3. SEDIMENT PROCESSES

3.1. Introduction

Water flow has the ability to carry the sediment material and in some cases deposit the sediment materials into an estuary, thus changing its bathymetry. Estuaries can be seen as sediment sinks. They may, with restricted input and output, act as reservoirs which can trap the sediment materials. Runoff from upstream rivers and incoming tidal currents can supply sediment material which may fill the estuary. An estuary has a life span which depends on the hydrodynamic processes within the estuary.

Sediment from the catchment areas may flow out to the sea during periods of heavy rainfall and during the ebb tide.

The cleaning up area in the catchments area has a great impact on the amount of sedimentation pouring to the reservoir. Any constructions which block the mouth of estuary can trap the sediment materials in the estuary.

Sections 3.2 to 3.5 present fundamental information and definitions required to understand and carry out the sediment analysis.

3.2. Sediment Classifications

Basically, there are two categories of sediment materials in the estuary: cohesive material (clay, silt) and non cohesive material (sand, gravel, cobble, and boulder). *Table 3.1* shows the classification of sediment material based on its diameter.

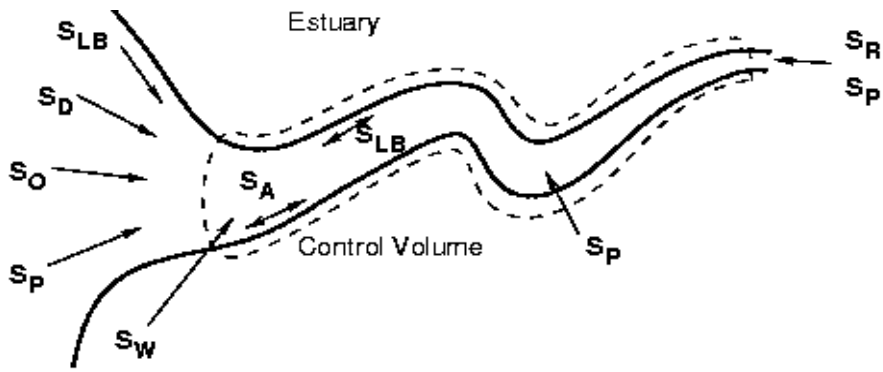
Class name (1)	Size range (mm) (2)
Clay	**d_s < 0.002 to 0.004 mm**
Silt	**0.002 to 0.004 < d_s < 0.06 mm**
Sand	**0.06 < d_s < 2.0 mm**
Gravel	**2.0 < d_s < 64.0 mm**
Cobble	**64.0 < d_s < 256.0 mm**
Boulder	**256.0 < d_s**

Table 3.1: Sediment classification (Chanson 2004)

Van Rijn (1993) defined sediment as a fragmental material, mainly formed by the physical and chemical disintegration of rocks from the earth's crust. They might be transported by gravity, wind or water. Transportation of sediment material by water can be called fluvial sediment transport.

3.3. Sediment Sources

An estuary as a reservoir receives sediment material from different sources. As the materials come from different places, the characteristics of sediment materials also varies. Sources of sediment materials are illustrated in Figure 3.1.



S_{LB}	Littoral drift and/or bank erosion
S_D	Return of dredged spoil
S_O	Entrance marine shoals and tidal delta sands
S_P	Disposal of domestic and industrial effluents and solid wastes
S_W	Wind erosion of coastal dunes and drying inter tidal shoals
S_A	Decomposition and excretions of marine and river plants and animals
S_R	Erosion of upper catchments and transport by rivers and streams

Figure 3.1: Sources of sediments material (McDowell & O'Connor, 1977)

River bank erosion and general catchments runoff produce large quantities of sand, silt and clay. Catchments runoff also delivers organic materials to the estuary. Littoral processes can supply large quantities of sand to an estuary. Wind action on dunes and inter tidal sand banks carries fine sand into an estuary.

Analysis bottom samples taken from Causeway Lake shows that the majority of sediment materials are: entrance marine shoal and tidal delta sands (S_O), decomposition and excretions of marine and river plants and animals (S_A), and erosion of upper catchments and transport by river and stream (S_R). As the Causeway Lake has a causeway and a bridge across the mouth of the estuary, it separates the lake from the dunes area. Thus, wind erosion of coastal dunes (S_W) contributes fewer materials in the lake. The Lake does not have long rivers upstream. There is no evidence of disposal of domestic and industrial effluents and solid wastes (S_P).

3.4. Sediment Transport Modes

Basically, transport of sediment particles is classified in various modes as described in Figure 3.3. Total Sediment Transport is a combination of bed material and wash load transport. Bed material transport is a mixture of bed load and suspended load transport. All developed formulas have been used to predict bed material transport, as they are based on hydraulic principles. It is much more difficult to evaluate formulas to calculate wash load. Some existing formulas are only representative for specific catchment for which they were developed, but cannot be transferred into other catchment conditions. Therefore in this thesis, only bed material transport will be analyzed.

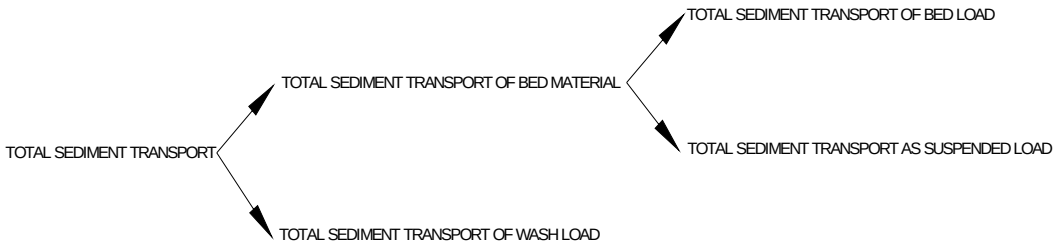


Figure 3.2: Modes of sediment transport (Graf 1998)

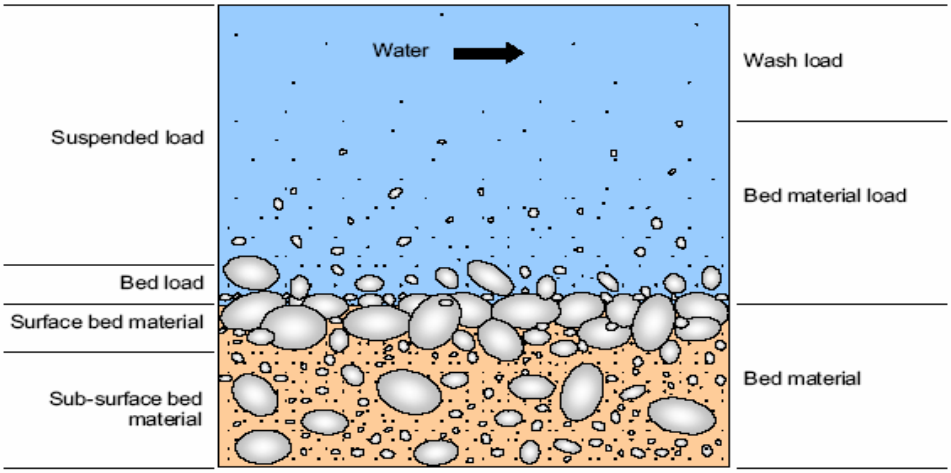


Figure 3.3*: Sediment transport modes (Northwest Hydraulic Consultants 2003)*

3.4.1. Bed Material Load Transport

Bed load transport load occurs when sediment particles are sliding, rolling and undergoing saltation (jumping) along the bed. During these forms of motion, these large particles have almost continuous contact with the bed.

Sediment particles which have less contact with the bed and are originally eroded from the river bed are also known as suspended sediment.

The mixture of bed load and suspended load (but excluding wash load) is known as the bed material load. It is this for which most of the sediment transport formulas have been developed.

3.4.2. Wash Load Transport

Wash load is that fraction of the suspended load which has a small probability of having contact with the bed. Usually wash load consists of small particles such as silt and clay. A rule of thumb is that the grain size separating wash and suspended load is about the 10% finer size given by sieve analysis of bed material. (Raudkivi, 1990)

3.4.3. Total Sediment Transport

Total sediment transport is the combination of all sediment transport modes: bed load and suspended load.

3.5. Sediment Transport Formulas

In agreement with the sediment transport modes, there are three types of sediment formulas: bed load transport, suspended load transport, and total load transport. If the contact between sediment particles and the bed occurs as a result of gravity forces only, it can be categorized as bed load transport. The combination of gravity force and turbulent eddies on sediment particles in the fluid can cause suspended load transport (Bagnold 1966). There are about 20 different theoretical and experimental formulas to evaluate sediment transport. In this thesis, only four have been discussed as they will be implemented in the numerical modelling analysis. *Table 3.2* shows these formulas, together with an indication of the transport mode types.

Formula	Total Mode	Bed Mode	Suspended Mode	Remark
Engelund and Hansen (1967)	**Yes**	**No**	**No**	
Ackers and White (1973)	**Yes**	**No**	**No**	
Brownlie (1981)	**Yes**	**No**	**No**	
Van Rijn (1984)	**Yes**	**Yes**	**Yes**	**Bed load and Suspended load determined separately**

Table 3.2: Overview of selected sediment transport formulas

3.5.1. Engelund and Hansen (1967)

Engelund and Hansen adopted the stream power concept to derive a sediment transport equation. This formula calculates the total sediment load. The equation can be used with moderately sorted bed materials having mean fall diameters larger than 0.15 mm and $\frac{d_{75}}{d_{25}} \leq 1.6$

$$q_t = \frac{0.05 \cdot \overline{u}^5}{(s-1)^2 \cdot \sqrt{g} \cdot d_{50} \cdot C^3} \qquad \text{(Equation 3.1)}$$

$$C = \frac{\overline{u}}{\sqrt{h \cdot I}} \text{ or} \qquad \text{(Equation 3.2)}$$

$$C = 18 \cdot \log\left(\frac{12 \cdot h}{k_s}\right) \qquad \text{(Equation 3.3)}$$

Where:

q_t = Total sediment load [m^3/m-s]

$\overline{u}$ = Mean flow velocity [m/s]

s = Specific gravity of sediment

g = Gravitational acceleration [m/s^2]

d_x = Particle diameter [m]

C = Chezy-coefficient [$m^{0.5}$/s]

h = Depth of flow [m]

I = Energy of slopes [-]

k_s = Effective river bed roughness

The formula is very popular and implemented around the world, even though it was developed in laboratory conditions.

3.5.2. Ackers and White (1973)

Ackers and White formula was developed based on about 1000 sets of laboratory flume studies and field data. This formula is used to calculate the total sediment load. They developed the formula as a function of several empirical coefficients and a dimensionless mobility parameter. The formula can be used for grain size greater than 0.04 mm and a dimensionless particle diameter $D_* > 1$.

$$q_t = K_a \cdot \bar{u} \cdot d_{35} \cdot \rho_s \cdot \left(\frac{\bar{u}}{u_*}\right)^l \cdot \left(\frac{Y - Y_{cr}}{Y_{cr}}\right)^m \qquad \text{(Equation 3.4)}$$

$$Y = \frac{u_*^{\,l}}{\sqrt{g \cdot (s-1) \cdot d_{35}}} \cdot \left(\frac{\bar{u}}{5.66 \cdot \log\left(10 \cdot \frac{10 \cdot h}{d_{35}}\right)}\right)^{1-l} \qquad \text{(Equation 3.5)}$$

Ackers and White coefficients are described in *Table 3.3*:

Coefficients	**$1 < D_* < 60$**	**$D_* \geq 60$**
l	$1 - 0.56 \cdot \log(D_*)$	0
m	$\frac{6.83}{D_*} + 1.67$	1.78
K_a	$10^{-3.46 + 2.79 \cdot \log(D_*) - 0.98 \cdot (\log D_*)^2}$	0.025
Y_{cr}	$\frac{0.23}{\sqrt{D_*}} + 0.14$	0.17
D_*	$d_{35} \cdot \left(\frac{(s-1) \cdot g}{v^2}\right)^{\frac{1}{3}}$	
v	$\left[1.14 - 0.0031 \cdot (T_e - 15) + 0.00068 \cdot (Te - 15)^2\right] \cdot 10^{-6}$	

Table 3.3: Coefficients for Ackers & White formula

Where:

q_t = Total sediment load [m^3/m-s]

K_a = A coefficient of Ackers and White

$\bar{u}$ = Mean flow velocity [m/s]

d_x = Particle diameter [m]

ρ_s = Density of sediment [kg/m^3]

u_* = Current related bed shear velocity [m/s]

l = A coefficient of Ackers and White

Y = Particle mobility parameter of Ackers and White

Y_{cr} = Critical particle mobility parameter of Ackers and White

D_* = Dimensionless particle dimeter

s = Specific gravity of sediment

g = Gravitational acceleration [m/s^2]

Te = Temperature (ºC)

3.5.3. Brownlie (1981)

Brownlie used a large number of observed values for the variables (laboratory and field works) the functional relation between the dimensionless variable can be fitted. Combined dataset of more than 7000 points has been used for his numerical analysis. Calculation of sediment concentration is given by:

$$c = 7115 \cdot cf \cdot \left(F_g - F_{go}\right)^{1.978} \cdot S^{0.6601} \cdot \left(\frac{R_H}{D_{50}}\right)^{--0.3301} \qquad \text{(Equation 3.6)}$$

$$F_g = \frac{\bar{u}}{\sqrt{\left(\frac{\rho_s - \rho}{\rho}\right) \cdot g \cdot D_{50}}} \qquad \text{(Equation 3.7)}$$

$$F_{go} = 4.596 (\tau_c^*)^{0.5293} S_o^{-0.1045} \sigma_g^{-0.1606} \qquad \text{(Equation 3.8)}$$

$$R_g = \frac{D_{50} \cdot \sqrt{g \cdot R_h \cdot S_o}}{\nu} \qquad \text{(Equation 3.9)}$$

$$\tau_c^* = 0.22\,Y + 0.06 \cdot 10^{(-7.7\,Y)} \qquad \text{(Equation 3.10)}$$

$$Y = \left(\sqrt{\frac{\rho_s - \rho_w}{\rho_w} \cdot R_g} \right)^{-0.6} \qquad \text{(Equation 3.11)}$$

Where:

c = Mass concentration of sediment in part per million (ppm)
1000 ppm equal to 1 kg/m^3

cf = Parameter, 1 for laboratory conditions and 1.268 for field conditions

S_o = Slope

F_g = Brownlie's parameter

R_g = Brownlie's parameter

F_{go} = The grain Froude number in lower regime flow (flow Froude number < 1) at slope S which is at critical shear

R_h = Hydraulic radius

$\bar{u}$ = Velocity at cross section

g = Gravity

ν = Fluid viscosity

σ_g = Sediment standard deviation

D_{50} = Median grain size

ρ = Fluid density

ρ_s = Sediment density

Range of data used in analysis by Brownlie is shown in *Table 3.4*.

Variable	Minimum	Maximum
Median particle size, D_{50} (mm)	0.088	2.8
Unit discharge, q(m^3/s/m) [Discharge Q(m^3/s)]	0.012 [0.0032]	40 [22,000]
Slope, S	3.0×10^{-6}	3.7×10^{-2}
Hydraulic radius, r (m)	0.025	17
Temperature, T (°C)	0	63
Also:		
Width-to-depth ratio, w/d	Greater than or equal to 4	
Geometric standard deviation of particle sizes, σ_g	Less than or equal to 5	

Table 3.4: Range of data used in analysis (Brownlie 1981)

3.5.4. Van Rijn (1984)

Van Rijn (1984) developed analytical models for bed load and suspended sediment transport and claimed that about 77% of tested field and laboratory cases are within 0.5 to 2 times compared with his model. A simplified formula when only the mean velocities, flow depth and particle size are known will be adapted to the Causeway Lake case where no other available historical data exists.

Van Rijn introduced an equation describing the mean critical velocity U_c, that initiates the sediment transport (Van Rijn, 1984a):

$$U_c = 0.19 D_{50}^{0.1} Log\left[\frac{12R}{3D_{90}}\right] \qquad 0.1 <= D_{50} <= 0.5 \text{ mm} \qquad \text{(Equation 3.12)}$$

Next he developed two formulas:

For Bed load transport

$$\frac{q_b}{Uy_0} = 0.005\left\{\frac{U - U_c}{[(S_s - 1)gD_{50}]^{1/2}}\right\}^{2.4}\left\{\frac{D_{50}}{y_0}\right\}^{1.2} \qquad \text{(Equation 3.13)}$$

And for Suspended sediment transport

$$\frac{q_s}{Uy_0} = 0.012\left\{\frac{U - U_c}{[(S_s - 1)gD_{50}]^{1/2}}\right\}^{2.4}\left\{\frac{D_{50}}{y_0}\right\} D_*^{-0.6} \qquad \text{(Equation 3.14)}$$

Where

q_b = Bed load transport rate; volume per unit width

q_s = Suspended sediment transport rate, volume per unit width

U = Flow velocity

y_0 = Flow depth

S_s = Specific gravity of bottom material

g = Gravitational acceleration

D_{50}, D_{90} = Characteristic particle sizes

D_* = $[(S_s - 1)g/\nu^2]^{1/3} D_{50}$ - Dimensionless grain size

Van Rijn recommended the simplified correlations for data covering y_0 = 1 to 20 m, U = 0.5 to 2.5 m/s and D_{50} =0.1 to 2 mm.

3.5.5. Sediment Transport Formula Summary

Browlie (1981) compared statistically 13 sediment transport formulas by using 1000 records from 31 sets of laboratory and field data, and compared their accuracy separately for field and laboratory data. *Table 3.5* presents the geometric mean and standard deviation of observed sediment concentration for both laboratory and field observations. A graphical display of the statistics is presented in Figure 3.4.

Investigator	Number	Laboratory Mean	Laboratory S.D.	Field Mean	Field S.D.
Ackers and White (1973)	998	1.150	1.758	0.694	2.027
Bagnold (1966)	999	2.155	2.718	1.173	2.537
Bishop et al. (1965)	973	0.695	2.300	0.443	2.488
Einstein (1950)	950	0.628	4.059	0.420	3.719
Engelund and Fredsoe (1976)	825	1.274	2.972	3.179	14.026
Engelund and Hansen (1967)	999	1.236	2.064	0.916	1.997
Graf (1971)	999	1.360	3.696	1.005	3.124
Laursen (1958)	972	1.296	2.532	0.420	3.098
Ranga Raju et al. (1981)	833	1.160	1.882	0.333	2.813
Rottner (1959)	999	0.920	2.101	0.603	1.904
Shen and Hung (1971)	940	0.866	1.656	0.432	2.973
Toffaleti (1968)	995	1.166	2.749	0.854	2.572
Yang (1973)	993	1.215	1.710	0.471	3.077
Brownlie (1981)	999	1.000	1.638	1.000	1.746

Table 3.5: Geometric Mean and Geometric Standard Deviation of selected sediment formulas (Brownlie 1981)

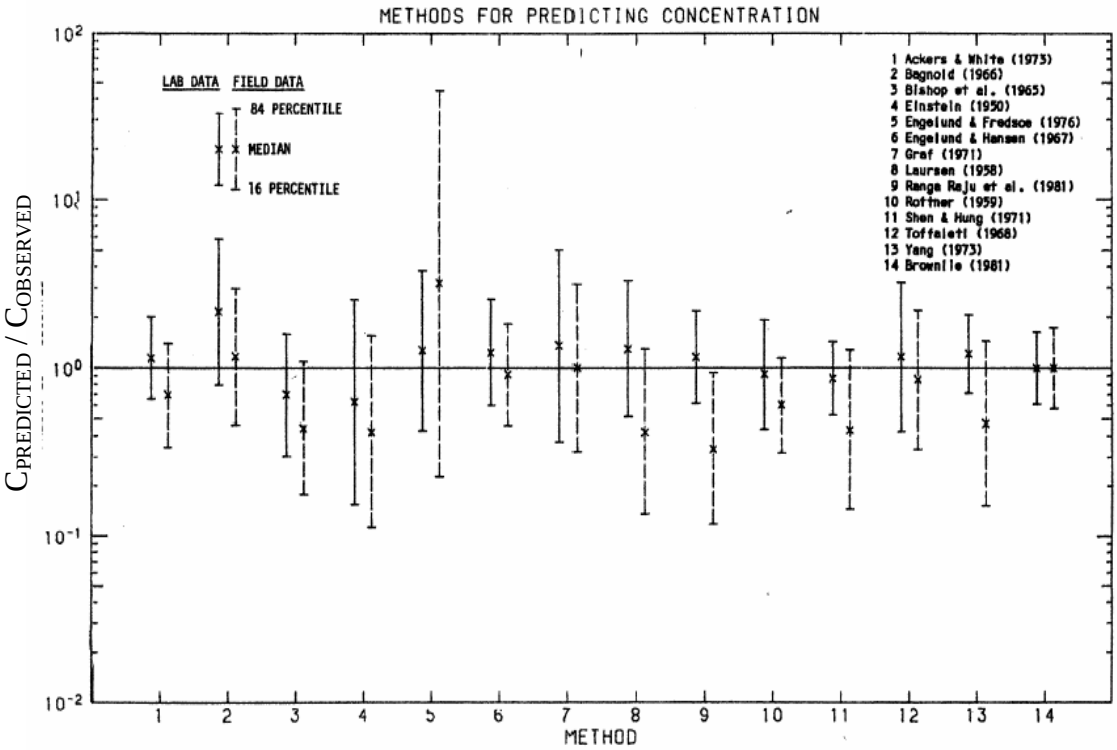


Figure 3.4: Comparison of methods for predicting sediment concentration (Brownlie 1981)

Figure 3.4 shows that Browlie's formula (No. 14) has the lowest standard deviation for both laboratory (1.638) and field data (1.746). Engelund and Hansen formula (No. 6) also has low standard deviations both of 2.064 and 1.997 for laboratory and field data respectively. Ackers and White is next to be considered as a reliable formula with standard deviations of 1.758 and 2.027 for laboratory and field work respectively. At that time (1981), Van Rijn's formula had not been proposed yet. Thus Van Rijn formula was not listed. Van Rijn's formula was released in 1984.

3.6. Sedimentation in Causeway Lake

Causeway Lake acts as a sink. It receives sediments from the sea side during the flood tide and from the catchment areas during periods of heavy rainfall.

During the high tides which exceed the bridge sill level of 1.4 m AHD, sea water flows into the Lake. It brings suspended sediment in during the flood tides. During the slack water in which water velocity is close to zero, the sediment will have a chance to settle in the lake. Then the ebb tide takes part of the suspended sediments out to the sea. The sill level is also obstructing sediment movement to the sea. Thus a considerable amount of sediment will settle in the area of the estuary most significantly affected by the tidal sediment inflow.

The amount of sediment inflow from the catchments depends on the rainfall intensity. During persistent heavy rainfall, discharge in the creeks carries sediment from catchment areas to the lake. Thus the rainfall data needs to be analysed to evaluate creek discharges and sediment transport evaluated based on empirical

equations discussed in the section. Human activities will also affect the amount of sediment. Clearing of the area and logging trees will directly affect the sediment rate. When the vegetation has been removed, sediment from ground surface (wash load) will easily erode away during the rainfall, and then will drain to the waterways. The initial qualitative impression about Causeway Lake sedimentation was based on analysis of available aerial photographs. Quantitative evaluation was based on measurement and experimental equations.

3.7. Qualitative Evaluation of Sediment Based on Aerial Photographs

Department of Natural Resources has been taking aerial photographs of the Capricorn Coast at irregular intervals for several years. In this report a few aerial photographs have been included to present visual changes to the lake layout for the period of analysis. They are selected to represent the nearest time to when bathymetry was measured. The bathymetry will be discussed in Section 3.9.1. The photos are comparable because, in each case, the elevation of the lake was around the fixed sill level under the bridge. Three parts of the lake can be distinguished:

1. Central part between the causeway and peninsula
2. Shoal Creek branch left from the peninsula (southern side)
3. Mulambin Creek branch right from the peninsula (northern side)

Figure 3.5 shows a photo from 22 August 1961. A sandbar is clearly visible in the central part of the lake. It is highly probably that this sandbar has been formed with sand transported under the bridge from the sea side by the tidal action.

