are defined for why lightweight sediment was not chosen.

- Past studies have shown that lightweight sediments accelerate differently due to oscillatory flow (waves) than prototype sand sediments, Paul (1972)
- Lightweight sediments move when the model is started and stopped
- PVC grains are very expensive in the quantity required for this investigation.
- Even when using lightweight sediment fall speed parameter ($H/wT$) cannot be conserved between model and prototype.

The search for sand

Ten sand samples were sourced for testing as possible model sediments, each sample was; sieved for $d_{50}$, fall speed measured and critical bed shear velocity measured.

From these initial tests it was evident that only two samples were possible model sediments; Stradbroke mineral sand and Stradbroke silver sand.

From the initial simple tests the Stradbroke mineral sand had the best properties for use in the model, however there was some question about the specific gravity. For this reason the prototype and the two possible model sands were tested for specific gravity.

The outcome of the specific gravity tests defined that the Stradbroke mineral sand ($SG = 2.97$) was much heavier than the prototype ($SG = 2.65$) and the silver sand ($SG = 2.66$). This outcome suggests that other sediment properties must play a significant role in the characteristics of the mineral sand, such as particle shape. Given this outcome the mineral sand could not be relied upon in modelling and so the silver sand was chosen as the model sediment. Following specific fall speed tests for sieve increments it can be defined that the model sediment has the same properties as the prototype material at the same size increments. However it was noted that significant shell material existed in the prototype sediment which significantly influenced sediment properties. Both prototype and model sand were retested accurately to define the final parameters on which to base the model scaling criteria which are highlighted below;

<table>
<thead>
<tr>
<th>Table 1. Sediment characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prototype</td>
</tr>
<tr>
<td>Stradbroke Sand</td>
</tr>
<tr>
<td>$d_{50}$ (mm)</td>
</tr>
<tr>
<td>$\omega$ (m/s)</td>
</tr>
<tr>
<td>$V_{crit}$ (m/s)</td>
</tr>
<tr>
<td>$\alpha$ submerged angle of repose</td>
</tr>
</tbody>
</table>
Testing Methods

Median Grain size

Standard sieve analysis undertaken on three samples and the average taken.

Fall Velocity Calculation, \( \omega \)

QGHL has a 0.9 m perspex tube with 0.1 m gradings on the side. The tube was filled to the 0.9 m grading with fresh water. Initial testing method included a set sample of material being placed on the water surface and the time measured for approximately 50% of the material to reach the bottom of the tube visually. This test was undertaken 8 times for each sample and the results were averaged.

For the final measurement of fall speed for the scaling criteria a method from Paul (1972) was used where samples were sieved and each sieve graduation from the sieved sample was tested in the tube using the method above. Then the resulting fall speeds were averaged.

The influence of salt water in the prototype was tested with a sample being tested in the field and in the laboratory. Although only one comparison was made it showed that fall speed is reduced in salt water. Hence prototype fall speed is slower in a real inlet, than defined in a fresh water test. The scale error associated with this was been noted.

Critical bed shear velocity, under uniform flow \( V_{\text{crit}} \)

A small flume was used to measure critical velocity of sediment incipient motion. A digital flow meter was hooked up to a pump system that inlet at the head of the flume into a baffle system before travelling down the flume to a dam gate. The velocity in the flume was controlled via adjustment of the dam gate. Each testing campaign began by starting the pump up and generating a specified flow rate (Q). Dye and adjustment of the baffles was undertaken to achieve a relatively uniform flow. Next a sample of sediment was added to the base of the flume at a level profile. The dam gate was started at a significant height and reduced in increments until incipient motion was visually observed and the depth of water in the flume was measured to allow the critical velocity to be calculated \( (V = Q/A) \). All tests observed when continuous bedload occurred, when ripples formed, and the formation of a crest shape at high velocity.

Submerged angle of Repose, \( \alpha \)

Submerged angle of repose was estimated by the use of a shallow galvanised tin box in the small wave flume used for the critical velocity tests. The box was filled with sand and levelled, one edge was fixed on the bed of the flume and the other corner was lifted at a steady rate until sediment motion began. This test was carried out several times and the finish height of the box corner was marked on the side of the flume. The angle was then averaged from the testing results.

Specific Gravity Calculations, \( SG \)

The specific gravity of the samples was tested by the Department of Natural Resources at the Rocklea Laboratory.
3.3 Final scale selection and modelling limitations

Given the modelling limitations the chosen design for the physical model was a scale 1:50 tidal inlet, it was the largest inlet that could fit in the modelling area and approximately represented a 1:50 scale model of Brunswick River inlet in northern NSW.

Following the dimensional analysis process and the selection of the model sediment the choice of the scaling criteria was largely dominated by practical considerations.

Dimensional analysis showed that the hydrodynamic interaction of waves and currents at the inlet could be effectively modelled with froude scaling however the froude scaling methodology would not produce simulation between wave and current sediment transport potential. The use of this criteria in the movable bed model would result in wave forces being relatively greater in instigating sediment transport than current forces. Because the project was investigating sediment transport due to waves and currents at a tidal inlet it was considered critical to maintain the ratio between the two forcing parameters. For this reason the initial scaling criteria chosen was the “SAND” criteria of Kamphuis (1975). This criteria involved the distortion of the inlet current velocity to achieve sediment transport similitude in the model. This methodology was based on the fact that bedload sediment transport dominates in the model. It was noted however that this choice would generate hydrodynamic distortions in the tidal inlet that must be considered in the analysis of results.

The other critical factor in selection of scaling criteria was the morphological time scale. The hydrodynamic parameters, in particular the waves, needed to provide sufficient forcing to promote sediment to the inlet and generate ebb delta growth within the time constraints of the project. For this reason the final wave condition chosen would not necessarily be a mean prototype wave condition. As defined in the preliminary testing this was found to be the case and a storm prototype wave condition was used in the final model study.

Physical Modelling Limitations

Model distortions are required to achieve a practical model that can give insight into inlet processes. However the modeller must always be aware of the limitations created by distortion of the model parameters and their impact on model outcomes.

The primary limitations of this particular physical modelling methodology are defined below;

- The distortion of the current velocity scale will distort the model hydrodynamics at the inlet.
- The proposed model inlet system represents a prototype tidal inlet with the predominant sediment material being 10 mm gravel under the action of storm waves. For this reason results will differ from a prototype inlet because unlike the desired prototype the primarily transport mode will be as bed load.
- The relatively large sediment in the model results in distorted a equilibrium beach profile in the model compared to prototype beaches.
- The use of prototype storm wave conditions to speed up morphological time scales in the model will distort the expected prototype ebb delta development process.
- The surfzone is not modelled correctly in comparison to the prototype because the surf similarity parameter (H/ωT) cannot be held in similarity at much less than the prototype scale. However the majority of inlet
relationships define that tidal prism and associated inlet dimensions have primary influence on ebb delta growth characteristics, longshore transport simply promoting the sediment to the inlet.

These limitations show that very few parameters can be held in similitude with prototype processes within the practical limitations of a Movable Bed Physical Model. However given these constraints, there exists little other option for the investigation of the complex and dynamic processes at tidal inlets.

The benefits of the chosen movable bed model design are as follows;

- The model inlet system is a controlled small scale prototype inlet that has all the primary characteristics of a prototype inlet.
- This model can generate beach and ebb delta response relationships against the model longshore transport rate. Although the inlet processes may not be the same they may instigate ideas for further research in prototype scale inlet processes.
- This system allows the qualitative observation of tidal inlet processes.
- This controlled inlet system can be used to generate a comprehensive data set to be compared to numerical model simulations and hence extend findings through the numerical model to prototype scales. The theory behind this being that even though the physical model does not correctly model the prototype processes a lot can be gained from testing the limitations of the numerical software against a simple controlled wave/current/sediment interaction system.
- Secondary advantages of this modelling method include its flexibility to undertake a scale series of models in order to better understand scale effects and its flexibility to be used for future movable bed model investigations given its generic layout.

4 MODEL CONSTRUCTION

A schematic of the constructed model is shown in Figure 1 and a number of photos showing the stages of construction are in Figures 2a to 2i.

The model was constructed with a compacted soil base which was built up from the shed floor level at 20 degrees to the back wall of the model. The design concrete profile was then formed according to a steep equilibrium beach profile that would allow a thin or thick veneer of model sediment to be formed over for final inlet testing.

At the back of the model three 6 m wave paddles were installed (Figure 3.5) and calibrated in series to generate a single 18m wide wave front. At each end of the wave paddles two 12 inch PVC pipes were installed with dissipaters to provide symmetric flood flow to the model. Adjacent to these pipes and set back from the paddles on each side were 2.5m weirs to control ebb flows from the model area. Figure 3.3 shows these features. Behind the weirs and along the back of the model a 1m wide return channel was constructed to allow the water from the model to be recycled to the storage tank under the QGHL modelling facility prior to being pumped back through the system. A pointer gauge was installed adjacent to the left offshore weir that was used to define water levels within the model area and later for surveying characteristics of the ebb delta.

At the front of the model a curved block-work channel was constructed that represented the model estuary. This included a 3 m weir and another 12 inch pipe
and dissipater to facilitate ebb and flood flows to and from the model respectively. A return channel was constructed down the right side of the model to recycle water through the system. Figure 3.2 shows the features of the estuary area. Two rubble mound breakwaters were constructed either side of the inlet channel with a geofabric spine to ensure sand could not flow through the structures.

Because the project needed to measure the longshore transport in the model a catch tank was constructed into the floor on the left hand or downdrift edge of the model beach. This catch tank consisted of a shallow galvanised iron tray hanging just below the level of the model beach from two brackets. An early photo of the equipment is shown in Figure 3.0. This photo does not include the false lid and wave guide that was installed to refine the catch tank after initial testing. The hanging mechanism included a load cell hanging from each bracket that was calibrated to measure the sand leaving the downdrift edge of the model as shown in Figure 3.4.

Following the construction of the model a series of cables were erected from the model area back to the model control room. The entire model was controlled and monitored using the GEDAP software system including the generation of waves, ebb and flood tidal flows, the measurement of sand in the catch-tank, the monitoring of water levels and wave conditions throughout the model.

5 PRELIMINARY PHYSICAL MODEL TESTING

In order for this project to be successful at the investigation of ebb delta develop the following issues had to be addressed:

1. Final design wave conditions had to produce a high enough sediment transport rate so ebb delta development could be measured in a practical time interval.
2. A stable equilibrium beach profile had to be defined for the chosen model sediment and design wave conditions before ebb delta tests could commence.
3. The longshore transport tests of Kamphuis and Kooistra (1990) had to be simulated on the model beach to defined the transport rate being delivered to the inlet. This was critical for comparison with ebb delta growth characteristics.
4. Before final inlet testing commenced the flood and ebb flow system had to be calibrated and automated.
5. Finally all monitoring equipment and methodologies had to be tested, calibrated and refined where required.

The following sections formed the extent of the preliminary investigation;

- a) Fixed bed testing of design wave parameters and tidal operations.
- b) Formation of an equilibrium beach profile
- c) Generation of a uniform beach longshore transport to feed the model inlet
- d) Calibration of a model tidal system

5.1 Fixed-Bed Inlet Testing

Fixed bed testing was undertaken of the model inlet to test the following;

1. The pipework and weir design would achieve the desired flows for the scaling criteria.
2. The model sediment was placed on the concrete beach profile in thin lines and a variety of wave conditions were tested to assess longshore transport potential.
3. Flow patterns and basin circulations were investigated and assessed with tracer sediment, dye and drogues.
The methodology for the fixed bed testing was to first fill the model area to an offshore depth of 0.55 m at the wave paddles; this height was chosen because the capacity of the flapper wave paddles to generate waves is dependent on water depth. The flapper paddles were calibrated and 3 capacitance wave probes were calibrated to measure the offshore wave condition approaching the model. The wave probe calibration setup is shown in Figure 3.1, where the level of the frame was adjusted 4 times to plot the calibration curve. The GEDAP software system and a PC were then used to generate the required wave conditions for testing. At this early stage of the project the tidal system had not yet been automated so control volume calculations were used to define the position of the weirs and the input flow rates to achieve the desired inlet flow conditions. Ebb and flood flows were simulated via manually starting the pumps that were controlled with control valves and setting the weirs at specific heights. An Acoustic Doppler Velocimeter was used to measure velocities along with the water depth to assess the model flow rates in the inlet channel.

The results of the fixed bed testing showed that the pipework and weir system would achieve the required flow velocity through the inlet. Figure 5.1 and 5.2 represent examples of the fixed bed tests.

The sediment tracer tests showed that the median (prototype) design wave condition was not sufficient to generate significant longshore transport in the model and hence storm prototype wave conditions would have to be used for final testing as was done in Wang et al. (1996). After this testing $H_s = 0.065 \text{ m}$ and $T_p = 1.2 \text{ s}$ was adopted as the design wave to accommodate a more practical system and morphological time scale. Figure 5.0 shows an example of a tracer test and associated sediment transport. The area outlined in red defines the extent of the sediment migration along the model beach. Design wave conditions were measured with capacitance wave probes to verify the final design wave condition. The design wave condition was measured about 5 m from the wave paddles offshore of model beach.

Investigations of basin circulations showed that there was a problem with basin circulations due to the longshore transport current however this was addressed via making the downdrift boundary more permeable, so as to reduce the return flow effect in the model area. Updrift and downdrift boundary effects were evident. A particular weak point of the model was the sediment catch tank design. Because of its purpose the catch tank needed to be in the model area. The problem associated with this was that the wave on the downdrift boundary was extremely distorted by refraction due to the change in water depth. Even after installing a partial lid and wave guide on the trap there was a small downdrift accretion of sediment prior to entering the trap which tended to further influence wave conditions.

The fixed bed testing provided confidence in future model outcomes because the boundary effects were minimal and the hydrodynamic processes in the vicinity of the inlet reproduced prototype processes. Figure 5.3 shows tracer sediment being transported offshore along the updrift training wall following its path along the updrift beach.

Dye testing of the inlet and beach system seemed to show relatively good agreement with the natural system, in terms of longshore transport, ebb flow jet and the circulation cell downdrift of the training walls.
5.2 Equilibrium Beach Profile Generation

The Modelling project intended to reproduce the longshore transport tests of Kamphuis and Kooistra (1990) via plugging the inlet and moulding a straight beach. Kamphuis and Kooistra (1990) moulded a 1 in 10 sand profile onto their model beach and then ran waves on this profile for a period of time for a equilibrium profile to form before they commenced measuring longshore transport. Given this methodology the project inserted an undistorted average Gold Coast, QLD beach profile into the model beach after the inlet structures had been removed and the inlet plugged. The design wave conditions defined from fixed bed testing (Hs = 0.065, Tp = 1.15 s) were run on this profile and 5 beached profiles (Figure 6.21) were surveyed periodically across the model to assess when a stable equilibrium profile had formed. A total station and a rig suspended over the model area shown in Figure 3.8 and 3.6 were used to measure the profiles. After 30 hours the test was abandoned because the morphological time scale was going to be impractically long to form an equilibrium profile. The beach was then remoulded and a design storm wave (Hs = 0.08 m Tp = 1.2 s) condition was tested on the undistorted model profile and a stable equilibrium profile was not successfully generated after 20 hours of testing.

It was then decided that the generation of an equilibrium profile in the model basin would not be practical due to time constraints.

Given this constraint the project team instigated two flume tests to develop a design model equilibrium beach profile. The undistorted profile was inserted into the flume and the design wave conditions were run on the profile and periodic surveys were taken of the profile until a quasi equilibrium condition was reached. This took approximately 70 hours of testing time but because the flume could be run overnight this process was much faster than in the basin. A large mass of sand was transported offshore in the flume in the generation of the equilibrium profile. It can be seen in Figure 4.1. This was excess material that was not part of the offshore profile was removed and found to have no influence on the final equilibrium profile because it was below the depth of closure.

