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Wave Setup in River Entrances

Field Surveys of Wonboyn Lake Estuary and Tallebudgera Creek Estuary, Australia

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ABSTRACT

It is unclear whether wave setup exists in river entrances and previous studies have shown that it is virtually non-existent in trained river entrances. However, conclusive studies have not been made on semi-trained and untrained river entrances. In this study, two field surveys were conducted in Australia; one at Wonboyn Lake Estuary, New South Wales, and one at Tallebudgera Creek Estuary, Queensland.

At Wonboyn Lake, water surface elevations were recorded along the open coastline and inside the lagoon at a number of locations. Current data was also recorded along with topography and bathymetry. The data showed that there were significant elevation differences (*100-140 mm*) between the river entrance and offshore water levels, despite the measurement points being close to one another and the net flow in the entrance being zero. A similar dataset was recorded at Tallebudgera Creek, but due to problems with the weather, the data was not complete.

Two models, one analytical and one numerical, were used to analyze whether wave setup may have occurred at the river entrance in Wonboyn Lake. The analytical model was originally designed to calculate wave setup along the open coast, but an attempt was made to apply it to a river entrance. The resulting output from the river entrance was uncertain due to this, but showed there may have been a setup of approximately *70 mm* at the aforementioned point in time when the difference was *100-140 mm.*

The numerical model (HEC-RAS) was employed using bathymetry and topography data recorded in the Wonboyn Lake area. The model showed that the driver of the lagoon water level was the higher water level in the entrance and not the offshore MWL.

The conclusion of the Wonboyn Lake analysis was that there may have been wave setup, or some other phenomena, that raised the level of the water in the entrance from the offshore water level, since the water level in the entrance was the driver of the system. However, since the analytical model was too uncertain to be conclusive it was not possible to confirm or discard the occurrence of wave setup in the river entrance.

It was not possible to conduct a reliable analysis of the Tallebudgera Creek data, since it was too fragmented to apply any of the two previously mentioned models to it.

Finally, it is recommended that more research is made on the subject of wave setup, if it is to be concluded whether it exists in river entrances or not.

FOREWORD AND ACKNOWLEDGEMENTS

This master thesis on *'Wave Setup in River Entrances – Field Surveys of Wonboyn Lake Estuary and Tallebudgera Creek Estuary, Australia'* was written under the regime of the Faculty of Engineering at Lund University (Lunds Tekniska Högskola), during the period of March - August 2011. The field surveys in this report were conducted under the regime of the University of Queensland in Brisbane.

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Linköping, August 2011
Henrik Ölander-Hjalmarsson

CONTENTS

1 INTRODUCTION

The ongoing discussion of climate change has spurred a large interest in the consequences of more extreme weather and a raised sea level. The most exposed areas are next to coastlines, and this raises the demands on coastal engineers when it comes to design criteria for marine structures and to make water level predictions. Coastal areas are being developed in many parts of the world, and according to the UN Atlas of the Oceans (2010), 44 percent of the global population live within 150 km of the coast.

Furthermore, there is a growing migration of people to coastal areas, and this applies high pressure on metropolitan areas that, in many cases, are already over populated. For example, approximately 1000 people arrived daily to coastal cities in China alone in 2003 (PRB, 2003). With a rising sea level, more extreme weather and more coastal development, the damage may be more substantial for each storm that hits. According to NOAA (2011), the biggest life threatening danger during a storm is the rise in water level caused by storm surges.

One example was Hurricane Katrina that caused USD 108 billion (not adjusted for inflation) of damage in 2005. It was the costliest hurricane in U.S. history (NOAA, 2011). Hurricane Katrina might serve as an indication of what is to come in a future with denser coastal population. Due to this increasing risk, a better understanding of the dynamic properties of coastlines is needed in order to minimize costs of life and property during such events in the future (NOAA, 2011). Raised water elevations during storms do not only contain the storm surge, but also wave setup and tide (Hurricane Science, 2011). It may therefore be of interest that wave setup is investigated further in areas such as river entrances and not only along open coastlines.

1.1 Background

Wave setup is a concept that is widely known, but has been proven to be neglected in several storm surge prediction models used in Australia (Stewart 2010). A number of studies (e.g. by Hanslow et al. 1992, 1996) have been conducted to better understand how wave setup behaves in river entrances, but they have only been conclusive with respect to trained river entrances. Given the concerns raised by e.g. the NOAA (2011), there is an interest to find out more about water level changes during large wave events and storms.

There is similarly an interest in finding out more about wave setup in river entrances. For example; if it exists in river entrances, how does it affect the water surface elevations as well as the interchange of water between the estuary and the ocean?

Two field sites were chosen for this project: Wonboyn Lake and Tallebudgera Creek. In Wonboyn, the major focus was put on wave setup and how it would affect the water surface elevations in the river entrance and estuary.

In Tallebudgera Creek, the Gold Coast City Council had received concerns from residents around the estuary with regard to fear of flooding during heavy rainfall events. The City Council continuously dredges the river entrance and it wants to know if it is worth the cost. There was therefore an interest to see if it would be possible to measure wave setup in and around the Tallebudgera river entrance and connect that to any possible rise in water surface elevation due to an increased runoff.

1.2 Aim

The aim of this thesis is to find out if wave setup occurs in semi trained river entrances such as at Wonboyn Lake using field measurements in combination with analytical and computer models. Furthermore, the aim is also to find out if there is any risk of flooding in the Tallebudgera Creek Estuary during heavy rainfall events and if wave setup may contribute to any potential flooding.

1.3 Method

The work on this thesis officially began on the *1st* of March, 2011. First, a literature review of predominantly wave-setup related research material was conducted. The literature review in this report brings up the background history of how wave setup is understood in general term. The findings were summarized into chapter 2.

One dataset was collected during a field trip to Tallebudgera Creek River entrance when the author was enrolled as an occupational trainee at the University of Queensland. Unfortunately, during the field trip to Tallebudgera, the surf washed away a lot of measuring equipment resulting in gaps in the data. Despite this, it was still possible to extract some information.

One other dataset was recorded at Wonboyn Lake, NSW, in December 2010.

Wave data for each study in this thesis was obtained from wave buoys operated by the NSW Department of Commerce. Weather data was extracted from the website of Bureau of Meteorology (2011). There were no near-shore wave measurements made on-site since the offshore wave data was deemed to be enough for the post processing of each field survey dataset.

When the datasets had been collected, an analysis was conducted. The first step was to find erroneous data points and adjust or remove them from the datasets. The data was then plotted in order to visualize the water levels at each measurement point as functions of time. Once the data had been plotted into graphs, a visual analysis was conducted by looking at the trends of the data and what information those trends could convey about any possible wave setup in the river entrances.

The visual analysis was only a small part of the analysis process, but laid the foundation for the models described later in this report. Using models to try and understand if there was any wave setup present in the Wonboyn Lake river entrance was the prime focus of the analysis process, and also one of the most time consuming parts of the project.

Unlike the Tallebudgera dataset (2011), the Wonboyn dataset (2010) was completed without any major problems. Therefore, two models were applied to this dataset. Two models were used because one model alone may not be sufficient to support any possible claims regarding the wave setup conditions, whereas if two models were used, they could be compared and possibly validated towards each other's results. Unfortunately, none of the models could be applied to the data from the Tallebudgera field study.

The first model was an analytical solution to the general wave setup equation by Dean and Dalrymple (1991) and another model was a 1D computer model built in Aquaveo's Watershed Management System and simulated in USACE's HEC-RAS simulator. The results of these models were then compared with the measured data for validation, see chapter 6.

1.4 Limitations

This thesis is limited to the survey and analysis of the following two estuaries situated along the east coast of Australia. The dates the datasets were collected are written in the brackets.

- Wonboyn Lake Estuary, Eden, New South Wales (*14th-16th* of December 2010).
- Tallebudgera Creek Estuary, Gold Coast, Queensland *(3rd-5th* of April 2011).

The geographical limitations in each case was different, but limited to the distance between a beach transect and a water surface elevation measurement point somewhere upstream in the lagoon.

The time limit of the Wonboyn and Tallebudgera datasets was approximately *25-26 hours*. That time span was chosen because it would include the time it took to complete two full tidal cycles, plus an extra time buffer before and after the intended survey period.

There was also the overall limitation of the survey methods used in these surveys. The surveys could only be as good as the number of surveyors and the quantity of equipment used to record data points. Each survey was also highly dependent on the weather and other natural factors.

2 WAVE SETUP

Wave setup is a rise in mean water level (MWL) from the still water level (SWL) within the surf zone. This rise in water level is the result of wave-related momentum being transferred to the water column when waves break (Dean and Walton, 2010). The MWL is, unlike the SWL, not a horizontal line that moves up and down only with tidal variations, but a surface that is seen as the time mean of the instantaneous water surface, i.e., it changes when waves slow down, rise and break close to shore (Nielsen, 2009), see figure 1. If the MWL is below the SWL it is defined as wave setdown.

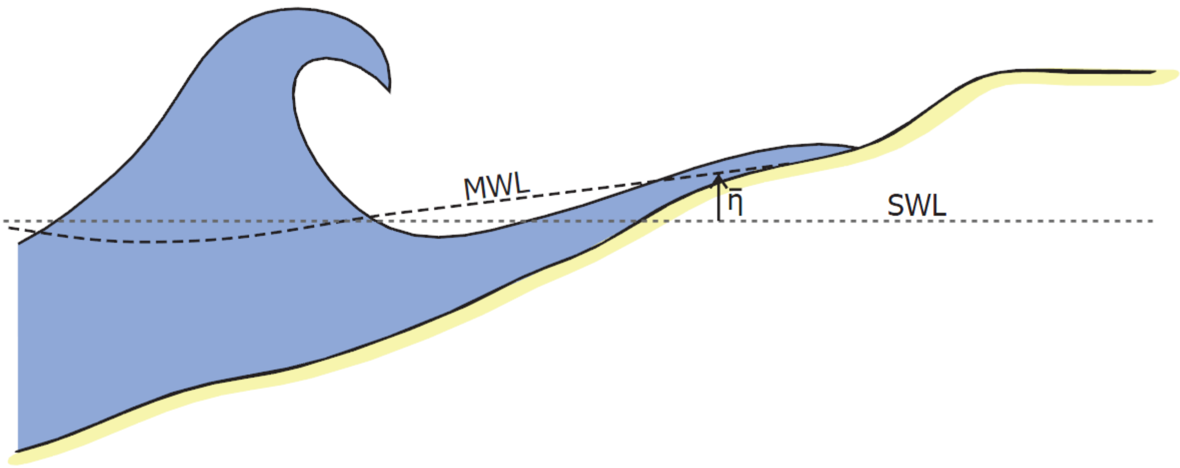

Figure 1 *Conceptual section drawing of beach and water surface profiles. The wave setup $\bar{\eta}$ is the distance above the still water level (SWL) to the mean water level (MWL).*

Understanding wave setup better may also be beneficial in preventing flooding events in estuarine areas since, e.g., beach berms may be nourished to prevent such events from happening.

Dunn (2001) pointed out in his doctoral thesis that to better be able to predict water surface elevations around developing areas close to estuaries may have large economic implications on the value of real estate. In other words, if water levels are overestimated there might be large economic consequences for real estate owners when prices plummet. On the other hand, if levels are underestimated, areas that in reality are unsuitable for development may be developed and damaged if there is a flood due to a wave-setup contributed water level.

There are several other engineering applications that can benefit from a better understanding of wave setup. Such applications may be to, e.g., improve harbor design, since the dynamic wave setup can contribute to unwanted and possibly damaging oscillations that arise due to resonating patterns (Dean and Walton, 2010).

2.1 Background

Wave setup was first observed in the late 1930's at Narraganset Pier and at New Port, Rhode Island, U.S. (Dean and Walton, 2010). In 1961, Dorrestein conducted some of the first experiments tracking the MWL using stilling wells fastened to floats and then recording the water levels in those wells using a film camera.

In the 1960's, Longuet –Higgins (1962) and Longuet-Higgins and Stewart (1963, 1964) developed some of the first theoretical relationships for describing wave setup using wave radiation stresses. During initial field observations, the water surface elevation next to the beach line was noted as higher than what was recorded at a tide gauge further out from the shore. This lead to the conclusion that waves had something to do with that water level rise (Dean and Walton, 2010).

In 1968, Bowen et al. (1968) performed some of the first laboratory experiments in trying to quantify wave setup using periodic waves. Bowen et al. concluded that the maximum wave setup was a major part of the maximum wave run-up height. He also produced figure 2 from his lab experiments that show the wave setdown that accompanies wave setup due to continuity.

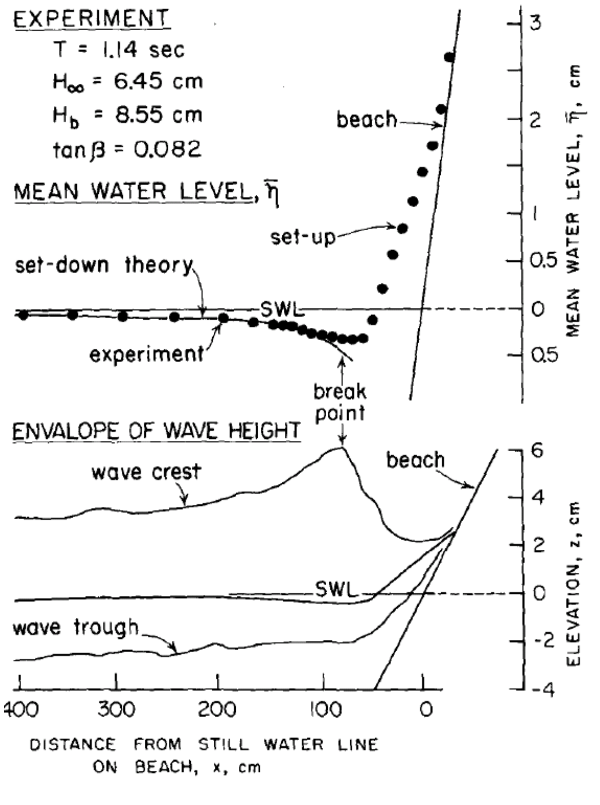

Figure 2 *Diagram produced by Bowen et al. (1968) based on laboratory experiments for wave setup. It shows the mean water level with its wave setup and setdown.*

Svendsen (1984) introduced the concept of the wave roller, which is the turbulent front of a breaking wave. This wave roller carries momentum towards the shore after the wave has broken, and that explains why there is a delay in the transfer of momentum to the water column. Tajima and Madsen

(2006) showed that wave setup can be fairly accurately predicted in the near shore surf zone in the laboratory when the roller is included in the radiation stress.

2.2 Open Coast

Stewart et al. (2010) showed that the wave radiation stress, and subsequently wave setup, is often excluded when modeling storm surges and this underestimates the predicted storm tide level. As Hanslow et al. (1996) and Dunn (2001) have showed this might not have a serious impact when predicting water surface elevations in trained river entrances. However, since wave setup is prominent along coast lines, underestimated storm surge level predictions without radiation stresses might prove dangerous if those predictions have been used as a theoretical base for where to develop coastal areas.

Dean and Bender (2005) investigated how internal drag forces from vegetation and surface forces due to bottom friction affects wave setup. Both linear and non-linear wave cases were investigated and the wave setup effect was considerably attenuated by vegetation in both cases. Theoretically, vegetation can also induce a wave setdown for the case of nonlinear waves, but Dean and Bender (2005) did not claim that an event such as that has been observed in the field.

A major part of the research that has been conducted on wave setup has revolved around open coast wave setup. Consequently, the phenomenon has not been investigated to the same degree in river and lagoon entrances partly due to there being many significant parameters to the problem: morphology, runoff flow, bottom friction, side wall friction, viscosity, water depth, wave height, wave period etc. Making models with appropriate simplifications without affecting accuracy is a delicate process and when the different variables are many it becomes difficult to find general relationships that can fit many different morphologies and lagoon types. Many of the relationships that have been derived in the past are typically only confirmed to apply to the field surveys and experiments they were derived from.

One example of the fairly large difference between several field studies and laboratory experiments can be studied in a chapter on wave setup written by Dean and Walton (2010), where they summarized several laboratory and field experiments on wave setup. They clearly state in their conclusion that more research needs to be done on the subject due to the differentiating nature of previous work. Results may differ substantially from survey to survey. One reason for this was noted to be the many different kinds of measurement techniques that were used by different science teams. Dean and Walton (2010) also declared that in its current state, the understanding of wave setup is still not deep enough to allow for engineering design applications due to these different results.

Apotsos (2007) managed to improve several models for wave setup by manipulating what he called a ‘free parameter’. In his sensitivity analysis of the different models he found that wave rollers, tidal water elevation changes and bottom stress significantly affects wave setup, and this also confirms the work by Dunn (2001).

The sensitivity analysis by Apotsos (2007) also showed that it is important to estimate the water depth profile and cross-shore wave height in order to accurately predict the wave setup. Apotsos (2007) finally concluded that it is possible to predict wave setup on both uniform and non-uniform beaches by using parametric wave models. He also stated that a simple 1D model that uses the shore-normal momentum balance and that also includes bottom stress and the concept of the wave roller is all that is required.

2.3 River Entrances

Hanslow et al. (1992, 1996) conducted surveys at the entrance of Brunswick River at Brunswick Heads, North South Wales, Australia. Hanslow et al. used a manometer system developed by Nielsen in 1988 to measure the MWL. Measurements were highly detailed and the result of the survey was that wave setup in the river entrance was either small or negligible. Hanslow (1996) concluded that the absence of wave setup in the river mouth was due to the momentum flux of the river flow and its influence on the incoming waves.

A dip in water surface elevation was also noted outside two breakwaters surrounding the river entrance. That dip was explained by the breakwaters being in the way of the longshore current. When the current veered ocean side to pass the breakwaters, the velocity of the current increased outside of the river entrance and consequently lowered the water surface. Large scour holes were also noted at low tide which indicated that large quantities of water passed through the breakwater into the river entrance. If these events had a significant effect on waves and subsequently wave setup was not investigated.

In 2001 Dunn attempted to further explain the data collected by Hanslow et al. (1992, 1996) by using different physical, analytical and numerical models. He concluded that two of the reasons for the small and sometimes immeasurable wave setup in trained river entrances were due to energy dissipation from breaking waves due to bottom friction and waves losing energy when rolling on side walls. Consequently, when waves break without any energy dissipation due to bottom friction and side wall friction the setup can be significantly higher (Dunn, 2001). This showed that these factors of energy dissipation that often are ignored when calculating wave setup are important to consider. Dunn (2001) also analyzed tide gauge data from Tweed Heads and Ballina, New South Wales, and from the Gold Coast Seaway, Queensland, and came to the same result that the wave setup was negligible in these locations as well.

Apparently, as in the case of the Brunswick River entrance case described by Hanslow et al. (1992, 1996) and analyzed further by Dunn (2001) a trained river entrance with an outflow of water does not produce any significant wave setup. However, this may not be the same for semi-trained or un-trained river entrances which are investigated in this report.

Stewart et al. (2010) investigated storm surge models and performed hindcast simulations for Tropical Cyclone (TC) Roger that hit the Australian coast in 1993. During TC Roger there were tidal anomalies that could not be fully explained and research is still being conducted at the University of Queensland on the subject. The results found by Stewart et al. (2010) support the previous work by Hanslow et al. (1996) and Dunn (2001) that there is a negligible wave setup in trained river entrances.

2.4 Wave Radiation Stress

Wave radiation stress is the key factor behind wave setup. When waves break they transfer their momentum to the water column (Dean and Walton, 2010), thus raising the water surface elevation close to shore which balances out the radiation stress. Radiation stress arises as a force in the direction of the wave propagation. Longuet-Higgins and Stewart (1964) defined radiation stress as “the excess flow of momentum due to the presence of waves”.

2.5 Governing Equations

Wave setup can be described using the force balance seen in figure 3. The wave setup is divided into two parts; one static and one dynamic part. The static wave setup is the wave setup that is seen as the raise of the MWL as a result of the momentum being transferred to the water column. The dynamic wave setup is the oscillatory part of wave setup (i.e., “surf beat”). The dynamic wave setup oscillations usually have a time period of 10-20 times longer than the mean wave period (FEMA, 2007).

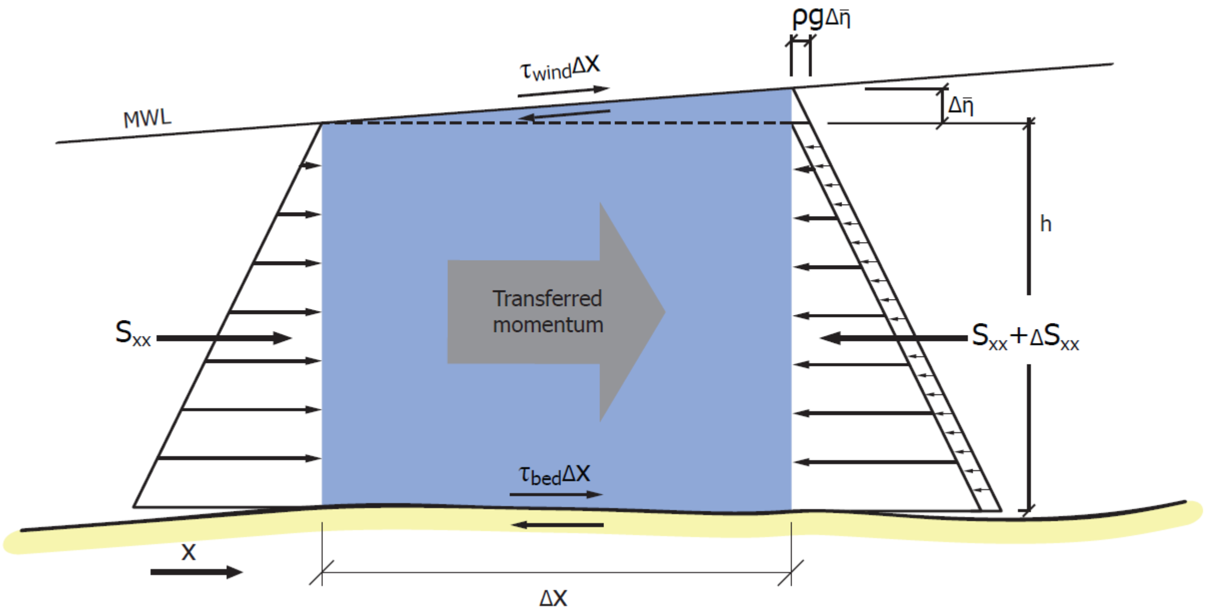

Figure 3 *Control volume in the surf zone. All forces are time averaged. Figure concept adapted from FEMA (2007) and Nielsen (2009).*

According to Longuet-Higgins and Stewart (1962) the conservation of momentum in the cross-shore direction is

$$\frac{\partial S}{\partial x} = \rho g(\bar{\eta} + h) \qquad (1)$$

The force balance in figure 3 can be written as

$$-\rho g h \Delta\bar{\eta} - \Delta_{S_{xx}} + \tau_{wind}\Delta x - \tau_{bed}\Delta x = 0 \qquad (2)$$

or, with regard to the setup slope as

$$\frac{\partial\bar{\eta}}{\partial x} = \frac{1}{\rho g h}(\tau_{wind} - \tau_{bed}) - \frac{1}{\rho g h}\frac{\partial S_{xx}}{\partial x} \qquad (3)$$

The first step to finding a simplified equation for the wave setup is by neglecting bed and wind shear stresses in eq. (3) (Longuet-Higgins and Stewart, 1962), which then becomes

$$\frac{\partial\bar{\eta}}{\partial x} = -\frac{1}{\rho g h}\frac{\partial S_{xx}}{\partial x} \qquad (4)$$

By then combining a shallow water wave theory with the radiation stress S_{xx} and wave height H and by assuming that waves approach the shore normally, the following equation for radiation stress is found (Nielsen, 2009)

$$S_{xx} = \frac{3}{16}\rho g H^2 \qquad (5)$$

To find wave setup as the function of the shore normal distance x, eq. (5) is substituted into eq. (4). The resulting equation is then

$$\frac{\partial\bar{\eta}}{\partial x} = -\frac{1}{h}\frac{3}{8}H\frac{\partial H}{\partial x} \qquad (6)$$

Bowen (1968) and Longuet-Higgins and Stewart (1964) that developed the earliest wave setup models assumed a linearly proportional relationship between wave height (*H*) and water depth (*h*), and that may be applied by

$$\frac{H}{h} \equiv \gamma \qquad (7)$$

where γ is normally set to 0.78 for monochromatic waves.