Figure 3.5: Causeway Lake, aerial photograph taken 22 August 1961. (Courtesy of the Department of Natural Resources and Mines.)

Figure 3.6 shows an aerial photo taken 28 May 1988. The tongue of sand in the central part of the lake, visible in the 1960s, becomes a wide underwater sand bar and siltation is visible along almost the whole bank of the lake. Mulambin Creek branch looks more silted than the other branch. The existence of seagrass is also visible.

Figure 3.6: Causeway Lake, aerial photograph taken 28 May 1988. (Courtesy of the Department of Natural Resources and Mines.)

Figure 3.7 shows the lake conditions in May 2001. There is progressing siltation in the central part of the lake. Accumulation of sediment is also visible at the southern end of the causeway. Sea grass has been covering a significant part of the Mulambin Creek branch. Seagrass is also growing in the central part and in the Shoal Creek branch. The banks of the Shoal Creek area are more silted as well.

Figure 3.7: Causeway Lake, aerial photograph taken 30 May 2001. (Courtesy of the Department of Natural Resources and Mines.)

Finally, Figure 3.8 is the most recent available aerial photo of the lake taken on 29 June 2004. There is continuous progressing in siltation in the central part of the lake and along the banks. Sea grass areas are larger as well.

Figure 3.8: Causeway Lake, aerial photograph taken 29 June 2004. (Courtesy of the Department of Natural Resources and Mines.)

Based on visual analysis of the series of aerial photographs taken between 1961 and 2004, the sedimentation is progressing during these years. Judging by brownish colour of sediment material, the majority of sediment material in the centre of the lake up to July 2004 could be from the sea.

3.8. Methodology

3.8.1. Sediment transported by tide

Suspended sediment is transported by flowing water. During the period of flood tides, suspended sediment enters the lake from the sea side. Flowing suspended sediment can be observed from the brownish color in the water. A point sampler (Figure 3.9) was used to collect suspended sediment samples at the time when velocities were measured.

Figure 3.9: Sediment Sampler

The sampler is made from a streamlined bronze casting (=10 kg), which has an intake nose hole. Collected samples have certain concentrations of sediment expressed in mg/L. In laboratory conditions, the concentration of sediment is measured using filtration method. Filtration can be carried out by suction. Figure 3.10 shows the filtration system.

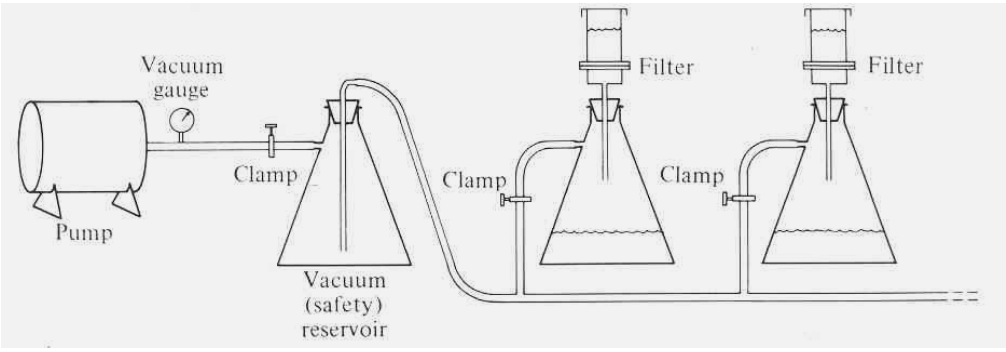


Figure 3.10: Filtration system using simple suction (Dyer 1979)

The concentration can be determined as:

$$C = \frac{G_d}{V} \qquad \text{(Equation 3.15)}$$

In which G_d is the dry mass of sediment and V is the water volume of each sample. Analyses of sediment concentration and discharges as a function of time allow evaluation of the volume of sediment transported into and out of the lake during a tide period.

$$S_r = Q \cdot C \qquad \text{(Equation 3.16)}$$

Where:

S_r = Suspended Sediment rate

Q = Water discharge

C = Concentration

3.8.2. Sediment transported from the catchments

Estimating the sediment rate from the creek side was based on the same principles as for determining sediment rate from the sea side. Collected water inflow samples were analysed in the laboratory to obtain C. To evaluate probable sediment transport into the estuary for extreme conditions, creek discharges must be evaluated and then experimental formulas used to estimate the amount of sediment. For analytical analysis, a wide range of flow rates and corresponding sedimentation inflow from creek discharges along with bottom material characteristics must be evaluated.

Cross sections of both creeks must be selected and measured. The following data needs to be obtained to evaluate discharges: cross sectional area, surface width, hydraulic mean depth, Manning coefficient, catchment areas, length of creeks, D_{50}, D_{90}, bottom material relative density and kinematic viscosity. The sediment transport will be initiated when velocities in the creek exceed the critical velocity (Equation

3.12) suggested by Van Rijn (1984a). To evaluate bed and suspended sediment transport separately, Van Rijn formulae were implemented (Equation 3.13 and 3.14).

3.9. Field Data and Analysis

3.9.1. Bathymetry of the lake

The oldest available surveying found during this study was a survey of the lake bottom in 1960 undertaken by Department of Harbours and Marine, Queensland which covered only 13 ha (Figure 3.11). The particularly valuable survey of the whole lake was taken by The Main Roads Department in 1986. It covered the whole 86 ha area of the lake (Figure 3.12).

A recent bathymetry survey of the Causeway Lake was conducted by the author on 24th October 2003. A Global Positioning System was used to record the position of survey points. 220 survey points have been collected covering a 66 ha area of the lake. These points were transferred into digital form using CivilCAD 6.3. The location of survey points are shown in Figure 3.13.

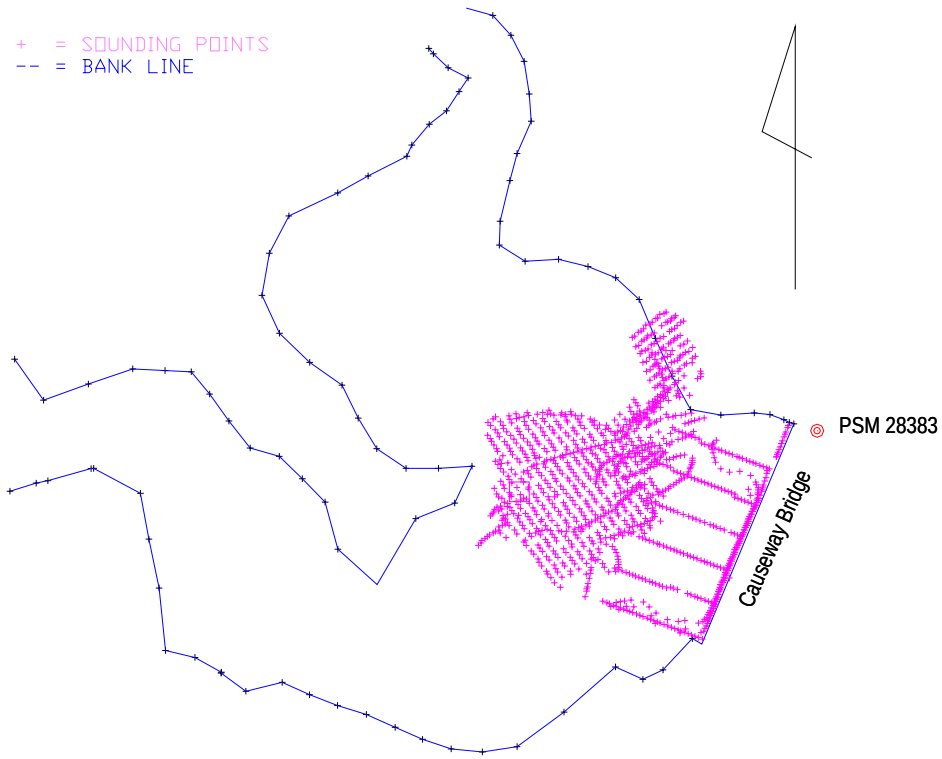


Figure 3.11: Causeway Lake bottom survey area in 1960.

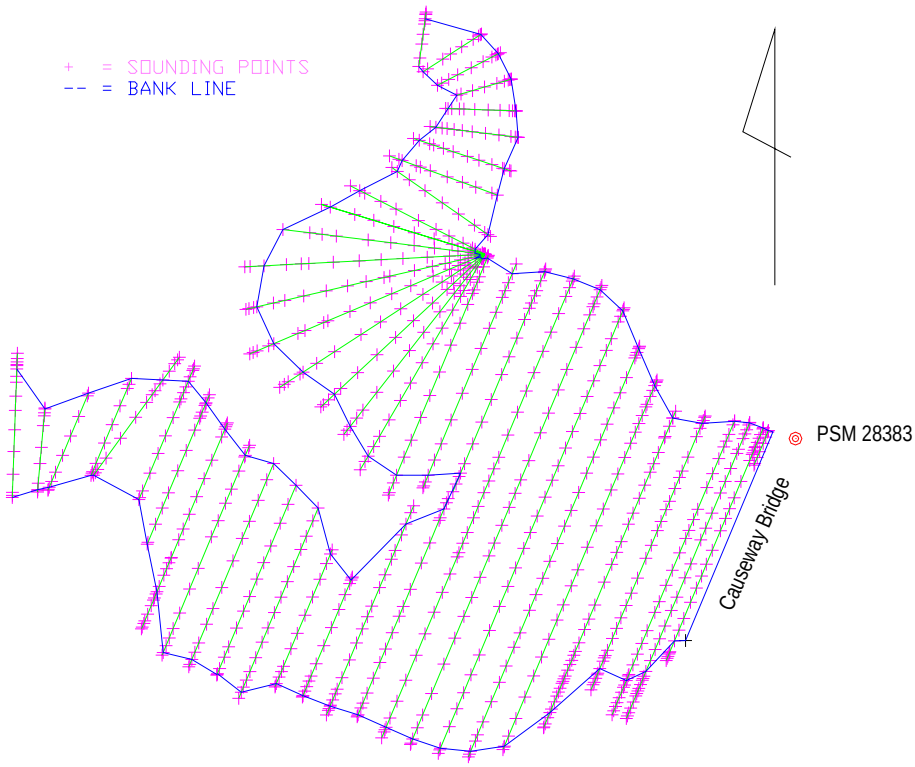


Figure 3.12: Causeway Lake bottom survey area in 1986.

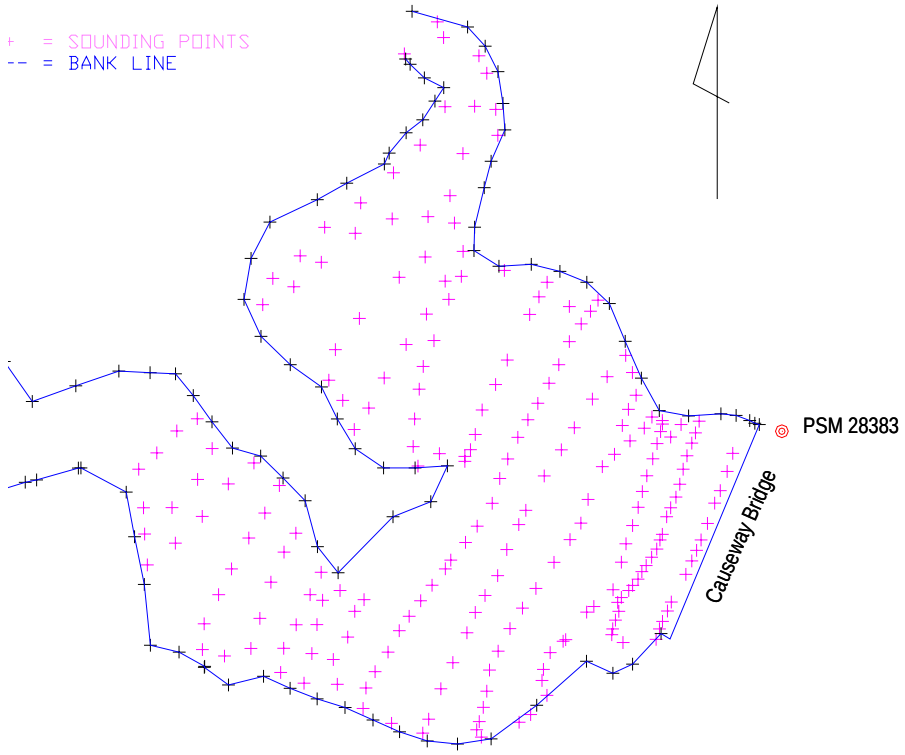


Figure 3.13: Causeway Lake bottom survey area in 2003.

Based on the surveying data, contour lines were generated automatically by CivilCad software. Figure 3.14 shows 1960 contour plan. Figure 3.15 shows 1986 contour plan with 0.5m intervals. Figure 3.16 shows 2003 contour lines with the same intervals. Contour lines are shown in relation to sill datum. At this datum the sill level under the bridge has the elevation of 0.0m. The sill level represents the surface level of the lake.

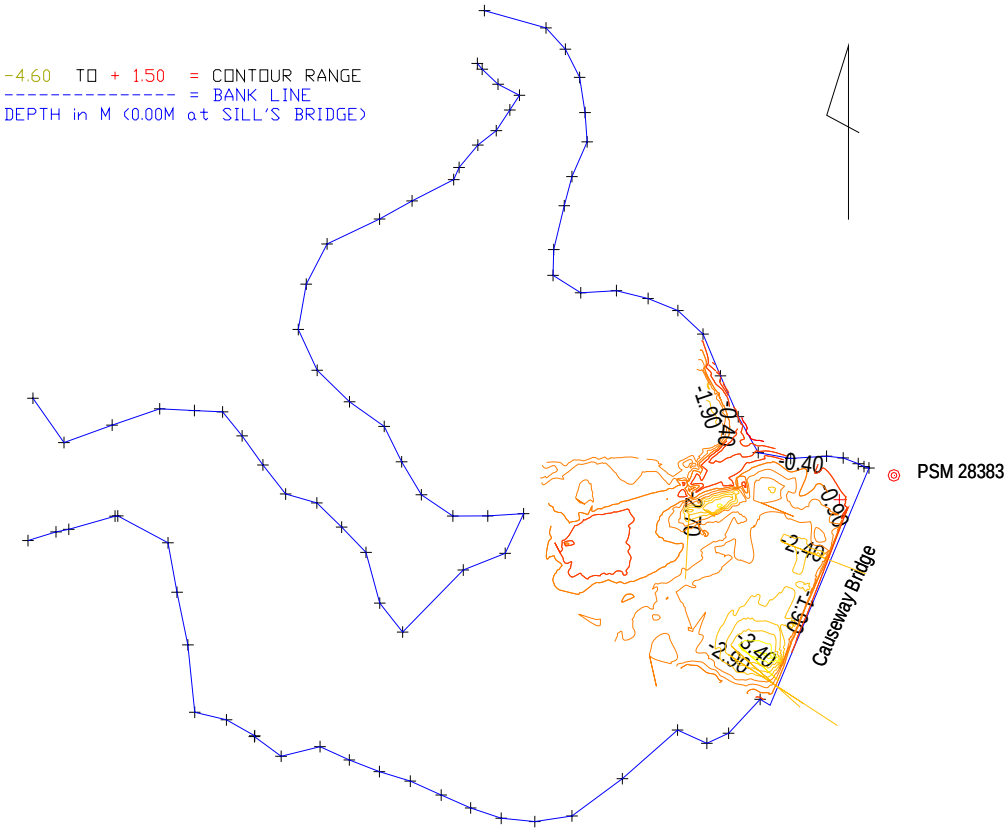


Figure 3.14: Causeway Lake, bathymetric plan 1960 (Depth related to the sill level under the bridge)

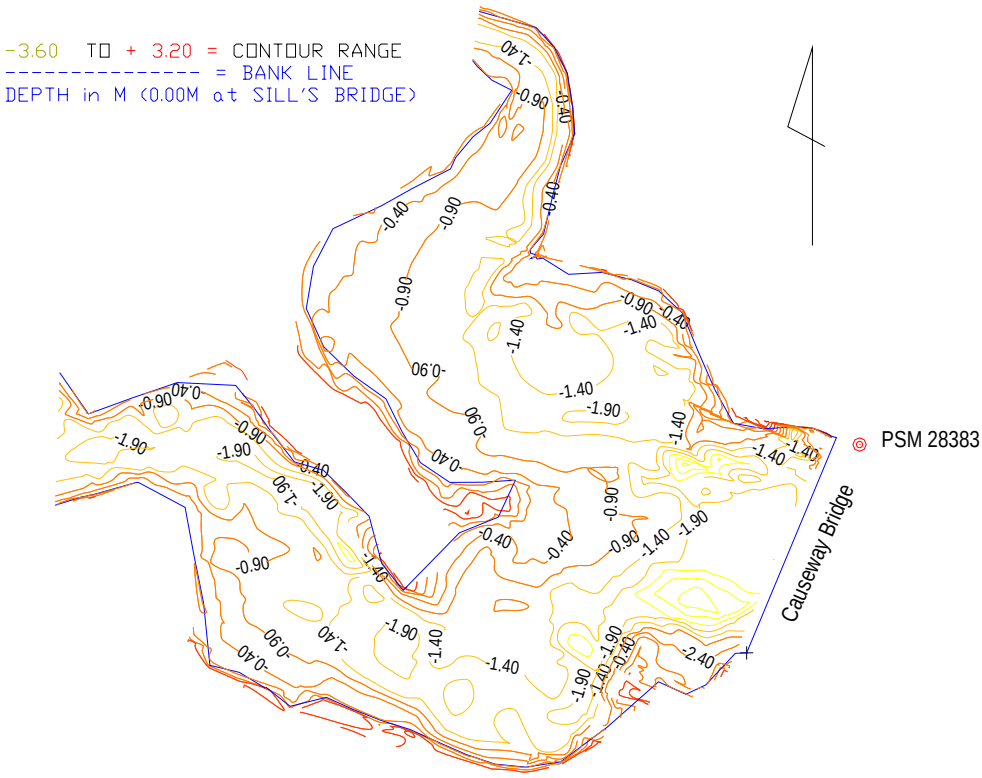


Figure 3.15: Causeway Lake, bathymetric plan 1986 (Depth related to the sill level under the bridge)

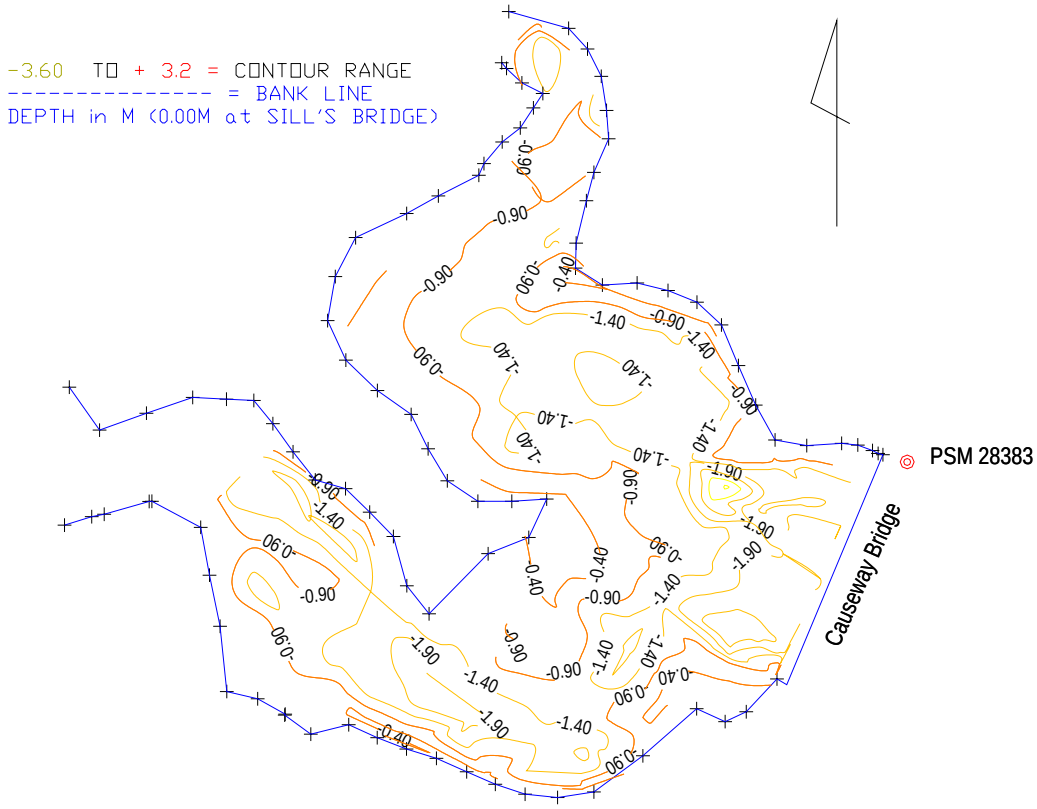


Figure 3.16: Causeway Lake, bathymetric plan 2003 (Depth related to the sill level under the bridge)

CivilCad was used to evaluate quantitative changes of the sedimentation in the lake. Based on surveying data from 1986 and 2003 on a 66 ha surveyed area, the total volume of sedimentation settled in the lake for this period was 42800 m^3. Thus the annual average sedimentation of the lake for the last 17 years is about 2500 m^3/y. Assuming a possible measured error of the depth ±0.01m on the 66 ha area, it gives the volume of sedimentation of 42800 ± 6600 m^3 in total or 2500 ± 400 m^3/y.

The central part of the lake with the total area of 28 ha (Figure 3.17), where the sand bar is predominantly visible, has a total sedimentation of 32500 m^3 for the period 1986 to 2003. It represents an average annual sedimentation of about 1900 ± 200 m^3/y. Thus the rest of the lake accumulated about 600 ± 200m^3/y transported from the catchment areas by creeks during rainy periods.

The deepest part of the lake was along the main channel of the original estuary, located on the south side of the causeway. Examination of the available bathymetric data showed the change of the maximum depth from 4.6m in 1960, to 3.6m in 1986 and only 2.9m in 2003.

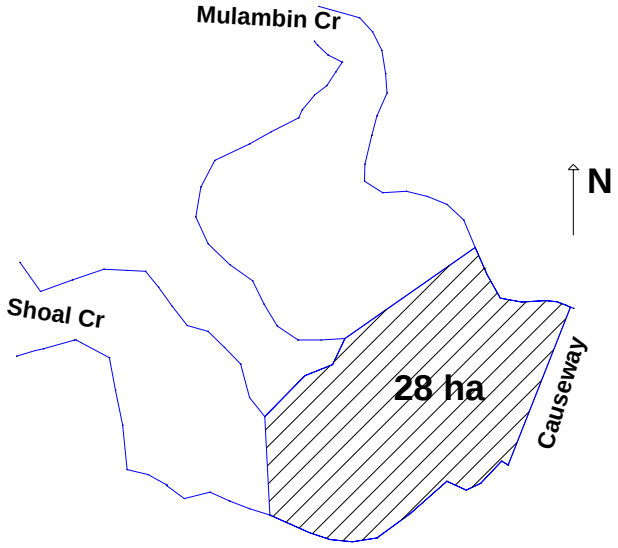


Figure 3.17: Approximate area of sedimentation caused by tidal actions

Based on the bathymetric plans, 15 cross section profiles were created as shown in Figure 3.18.