At the completion of flume testing two equilibrium profiles were generated one for the design wave condition and one for the storm wave condition. These match a Dean (1977) equilibrium profile with a coefficient A of 0.07. The measured flume profile for design wave conditions are shown in Figure 4.0 and a photo of flume testing in Figure 4.1.

Table 2 shows the simplified representation of the equilibrium profile that was moulded into the model basin for beach longshore transport tests and tidal inlet testing.

<table>
<thead>
<tr>
<th>Equilibrium Beach Profile used in inlet model</th>
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<tr>
<td>chainage(m)</td>
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<tr>
<td>0.96</td>
</tr>
<tr>
<td>3.32</td>
</tr>
<tr>
<td>3.88</td>
</tr>
<tr>
<td>5</td>
</tr>
<tr>
<td>7.9</td>
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<td>11.1</td>
</tr>
<tr>
<td>12.5</td>
</tr>
</tbody>
</table>
The design wave profile was moulded onto the concrete profile via the use of 6 steel templates. The excess sand was removed from the basin with a loader via the back of the inlet model. The moulding of the model was an extremely labour intensive process because the sand was not dry and any mechanical means of levelling created more disturbance or simply did not work. Hence all reprofiling had to be done by hand. This process was particularly time consuming because of the large modelling area required to investigate ebb delta evolution.

### 5.3 Longshore Transport Verification

The purpose of the longshore transport testing was to achieve a quantitative indication of sediment transport across the model. To commence these tests the generated equilibrium profile was moulded onto the model beach and the methodology of Kamphuis and Kooistra (1990) was followed. This methodology included:

- Fill the model to 0.55 m offshore water depth
- Calibrate wave probes
- Start the wave paddles at the desired design wave condition.
- Insert sand at an updrift beach at the rate specified by Kamphuis and Kooistra (1990) formula below.
- Periodically survey 5 beach profiles across the model beach to assess beach profile stability (Figure 6.21)
- Hourly stop the waves and measure the submerged sand volume in the downdrift sand catch tank.
- On restarting the waves sample offshore waves for 5 minutes to ensure uniform water levels (stilling wells) and wave conditions
- Ripple geometry was measured on the surveyed profiles
- Water temperature measured

\[
\frac{Q}{\rho H_{sb}^3} = 1.3 \times 10^{-3} \left( \frac{H_{sb}}{L_{op}} \right)^{1.25} m b^{0.75} \left( \frac{H_{sb}}{D_{50}} \right) \sin^{0.6} (2\alpha_p)
\]

Kamphuis and Kooistra (1990)

Longshore transport measured for these tests showed a good correlation with Kamphuis for the first 6-7 hours. Following this initial period basin effects starting influencing the model. In particular a shallow diagonal offshore bar formed which influenced the incoming wave front and caused the beach to perturbate into a system of cusps on the foreshore. The offshore profile was relatively stable except for this very subtle shallow bar. A second test was undertaken and the results were exactly repeated.

The project got some advice from Professor William Kamphuis about the basin effects and installed some additional dead water on the updrift side of the model via an additional wave guide.

The slight slope to the floor in the deep water before the model was considered a possible source of error and so the floor was built up to match the paddles that were mounted level.
A rubber skirt had been placed at the bottom of the paddles in a prior physical model to stop rocks from jamming underneath. From close inspection this rubber skirt was having a significant influence on model circulations. The skirt allowed water under the paddles on the forward stroke but on the backward stroke they blocked the flow and caused the water to rush around the two extreme ends of the paddles. This decreased the paddle movement and hence the wave heights and at the same time drove a basin circulation.

Visual observation seemed to show no indication of a cross-wave at the paddle however two central wave guides were placed adjacent to the paddles to minimise any possible effect.

The amount of rutile in the sediment was also an issue of concern. But given the volume of sand it was not viable to screen it out. Tests were carried out to assess if armouring due to the rutile was significant however no measurable differences were found between a beach where the rutile was scraped and a beach where scraping did not occur.

A particular basin effect was the formation of a cusp and an adjacent eroded area in the centre of the model. It was thought that the location of the inlet under the beach may be responsible due to the perturbation in the model beach watertable. The inlet base was plugged up to a greater extent to minimise this influence.

After these modifications a further three tests were undertaken, using transport rates as defined from before which matched well with Kamphuis’s formula. These tests with amendments generated an extremely stable beach profile with no offshore features, across the 5 model profiles for the 18 hours of testing. Figure 6.2 shows the monitored profiles for beach test 1. These results were reproduced in beach test 2 and beach test 3 giving confidence in the model setup to monitor ebb delta development. The centre foreshore of the beach however did still form a small perturbation that could not be explained however the updrift and downdrift beaches were relatively stable. Transport rates measured were similar to earlier tests with the expected transport rate being achieved for the straight beach until foreshore perturbations occurred in the centre of the model and adjacent to the sand trap.

The research timeline was too tight to investigate further the model laboratory effects associated with the beach. The perturbation at the centre of the model could not be explained but it reached an equilibrium relatively quickly and it would be assumed it would not further influence the longshore transport in the model. Retrospectively assessing the testing from some survey levels taken adjacent to the trap showed that a significant amount of material had built up adjacent to the trap and offshore. From a rough volume calculation of down-drift beach it is possible that the decrease in sediment transport measured in the catch tank may be accounted for because of boundary effects.

Also in retrospect the distance between the model beach and the paddles was probably too small and because of the steep foreshore in the equilibrium profile reflected waves may have played a role.

The outcome from the longshore transport testing was that longshore transport measured in the model was effectively predicted by the Kamphuis and Kooistra (1990) formulation and the design equilibrium beach profile was stable. Figure 6.0 shows the model longshore transport testing results in comparison with the Kamphuis formula. Figure 6.1 shows beach longshore transport testing underway. Given the results of these validation tests the Kamphuis formulation was used to
define the sand input rate to the updrift beach of the inlet model and provided confidence in the model system to accurately measure the ebb delta development in a stable beach system.

5.4 Tidal System Verification

In order to investigate the formation of the ebb delta due to waves and tidal flows. The model needed to develop an automated system of ebb and flood flows. Ideally the model would develop the inlet flow rates and water level changes that are the primary features of tidal flows.

On first design of the Movable Bed Inlet Model it was proposed to generate a sinusoidal tide within the model that reproduced both ebb and flood flows in the inlet as well as the changes in water level. When undertaking the beach longshore transport testing a set of simple tests was made to test the practicality of producing a sinusoidal change in water depth alone. A closed loop control was set up in the GEDAP model control system with the assistance of Mr Gary Hart (EPA). This input water at a set rate into the model through digital control valves and used the back weirs to adjust water level to fit a desired time series given realtime feedback from a capacitance stilling well in the model area. These tests shows that the weirs and drive motors did not have the response time to generate a smooth sinusoidal tide they were always overshooting. Due to the complete non-linearity of the weir response the only possible way to achieve a reasonably sinusoidal water level change was to define specific points on the curve in situ for the weir to drive to. Due to the complexity in reproducing a sinusoidal water level change alone without matching the inlet current velocities, that were critical to the project, the sinusoidal system was set aside in favour of a quasi-steady state system.

This system was proposed to consist of steady state flows in the ebb and flood direction at the required scale velocity of 0.3 m/s and 0.14 m/s and at a fixed offshore design depth of 0.55 m. Because of the time limitations we wished to automate these steady state conditions in a smooth manor in order that the model could be run continuously without stopping between ebb and flood cycles. Effectively running a blocked velocity time series. This was considered to be acceptable as the primary sediment transport within inlets result from the maximum discharge conditions that would be simulated in the steady state system.

The first step of this process was to define two steady state systems. Because the system was steady state the primary purpose of the tests was to achieve the required inlet velocity at a static design water level. The dimensions of the original inlet and the required scale inlet velocity were used to define the flow rate required into the model for both flood and ebb flow conditions.

In order to achieve this a stilling well was set up in the centre of the model as a control for the water level. A piece of code was written to read the original water level for 10 s (design level). The code ramped up the control valves to the required flow and then the weirs were ordered to hunt to maintain the water level at the design value. This water level associated with the inflow rate generated the required inlet current velocity that was verified with ADV measurements. When the steady state condition was reached the weirs stopped. This methodology was not used for the inlet testing because of the time lag for the steady state to be achieved and the fact that conditions were not exactly reproducible.

For this reason we defined the steady state as a number of digital hits on the weir motor from the starting point. When this was set up it was discovered that the system
could not be controlled via the digital hits because the weir travel was completely unrelated to the digital hits. Weir travel down was much further than travel up under the same amount of hits because of the weight of the water. This finding meant that reproducibility could not be achieved unless everything was strictly calibrated \textit{in situ} and the only way of resolving this was to define the position of the weirs with other instrumentation.

Mr Gary Hart (EPA) suggested using potentiometers to measure the rotation of the weirs and convert the voltage measured to mm of travel. These were installed and calibrated by measuring the change in length. The equilibrium position hunting tests were repeated and the location of the start points and equilibrium points were defined from the potentiometers. This achieved a reproducible steady state system for both flow conditions to be tested.

The next phase of the process was to tweak the transitions between the steady states in order to minimise water level fluctuations and fit the ebb/flood cycle into a semi-diurnal time scale of \(~106\) minutes. Because the weirs were the biggest limitation the transitions were restricted by their speed in getting down to the steady state position and getting back up above the water level. So the weirs were set to go at top speed down and up and then the control valve inflows were adjusted to be matched and minimise changes in water level.

This was a particularly difficult process as the weir and control valve responses were both non-linear. However it was discovered very early that a small loss in water level was more advantageous than a gain because the next steady state would quickly level things up. However an increased water level in the model would take some time to be removed from the system. So the calibration process erred towards the former result.

All in all because the flood weir had further to travel in opening and closing the flood transition was longer than the ebb transition. In total the ebb transition took \(6:30\) minutes to get to steady state and about the same to come back above the water level. It is important to note that we short cut the transition by changing over once the weirs were above the water surface. The flood transition took \(8:30\) minutes to reach steady state and slightly longer to come back to the water level. This equated to a total transition time of \(30\) minutes for a complete ebb/flood cycle. Allowing two \(38\) minute steady state ebb and flood conditions.

Following the calibration process the methodology of ebb and flood flows is described below:

The front control valve is open at a specified discharge curve. The two weirs at the back of the model are driven down by drive motors at maximum speed to the steady state position defined from the calibration. When the weirs reach the steady state position the flow into the model is at its maximum and the steady state inlet flow conditions are achieved. This steady state continues for \(38\) minutes and then the inlet discharge pipe starts to decrease flow into the back of the model and the offshore weirs begin to wind up with the turn of the tide. When the back weirs reach the specified still water level position the inlet flow has stopped and the offshore control valves begin to open and the inlet weir begins to drive down to its steady state position. While this transition to flood flow steady state continues the offshore weirs continue to drive up to their starting point prior to the next ebb flow cycle. When the flood flow steady state is reached this runs for \(38\) minutes before the system makes the transition back to the ebb condition. This process provides a reproducible steady state system at a constant still water level.
To ensure the reproducibility of this methodology current velocities and water levels were measured throughout the model inlet testing. Figure 7.0 and Figure 7.1 show examples of the inlet current velocity profiles measured during the inlet model testing to show the structure of the two different flow conditions used. An Acoustic Doppler Velocimeter was used to take these measurements pictured in Figure 3.7.

6 INLET TESTING OF EBB DELTA DEVELOPMENT

6.1 Testing parameters
The final design parameters used in the inlet testing were a result of the literature review, dimensional analysis and importantly the practical limitations highlighted in the preliminary testing. Given the extensive effort required to develop such a model it was unfortunate that time constraints limited the project to four final inlet tests. Further investigation of additional parameters in the model would be required to improve understanding of ebb delta development processes.

Table 3 shows the testing parameters used in this study. The primary parameters investigated in the model study were the influence of inlet flow rate (which can be linked to prototype Tidal Prism) and the influence of artificial sand bypassing on ebb delta development and adjacent beach response.

An extension to this testing would be to investigate the influence of the following parameters:

- Changes in water level at the inlet
- The response of the ebb delta and adjacent beach to dredging of the ebb delta.
- The response of the inlet to episodic storm wave events.
- Investigation of a natural inlet configuration
- Assess how different coastal structures influence inlet and adjacent beach response

6.2 Inlet Testing Methodology
The inlet testing methodology was developed to monitor ebb delta development and response of the adjacent beaches under the action of the parameters defined in Table 3.

At the start of each test the equilibrium beach detailed in Table 2 was manually moulded into the model beach with a series of templates. To give this task scale this took two days for 3 people to complete at the start of each test. The Inlet itself was also moulded with a 0.1 m veneer of sand.

When the sand profile was complete the model was filled with water up to a depth of 0.55 m offshore. The seven capacitance stilling wells and wave probes within the model were calibrated and an original survey of the model area was undertaken. The entire model was defined in a grid system with the origin being the corner of the model adjacent to the catch tank. A total station was used to undertake surveys the first survey of each test consisted of;

1. the Mean Water Line (MWL) position on the beach,
2. an inlet throat cross-section survey
3. an inlet centreline survey offshore
4. two beach profiles 3 metres north and south of the channel centreline
5. the locations of the measuring instruments eg wave probes and ADV
6. A detailed survey of entire model area.

Once the initial survey was complete the GEDAP automated system generated the design wave conditions and the steady state ebb flow as defined in Section 4.4. Stilling wells within the inlet and offshore were used to monitor the water levels to ensure reproducibility and at the start of each day 10 minutes of offshore wave data were collected to verify that the design wave condition was achieved. ADV measurements were also taken to verify initial design inlet current velocities. In accordance with the Longshore transport tests 33 kg (submerged weight) of sand was added to the updrift beach per hour to maintain the longshore transport to the inlet system. Because this quantity of sand is associated directly with the measurement from the catch tank (as in Kamphuis and Kooistra, 1990) a systematic process of sand addition was developed and calibrated with the catch tank. This included the addition of sand to a standard vessel and wetting of the sand until it settled completely. The excess sand was then scraped to form a level surface on the top of the vessel and this volume was placed in the catch tank and relative mass measured. This methodology effectively maintained a stable updrift beach profile as required for the inlet testing.

At the end of each testing period the following parameters were measured;
- the Mean Water Line (MWL) position on the beach,
- an inlet throat cross-section survey
- an inlet centreline survey offshore
- two beach profiles 3 metres north and south of the channel centreline
- the locations of the measuring instruments if moved.

Following the collection of this data with the total station the model area was drained in increments to monitor the extent and characteristics of the ebb delta. The ruling depth of the ebb delta was defined as first point to break the surface when draining occurred. A stilling well at the side of the model was used to measure water levels while the total station and pole defined the 2D extent of the topographic features. The initial ebb delta surveys covered quite a large area in the model to ensure they captured the full extent but later testing refined this area to reduce monitoring time. Photos of the draining process were also taken to assist in the analysis of the measured data. The process of running and monitoring the inlet was repeated until the end of each test. At the end of each test a full survey of the model area was undertaken for comparison with the original profile.

Observations of inlet processes were taken during the four tests and tracer sediment was used to investigate inlet flow patterns and natural inlet sand bypassing mechanisms.

A goal of the research was to investigate the influence of artificial sand bypassing on the formation of the ebb delta. In order to achieve this in T2 and T3 a jetty structure was erected as shown in Figure 1 and sand was removed at the specified rate in a defined channel along the jetty using a shovel. The intention of the methodology was to remove the entire calculated longshore transport rate each hour prior to it entering the inlet. Hence analysing the influence of starving the inlet of sand in a controlled system.
a. Data Collected from Inlet Physical Model

Four datasets were developed from the inlet physical modelling for use in comparison with numerical model simulations and for the generation of physical model inlet relationships. All equipment positions and monitoring locations were defined according to the single model grid which enables direct transfer of monitoring results to numeric model grids.