Eq. (6) is then as follows

$$\frac{\partial\bar{\eta}}{\partial x} = -\frac{3}{8}\gamma^2\frac{\partial h}{\partial x} \qquad (8)$$

where the water depth $h = D + \bar{\eta}$. D is the height from the bottom to the SWL. Nielsen (2009) solved this equation with the boundary condition that $\bar{\eta}(h_b) = \bar{\eta}_b$ to find the wave setup as a function of the total depth where $_b$ denotes a breaking wave condition. The solution to that equation, given by Nielsen (2009), is then

$$\bar{\eta}(h) = \bar{\eta}_b + \gamma^2 \frac{3}{8}(h_b - h) \tag{9}$$

Through this equation, the shoreline setup can be calculated by assuming $h=0$. However, according to Nielsen (2009), the equation is often inaccurate in predicting the shoreline setup when compared to actual field data and will likely underestimate $\bar{\eta}(0)$, i.e. the shoreline setup elevation.

2.6 Examples of Empirical Wave Setup Equations

Nielsen (2009) proposed an empirical relationship for wave setup based on measurements taken from South Beach at Brunswick Heads in 1989. According to Nielsen (2009) the formula provides an accurate fit for data collected at sandy beaches with varying morphologies

$$\bar{\eta} = \frac{0.4H_{orms}}{1+10\frac{h}{H_{orms}}} \tag{10}$$

When finding the shoreline setup elevation, i.e. the approximate seepage face, where $D + \bar{\eta} = h = 0$, eq. (10) becomes

$$\bar{\eta}_s = 0.4H_{orms} \tag{11}$$

where H_{orms} is the deepwater root-mean-square wave height. However, this relationship only serves as an approximation. How well it fits with the wave setup elevation at Wonboyn Lake is seen in chapter 5.1.

Stockdon et al. (2006) found a relationship that depends on beach slope, taking into account the trends of one of their datasets (Nielsen, 2009);

$$\bar{\eta}_s = 0.35\beta_{Beach}\sqrt{H_{orms}L_0} \tag{12}$$

Baldock (2004) concluded that the amplitude of the dynamic wave setup is likely to depend on the beach slope face. The data that Nielsen used to find the correlation seen in eq. (10) did not show any tendency to rely on the beach face slope, but in the case of Stockdon et al. (2006) the correlation was improved utilizing the slope (Nielsen, 2009).

3 FIELD SITES

3.1 Wonboyn Lake Estuary

Wonboyn Lake is situated in New South Wales near the south-eastern corner of Australia at 37.250 S, 149.967 E Lat/Long (DNR, 2007) a short distance north of the southern NSW border (figure 4). The surrounding area is sparsely populated and industrialized which has resulted in the estuary largely being in an undeveloped and non-industrialized state with the exception for oyster farming. Wonboyn Lake is situated in the temperate zone with fairly stable average temperatures throughout the year. The temperature hovers between *~8-14 °C* (min/max) during winter and *~16-22 °C* (min/max) during summer (BOM, 2011). The annual rainfall the past ten years has been in the range of *450 mm* in 2009 to over 980 mm in 2010.

Wonboyn Lake Estuary consists of a river and a lake and its catchment area is approximately *320* km^2. The area of the waterway is *3.6* km^2 (DNR, 2007). It is possible to navigate by boat through from the lake to the entrance of the estuary, but due to the wide part of the entrance being shallow *(~0.5 m*) (DNR, 2007) during ebb it may be difficult to navigate through. Also, the narrow part of the entrance, seen in figure 5 was prone to relatively high flow velocities at the time of the field survey.

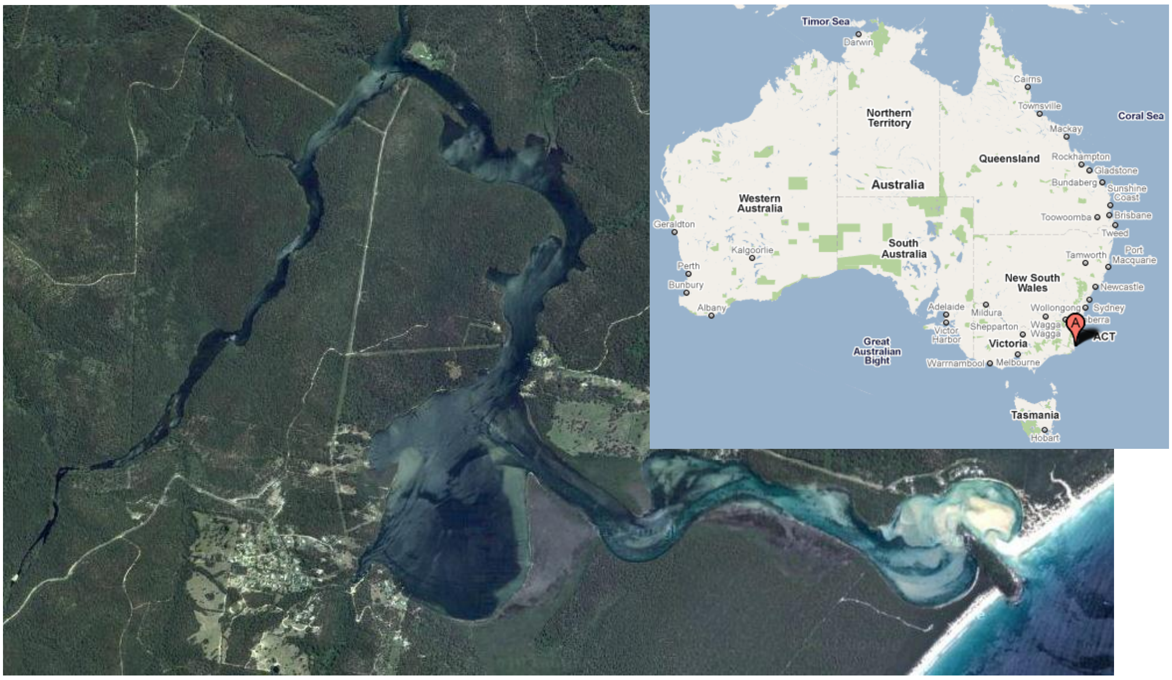

Figure 4 *Satellite image of Wonboyn Lake Estuary and its location (A) in Australia (Google, 2005)*

Figure 5 *The lagoon entrance as seen from a satellite photo taken in 200 5 (Google).*

It should be noted that the satellite pictures of Wonboyn Lake Estuary are from 2005, and the morphology has changes since then. The spear of the spit has changed and eroded down to a narrower shape. Figure 6 was taken during the field trip and shows shape of the spit at the time of the field trip. Figure 19, a virtual image of the spit, was created using the topographical data points recorded with a Total Station.

Figure 6 *The lagoon entrance along with the shape of the spit. Photo taken during the field trip described in this report (photo by Peter Nielsen, 2010).*

The lagoon entrance to Wonboyn Lake is, as shown in the pictures, naturally trained on one side against the cliff of Bay Rock. The lake is connected to a small river, ‘Wonboyn River’, a few kilometers upstream. The entrance closes from time to time and the locals have had to excavate it in the past due to flood risks. The photos below (figures 7, 8 and 9) were taken in 2004 when the estuary closed up completely which caused flooding in the general Wonboyn Lake area. Due to the fear of further floods, an excavator was called in to open the entrance (figure 9).

The entrance closed up entirely in 2009 again, but in early 2010 it reopened by itself by the pushing force of runoff water from heavy rainfall (Wonboyn Cabins, 2010).

Figure 7 *The closed up lagoon entrance in 2004 seen towards the inland and lake (photo by Alan Hay, 2004).*

Figure 8 *The closed up lagoon entrance in 2004 seen towards the ocean (photo by Alan Hay, 2004)*

Figure 9 *The entrance just after the excavation was completed in 2004 (photo by Alan Hay, 2004)*

During the field trip, a local oyster farmer explained that a large part of the lake is used for oyster farming, so it is important that the salinity is acceptable and preferably optimal. The oysters grown in Wonboyn Lake thrive in brackish water and if the salinity is too high or too low, crops may get hindered in growth or even die. It is therefore important for the oyster farmers that the entrance is kept open to allow an interchange between fresh and salt water.

The waterway upstream of the lagoon entrance was at the time of the field trip shallow in many places and the propellers of the boats that were used as transport to the location frequently hit the bottom. The wider part of entrance (ocean side) was not particularly deep and it was possible to wade across fairly easily during low tide.

3.2 Tallebudgera Creek Estuary

The river entrance to the estuary (figure 10) is situated just north of Palm Beach (28.095 S,153.462 E Lat/Long.) on the Gold Coast just south of Brisbane. The creek has a total network length of *219 km* and the catchment has an area of *110* km^2 (Healthy Waterways, 2010). Just upstream of the creek, on the other side of the Gold Coast Highway, is a number of artificial lakes. The area is developed and populated to a large extent, but according to Healthy Waterways (2010) the state of the estuary is in moderate to good condition in terms of water quality. The climate in the region is classified as humid subtropical climate with hot humid summers and cool winters. The average yearly precipitation is about 1300 mm with most of the rainfall between October and June (BOM, 2011).

The Gold Coast City Council regularly dredge the lagoon entrance to Tallebudgera Creek Estuary due to rising concerns of the local because they have expressed their worries about the risk of flooding during heavy rainfall. A groin was constructed in the 1970’s on the south side of the river entrance to protect it and the surrounding beach from wave erosion.

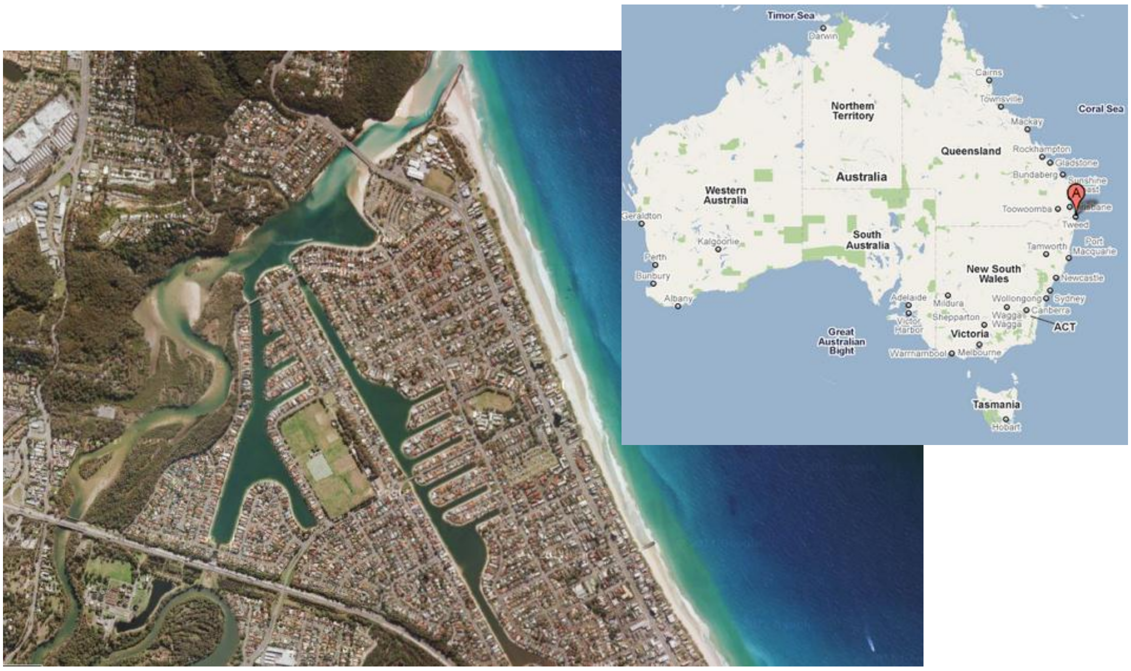

Figure 10 *Satellite photo of Palm Beach and Tallebudgera creek river entrace at the top. The artificial lakes and channels are clearly visible in this photo (Google, 2008).*

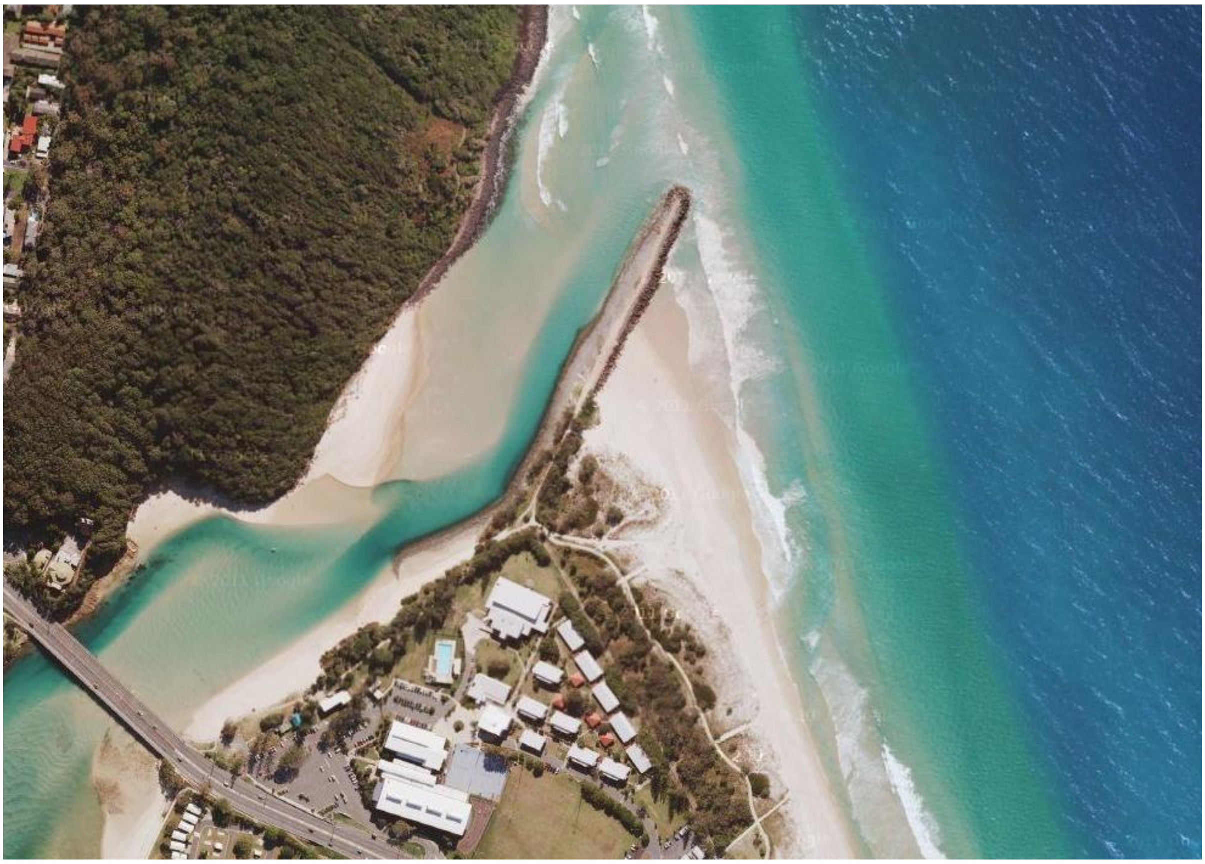

Figure 11 *Satellite photo showing the field site. The rock wall is seen in the middle of the picture. (Google, 2008)*

Below is a set of satellite photos (figure 12) from Google Earth of the river entrance of Tallebudgera Creek taken in 2003, 2004, 2008, 2009 and 2010. The changes in the entrance have been quite significant between the different years. This is due to sediment being transferred as a result of different marine processes. During normal conditions there is a slow accretion of sediment towards the shore and during storm events sediment is carried offshore. When there is heavy rainfall, the flow in the river is larger than usual, thus bringing finer sediment along with it. Towards the entrance, when the river widens and eventually reaches the ocean, the sediment settles as the velocity of the flow decreases.

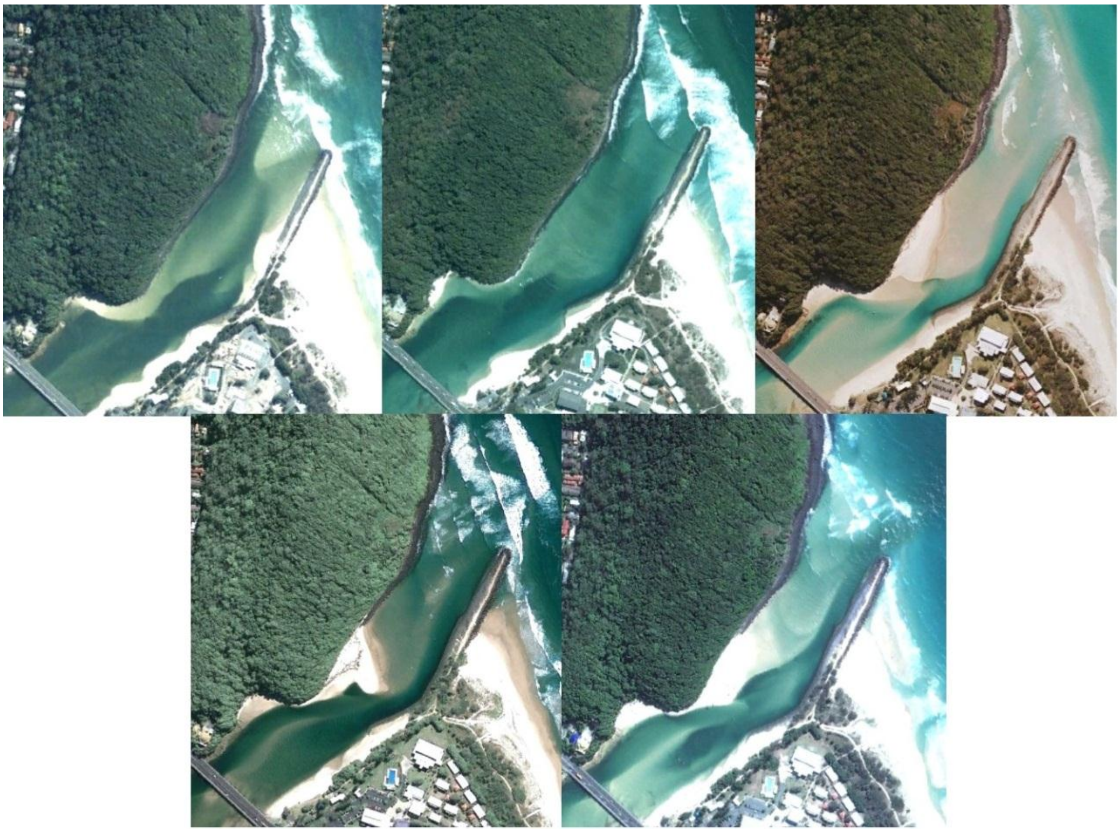

Figure 12 *The morphology changes constantly in the river entrance. From top left: 2003, 2004, 2008, 2009 and 2010.*

Figures 13 and 14 were taken during the field trip. Figure 14 shows how the sand has accreeted on the north side of the river. Figure 15 gives and indication to the wave conditions at the river entrance.

Figure 13 *Panoramic photo merge taken on top of the groin.*

Figure 14 *Photograph taken towards the mainland showing the bridge with the Gold Coast Highway in the background.*

Figure 15 *Wave conditions by the river entrance.*

4 FIELD MEASUREMENTS

The author attended the surveys at Wonboyn Lake on the *14th-16th* of December 2010 and Tallebudgera Creek River entrance on the *3rd-5th* of April 2011. The surveys were conducted under the management of the University of Queensland. The official academic supervisors on the field trips were A/ Professor Peter Nielsen and Dr. David Callaghan at the departments of Coastal and Civil Engineering.

The recorded datasets belong to the University of Queensland and will be referenced to in the text as the “Wonboyn dataset” and “Tallebudgera dataset”, respectively. To make sure that the data would be as easy as possible to compare between the different surveys, the surveys were set up in a similar and consistent manner.

It is important that surveys like the ones described in this report can be replicated easily by others who want to conduct research in the same area of study. Since most of the equipment used in this report is inexpensive and easy to manufacture and use (Nielsen, 1999), this helps a great deal in spreading the survey method to other researchers.

It should be noted that it is a difficult task to perform a successful field survey, as evident from several of the problems that arose during the Tallebudgera field trip. Weather, taking correct measurements, calibrating equipment, beach erosion, the human factor etc. all play a large part in whether a field survey is successful or not. A field survey dataset will likely never be perfectly representable of reality, but as long as great care is taken when conducting the surveys and post processing the data, many productive conclusions can be drawn from it.

4.1 Field Measurements in Wonboyn

4.1.1 Previously conducted studies

The Department of Land and Water Conservation – Estuary Management Program, conducted a bathymetry survey of the Wonboyn estuarine lake in 1998. In the survey, depths and drying heights were recorded using a dual frequency echo sounder attached to a boat. The location of each depth measurement was recorded using a GPS. The outline of the estuary was drawn up using aerial photographs taken in 1997.

4.1.2 Water Surface Elevation Survey

Peter Nielsen has used a survey method of stilling wells in the past and has evaluated and validated its efficiency of producing reliable results, e.g., in his report from 1990 (Nielsen, 1990).

The stilling wells, commonly made out of clear plastic pipes, made up the bulk of the water level measurement equipment. Each well had a measurement tape securely strapped to it to make it possible to fairly easily measure the water level in each well. The wells themselves were strapped to star pickets that had been driven into the sand. To make sure the bottom of the wells would not be in open air in case of fast erosion rates they were dug down into the sand. The filter in the bottom of each well acted as a damper to remove any short oscillations from incoming waves. If all wells were calibrated correctly with the same time constant, a representative MWL could be found by setting up the wells in a shore-normal line, see figure 17. The equation for finding the time constant using the half-life of the well is shown here

$$H = H_0 e^{\left(\frac{-t}{T}\right)} \qquad (13)$$

where H_0 is the water column in the well and H is half of the water column, see figure 16. A time constant of $T=100\ s$ was used to calibrate all the filters of the wells to not be susceptible to any short wave oscillations. The time t it took to drain the well half a water column using the aforementioned T took approximately $70\ s$.

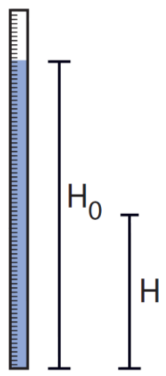

Figure 16 *Conceptual drawing of well illustrating* H_0, *and* H.