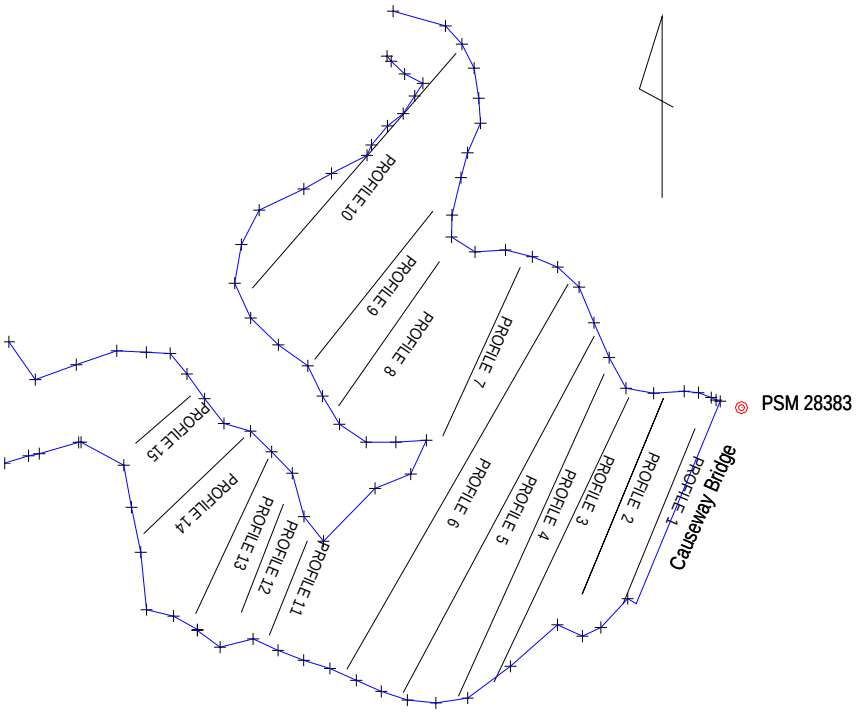
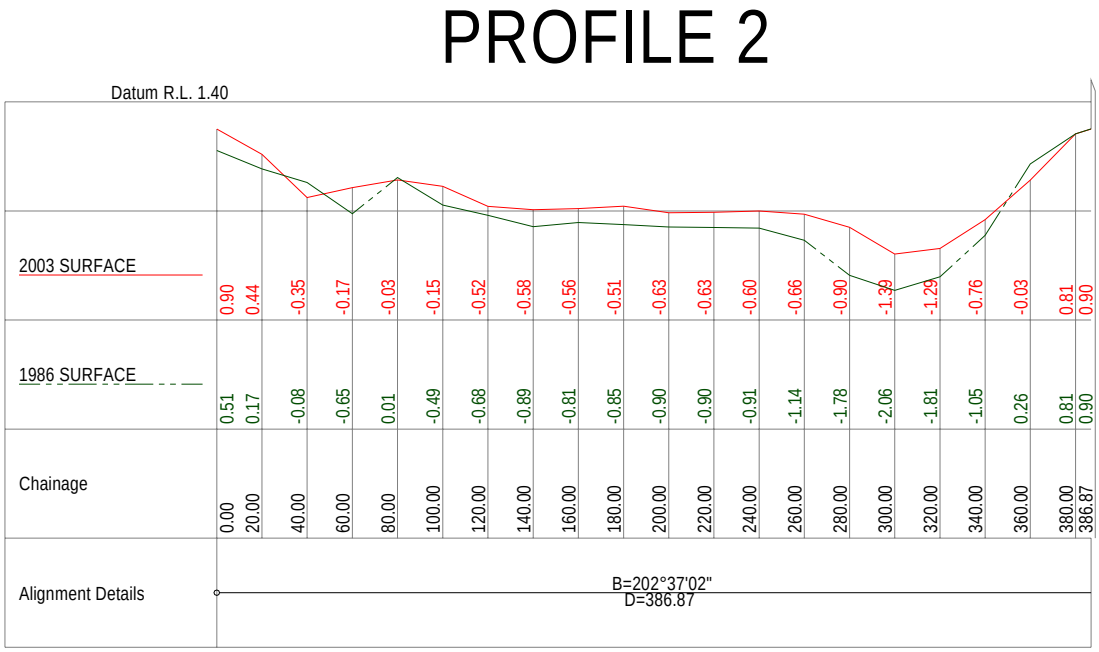


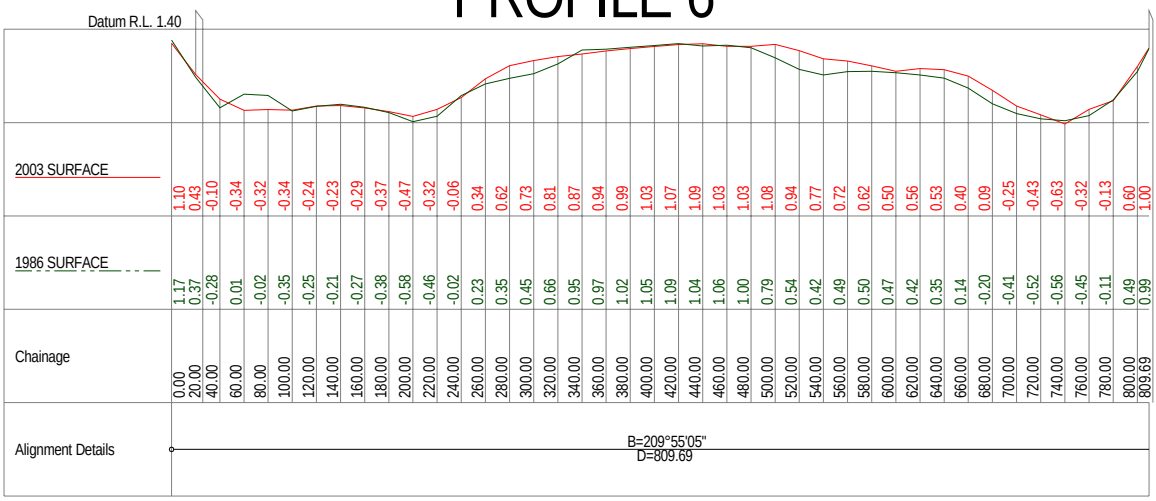
Figure 3.18: Location of cross-section profiles

Characteristic profiles 2, 6, 10, and 15 are shown in Figure 3.19. The elevation for cross sections is based on Australian Height Datum.

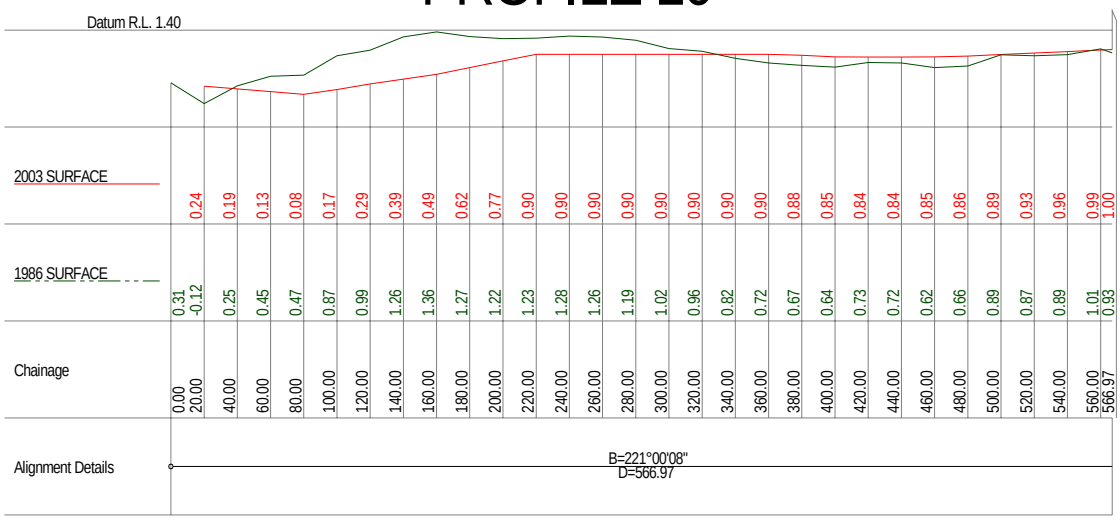
Profile 2 is the cross section close to the mouth of the estuary, close to the bridge sill. The deepest point at chainage 300m has become shallower by about 0.67m during the last 17 years (Figure 3.19). At other chainages on the same cross sections, there is an increase in sediment depth of about 0.35m. Profile 6 is the cross section along the sand bar. During the last 17 years this has also become shallower by about 0.30m. Profile 10 is the cross section along the Mulambin Creek, close to the Bank Line. It has become significantly shallower by about 0.80m during the last 17 years. Cross section at Profile 15 at Shoal Creek similarly becomes shallower by about 0.07m.



PROFILE 6



PROFILE 10



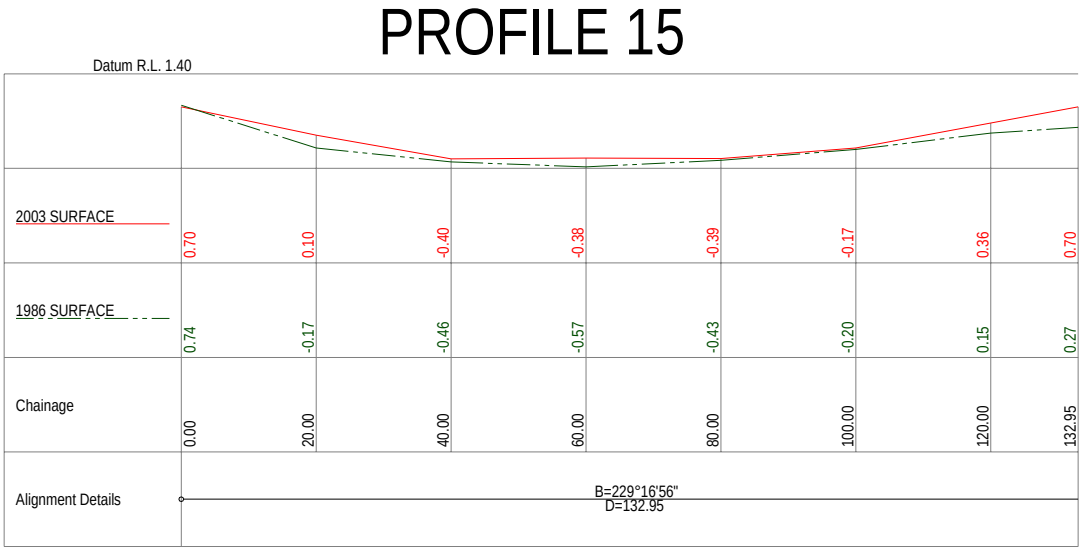


Figure 3.19: Selected profiles across the Causeway Lake. Chainage 0.00 is on the left bank of the lake facing the sea

3.9.2. Characteristics of bottom material

Samples of the bottom material were collected using a grab sampler. A total of 37 samples were collected in November 2003. The locations of sampling, shown in Figure 3.20, were noted using GPS.

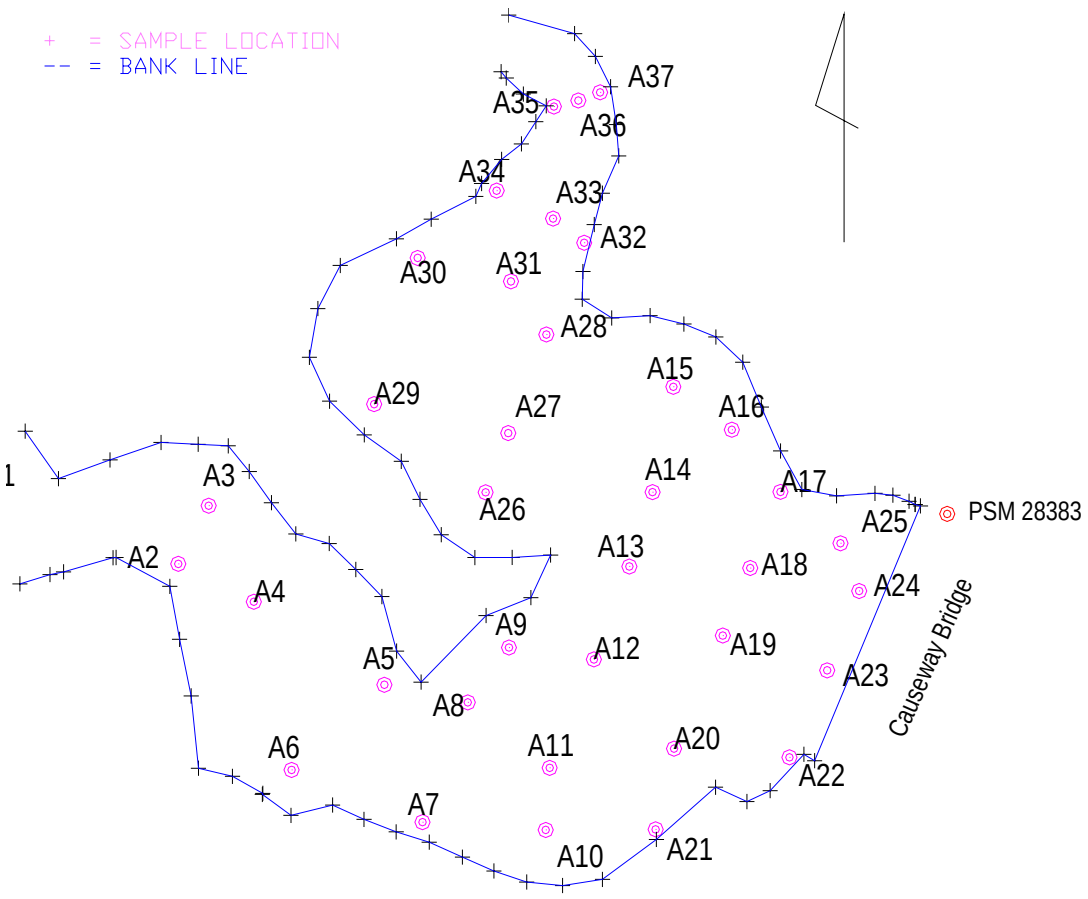


Figure 3.20: Location of bottom sampling

Sieve analysis and pH level measurement were next carried out and the results are shown in *Table 3.6*. The percent of material which passed the 75 µm sieve was considered as wash load. Figure 3.21 shows distribution of D_{50} particles in the lake.

Analysis of data, shown in *Table 3.6*, indicates that the main sand bar in the central part of the lake is made of sand with almost no silt particles. Wash load in the samples A12, A13, A17, A18, and A25 is very low, in the order of 1%. The Mulambin Creek branch has the highest percentage of silt (over 20%) as found in the samples A14, A16, and A26. The Shoal Creek branch has lower rates, generally about 10 – 20% of silt.

Sample	Grain size [mm]		Finer than 10% sieve	pH
No	D_{50}	D_{90}	%	
A1	0.25	N/A	11.8	7.9
A2	0.13	0.26	6.8	8.2
A3	0.20	0.74	18.9	
A4	0.17	0.30	13.2	
A5	0.19	1.00	19.1	7.7
A6	N/A	N/A		
A7	0.19	0.28	2.6	8.4
A8	0.11	0.30	22.3	8.1
A9	0.26		11.0	7.8
A10	0.16	0.49	6.6	
A11	0.17	0.41	13.0	8.1
A12	0.16	0.27	1.8	8.4
A13	0.19	0.29	1.0	8.5
A14	0.14	0.46	22.4	8.1
A15	0.17	0.80	17.0	8.0
A16	0.15	0.80	25.8	
A17	0.18	0.28	0.4	8.2
A18	0.00	0.31	0.9	8.5
A19	0.19	0.45	13.4	8.1
A20	0.13	0.50	22.2	
A21	0.19	0.47	6.5	8.1
A22	0.18	0.40	6.5	8.1
A23	0.30	1.20	8.2	8.0
A24	0.41	N/A	18.0	8.0
A25	0.20	0.30	1.2	8.5
A26	0.12	0.85	19.7	7.7
A27	0.12	0.34	13.2	8.0
A28	0.20	0.29	1.0	8.2
A29	0.12	0.27	19.8	
A30	0.10	0.20	28.4	8.1
A31	0.22	0.92	17.4	
A32	0.14	0.50	22.2	
A33	N/A	N/A		
A34	0.12	0.22	14.1	
A35	0.12	0.50	20.9	
A36	0.22	1.10	20.9	
A37	0.18	0.32	1.8	8.1
min	0.10	0.20		
max	0.41	1.20		
avg	0.18	0.49		

Table 3.6: Characteristics of the Causeway Lake bottom material

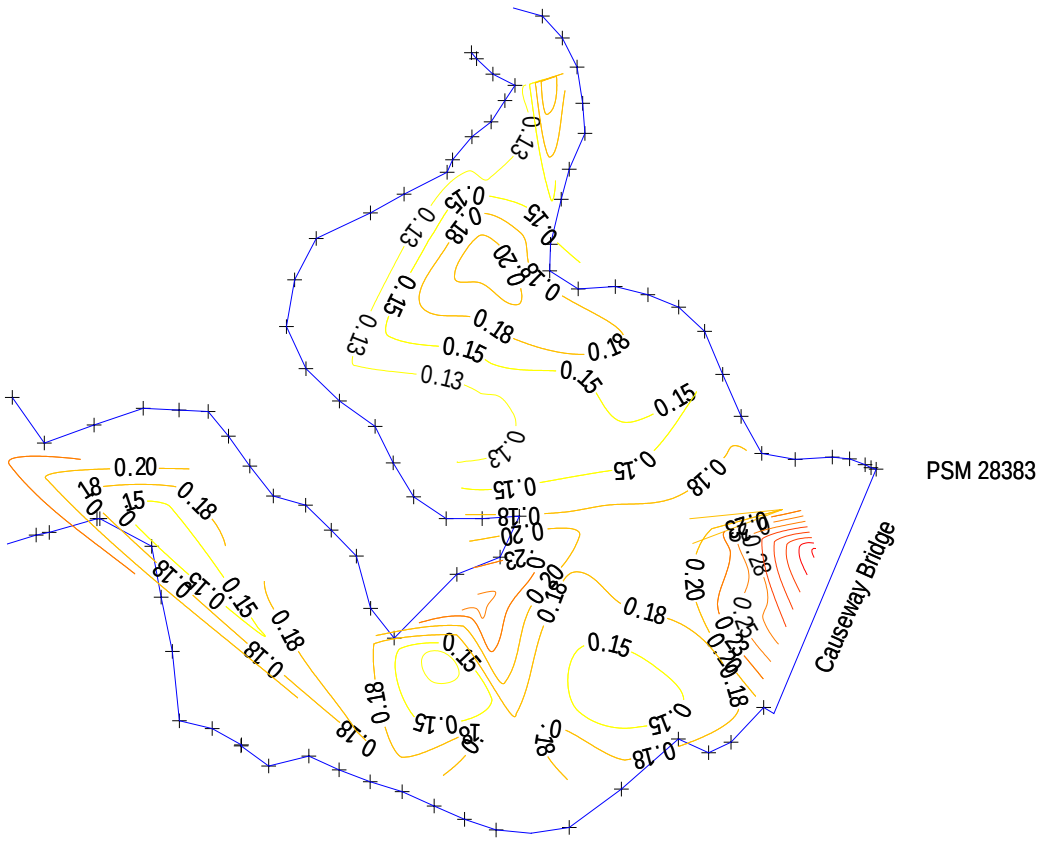


Figure 3.21: Distribution of D_{50} [mm] in the Causeway Lake

The last column in *Table 3.6* shows the level of pH in the bottom samples. The pH scale ranges from 0 to 14, with acidic solutions having a pH below 7 and alkaline ones over 7. The level of the pH is around 8 indicating that the bottom material is free of acid.

3.10. Field Measurement of Sediment Transport

3.10.1. Sediment transported by tide

This section summarises analysis to evaluate total suspended sediment entering the lake during flood tides exceeding the sill level of the bridge. Figure 3.22 to Figure 3.24 show average measured concentration of suspended sediment during flood and ebb tide periods. Concentration was evaluated in the laboratory as described in section 3.8.1.