The extent of the dataset for the four inlet tests included the following:

- 1 wave condition offshore measured at the start of each modelling interval. Figure 8.0 shows a measured sample of the final design offshore wave conditions.
- 3 wave conditions measured at the start of each modelling interval starting from the extent of the movable bed up to just before the end of the breakwaters all on the centreline of inlet.
- 3 stilling well water level measurements taken at the start of testing intervals and at random times.
- ADV and current measurements within the inlet, over the ebb delta and across the beach profile at random intervals.
- Geometry of ripples developed in the model due to waves and tidal currents.
- Time series of MWL movement showing beach retreat and accretion.
- Profiles of channel centreline, north and south beach profiles and inlet cross-section as defined in the schematic above Figure A2.
- Detailed ebb delta survey for each time interval which shows ruling depth height and location and plan form of ebb delta.
- A full model area survey at the completion of each of the four tests.

a. Outcomes of Ebb Delta development tests

**T1**

Test T1 was a current-dominated unbypassed inlet. Figures A1.0-A1.4 show the measured ebb delta formation and the associated measured changes in beach mean water level for the period of T1. Figure A2.0 shows profiles of the channel centreline, beach profiles north and south of the inlet and the inlet minimum cross-sectional area for T1.

T1 showed rapid growth of up-drift beach adjacent to the up-drift training wall and the subsequent erosion of the down-drift beach. The ebb delta formed relatively quickly growing on the ebb flow and shrinking on the flood flow. The high velocity current during the ebb cycle caused significant wave refraction which influenced ebb delta formation. The Inlet cross-sectional area was maintained relatively stable throughout the test however the channel migrated down-drift due to the infill of sand from the up-drift beach resulting in a narrower but deeper channel. A significant volume of material was scoured from the inlet channel. The inlet centreline survey showed the scour hole in line with the end of the training walls which is a feature of prototype inlets. Tracer tests showed the nature bypassing had commenced prior to the end of testing. The natural bypassing route is defined in Figure 15.

**T2**

Test T2 was a current dominated bypassed inlet.

Figures B1.0-B1.4 show the measured ebb delta formation and the associated measured changes in beach mean water level for the period of T2. Figure B2.0 shows profiles of the channel centreline, beach profiles north and south of the inlet and the inlet minimum cross-sectional area for T2.
By artificially bypassing the T2 inlet the beach alignment is effectively maintained up-drift and down-drift of the inlet. The ebb delta formed relatively quickly as in T1. The inlet cross-sectional area and dimensions were stable for the entire testing period as little sand flowed in from the up-drift beach. As in T1 a significant amount of material was scoured from the inlet channel. The inlet centreline survey showed the scour hole in line with the end of the training wall as in T1.

**T3**

Test T3 was a wave dominated unbypassed case. Figures C1.0-C1.4 show the measured ebb delta formation and the associated measured changes in beach mean water level for the period of T3. Figure C2.0 shows profiles of the channel centreline, beach profiles north and south of the inlet and the inlet minimum cross-sectional area for T3.

T3 proved to be an unstable inlet. As in T1 the up-drift beach accreted and the downdrift beach eroded. The inflow of sediment from the up-drift beach was greater than the ebb current’s ability to remove the material and hence the inlet moved towards closure. Closure did not occur however because flow was forced in the model so an over-height build up and scoured the inlet pushing significant material offshore.

**T4**

Test T4 was the wave dominated bypassed case. Figures D1.0-D1.4 show the measured ebb delta formation and the associated measured changes in beach mean water level for the period of T4. Figure D2.0 shows profiles of the channel centreline, beach profiles north and south of the inlet and the inlet minimum cross-sectional area for T4.

In test T4 artificial bypassing maintained a relatively stable inlet cross-section after some initial infilling. T4 effectively retarded ebb delta growth for the period of the test. Bypassed material was recycled on the up-drift beach and this resulted in erosion to the down-drift beach. The channel centreline survey showed only a small sign of a scour hole at the inlet entrance.

**Comparisons and Observations**

The rate of erosion and accretion measured on the adjacent beaches for T1, T3 and T4 (downdrift) appeared to be independent of inlet flow conditions.

Observations of the trained inlet tests showed that material piled up against the up-drift training wall until the profile angle was parallel to the wave crests. At this point the beach steepened up until plunging breakers began to suspend the material and transport it offshore (cross-shore) along the training wall and into the inlet. As shown with the tracer material in Figure 14. Where no bypassing occurred the beach continued to grow until it reached close to the end of the training wall this location was consistent for T1 and T3. This formed an unimpeded conveyor belt of sand into the inlet.

In T2 and T4 the bypassing jetty provided a permanent rip cell in the beach area trapping sediment from adjacent beaches. Both tests observed that the bypassing mechanism of deepening the channel resulted in a magnification of the rip cell offshore and towards the back of the ebb delta Figure 16.

Figure 10 shows the ruling depths measured during tests T1 to T4. The plot showed that the current dominated tests (T1 and T2) produced a higher ebb delta ruling
depth than the wave dominated tests (T3 and T4). This plot also showed that the depth over the ebb delta was shallower for both the artificially bypassed tests (T2 and T4).

Figure 11 shows the position of the ebb delta crest for the four inlet tests. This plot shows that the ebb delta crests were inshore in the current dominated tests and further offshore for the wave dominated tests.

Figure 9.0 shows the ebb delta growth rate measured and calculated for the 4 tidal inlet tests. The current dominated tests showed a greater growth rate and ebb delta volume over the same testing time period compared to the wave dominated tests. Artificial bypassing did not significantly influence growth rate of the ebb delta for the current dominated tests over the period of testing. Artificial bypassing did retard the growth of ebb delta for the wave dominated inlet tests.

Bypassing mechanisms observed were similar in all inlet tests as shown in Figure 15. All tests showed similar bypassing mechanisms once the material entered the inlet. On a flood tide the material was pushed into the inlet via current and wave pulsing. Figure 7.2 shows raw current velocity data from T1 which shows how the penetration of wave energy was increased during flood flows. The inflow of material formed a flood delta inside the up-drift training wall. When the tide changed direction the material was pushed out onto the outside up-drift edge of the ebb delta where it was trapped until the end of the ebb flow cycle. When current eased the wave action pushed the material across the top of the ebb delta onto bridging shoals to access down-drift beaches.

Figure 13 shows an isopach image representing the original moulded inlet profile subtracted from the final survey of tests T1 to T4. This gives an indication of the ebb delta development processes however the tests were not carried out over the same time intervals so the ebb delta extents are not indicative of growth rates.

Figure 14 shows wave diffraction observed between the training walls.

7 ANALYSIS OF MODEL OUTCOMES

The Physical Modelling project came up with a number of important outcomes.

- The comparison of the P vs. A for the model tests with Van Kreeke (1992) is shown in Figure 13. Gourlay et al. (1980) measured P vs. A parameters for low littoral drift coastlines and found them to possess relatively larger cross-sectional areas than O'Brien (1969) which was a basis for Van Kreeke (1992). A goal of this research was to investigate if artificial bypassing influenced the P vs. A relationship. Results suggest that in a stable inlet artificial bypassing does influence inlet channel configuration but does not influence the inlet cross-sectional area. However, inlet stability is dependent on sediment transport to the inlet. This is shown by comparison of T3 and T4 which shows the artificially bypassed inlet was maintained.

- Figure 12 shows the ebb delta volume vs. tidal prism plot for the 4 inlet tests in comparison to Walton and Adams (1976). Walton and Adams generated a relationship between ebb delta volume and tidal prism for a number of US inlets. Given that none of the inlet tests achieved an equilibrium profile, results suggest that ebb delta volumes will be significantly greater than predicted by Walton and Adams.

- The ebb delta volume analysis showed that for the wave dominated case the ebb delta growth was retarded by the artificial bypassing system. However,
neither test was carried out for long enough to see if artificial bypassing influenced the final equilibrium delta volume. Tests did show however that bypassing appears to be an effective way of managing beach amenity in the vicinity of tidal inlets while offering some respite to channel maintenance.

- The observations showed that cross-shore transport has a significant influence on sand supply to trained tidal inlet ebb delta.
- Measurements of ruling depth showed that the current dominated tests showed shallower ruling depth compared to the wave dominated cases. From observations this is due to current generated wave refraction. The Artificially bypassed tests also showed shallower depth over the bar crest. Observations suggest that this resulted from the lack of a sand bridge onto the ebb delta from the updrift beach. Both of these observations are shown in Figure 17.
- The ebb delta crest was measured to be much further offshore in the wave dominated case than the current dominated case. This is shown in Figure 11. This was an unexpected result that may have been caused by sediment transport being primarily bed load in the physical model in contrast to suspended sediment transport dominating in a prototype inlet.
- The cross-section area of T4 was relatively stable during the period of testing which suggests that artificial bypassing may be an effective tool to maintain unstable inlets.
- The inlet current conditions had no measurable influence on adjacent beach accretion and erosion trends.
- In all four tests natural inlet bypassing was observed through tracers and the development of down-drift bar systems well before the generation of an equilibrium ebb delta volume.
- The current dominated tests when compared to the wave dominated tests showed that ebb delta growth was much faster and produced larger deltas as observed at the prototype scale.
- In the artificially bypassed cases (T2 and T4) the bypassing system generated a rip cell with return flow along its axis. This rip cell as intended pulled material into it off adjacent beaches but when sand was removed the rip cell was observed to magnify in size and intensity and could be a pathway for leakage to the back of the ebb delta. This observation should be given further consideration in bypassed prototype inlets as it may provide a pathway for leakage to the ebb delta.

8 CONCLUSION/DISCUSSION

The purpose of this movable bed physical modelling project was to investigate key parameters of the growth and response of ebb tidal deltas at tidal inlets. Because of the limitations on movable bed physical models it is not practical nor sensible to extrapolate model outcomes to prototype scale processes however the model represents an insight into a complex system of waves, current and sediment processes that could not otherwise be gained without large scale expensive field monitoring exercises.

The construction and calibration of the idealised model system and the choice of appropriate modelling parameters took up a considerable percentage of the allocated project time due to the often subjective nature of movable bed physical modelling and the limited amount of past research in this area.
In total the physical modelling project was successful in achieving its objectives of:

1) producing a comprehensive dataset of measured inlet response for use in numerical model simulations and validation to extend understanding to the prototype scale.

2) investigating ebb delta development and beach response at the model scale which flagged the following outcomes that could have significance at the prototype scale inlet processes.

At the model scale:

- Artificial bypassing was effective in maintaining beach alignment in the vicinity of tidal inlets
- Artificial bypassing retarded the growth of the ebb tide delta for the wave dominated inlet.
- Artificial bypassing generated a large rip cell with a return flow along its axis. Observations showed when material was removed the rip cell magnified in intensity and could be a potential flow path for sediment leakage onto the back of the ebb delta.
- For the current dominated inlet tests it was found that the P vs. A relationship was independent of artificial bypassing. The inlet channel configuration varied but the cross-sectional area remained the same for the bypassed and unbypassed cases.
- Natural bypassing processes for all the inlet tests are defined in Figure 15.
- The two inlet tide conditions had no influence on the erosion and accretion trends on adjacent beaches.
- Ruling depths were higher for the current dominated inlets. Observations suggest that current generated wave refraction is responsible.
- Artificial bypassing resulted in slightly higher ruling depths. Observations suggest the lack of a sand bridge updrift of the ebb delta magnified wave refraction around the ebb delta.
- The observations showed that cross-shore transport has a significant influence on sand supply to trained tidal inlet ebb deltas.
REFERENCES


Yalin M.S. and Price W.A. (1975),”Time growth of tidal dunes in a physical model”, Symposium on Modeling techniques, 2nd annual symposium of waterways, Harbors and Coastal Engineering Division of ASCE, California 1975
Figure 1. Schematic of Physical Model
Figure 2a. Model construction commenced with shaping of fill

Figure 2b. QGHL staff, John Mohoupt and John Brinkley moulding concrete offshore profile.
Figure 2c. Beach profile moulded, catch tank basin on the left and tidal system weir and return channel at top.

Figure 2d. Inlet flood return channel under construction
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Figure 3.6. Rig used to measure beach profiles, ebb shoal detail and current velocities. This photo was taken during the straight beach longshore transport tests.
Figure 3.7 2D Acoustic Doppler Velocimeter (ADV) measuring model longshore velocities left and a Capacitance wave probe on the right.

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Figure 7.0 Moving average current velocities in the centre of the inlet channel measured in T1 and T2

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Figure 8.0 A sample of measured offshore wave conditions used for T1 to T4
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Figure 10.0 Ebb delta ruling depth plot
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Figure 12. Ebb Delta Volume vs Tidal Prism
Movable Bed Physical Modelling of Ebb delta Growth Characteristics

Figure 13. Tidal Prism vs Inlet cross-sectional area compared to Van Kreeke (1992)

Table 3. Parameters used in final inlet model testing

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Test 1 (T1)</th>
<th>Test 2 (T2)</th>
<th>Test 3 (T3)</th>
<th>Test 4 (T4)</th>
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<td>Significant Wave Height (m)</td>
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<td>Tidal period (min)</td>
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<td>Inlet discharge (m³/s) – ebb and flood</td>
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<td>Offshore water depth (m) –below SWL</td>
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Figure 14. Isopach Comparison of final survey of 4 inlet tests

Figure 15. Photo showing the wave refraction within the tidal inlet
Figure 16. The natural bypassing mechanism observed in the movable bed physical model tests.

Figure 17. Shows the RIP cell generated by the artificial bypassing system and how it was observed to magnify in size and intensity when material was removed.
Strong Ebb flow magnifies the wave refraction over the delta

Bypassed inlet doesn't allow a sand bridge to form

Unbypassed inlet forms a sand bridge onto the ebb delta

Longshore Transport Direction

Figure 18. Showing the observed wave refraction intensity of different modelling conditions
Figure A1.0
Figure A1.1
Figure A1.2
Figure A1.3
Figure A1.4
Figure A2.0  TEST:T1  Profile: PN
Figure A2.1  TEST:T1  Profile: PS

Figure A2.2  TEST:T1  Profile: T  (inlet centreline)
Figure A2.3   TEST:T1 Profile: Inlet cross-section (downdrift is positive chainage)
Figure B1.0
Figure B1.2

TEST: T2
DATE: 231002
ELAPSED TIME: 1664 min

0 min
416 min
946 min
1664 min
2080 min
Figure B1.3
Profile Location Diagram

Figure B2.0    TEST:T2    Profile: PN
Figure B2.1 TEST:T2 Profile: PS

Figure B2.2 TEST:T2 Profile: T (inlet centreline)
Figure B2.3  TEST:T2 Profile: Inlet cross-section (downdrift is positive chainage)
Figure C1.1
Figure C1.2
Figure C1.3

TEST: T3
DATE: 201102
ELAPSED TIME: 2080 min

0 min
416 min
832 min
1352 min
2080 min
2600 min
3120 min

Movable Bed Physical Modelling of Ebb delta Growth Characteristics
Figure C1.5
Profile Location Diagram

Figure C2.0   TEST:T3   Profile: PN
Figure C2.1  TEST:T3  Profile: PS

Figure C2.2  TEST:T3  Profile: T (inlet centreline)
Figure C2.3  TEST:T3 Profile: Inlet cross-section (downdrift is positive chainage)
Figure D1.0
Figure D1.1
Figure D1.3
Profile Location Diagram

Figure D2.0  TEST:T4  Profile: PN
D2.1 TEST:T4 Profile: PS

Figure D2.2 TEST:T4 Profile: T (inlet centreline)
Figure D2.3 TEST:T4 Profile: Inlet cross-section (downdrift is positive chainage)
APPENDIX A.