The wells are cost-effective pieces of equipment that, when calibrated correctly, provide a good representation of the MWL. They are cheap to manufacture and replace if broken or lost in the surf. The wells are illustrated in figure 18.

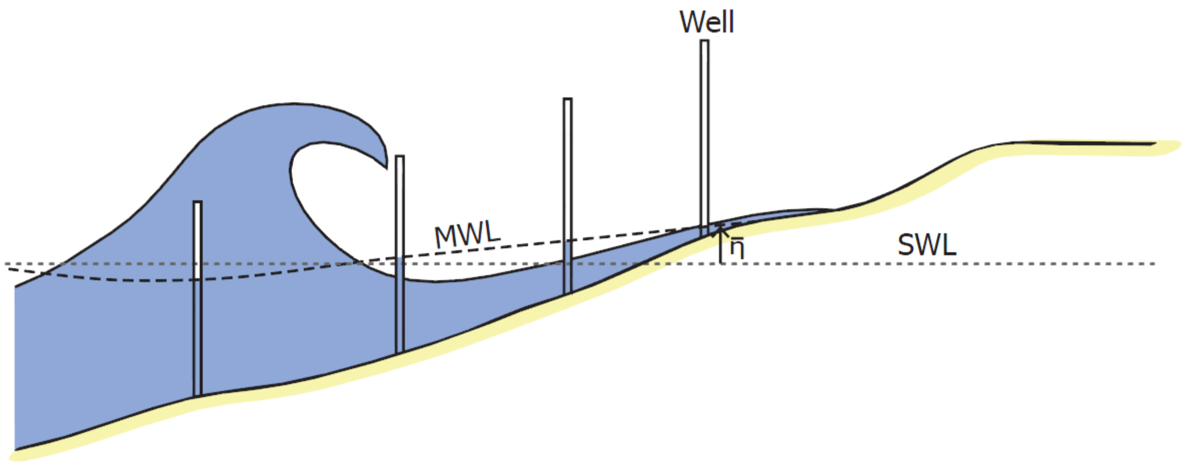

Figure 17 *Conceptual image showing how the wells portray the MWL and subsequently wave setup.*

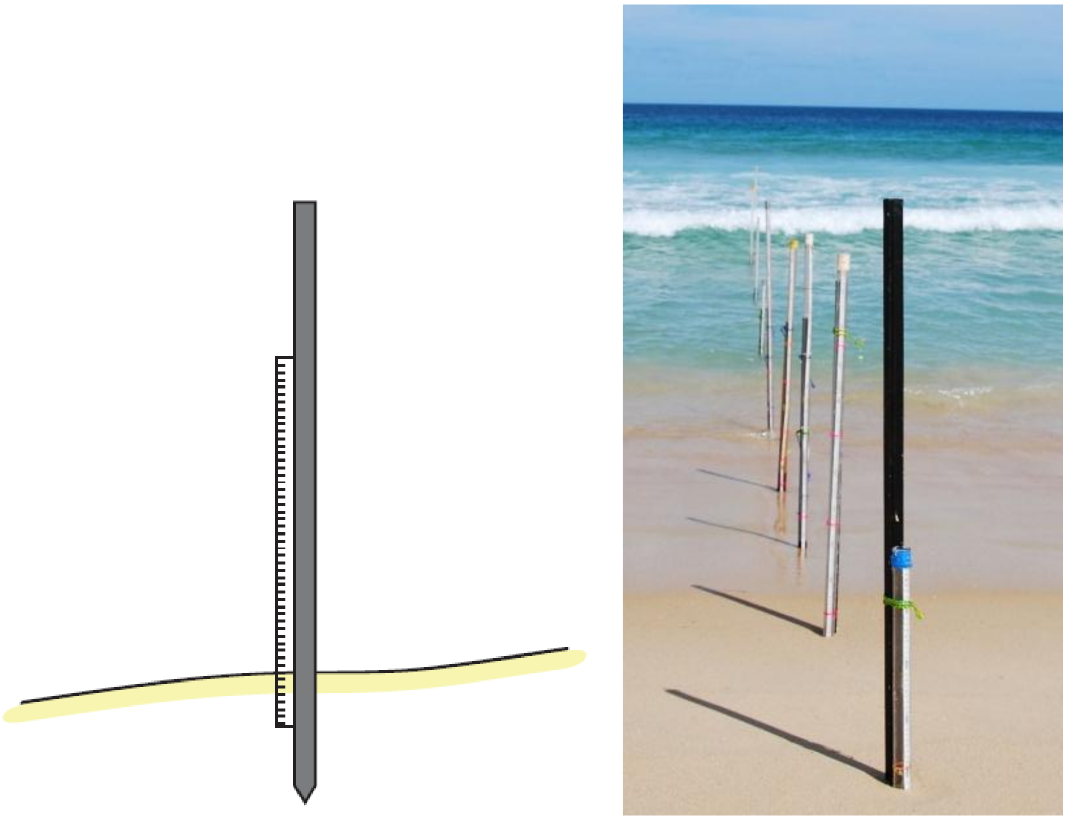

Figure 18 *Left: Conceptual image of a well fastened to a star picket. It is important that the well is dug down a bit into the sand if significant erosion occurs. Right: Wells in the beach transect at Wonboyn.*

Measurement tapes were set upside-down with *0 m* being at the top of the well. The reason for that was that it would be easier to connect the measured water level with the level survey. A color additive was also added to water in the wells to make it easier to take readings. The readings were made at millimeter level accuracy, but were prone to an error margin, see 4.3.1.

Wells were set in three separate major locations (figure 19);

- One beach transect consisting of nine wells in a straight line (figure 21).
- Three entrance wells positioned around the lagoon entrance (figures 19 and 21).
- One lagoon well was positioned further upstream in the lagoon (figures 19 and 21).

All readings from the wells were taken manually by looking at the water level and noting the reading on the strapped-on measurement tapes. They were taken once every *15 minutes* from each well throughout the survey. During low tide, the wells that were furthest inland would be in the dry and therefore no measurements could be taken from them at that time.

Figure 19 is a conceptual image that was derived from the Total Station survey data collected during the field trip. It shows the sand spit at the entrance of the lagoon. The color coded symbols represent the approximate locations of the different stilling wells. Well 1 was not included in this image and no readings were taken from it since the water never reached its elevation.

Figure 19 *Conceptual image of the lagoon entrance at Wonboyn Lake. All dimensions are only approximate. The symbols in the image denote the different wells used in the survey and their approximate locations. The ocean is to the right in the image and the lagoon entrance is next to the red ring. The north direction is straight up vertically in the figure.*

Figure 20 *Enhanced image of the lagoon entrance along with the location of the Entrance Wells and Lagoon Well. The Beach Transect is further to the right outside of the image (photo by Peter Nielsen, 2010).*

The image below (figure 21) shows the beach transect during mid-tide. Well 1 was never wetted, but was marked in this image nonetheless. It was from these wells that a MWL and subsequently the wave setup could be plotted. The results of these surveys are found in chapter 5.

Figure 21 *The Beach Transect wells with well numbers. The symbols along with the colour codes are the same as in chapter 5.1 for easier overview of the data (photo by Peter Nielsen, 2010).*

4.1.3 Sand Surface Elevation Survey

The sand level at the wells was recorded when possible, i.e., when the water level was not too high to accurately measure the sand level against the measurement tape strapped to the wells. However, due to scouring around the wells most of the time, the accuracy of the sand level measurements varied in the order of several centimeters. The sand level measurements were used to calculate the shoreline elevation and to get an overview of how much erosion and accretion that occurred next to the shoreline.

4.1.4 Current

Two current monitors were used to determine when the flow changed direction due to the turn of the tide. The type of current meters that were used consisted of a simple manometer construction with two glass tubes protected by a casing made of wood. The two tubes were connected at the top so the pressure over the water level is equal in both tubes. They were also connected to two plastic pipes at the bottom that were filled with a filter to make sure the small wave variations got filtered out from the data.

First, some water was sucked in to the current meter raising the water level to make taking readings possible. When the water levels were equal on both sides the current meter could be used to take measurements. In order to see the direction of flow, the levels of the two manometer tubes were noted. If the flow was coming from the left, the water level in the left manometer tube would be higher than the right and vice versa. The current meters were calibrated using a known flow in a flume and care was taken to make sure the filters were of equal density.

The locations of the currents monitors were approximately in the area of the entrance wells.

Due to large quantities of sediment being transported in the entrance, the current monitors were buried at around *03:30* on the morning of the *15th*. However, the direction of the current was noted by visual observation after that.

4.1.5 Level survey

In order to tie the collected data together, level surveys were made manually using a Dumpy level (figure 22). The elevations for all of the wells were taken several times to assure that errors would be small.

Figure 22 *Image of a dumpy level and measurement rod (photo by Paul Moric, 2011).*

The level surveys using dumpy levels were given an error margin of approximately *2-4 mm* to make sure the levels would be accurate enough. As discussed in section 4.3.1, the water level measurements were noted in millimeters, but since the water surface inside the wells slowly moved up and down with the waves the accuracy of those measurements was more likely at centimeter level.

In studies like this, it is of utmost importance that the elevation surveys are conducted thoroughly, since the elevations had to match for all wells. Otherwise it would not have been possible to make accurate assumptions with regard to the data and a possible wave setup scenario. If the levels would have been off by too much the data would be of poor quality and unusable, since the sought after precision was on the level of a maximum of a few centimeters.

State benchmark 83667 with an AHD (Australian Height Datum) of *1.347 m* (i.e. *1.347 m* above the approximate offshore mean sea level) was used to connect the elevations of all the wells.

A Total Sation was also used to determine the bathymetry and topography of the area. Several hundred points were recorded for the possible use of creating a 3D model of the area. The Total Station was the instrument of choice for finding the bottom profile for the beach transect.

Figure 23 *A Total Station in the background on its tripod on the sand spit in Wonboyn.*

4.2 Field Measurements in Tallebudgera

4.2.1 Water Surface Elevation Survey

The same types of wells were used as described in 4.1.2.

The wells were positioned in strategic locations in order to get a representative overview of the water surface elevations during the duration of the survey. The following locations were used (see :

- A beach transect originally consisting of 12 wells in a shore normal straight line (figure 24). Two transducers were also used. One was strapped to the offshore well (Well 12) and another to a star picket. Unfortunately the elevation could not be measured for the outer most transducer. This transect made it possible to see the setup of the MWL as a function of time.
- Three wells upstream of the river entrance. A transducer was strapped to the well furthest inland (the ‘lagoon well’) so no manual readings were needed to be taken from it except for a few that were used as calibration values.

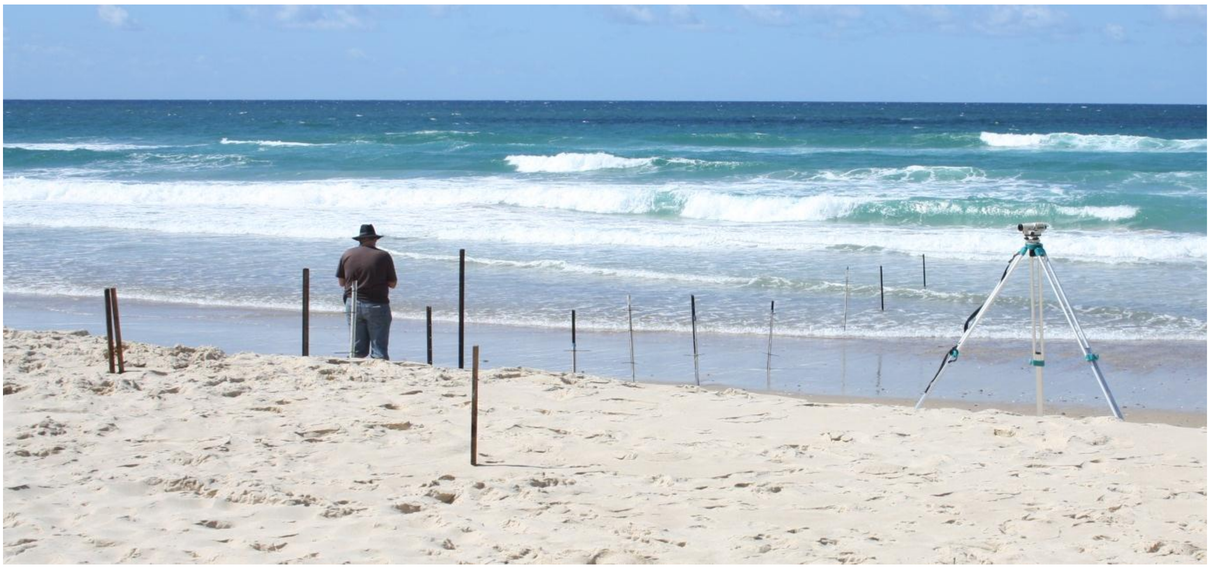

Figure 24 *The beach transect during the second day of surveying (photo by Paul Moric, 2011).*

Readings were taken every *15 minutes* throughout the entire surveys this way with a few exceptions. Due to night time safety reasons for the surveyors, measurements were only taken once every 30 minutes for the two wells closest to the ocean throughout the night.

Figure 25 *Locations of the Creek wells and transducer as well as the beach transect. The dark blue line to the right shows the location of the beach transect.*

The pressure transducers were programmed to take a reading once every second. Since the pressure transducers recorded the total pressure, the data needed to be corrected for atmospheric pressure variance in order to get the pressure from the water column. To get an elevation reference for the data recorded by the transducers, a well was placed on the same spot. Several readings were taken manually from the well, and those readings were used to make an appropriate fit of the transducer data.

Unfortunately, due to a higher rate of beach erosion than expected, several wells were lost and fell in the surf after only a few hours of surveying. This led to losses of data, which is evident in 5.2. Since a survey like this requires accurate height elevations of each well, problems arose during dark hours when it became increasingly difficult to survey elevations of wells that were re-installed after they had fallen down or been moved. When the data was put together after the field trip, several inconsistencies in the elevation data were found due to this.

The inconsistencies were believed to be due to the confusion and stress of the survey crew that arose when the equipment began to fail and also due to the high rate of beach erosion that may have moved the locations of the equipment. As a result, relatively large pieces of data needed to be discarded from the beach transect survey. Even though there were many problems surrounding the Tallebudgera field trip, the author learned a lot about the art of surveying and organizing a field trip. However, it was still possible to draw a few conclusions from the remaining data, see 6.2.

4.2.2 Current

The same type of current meters as in 4.1.4 was used in this survey. Since the bathymetry of the river entrance was not measured due to time constraints and practical reasons, the actual flow in the river was not measured, only the direction change of the flow.

4.2.3 Level survey

The same type of level survey as in 4.1.5 was utilized.

All wells were surveyed using this technique and the levels were tied back to permanent survey marker 182011 that had a level of *2.645 m* above AHD.

4.3 Sources of Error

4.3.1 Water and Sand Surface Elevation Survey

The human factor is a main source of error when taking measurements. Blunders such as making an incorrect reading, forgetting that the measurement tapes are upside-down, accidently moving a well etc, are always a possibility, especially during rough conditions in the surf zone at night time. The human factor may also have been involved when calibrating the wells, making incorrect readings when checking the well time constants and tuning the filters.

When the measurements were taken they were recorded down to millimeter accuracy. However, since the water level in the stilling wells was constantly moving slowly up or down, a more probable accuracy is at centimeter level.

It is quite likely that there was a slight time offset in the data since the wells, in theory, should be measured at the exact same time. However, since there were only two people doing readings at the beach transect at any given time, doing an instant reading on all of them was not humanly possible and it usually took a few minutes to take all the readings manually. Because this offset was deemed to be relatively small, it will be neglected in this report.

4.3.2 Current

If a current meter was not completely sealed over the water surface, the water that had been sucked in would leak out and that would have made taking correct measurements impossible. Also, if the filters were not dense enough, water would escape out purely by the pressure from the water column inside the tube.

4.3.3 Level survey

Level surveys are subject to many possible errors, including human mistakes and blunders. Out of these mistakes and blunders, some of the most common errors according to the Montana Department of Transportation (2011) are;

- Reading a value incorrectly off the measurement rod
- Looking at the wrong crosshair in the Dumpy Level
- Misplacing decimal points
- Misunderstanding between the person that holds the measurement rod and the person looking through the level survey

Another source of error is of systematic nature. That means that it is an error that occurs during a specific situation or all the time.

A few examples of common systematic errors (Montana Department of Transportation, 2011);

- Malfunctioning Dumpy level, e.g., the prism inside does not line up correctly and causes an angular error.
- Atmospheric refraction, i.e., the refraction of light through air that is temperature stratified. Air close to the ground has a higher density than air higher up, which means that light will refract downward.
- Curvature of the Earth. This might cause readings to be too high when the distance between the measurement instrument and the measurement rod is long.

To eliminate errors such as these, care was taken to make sure the steps in between level points were not too long. Circle closing the surveys with satisfying results was another important way of checking the precision and accuracy of the surveys.

A Total Station is also subject to several of these errors, and it is often difficult to get a millimeter precision using that kind of instrument. The Wonboyn dataset (2010) became an evidence of this when a few points that were measured and then re-measured showed differences of several centimeters. That may have happened due to several reasons, e.g., the staff with the reflection prism may have been sunk down too far into sand or may have been held at an angle.

The bottom profile which is used in chapters 5 and 6 was recorded only once. Since sand eroded and accreted onto the beach, the profile would have been prone to changes. For all analysis purposes, those changes were assumed to be negligible.

The bathymetry dataset from 1998 recorded by the ‘Department of Land and Water Conservation – Estuary Management Program’ may also have contained errors. It is unclear if they corrected their data for small, or large, variations in the water surface elevation when using the echo sounder. However, since that possible error mostly would have applied to the data that was recorded close to and/or in the ocean, it can likely be neglected for two reasons.

First, the data was tied together with the new Wonboyn Dataset (2010), which did not contain errors of that kind. Second, if there were any errors like that inside the lagoon it is not likely they will affect the end result significantly since the HEC-RAS model is merely a simplification of the measured bathymetry and since the most sensitive part of the model is in the river entrance.

5 DATA COLLECTED

5.1 Wonboyn Lake Estuary Data

This dataset was collected during approximately *26 hours* on the *14th-15th* of December 2010. All of the gathered water surface elevation data points are seen in figure 26. The locations of the wells are shown in figures 19, 20 and 21. Well 2 was the well located furthest up on the beach and well 9 was furthest offshore. Entrance wells 1, 2 and 3 were placed close to the narrowest point of the entrance on the sand spit side. The lagoon well was placed further upstream in the lagoon as seen in figure 20.

Wells 2, 3, 4 and 5 could not be read the entire survey since they would get dry during low tide. This explains the gaps for those wells in the data. The water surface elevations in the lagoon well were attenuated by the slowing reaction time of the lagoon itself, as seen in figure 26.

The full surface water elevation data is seen in figure 26, where time in hours is along the x-axis and the elevation in millimetres AHD is on the y-axis. The zero datum line for the y-axis is the AHD which approximately represents the offshore mean sea level. The interpolated shoreline elevation is shown as the black hollow diamonds. The large × signs show where the current changed direction, and “in” and “out” denotes whether water was flowing in or out of the river entrance.

The attenuated appearance of the curves for the water level in the most shoreward wells (2-6) is due to the waves getting smaller after they break and eventually become runup on shore. Wells 7-9 on the other hand continuously record the tidal signal and therefore exhibit an appearance with a larger amplitude.

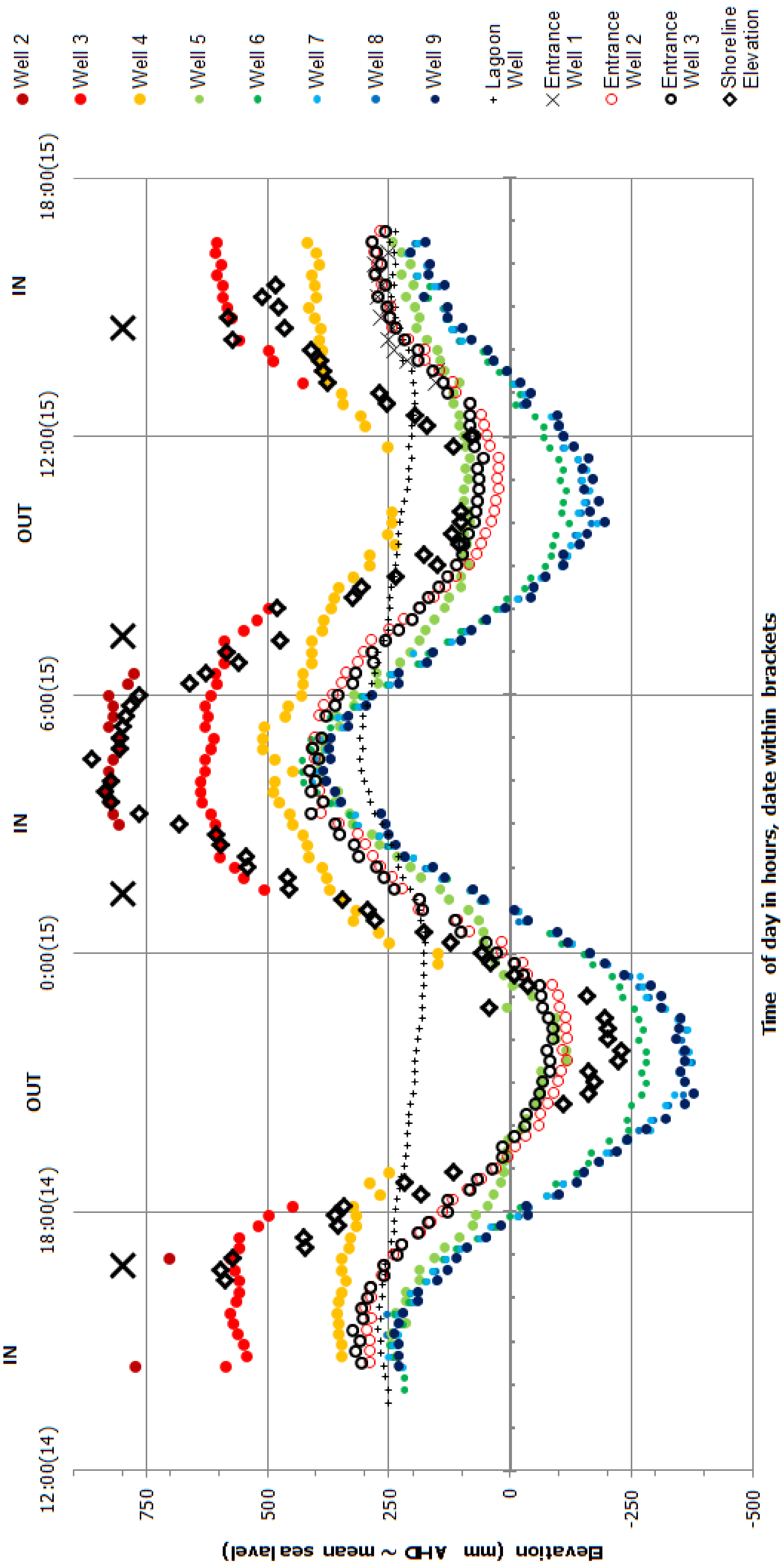

Figure 26 *Wonboyn water surface elevation data. The large ×'s represent the time when the current changed direction.*

The shoreline elevation was calculated by using the equation for finding the intersection point between two lines, see figure 27

$$\bar{\eta_s} = h_{os} + \frac{(h_{is}-h_{os})(h_{ow}-h_{os})}{(h_{is}-h_{os})(h_{iw}-h_{ow})} \tag{14}$$

where h_{is} is the sand elevation shoreward, h_{os} is the sand elevation ocean side, h_{iw} is the water surface elevation shoreward and h_{ow} is the water surface elevation ocean side.