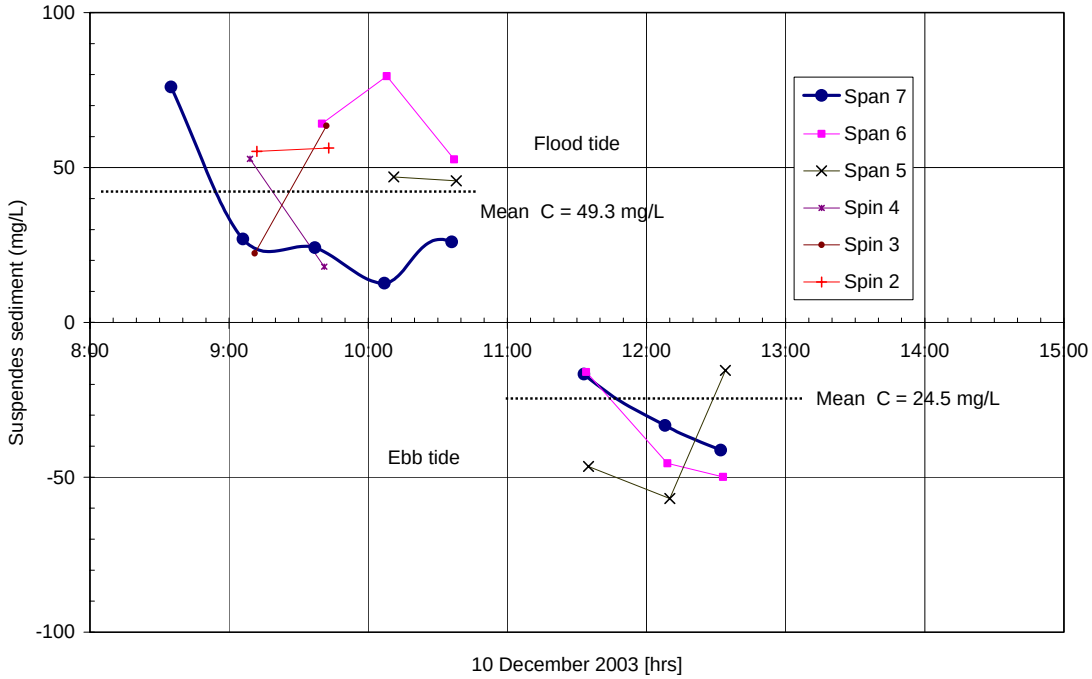


Figure 3.22: Suspended sediment concentration during tide on 10 December 2003

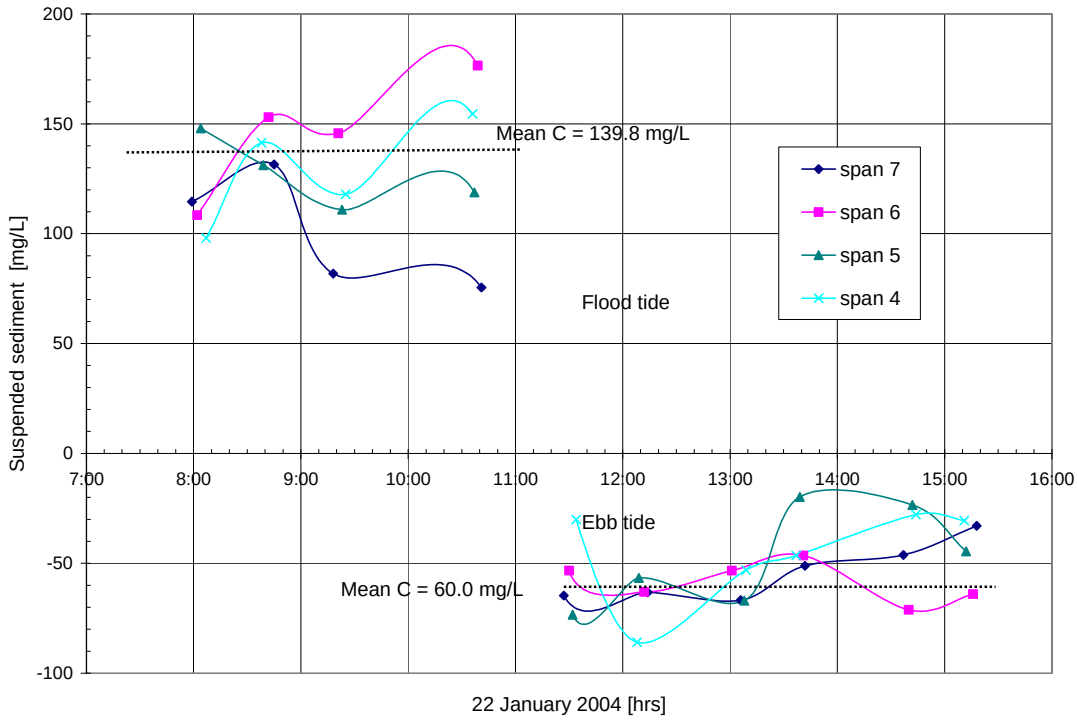


Figure 3.23: Suspended sediment concentration during tide on 22 January 2004

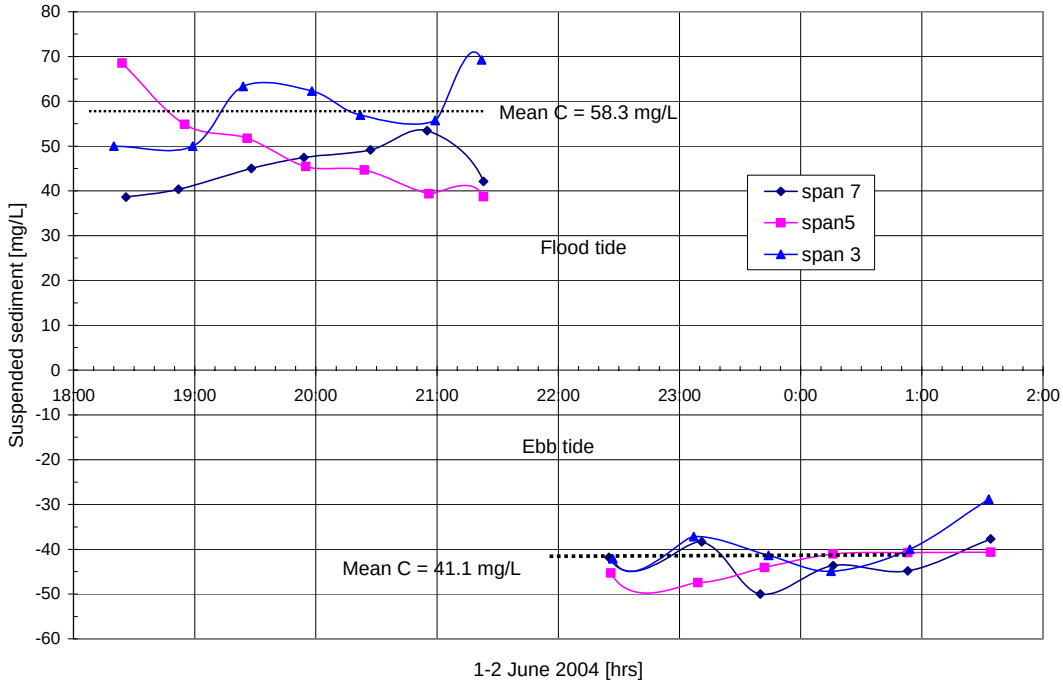


Figure 3.24: Suspended sediment concentration during tide on 1/2 June 2004

The relationship between sediment concentration and Tide Level can be drawn as shown in Figure 3.25.

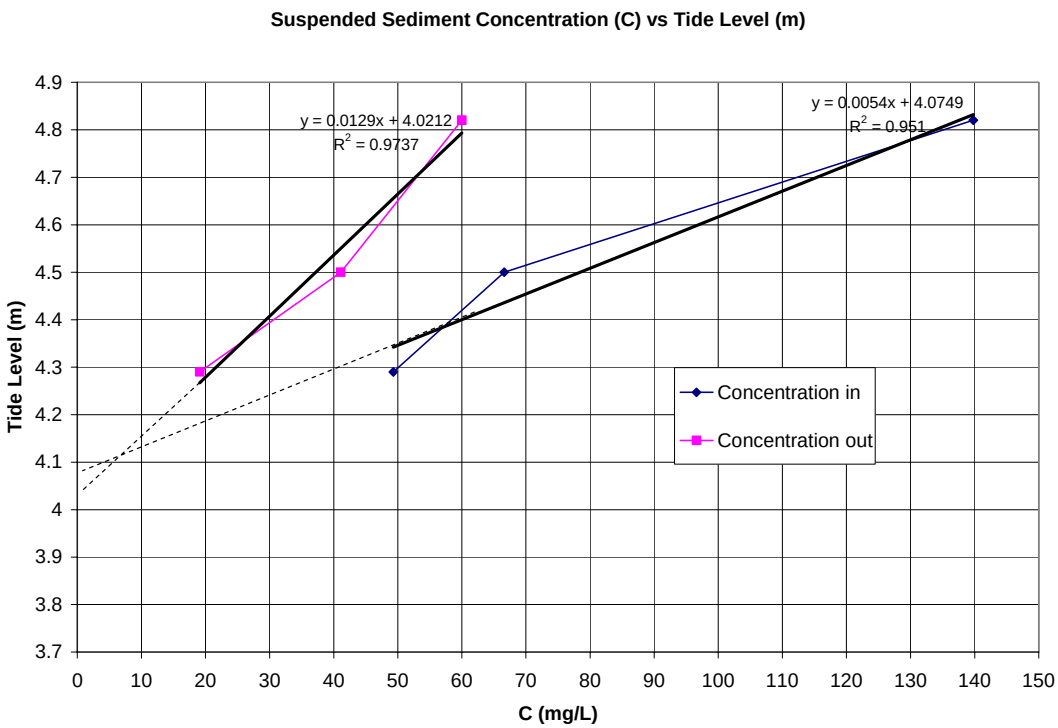


Figure 3.25: Estimated sedimentation concentration in the Causeway Lake

Figure 3.25 shows the relation between Tide Level and Concentration (In/Out). The equations can be expressed as:

C=185*(HW-4.10) mg/L Flood Tide (Equation 3.17)

C=77.5*(HW-4.04) mg/L Ebb Tide (Equation 3.18)

Having an average concentration of suspended sediment and the total volume of tidal prism entering the lake, the total volume of sediment entering the lake from the sea was evaluated. Based on the evaluated volume of water leaving the lake during ebb tides, the volume of suspended sediment leaving the lake was also estimated. It was assumed that during ebb tides the velocities in the lake are much lower, therefore any volume of water left in the lake after the next flood tide was contributing to the lake sedimentation. The results of calculations are shown in *Table 3.7*.

	Flood Tide			Ebb Tide			Siltation in the lake
HW [m LAT]	Vol in [ths m^3]	C [mg/L]	Mass in [t/tide]	Vol out [ths m^3]	C [mg/L]	Mass out [t/tide]	Sediment [t/tide]
4.29	93	49.3	4.59	71	24.5	1.74	2.85
4.50	306	58.3	17.84	227	41.1	9.34	8.50
4.82	838	139.8	117.15	600	60	36.00	81.14

Table 3.7: Measured sedimentation in the Causeway Lake

Obtained results of tidal sedimentation in the lake are graphically presented in Figure 3.26 and can be expressed by the following equations:

$$S = \left[\frac{HW}{4.07}\right]^{30} \qquad \text{(Equation 3.19)}$$

Where:

S = Sedimentation Settle in the Lake (Tonnes / tide)

HW = High Water Elevation (m LAT)

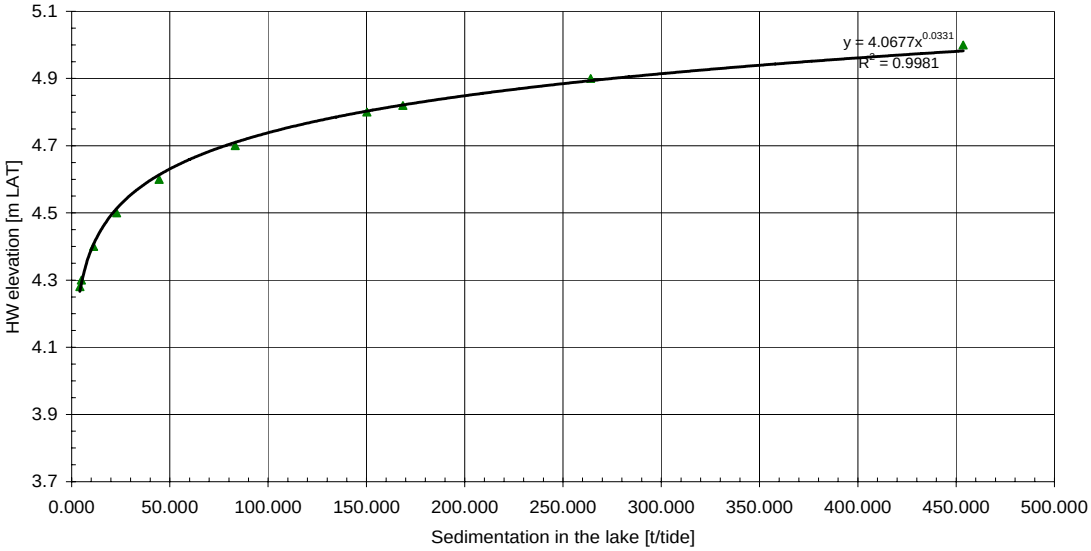


Figure 3.26: Estimated sedimentation in the Causeway Lake caused by suspended sediment entering the lake during tides exceeding the sill level of the bridge

The equation 3.19 is a final effect of analysis of volume of flow and sediment concentration for the flood and ebb tide periods. It is the result of sedimentation in the lake for the average weather conditions.

The equations were used to evaluate annual sedimentation in the Causeway Lake cause by tidal action. Average numbers of tides per year exceeding the bridge sill level, 3.76m LAT, were evaluated based on data from Maritime Safety Queensland shown in *Table 3.8*. The analysis found that about 2743 tonnes X 2.65 t/m^3 = 1035 m^3 of sediment from the sea has annually been settling in the lake.

HW	Flood tide		Ebb tide		Siltation in the lake 5 = 2-4	No of tides per year	Total annual siltation	
	Tidal prism [Eq 2.16] 1	Suspended sediment [Eq 3.17] 2	Tidal prism [Eq 2.17] 3	Suspended sediment [Eq 3.18] 4				
[m]	[mil m3]	[t/tide]	[mil m3]	[t/tide]	[t/tide]		[t/year]	
3.70	0.00	0.00	0.00	0.00	0.00	25	0	
3.80	0.01	0.00	0.01	0.00	0.00	27	0	
3.90	0.02	0.00	0.02	0.00	0.00	24	0	
4.00	0.03	0.00	0.03	0.00	0.00	23	0	
4.10	0.05	0.92	0.05	0.35	0.57	20	0	
4.20	0.08	3.03	0.07	1.06	1.97	20	39	
4.30	0.13	7.39	0.10	2.37	5.01	17	85	
4.40	0.21	15.85	0.15	4.70	11.16	13	142	
4.50	0.34	31.55	0.22	8.63	22.92	10	226	
4.60	0.54	59.68	0.33	15.10	44.58	8	346	
4.70	0.84	108.66	0.47	25.47	83.19	6	469	
4.80	1.30	192.02	0.68	41.77	150.25	3	424	
4.90	1.99	331.03	0.97	66.92	264.11	2	559	
5.00	3.02	558.78	1.37	105.14	453.64	1	454	
					Total	198	2743	tonnes
					Siltation Volume		1035	m3

Table 3.8: Average annual siltation in the Causeway Lake caused by tides action

During the period of a tide cycle, sediment concentration in is not necessarily equal to sediment concentration out. There are several factors which affect the sedimentation into the estuary.

In Causeway Lake, the bridge is obstructing the flushing of sediment out during the ebb tide. During the flood tide, when sea water is entering the estuary through the narrow channel, water velocity is considerably high. It brings sediment into the estuary. At the end of the flood tide, conditions become very calm, and finally reaches the point when the water flow is almost zero. This condition is called slack water. During the ebb tide, water flows out to the sea with low velocity, and sediment is again having a chance to settle in the lake.

3.10.2. Sediment Transportation from the catchments

3.10.2.1. Predicted sediment discharge into the lake

The bottom sediment material could be transported into the Causeway Lake through two creeks: Mulambin Creek and Shoal Creek (Figure 3.27 and Figure 3.28). The narrower cross-sections of the creeks were measured (Figure 2.36 and Figure 2.40) and additional data for sedimentation analysis is shown in *Table 3.9*.

Figure 3.27: View of Mulambin Creek entering the Causeway Lake

Figure 3.28: View of Shoal Creek entering the Causeway Lake

	Shoal ck	Mulambin ck
Manning coefficient n	0.05	0.05
Catchment [km^2]	10.7	2.3
Length of creek {km]	3.5	2.1
D_{50} [mm]=	0.12	0.12
D_{90} [mm]	0.8	0.5
Bottom material relative density s=	1.8	1.8
Kinematic viscosity [m^2/s]	1.00E-06	1.00E-06

Table 3.9: Characteristic cross-sections of the creeks approaching the Causeway Lake

There are various sediment transport relationships with the most recent formulas based on computer optimisation of observed data using various dimensionless parameters (Some formulas discussed in Section 3.5). Since all the formulas are based on experimental data, the quality of predictions depends on whether the conditions correspond to those on which the particular formula was based. The assessment of the quality of any formula does not only depend on the formula but also on the fact that sediment transport in nature is extremely difficult to measure accurately. Therefore results of calculation are considered as good ones if predicted sediment transport rates are within 0.5 to 2 times the observed values (Van Rijn 1984a).

The mean critical velocity initiating sediment transport from the catchments to the lake calculated with Equation 3.12 for the possible range of average depths is shown in *Table 3.10*. The critical mean velocity U_c, can be assumed as 0.30 m/s in both creeks.

Depth [m]		Velocity U_c [m/s]	
Shoal Creek	Mulambin Creek	Shoal Creek	Mulambin Creek
1.30	0.75	0.31	0.29
1.40	0.85	0.31	0.30
1.50	0.95	0.31	0.30
1.60	1.05	0.32	0.30
1.70	1.15	0.32	0.31
1.80	1.25	0.32	0.31
1.90	1.35	0.32	0.31
2.00	1.45	0.32	0.31
2.10	1.55	0.33	0.32
2.20	1.65	0.33	0.32
2.30	1.75	0.33	0.32

Table 3.10: Mean critical velocity causing sediment transport

Predicted bottom and suspended sediment transport in Shoal Creek and in Mulambin Creek were next calculated using Equation 3.13 and Equation 3.14. In this

calculation density of bottom material was assumed to be equal to 1800 kg/m^3 compared with 2650 kg/m^3 for sediment transported by flood tides from the sea side.

Van Rijn equations are used to calculate sediment transport. Calculations shown in *Table 3.12* and *Table 3.13* indicate that the bottom transport is less significant in analysis, as it is of the order of 10 times smaller than suspended sediment transport. The results show the quantity of sediment transport in units of discharge (m^3/hr). Mulambin Creek transported more sediment than Shoal Creek. To obtain the total possible average annual volume of sediment transported into the lake from the catchment areas, first of all an analysis of the flows in the creeks was carried out. Using Manning equation (Equation 2.12) the flow in creeks were evaluated. (*Table 3.11*).

There are several assumptions at this stage of study, particularly related to the surface slope. Mean surface slope of the creeks were evaluated base on local surveying. However with the increase of discharges, it's also increasing mean depth of the slope's creeks. It is presented in the Table 3.11

Mean hydraulic depth D in cross-section [m]		Surface slope S_f	Mean velocity U [m/s]		Discharge Q [m^3/s]	
Shoal Creek	Mulambin Creek		Shoal Creek	Mulambin Creek	Shoal Creek	Mulambin Creek
1.3	0.6	0	0	0	0	0
1.4	0.7	0.00002	0.14	0.09	20	6
1.5	0.8	0.00004	0.21	0.14	31	11
1.6	0.9	0.00006	0.26	0.18	42	16
1.7	1.0	0.00008	0.32	0.22	54	22
1.8	1.1	0.0001	0.37	0.27	67	29
1.9	1.2	0.00012	0.42	0.31	80	37
2	1.3	0.00014	0.47	0.35	94	46
2.1	1.4	0.00016	0.52	0.40	109	55
2.2	1.5	0.00018	0.57	0.44	125	66
2.3	1.6	0.0002	0.62	0.48	142	77

Table 3.11: Probable fresh water discharges in the creeks

Based on theoretical calculations shown in *Table 3.11*, probable discharges from both creeks are as follows:

- Mulambin Creek, D = 0.75, total Q = 8 m^3/s.
- Shoal Creek, D = 1.35, total Q = 10m^3/s.

Assumptions are correct for the theoretical analysis as we get similar values based on field measurement data from *Table 2.12* for Mulambin Creek and *Table 2.13* for Shoal Creek.

U [m/s]	transport m^3/s.m width		transport m^3/s in cross-section		Total
	bottom	suspended	bottom	suspended	[m^3/hr]
0.35	1.75E-07	1.44E-06	0.0000	0.0002	0.7
0.40	8.76E-07	7.17E-06	0.0001	0.0010	4.0
0.45	2.44E-06	2.00E-05	0.0003	0.0027	10.8
0.50	5.22E-06	4.27E-05	0.0007	0.0058	23.4
0.55	9.61E-06	7.87E-05	0.0013	0.0106	42.9
0.60	1.60E-05	1.31E-04	0.0022	0.0177	71.6
0.65	2.49E-05	2.04E-04	0.0034	0.0275	110.9
0.70	3.66E-05	3.00E-04	0.0049	0.0405	163.4

Table 3.12: Evaluation of sediment transport in Mulambin Creek as a function of mean velocity in cross-section "M"

U [m/s]	transport m^3/s.m width		transport m^3/s in cross-section		Total
	bottom	suspended	bottom	suspended	[m^3/hr]
0.35	6.53E-08	3.97E-07	0.0000	0.0001	0.4
0.40	5.08E-07	3.08E-06	0.0001	0.0005	2.2
0.45	1.64E-06	9.93E-06	0.0003	0.0015	6.5
0.50	3.77E-06	2.29E-05	0.0006	0.0035	14.8
0.55	7.24E-06	4.40E-05	0.0011	0.0068	28.4
0.60	1.24E-05	7.54E-05	0.0019	0.0117	49.0
0.65	1.97E-05	1.20E-04	0.0031	0.0185	77.8
0.70	2.94E-05	1.79E-04	0.0046	0.0277	116.3

Table 3.13: Evaluation of sediment transport in Shoal Creek as a function of mean velocity in cross-section "S"

Based on findings from *Table 3.12* and *Table 3.13,* the sediment discharges from the catchments occur when the mean water velocity is in the order of 0.4 m/s or more. It represents discharge in the order of 70 m^3/s from Shoal Creek and 55 m^3/s from Mulambin Creek (Highlighted in *Table 3.11*). It can be conservatively assumed that part of this flow, of the order of 20 m^3/s will be fresh water flow.

3.10.2.2. Catchment sediment transported into lake

When combining the quantity of sediment transported by creeks into the lake (*Table 3.12* and *Table 3.13*) with probable discharges and number of rainy days (*Table 2.5)*, the expected volume of sediment transport from the catchments into the lake was achieved and the results are presented in *Table 3.14*. The estimated volume of transported sediment was based on assumptions that flow into the lake may occur for 6 hours of an ebb tide period when water elevation exceeds the elevation of the sill under the bridge.

Sample calculation for mean velocities of 0.4 m/s:

- Sediment transport at Mulambin Creek is 4 m^3/hour (*Table 3.12*), thus in 6 hours is 24 m^3/ebb tide
- At Shoal Creek total sediment transport is 2.2 m^3/hour (*Table 3.13*), thus in 6 hours duration is 13.2 m^3/ ebb tide,
- Total discharge from both creeks for each ebb tide is 13.2 + 24 = 37.2 m^3/ ebb tide
- Then from *Table 2.5*, the occurrence of rainfall which exceed 60 mm/day can be found
- Thus the quantity of sediment annually can also be found.

Mean velocities in Creeks	Total sediment transport	No of occurrence 1940 - 2003	Sediment transport [m^3]	
m/s	m^3/ebb tide	days	total	Means annual
0.4	37.2	152	5654	90
0.5	229.2	38	8710	138
0.6	723.6	22	15919	253
0.7	1678.2	23	38599	613
			Total	1093

Table 3.14: Estimated order of average annual sediment transported from catchments into the Causeway Lake

Thus expected sediment transport from the catchments is of the same order as from the sea giving together an average annual sedimentation in the order of 2000 m^3 without considering wash load that comprised about 20% of the total load (*Table 3.14*). The accuracy of estimation can only be verified by comparison with the bathymetry surveys showing average siltation of the lake in the order of 2500 m^3/year.

To evaluate the accuracy of the computed and measured values, a discrepancy ratio (r) has been used, defined as:

$$r = \frac{siltation_transport_computed}{siltation_transport_measured}$$

In our case it will be:

$$r = \frac{field_data_analysis}{bathymetry_analysis} = \frac{2000}{2500} = 0.8$$

Analysis of various field data with several experimental formulas describing sediment transport (bed-load and suspended) carried out by van Rijn (1984), shows deviations in the results up to a factor of r = 2. The relatively low accuracy of existing equipment for measuring sediment transport also justifies the use of simple approximation functions with less accuracy, to avoid the use of complicated numerical solution methods.

3.11. Causeway Lake Sedimentation Findings

Available bathymetry of the lake from 1986 was used as a comparison with the existing bathymetry surveyed in 2003. The average sedimentation on the 66ha of the lake was evaluated at about 2500 m^3/year. However bathymetry analysis itself does not answer the following questions:

- What is source of sediment?
- What is the distribution of sediment from the sea and from the catchments?

Therefore a set of field data measurement and analysis has been undertaken.

Based on aerial photos and site measurements, there is visible separation between sediment transported by tide (basically limited by a sand bar) and that transported from the catchments by two creeks approaching the lake: Mulambin Creek and Shoal Creek. The central part of the lake with a total of 28 ha of the lake area with the visible sand bar, has sedimentation in the order of 2000 m^3/y.

The central part of the lake has coarse bottom material with D_{50} in the range of 0.2 – 0.3 mm whereas closer to the entrances from the Shoal Ck and Mulambin Ck, bottom material diameter is reduced to 0.1 mm and a significant thickness of mud is observed (up to 0.2m). Wash load based on sieve analysis is on average 10 to 25%. Also near the entrances from both creeks, significant amounts of sea grass and weeds are observed.

Measurements of discharges under the bridge during flood and partly ebb tides, together with measurements of suspended sediment concentration, concluded that

about 1050 m^3/y can be transported into the lake by tide currents when tide levels exceed the bridge sill level.

Analyses of the quantity of sediment transported from the catchments were more difficult and were carried out based on several assumptions. Van Rijn's empirical formulas were used to evaluate relation between flow velocities and sediment transport. Next, flow velocities and discharges were related to the catchment characteristics and rainfall intensity using the Rational Method. Having only daily rainfall data, it was justified that significant transport of sediment could happen if daily rainfall exceeded 60 mm. It was also assumed that tidal prism in the creeks' cross-sections has insignificant influence on the sediment transport. All these assumptions and deductions brings us to the conclusion that average annual sediment transport from the catchment areas is of the order of 1100 m^3.