Moveable Bed Physical Model
Tidal Inlet

Design Report
THE PHYSICAL MODEL DESIGN

The initial focus on this project has been to develop a Coastal Movable Bed physical Model (CMBM) at Queensland Government Hydraulics Lab (QGHL) to investigate the development and response of ebb tidal deltas. An extensive literature review has been undertaken, dimensional analysis (attachment A) of contributing factors to the inlet system has been completed and a scaling table has been constructed (attachment B). This investigation has supported the outcome defined by past researchers that tidal Inlets are one of the most complex coastal systems and that Coastal Movable Bed Modelling is largely an “art” (Kamphius(1975)). Practical physical models cannot reproduce the complex sediment transport processes in a prototype physical system because the model sediment cannot be scaled down correctly from the prototype scale thus distortion of parameters is required in order to compensate for this lack of similarity. When distortion is required significant errors can be introduced and for this reason the interpretation of results are the most important part of a CMBM.

There are two types of CMBM they are (1) Prototype models which are used to define real world response and (2) Idealised models which are used to investigate complex systems with a simplified physical system. Prototype models primarily focus on specific sites and processes and use distortion of parameters in order to calibrate with usually limited prototype data. This method gives qualitative results, based on the assumption that because the model gives the same end condition as the prototype it will respond the same to testing conditions. When using this modelling method great care is required in interpreting results because the model does not reproduce prototype physical processes. The uncertainty in this modelling method and the expense of CMBM’s is a primary reason why they are not widely used in coastal engineering design.

Idealised models are important tools in understanding complex systems and are often used to form empirical relationships and calibrate numeric models. It is important to note however that idealised models are a simplified physical system not a representation of the prototype situation so again results must be carefully interpreted for application to prototype situations. A short database of past research experiences of coastal movable bed modelling have been tabulated in attachment C.

THE PROPOSAL

The proposal put forward is to produce an undistorted scale physical model as large as possible in the facilities available and primarily treat this model as a idealised tidal Inlet. This model shall produce all engineering processes (e.g. reversing tidal flow, longshore sediment transport and wave attack) however detailed sediment transport processes shall not be reproduced accurately. The modelling method is after the “sand model” of Kamphius(1975) and uses a similar modelling methodology to Seabergh(1999). The primary downfall of the model is that the surfzone sediment transport is modelled primarily as bedload. The decision to go with this bedload model was made because
the modeller can only model bedload or suspended sediment transport, in choosing suspended sediment transport the model would require significant geometric distortion and all areas outside the surfzone would not be modelled closely. The modelling method chosen will generate slower sediment transport, however it is considered from measured field inlet relationships (Jarrett(1976), Walton and Adams(1976), that bedload dominated forces show a strong correlation with ebb delta response. The primary benefit of the proposed modelling methodology is its flexibility in testing options and its simplicity for the purposes of result interpretation.

There are many different types of tidal inlets that could be investigated as part of this project. The initial research shall be based on the following inlet type;

1. Inlets with two training structures
2. Inlets on exposed high wave energy coastlines
3. Inlets on open coasts without other influencing structures (eg headlands)
4. Inlets with dominant fine sand sediment $d_{50} = 0.2-0.3\text{mm}$
5. Inlets in semi-diurnal tidal environments

**METHODOLOGY**

The basis of the methodology is that in this model we can control the inlet system tidal flow and longshore sediment transport and therefore define relationships between these parameters and the ebb delta growth/response and adjacent beach response. This methodology is further strengthened by maintaining measured field inlet relationships in the model and validating the model against a prototype event in order to link the model to a real world inlet. The very least we can gain is an extensive model data set of a simplified inlet that can be used for numerical model calibration. Another benefit of the modelling method is it will generate an accurate 3D hydrodynamic model of a tidal inlet. The reproduction of detailed hydrodynamic processes in an inlet by themselves would be advantageous in the calibration of a numerical model. The model project aim is to generate a “quasi – equilibrium ebb delta” as described by Walton and Adams(1976) and measure its features; growth rate, plan form and ruling depth and its response and the response of adjacent beaches to; artificial bypassing, delta dredging, changes to tidal prism (tidal flow) and changes to inlet aspect ratio.

Following the initial set of tests, specific testing can be undertaken to investigate further areas of importance. In particular further numeric model simulations may be run to refine numeric model predictions of the simplified physical system response. The flexibility in the model allows a great variety of additional research scope and even could be used to investigate specific inlet systems. Further research possibilities defined are highlighted in attachment D.

The real challenge of this project is to link the physical model results to real world inlets. As explained earlier CMB Modelling is and “art” not an exact science so the interpretation and understanding of results is very important in extending results to real world inlets. In order to better understand the limitations of the model the project aims to calibrate and validate the physical model. Three possible methods are proposed;

1. The model shall be initially run as a simple beach and the longshore transport rate shall be measured for specific wave testing conditions. This transport rate shall then be compared to calculated prototype transport rates in order to define an estimate of the morphological time scale in the model. The transport processes in the model will be primarily via bedload, and the prototype is a mixture of suspended sediment and bedload. Hence this calibration is not quantitatively correct, however a qualitative result should be gained.
2. The second method of calibration shall be via prototype data sourced from the Brunswick River Inlet. A dredging campaign of the ebb delta was undertaken in September 1996 and a survey was taken 4 months later. This prototype data is
limited however it is hoped that another qualitative indication of morphological time scale can be defined. Details of the prototype data can be found in attachment E.

3. Due to the flexibility in the model another calibration method may be used if required via the use of a scale series of models. This method would include the construction of two models of different scale in the basin with specific testing undertaken for each. Results would then be compared to aid the understanding of scaling errors between the models and thus scaling errors between the model and the prototype.

In calibration of the model the testing methodology proposes to maintain proven prototype inlet relationships that are define in more detail in attachment F. Holding these proven inlet relationships will provide an additional calibration tool.

**Flexibility in the Model**
The flexibility in the model allows;
1. Investigation of a geometrically distorted suspended sediment transport model using the empirical scale criteria of Vellinga(1982).
2. An estuary or a bay can be reproduced in the model
3. Inlet W/D relationship can be varied via movement of the downdrift training wall and excavation of the channel sand.
4. The model can be tested purely as a fixed bed froude hydrodynamic model
5. Tracer studies could be undertaken in the model facility
6. Wave attack angle can be 10, 20, 30 degrees SE and a single paddle can be used to simulate a NE wave attack angle.
7. The steep concrete beach design with sand filled to the profile will facilitate the investigation of natural untrained inlets and breakout processes.

**Model Specifications**

**Type:** Undistorted, short wave model, bedload dominated,

**Scale:** 1:50 (2m maximum inlet width)

**Prototype:** Idealised (but size of Brunswick River Inlet, NSW)

**Model layout** – The complete layout of the model can be viewed as an AutoCAD image attachment G.

**Sediment:** sand $d_{50} = 0.188$

**Hydrodynamic time scale** – Froude $N_t = NT = 7.07$

**Current Velocity scale** – distorted to reproduce required bed shear stress $N_v = 2.89$, 2.4 times greater than froude, therefore inlet velocity of 0.415m/s is required for a 1.2m/s maximum current measured at Prototype Inlet. $Q_{model} = 0.083m^3/s$ (max lab discharge capacity ~0.1m^3/s.)

**Tidal period** 102 minutes from froude

**Depth of closure** $h_p = 12m$, $h_m = 0.24m$ (This is the extent of survey data, also no definable change in bathymetry measured below 10m depth) Model bed starts at −15m AHD
Equilibrium Profile concrete – Beach Profiles from Brunswick Heads and the Gold Coast Beaches were compared and the following baseline profile was assumed to allow flexibility in the model. A 0.2m wide walkway will be present adjacent to the bay area at the back of the beach. This shall be 0.75m above the floor level 0m. Concrete will start in front of the walkway at 0.55m, and be horizontal for 1m. At the 1m point an equilibrium profile(A=0.24) shall extend to the closure depth of 0.25m. At this point a flat bed will be in place until the wave transition slope near the paddles graduates to 0m. The transition slope will be constructed of gravel material to allow flexibility in changing wave angle of attack.

Inlet depth = 0.2m(Prototype -10m AHD) cut into the entrance below MWL then layered with sand 0.1m thick on the channel bed. This allows flexibility in testing inlet aspect ratios and allows considerable leeway for inlet scour testing.

Sand Volume: Because of the flexibility built into the model(with the ability to reproduce a natural inlet) it will require ~30 cubic metres of model sediment to fill to the undistorted profile and generate the required alongshore transport.

Beach angle – 20 degrees, This is an angle that we know will produce longshore transport. The model has been constructed to allow testing with 10,20 and 30 degree wave angle by the movement of the wave paddles. A single wave paddle will also be available to generate a NE wave direction for limited testing directly on a formed ebb shoal.

Wave Conditions
Irregular Jonswap
Prototype data measured at Cape Byron, Hs (mean energy) = 1.72m T =10s, max Hs = 4.0m, T=8s
Therefore Model Hs mean = 0.034 T =1.4s, max = 0.08, T=1.13s, will be used in the model for initial testing.

Hs = 0.034m T=1.4, Model Mlong = 14.15 kg/hour = 0.043 m³/8 hour

Estimated morphological time scale – by comparing the above estimation with a calculation using the actual prototype data wave conditions a morphological time scale of tm ~ 4 was defined.

Permeable breakwaters will be used in construction. The breakwaters will be 0.2m above MWL which is 0.75m above the paddle level at 0m. The breakwaters shall be mobile in order to test different configurations.

Testing Methodology

The testing methodology is defined in detail in testing methodology spreadsheet see attachment H, a summary is shown below;

- Construction of model and calibration of all measuring equipment will be undertaken. Preliminary testing without sand shall be undertaken of working model.
• The model shall be run as a hydrodynamic model and measurements will be taken to calibrate with Delft3D (FLOW and WAVE).
• The model will be run as a simple beach as done by Kamphius and Kooistra (1990) and longshore transport rate measured for specific wave conditions. This will generate empirical relationships for longshore transport in the model. Measurements will be taken on the simple beach for use in Delft3D calibration.
• An inlet shall be installed offset towards the updrift side of the beach and ebb shoal formation will be generated via tide and wave action. The model will be calibrated in order to generate a practical workable model.
• Model validation shall then be attempted with Prototype data from the Brunswick entrance and if required from scale series tests. (time dependent)
• The model shall then be tested for the following responses:
  1. The growth rate, plan form, ruling depth and adjacent beach response of the ebb delta shall be measured in the model for specific wave and tidal conditions.
  2. The influence of sand bypassing shall be measure on the ebb delta and the adjacent beach.
  3. Ebb delta recovery from dredging shall be investigated, via adjacent beach measurement and ebb delta measurement.
  4. The influence of changes to the tidal current flow will be measured on the ebb delta and adjacent beaches.
  5. The Inlet aspect ratio will be adjusted to investigate ebb delta and adjacent beach response.
  6. Empirical Inlet relationships defined in the past shall be tested against physical model results. e.g. Floyd (1968) and Walton and Adams (1976)
• A Delft3D model shall be set up at the physical model dimensions and the physical and numeric model will be used together in the development of a better numeric model prediction giving careful considerations to the limitations of both modelling methods.

Following the completion of the initial testing further testing will be undertaken in line with the findings of initial testing.

REFERENCES


ATTACHMENTS
CRC INLET MODEL DESIGN – Similarity Criteria

DIMENSIONAL ANALYSIS

The objective of this project is to investigate contributing factors to the formation and response of ebb shoals at tidal inlets and their associated impact on adjacent beaches.

The design process begins by outlining the contributing parameters to ebb shoal formation and response at tidal inlets. This will show exactly what is being modelled and assist with interpretation of results. These factors can be broken into two areas (1) Hydrodynamic similitude and (2) sediment motion similitude.

Using the continuity equation from Hughes (1993) the following common physical similarity criteria for waves and currents can be defined;

1. Froude Criteria (gravity forces/inertial forces) \( \frac{N_v}{(N_g N_L)^{1/2}} = 1 \)
2. Reynolds Criteria (viscous forces/inertial forces) \( \frac{N_v L N_b}{\rho / \nu} = 1 \)
3. Weber Criteria (surface tension force/inertial forces) \( \frac{N_{b s}}{(N_v)^2 N_b L / \sigma} = 1 \)
4. Cauchy criterion (Elasticity forces/inertial forces) \( \frac{N_{b s}}{(N_v)^2 N_b L / E} = 1 \)
5. Euler Criterion (pressure forces/inertial forces) \( \frac{N_p / N_b L v^2}{N_{b s}} = 1 \)
6. Strouhal Number (temporal inertial force/convective inertial force) \( \frac{N_s / N_b N_t}{N_b L} = 1 \)

These values have physical basis and can now be used in dimensional analysis to define the remaining similarity conditions.

We wish to model correctly the movement of sediment as a result of the interaction of waves and currents for this reason we have selected the following independent parameters (Kamphius (1985), Dalrymple (1989)).

- \( H \) = wave height
- \( T \) = wave period
- \( L \) = wave length
- \( X \) = horizontal length scale
- \( Z \) = vertical length scale
- \( t \) = time scale
- \( h \) = local water depth
- \( g \) = acceleration due to gravity
- \( k_s \) = bottom roughness
- \( \rho \) = fluid density
- \( \nu \) = fluid kinematic viscosity
- \( d \) = sediment diameter
- \( \rho_s \) = sediment density
- \( \tau_b \) = bottom shear stress (link between fluid and sediment)
- \( \alpha \) = sediment particle shape
- \( p \) = external pressure
- \( \sigma \) = surface tension
- \( E \) = Elasticity

From literature it has been defined that the sediment fall speed is an important parameter which will be part of the scaling however it is not in the initial list because it is not an independent parameter.
\[ \omega = \text{fall speed velocity} = f(d, \alpha, \rho, \rho_s, \nu) , \] because all parameters of this function can be assumed to be constant for an individual sediment and testing fluid, in development of the dimensionless parameters we will insert \( \omega \), in the place of the grain shape factor.

**Using the Buckingham \( \pi \) Theorem**

Basic units include Mass(M), Length(L) and Time(T) – \( n = 3 \)

From above there are 18 independent variables – \( m = 18 \)

Therefore there must be 18-3 = 15 dimensionless parameters.

In a similar way to Kamphius(1985) we will divide the parameters into Hydrodynamic parameters and Sediment motion parameters.

**HYDRODYNAMICS SIMILITUDE**

\[ \Pi_H = f \left( \frac{H}{L}, \frac{h}{L}, \frac{x}{L}, \frac{z}{L}, \frac{k_s}{L}, \frac{T}{t}, \sqrt{\frac{pt^2}{\rho L^2}}, \frac{\rho L^3}{\kappa t^2}, \frac{t(g/L)^{1/2}}{\rho L^2/Et^2}, \nu / L(g L)^{1/2} \right) \]

- The first four dimensionless parameters are satisfied for a geometrically undistorted model. If the vertical dimension is distorted scale effects are introduced to hydrodynamic response in particular phenomena acting in a vertical and horizontal plane (e.g. wave diffraction)
- \( k_s/L \) is only in similitude if model boundary roughness is the same as the prototype. Initial flume tests show that this will not be the case, firstly because the model sediment is not scaled down and secondly because considerable ripples form in the sediment under wave action and under current. This scale effect can be minimised by keeping conditions rough turbulent in the model.
- \( T/t \) this parameter defines that the hydrodynamic time scale must be equal to the wave period oscillation in order to negate scale effects. Hence only short waves OR long waves (tides) can be modelled in similitude in a single model. In our case we have chosen to model as a short wave model with a tidal current superimposed.
- \( t(g/L)^{1/2} \) this parameter is a form of wave propagation due to gravity (gravity balancing inertia forces) and hence to be correctly simulated the flow Froude Criteria must be met.

\[ N = \left( \frac{N_i}{N_v} \right)^{1/6} = 1 = \left( \frac{1}{N_v} \right)^{1/6} \]

- \( N_i/N_v \) is a form of the Euler Number, external pressure forces balancing inertia forces, for free surface flow this term can be neglected because external pressures are negligible.
- \( \rho v^2/\sigma \) is the form of the Weber Number that balances surface tension with inertia forces. This parameter controls how small the model scale can be before scale effects become significant. It is assumed that surface tension effects can be neglected when Wave length >0.12m De Vries(1982), Le Mehaute(1976) \( T > 0.35s, h > 0.02m \).
• \( \rho v^2/E \) Cauchy Number, Elastic forces balancing inertia forces, Model fluids are usually assumed incompressible and therefore scale effects are neglected. However scale effects do exist as a result of temperature changes and as a result of excess air entrainment (due to wave breaking) in the model relative to the prototype.