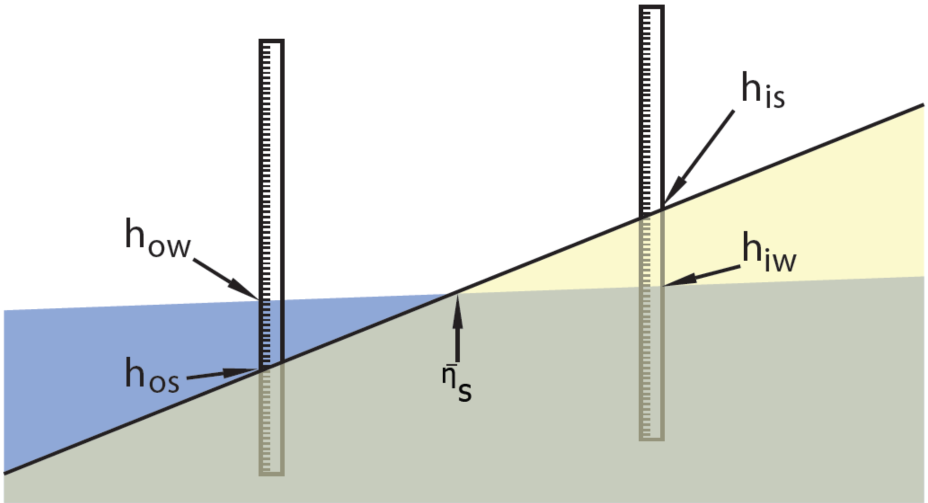

Figure 27 *Interpolated shoreline elevation.*

When comparing Well 9 with this interpolated shoreline elevation, the setup next to the open coast was in the order of approximately *200 mm* (min) at low tide and approximately *400 mm* (max) at high tide.

The slower reaction time of the lagoon is seen in figure 28, where only the Lagoon Well and wells 7, 8 and 9 were plotted. The lagoon did not keep up with the elevation changes of the ocean and entrance, which is seen by the elevation of the lagoon water surface not changing as much with time as the other water surface elevations.

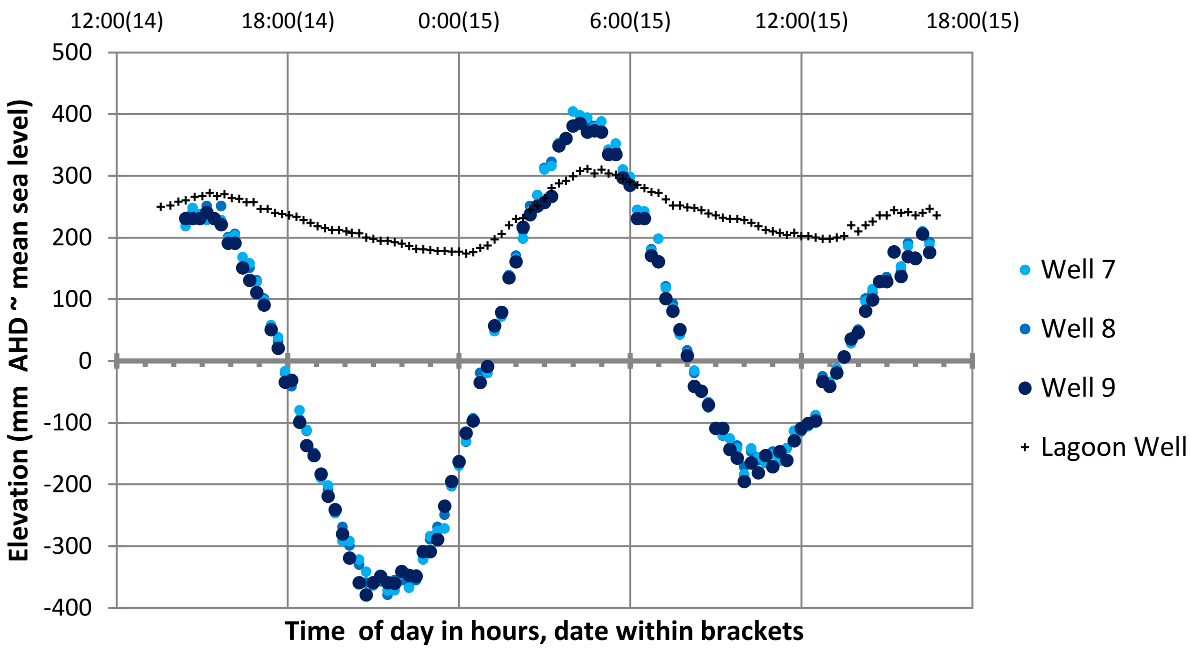

Figure 28 *The lagoon water surface elevation compared with the surface elevations in wells 7, 8 and 9. Well 9 is the well furthest off shore.*

Figure 28 shows that the lagoon level began to rise before it intersects the Well 7, 8 and 9, but that it began to fall almost instantaneously at the turn of the high tide.

The current monitor data shows that the current changed direction (marked with the large ×'s in figure 26) at the point where the entrance well levels were at the same level as the lagoon water level. With that in mind, that means that the lagoon and entrance well levels were approximately *100-140 mm* higher than the offshore water level when the current was at its turning points.

The data also clearly shows that to the left of the first × from the left, the offshore water level was lower than the lagoon water level, but that there was an inflow of water at the same time. The same phenomenon occurred to the right of the first × from the right.

The following set of four figures (29-32) show the measured setup profiles from the beach transect at different points in time. Figure 29 shows the setup profile from the start of the survey to the turn of the tide at about *21:30*. Figure 30 then follows in the same manner but during rising tide. Figures 31 and 32 follow the same pattern as figures 29 and 30.

The bottom profile found by using the Total Station has also been included. However, since this profile was surveyed only once, it should be noted that the bathymetry may have changed from when the water levels were recorded.

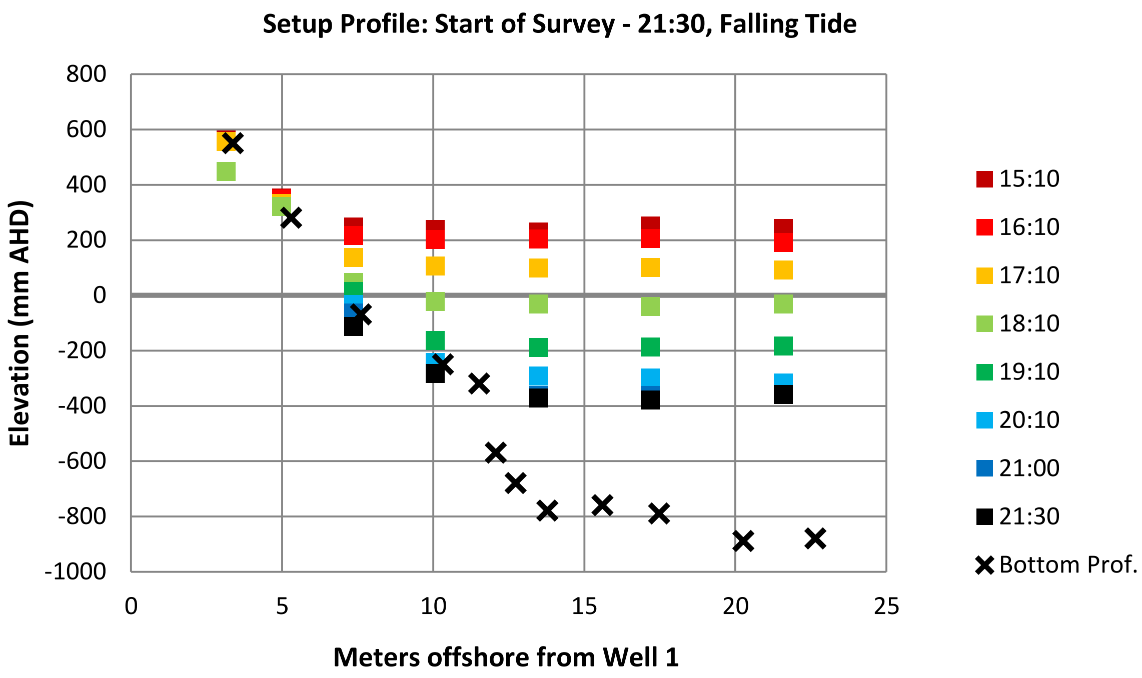

Figure 29 *View of the setup profile at different times from the start of the survey during falling tide to the turn of the tide at about 21:30 on 14/12. The point furthest to the right (offshore) is Well 9.*

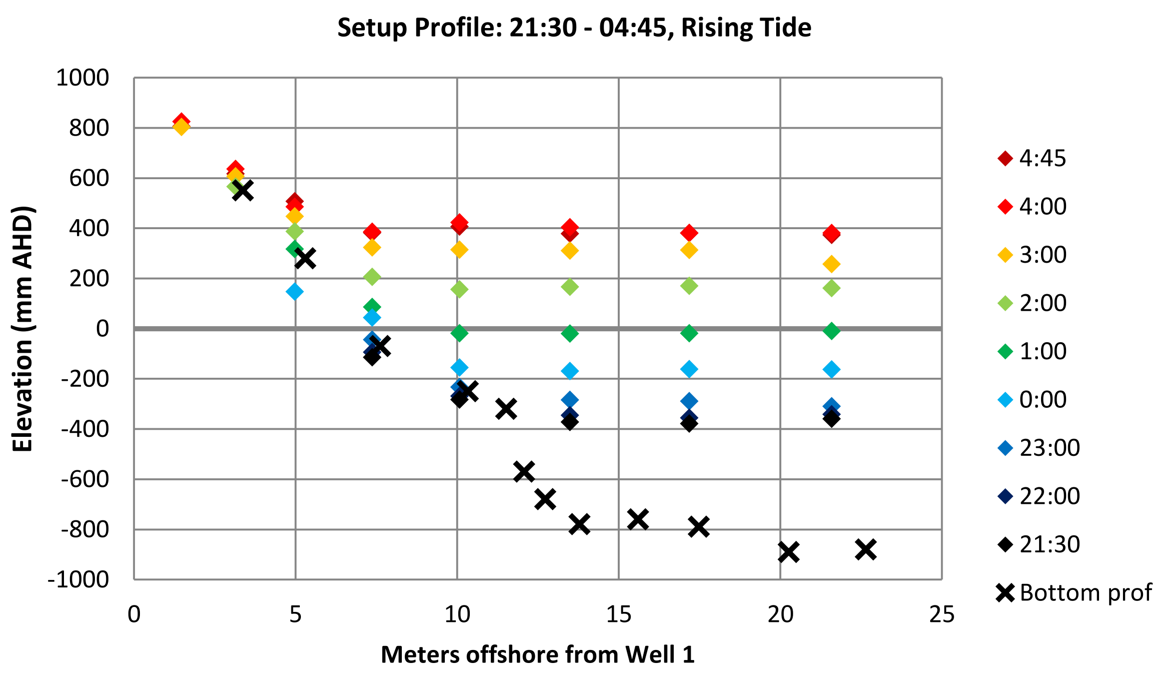

Figure 30 *View of the setup profile at different times during the rising tide from 21:30 (14/12) to the turn of the tide at approximately 04:45 (15/12). The point furthest to the right (offshore) is Well 9.*

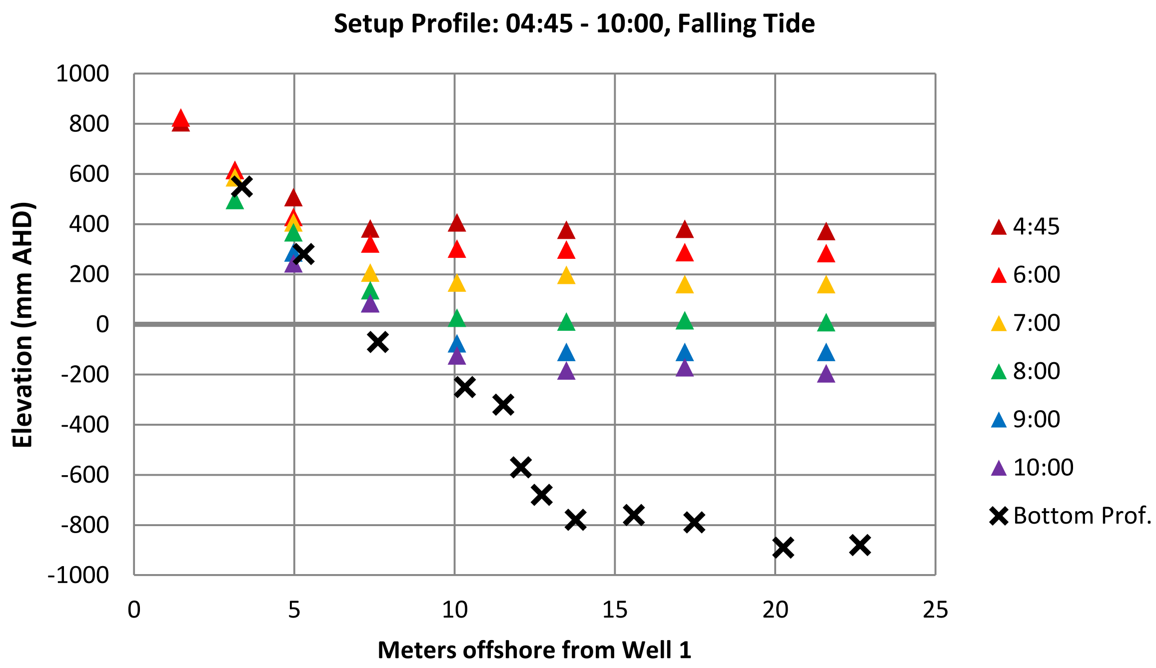

Figure 31 *View of the setup profile at different times during the falling tide from 04:45 (15/12) to the turn of the tide at approximately 10:00 (15/12). The point furthest to the right (offshore) is Well 9.*

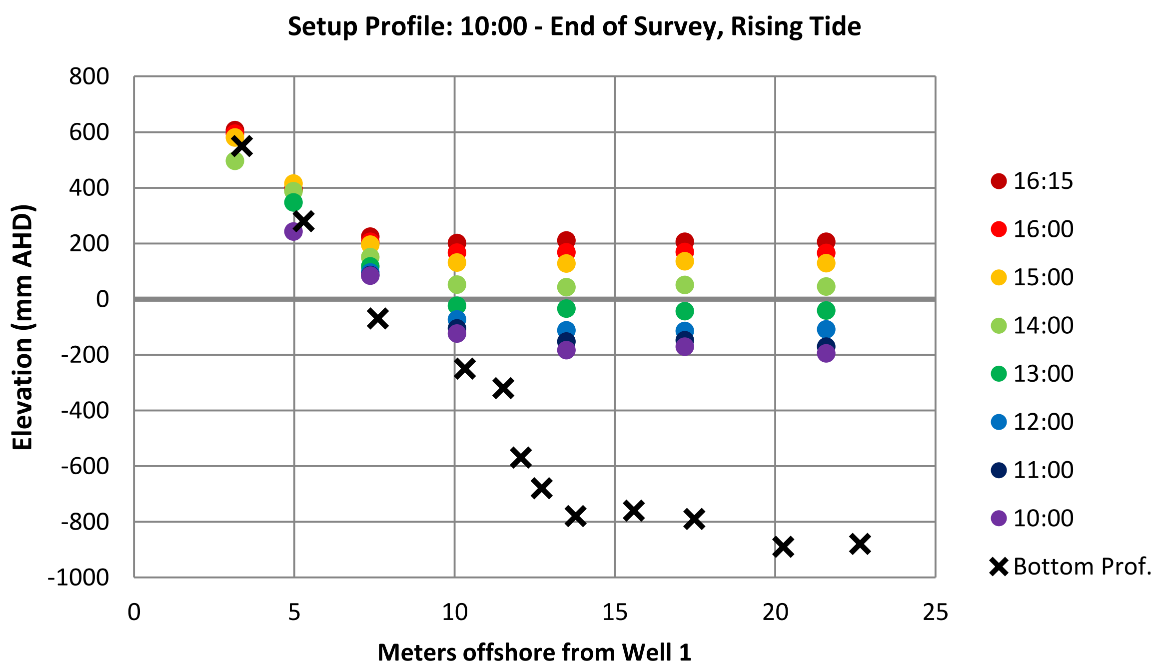

Figure 32 *View of the setup profile at different times during the falling tide from 04:45 (15/12) to the turn of the tide at approximately 10:00 (15/12). The point furthest to the right (offshore) is Well 9.*

By utilizing a Total Station during the Wonboyn field trip and taking several hundred data points, a virtual image of the survey area was created (figure 33) using a contour plot function in MATLAB.

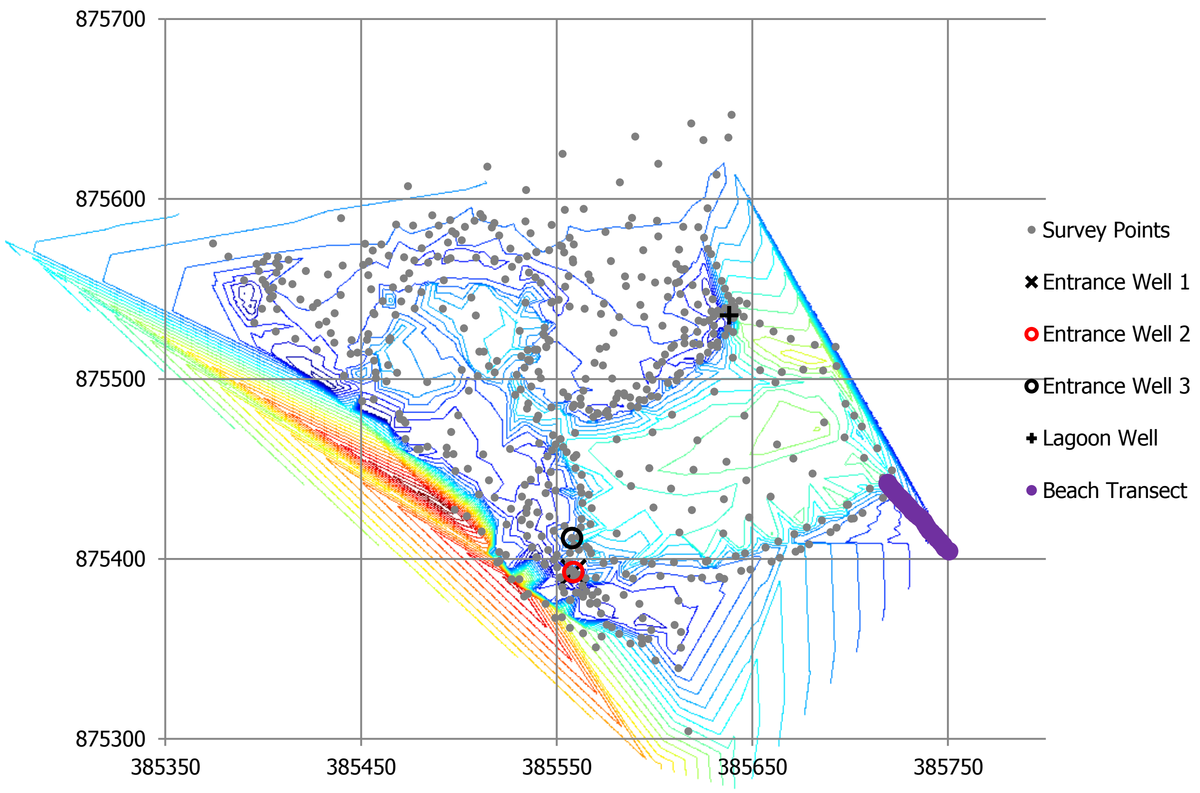

Figure 33 *Contour plot created by Paul Moric (2011) of the Total Station survey data from the Wonboyn Lake field trip. The x-axis is the easting in meters and the y-axis is the northing in meters. Light blue indicates lower elevations while darker blue and red indicate higher elevations. The different stilling wells are marked in the figure as explained by the legend.*

This scatter point data was later used to construct the 3D-model from which river cross sections were extracted in 6.1.3.

5.2 Tallebudgera Creek Estuary Data

The Tallebudgera dataset was collected during approximately *25 hours* on the *3rd-4th* of April 2011. There were originally 12 wells that were used in the survey. However, due to the special circumstances surrounding the severe beach erosion that claimed several of the wells, the lost wells needed to be reinstalled. There were therefore 'effectively' 18 wells, since the wells were renamed once they had been reinstalled.

The full plotted dataset is seen in figure 34. The black diamonds in the figure represent the interpolated shoreline elevation. Well 12 was furthest offshore in the first beach transect setup. Once all of the wells that fell had been reinstalled, New Well 7 was at approximately the same location as the old Well 7. Wells C, D, E, F and New G were shoreward of New Well 7with Well C furthest onshore.

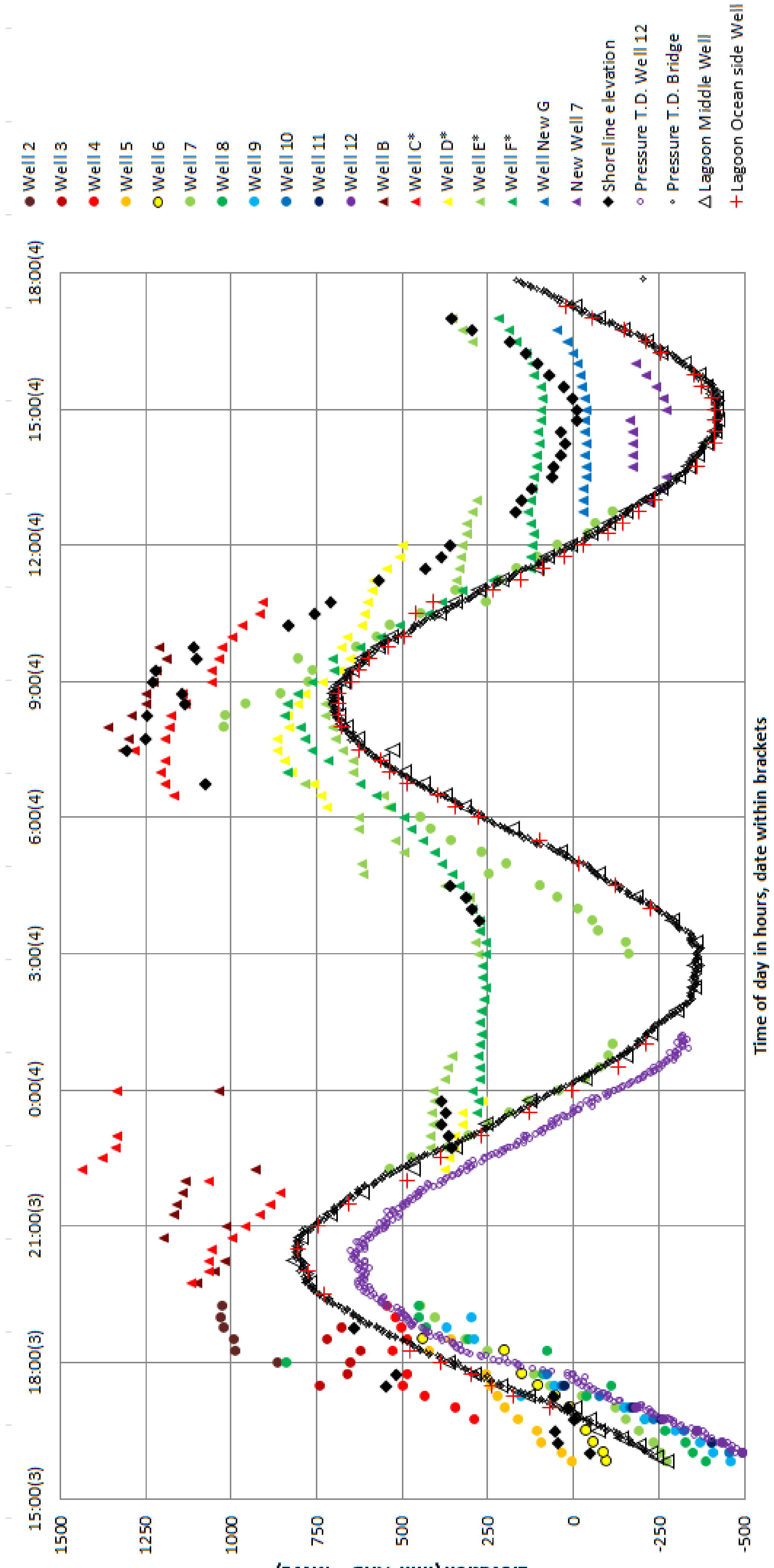

Figure 34 *The Tallebudgera Creek Estuary water surface elevation data.*

As seen in figure 34, the data from the beach transect was fragmentized during the first and middle part of the survey while it was possible to get consistent measurements during the last third of the survey.