4. ESTUARINE HYDRODYNAMIC AND SEDIMENT TRANSPORT NUMERICAL MODELS

4.1. Introduction

The other chapters of the thesis were connected with the modelling hydrodynamics and sedimentation processes in the Causeway Lake. To understand principles of numerical modelling some basic information related to the models, types and their application was initially described in this chapter.

Predicting the future changes of estuary bathymetry due to flow and sedimentation can be done by using surface water modelling. By nature, water flow or tidal current is very complex and hard to predict. When turbulent flow is involved or different water density is identified, then modelling will be more difficult to simulate. Modelling software is usually a simplification of the real situation, however major components of hydrodynamics are fully represented. When the density variation is identified, flow can be categorised as stratified, otherwise it is called homogeneous. Numerical modelling can be categorised by five major types (King 2004).

4.1.1. Fully three dimensional flow

Three dimensional models are the most complex among models. The inertial component of water velocity in the vertical direction needs to be considered. Navier-Stokes Equations for conservation of momentum in all three directions must be solved. Additional transport equations may be required to simulate density-influencing parameters. Fully Three dimensional models can be well applied for high speed flow, flow close to intakes for pumps and turbines in reservoirs or streams, flow obstructed by baffles or screens and flow near artificial de-

stratification devices in reservoirs, where the propellers or bubblers introduce vertical momentum (King 2004).

4.1.2. Three dimensional flow - the hydrostatic assumption

When the vertical accelerations are small, the hydrostatic assumption is used. It is less complex as the vertical momentum equation can be replaced with an algebraic one. These equations are called the Shallow Water Equations. This modelling is suitable for stratified flow in coastal and estuarial systems, where the water depth is less than 50 meters, stratified flow in deep reservoirs with small velocities, homogeneous flow systems where wind stresses are large enough (King 2004).

4.1.3. Two dimensional depth averaged (or vertically integrated) flow

This is a depth averaged version of the Shallow Water Equations. The assumption is that two average horizontal velocity components can represent the vertical distribution of horizontal velocities. The governing equations are fully two dimensional in the horizontal plane. Two dimensional models suit shallow lakes, reservoirs and coastal systems where wind stresses and stratification are not significant, estuaries influenced by tidal flow and rivers and streams (King 2004).

4.1.4. Two dimensional laterally averaged flow

This approach uses laterally averaged versions of the of the Shallow Water Equations. The simplification consists of assuming that a single average horizontal velocity can represent the lateral distribution of horizontal velocities. The governing equations then become fully two-dimensional in the vertical plane. This type of

formulation is suitable for narrow deep reservoirs that are subjected to stratified flow conditions, and homogeneous flow along rivers with obstructions in the bed, such as submerged weirs (King 2004).

4.1.5. One-dimensional cross-section averaged flow

One dimensional flow uses cross-section averaged versions of the of the Shallow Water Equations. The simplification consists of assuming that a single average velocity can represent the cross sectional distribution of horizontal velocity. The governing equations then become fully one-dimensional along the horizontal axis of the system. It suits a river or stream system that is relatively narrow with little variation across the section (King 2004).

4.2. Numerical Model suited to the Causeway Lake condition

Causeway Lake is an estuary which has three boundaries: Mulambin Creek, Shoal Creek and Sea Boundary. The estuary can be categorized as a shallow estuary with average depth less than 3 meters. The major influence of hydrodynamic force is by tidal currents during the high tide periods and flow from the creeks during heavy rainfall periods. The influence of waves is minimal as the estuary is blocked by the 1.4 m AHD bridge sill. Based on the explanation above a two dimensional depth averaged model is well suited to simulate the hydrodynamic and sedimentation processes of Causeway Lake.

The following surface water models are as the two dimensional depth averaged type suitable to simulate hydrodynamic and sediment transport in a shallow estuary: MIKE21, Delft3D and Resource Modelling Associates (RMA).

4.2.1. MIKE21

MIKE21 is a two dimensional depth averaged hydrodynamic model developed by the Danish Hydraulics Institute (DHI). This model can be used to simulate waves, currents and sediment transport. The MIKE21 modules are as follows (Johnson 2002):

MIKE21 HD

The hydrodynamic module is based on the equations of motion for large water bodies. The function of this module is to simulate the tidal flows and water levels. MIKE21 HD includes formulations for many marine processes including convective and cross momentum, bottom and wind shear stress and wave-induced currents.

MIKE21 NSW

The Nearshore Wave module is a spectral wind-wave model. It describes the propagation, growth and decay of short-period waves in the nearshore environment. The model includes the effects of refraction and shoaling due to varying depth, wave generation due to wind and energy dissipation due to bottom friction and wave breaking.

MIKE21 PMS

The Parabolic Mild Slope module of MIKE21 solves the mild slope equation using a parabolic approximation to allow resolution of wind waves on gently sloping bathymetry. The model can be applied in any water depth and includes the effects of shoaling, refraction, dissipation due to wave breaking and bed friction, forward scattering and partial diffraction.

MIKE21 BW

This is a Boussinesq Wave module and is used for detailed assessment of flow in harbours where directional wave measurement and wave-wave interaction are important. The processes included in this model are shoaling, refraction, diffraction, frequency dispersion and partial reflection. The equations are non-linear and can be used for areas with complex bathymetry.

MIKE21 ST

Non-cohesive sediment transport modelling is performed using MIKE21 ST which uses inputs from the wave and hydrodynamic modules to determine sediment transport rates in coastal areas. Initial bed level changes are calculated and coastal structures, such as breakwaters, can be included. MIKE21 ST includes the effects of combined wave-current flow, breaking and non-breaking waves, graded or uniform bed material and plane or rippled beds.

4.2.2. DELFT-3D

Delft3D is a 2/3 dimensional hydrodynamic and sediment transport simulation program which calculates flow and sediment transport. It is mostly used for the modelling of coastal, river and estuarine areas. Delft3D is developed by Delft Hydraulics - Netherland and has the following modules (Delft Hydraulics 2003):

FLOW

The FLOW module of Delft3D is a multi-dimensional (2D or 3D) hydrodynamic and transport simulation program which calculates non-steady flow and transport phenomena resulting from tidal and meteorological forcing on a curvilinear, boundary fitted grid.

WAVE

The Delft3D-WAVE module can be used to simulate the propagation and transformation of random, short-crested, wind generated waves in coastal waters which may extend to estuaries, tidal inlets, barrier islands with tidal flats, and channels. The module computes the evolution of waves over arbitrary depths for certain wind, flow and water level fields.

SED

The SED module of Delft3D can be applied to model the transport of cohesive and non-cohesive sediments, e.g. spreading of dredged materials, to study sediment/erosion patterns.

4.2.3. RMA (Resource Modelling Associates) – Ian King Version

The RMA suite of hydrodynamic and water quality models is developed by Dr. Ian King, Resource Modelling Associates (RMA), NSW. Model may be used for coastal, estuarine and river simulation in steady state or dynamic mode. Modules which include one/two/three dimensional elements may be used in the same network. RMA has the following modules:

RMA2

RMA2 is a two dimensional depth averaged finite element hydrodynamic numerical model. It computes water surface elevations and horizontal velocity components for subcritical, free-surface flow in two dimensional flow fields. RMA2 computes a finite element solution of the Reynolds form of the Navier-Stokes equations for turbulent flows. Friction is calculated with the Manning's or Chezy equation, and eddy viscosity coefficients are used to define turbulence characteristics. Both steady and unsteady state (dynamic) problems can be analyzed.

RMA11

RMA11 is a finite element water quality model for simulation of three-dimensional estuaries, bays, lakes and rivers. It is also capable of simulating one and two dimensional approximations to systems either separately or in combined form. It is designed to accept input of velocities and depths, either from an ASCII data file or from binary results files produced by the two-dimensional hydrodynamic model, RMA2. Results in the form of velocities and depth from the hydrodynamic models are used in the solution of the advection diffusion constituent transport equations.

RMAGEN

RMAGEN is a graphically interactive module which may be used for creation and editing of finite element networks.

RMAPLT

This module is designed to run in conjunction with a mouse and a graphics display screen. Its purpose is the creation of velocity vector displays, contours, and time histories from the RMA series of finite element programs.

4.2.4. Selected Numerical Model

These models: MIKE21, DELFT3D and RMA are all suitable to simulate hydrodynamic and sediment transport in Causeway Lake. MIKE21 and DELFT3D are widely used worldwide by environment and surface water consultants. However RMA is also widely used by educational organisations. In terms of user friendly interface and presentation of output, DELFT3D and MIKE21 are outstanding compared to RMA. However the RMA package includes the source code which can be modified for future extension of the model into other areas. Thus for educational and research purposes including providing opportunity to develop numerical modelling skills and also for cost consideration, RMA is the selected model to simulate the Causeway Lake.

4.3. RMA (Resource Modelling Associates)

4.3.1. RMA-2

The governing equations consist of two conservation of momentum equations in the x and y direction and an equation of conservation of mass. The dependant variables are velocities (*u*) and (*v*) and depth (*h*) (See Figure 4.1). The variable for depth (h) is measured from the bottom elevation (a) (King 2004).

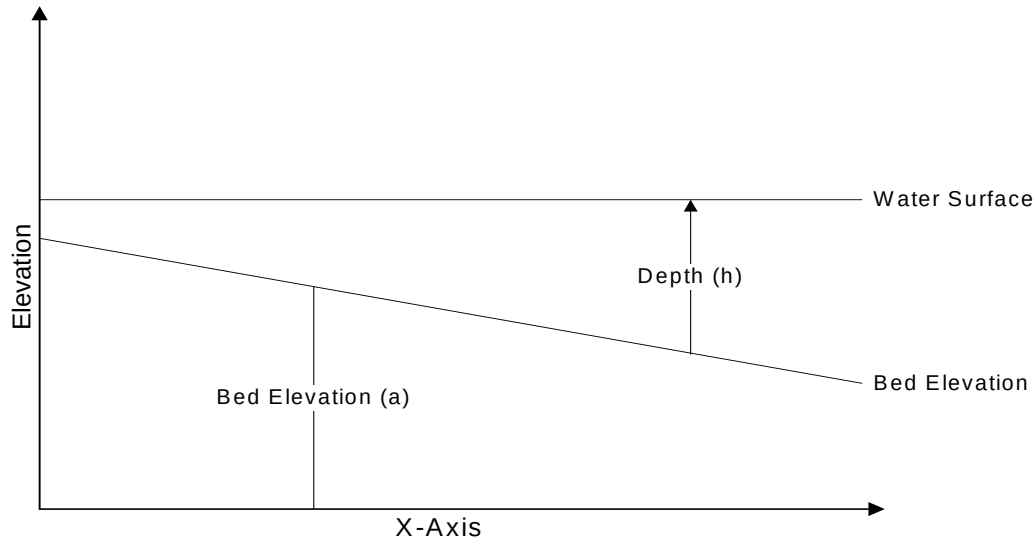


Figure 4.1: Two Dimensional Depth Averaged Flow Definition (King 2004)

The RMA-2 data input file consists of a series of blocks (*Table 4.1*).

1	**FILE DATA** Information on files that will be required for input or output *Required*
2	**CONTROL DATA** Control data defining this run *Required*
3	**SYSTEM PROPERTY DATA** Water, bed and system properties *Required*
4	**STEADY STATE STEP DATA** Time step and boundary condition data *Required*
5	**TRANSIENT STEP DATA** Time step and boundary condition data *Optional*

Table 4.1: Block Structure of RMA 2 (King 2004)

4.3.2. RMA 11

4.3.3. Sand Transport

Sand transport is modelled using the concept of sand transport potential. A sand transport formulation is used to define this potential which is assumed to depend on sand and flow parameters. In general terms the bed source or sink term is given by

$$S = \frac{C_{eq} - C}{t_c} \qquad \text{(Equation 4.1)}$$

Where:

S = Source term,

C_{eq} = Equilibrium concentration (transport potential),

C = Sand concentration in the water

t_c = Characteristic time for effecting the transition; this depends on whether deposition or erosion is occurring.

It is the selection of t_c that presents the most empirical step of the analysis. Model RMA 11 has four options allowed for computation of the sediment transport potential. All four options are basically empirical equations developed from:

1. The Ackers-White (1973) equations.

2. The Van Rijn (1984a and b) method derived from the 1984 papers.

3. The Brownlie (1981) method.

4. An updated Van Rijn method that allows for shear impacts of waves.

RMA 11 data input block is presented in *Table 4.2*.

1	**FILE DATA** Information on files that will be required for input or output *Required*
2	**CONTROL DATA** Control data defining this run *Required*
3	**SYSTEM PROPERTY DATA** Water, bed and system properties *Required*
4	**INITIAL CONDITION DATA** Initial condition and rate terms for all nodes *Required*
5	**TRANSIENT STEP DATA** Time step and boundary condition data *Required*

Table 4.2: Block Structure of RMA 11 (King 2004)

RMA-11 requires additional file of R4Q. The R4Q file contains information on sediment transport parameters. The general structure of the R4Q file is presented below (*Table 4.3*).

1	**SYSTEM CONTROL DATA** Information on constituents that are to be used overall prototype location. *Required*
2	**GLOBAL WATER QUALITY DATA** Water quality parameters that are common to the whole system *Required*
3	**ELEMENT TYPE DEPENDENT WATER QUALITY DATA** Water quality parameters that vary by element type number *One block required for each element type*
4	**METEOROLOGIC DATA** Weather data in old format *Optional*

Table 4.3: Block Structure of R4Q (King 2004)

4.4. Field Data for Model Calibration

RMA2 and RMA11 are based on the bathymetry of the area being modelled. In addition to the bathymetry data, the models require flow and tide data to form the boundary conditions. The models require inflow data $Q = f(t)$ at all inlets from catchments, and tide elevation data $H = f(t)$ on the ocean side. Wind, precipitation and evaporation may be considered by the model but were not included in this study. Thus we use upstream discharge (Q) and downstream water level (WL) based on measured data to verify upstream WL and downstream Q outputs from the model.

4.4.1. Bathymetric data

The bathymetry survey of the Causeway Lake undertaken by the author on 24 October 2003 was used for numerical modelling. The full analysis of this survey and its comparison with the previous ones was presented in Section 3.9.1. For modelling purposes, the sill level of 1.4 m AHD was converted to 0.00m datum. All survey coordinates are related to the PBM No 28383. The datum system is GDA94, Zone 56 and coordinates are 273709.011 (Easting) and 7432710.767 (Northing)

4.4.2. Water Discharges

The model requires information about discharges from sea side and creeks simultaneously, and the field measurements undertaken on 10 December 2004 were used to obtain data for the model calibration.

In the calibration stage, the water discharges from creeks were used as boundary conditions at the upstream side of the lake. As verification, the model prediction of

water discharge under the bridge and water elevations in the creeks' cross-sections was compared with the measured data. Adjustment of the model was made by proper selection of the Manning and turbulence coefficients (Sections 4.5.3.1 and 4.5.3.2).

4.4.3. Water Elevations

Tide is a major driving force of the Causeway Lake system. Tidal measurements were taken on 10 December 2004. Simultaneous measurements of velocities and water level elevations were taken at the inflows from Mulambin Creek and Shoal Creek into the lake, as well as inflow from the sea side under the Causeway Bridge. A water level change under the bridge was used by the model as boundary conditions on the sea side. Water level predicted by the model at the creeks' cross sections was compared with the field data measurement during the process of model calibration.

Recorded water levels for 10 December 2004 are presented in Figure 4.2. There is a visibly significant difference between water level "under the bridge" and "near the bridge", the latter representing elevation of water level inside the lake near the causeway. On the other side of the lake, the changes of recorded water levels were not significantly different at the creeks's cross sections, and they are in the range of accuracy of the pressure transducers used.

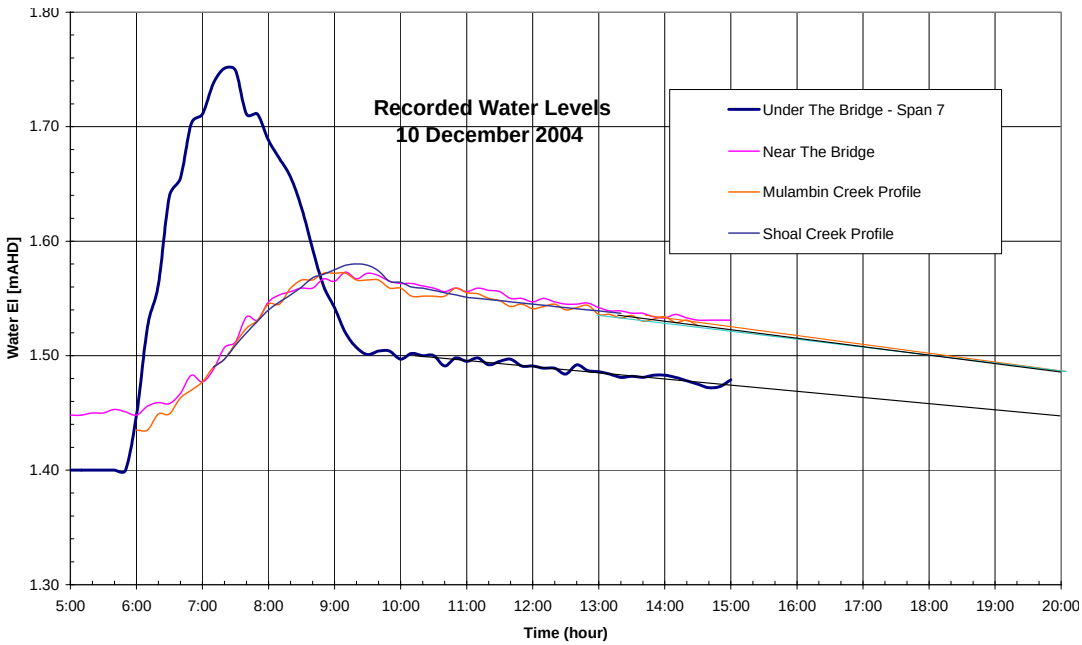


Figure 4.2: Causeway Lake recorded water levels 10 Dec 2004

More detailed information for water levels during the 10 December 2004 field data collection can be found in Section 2.8.2.4. It should be noted that more accurate evaluation of the water level was exceeding my possibility during the field test. However it teaches me the importance of accuracy as far as it is possible as it is the fundamental for the proper input into numerical model.

4.4.4. Sediment concentration

Sediment concentration at the sea side boundary, under the Causeway Lake Bridge, were obtained after analysing field data (Chapter 3.10.1). Then Equations 3.17 and 3.18 were derived for sedimentation rate during flood and ebb tide. Based on these analysed field data, concentrations of suspended sediment transport for use in the numerical model of 47 mg/L for upstream flow (flood tide) and 37 mg/L for downstream flow (ebb tide).

Sediment concentration from the creeks was not measured directly. It was evaluated based on the following assumption.

Using Van Rijn approach to evaluate sediment transport from the catchment, it was found that if the mean velocities are below 0.3 m/s, sediment transport is of low significance from Mulambin Creek and Shoal Creek catchments. Extrapolating the results of sediment transport from *Table 3.14* (Section 3.10.2.2) for average velocities of the order of 0.10 to 0.15 m/s in the measured cross-sections, the sediment transport could be of the order of 1 m^3/ebb-tide period, equivalent to a concentration of the order of:

$$C = \frac{S_v \cdot \rho_s}{Q_{ebb}} = \frac{1 \cdot 1800}{(220{,}000 + 160{,}000)} \cdot 10^6 \cdot 10^{-3} = 5\frac{mg}{L} \qquad \text{(Equation 4.2)}$$

Where:

S_v = Sediment Vol (m^3/ebb-tide)

ρ_s = Sediment density (kg/m)

Q_{ebb} = Vol of downstream water from both creeks (m^3/ebb-tide)

With a similar approach when considering the flood tide period, the evaluated sediment concentration will be of the order of 10 mg/L. These figures were assumed for model calibration.

4.5. Numerical Model Analysis

4.5.1. Grid Generation

The finite element grid was generated using survey points from a topographic survey. The latest survey data from year 2003 was used for this modelling.

The grid generation process was simplified by the use of the RMAGEN. The digitized bathymetry data were imported to RMAGEN, and a finite element grid was generated to represent the estuary. Reference water depths at each node of the model were interpolated from bathymetry data.

220 topographic data points are derived from survey data of the Causeway Lake estuary to form the model grid/mesh. The XY coordinates and elevations of the 220 points form the geometry of the model. *Figure 4.3* is the finite element model mesh that was developed for the Causeway Lake hydrodynamic and sedimentation study.

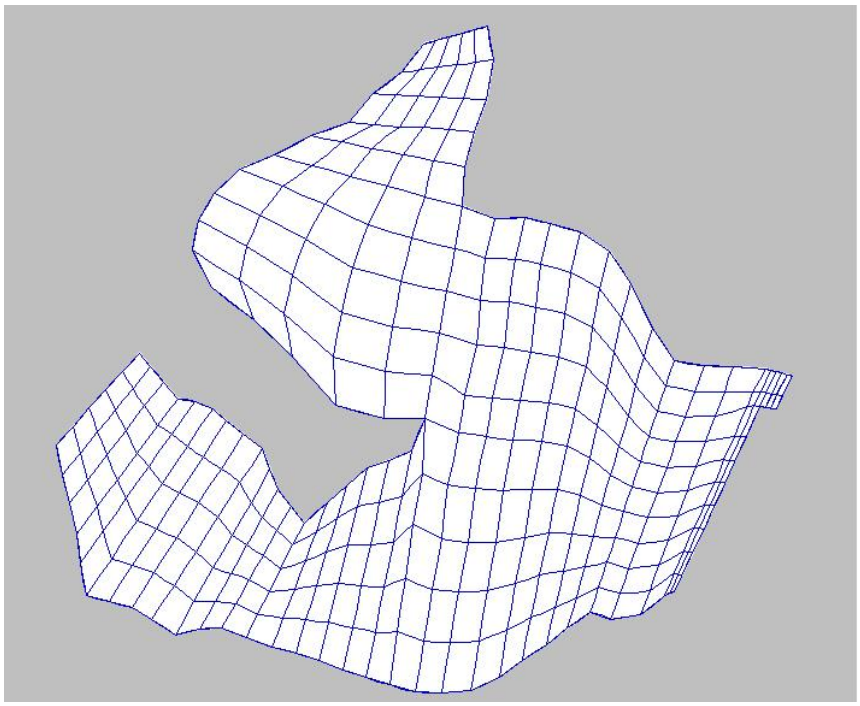


Figure 4.3: Finite Element Grid for Causeway Lake

4.5.2. Boundary condition specification

The hydrodynamic part of the modelling represented by the model RMA2 requires the following boundary conditions:

- Downstream boundary conditions (the Causeway bridge): a time-varying water surface elevation $H = f(time)$

- Upstream boundary conditions (Mulambin Creek and Shoal Creek): a time varying discharges $Q = f(time)$
- During calibration, discharges under the bridge and water surface elevation at the entrances to the Mulambin Creek and Shoal Creek were verified to obtain similar model output results to the data from the field measurements
- The variables in the calibration process were mainly bottom roughness coefficient and turbulent exchange coefficient.
- The required boundary data for calibration and for verification of the calibration were analysed in Section 4.4.3. The assumed time step in the model was 10 min.

4.5.3. Hydrodynamic model RMA 2 calibration

Numerous model simulations were required, specifying a range of friction and eddy viscosity coefficients, to calibrate the model. Calibration of the flushing model requires a close match between the modelled and measured tides in each of the cross sections where tides were measured. Initially, a one day period was calibrated to obtain visual agreement between modelled and measured tides. Once visual agreement was achieved, a one tide cycle period was modelled to calibrate the model.