• It is also noted that scale effect occurs with the use of fresh water in the model when the prototype is Salt water. This error has been defined in Le Mehaute (1976) as up to ~3%.

Following all these considerations it can be concluded that the best way to hold similitude for hydrodynamic processes is to use a geometrically undistorted model and use Froude scaling criteria for the Hydrodynamic characteristics, (because gravity restoring forces dominate). The scale effects associated with this decision are reasonably well understood;

• Viscous effects
• Bed Roughness
• Long wave effects
• Air entrainment , temperature changes
• Fluid density differences
• Surface tension

Now that we can model hydrodynamics the next task is to simulate sediment motion in the model.

**SEDIMENT MOTION SIMILITUDE**

Dalrymple(1989) similar to Kamphius(1985), but includes the Dean number for sediment fall speed similitude.

\[
\Pi_{sm} = g \left( \frac{\nu d}{\nu}, \frac{\rho v^2}{\gamma d}, \frac{\rho_s}{\rho}, \frac{H}{\omega T} \right) (u^*/\omega)
\]

1) The first parameter is Grain size Reynolds number which represents the viscous component of incipient sediment motion. This parameter along with the densimetric froude number makes up the Shield’s Parameter for incipient motion of bedload sediment transport. Again because water is used in both the model and prototype the viscosity ratio is 1. Therefore the only way to achieve similarity is via the use of a large diameter light weight sediment. Experienced researchers suggest against the use of lightweight sediments in Coastal Movable Bed Models. Scale effects related sediment motion are significant.

2) The second parameter is the Densimetric Froude number which represents the gravity component of incipient sediment motion. And as defined above this parameter is a primary requirement for bedload sediment transport due to shear stress. The practical difficulty in achieving similarity is the requirement that the sediment diameter must be scaled down with the geometric length scale.

3) The third parameter is Relative density between sediment and fluid. Because our prototype sediment is sand, if we use sand in the model and the model fluid is water we are conserving this parameter. (However the minor scale effect associated with salt water in the prototype must be noted)

4) The final parameter(1) is the sediment Fall speed parameter. Studies have shown that this parameter is very significant in coastal modelling because it allows similitude of suspended sediment transport due to waves, which is the primary mode of transport in the surf zone. Several researchers (Dean(1977),
Gourlay (1980) have shown that surf zone turbulent conditions are best modelled maintaining this parameter as a dominate scaling criteria. Unfortunately except at very large scales or with the use of lightweight sediment this parameter cannot be modelled in an undistorted model.
The final parameter (2) is the shear velocity/fall velocity relationship which accommodates suspended sediment transport due to shear stress.

In order to have complete similarity between model and prototype all the above dimensionless parameters must be equal between model and prototype ie there ratios equal to 1. As defined extensively in literature very few of the dimensionless parameters can be held in similitude.

All past experience with Coastal Movable bed modelling involved minor similarity (if any) with physical laws and distortion of the geometric scale or hydrodynamics (via empirical methods or maintaining important relationships) in order to achieve calibration of a prototype event.

It appears the modeller must begin by building a model that will fit into the boundaries and limitations of the facilities, equipment and sediment available. Then chose the most important parameters and go through a systematic process of logical steps in order to generate the desired outcome. Interpretation of results correctly being the primary task.

From Hughes (1993) the modeller has 4 ways to achieve similitude in a Coastal Movable Bed Model.
1. Via dimensional analysis – It is obvious from the above information that there is very little conservation of similitude using this method and can in no way be relied on by itself.
2. Similitude via calibration with prototype data – This is possible for the current project however the detail of prototype data is not available and even with calibration, this does not define that the physics are the same in the model and the prototype, just that given systematic distortion the model has achieved a prototype outcome.
3. Similitude via a scale series – By carrying out a number of different scale tests we may achieve a better understanding of scale effect. This has a lot of merit however this project doesn’t really have the resources to do several tests.
4. Similitude via known physical relationships – This is a good option for the present project.

This project proposes to use several of these methods in order to better understand scale effects in the model and so be better equipt for interpretation of modelling results and their relationship to prototype inlets.
## Undistorted Physical Model Design - Scaling requirements

Kamphius (1991) "sand Model", undistorted, shortwave, bedload dominated.

<table>
<thead>
<tr>
<th>Prototype</th>
<th>Model</th>
<th>Scale factor</th>
<th>lab limits</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>X Horizontal Scale</td>
<td>1</td>
<td>50.000</td>
<td>50.000</td>
<td></td>
</tr>
<tr>
<td>Z Vertical Scale</td>
<td>1</td>
<td>50.000</td>
<td>50.000</td>
<td></td>
</tr>
</tbody>
</table>

### Gravity

| g (assume constant) | 10 | 10.000 | 1.000 |

### Sediment: (sand)

| d50 | 0.000266 | 0.00019 | 1.400 |
| u | 0.09 | 0.039 | 1.429 |
| ω | 0.238 | 0.211 | 1.128 |
| submerged angle of repose | 35.2 | 39.000 | 0.903 |

Bed permeability has been shown to be an important parameter, however a very large scale model must be constructed to study the influence of permeability, on beaches alone would be a good start. At the scale chosen a much larger particle would be required than the prototype particle to reproduce permeability in scale because of surface tension and viscous scale effects. Hence permeability of the model sediment was not found and scale effects associated with permeability must be accepted.

| ks ripple bed | 0.005 | 0.003 | 1.667 |
| Ks flat bed = d | 0.000266 | 0.00019 | 1.400 |

| Waves: Irregular |
| Hs mean (m) | 1.72 | 0.034 | 50.000 |
| Hs max (m) Prototype data | 4.5 | 0.090 | 50.000 | 0.120 |
| Tp max wave (s) | 9.5 | 1.344 | 7.071 |
| Tp max period (s) | 15.1 | 2.135 | 7.071 |
| L wavelength | 50.00000 |
| direction (degrees) from East | 10.20 | 20.000 | 1.000 |
| spectrum JONSWAP | The wave spectrum usually used at GGGHL, no further research undertaken into wave spectrum. |

### Tidal Current

<p>| fluid density (kg/m3) (10 degrees) | 1003 | 1000.000 | 1.003 |</p>
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value 1</th>
<th>Value 2</th>
<th>Value 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kinematic viscosity (10)</td>
<td>0.0000014</td>
<td>0.00000131</td>
<td>1.069</td>
</tr>
<tr>
<td>Dynamic viscosity (25)</td>
<td>0.0000891</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hydrodynamic time scale</td>
<td>43200</td>
<td>6109.403</td>
<td>7.071</td>
</tr>
<tr>
<td>Proude Criteria Velocity (current)</td>
<td>1</td>
<td>0.141</td>
<td>7.071</td>
</tr>
<tr>
<td>Velocity distortion (Kamphius), bed shear stress m/s</td>
<td>1</td>
<td>0.331</td>
<td>3.021</td>
</tr>
<tr>
<td>Vmax, Equilibrium Discharge Relationship</td>
<td>1.2</td>
<td>0.649</td>
<td>1.850</td>
</tr>
<tr>
<td>Velocity of interest</td>
<td>1</td>
<td>0.331</td>
<td></td>
</tr>
<tr>
<td>Inlet depth</td>
<td>5</td>
<td>0.100</td>
<td></td>
</tr>
<tr>
<td>Inlet width</td>
<td>100</td>
<td>2.000</td>
<td></td>
</tr>
<tr>
<td>Inlet Area</td>
<td>500</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>Discharge (m3/s)</td>
<td>500</td>
<td>0.0662</td>
<td>0.100</td>
</tr>
<tr>
<td>h (maximum)</td>
<td>1</td>
<td>0.020</td>
<td>50.000</td>
</tr>
<tr>
<td>Tidal signal (saw tooth flow)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Optimum inlet depth</td>
<td>3</td>
<td>0.060</td>
<td>50.000</td>
</tr>
<tr>
<td>Equilibrium Beach Profile</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Profile shall be a gangway 0.2m at back (0.75m high), 0.2m below top of gangway we will have a flat section 1m long at head of flat section a equilibrium profile (A=0.24) shall extend to the zero chainage point 0.1m below the proposed MWL, at this point a linear slope shall extend to the depth of closure, beyond the depth of closure a linear slope to bed 0.3m. Then a 0.25m transition slope shall be installed before to the wave paddles.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth of closure</td>
<td>12</td>
<td>0.240</td>
<td>50.000</td>
</tr>
<tr>
<td>Beach angle (deg.)</td>
<td>20.000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand quantity required</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Morphological time scale</td>
<td>~4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Im (Model Calibration)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longshore Transport</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kamphius</td>
<td>0.043 m3/day</td>
<td>(rough estimate - 113kg/day(8 hours))</td>
<td></td>
</tr>
<tr>
<td>Model slope undistorted</td>
<td></td>
<td>1:25</td>
<td></td>
</tr>
<tr>
<td>Breakwaters</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Create as permeable structures</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inlet Cut (below MWL)</td>
<td>5</td>
<td>0.100</td>
<td>50.000</td>
</tr>
<tr>
<td>Kinematic Similarity sediment</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Some interest in diffraction characteristics, even though viscous scale effects will remove any quantitative assessment.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Allow 0.1m depth of sediment above concrete base, therefore depth 0.2m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>This also allows flexibility in W/D relationships, and testing of equilibrium scour trends.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Parameter</td>
<td>Value</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-----------------------------------------------</td>
<td>---------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear velocity (long waves) ( u_2 )</td>
<td>4.622</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear velocity (short waves) ( u_1 ) -</td>
<td>1.975</td>
<td></td>
<td></td>
</tr>
<tr>
<td>undistorted</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Dimensionless Parameters

#### Hydrodynamics

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( H/L ) (wave steepness)</td>
<td>1.000</td>
</tr>
<tr>
<td>( h/L )</td>
<td>1.000</td>
</tr>
<tr>
<td>( X/L )</td>
<td>1.000</td>
</tr>
<tr>
<td>( Z/L )</td>
<td>1.000</td>
</tr>
<tr>
<td>( k_0/\lambda )</td>
<td>0.033</td>
</tr>
<tr>
<td>( T/t ) short wave model</td>
<td>1.000</td>
</tr>
<tr>
<td>( t(g/L)^{1/2} ) flow froude</td>
<td>1.000</td>
</tr>
</tbody>
</table>

**Reynolds Number Value; ensure turbulent**

\[ Re = \frac{\nu}{\nu} \]   

\[ \frac{\rho u^2}{\gamma} \]   

\[ \frac{\rho s}{\rho} \]   

\[ \frac{H}{\alpha T} \]   

\[ \frac{u}{\alpha} \]   

**Bed Ripple scale effect, also sediment is not scaled down correctly.**

**Viscous scale effects, Big issue when considering sediment transport.**

**Given the velocity required to gain proportionality between waves and currents maintaining turbulent conditions should be reasonably easy.**

**Given depth of model and wave size, neglible scale effect**

**Technically not incompressible because of air entrainment in wave breaking processes.**
### Past Movable Bed Modelling (MBM) experience

<table>
<thead>
<tr>
<th>Organisation / Researcher</th>
<th>Project</th>
<th>Year</th>
<th>Basin Size</th>
<th>Scale Geometry</th>
<th>Distortion</th>
<th>Time Scale</th>
<th>Sediment</th>
<th>Slope</th>
<th>Wave Scale</th>
<th>Waves</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>GITI (Old US Army Corps)</td>
<td>An evaluation of Movable Bed Total Inlet Models</td>
<td>1979</td>
<td>2g X 50</td>
<td>1:200-500</td>
<td>1:150-500</td>
<td>0.19-0.32 (sand)</td>
<td>0.18-0.25</td>
<td>L/T</td>
<td>regular</td>
<td>Highly Distorted models</td>
<td></td>
</tr>
<tr>
<td>GITI (Old US Army Corps)</td>
<td>Masonboro Inlet</td>
<td>1977</td>
<td>2g X 46</td>
<td>1:60</td>
<td>0.25</td>
<td>0.13</td>
<td>T</td>
<td>1:7.73</td>
<td>regular</td>
<td>Fixed bed model, basically a hydraulic model, defined that processes oceanward of the entrance needed to model waves correctly</td>
<td></td>
</tr>
<tr>
<td>GITI (New US Army Corps)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Current Dominated</td>
<td></td>
</tr>
<tr>
<td>Seaberg (1995)</td>
<td>Large scale idealised inlet, constructed</td>
<td>1999</td>
<td>46 X 99</td>
<td>1:25</td>
<td>0.34</td>
<td>T</td>
<td>1:7.07</td>
<td>regular</td>
<td></td>
<td>Genetic Entrance, for general testing</td>
<td></td>
</tr>
<tr>
<td>Delft - 74 Naranj</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>wave height, orbital motion and current velocity remained fixed to be changed to reproduce morphological conditions</td>
<td></td>
</tr>
<tr>
<td>Wang (1993)</td>
<td>Movable Bed Modeling of Sebastian Inlet, Florida</td>
<td>1993</td>
<td>30 X 34</td>
<td>1:60</td>
<td>1:40</td>
<td>0.35</td>
<td>T</td>
<td></td>
<td>Prototype waves 2m, tidal current velocity 1.3-2.0ms, used Wang's scaling law using fall speed parameter and distortion. Used adjacent beach response as the primary scaling criteria</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wang (1996)</td>
<td>Idealised inlet - Investigation</td>
<td>1996</td>
<td>26 x 30</td>
<td>1:200</td>
<td>0.39</td>
<td>0.19</td>
<td>wave Ht (QH^2)</td>
<td>Troude</td>
<td>Troude</td>
<td>An experimental inlet study, Morphological time scale was given trouble scaling also, wave height is distorted relative to the vertical and horizontal scale?</td>
<td></td>
</tr>
<tr>
<td>Kamphius and Larson (1987)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Volume of erosion was proportional to: wave steepness, wave length with respect to island size, excess shear stress and a grain size related parameter representing sediment suspension and bottom bedform</td>
<td></td>
</tr>
<tr>
<td>Deguchi, Shi and Sawaragi (2000)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Investigated influences of height of bar, river discharge and rate of increase of discharge on flushing of the bar.</td>
<td></td>
</tr>
<tr>
<td>Kamphius and Larson (1987)</td>
<td>Movable Bed Experiments of Sebastian Island</td>
<td>1987</td>
<td>22 x 29</td>
<td>200, 100, 75, 50</td>
<td>1</td>
<td>0.96, 0.1, 0.1</td>
<td>regular</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fowler and Hughes (1991)</td>
<td>Beach Fill Erosion at Ocean City, Maryland - Beach Profile Modelling Guidance, Turbulence dominated, erosion by waves</td>
<td>1991</td>
<td>20 wave flume 2m x 75m</td>
<td>1:100</td>
<td>0.12</td>
<td>0.13</td>
<td>4 Beach</td>
<td>Troude</td>
<td>Troude</td>
<td>Turbulence-dominated sediment transport has been validated for surf zone modelling, a flume model was tested again another. Scaling undertaken by fall velocity parameter criterion</td>
<td></td>
</tr>
<tr>
<td>Hughes and Fowler (1990)</td>
<td>Validation of Movable Bed Modelling Guidance, Turbulence dominated, erosion by waves</td>
<td>1990</td>
<td></td>
<td></td>
<td></td>
<td>0.225</td>
<td>1:7</td>
<td></td>
<td>Troude</td>
<td>Troude</td>
<td></td>
</tr>
<tr>
<td>Deguchi and Sawaragi (1988)</td>
<td>Effects of structure on deposition of discharged sediment around inlets</td>
<td>1988</td>
<td>15</td>
<td>0.35 and 0.15</td>
<td>0.46</td>
<td>0.22</td>
<td>Troude</td>
<td>Troude</td>
<td>Troude</td>
<td>Tested different structures, Lb and Le, Ls = length of structure, Lc = wave length(deep water), B = depth</td>
<td></td>
</tr>
<tr>
<td>Kwon (1975)</td>
<td>Lab Tests in Coastal Movable Bed Models - Beach profiles</td>
<td>1975</td>
<td>32</td>
<td></td>
<td>0.21</td>
<td>1:20</td>
<td></td>
<td>Troude</td>
<td>Troude</td>
<td>Final profiles are affected by inlet profile shape</td>
<td></td>
</tr>
<tr>
<td>Hughes and Schwennetberg (1989)</td>
<td>Current-induced scour at Ventura Harbor</td>
<td>1999</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Troude</td>
<td>Current dominated, calibration was achieved by adjusting flow discharge until equilibrium scour depth was achieved from the prototype. Distorted the velocity scale</td>
</tr>
<tr>
<td>Dalrymple, Loo and Oyer (1999)</td>
<td>Waves and Wave Induced Flows in Jetted Flumes</td>
<td>1999</td>
<td>0.32 x 44mm</td>
<td>1:30</td>
<td>0.19</td>
<td>0.13</td>
<td>T</td>
<td>T</td>
<td>T</td>
<td>T</td>
<td>T</td>
</tr>
<tr>
<td>Wernli and Talsma (1989)</td>
<td>Definition of time scale for beach profile development</td>
<td>1988</td>
<td>28 x 0.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>T</td>
<td>Troude</td>
</tr>
<tr>
<td>Vitek, Wheaton and Guanzalma (2000)</td>
<td>Sediment and for Movable Bed Simulation</td>
<td>2000</td>
<td>33 x 3.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>T</td>
<td>Troude</td>
</tr>
<tr>
<td>Turner, Leyden, Cox, Jackson and Mograh (1999)</td>
<td>Evaluation of Lab and Scale effects on a 3D Movable-bed Model - temporary groin structure</td>
<td>1998</td>
<td>33.5 x 3.5</td>
<td>1:20</td>
<td>0.20</td>
<td>0.13</td>
<td>T</td>
<td>Troude</td>
<td>Troude</td>
<td>Hybrid fixed bed tracer model, granulated PVC used as sediment material. Used an offshore breakwater (field) relationship to verify the real model from Hu and Silvester (1990)</td>
<td></td>
</tr>
</tbody>
</table>
FURTHER RESEARCH OPPORTUNITIES