The wells that are marked with an asterisk (Wells C, D, E and F) were fitted to the data within a reasonable range, but their real exact elevations are unknown. Therefore, they did not have any other function than helping to visualize the plot of the full dataset.

All wells, except for Well 7, fell down within a few hours of surveying and had to be reinstalled. Since the pressure transducer at Well 12 also fell, there were no offshore measurements recorded.

However, the transducer data from the lagoon position by the bridge as well as the middle and ocean side wells were not affected by the conditions out at the shoreline as seen in figure 35.

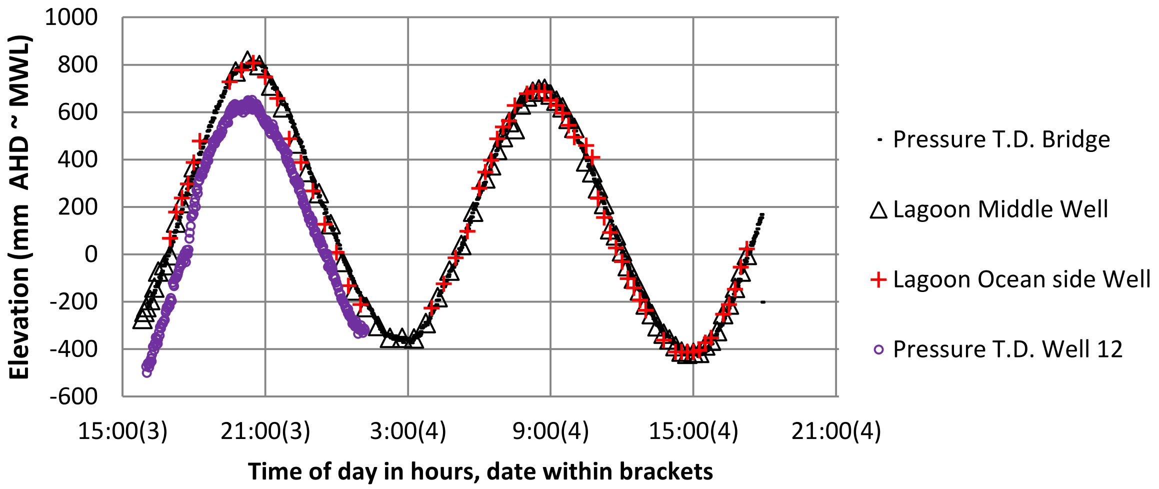

Figure 35 *The two lagoon wells (Middle and Ocean side) that produced consistent data throughout the survey period.*

There was also current monitor data available from two current monitors. However, since the bathymetry of the creek was not known, it was not possible to determine the flow. It was possible to ascertain whether there was an in- or outflow at each current monitor with each reading. In figure 36 the differences between the two manometer pipes inside the current monitors were plotted. A positive value indicates an inflow to the estuary while a negative value denotes an outflow.

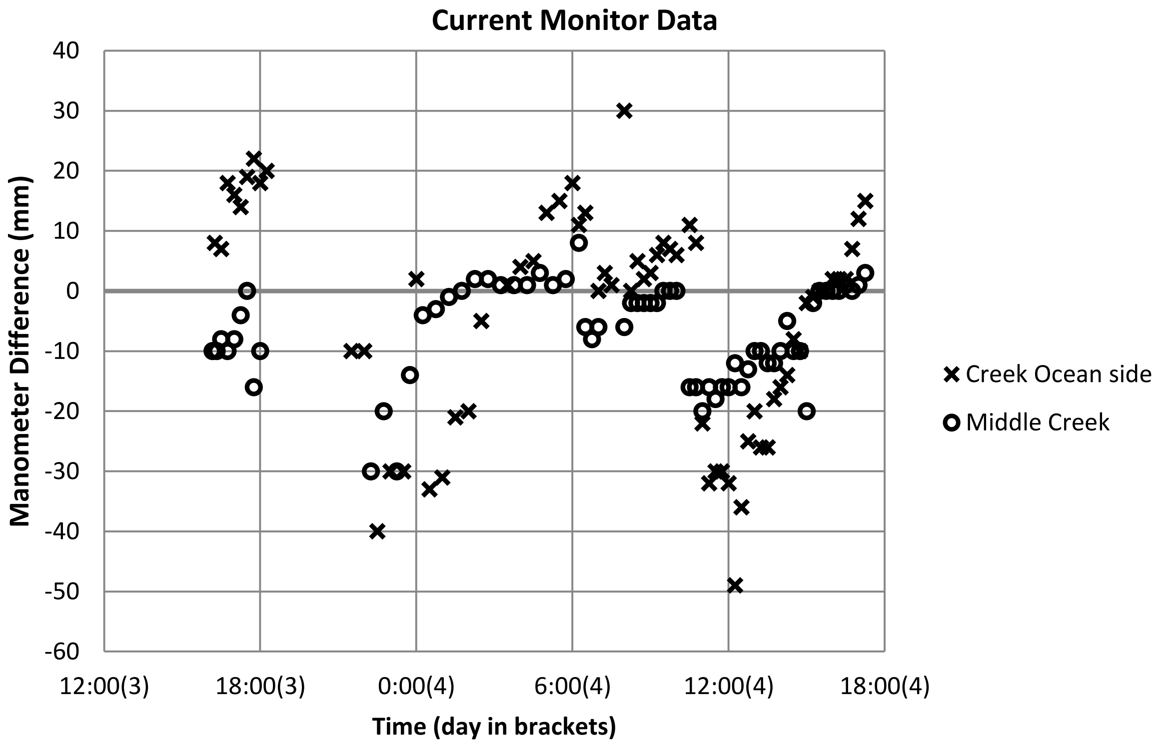

Figure 36 *Current monitor data from inside the creek. A positive value denotes an inflow while a negative value denotes an outflow.*

The current data shows that the two current monitors generally follow the same pattern with regard to in- and outflow except for in the beginning of the survey.

6 ANALYSIS AND MODELLING RESULTS

6.1 Wonboyn Lake Estuary

The Wonboyn data shows that the processes that were at work in the river entrance were complex. The offshore water surface elevations of wells 7, 8 and 9 were lower than the Lagoon Well and entrance wells 1 and 2 levels at the time of the turning current (when the net flow was zero). At that point in time there was a difference of about *100-140 mm* between the water surface elevations offshore and in the entrance.

The first hypothesis was that it was a measurement error in the level survey. However, two separate and unique level surveys were made and both came to the same conclusion within a few millimetres. These surveys were also double checked to make sure there were no intermediate level errors between the wells that may have been overlooked. That made it unlikely there was a measurement error in the survey.

Another hypothesis was that the water level difference may have had something to do with wave setup. However, the previous reports by Hanslow et. al (1992, 1996) and the analysis of their data by Dunn (2001) suggested that there is no wave setup in trained river entrances. Since the river entrance in Wonboyn is only trained against rock on one side it may behave differently compared to a completely trained river entrance.

The data shows that there was a clear measurable setup along the open coastline of up to approximately *400 mm.* The measured open-coast wave setup, along with a measured current of water flowing into the entrance during high tide, despite there being a lower water level out at sea compared to the entrance, is a clear indicator that wave setup (or some other phenomena) was raising the water surface level close to the entrance and thus enabling an inflow.

If the difference in surface elevation had something to do with wave setup, it can be hypothesised that the wave setup was mostly present during low tide when the water depth at the lagoon entrance was small, since wave setup increases as depth decreases.

Below is a figure of the interpolated shoreline elevation along with a plot of an empirical relationship (eq.11) for wave setup along a shoreline. That empirical formula shows that the calculated wave setup elevation at the shoreline was within a reasonable range. However, it does not fully agree at around *22:00*. Eq. 11 relies only on wave conditions and since the conditions may change from the wave buoy to the shore, the empirical shoreline elevation is probably still within reasonable range.

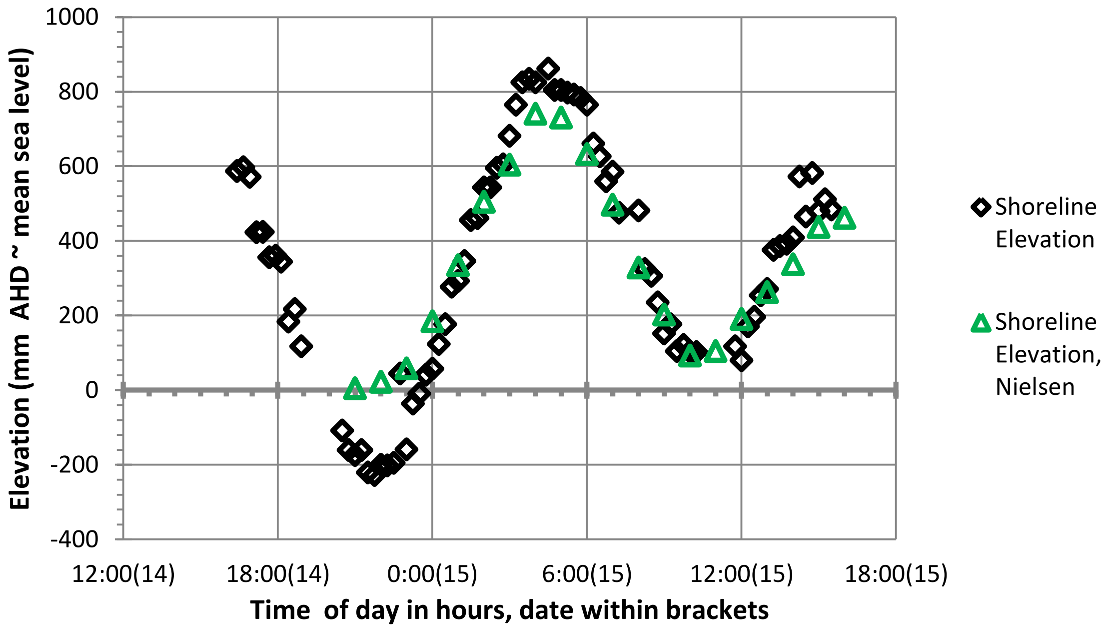

Figure 37 *The interpolated shoreline elevation compared with the theoretical calculated shoreline setup using Peter Nielsen's eq. 11.*

6.1.1 Analytical Wave Setup Model

Due to the water surface elevation difference found in the entrance between the offshore wells and entrance wells there was enough reason to hypothesize that there may have been wave setup in the river entrance to some degree. An analytical wave setup model was hence made in a spread sheet to see if the aforementioned hypothesis carried any weight. The first part of the process was to model the MWL along the shoreline and then, once it had been calibrated, attempt to model the water surface elevation in the river entrance.

The analytical model used a more general solution to the wave setup equation than that given by eq. 9 from Nielsen (2009) and the solution to this analytical wave setup equation by Dean and Dalrymple, (1991) is as follows

$$\bar{\eta} = \bar{\eta}_b + \frac{\frac{3}{8}\gamma^2}{1+\frac{3}{8}\gamma^2}(h_b - h) \tag{15}$$

where h_b is the breaking wave depth, $_b$ denotes breaking wave conditions and h is the water depth from the bottom to the SWL. It would have been possible to use the analytical solution by Longuet-Higgins for wave setdown at the breaking point, but for this case the wave setdown was assumed to be zero for simplification purposes.

The bathymetry data from the Total Station survey and the offshore water surface elevation (Well 9) were both extracted from the Wonboyn Dataset (2010). The bottom profile of the entrance is seen in figure 38.

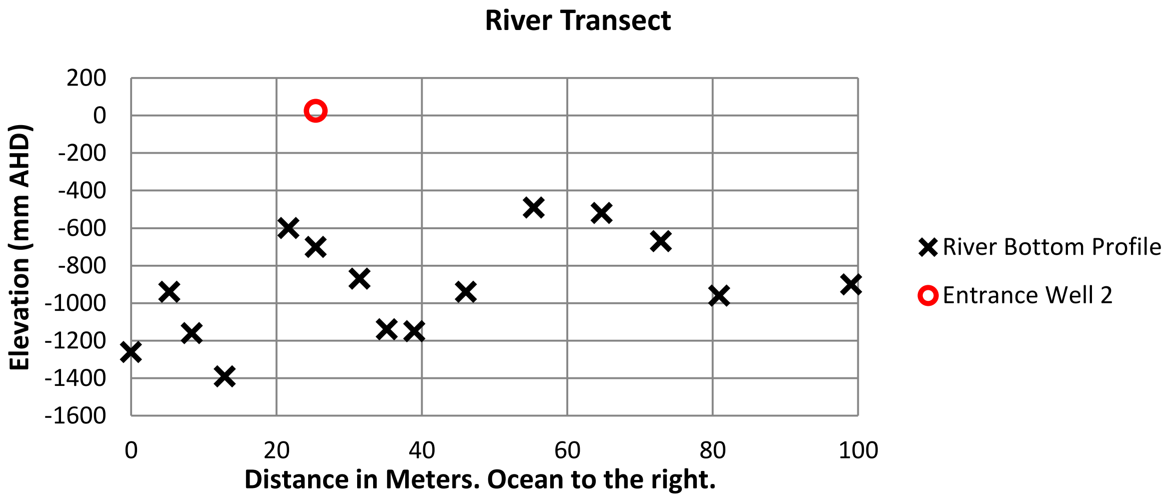

Figure 38 *The bottom profile of the river entrance. The red circle marks the approximate x position of Entrance Well 2.*

The waves were assumed to be monochromatic and that they approached the shore normally. The break index γ was subsequently set to *0.78* for monochromatic waves.

Since waves broke mainly outside of Well 9 during low tide there was the possibility that Well 9 already included some degree of wave setup, at least during low tide. The following analytical model was calibrated with the boundary condition that it had to use the same water surface elevation as Well 9 at the offshore point. In order to achieve that without using a variable breaking water depth (e.g. to use the depth from Well 9 directly from each data point), the following variant of eq. 15 was derived. For the newly derived equation to work the assumption that $\bar{\eta}_m << h_m$ needed to be made. First, let

$$K = \frac{\frac{3}{8}\gamma^2}{1+\frac{3}{8}\gamma^2} \tag{16}$$

If waves with a breaking height H_b break at a water depth h_b, then the wave setup η_m at some shoreward located point x_m, where the water depth is h_m, is given by

$$\bar{\eta}_m = \bar{\eta}_b + K(h_b - h_m) \tag{17}$$

If eq. 15 is rearranged, it yields

$$h_m = h_b - \frac{\eta_m}{K} \tag{18}$$

assuming that the setdown at the break point is neglected. If the MWL is measured at the point x_m where $d_m = h_m + \bar{\eta}_m$, eq. 16 can be re-written as

$$\bar{\eta}_m = (h_b - d_m)\frac{K}{1-K} \tag{19}$$

From eq. 17, the SWL is obtained as $h_m = d_m - \overline{\eta}_m$. Thereafter, the SWL can be calculated for the other shoreward points as long as the bottom profile is known when the MWL is calculated using eq. 15.

Komar and Gaughan (1972) proposed a semi-empirical formula for breaking wave height which is written as

$$\frac{H_b}{H_o} = 0.56\left(\frac{H_o}{L_o}\right)^{-1/5} \qquad (20)$$

where the subscript $_o$ refers to deep-water conditions and L is the wavelength (the deep-water wave length $L_o = 1.56T^2$, where T is the wave period). Refraction was neglected in eq. 18 and the waves were assumed to approach the beach close to shore normal. The breaking depth was finally given by

$$h_b = H_b/\gamma \qquad (21)$$

Three main periods of interest were chosen for the model: *21:30 (14/12)* low tide, *01:30 (15/12)* at the time of no in- or outflow in the entrance and at *04:30 (15:12)* during high tide.

An average of the mean deep-water waves from the wave buoy data obtained from the NSW Department of Commerce (2010) was calculated to $H_{mean} = 0.75$ *m* and the average period T, was calculated to $T = 5.0$ *s*.

Open Coast

The model was then set up with the input shown in table 1 and was combined with the surface elevation at Well 9. The result of the calculated setup profile is seen in figure 39.

Table 1 *The input for the analytical wave setup model.*

Parameters	**Values**
γ	0.78
K	0.186
H_b (m)	0.926
L_o (m)	39.0
T (s)	5.0
H_o (m)	0.75
h_b (m)	1.187

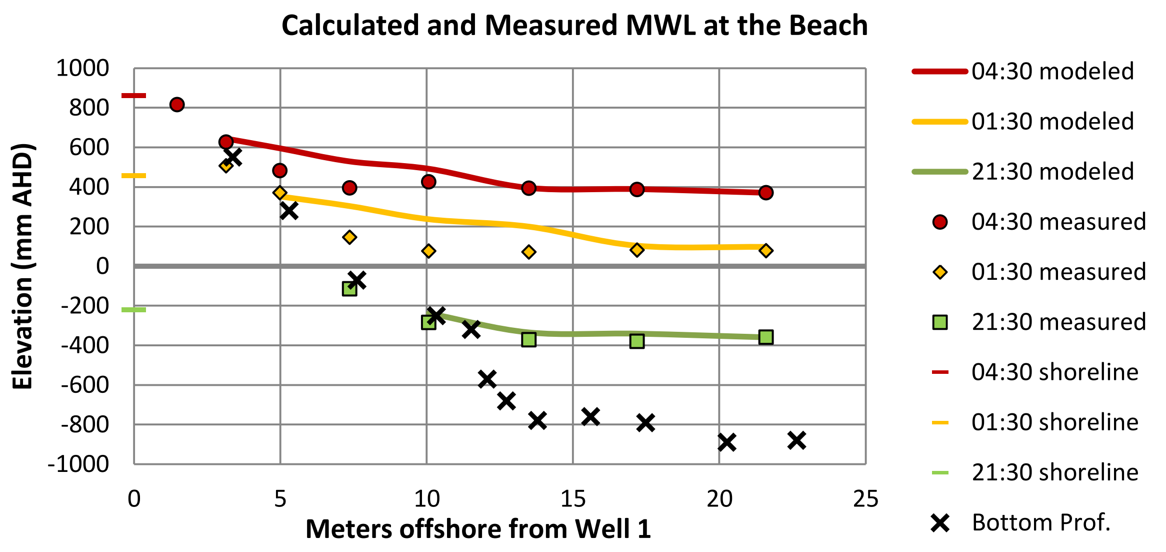

Figure 39 *The analytical solution to the wave setup plotted against the measured MWL. The interpolated shoreline elevation is seen as the short lines to the left in the chart.*

Figure 39 shows that the model overestimated the wave setup in general, but underestimated the shoreline elevation. The exception is at low tide, where the model was in fairly good agreement with both the measured MWL and the shoreline elevation. This might suggest that at low tide there was already some degree of setup included in the readings from Well 9 since the waves broke outside Well 9. There may also have been a small degree of setup at high tide since some waves broke outside Well 9 then as well, which is discussed in the next paragraphs.

When calibrating the model using Well 9 as the boundary condition the setup (without the tidal signal) was obtained through eq. 19, see figure 40.

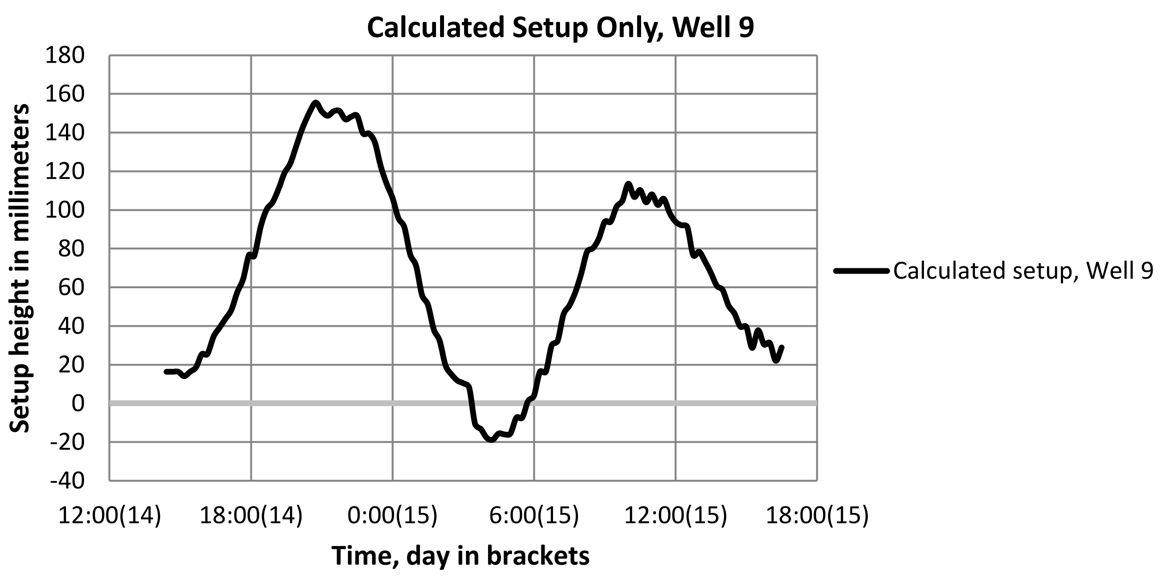

Figure 40 *Calculated setup at Well 9 as a function of time.*

Figure 40 shows a negative setup (positive setdown) at low tide. However, since there was no evidence of a measured setdown, an assumption that the setup was zero at that point should be made. It is therefore plausible that there was no setup when the water surface elevation was at its highest point in the middle of the survey (around *04:00*). However, the model indicates there may have been a small degree of setup in the beginning and at the end of the survey, which correlates with some waves being observed breaking outside Well 9 at several points in time during high tide.

Figure 40 correlates with the data in figure 26 as well, where Well 9 was lower than the entrance wells at the beginning and end of the survey, even though there was an inflow into the estuary.

River Entrance

Some of the most interesting points in time during the survey were when the current changed direction in the river entrance. Those time points were marked as large ×'s in figure 26. At those locations, the current in the entrance was recorded as being zero by the current monitors, even though there clearly was a difference of approximately *100-140 mm* at each zero current time point between Well 9 and the entrance wells. As previously discussed, it is not likely that the discrepancy was due to measurement error since two different level surveys were made, and both came to the same result.