4.5.3.1. Friction coefficients

Friction inhibits flow along the bottom of estuary channels or other flow regions. Friction is a measure of the channel roughness, and can cause both significant amplitude damping and phase delay of the tidal signal. Friction is approximated in RMA 2 as a Manning's coefficient. The Causeway Lake was basically divided into 3

parts based on field observation, the central part of the lake in which sandy material from the sea side has been settled, and then the two branches of the lake near the Shoal and Mulambin Creeks entrances into the lake in which sea grass has been growing. The bridge was considered as the sill concrete construction.

The Manning's coefficients representing each part of the lake were found by trial and error and are shown in *Table 4.4*.

Area	Manning's coefficients (Metric unit)	
Central Part of the lake involving sand bar	0.020	Sandy bottoms with average diameter of 0.2mm
Shoal and Mulambin Creek	0.032	Large amount of sea grass within those areas
Concrete Sill	0.011	Concrete materials

Table 4.4: Adopted Manning's coefficient n for simulating hydrodynamic processes in Causeway Lake

4.5.3.2. Turbulent exchange coefficients

Turbulent exchange coefficients approximate energy losses due to internal friction between fluid particles. The significance of turbulent energy losses increases where flow is swifter, such as inlets and bridge constrictions. According to King (2004), these values are proportional to element dimensions (numerical effects) and flow velocities (physics). In most cases, the modelled systems were relatively insensitive to turbulent exchange coefficients because there were no regions of strong turbulent flow. Several options can be chosen to assign the turbulence coefficients. In this project, the Peclet number (P) was used to set up the turbulence coefficients. Eddy

Viscosity (E) and the Peclet number are inversely related. Recommended values of P are in the range 15 to 40. For marsh or sea grass area P can be more than 40.

$$P = \frac{\rho \cdot u \cdot dx}{E} \qquad \text{(Equation 4.3)}$$

Where:

ρ = fluid density

u = average velocity of an element

dx = length of element in stream wise direction

E = Eddy viscosity

Adopted Peclet Numbers for the model are shown in *Table 4.5*.

Area	Peclet Number	
Central Part (Sand Bar)	38	Sand Bar
Shoal and Mulambin Creek	55	Sea grass covered area
Concrete Sill	28	Concrete materials

Table 4.5: The Peclet Numbers adopted for simulating hydrodynamic processes in Causeway Lake

4.5.3.3. Results of model calibration

After several trial and error iterations based on introduced boundary conditions, the calibration of the model achieved acceptable levels of accuracy.

Discharges under the bridge calculated by the model and those from the field measurements are compared as shown in Figure 4.4.

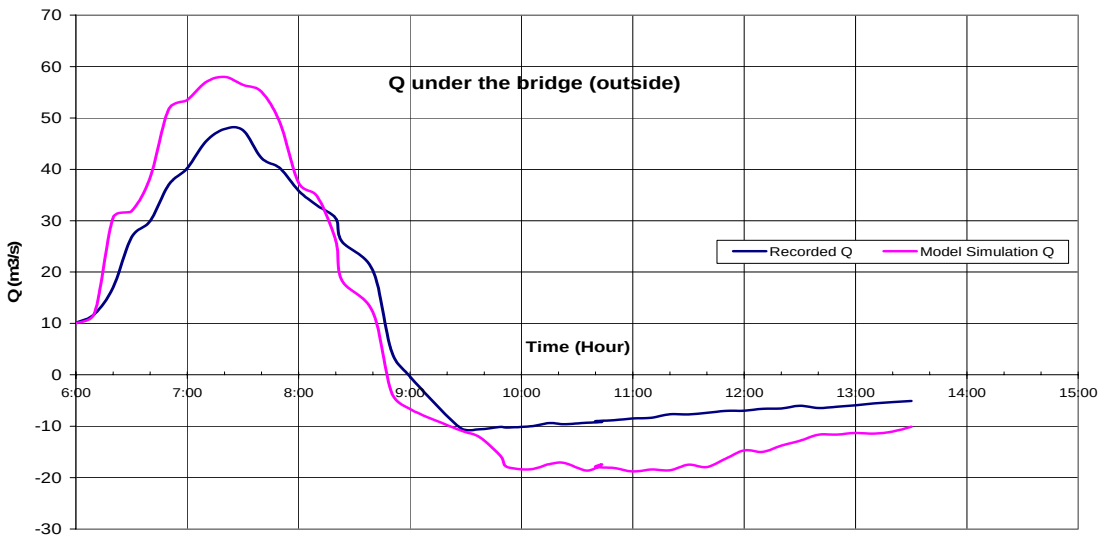


Figure 4.4: Measured and simulated discharges under the bridge for the period of one tide, 10 Dec 2004

Satisfactory qualitative agreement was achieved between model outputs and measured observations. However the model simulates higher discharges during both flood and ebb periods.

The model output water elevation in the Mulambin Creek cross-section follows the same trend as the field observation (Figure 4.5).

The model output water elevation in the Shoal Creek cross-section follows also the same trend as the field observation, and results are close to each other with differences generally below 1-2 cm (Figure 4.6).

Generally, the calibration indicates that evaluations of discharges from the creeks based on measurements were too high and requires additional more accurate measurements.

It should be noted that vertical axes in both Figures represent level of water surface above bridge sill level (1.4m AHD).

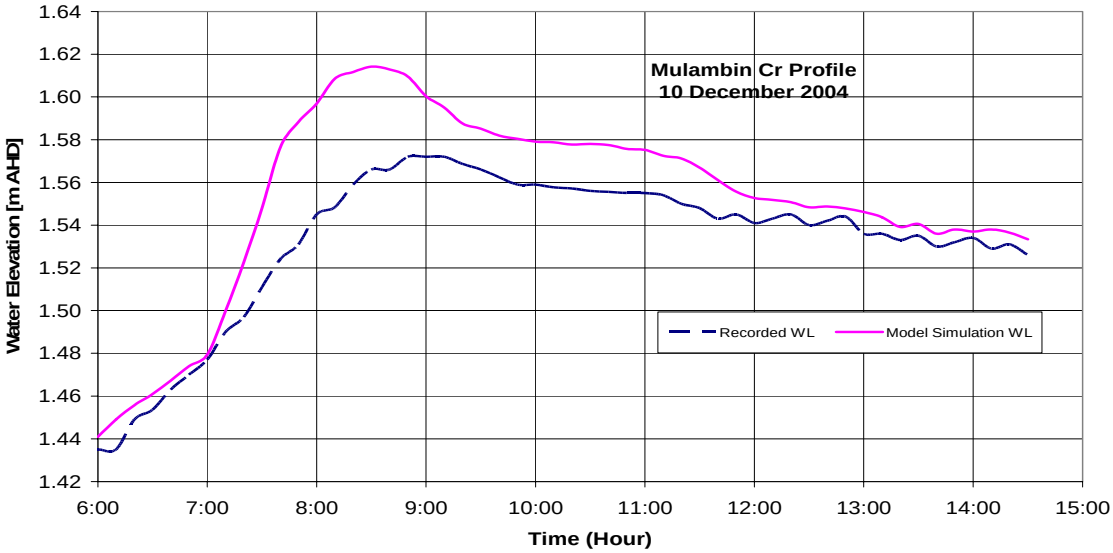


Figure 4.5: Model output vs. measured water levels in the Mulambin Creek cross-section

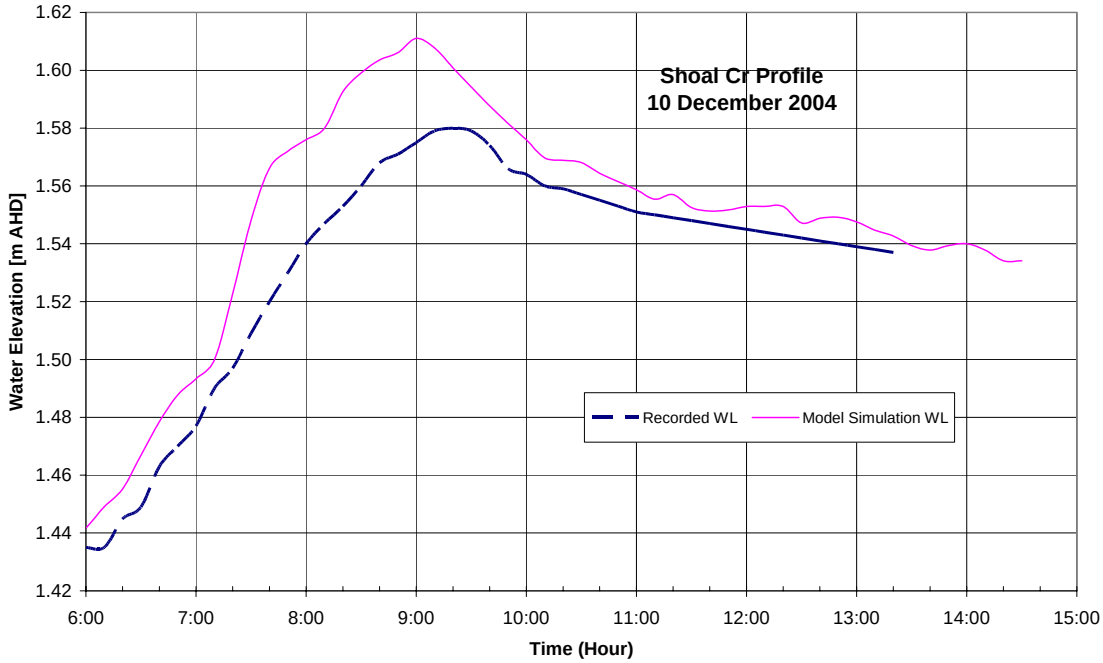


Figure 4.6: Model output vs. measured water level in the Shoal Creek cross-section

Figure 4.7 and Figure 4.8 respectively are snapshots of the hydrodynamic process during the flood tide, showing distribution of velocities. Initial flow towards the two creeks becomes more circular after a short period of time. The model simulated a full

tide cycle on year 2004, day 344 (Julian day which is referring to 10 December). Figure 4.7 is a snap shot of flood tide at 7.00am (hour 6.999), meaning 30 minutes before HW. The arrows show the velocity distribution at that time. The velocity distribution was differentiated by different colour. The highest velocity is under the bridge at this point of time. Figure 4.8 is a snap shot of the flood tide at 7.10am (hour 7.166). It shows that water makes a circular movement when reaching the centre of the lake at this stage of the tide.

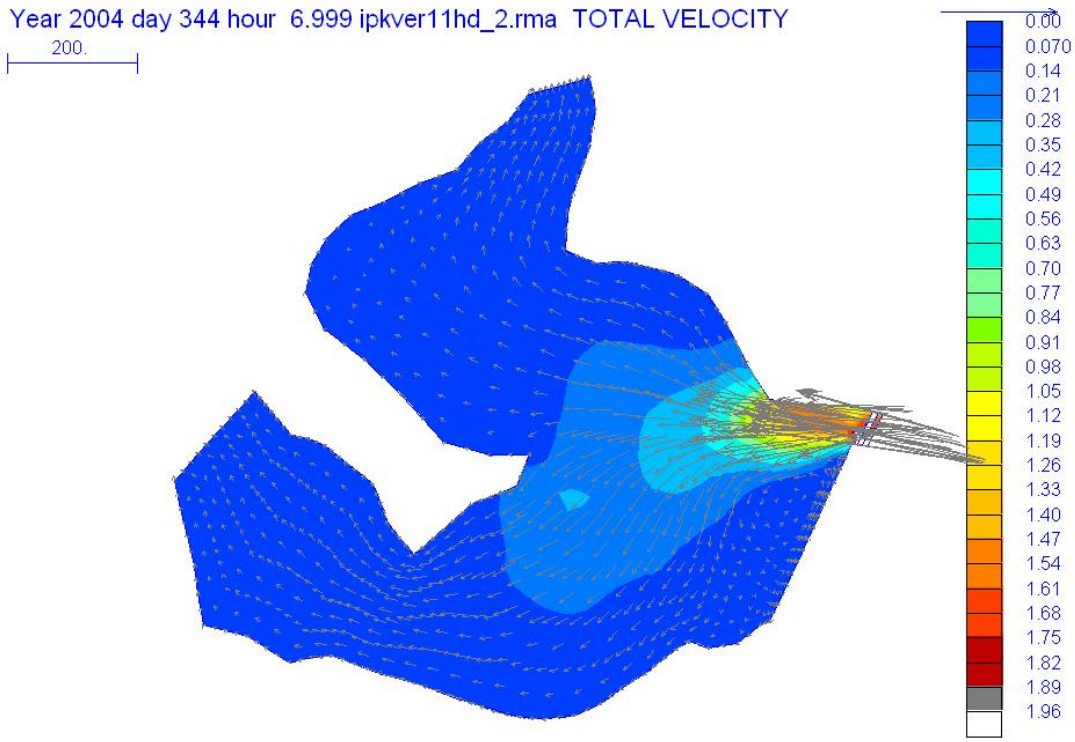


Figure 4.7: Simulated Velocity Distribution (m/s) during Flood Tide (30 min before HW)

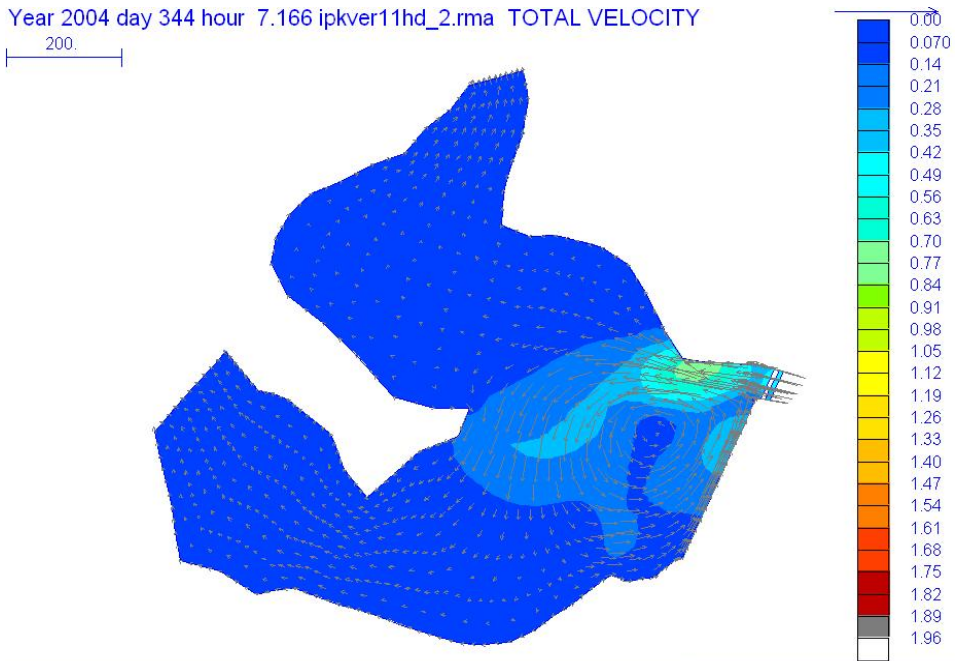


Figure 4.8: Simulated Velocity Distribution (m/s) during Flood Tide, 20 min before HW

Figure 4.9 is showing velocity distribution about 1 hour 40 minutes after HW when the elevation of WL is about 10 cm above the bridge sill level. Water starts to flow out of the lake at 9.10am (hour 9.165). It is clear there has been a reduction in water velocities in the lake by two hours after HW (hour 9.498) as water discharge from the lake slows (Figure 4.10).

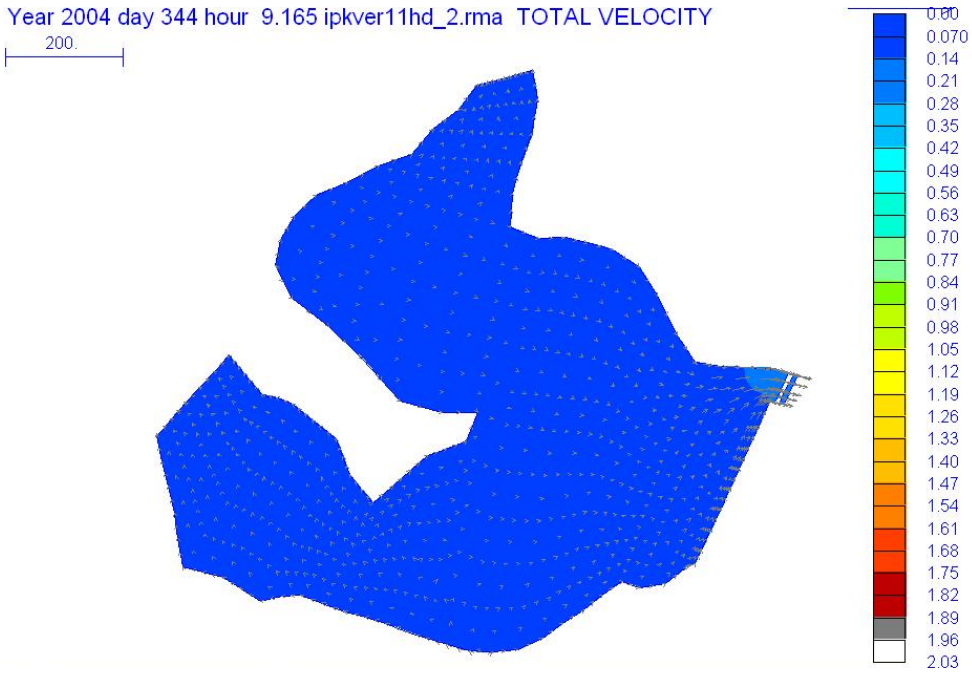


Figure 4.9: Simulated Velocity Distribution (m/s) during Ebb Tide, 1hr 40 min after HW

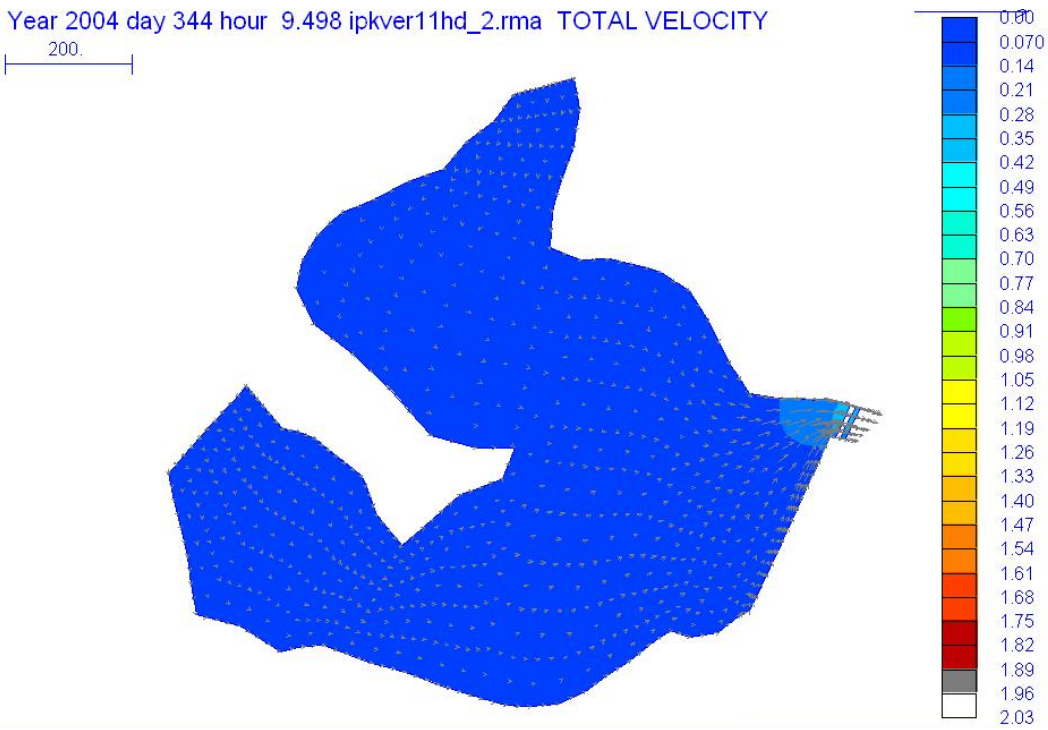


Figure 4.10: Simulated Velocity Distribution (m/s) during Ebb Tide, 2hrs after HW

The pattern of flow obtained from the model is in agreement with the field observations. Full agreement between field measurements and simulation was not achieved. There is still not enough full field data to make calibration more accurate. The main reason lay in the problems of accuracy in recording water levels, as well as in using the velocity-area method with limited numbers of measurement locations and with the low velocities being on the edge of the accuracy of reading of the current meter. Additionally, the densely growing mangroves that cover the banks of the creeks may have an influence on the results of the field data analysis. Given these limitations, this part of the modelling can be considered to provide a reasonable qualitative description of the hydrodynamics processes in the lake, with some cautions about the accuracy of the quantitative values.

4.5.4. Sediment Transport Model RMA 11 calibration

The velocities and water surface elevations provided by the RMA 2 model were used by the RMA11 model to calculate changes in bed elevations due to erosion, transportation, and deposition of sediment.

In order to obtain an indication of sediment transport processes within the entrance to Causeway Lake, a control volume was established and the net amount of accretion was calculated by comparing the Digital Elevation Models using Civil CAD 6.3 of 1986 and 2003 data. Based on bathymetry data, it was established that a net volume of approximately 2500m^3/year of sediment was deposited within the control volume of the lake during that period (Section 3.9.1).

The model was run for a variety of combinations of the input parameters and the net amount of accretion was calculated by the model, within the control volume and over a single tidal cycle. Using this procedure, it was determined that appropriate values for calibration were as follows:

- Coefficient of deposition (CLDE) = 1.0 (as recommended in RMA documentation)
- Coefficient of erosion (CLER) = 10.0 (as recommended in RMA documentation)
- Falling Velocity = 0.0012 m/s (based on sample analysis)
- Sediment concentration under bridge and at the Mulambin and Shoal Creeks profiles as discussed in Section 4.4.4.
- No vertical stratification in the lake waters
- Bottom roughness coefficient used in the model was Manning's coefficient *n*

Initially all four available formulas in the model were used to calculate sediment transport were tested:

1. The Ackers-White (1973)
2. The Van Rijn (1984)
3. The Brownlie (1981)
4. An updated Van Rijn method that allows for shear impacts of waves

From the bathymetry data analysis it was estimated that the average annual sedimentation rate of the lake is 2500 m^3. Other analyses (field data and historical rainfall data) gave an average annual sedimentation of the order of 2100 m^3, excluding wash load.

In the present calibration of the RMA 11 sedimentation model, the assumed tide outside the bridge was 4.28m LAT representing the situation during field testing for the period of one tide. The average annual conditions of the tide was next introduced as MHWS (4.11m LAT) and the model was run with the average sediment concentration for the period of one year having 198 tides exceeding the bridge sill level.

The obtained results with the different sediment transport formulae are:

- The Ackers-White: 3700 m^3/y.
- The Van Rijn: 2900 m^3/y.
- The Brownlie: 3100 m^3/y.
- An updated Van Rijn: 3700 m^3/y.

There are no significant differences between results when using any of these four formulas. For the further analysis, the Van Rijn formula was selected as his approach was also used in previous analysis of the field data.

The model presents final changes to the bathymetry, however to evaluate distribution of sediment from the sea side and from the catchments, the analysis of model output results were carried out with Van Rijn formula and obtained:

Sediment transport from the sea side: 1800 m^3/year

Sediment transport from catchments: 1100 m^3/year

Considering the previous discussion regarding possible errors in analysis, the results are in satisfactory agreement with the field data analysis. The factor of discrepancy < 2 is generally accepted in the literature for sediment transport quantitative analysis.