1. Investigate the influence of different wave angles, including NE wave direction
2. Investigate a composite wave signal eg storm events vs average energy conditions
3. Investigate the influence of W/D relationships on ebb delta development
4. Test a river flooding event, measure bar response and recovery
5. Test a Natural entrance, and attempt to reproduce downdrift spit migration and breakthrough cycle
6. Investigate inlet closure and breakout processes
7. Maintain similarity of equilibrium discharge relationships to investigate processes within the inlet(Hughes(2000)).
8. Run two scales in the model to better understand scale effect
9. Investigate different training wall configurations
10. Investigate the influence of diffraction as a result of permeable breakwaters, generation of secondary currents
11. Investigate the influence of wave setup differential between beach face and the inlet on currents and hence the ebb delta
12. Investigate field measured inlet relationships with the idealised model and try to determine primary contributors – for use in Delft3D modelling
13. Install a headland updrift of the inlet like many on the coast and observe transport characteristics
14. Investigate the influence of changing inlet length
15. Test the response of dumping dredged material on the downdrift beach with associated transport processes.
PROTOTYPE DATA ANALYSIS

The proposed inlet investigation wished to reproduce a prototype inlet and then measure specific parameters relating to ebb delta and adjacent beach response. Having completed the dimensional analysis and constructed a practical design it is obvious that reproduction of a prototype inlet is not possible and the project must be satisfied with a conceptual tidal inlet model. Given the practical difficulty in producing measurable longshore transport and a proportional ebb and flood tidal flow, a sufficient outcome would be to model the inlet as a prototype and use the results directly to calibrate a numerical model.

This result would be advantageous to better understanding inlet processes, however a gap must still be bridged between the physical model and a prototype inlet system.

In order to stretch the project further we must validate the physical model in order to gain an understanding of the introduced scale effects in the model. There are two possible ways in which to achieve this:

1. Use prototype data. In order to achieve some indication of morphological time scale. At the same time maintaining important inlet relationships
2. Produce a scale series of models. In order to understand better the scale effect associated with the model. At the same time maintaining important inlet relationships.

The model design has been made at as large an undistorted scale as possible under the constraints of the modelling facility, in order to minimise scale effects. This model is a 1:50 scale of the Brunswick River Entrance in Northern NSW. As with almost all other movable bed modelling projects there is limited prototype data to work with and simplifying it into something that can be used in the model is difficult. This chapter will describe the methodology behind the simplification of prototype data.

Brunswick Prototype Data Manly Hydraulics Laboratory and NSW Department of Land and Water Conservation. (September 1996-March 1997)
Wave Data – Hs and Tp, non-directional
Tide Data – tide levels 150m upstream of Brunswick Inlet
Bathymetry Surveys;
- September 1996 (post ebb shoal dredging program)
- March 1997 (Ebb shoal reformed)
- General bathymetry survey PWD(1989)

Chris Voisey Undergraduate Thesis
- Inlet current velocity measurements – 24/9/1996 tidal cycle, spring tide
- 3 dune top to –1m beach transect surveys – south of inlet

Tweed region, Wave data recording program 1995-1997, Department of Environment(EPA).
- Directional wave data for measurement period

BATHYMETRY

Beach Profiles:
Because the aim of this project was to produce an idealised model it was proposed to produce a generic beach profile. The primary objective of the beach profile was to facilitate all proposed testing profiles and not expose the fixed bed.
For this reason the available beach profiles from Brunswick south beach and Gold Coast beaches were plotted on the same chart. This defined that all profiles were relatively similar. Kamphius et al(1990) used 0.1m thickness of sand on his movable bed profiles and with this thickness in the nearshore zone all possible events could be allowed for with a profile of the following description.

At the boundary of the bay and beach a concrete walkway 0.2m wide exists followed by a 1 metre flat section of dune(0.1m above MWL) and then an equilibrium beach profile of Dean(1977), \( h = 0.24x^{0.67} \) down to a depth of \( X \), this was followed by a linear grade to the depth of closure ~0.3m, Following this a flat bed extends until a wave transition slope dropped down to the paddle 0.55m. In order to achieve optimum wave conditions from the paddles.

The inlet bathymetry surveys would be reproduced in the model, however areas where no data existed would have to be extrapolated prior to calibration. Difficult areas included the offset nature of the north training wall and reef structures north of the inlet. These estimations would add more experimental error to the calibration/validation process.

**TIDE**

Tidal information from the Brunswick inlet showed the maximum amplitude to be about 1.9m. It is a very difficult decision in choosing tidal variation to use in the model because it will have a direct impact on the calibration. If we choose to use the largest spring tide this will provide the maximum flows and thus sediment transport due to tides, however this will influence the wave impacts. If we use a mean spring tide the morphological scale may vary because there is less flow circulating the sediment transported onshore by the wave action. This impact must be assessed as the project develops. In order to calibrate the model with the current velocity data the tidal range must be between +0.97 and –0.5m (at the time of measurement). However for validation and testing of the model a mean spring tide of +0.91m to –0.73m will be used.

It must be noted that the model generates the proposed tide in one side of the model and out the other it does not provide a tidal response.

From the limited current data available we will use the measured maximum values of 1.2m/s and 1m/s for ebb and flood flow respectively.

**WAVES**

The important characteristics of waves we need to simulate in the model include; magnitude of wave energy flux, wave direction, therefore magnitude of longshore energy flux and wave frequency therefore defining nett annual energy flux.

Wave data sourced from Cape Byron for the prototype period was found to be relatively consistent(Mean Energy, \( H_s = 1.7m \)) with 5 notable spikes of \( H_s \) wave conditions > 3.5m. It is important to include these storm events if we wish to calibrate the model with Brunswick prototype data. A way to calibrate the inlet may be to program a composite wave signal, with a mean energy wave signal (95%) and specific storm events(5%). This must be further considered when calibration is attempted and a methodology must be chosen. It also must be noted that some decrease in wave magnitude and angle of attack would be expected due to refraction and diffraction between Cape Byron and the Brunswick inlet however this should be limited to less than 10% and will be counteracted by the limitations of the modelling method in the use of greater than scale model sediment. The testing mean wave was calculated via the use of the wave energy
parameter $H^2T^2$, Walton and Adams (1976). The average wave energy for the data set was calculated and this correlated to a mean testing wave of $H_s = 1.72m$ and $T = 10s$. Storm wave events were taken as $H_s = 4m$, $T = 8s$.

Directional data was derived from the Tweed Wave Rider in summary form showing that the majority of waves approach the coast from a ESE direction between east (90) and SE (135) (~67%) during the summer prototype period. This data suggests that a wave approach angle of 20 degrees EAST should be adequate to represent the prototype conditions and at the same time generate a practical model longshore transport. The model wave paddle setup has been designed to facilitate 10, 20 and 30 degree angles of ESE wave attack angles and a single paddles can be used to simulate a NE wave attack angle on the ebb delta.

It was noted from the direction wave data summary that almost all large wave events i.e. >4m occurred from the ENE direction, although at very small frequencies). This is an important note that maybe should be tested in the model in order to investigate ebb shoal response to large wave conditions for short periods?
INLET RELATIONSHIPS

As part of this project an extensive literature review was undertaken of tidal inlets and associated investigations. This section defines a summary of Inlet relationships defined from observation and measurement of field inlets. It is proposed to use a number of these relationships to assist in the calibration of the physical model in conjunction with prototype data.

One of the earliest and most well known is the Tidal Prism vs Minimum inlet Area \( (P \text{ vs } A) \) relationship of O’Brien(1969) and later Jarrett(1976). The findings of Jarrett showed a strong correlation between these two parameters for over 100 inlets in the United States. This relationship has been used in the development of scaling criteria by Hughes(2000). Hughes has used this relationship to define an “equilibrium discharge relationship” within an inlet via the theoretical distortion of the froude velocity scale. This has been shown to work effectively however results are only accurate within the inlet where wave forcing is insignificant.

It is proposed to maintain this relationship in the physical model in the calibration process. Because the physical model does not model tidal prism essentially the current velocity vs Minimum inlet area will be maintained, so the position on the inlet stability curve of Escoffier(1977), shall be maintained.

A number of researchers have defined the importance of the sediment fall speed parameter \( (H/wT) \) in modelling surf zone processes. This has been a major stumbling block for this project. Because this parameter cannot be conserved for a model scale of any smaller than about 1:10. This scale size is much too large even for a very small inlet. Conservation of this parameter means that only storm conditions can be modelled and only surfzone processes where suspended transport due to wave induced turbulence dominates. Similitude is at the expense of bedload dominated processes outside of the surf zone. Empirical geometric model distortion has been used to maintain similarity for equilibrium beach profile modelling however the impacts of this distortion are complex and difficult to ascertain. Given the experience of the research team in this field it would be unwise to add further complexity to the already complex tidal inlet system.

Wave steepness \( (H/L) \) is a factor that is important in short wave modelling. This will be held in similarity by maintaining an undistorted model. Komar(1998) showed that High wave steepness is associated with erosion and low wave steepness with accretion.

Permeability is an important parameter in surf zone processes and training wall permeability can have a significant impact on sediment transport processes. Dalrymple et al(1999) has undertaken tests and shown that the diffraction of waves and energy loss due to a permeable training wall can generate a surface level overheight within the structure and thus generate secondary inlet currents. The physical model proposes to use semi-permeable breakwaters to investigate the influence of this secondary current.

Hanslows and Nielsen(1996) have shown from field measurements that there is a significant difference between wave setup within tidal inlets and wave setup on adjacent beaches. This overheight difference could also greatly influence tidal inlet processes due to associated secondary currents. For this reason water level measurements should be taken within the inlet and on the adjacent beach face.

Brunn and Gerritsen(1959), defined a relationship, from field observations, between tidal flow and longshore sediment transport. This relationship primarily defines the ruling bypassing mechanism, according to the ratio
\[ r = \frac{M}{Q} \]

- \( M \) = longshore sediment transport (m³/year)
- \( Q \) = maximum river discharge at spring tide (m³/s)

\( r > 200-300 \) – bypass sand by wave action across edge of delta
\( r < 10-20 \) – bypass sand by current action or shoal migration cycling

It is proposed to maintain this relationship as best we can in the calibration of the physical model with the prototype.

Brunn (1978)’s inlet stability relationship (Tidal Prism)/ (Sediment available to enter the inlet) \( \frac{P}{M} \) is a good relationship to hold in the physical model because the relationship takes the focus off the surfzone transport processes (which are not modelled correctly in the model). The waves in the model are a forcing mechanism attempting to infill the inlet at an available rate and the tidal system counteract this process. Hence calibration will attempt to find this equilibrium point calculated from the prototype inlet.

- \( \frac{P}{M} > 100 \) flows large and therefore stable
- \( \frac{P}{M} = 50-100 \) large ebb shoals but still deep and stable
- \( \frac{P}{M} < 50 \) shallow and unstable.

Dowbrowski (1996) investigated inlet relationships associated with ebb delta volume;

- Wave Height vs Tidal Prism,
- Inlet Width/Depth ratio,
- Inlet area below MWL and Tidal amplitude.

Dowbrowski described a quasi-equilibrium condition of ebb delta volume when the ratio between wave energy and tidal energy is representative of the long-term wave and tidal conditions of the entrance.

Floyd (1968) investigated ebb delta relationships for the east coast inlets of New South Wales and found relationships linking ebb delta ruling depth and channel depth (1) AND channel depth and distance offshore of ebb delta (2).

The limiting depth of ebb tide deltas is defined by wave breaker depth or alongshore transport depth of closure.

Walton and Adams (1976) defined that every inlet has a dynamic equilibrium ebb delta volume. They generated an empirical relationship for the volume in the following form \( V = aP^b \). The relationships defined tend to under predict ebb delta volume in locations where deeper water depths occur close offshore. In cases where sediment can be lost from the active system.

Fitzgerald et al (1977) defined that Inlet cross-sectional areas react rapidly to changes in flow and slowly to changes in Ebb tide delta. They also defined a relationship between Inlet cross-sectional area and flood tidal range was also found.

Gibeaut and Davis (1993) showed that ebb-tidal delta size does not correlate with morphology, but when normalised by their throat cross-sectional areas, wave-dominated inlets have oversized and tidal-dominated inlets have undersized ebb deltas compared to the equilibrium sizes of mixed energy inlets.

Machemehl, Herbich and Jo (?) defined a relationship between Inlet width and Inlet length.
Kieslich (1981) showed that channel thalwag migrates towards single training wall after construction regardless of Longshore transport direction, ie updrift or downdrift. Suggesting channel migration may be significantly influenced by tidal flow as well as longshore transport rate.