When running the model using the bottom profile data for the river entrance, the following profiles were calculated (figure 41). The reader should note that the simulation was stopped after the first 'bump', i.e. after the first shallow part to the right in the figure since the setup model only applies with slopes that have the same sign as the slope of the nearby beach. The straight lines to the left of approximately $x = 55\ m$ in the figure are merely illustrative and show what the raised water surface would look like if the wave setup did raise the water surface elevation as calculated.

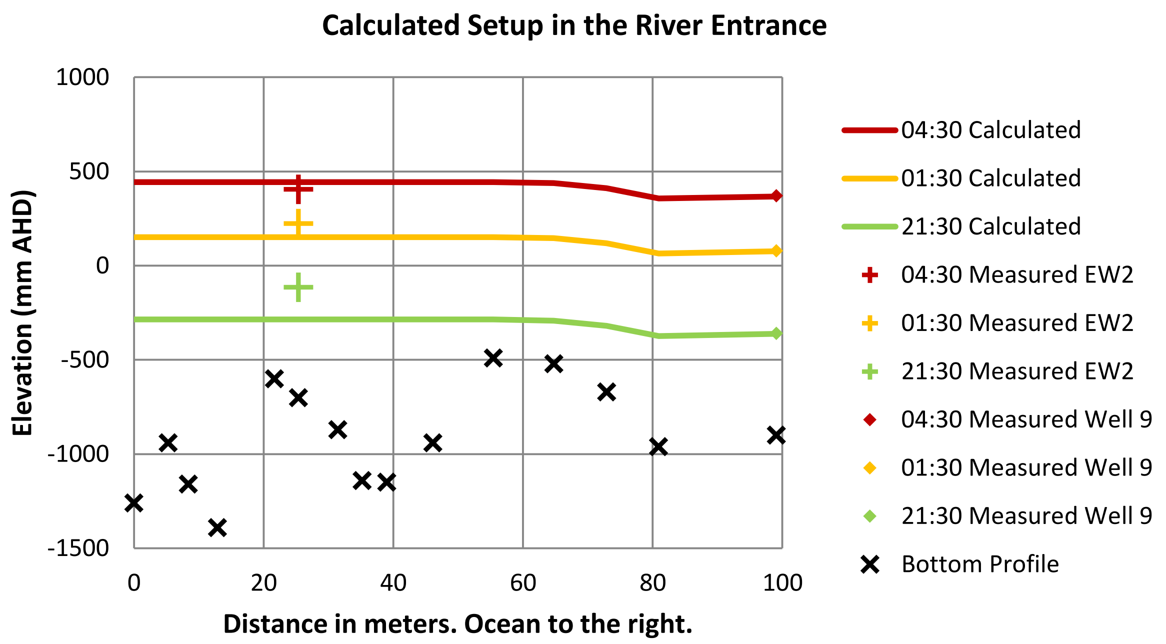

Figure 41 *Output from the analytical wave setup model when the river entrance bottom profile is used as input. The crosses are the measured water surface elevations in Entrance Well 2.*

The output in figure 41 underestimates the water surface elevation at both *21:30* and *01:30*. At *21:30* it was not surprising that it would underestimate the water surface elevation since there was a significant outflow from the lagoon at that time and the water level of the lagoon was still fairly high. That the model overestimated the water surface elevation at *04:30* was also acceptable since there was an observed inflow into the estuary. If there was an inflow, the wave-current interaction should have enabled the waves to break later, or not at all, around and inside the river entrance. If the waves were dissipated as they went in the same direction of the current, there should not have been any wave setup.

As mentioned before, *01:30* was one of the most interesting points in time since the net flow was zero in the entrance. The difference between the modeled water surface elevation at Entrance Well 2 and the measured elevation was approximately *70 mm*. Since the difference between Well 9 and Entrance Well 2 at *01:30* was approximately *140 mm*, the calculated setup may have accounted for half that difference if the model was accurate.

However, since eq. 15 was mainly derived for calculating wave setup along an open coastline it was difficult to assess how well it would represent wave setup in a river entrance. The concept around wave setup is that the coastline stops the momentum flux from propagating further onshore, which raises the MWL. If there is nothing to effectively stop this flux, e.g., there is an open river entrance next to the shoreline, the logical conclusion would be that there should not be any wave setup. However, as the data shows, there was a clear difference in water surface elevation when the flow in the entrance was zero which contradicts that conclusion.

6.1.2 Modelling the Estuary

It is possible that the morphology of the estuary can explain the occurring phenomena in the lagoon entrance and that the friction in the first *500 meters* enabled the water surface elevations to differ by a fairly large amount over relatively small distances.

It was therefore decided that the estuary should be modelled using a 1-dimensional hydraulic model to investigate if it was possible to replicate the measured conditions in a simulation. HEC-RAS (*Hydrologic Engineering Center's River Analysis System, version 4.1.2*), developed by the USACE, was chosen as the simulator since the program is free to use without charge and is easy to set up.

HEC-RAS can be used for many different kinds of river analysis projects and can run both steady and unsteady simulations. Among the standard hydraulic functions it can also simulate, e.g., sediment transport and temperature dispersion. Another program made by Aquaveo, called WMS (*Watershed Management System, version 8.4*), was used to model the estuary bathymetry and cross sections due to an efficient graphical interface, a function that HEC-RAS still lacks.

6.1.3 Building the Model in WMS

The first step was finding bathymetry data for the estuary that would be representative in a simple computer model. All of the bathymetry data from the 1998 study by the Estuary Management Program (see 4.2.1) was made available in an xyz scatter plot containing easting, northing and bathymetry data which enabled an easy transaction from scatter points to triangulated surfaces, see figure 42.

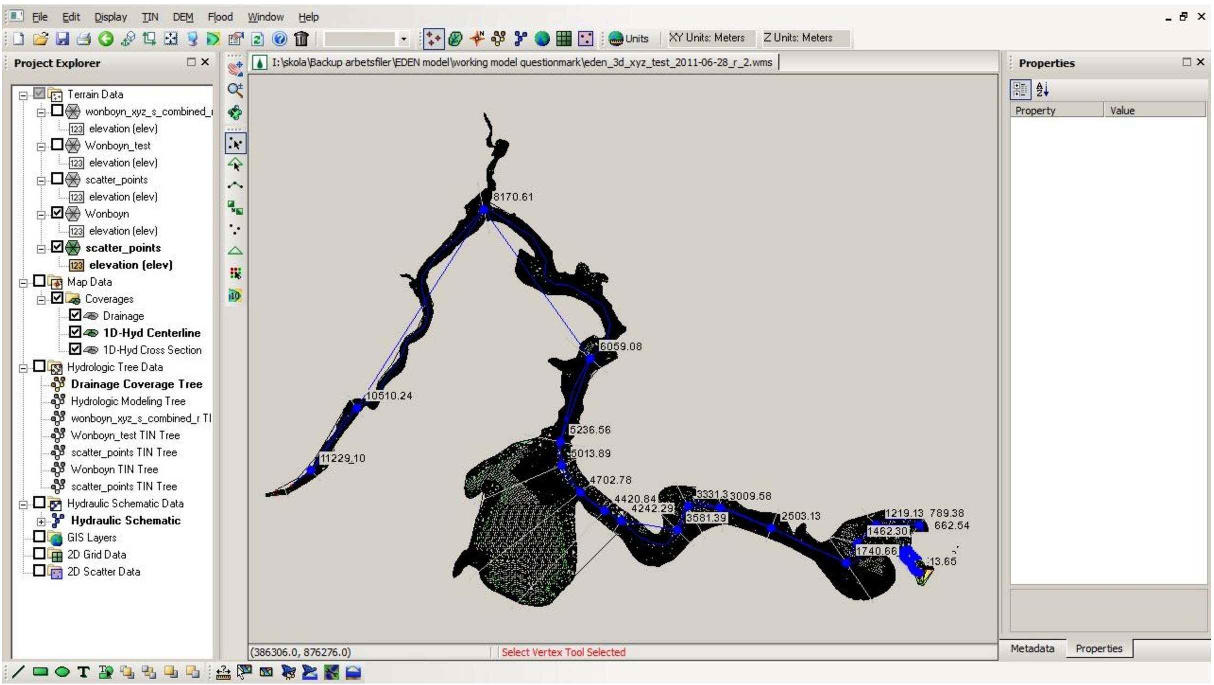

Figure 42 *Overview of the model of Wonboyn estuarine lake as seen in WMS 8.4.*

However, since the xyz scatter data had been recorded in 1998 there had been significant changes in the morphology of the estuary entrance. There had likely been changes in the morphology of other parts of the estuary as well, but since there was no data more recent than the 1998 data that had to suffice. The bathymetry data that was recorded during the 2010 December field trip therefore needed to be combined with the 1998 dataset in order to get a fairly up-to-date model. To accomplish that, the older data points surrounding the entrance were removed and the newer data points were inserted at their correct coordinates instead. The result of that combination is seen in figures 43 and 44.

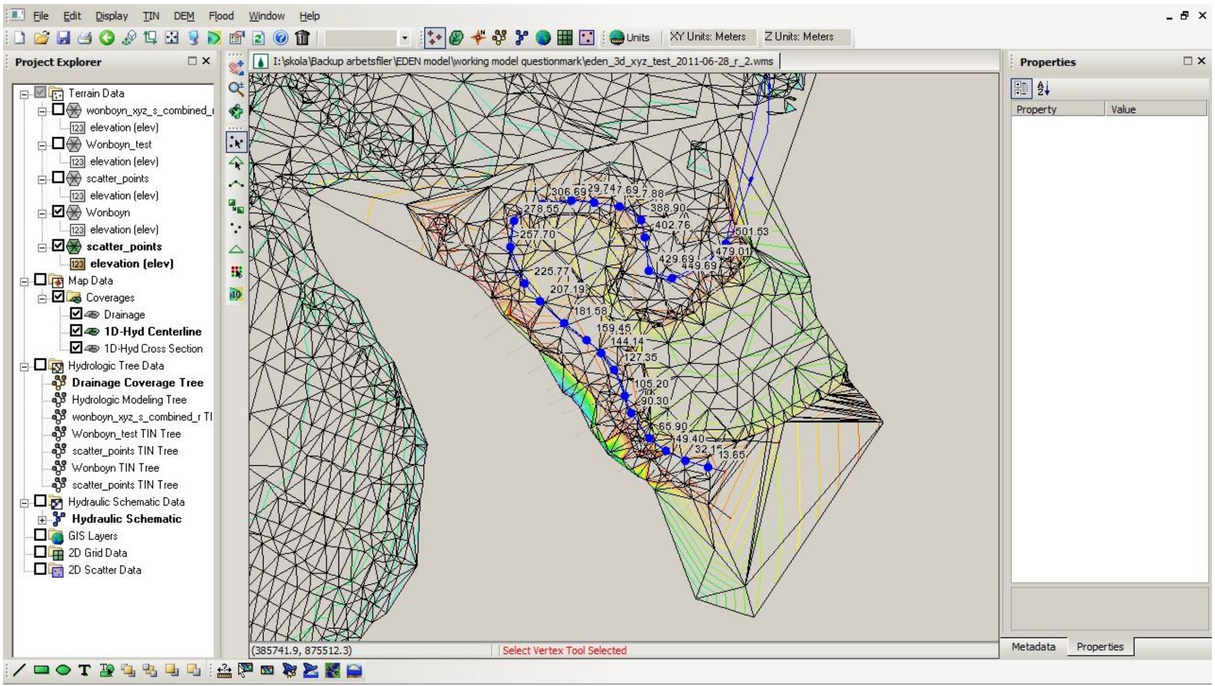

__Figure 43__ Image from WMS 8.4 of the triangulated entrance after the 1998 and 2010 data points had been combined.

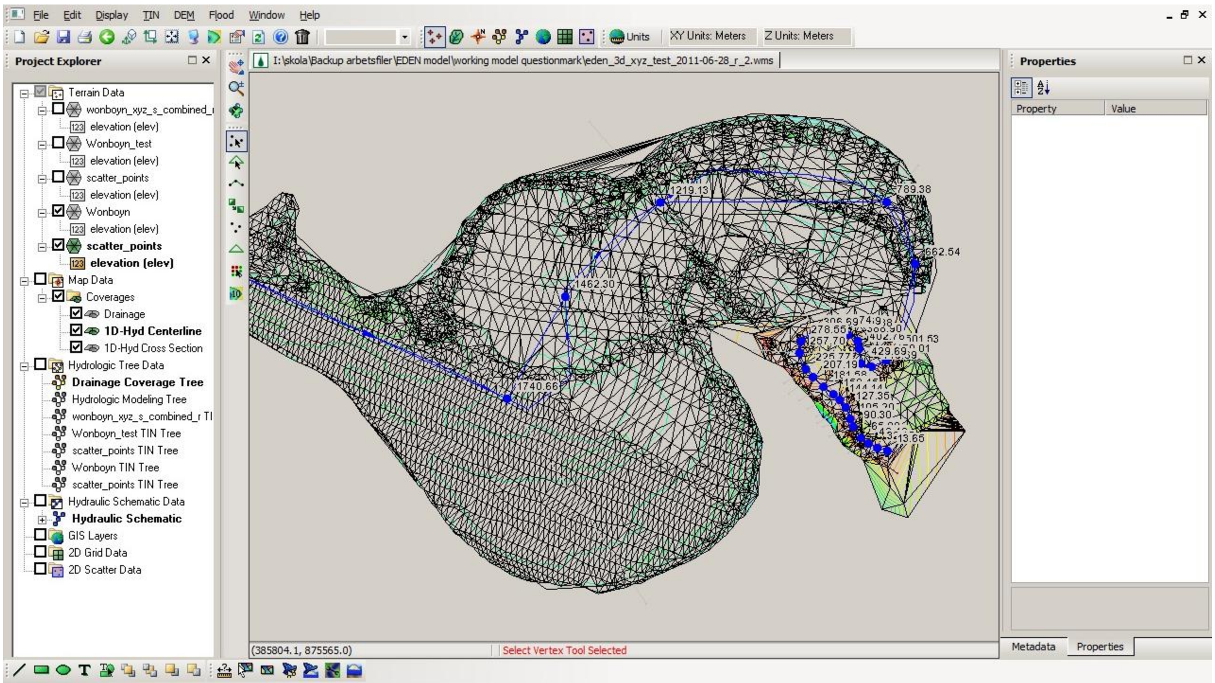

Figure 44 *Image from WMS 8.4 with a larger overview of the triangulated entrance to the estuary after the data points had been combined.*

When the bathymetry had been triangulated, an outline of the river was laid out. Its full layout is seen in figure 42 as the blue line. A number of cross section locations were chosen to represent the bathymetry for HEC-RAS. Since the entrance was of highest interest, the concentration of cross sections was made higher in that area over a distance of approximately *500 meters* upstream from the sea. The furthest upstream reaches of the river system were neglected to simplify the model. When the cross section locations had been chosen, the model was exported to HEC-RAS for simulation. The simplified model in HEC-RAS is seen in figure 45. A zoomed in image of the entrance is found in figure 46.

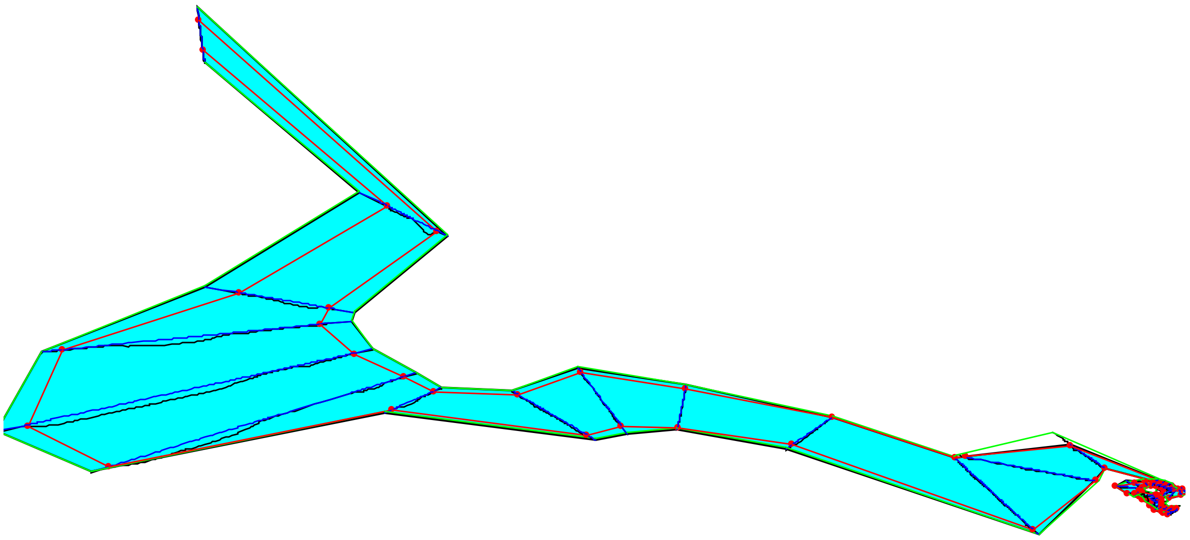

Figure 45 *The model seen in its simplified state in HEC-RAS 4.1.2 with the location of the cross sections.*

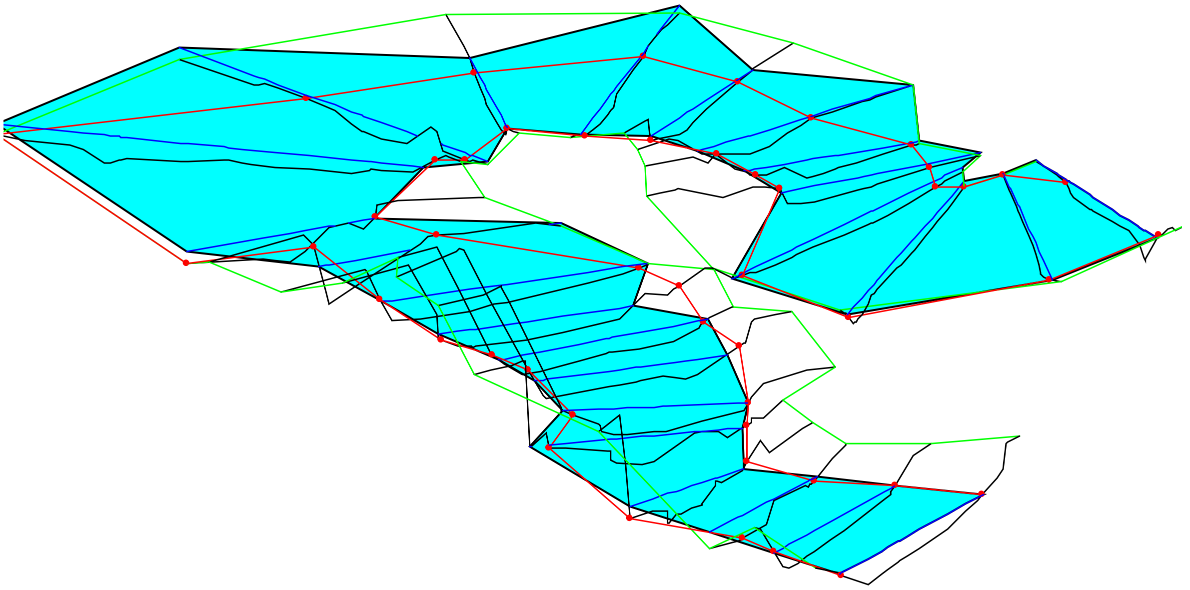

Figure 46 *Close-up of the simplified entrance and the location of the cross sections.*

The grid at the entrance was chosen to run as far out into the entrance as shown in figure 46. It is possible the model would have reacted differently if a larger grid further out into the ocean had been chosen.

One simplification with the model was that the 1998 xyz-scatter point data did not include any topographical features around the estuary itself. In other words, the model automatically used vertical walls along the outlines in those cases. An example of such a cross section is seen in figure 47.

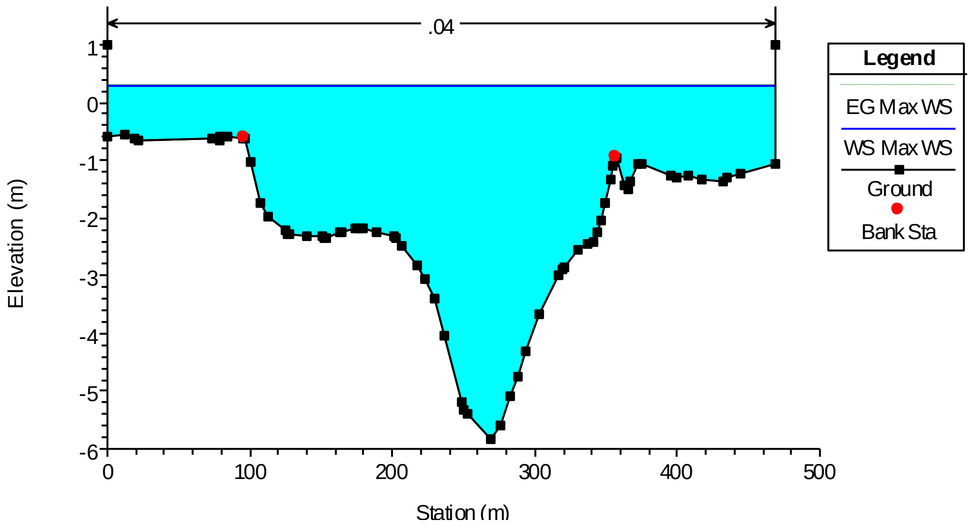

Figure 47 *Cross section of Wonboyn Lake showing the lack of topographical data along the edges.*

However, in most cases, the cross sections were complete and looked similar to figure 48.

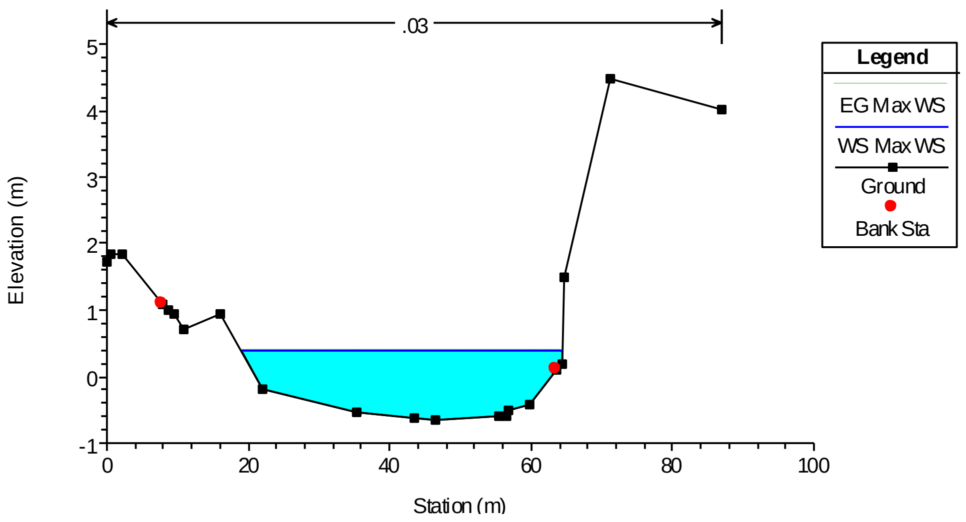

Figure 48 *Cross section from the seaward side of the lagoon entrance showing the bathymetry.*

6.1.4 Setting Up and Running the Model in HEC-RAS

Before the sensitivity analysis, a few simulation runs using the default program values were conducted to see if the model would work properly and if its output would resemble the measured values without any calibration. Manning's number was set by default to $n = 0.03$ for the entire estuary and the inflow, i.e., runoff, was set to $0\ m^3/s$. To start the model, a base flow larger than $0\ m^3/s$ needed to be entered. A value of $Q = 0.1\ m^3/s$ was chosen since a flow small such as that would not significantly affect the water surface elevations.