4.6. Simulation of Sedimentation in the Lake

Initially calibrated models RMA 2 and RMA 11 were used to simulate trends in the lake siltation for the following scenarios (Figure 4.11):

Scenario 1: Do nothing, it means leaving the lake without any human interventions

Scenario 2: Partial dredging in the central part of the lake including sand bar (28 ha)

Scenario 3: Partial dredging the branch of the lake next to Mulambin Creek (24 ha)

Scenario 4: Full dredging of the lake (66ha)

Based on existing bathymetry data, the evaluated volume of sediment that must be dredged to keep the minimum depth of the lake 1.5 m below the bridge sill level is:

- Central part of the lake, 28 ha area: 125.000 m^3
- Part next to Mulambin Creek, 24 ha area: 141.000 m^3
- Whole lake, 66 ha area: 340.000 m^3

Conservatively it is assumed that up to 20% of this bottom material is silt and clay fractions.

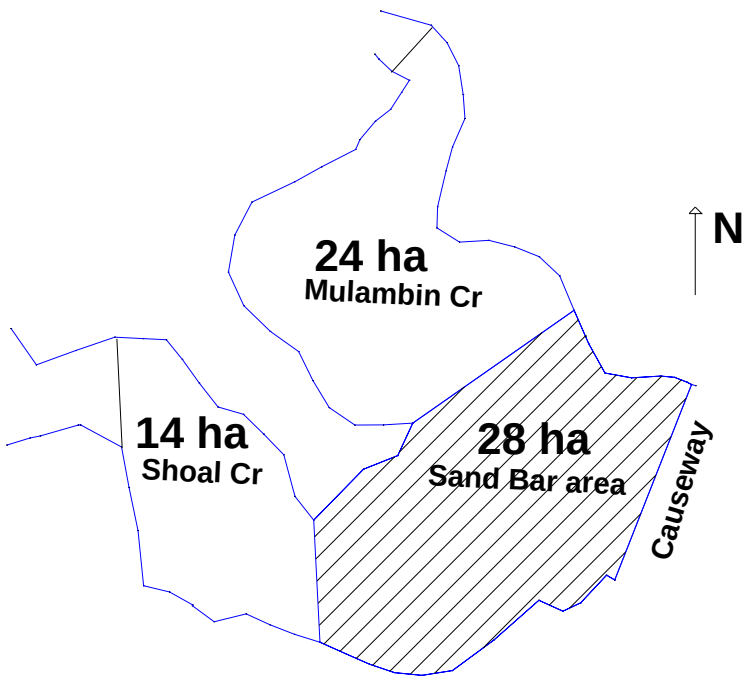


Figure 4.11: Suggested dredging areas

4.6.1. Scenario 1: Do Nothing

Figure 4.12 to Figure 4.14 show existing bathymetry in 2003 and the predicted changes after 5 and 10 years respectively. The vertical datum 0.0m in these Figures is at bridge sill level.

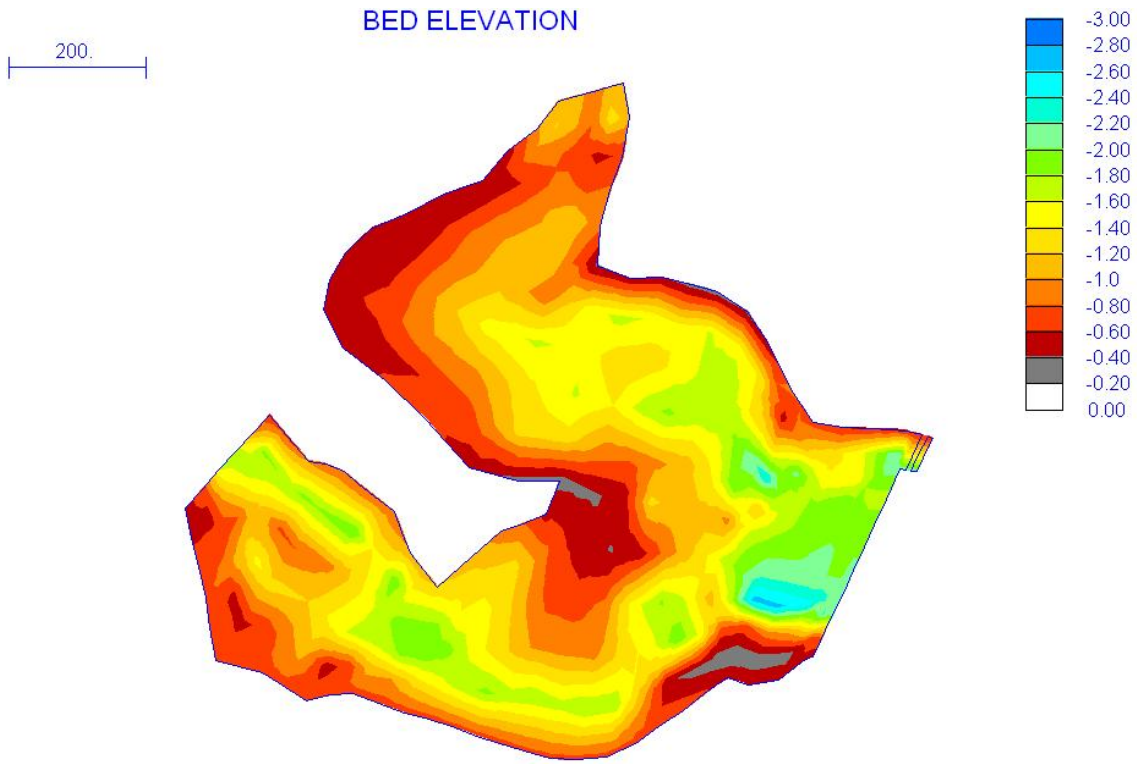


Figure 4.12: Existing Bathymetry of the Causeway Lake in 2003 - depths in metres (Scenario 1)

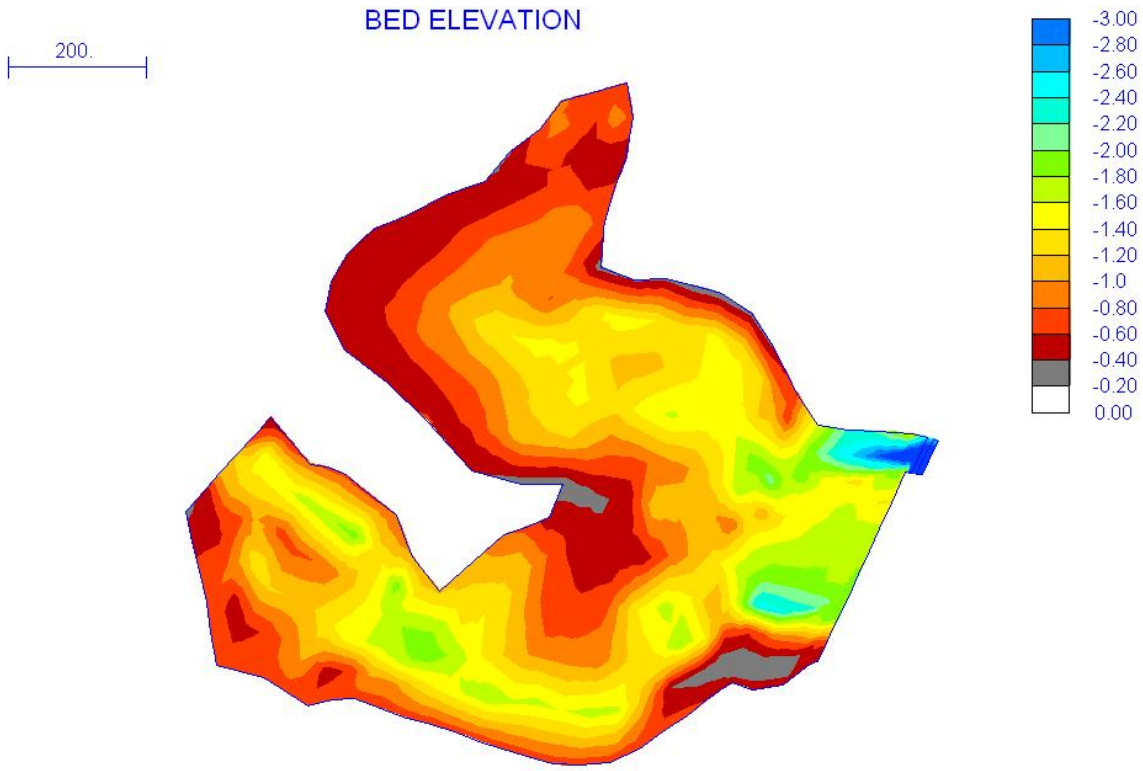


Figure 4.13: Simulated Bathymetry of the lake after 5 years - depths in metres (Scenario 1)

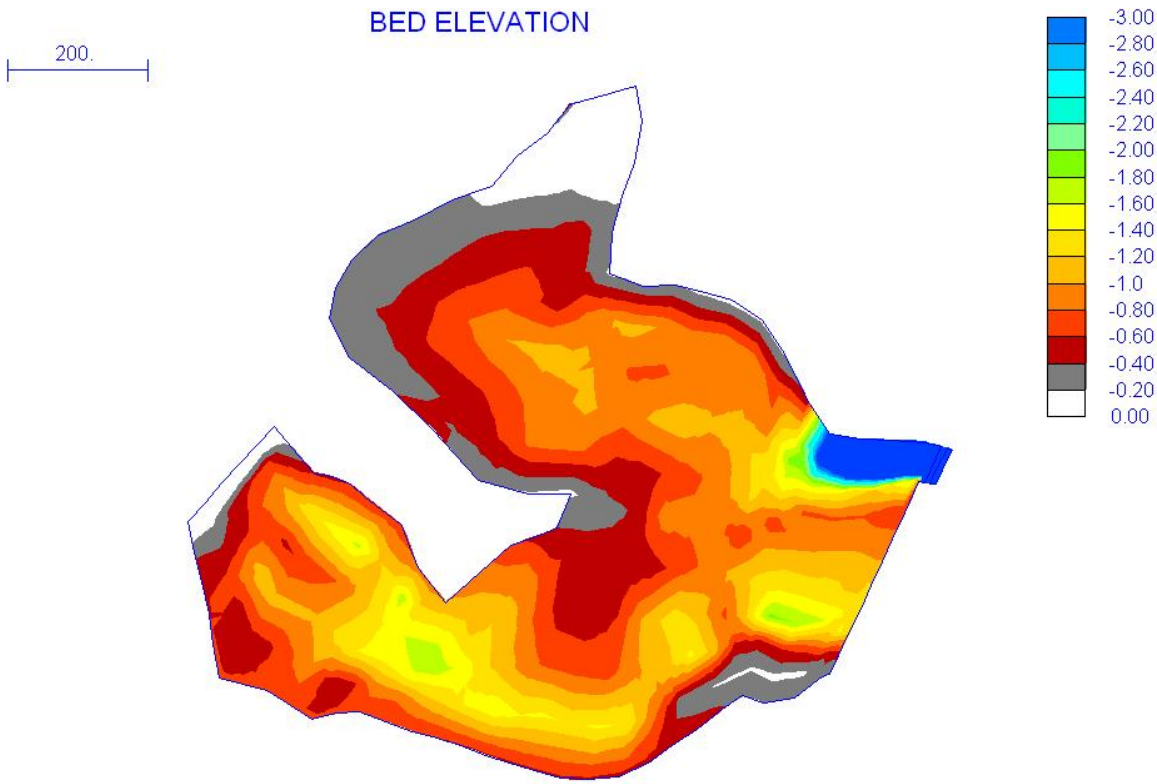


Figure 4.14: Simulated Bathymetry of the lake after 10 years - depths in metres (Scenario 1)

Analysis of the bottom changes in the section of the lake indicate progressive shallowing of the whole lake. The figures in *Table 4.6* are the average depth in particular sections of the lake below the bridge sill level. On average the whole lake will be progressively shallowed. This process of shallowing may have significant influences on the whole life of the lake with significant potential of its eutrophication particularly near Mulambin Creek. It may go as far as changing this part of the lake into marshes

	Year 0		Year 5		Year 10	
Scenario 1	[m]	[%]	[m]	[%]	[m]	[%]
Shoal	-1.34	100	-1.27	95	-1.06	79
Central	-1.31	100	-1.22	92	-0.95	72
Mulambin	-0.98	100	-0.86	88	-0.50	51

Table 4.6: Average depths reduction with time for Scenario 1: Do Nothing (depth measured below the bridge sill level and % is of the initial depth in Year 0)

4.6.2. Scenario 2: Dredging the central part of the lake

The bathymetry after dredging of 125,000 m^3 of bottom material in the central part of the lake is shown in Figure 4.15. Practically dominant part of the lake will have depth over 1.4m below the bridge sill level. Simulations of the lake behaviour 5 and 10 years after dredging are shown in Figure 4.16 and Figure 4.17.

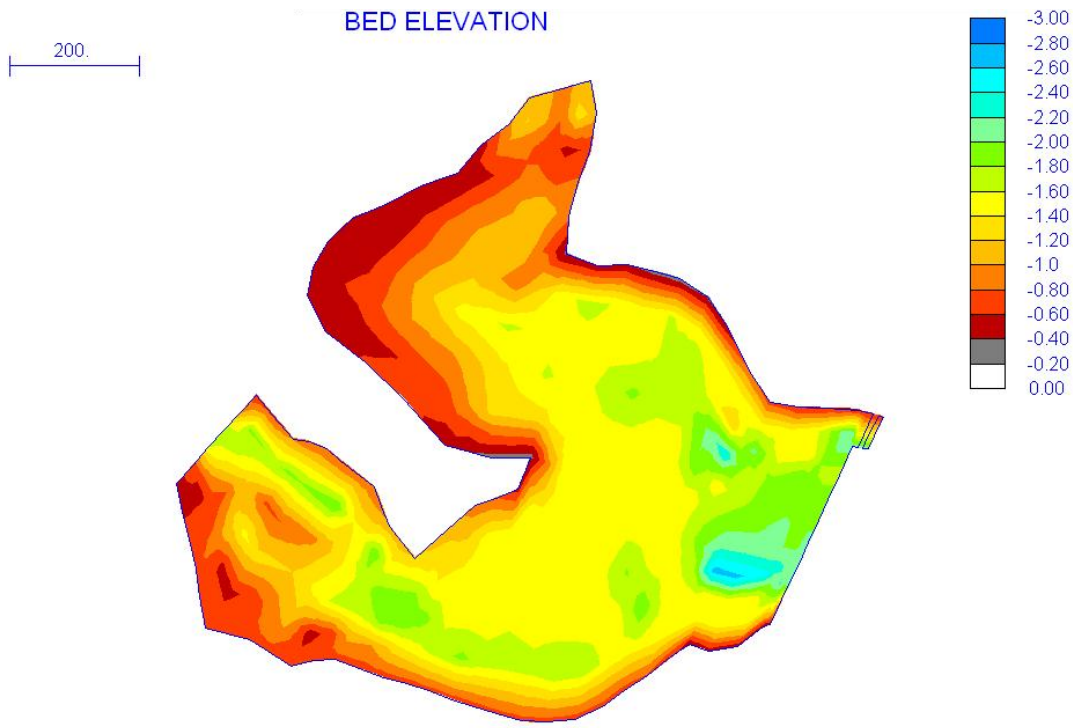


Figure 4.15: Bathymetry after dredging central part of the lake - depths in metres (Scenario 2)

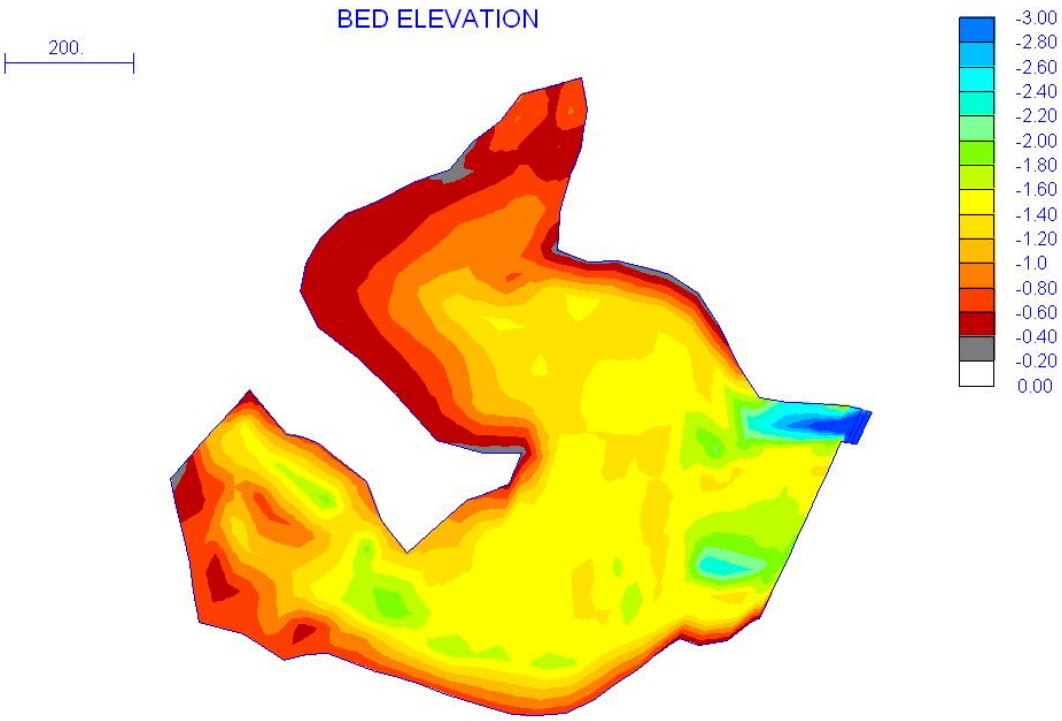


Figure 4.16: Simulated Bathymetry of the lake after 5 years - depths in metres (Scenario2)

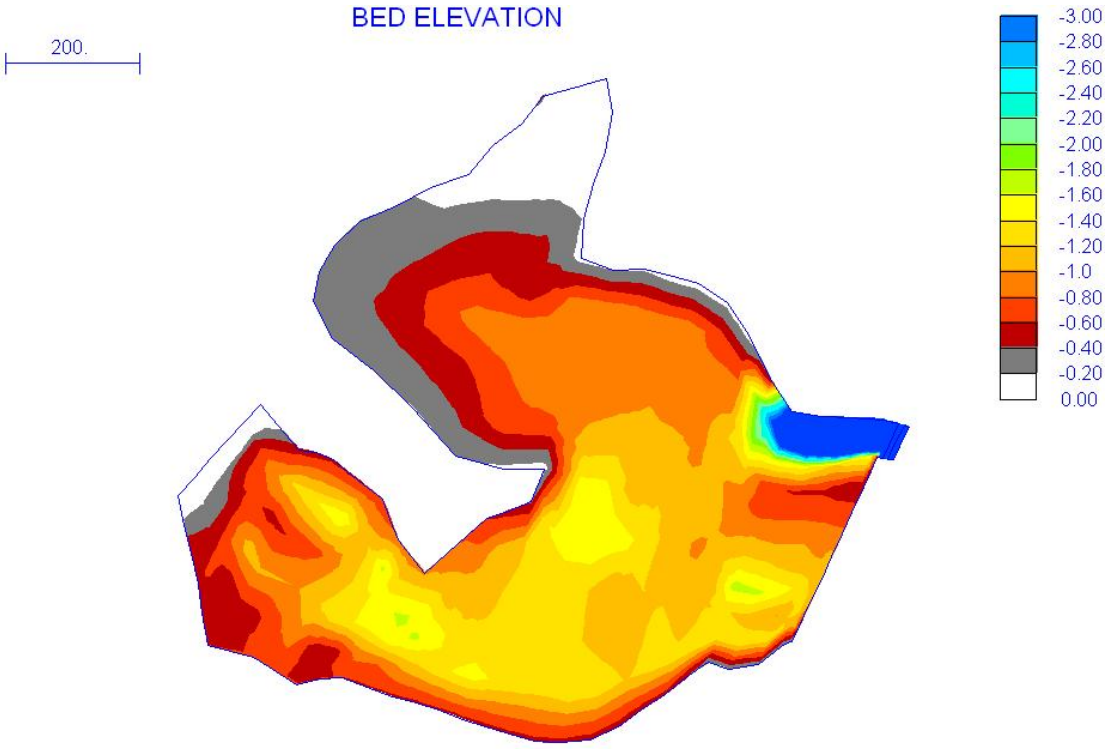


Figure 4.17: Simulated Bathymetry of the lake after 10 years - depths in metres (Scenario2)

Predicted siltation of the lake is shown in *Table 4.7*. The lake retains a reasonable depth even after 10 years.

	Year 0		Year 5		Year 10	
Scenario 2	[m]	[%]	[m]	[%]	[m]	[%]
Shoal	-1.39	100	-1.31	94	-1.05	74
Central	-1.63	100	-1.54	94	-1.27	78
Mulambin	-0.98	100	-0.85	87	-0.40	41

Table 4.7: Average depths reduction with time for Scenario 2: Central part dredged (depth measured below the bridge sill level and % is percentage of the initial depth in Year 0)

4.6.3. Scenario 3: Partial dredging the branch of the lake next to Mulambin Creek

The other scenario considers the option of dredging the Mulambin branch of the lake which is currently significantly silted. After dredging 141,000 m^3 from this area, the bathymetry of the lake is shown in Figure 4.18. Simulations of the lake behaviour 5 and 10 years after dredging are shown in Figure 4.19 and Figure 4.20.