**SUMMARY**

The relationships above show a clear picture that inlet processes show a strong link to Inlet cross-sectional area and tidal prism. No studies researched show a strong correlation between longshore sediment transport and inlet processes. It is noted that longshore transport is a part of the balance which maintains the stability in the Inlet, and thus does influence inlet cross-sectional area. Because this study is focussed on ebb delta development and modelling limitations don’t provide any other options, it is assumed acceptable to model surfzone transport as bedload, provided the inlet infilling rate supplied by the waves can be quantified. This modelling methodology should allow the use of the above measured inlet relationships in the calibration of the model.
APPENDIX B.

Tidal Inlets Research Database
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<tr>
<th>ID</th>
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<td>Erosion adjacent</td>
<td>Erosion of beaches, Inlets, HwT</td>
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| Stability of Tidal Inlets, Theory and Engineering, Developments in Geotechnical Engineering Vol 23, Elsevier Scientific Publishing Company | 1978 | Nathan Library | Theory, field experience | Tidal Inlets | Tidal Prism relationship, Tidal inlet dynamics | Must separate material that bypasses the entrance on bars and the material that enters the inlet with flood currents. Also important to consider the net sediment transport in both directions. Bruun defines the net longshore drift as $\Delta M_{tot}$. Also must consider local circulations. Defines 4 parts to an inlet: (1) GORGE, (2) OCEAN ENTRANCE, (3) INTERMEDIATE SECTION, (4) BAY CHANNEL. The difference between tidal inlets is mainly in the morphology of the ocean bar where a battle of forces occurs: (a) tidal currents, (b) sediment transport including wave induced littoral drift and finally the flux of wave energy into the entrance which in turn depends upon the character and intensity of the wave action and bar morphology. Flood currents are more spread on the ocean side and more concentrated on the bay side, visa versa for ebb currents. What is less known is the quantity of sediment drift at the inlet and the detailed mechanism of bypassing in a quantitative sense. Proposes $\Delta P/M_{tot}$.


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Gourlay M.R.  Dec-85  EPA Lib  Lab testing  Lab testing of sediments  ED testing were done systematically changing sediment types and wave conditions. The profile type and dimensions, such as berm width and maximum wave setup, are found to be determined by the sediment mobility parameter (H/wT) and sediment properties, particularly permeability. Accreting beaches are particularly influenced by sediment permeability. Breaker types were influenced by the profile shape and sediment characteristics (particularly the permeability), as well as wave steepness and beach slope.

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Gravens M.B.  1996  Nathan Library  Numerical Model  Inlet Modelling and Beach evolution  Inlet Modelling, Beach evolution, empirical formulation, ebb shoal

Ebb shoal tends towards an equilibrium volume related to the tidal prism - Walton and Adams (1976), Marino and Menta (1987). This is the sort of model we wish to develop for long term planning. Factors impacting on inlet impact on littoral drift: net sediment impoundment, jetty leakage, channel shoaling, shoal formation


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APPENDIX C.

Inlet Relationship Summary Report
APPENDIX C.

INLET RELATIONSHIPS SUMMARY REPORT

Open Coast: Task 2

1. Purpose of the Working Paper?
   This working paper was instigated under the Open Coasts Theme by the CRC for Coastal Zone, Estuary and Waterway Management. The purpose of this brief paper is a tool for managers and planners to easily access the state of knowledge with respect to tidal inlets. This paper is not designed to be a comprehensive manual of tidal Inlet relationships but a brief overview of inlet relationships as a starting point with a database of references for managers that are tackling tidal inlet management problems.

2. INLET RELATIONSHIPS

   EBB DELTA CHARACTERISTICS

   TOPIC: Ebb Delta Characteristics
   NAME: Capacity of Inlet Outer Bars to Store Sand
   RESEARCHERS: Walton and Adams (1976)
   DATA SOURCE:
   Walton and Adams (1976) compared the idealised “no-inlet” hydrographic charts with actual hydrographic charts for 44 inlets around sandy portions of the United States coast to determine the volume of ebb-tidal deltas.
   RELATIONSHIP DESCRIPTION:
   
   \[ V = aP^b \]
   
   Where \( V \) = volume of sand stored in the outer bar/shoal of the inlet (in cubic yards of immersed sand)
   
   \( P \) = tidal prism of the inlet (ft³)
   
   \( a, b \) = correlation coefficients
   
   \( V = 8.7 \times 10^{-5}P^{1.23} \) (Nerang highly exposed coasts)
   \( V = 33.1A^{1.28} \) (linear regression data from 44 tidal inlets)
   
   COMMENTS:
   Dynamic Equilibrium, empirical relationship. Limitations it only applies where all sand remains in the active system and is not lost offshore (ie Nerang). They concluded that more sand is stored in the ebb-deltas of low energy coasts than that of high energy coasts. Limitations to this model exist in that other factors such as longshore energy flux(sand supply available to the inlet) and sediment size that could present an upper limit to the storage capacity weren’t considered in the study.
Relationship between P and Ebb delta equilibrium volume. Mainly accurate for high wave energy coastlines, much more scatter for low littoral drift coastlines.

**TOPIC:** Ebb Delta Characteristics  
**NAME:** River Mouth Training in New South Wales, Australia  
**RESEARCHERS:** Floyd (1968)  
**DATA SOURCE:** 16 trained inlets on the NSW Coastline, Australia  

**RELATIONSHIPS DESCRIPTION:**  
The study suggests that the minimum bar depth expected would be given by the relationship (Floyd, 1968):  
\[ D_B = 0.5D_C \]  
Or  
\[ D_B = 0.21 \left( \frac{Q_m}{W} \right) \]  
Where \( Q_m \) = maximum discharge in cusecs for a tide of mean spring range  
\( D_C \) = maximum channel depth below the surface when \( Q_m \) occurs  
\( D_B \) = depth over the bar saddle at the same time  
\( W \) = width of the channel at the same time  

**channel depth** is also linked to **offshore distance of ebb delta**  

**COMMENTS:**  
“Floyd (1968) concluded that it is not possible to remove a bar by training jetties (other than in exceptional circumstances) and that the best depth across the bar can only be increased to the same extent as it may be practicable to increase the depth of the channel. These observations are valid for particular conditions, namely that littoral drift depends only on waves and wave induced currents, there are no appreciable ocean currents, offshore contours are regular, and entrance alignment is approximately perpendicular to the coastline.”  

Floyd (1968) investigated the relationships between ebb delta-depth and channel depth for 16 trained river entrances on the New South Wales coast. His research revealed that despite the complex processes involved in the formation of ebb-deltas, a simple correlation exists between channel depth and ebb-delta depth. Further, this relationship can be applied to all inlets with ebb-deltas ranging from 0.6 – 20m.  
Relationships for trained river entrances - Primarily inlet depth(max below mean tide) vs bar depth and bar distance from training walls. \( D_{max} = 1.3D_{mean}(A/W) \) uniform flow, \( D_{max} = 1.65 D_{mean}(eccentric ~ flow) \).  
\( D_B = 0.5D_C \) and \( D_C = 0.015B(d\) expansion point) Floyds relationships can be explained conceptually by Tomlinson(1991) in that after initial training the ebb delta is formed primarily by ebb currents with the jet magnitude defining the distance offshore of the deposition because the depth is below the depth of breaking for waves. This explains why the ebb delta is a set distance from the training walls in proportion to inlet depth which is proportional to Tidal Prism. (However I cannot work out why Floyd used the maximum depth?). So the ebb delta is pushed out to the reach of the current therefore defining the ebb
delta location it then builds up to a point where tidal energy and wave action form a quasi-equilibrium depth. This is the point where sediment bypassing begins.

**TOPIC:** Ebb Delta Characteristics
**NAME:**
**RESEARCHERS:** Dowbrowski (1996)
**DATA SOURCE:**
**RELATIONSHIPS DESCRIPTION:** Inlet width depth ratio, Inlet area below MWL and Tidal Amplitude. Ebb shoal quasi-equilibrium volume long term wave and tidal energy
**COMMENTS:** Dombrowski(1996) Model of Ebb Shoal evolution, tested against 5 Florida, USA inlets. Investigated the Impact of (1) Deepwater wave height, (2) Sediment Grain size (3) Suspended sediment concentration.

(1) an increase in H lead to a decreased ebb shoal formation rate and a decreased ebb volume
(2) This factor effects Tcrit and w, an increase gave an increase in growth rate and an increase in equilibrium delta volume
(3) An increase in suspended sediment concentration gave an increased growth rate but the same equilibrium volume
Effects of H/P ratio on Equilibrium delta volume(defined dependence) were that the growth of the delta is determined by the rate at which the sand, supplied by the littoral system, is deposited by the ebb tidal flow. This partly explains the fluctuations in delta volume. Concluded that an equilibrium volume may never be reached. Therefore a “Quasi-equilibrium condition” would occur. This value would be consistent with a value of the wave energy to tidal energy ratio representative of the long-term wave and tidal conditions at the entrance.

**TOPIC:** Ebb Delta Characteristics
**NAME:** Statistical Geomorphic Classification of Ebb-Tidal Deltas Along the West-Central Florida Coast
**RESEARCHERS:** Gibeaut, J. C. and Davis Jr, R. A. (1993),
**DATA SOURCE:** A statistical analysis performed by Gibeaut and Davis (1993) on the ebb-tidal deltas of 71 inlets on the Florida coast indicated that the delta size does not correlate with morphology.
**RELATIONSHIPS DESCRIPTION:** Gibeaut and Davis (1993) showed that ebb-tidal delta size does not correlate with morphology, but when normalised by their throat cross-sectional areas, wave-dominated inlets have oversized and tidal-dominated inlets have undersized ebb deltas compared to the equilibrium sizes for mixed energy (Walton and Adams)
**COMMENTS:**
By normalising ebb-delta area with the cross-sectional area of the inlet throat, they were able to show that wave-dominated inlets have oversized, while tide-dominated inlets have undersized ebb-deltas when compared to the equilibrium sizes of mixed-energy inlet deltas. Walton and Adams (1976) modified their previously mentioned relationship between ebb-delta storage
volume and tidal prism to represent a relationship between ebb-delta storage volume and inlet throat cross-sectional area, such that $V = a' A^{b'}$

where $V =$ volume of sand stored in the outer bar/shoal (in cubic yards of immersed sand)

$A =$ inlet cross-sectional area at the throat ($\text{ft}^2$)

$a', b' =$ correlation coefficients

Although this relationship displayed less scatter than that which was derived between tidal prism and ebb-delta volume, care must be taken when using it as it may be unrepresentative of equilibrium conditions and does not consider longshore energy flux (which moves the sand to the inlet where ebb tidal currents can deposit it on the outer bar), and size distribution of the littoral material (Walton and Adams, 1976).

**TOPIC:** Ebb Delta Characteristics  
**NAME:**  
**RESEARCHERS:** Wright(1976)  
**DATA SOURCE:**  
**RELATIONSHIPS DESCRIPTION:**  
**COMMENTS:**

Morphodynamics of a Wave-dominated River Mouth - River mouth depositional patterns depend on three primary processes; (1) turbulent diffusion, (2) turbulent bed friction; and (3) buoyant expansion.(from flood flows) - However wave and tidal influence play varying roles

**TOPIC:** Ebb Delta Characteristics  
**NAME:**  
**RESEARCHERS:** Niemeyer(1986)  
**DATA SOURCE:**  
**RELATIONSHIPS DESCRIPTION:** The dissipative efficiency of the ebb delta bar(and hence sediment transport capacity) is not only based on the restricted water depth on the highest part of the shoals but also to a larger extent on the shoaling water depth prior to the ebb delta.  
**COMMENTS:**

**TOPIC:** Ebb Delta Characteristics  
**NAME:**  
**RESEARCHERS:** Marino and Mehta(1987)  
**DATA SOURCE:**  
**RELATIONSHIPS DESCRIPTION:** The ebb delta volume(18 cases US) is dependent on: W/D(waves and current), the flow cross-sectional area, tidal amplitude  
**COMMENTS:**

**TOPIC:** Ebb Delta Characteristics  
**NAME:**  
**RESEARCHERS:** Sha & Van den Berg, 1993, Fitzgerald, 1996  
**DATA SOURCE:**  
**RELATIONSHIPS DESCRIPTION:** revealed that ebb delta average depth, horizontal extent and sand volume of outer deltas are positively correlated with the relative contribution of tides and waves
COMMENTS:

**TOPIC:** Ebb Delta Characteristics

**NAME:**

**RESEARCHERS:** New Zealand studies(10th conference 1991) EPA Lib, smaller inlets have much faster morphological time scales in the field.

**DATA SOURCE:**

**RELATIONSHIPS DESCRIPTION:**

**COMMENTS:** Hence Physical Models have faster morphological time scales

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**INLET STABILITY RELATIONSHIPS**

**TOPIC:** Inlet Cross-sectional Stability

**NAME:**

**RESEARCHERS:** Escoffier(1977)

**DATA SOURCE:**

**RELATIONSHIPS DESCRIPTION:** Maximum current speed is a measure for the sediment transport capacity of the inlet currents. Sand is carried into the inlet by littoral transport and Vc (critical) is the critical velocity at which point the inlet can scour the material being pushed into the inlet. The value Vc has been measured around the world as being in the order of 1m/s with the exact value depending on the littoral drift, sediment characteristics, wave climate and tidal period. Insert Escoffier Closure Curve


**TOPIC:** Inlet Stability

**NAME:** Brunn (1978)

**RESEARCHERS:** Brunn(1967) and Bruun and Gerritsen(1957) –

**DATA SOURCE:** 50 inlets on the USA and European coasts considered by Brunn, almost all the values of peak inlet ebb current velocity are in the range 0.8 – 1.2 m/s.

**RELATIONSHIPS DESCRIPTION:**

- related inlet stability to the ratio of the net longshore drift to the maximum inlet discharge. Brunn (1975) (Tidal Prism/total sediment transport into the inlet)(P/M)

P/M > 100 large flow inlet compared to littoral drift, STABLE.

P/M ~50-100 large offshore bars develop, but bars are deep

P/M < 50 , Shallow bars are common, Inlet UNSTABLE

P = Tidal Prism (m³)

M = Annual Littoral transport to the Inlet (m³)

**COMMENTS:**
This is the basis for the accepted relationship that stable open coast inlets have a mean max. velocity of approximately 1m/s. This seems to be a balance point where the wave transport processes ability to infill the channel is counteracted by the tidal flows ability to scour the channel. As noted by Bruun (1978) the definition is so simple but quantifying the sediment volume available to enter the system and the material that actually does (local forces) is the tricky part.

Defines differing time scales for equilibrium
- P v A relationship (ie O’Brien) defines a Diurnal time scale equilibrium (end of each tidal cycle) Empirical relationship
- Longer term equilibrium is heavily dependent on instantaneous Littoral transport rate, ie localised wave climate

TOPIC: Inlet Stability
NAME: Can Multiple Tidal Inlets be Stable
RESEARCHERS: Van de Kreeke J.
DATA SOURCE: Used the work of Escoffier (1940) and expanded it to multiple inlets. Used a linearized lumped-parameter model for N inlets.

RELATIONSHIPS DESCRIPTION:
“For multiple inlets connecting the same bay to the ocean:

- The existence of set(s) of equilibrium flow areas requires that the bay surface area and/or tidal amplitude and/or tidal frequency is sufficiently large;
- The required values of bay surface area, tidal amplitude and tidal frequency increase with increasing values of equilibrium velocity;
- At best, there exist two sets of equilibrium flow areas

For a two-inlet bay system there exists no set of stable equilibrium flow areas. This implies that ultimately one or both of the inlets will close. Which inlet remains open can best be determined from the position of the inlets and equilibrium flow curves in the [A1, A2] plane.”