The model was given the downstream boundary condition that it would utilize the water surface elevation data recorded at Well 9 as the driver for the model. The output from that simulation is seen in figures 49 and 50.

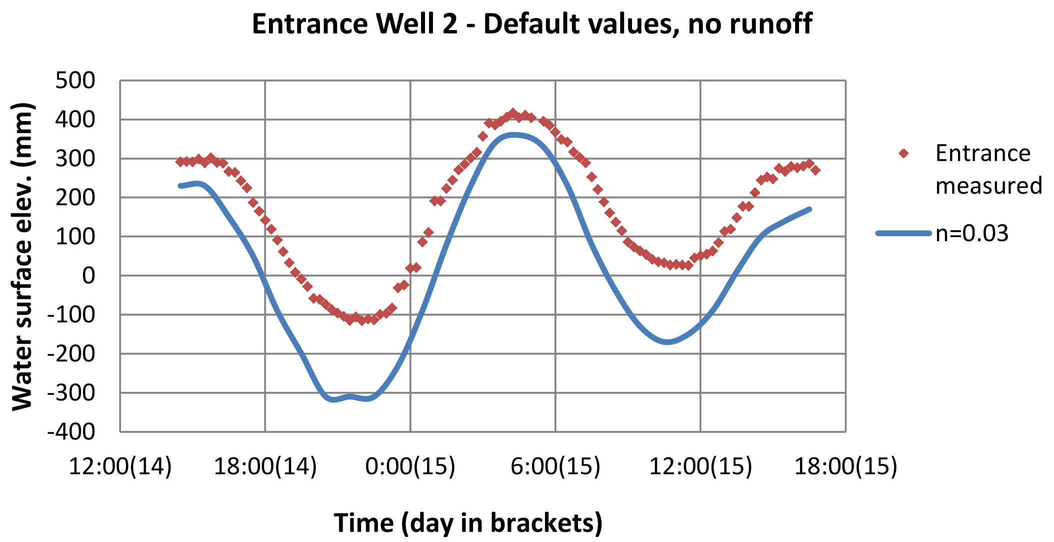

Figure 49 *The output at the entrance when the default HEC-RAS values were used, i.e. no runoff and* n=0.03.

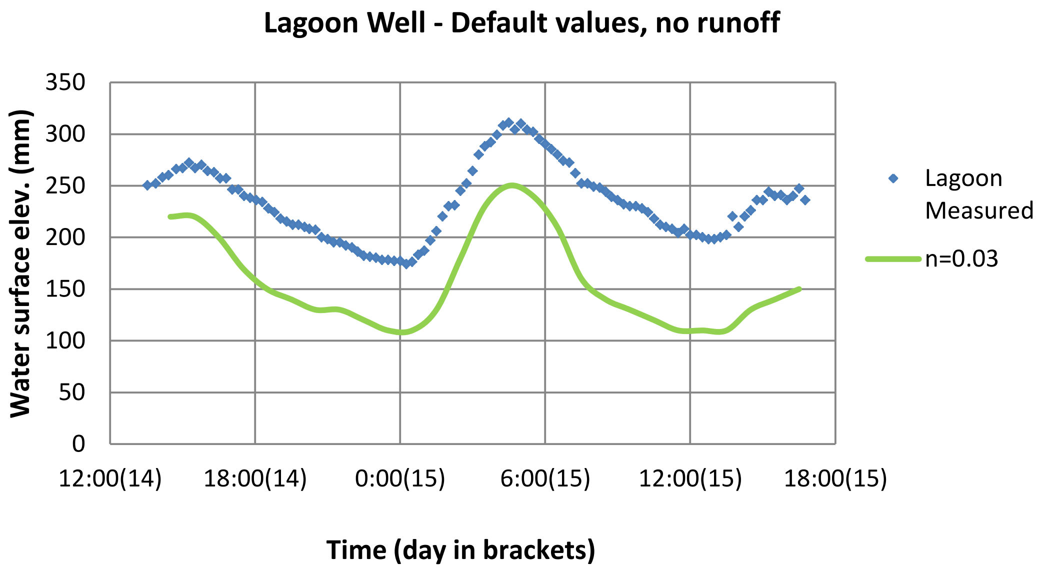

Figure 50 *The output at the Lagoon Well when the default HEC-RAS values were used, i.e. no runoff and* n=0.03.

As the figures show, the model output did not fit the measured data. In the entrance, the modelled curve shows a larger difference during low tide than during high tide. The shape of the modelled Lagoon Well curve shows a similar shape, but that the water surface elevation is underestimated.

The hypothesis that there may have been wave setup to some degree in the entrance could explain the larger difference between measured data and the model at low tide. If the model underestimated the water surface elevations in the estuary that means there is some extra driving force at hand that is missing in the model.

However, the Manning number and runoff had in this stage not been calibrated at all and such changes would likely produce another output.

6.1.5 Sensitivity Analysis

A sensitivity analysis was made changing Manning's number n and the runoff Q_r to see how those parameters would change the output of the model. The first step was to change n to different values for the entire estuary. The second step was to change the runoff flow Q_r from the river, and the third and step was to manipulate n for individual river cross sections and combine those changes with changes in Q_r.

In this first step (figures 51 and 52) the value of n was changed to values ranging from *0.03-0.1* and the output was compared with the measured water surface elevation in the entrance recorded at Entrance Well 2.

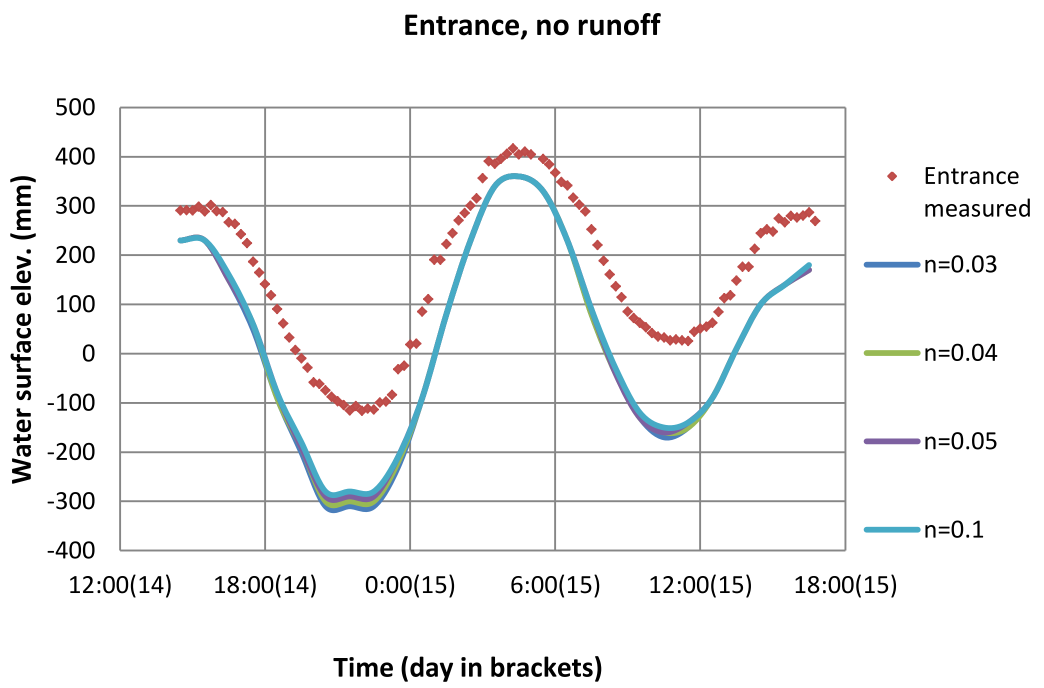

Figure 51 *First sensitivity analysis of changing the values of Manning's number* n*. The lines are the outputs at the river entrance from the different* n *values and the red dots are the measured water surface elevations at Entrance Well 2.*

As figure 51 shows, the difference in result was not large between the different Manning numbers in the entrance and it did not bear a close resemblance to the measured data at low tide. If the hypothesis that there may have been wave setup at low tide is correct, it might explain the greater offset at low tide between the measured data and modeled output since there should have been more wave setup when the water depth was small.

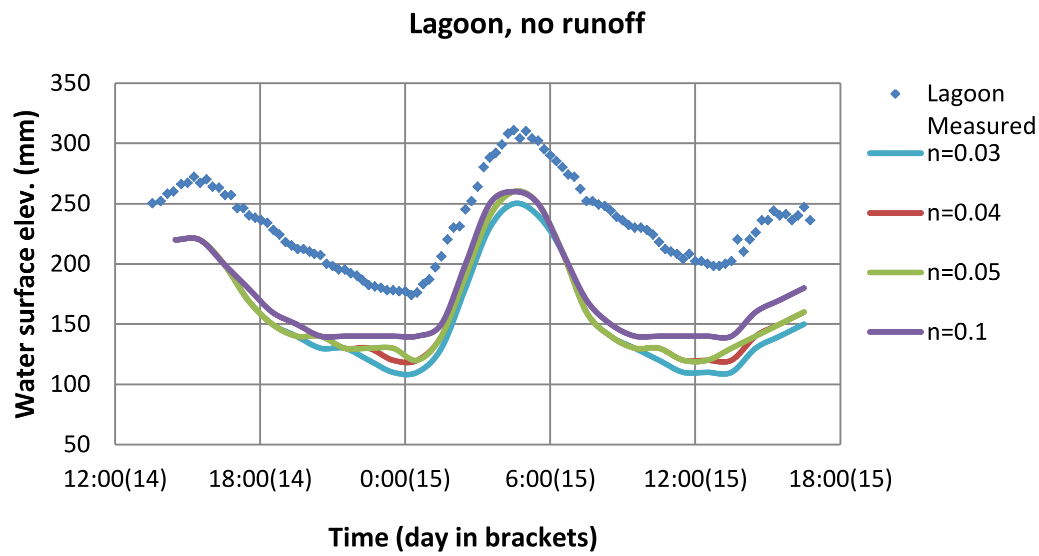

Figure 52 *First sensitivity analysis of changing the values of Manning's number* n*. The lines are the outputs at the Lagoon Well location from the different* n *values. The blue dots are the measured water surface elevations at the Lagoon Well.*

Figure 52 shows that the model predicted the shape of the measured water surface at the Lagoon Well fairly well. However, as seen in the image there was an offset of approximately *50-100 mm* between the modeled and measured values. Since the original runoff was small (*0.1* m^3/s) for the size of the river system, a bigger runoff was applied to see what kind of difference it will make.

Figure 53 shows a split section through the entire modeled part of the river system. The modeled water surface profile gives an overview of the hydraulic behavior of the estuary. The first *500 meters* upstream from the ocean show how the entrance was affected by the changes in water surface elevation offshore.

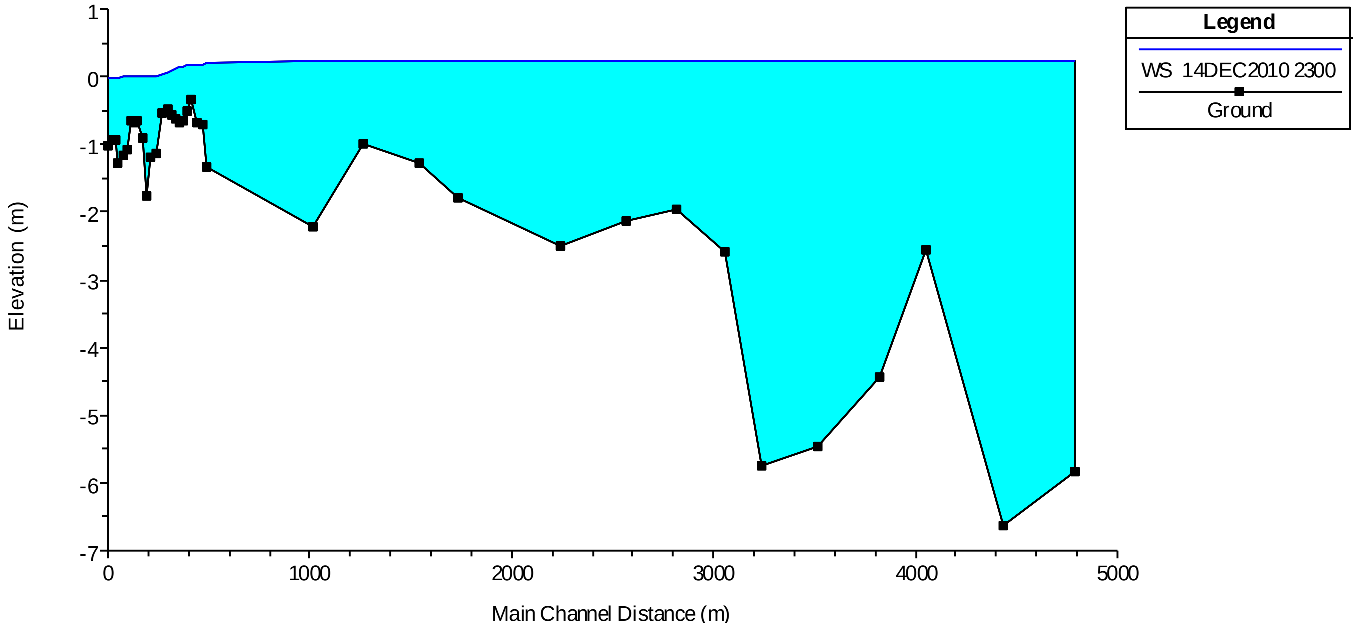

Figure 53 *Section through the entire part of the modeled Wonboyn river system. The ocean is to the left and the river and lake is to the right.*

Since changing the Manning's number did not alter the output by a very large amount, not even when changing n to the fairly high value of *0.1*, the runoff flow Q_r was altered with values between *1-5* m^3/s to generate a different result. The results of these alterations are seen in figures 54 and 55.

Since the estuary is used for oyster farming it is imperative that there is runoff into the estuary that can balance the salinity. It is therefore reasonable to assume that runoff plays an integral part in controlling the water surface elevation inside the estuary.

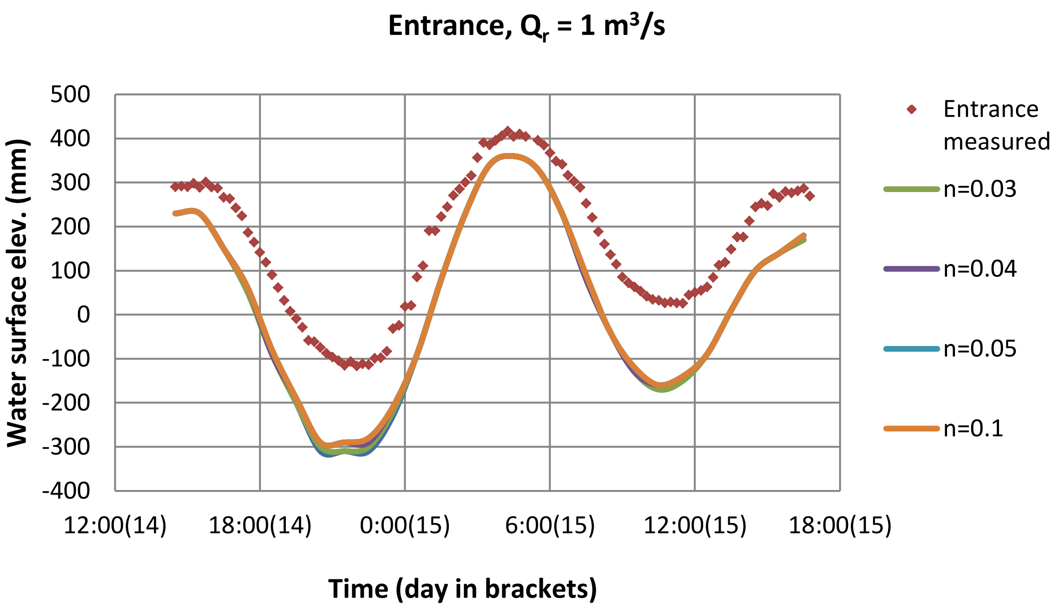

Figure 54 *Sensitivity analysis of how the runoff flow* Q_r *affects the water surface elevations by Entrance Well 2. In this attempt, an upstream boundary flow of 1* m^3/s *was set.*

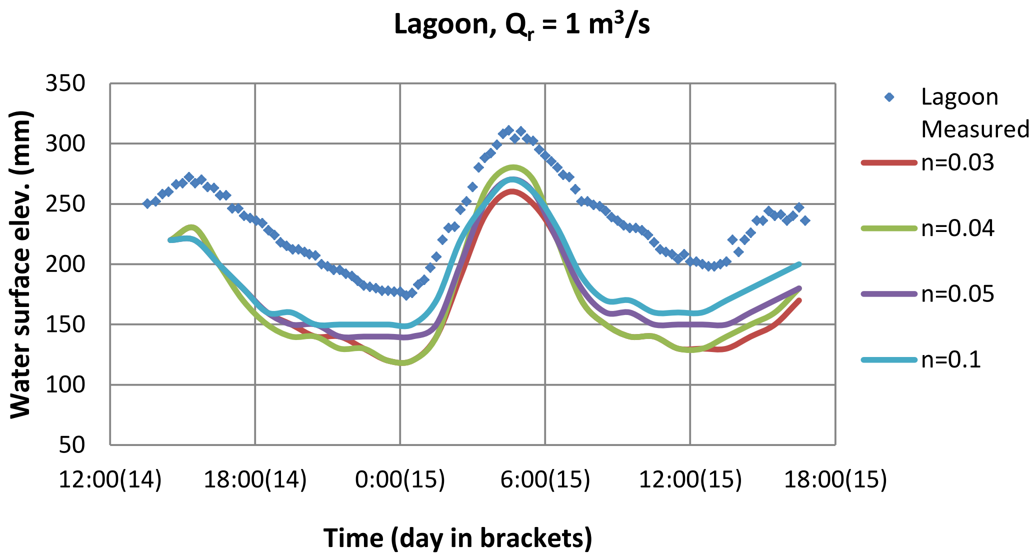

Figure 55 *Sensitivity analysis of how the runoff flow* Q_r *affects the water surface elevations by the Lagoon Well. In this attempt, an upstream boundary flow of 1* m^3/s *was set.*

In figure 55 it appears as if the added flow enabled the simulated water surface elevations by the Lagoon Well to fit the measured values better than without any flow at all. However, the elevations in the entrance (figure 54) appear to have been virtually unchanged by the added flow.

Several attempts with different assumed flows were made, and the flow that provided the best overall fit for different values of *n* was *5* m^3/s, see figures 56 and 57. However, it must be noted that there was a fairly large degree of uncertainty in these results due to several simplifications involving the bathymetry, Manning's *n* values and that the runoff flow Q_r was given as an upstream boundary condition. Other types for flows such as lateral inflow along the river and groundwater interflow were ignored.

It should be noted that the *n* values in figures 56 and 57 still apply for the entire estuary and had not yet been calibrated for any cross sections individually.

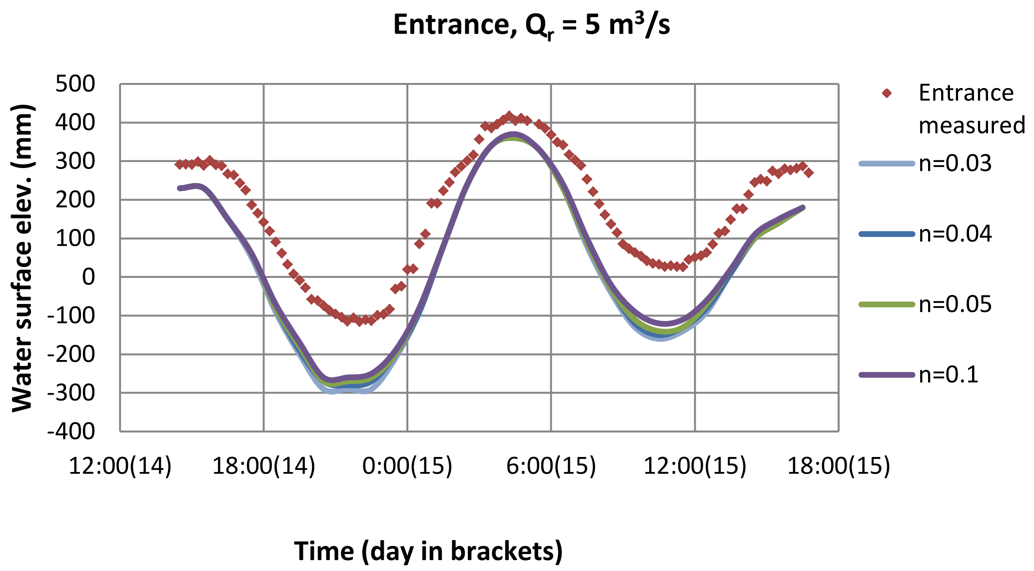

Figure 56 *Sensitivity analysis of how the runoff flow* Q_r *affects the water surface elevations by Entrance Well 2. In this attempt, an upstream boundary flow of 5* m^3/s *was set.*

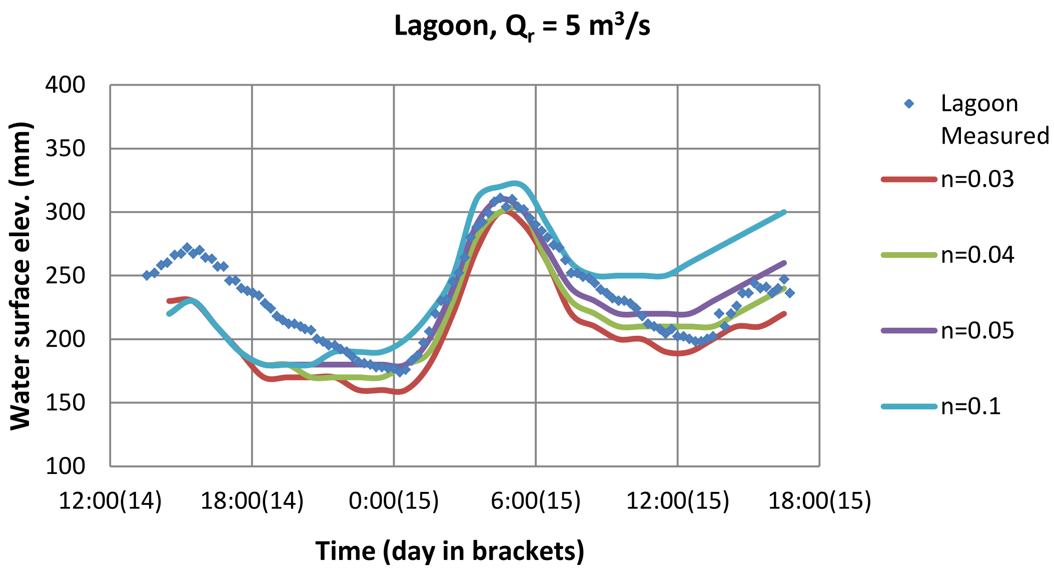

Figure 57 *Sensitivity analysis of how the runoff flow* Q_r *affects the water surface elevations by the Lagoon Well. In this attempt, an upstream boundary flow of 5* m^3/s *was set.*

It was difficult to get a good fit for the first simulated hours, but from around *00:00* on the 15th the curve fitted fairly well for several of the *n* values. However, since the hydraulic properties of the estuary varied from location to location, the *n* values were changed along the first *500 meters* upstream from the offshore given values between *0.03* and *0.06* to account for different types of bathymetry. Narrow sections were given a higher *n* value to account for larger friction.