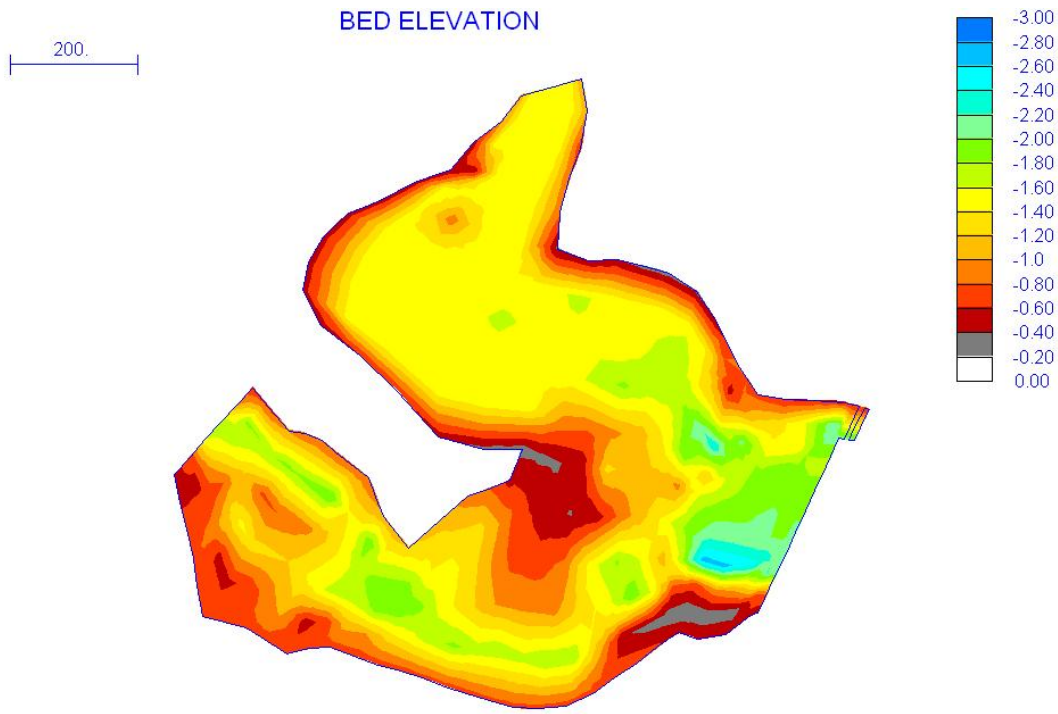


Figure 4.18: Bathymetry of the lake after dredging Mulambin branch of the lake - depths in metres (Scenario 3)

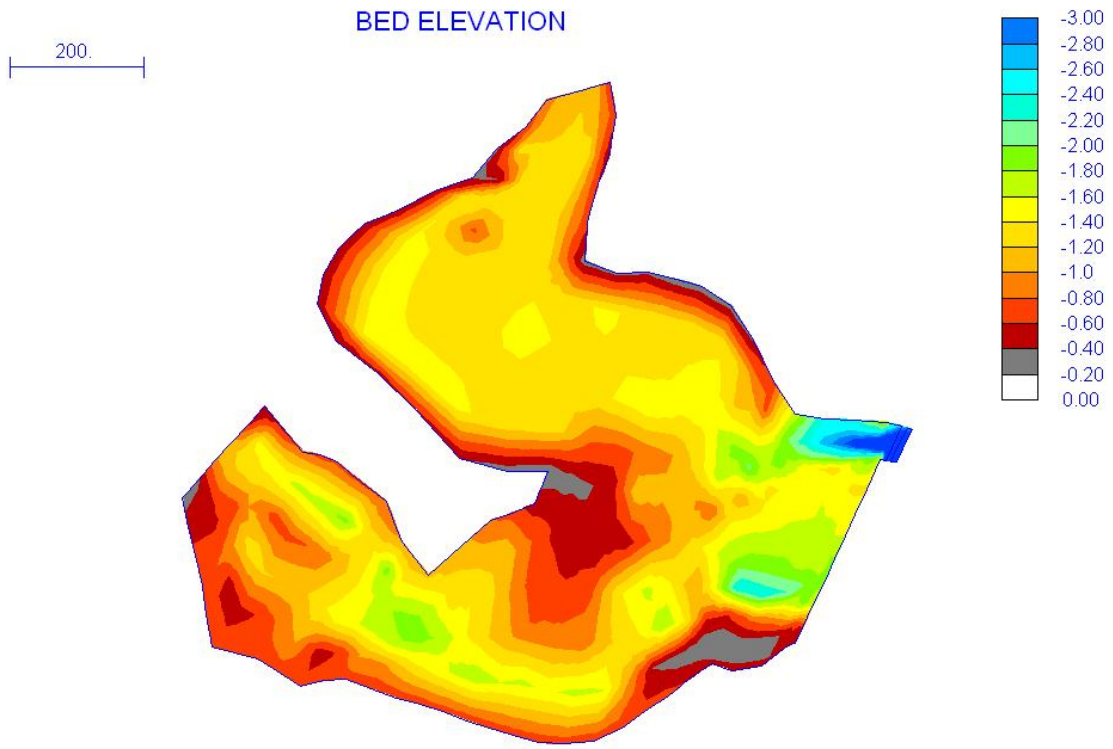


Figure 4.19: Simulated Bathymetry of the lake after 5 years - depths in metres (Scenario3)

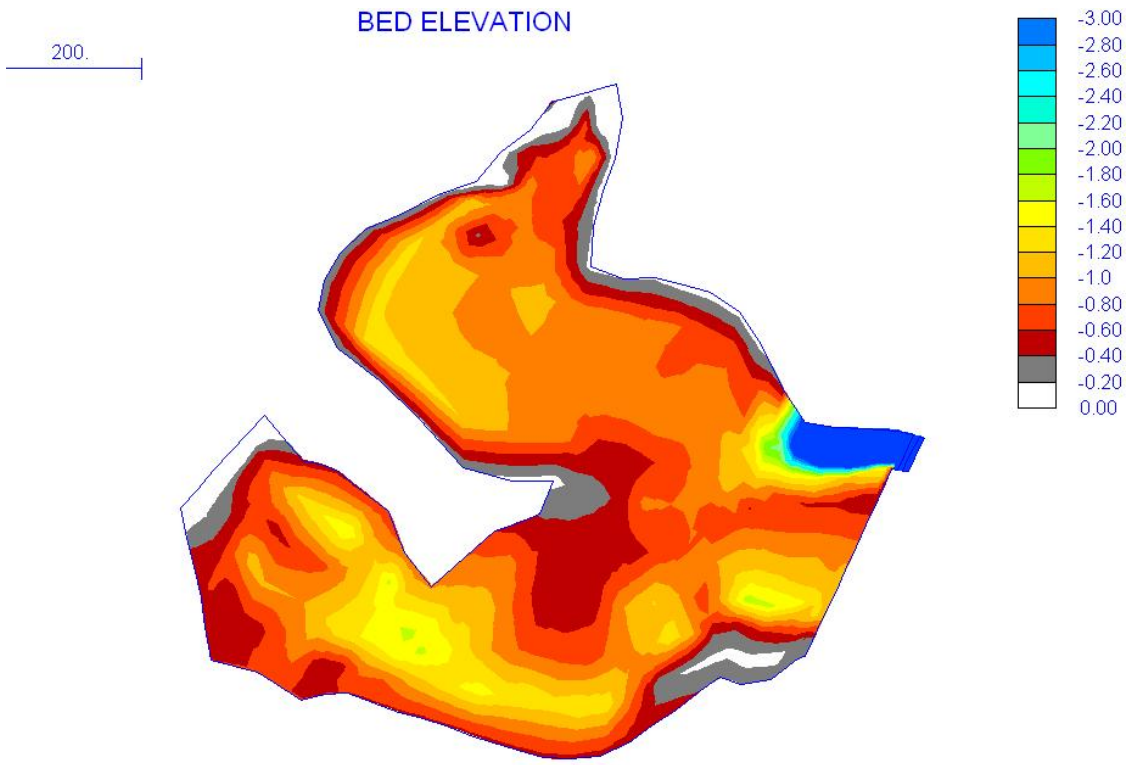


Figure 4.20: Simulated Bathymetry of the lake after 10 years - depths in metres (Scenario3)

Simulated siltation in the lake for this scenario following dredging near the Mulambin Creek predict changes in the bathymetry of the lake as shown in *Table 4.8*.

	Year 0		Year 5		Year 10	
Scenario 3	[m]	[%]	[m]	[%]	[m]	[%]
Shoal	-1.34	100	-1.26	94	-1.01	75
Central	-1.33	100	-1.25	94	-0.97	73
Mulambin	-1.50	100	-1.37	91	-0.93	62

Table 4.8: Average depths reduction with time for Scenario 3: Mulambin side dredged (depth measured below the bridge sill level and % is percentage of the initial depth in Year 0)

4.6.4. Scenario 4: Full dredging of the lake

The earlier scenarios considered options of dredging various parts of the lake. This scenario considers the effect of dredging the whole lake to the minimum depth of 1.5 m below the bridge sill level. After dredging 340,000 m^3 the bathymetry of the lake is shown in Figure 4.21. Simulations of the lake behaviour 5 and 10 years after dredging are shown in Figure 4.22 and Figure 4.23.

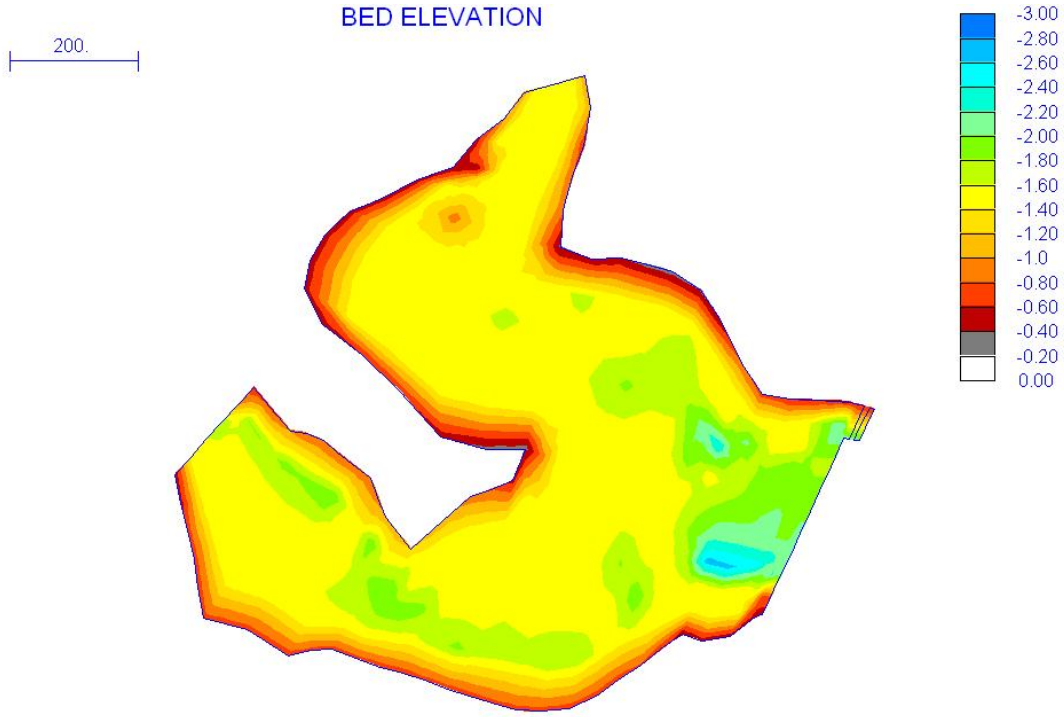


Figure 4.21: Bathymetry of the lake after dredging of the whole lake - depths in metres (Scenario 4)

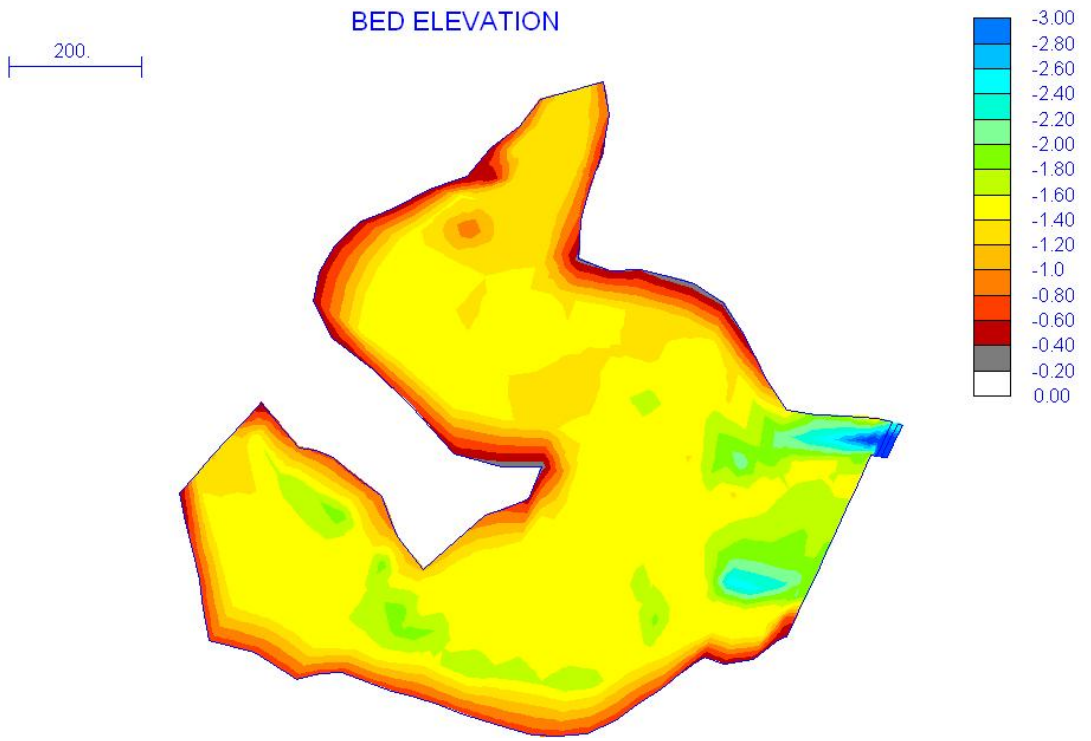


Figure 4.22: Simulated Bathymetry of the lake after 5 years - depths in metres (Scenario 4)

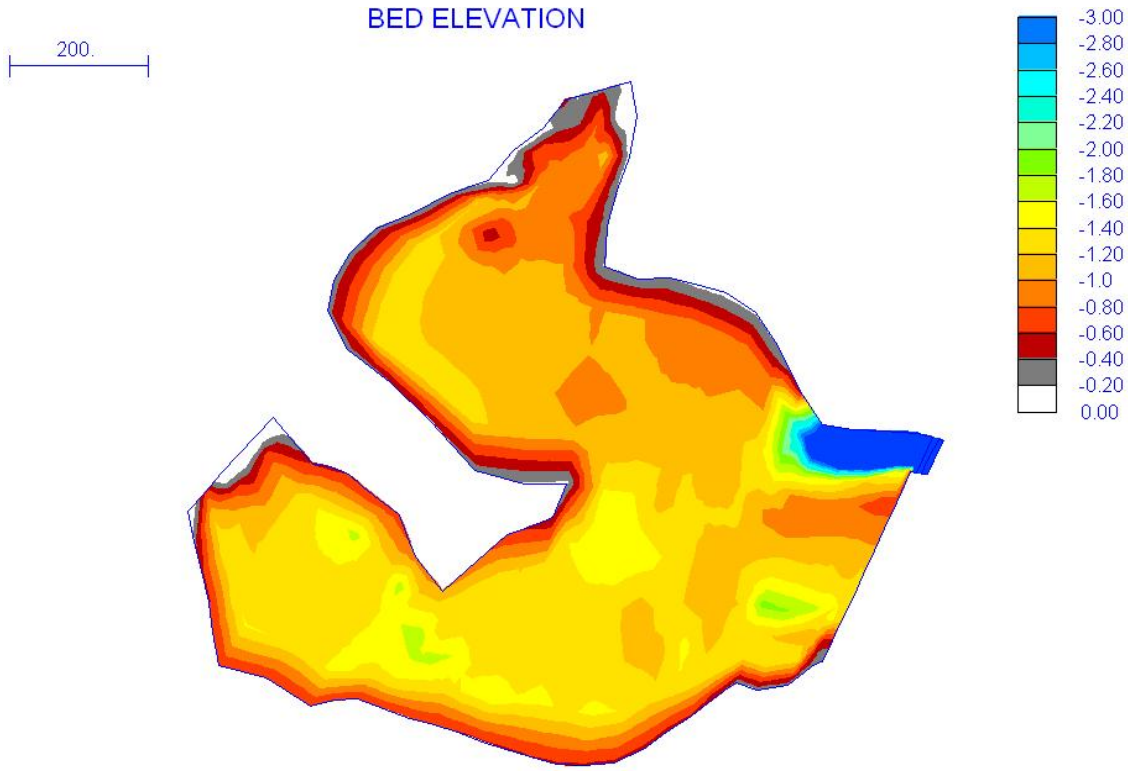


Figure 4.23: Simulated Bathymetry of the lake after 10 years - depths in metres (Scenario 4)

Predicted siltation of the lake is shown in *Table 4.9*. The lake retains a reasonable depth even after 10 years.

	Year 0		Year 5		Year 10	
Scenario 4	[m]	[%]	[m]	[%]	[m]	[%]
Shoal	-1.60	100	-1.52	95	-1.27	86
Central	-1.65	100	-1.57	95	-1.28	78
Mulambin	-1.50	100	-1.37	91	-0.93	62

Table 4.9: Average depths reduction with time for Scenario 4: Whole lake dredged (depth measured below the bridge sill level and % is percentage of the initial depth in Year 0)

4.6.5. Scenarios Analysis after 10 years

Scenario 1 assumed leaving the lake as it is. No significant changes are expected in the Shoal Creek part. In the central part, depth may reduce by up to 30%. The Mulambin Creek part will be very significantly silted with depth reducing by up to 50%, this may be considered as the effective death of this part of the lake, and may change it into marshes.

The only alternative for such situation will be dredging the lake. Some initial analysis of the bottom material discussed in Section 3.9.2 showed that the level of pH is around 8 indicating that the bottom material is free of acid. More detailed research analysis may be recommended in relation to checking if heavy metal elements are present. In analyses assumed dredging to the depth 1.5 m below the bridge sill level which generally indicate lowest water level in the lake.

Scenario 2 analysed the effect of partial dredging the central part of the lake, removing a total about 125,000 m^3 of sediment. The Shoal Creek part of the lake will behave as in scenario 1 without any significant changes. Mulambin Creek part will be worse than in scenario 1 and depth will be reduced by up to 60 %. It is expected severe siltation will occur in this part of the lake.

Scenario 3 analysed the effect of partial dredging the Mulambin Creek part of the lake, removing a total about 141,000 m^3 of sediment. Shoal Creek part of the lake as in previous scenarios will not have any significant changes. In the central part, depth may be reduced by up to 25%. Mulambin Creek part may be silted up to 40%, but still having a reasonable average depth of about 1 metre.

Scenario 4 assumed full dredging of the lake to a depth 1.5 m below the bridge sill level, removing a total of 340,000 m^3 of sediment. The results of simulation indicate that Shoal Creek part will be silted up by about 15%, central part by up to 20% and Mulambin Creek part by up to 40% of the dredged depth.

For the dredging scenarios it seems that partial dredging of the Mulambin Creek part of the lake seems more economical as the expected average reduction in the depth after 10 years is very similar to scenario 4 which requires full dredging of the lake.

4.7. Numerical Model Findings

Causeway Lake is a natural sink for sediment transported from the catchments as well as from the sea during tidal floods exceeding the bridge sill level. After 67 years since it was artificially created, the lake is close to losing its usefulness for several purposes such as tourist attractiveness, fishing and yachting. This section analysed hydrodynamics and sedimentation processes in the lake based on measurements and numerical modelling, and recommended dredging to the depth of 1.5 m below the bridge sill level at least in the Mulambin Creek part of the lake (scenario 3).

However before the final decision will be taken the following recommendations are suggested:

1. The present study includes only sieve analyses and pH level of the bottom material. It has shown that the pH level is around 8 indicating that the bottom material is free of acid. However, in the Mulambin Creek part of the lake, wash load (including silt and clay fractions) is up to 25%. Therefore more detailed bottom material quality analyses are recommended before using this material for any purposes, particularly to verify existence and level of heavy metals.
2. Before any dredging work started, acceptance must be obtained from the Environmental Protection Agency regarding sea grass and mangroves protection acts.

5. CONCLUSION

The goal of the research was to understand the Hydrodynamic and Sedimentation processes of the Causeway Lake, Queensland. Causeway Lake is an artificial lake created in 1939 by closing 80% the Mulambin Creek estuary entrance. Ever since, the lake has been a sink (reservoir-like) for sediment material transported from the sea and catchments.

To identify the cause of the sedimentation problem, we have to understand the hydrodynamic process in the estuary, tidal processes, discharge from catchments and the characteristics of catchments.

Tidal hydrodynamics involve tidal ranges, tidal current and tidal prism. Tidal ranges in Causeway Lake were up to 5.1 m LAT prior to the construction. After closing part of the estuary, it reduced to up to 1.2m LAT maximum inside the lake.

Tidal sedimentation was evaluated based on sediment concentration (Section 3.8.1 and 3.10.1) and tidal prism entering and leaving the lake.

The Tidal prism is the volume of water entering and leaving the lake and was evaluated based on field measurement. Based on measured Tidal Prism entering and leaving the lake, Equation 2.16 and Equation 2.17 were described.

Combining measured sediment concentration equations (Equation 3.17 and 3.18) and tidal prism equations described above allowed the prediction of average annual sedimentation in the lake caused by tidal action (Equation 3.19). It allows the

evaluation of sedimentation for average conditions (no heavy storms). Based on this analysis, 1050 m^3/year was found to be coming into the lake from the sea side.

The other source of sedimentation is catchments. Analysis of sediment transport rate and probable discharges from catchments was undertaken based on the Rational Method and Manning formula (Equation 2.13). Creek discharges are non-uniform due to tidal influence, therefore the analysis must be conducted in a very simplified way to achieve average quantitative values. Measurement confirmed the order of predicted discharges. Having knowledge about discharges, Van Rijn's sediment transport formula was implemented. It is an arguable assumption that discharge from the creeks of over 20 m^3/s brings significant sediment. Assumed discharges were based on analysis and visual field observation of low sediment concentration. Having discharge and combining them with historical rainfall data, it was possible to conclude the annual sediment transported from the catchments of 1100 m^3/y

In total, it is about 2150 m^3/year of sediment is transported into the lake (wash load not included).

The verification of this analysis was bathymetry of the lake taken in 1986 by Main Roads Department and in 2003 by the author of the thesis. It shows about 2500 m^3/year of average annual sedimentation. Expected error of field measurement is about 20%, giving a possible range of 2000 – 3000 m^3/year.

Analysis of different numerical models: DELTF3D, MIKE21 and RMA2 were conducted. The RMA model was selected based primarily on cost and it required

additional effort to study how to run the software. Field measurement data from 10 December 2004 were next implemented into this numercial model.

Limited model calibration was conducted until obtaining acceptable qualitative (not quantitative) results. It was found that the model is very sensitive to lake condition characteristics. Slight changes in Manning's coefficients, turbulence friction coefficients and Peclet factors affect the final result significantly (Section 4.5.3)

Sediment transport in the Causeway Lake based on output from RMA is in the order of 2900 m^3/year (Van Rijn Equation) to 3700 m3/year (Ackers-White Equation).

RMA was also used to predict the sedimentation and lake bathymetry changes over the next 10 years (Section 4.6). Four scenarios were analysed, however, the scenario connected with dredging of the Mulambin Creek part of the lake is the most efficient as its give similar results to dredging the whole lake.

Estuary sedimentation cannot be prevented, but it may be retarded. One way of doing this is to prepare a locally suited catchment management plan including:

1. Use of soil-conservation methods within the catchment area. Terraces, strip cropping, contour ploughing, and similar techniques retard overland flow and reduce erosion.
2. Examine the possibility of construct wetlands in the upper parts of the Mulambin and Shoal Creeks. Constructed wetlands have broad applications, having been employed, for example, to treat acid mine drainage, landfill leachate, urban runoff. Wetlands are usually effective for heavy metal

removal, making them ideally suited for the treatment of highway runoff, which often contains elevated levels of lead, copper, and zinc.

Sediment accumulation in the lake may also be reduced by providing a means of reducing flood tides entering the lake. On average, 198 tides/year enter the lake. Increase of the bridge sill level by up to 0.20 m may have significant influence on the total volume of sediment transported from the sea. This option requires detailed engineering analysis considering potential problems from permanent raising of the lake water level by 0.20m.

Physical removal of sediment deposits is rarely feasible. Sluice gates near the base of the causeway may permit flushing some sediment downstream, but the removal will not extend far upstream from the causeway.

Finally the thesis gave the opportunity to gain basic fundamental knowledge about tidal hydrodynamic, methodology of field measurement and analysis of unsteady flow, methodologies of analysing sedimentation processes. Experience in selection process of numerical models, data preparation, calibration, verification and implementation for the future sedimentation process in the Causeway Lake were also gained.

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APPENDICES

A.1. Bathymetry data (File name: bathymetry.doc)

A.2. Rainfall Intensity data (File name: rainfall.doc)

A.3. Field data and calculation 10 December 2003 (File name: 10dec03data.doc)

A.4. Field data and calculation 22 January 2004 (File name: 22jan04data.doc)

A.5. Field data and calculation 1-2 June 2004 (File name: 1june04data.doc)

A.6. Field data and calculation 10 December 2004 (File name: 10dec04data.doc)

A.7. Numerical Model (RMA) Input Data (File name: rmainput.doc).

Note: Appendices data are available from attached CD ROM Media.