COMMENTS:

TOPIC: Inlet Stability
NAME: Study of Tidal-Inlet Stability
DATA SOURCE:
RELATIONSHIPS DESCRIPTION: Machemehl. Herbich and Jo defined a relationship between Inlet width and Inlet Length.
COMMENTS: Research performed by Machemehl et al (1991) focused on developing a relationship between inlet width and length, and inlet stability for 51 tidal inlets in the United States. Using data sets obtained from aerial photography performed by the US Army Corps of Engineers over a 30-year period, they studied the relationship between an inlet's hydraulic radius and its stability to arrive at their conclusion. They found that the relationship between width and length is not linear, but up to a certain point, as the width of the inlet increases, so does the length.
Past this point, the length of the inlet is not affected by the width (Machemehl et al, 1991).

**TOPIC:** Inlet Stability  
**NAME:**  
**RESEARCHERS:** O'Brien and Dean(1972)  
**DATA SOURCE:**  
**RELATIONSHIPS DESCRIPTION:** Inlet stability methodology using prototype data  
**COMMENTS:**

**TOPIC:** Inlet Stability  
**NAME:**  
**RESEARCHERS:** Meng et al(2000)  
**DATA SOURCE:**  
**RELATIONSHIPS DESCRIPTION:** Theory on Stability of tidal inlets, Good resources to use related to STABILITY velocity required depending on given longshore transport  
**COMMENTS:**

**TOPIC:** Inlet Stability  
**NAME:**  
**RESEARCHERS:** Jenkins and Inman(1999)  
**DATA SOURCE:**  
**RELATIONSHIPS DESCRIPTION:** Equilibrium/Stability in Tidal Inlets – The influx volume of sand transported into the lagoon by the flood tide and waves is less than or equal to the outflux volume transported by the ebb tide. With a non-equilibrium inlet, if the tidal prism is not large enough to support the “mechanics-based equilibrium condition”, the controlling factors are (1) the amount of deficiency in the Tidal Prism (2) The proportion of the available prism below MSL. In restoration if (1) is fixed you have a larger inlet, but if you increase (2) you just decrease the flood dominance of the inlet and therefore balance up the equation  
**COMMENTS:**

**EBB JET THEORIES**

**TOPIC:** Tidal Jet Theories  
**NAME:**  
**RESEARCHERS:** Hughes (1993)  
**DATA SOURCE:**  
**RELATIONSHIPS DESCRIPTION:** Equilibrium Discharge Relationship  
**COMMENTS:** Equilibrium discharge relationship, primarily bed load transport by steady flow. Critical shear stress – Can use to define tidal velocity required in the inlet. THE EQUILIBRIUM CONDITION, this is a scale relationship derived from the field P vs A relationship.
TOPIC: Channel Characteristics
NAME: Factors Influencing Tidal Inlet Throat Geometry
DATA SOURCE:
Cross-sectional profile data for a three-year period was used by Fitzgerald et al (1977) to indicate the volume of the ebb-tidal delta increased significantly at a number of inlets in South Carolina. This had the effect of reducing the quantity of sand contributed to the inlet throat, resulting in an increase of the inlet’s cross-sectional area. An additional in-depth study performed at the end of this period indicated that similar changes to the throat’s cross-sectional area occur in the period of one tidal cycle, leading to the conclusion that inlets respond faster to variations in flow conditions than to changes in the ebb-tidal delta.

RELATIONSHIPS DESCRIPTION: inlet cross sections respond rapidly to changes in flow (eg change of tidal range, tidal anomalies, storm surge, river flooding) and slowly to changes in Ebb-tidal delta.

COMMENTS: Study of South Carolina inlets shows that inlet throat symmetry is related to 3 controlling factors: (1) the meandering of the channel thalweg, (2) the shoreline configuration and (3) the dominant longshore transport direction. The Price inlet responses quickly to changing flow conditions and more slowly to changes in the ebb-tidal delta. Good correlation between inlet x-section area and flood tidal range before measurement.

TOPIC: Ebb Jet Theories
NAME:
RESEARCHERS: Militello and Kraus (2001)
DATA SOURCE:
RELATIONSHIPS DESCRIPTION: Channels that are maintained through an inlet entrance often tend to realign at an angle to the inlet centreline. Found that eddies created by ebb-tidal jet are a contributing factor to channel realignment. Eddy formation is a non-linear process associated with the advective terms in the hydrodynamic equations. Previously it was believed that channel migration was related to longshore sediment transport, skewed geometry of the backbay or orientation of the entrance jetties. EDDIES CREATED BY ADVECTIVE ACCELERATIONS ASSOCIATED WITH EBB FLOW. The location of inlet channels is controlled by the Ebb jet and thus is directly related to training wall configuration. The jet position also holds implications for the location and shape of the ebb shoal because of the spatial and temporal distribution of velocity associated with the ebb jet and eddies. Relationships must be developed between eddy formation, bay and shoal geometry, jetty configuration and wave climate.
COMMENTS:

TOPIC: Ebb Jet Theories
NAME:
DATA SOURCE:
RELATIONSHIPS DESCRIPTION:
COMMENTS:
Effect of Offset Jetties on Tidal Inlet Flood Flow - this could also be linked to
a reverse situation to the above theory. That channel migration is driven on the
ebb and flood tidal flow. Both were assessed from the Shinnecock Inlet

**TOPIC:** Ebb Jet Theories

**NAME:**

**RESEARCHERS:** Tomlinson(1991), Hughes(2000) Jet flow and offset jetties

**DATA SOURCE:**

**RELATIONSHIPS DESCRIPTION:** Migration and Breaching process occurred every ~30 years, Typical bar generated from a steady jet is a horse shoe crescent shape. This pattern is modified by flood currents and wave related sediment transport. Conceptualised trained inlet response; structure traps littoral drift behind it, or material goes into inlet or trapped on ebb delta therefore very little sediment reaches downdrift beaches. Because of the extension of training walls into deep water ebb delta forms mainly under the action of the ebb current in deep water. (Like if under a steady jet). Once ebb delta reaches critical depth for wave action the ebb delta growth will slow until the dynamic equilibrium between tidal and wave energy is reached. Therefore maximum depth will be in line with limit of wave breaking.

**COMMENTS:**

**INLET BYPASSING MECHANISMS**

**TOPIC:** Inlet Bypassing Mechanisms

**NAME:**

**RESEARCHERS:** Brunn and Gerritsen(1959)

**DATA SOURCE:**

**RELATIONSHIPS DESCRIPTION:**

Three methods by which sand moves past tidal inlets: (1) through wave induced transport along the periphery of the ebb shoal, (2) Through transport of sand in channels by tidal currents and (3) Through sand bar and tidal channel migration. \[ R = \frac{M}{Q}, M = \text{longshore transport per annum}, Q = \text{Maximum river discharge at spring tide}. \] \( r > 200-300 \) (bypass by (1)), \( r < 10-20 \) (bypass via (2) & (3))

**M** = Annual Littoral transport

**Q** = max spring tide discharge

**COMMENTS:**

Inlet characteristics are a result of the tidal currents and local wave conditions a balancing process.

**TOPIC:** Inlet Bypassing Mechanisms

**NAME:**

**RESEARCHERS:** Fitzgerald(1982)

**DATA SOURCE:**

**RELATIONSHIPS DESCRIPTION:**
COMMENTS: Add Ebb Delta Breaching and Stable inlet processes to Brunn's list of bypassing mechanisms. (Tidal Dominated entrances). Note the importance of SWASH BARS and the processes which drive these!!!

TOPIC: Inlet Bypassing Mechanisms
NAME:
DATA SOURCE:
RELATIONSHIPS DESCRIPTION: Sand bypassing pathways
COMMENTS:

TOPIC: Inlet Bypassing Mechanisms
NAME:
RESEARCHERS: Gaudiano and Kana(2001)
DATA SOURCE:
RELATIONSHIPS DESCRIPTION: intervals between shoal bypassing (I = 0.046Tp + 4.56) and bypassed volumes(S = 6.42Tp + 113.4) were related to tidal prism. Larger inlets shoal bypassed less frequently, but produced more bypassing volumes.
COMMENTS:

TOPIC: Inlet Bypassing Mechanisms
NAME:
RESEARCHERS: Fitzgerald, Kraus and Hands(2000) – Factors effecting inlet sand bypassing; tidal prism, inlet geometry, wave and tidal energy, sediment supply, spatial distribution of backbarrier channels, regional stratigraphy, slope of the nearshore, engineering modifications. Defines specific processes of bypassing.
DATA SOURCE:
RELATIONSHIPS DESCRIPTION:
COMMENTS:

TIDAL PRISM INLET RELATIONSHIPS

TOPIC: Tidal Prism vs Minimum Inlet Cross-sectional Area
NAME:
RESEARCHERS: O’Brien(1931), (1969)
DATA SOURCE: 1969 work Pacific Coast Inlets 28 inlets, 9 on the atlantic coast, 18 on the pacific coast and 1 in the gulf coast
RELATIONSHIPS DESCRIPTION: Jettied entrances A = 4.69 x 10-4P0.85 and unjettied entrances A = 2.0 x 10-5P
COMMENTS: true if a minimum longshore transport exists, if longshore transport is too low a balance is not attended to? Minimum inlet cross-sectional area below MSL vs Spring Tidal Prism, small inlets are not dynamic models of larger inlets with the same tidal prism. Because smaller inlets have greater effective depth variations between high and low tide. Relationships hold true in the inlet gorge however the characteristics of outer deltas could be controlling factors under certain conditions.
TOPIC: Tidal Prism vs Minimum Inlet Cross-sectional Area  
NAME: Tidal Prism – Inlet Area Relationships  
RESEARCHERS: Jarrett J.T.(1976)  
DATA SOURCE: An attempt was made to determine whether or not differences exist between the tidal prism –inlet area relationships for 108 inlets on the Atlantic, Gulf and Pacific Coasts in the USA. Given their developed and undeveloped entrance configurations  
RELATIONSHIPS DESCRIPTION: Unjettied and single jettied inlets on the three coasts do exhibit different P vs A relationships as a result of the differences in the tidal and wave characteristics between these three coasts. However the available data support the findings of O’Brien(1969). Maybe include a plot.  
COMMENTS: true if a minimum longshore transport exists, if longshore transport is too low a balance is not attended to?

TOPIC: Tidal Prism vs Inlet cross-sectional area  
NAME: Characteristics of Inlets/ Estuaries Discharging into Sheltered Waters  
DATA SOURCE: In order to obtain relationships for stable inlets in sheltered environments, with low longshore transport rates, field studies were undertaken to measure the characteristics of inlets in protected waters. Measured tidal prism, channel cross-sections and mean max. velocities (After Brunn(1978) for four inlets in South-east Queensland. Beelbi Creek in Hervey Bay and Tingalpa, Serpentine and Burpengary Creeks in Moreton Bay. Tidal prism 106 m³  
RELATIONSHIPS DESCRIPTION: Relationship was found to drop considerably below open coast P vs A predictions – O’Brien and Jarratt. Because mean max. velocity seems to be relatively constant for all open coast inlets (Brunn(1978)) they suggest that mean max velocity may be an appropriate design figure for the stability of sheltered inlets. P vs A relationship is controlled by the magnitude of littoral drift, all exposed coast relationships are the same because of continuous infilling (INCLUDE THE PLOT)  
COMMENTS: The cross-sectional area is larger relative to the tidal prism for sheltered inlets in comparison to higher longshore transport coastlines because less material is being forced into the entrance. Gourlay and Reidel (1980), define that P vs A relationships are dependent on alongshore transport rate. Order of magnitude less for low longshore drift entrances. Lower P vs A for estuaries upstream of littoral drift influence. Upstream of the influence of littoral drift all estuaries have the same relationship P vs A.

The littoral drift rate effectively controls the entrance cross-sectional characteristics, but upstream from the influence of littoral transport all estuaries have a similar P vs A relationship.
WAVE INTERACTIONS

TOPIC: Wave Setup in River entrances
NAME: 
RESEARCHERS: Hanslow and Nielsen
DATA SOURCE: Brunswick River, NSW, Australia
RELATIONSHIPS DESCRIPTION: Wave Setup in River entrances reduced by diffraction.
COMMENTS: Nielsen and Hanslow (1992) highlight from measurements that wave setup in River entrances is significantly less than on adjacent beaches. This was shown to produce significant scours through the breakwater to the channel. The Brunswick is a shallow Inlet and hence there is no reason why significant setup would not occur. Therefore a significant amount of potential energy exists adjacent to the entrance which can have a significant influence on local sediment transport processes surrounding an inlet.

INLET STRUCTURE INTERACTIONS

TOPIC: Wave / Training wall interactions
NAME: 
DATA SOURCE: 
RELATIONSHIPS DESCRIPTION: diffraction of waves and energy loss due to permeable breakwaters. Wave interactions generate secondary inlet currents.
COMMENTS: 

TOPIC: Inlet Structure Interactions
NAME: Tidal Inlet Response to Jetty Construction
DATA SOURCE: By observing the response of inlet channels to single jetty construction at thirteen locations in the United States
RELATIONSHIPS DESCRIPTION: Kieslich (1981) showed that channel thalweg migrates towards single training wall after construction regardless of longshore transport direction, suggests channel migration is significantly influenced by tidal flow as well as longshore transport.
COMMENTS: Kieslich (1981) found channel thalwegs migrate towards the jetty, regardless of a number of key factors considered. Thalweg migration ranged from a maximum of 31% to 49% of the total distance available for migration for single downdrift and updrift jetties respectively. In some situations, the migration was in opposition to the tidal regime at the inlet, with sediment entering the entrance from the natural side of the channel as a result of longshore transport. The inlet current regime causes scouring of the jetty side of the channel, and alone may be enough to cause thalweg migration. If the permeability of a jetty is high, the channel will not migrate toward it, and in the case of twin jettied entrances, migration occurs away from a deteriorated
jetty. Investigate the role of training wall permeability on channel stability.

**TOPIC: INLET STRUCTURES**
**NAME:**
**RESEARCHERS:** Dalrymple(1999) Shore and Beach
**DATA SOURCE:**
**RELATIONSHIPS DESCRIPTION:** Wave induced current produced by jettied entrances – Wave diffraction and decay along jetties generates a current in against the wall and within the wall shoreward. When the wave has decayed to a point a return flow must be generated that moves to the centre of the channel.
This is very interesting because it may suggest if wave conditions were great enough this wave induced circulation could significantly assist Ebb Tidal Flow at the oceanward entrance and retard flood flow at the entrance because of the spread out nature of flood flows at the entrance, However with these large wave conditions comes significant mass transport on shore. Remember the impact of this effect on the outside of the breakwaters. I have seen this circulation at Brunswick Heads and Tweed Entrance. It may have more impact on the outsides of the jetties.
**COMMENTS:**

**FLOOD DELTA CHARACTERISTICS**

**TOPIC:** Flood Delta Characteristics
**NAME:**
**RESEARCHERS:** Mehta et al(2001)
**DATA SOURCE:**
**RELATIONSHIPS DESCRIPTION:** Study of 61 Inlets, flood delta AREA and VOLUME, weak correlation defined between these parameters and tidal prism. Increased tidal prism equals larger AREA and VOLUME. While the deposit thickness remains invariant.
**COMMENTS:** These is explained by the fact that much of the sand ingress to an inlet may fall out of the active system and hence additional volume would occur according to the specific geometry and age of the inlet. This can be equally true for ebb deltas such as the Gold Coast Seaway.

**TOPIC:** Flood Delta characteristics
**NAME:**
**RESEARCHERS:** Bhogal(1989)
**DATA SOURCE:**
**RELATIONSHIPS DESCRIPTION:**
**COMMENTS:** Sebastian Inlet Investigation, Flood Delta is much larger than Ebb delta even though inlet is Ebb flow dominated because inlet bay conditions allow sediment to settle out of suspension, out of reach of ebb flows, sediment transport; degree of bottom roughness, sediment fall speed and the threshold velocity.
References