Since the best overall fit from the estuary in figure 57 was with *n=0.04*. Furthermore, *n=0.04* was chosen for the remaining parts of the estuary as well. The result of the output using the calibrated *n* values is seen in figures 58 and 59.

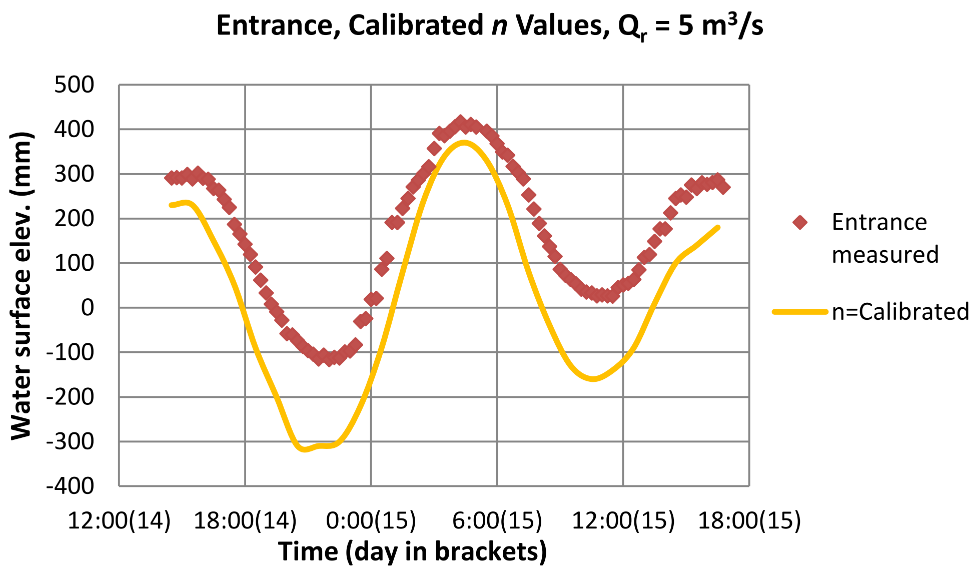

Figure 58 *Sensitivity analysis of how the runoff flow Q_r affects the water surface elevations by the Entrance Well 2. In this attempt, an upstream boundary flow of 5 m^3/s was set and the Manning's* n *was calibrated to fit the measured lagoon water surface elevation.*

As seen in figure 58, it was not possible to get a good fit even though the *n* and Q_r values were calibrated. Since the model kept underestimating the modeled water level in the entrance despite different *n* and Q_r values, that may serve as an indication towards there actually having been wave setup or some other kind of phenomenon in the entrance.

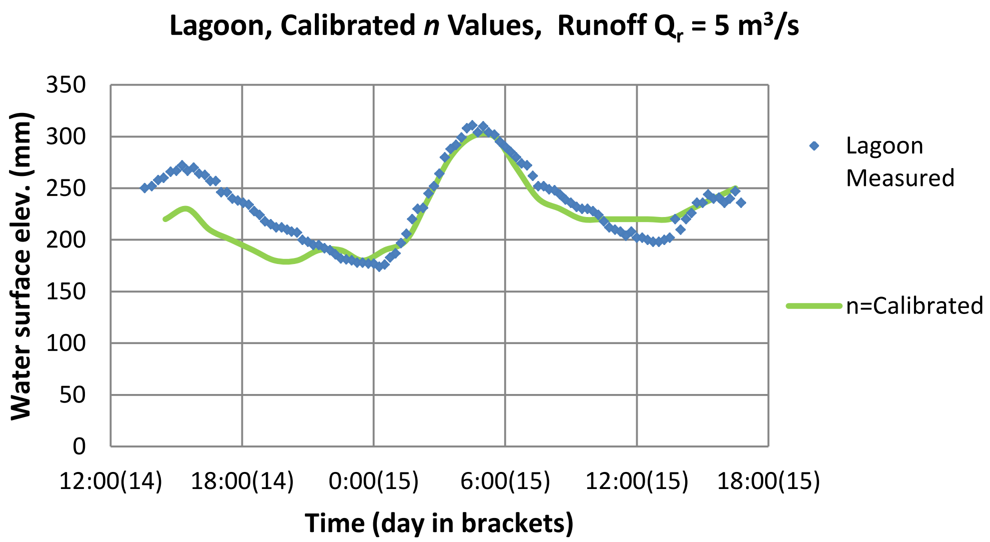

Figure 59 *Sensitivity analysis of how the runoff flow Q_r affects the water surface elevations by the Lagoon Well. In this attempt, an upstream boundary flow of 5 m^3/s was set and the Manning's n was calibrated to fit the measured lagoon water surface elevation.*

However, as seen in figure 59, the upstream boundary flow of $Q_r = 5\ m^3/s$ and calibrated *n* values provide a reasonably good fit for the most part of the measured Lagoon Well water surface elevations.

Measured data from the current monitors (Wonboyn Dataset, 2010) was also plotted against the modeled entrance velocity (figure 60). A simple Bernoulli equation was used to calculate the measured velocity. Since there was no calibration data available for the current monitors the calculated velocity from the current monitor data must be judged critically and may chiefly serve as an indication as to how the model predicted the change of direction for the current.

According to the data, the model under predicts the velocity. There was only data available from the current monitors until about *05:00* on the *15th*, after that they were buried in sand by the large amount sediment that was transported by the current in the entrance.

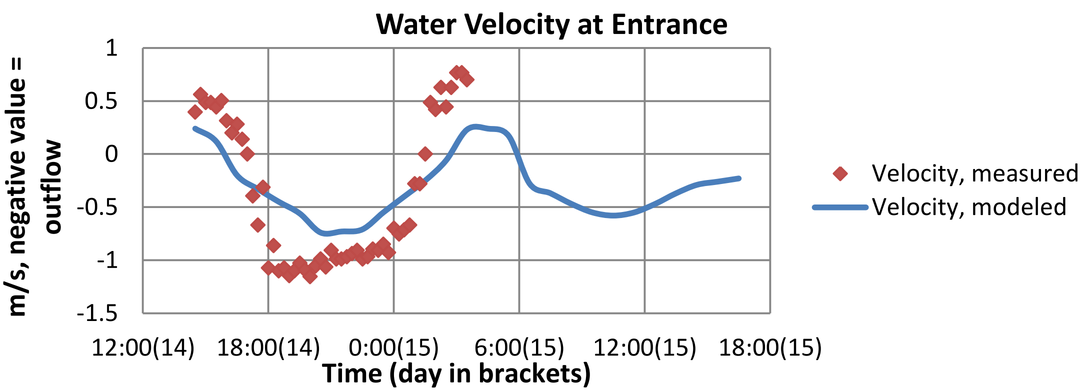

Figure 60 *Measured velocity from the current monitors plotted against the modeled velocity from HEC-RAS.*

Combining the results from all of these HEC-RAS analyses (figures 49-60) it is clear that the model underpredicted the water surface elevations regardless of different Manning *n* values, when there was no runoff flow into the estuary. The model could only partly model the water surface elevation at the Lagoon Well location when there was a more significant flow of $Q_r = 5\ m^3/s$, but the modeled entrance water surface elevation remained almost the same despite changing Q_r and *n*.

If there was wave setup in the entrance, which would have raised the water surface elevation in the entrance and subsequently raised the water surface elevation at the Lagoon Well, the upstream boundary flow Q_r could have been lowered in the model.

To further validate the theory that the runoff flow of $Q_r = 5\ m^3/s$ was too high, a simulation was made using the water surface elevation at Entrance Well 2 as the downstream boundary condition (instead of using Well 9) and driver of the system. The calibrated *n* values were the same as for figures 58 and 59 and flows of $Q_r = 0\ m^3/s$ and $Q_r = 5\ m^3/s$ were modeled. The results of these simulations are found in figures 61 and 62.

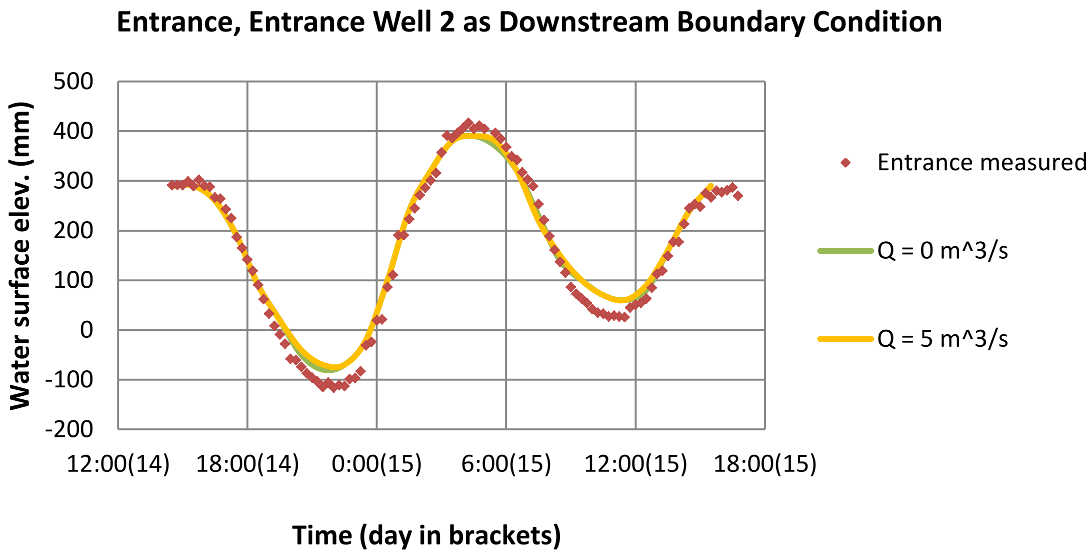

Figure 61 *Model output of the water surface elevation in the entrance when using the water surface elevations at Entrance Well 2 as the downstream boundary condition and two upstream flows of 0 and 5* m^3/s *respectively. The calibrated* n *values are still being used here.*

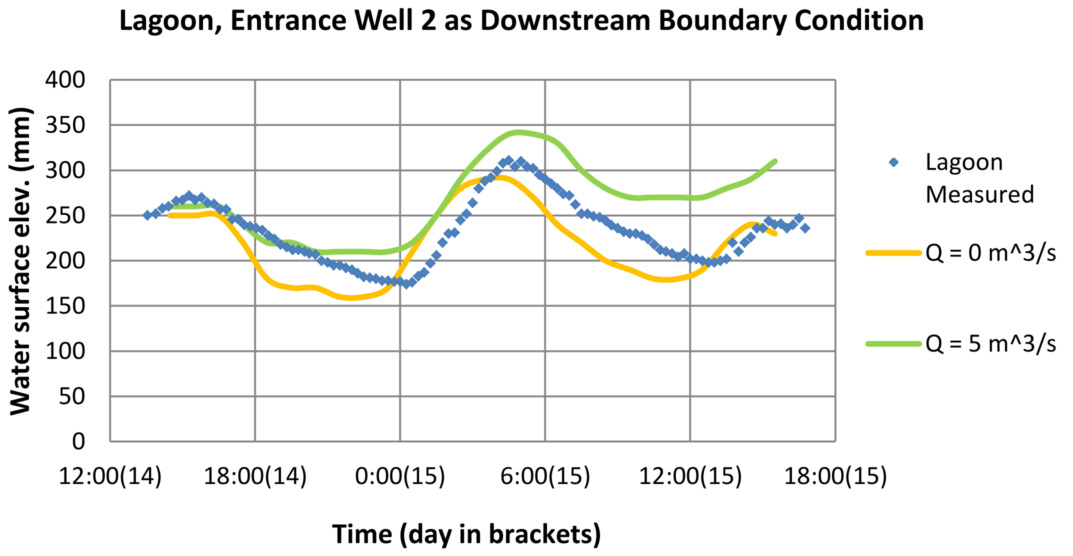

Figure 62 *Model output of the water surface elevation at the Lagoon Well when using the water surface elevations at Entrance Well 2 as the downstream boundary condition and two upstream flows of 0 and 5* m^3/s *respectively. The calibrated* n *values are still being used here.*

That the modeled water surface elevation in the entrance matched the measured water surface elevation is not surprising since the downstream boundary condition was the data from Entrance Well 2 which was located close to the model output point. The elevations by the Lagoon Well fitted fairly well with regard to the surface elevations, but the timing of the curve was off when $Q_r = 0\ m^3/s$.

The timing of the curve was better with $Q_r = 5\ m^3/s$, but it is clear that this flow was too high since the entrance could not let enough water out from the estuary, resulting in a constant rise in the surface elevation at the Lagoon Well. The modeled surface elevation was slightly lower than the measurements, which indicates there was some degree of runoff into the estuary.

The output in figure 62 indicates that there was runoff in the estuary since $Q_r = 0\ m^3/s$ provided water surface elevations that were slightly lower than the measured elevation and that $Q_r = 5\ m^3/s$ was clearly shown to be too large.

The final flow that was tested was $Q_r = 2\ m^3/s$ and the result is seen in figure 62.

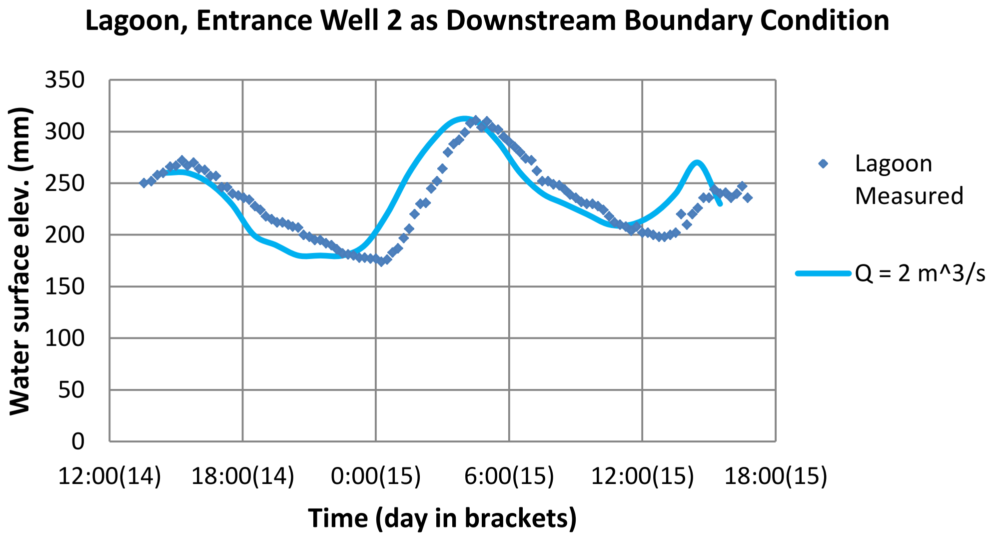

Figure 63 *The output at the approximate location of the Lagoon Well with a runoff flow of 2 m^3/s.*

With $Q_r = 2\ m^3/s$ it appeared as if the elevations correlated better in the beginning and middle, even though there still was a slight delay of the calibrated curve.

6.2 Tallebudgera Creek Estuary

One of the main concerns of the Gold Coast City Council and the major reason to request the field trip to Tallebudgera was the fear of raised water surface elevations in the estuary during heavy rainfall. Since there were many problems during the field trip due to wave conditions and severe beach erosion, it was unfortunately not possible to conduct a similar analysis as on the Wonboyn dataset.

One of the major problems was that the offshore well and pressure transducer was knocked down. There was therefore no offshore water surface elevation measurements to compare with the measurements taken at the more shoreward wells. Without any offshore measurements, it was not possible to establish the amount of wave setup that was present along the open coast. In hindsight, there should also have been at least one more stilling well placed closer to the river entrance. However, because of logistical difficulties, getting there and safely installing a stilling well or pressure transducer, was not possible at the time.

Since there was a high rate of beach erosion, a Total Station survey of the bottom would not have been able to provide reliable data unless bottom profile measurements would have been made frequently. Without a bottom profile it is not possible to use an analytical model such as the one described in 6.1.1.

Had the author been more familiar with computer modelling such as HEC-RAS before the field trip was made, a more comprehensive Total Station survey would have been made of the entrance. However, such a survey would have posed problems since some parts of the entrance were too deep to allow accurate measurements with the prism-pole pole used in taking measurements with the Total Station and other types of equipment should probably have been used, e.g., an echo sounder.

However, even though the data is incomplete, one important observation was made. The three water surface elevation data points in the channel all recorded basically the same elevations throughout the entire survey with hardly any hydraulic gradient, as seen in figure 64. That means that between those data points, water does not have any problem flowing freely in and out of the river system. In other words, if there would be heavy rainfall, the river entrance should let runoff water flow out without water elevations rising significantly.

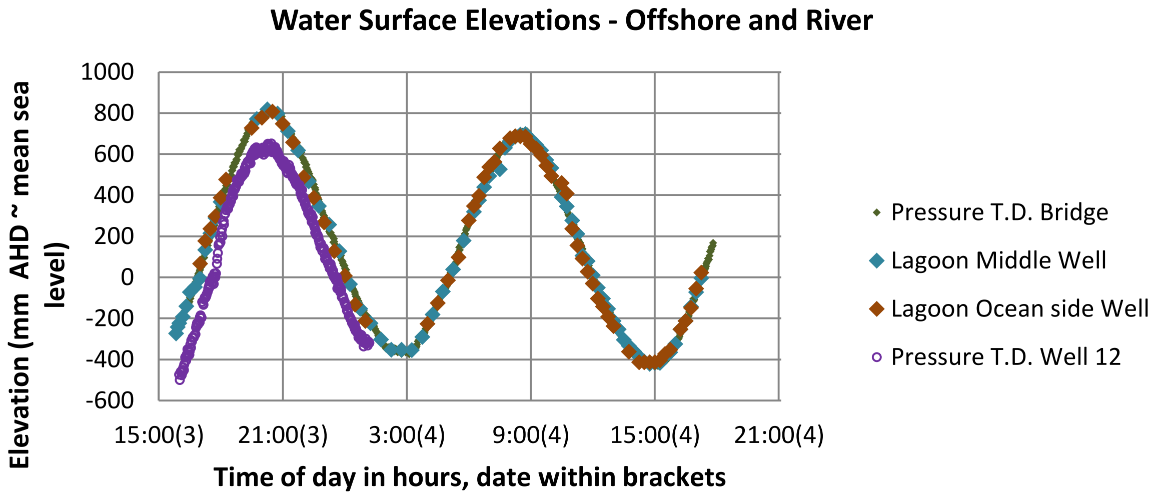

Figure 64 *Graph showing the elevations of the water surface recorded by the two wells and pressure transducer in the river channel.*

However, since the morphology of the entrance changes continuously, as seen in figure 12, water will not always flow as freely as during the time of the field trip. The fact that the river entrance had been dredged a few months earlier, as observed by the author, may have influcenced the lack of a hydraulic gradient and that the dredging improved the flow capabilities of the river entrance. Furthermore, since there was no well closer to the entrance it is not possible to estimate if there was any large hydraulic gradients there.

The data showed that the offshore water surface elevation was lower than the river wells, at least in the beginning of the survey when there was offshore water level data. There does seem to be a similarity to Wonboyn in that the offshore water elevation is lower than the river water surface elevation which may implicate there is a larger hydraulic gradient closer to the river entrance. However, since the offshore data does not cover the entire survey, it is not possible to draw any conclusions regarding those elevations and this survey serves primarily as an example of how difficult it can be to conduct a successful field survey.

7 CONCLUSIONS

7.1 Wonboyn Lake Estuary

The data shows that there was inflow to the estuary even though the offshore water surface elevation at Well 9 was lower than the lagoon and entrance wells at high tide. That suggests there was some process that raised the water level in the entrance and pushed water in. The data also shows that there was a *100-140 mm* difference in water surface elevations between Well 9 and the entrance wells, even though the current monitors showed that the net flow was zero in the entrance. This also supports the previous statement.

The analytical model showed that it generally overestimated the MWL at the open coast, but at the same time underestimated the MWL close to the shoreline. That makes it difficult to discuss whether it accurately can model wave setup in a river entrance since there effectively is no shoreline there. However, using the data that was available and running the model on the river entrance instead of the open coast resulted in a possible maximum setup of approximately *70 mm* at *01:30* when there was no net flow in the entrance. This result should mostly serve as a guideline since there are many uncertain factors and a number of assumptions were made.

The conclusion to the HEC-RAS analysis is that the model underestimates the water surface elevation at the location of the Lagoon Well if Well 9 is used as the downstream boundary condition. The only approach to which it was possible to get the elevations to match was to use a large runoff flow as input. However that runoff was too large when checked with Entrance Well 2 as the downstream boundary condition. It was shown that the water level in the entrance wells are the drivers of the lagoon water level since each time the lagoon and entrance wells went below/above each other the flow in the entrance changed direction.

According to the HEC-RAS analysis, Well 9 could not drive the water surface elevation in the lagoon to match the measured elevations in the model. There is therefore enough reason to conclude that it is *possible* there was wave setup (or some other phenomena, e.g., wave pumping as described by Callaghan et al., 2006) that drove the water surface elevations in the entrance wells to a higher elevation than the offshore water surface elevation. That the raised water level in the entrance, despite of a lower offshore water level, could drive the lagoon water level should be of significant interest for future discussions on the subject.

Furthermore, there was a clear measurable wave setup along the open coast. Since there was an inflow into the entrance even though the water level out towards the sea was lower than in the entrance there must have been wave setup, or some other phenomena, that raised the water surface level close to the entrance and thus enabling water to flow into the lagoon.

7.2 Tallebudgera Creek Estuary

The collected data was fragmented and lacked several vital parts needed for a more comprehensive analysis on the occurrence or non-occurrence of wave setup in the river entrance. The field trip serves as a prime example of how difficult field work can be since researchers always are at the mercy of Mother Nature.

Furthermore, due to the special circumstances that arose on the field trip, the aim with regard to Tallebudgera Creek could unfortunately not be met with certainty. It was only possible to speculate if there was any risk of flooding.

The three wells inside the river/lagoon were likely not enough since they all pointed to there being a very low hydraulic gradient in the river; even though the offshore water level was noticeably lower in the beginning of the survey when there still was data from the pressure transducer at Well 12. However, a low gradient in the river is a good thing since it shows that the water flows freely without hindrance. If the runoff flow gets large, it should be no problem for a larger flow to exit the estuary. However, that is merely speculative. More equipment should have been installed closer to the river entrance, which is something that should be taken into account when planning future surveys.

8 RECOMMENDATIONS

It is recommended that wave setup in river entrances is investigated further. This report indicates towards the possibility of a wave setup in the river entrance at the field site in Wonboyn. However, since previous studies have shown that wave setup may or may not be a factor in driving lagoon water surface elevations, other possible causes resulting in water surface elevation differences in river entrances should be investigated.

It is also recommended that more comprehensive studies than the ones in Wonboyn and Tallebudgera are conducted in the future. Understandably, it is a question of time, money and manpower. However, if more equipment could be deployed in and around the river entrance in particular, and if more focus was given to that particular area, it may be possible to come to a more definitive conclusion whether wave setup exists in some types of river entrances or not.

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