

4. HAZARD MANAGEMENT

4.1 DGR Item References

The following DGR items are addressed in this section:

RAINBOW BEACH CONCEPT PLAN (CP)

Item	Topic
CP4.3	Identify the presence and extent of acid sulphate soils on the site and if necessary, appropriate mitigation measures.
CP4.4	Provide an assessment of any flood risk on site in consideration of any relevant provisions of the NSW Floodplain Management Manual. Consider the potential impacts of any filling on the flood regime of the site and adjacent land.

PROJECT APPLICATION – OPEN SPACE CORRIDOR & CONSTRUCTED WETLAND (PA)

Item	Topic
PA5.3	Provide an assessment of any flood risk on site in consideration of any relevant provisions of the NSW Floodplain Development Manual (2005). Consider the potential impacts of any filling on the flood regime of the site and adjacent lands.
PA6.3	Identify the presence and extent of acid sulphate soils on the site and if necessary, appropriate mitigation measures.

4.2 Potential Development Impacts

Potential hazards which have been raised in the DGR documents and which are investigated in this section of the report include:

- Flooding and the requirements of the NSW Floodplain Management Manual
- Management of acid sulfate soils

With regard to flooding risk, it should be noted that the Section 149 Planning Certificate for the site provided by Port Macquarie Hastings Council states that the site is not “flood-prone”.

4.3 Major Storm Runoff Events

4.3.1 Introduction

This study has been carried out to determine the impact of the proposed development on the flood behaviour of Duchess Gully. Figure 1 is a locality plan showing the approximate site boundaries.

Urban development tends to increase the magnitude of runoff, and reduce the response time. This study quantifies the effects of urbanisation of part of the catchment of Duchess Gully, and investigates the mitigating effect of the creation of a constructed wetland.

4.3.2 Catchment Description

The catchment area of Duchess Gully is 782 hectares. The boundaries of the catchment are shown in Figure 25.

The western boundary of the catchment is formed by a ridge approximately 2 km west of Ocean Drive with elevations varying from about 80 metres to 200 metres AHD. The drainage is fairly well defined west of Ocean Drive, but poorly defined east of Ocean Drive. A number of shallow drains have been constructed east of Ocean Drive to drain this area.

West of Ocean Drive, the catchment is generally timbered and uncleared, except for a rural residential subdivision which occupies around 33 hectares. East of Ocean Drive, the catchment area is generally cleared. There are some areas of residential subdivision near the southern boundary of the catchment east of Ocean Drive. The development is evident in the aerial view Figure 3.

4.3.3 Development Scenarios

This assessment has considered the catchment in three conditions of development as follows:

- **Natural State** – no development
- **Existing Development**, including rural residential subdivision west of Ocean Drive (about 33 hectares), residential development to south of the site, but within the catchment area of Duchess Gully (about 28 hectares), and the existing lagoon (about 6.8 hectares in area).
- **Proposed Development** of the site. As above, but including an additional 89.5 hectares of residential development, with an additional constructed wetland of 12.7 hectares in area. The extent of the proposed residential subdivision is shown on the catchment subarea plan Figure 25.

4.3.4 Methodology

Design storm events have been estimated for the site by the design storm-loss rate-runoff routing method. The Watershed Bounded Network Model (WBNM) has been used. This model was used in previous drainage studies of the site, and has been used for this flood study to maintain consistency with previous studies. 5-year, 10-year, 20-year, and 100-year ARI event hydrographs were estimated for a range of storm durations, and catchment development conditions. A runoff-routing model was used so that complete hydrographs for the whole storm event could be obtained for use as input to a hydrodynamic model (based on MIKE11) of the Duchess Gully watercourse.

The WBNM was “calibrated” against peak discharges estimated by the Probabilistic Rational Method. The PRM method is the most appropriate to use in these circumstances because:

- There are no actual recorded storm events for this catchment, which could be used for model calibration.
- Regional parameters for the probabilistic rational method have been developed using a significant amount of recorded storm data. Regional parameters are available for this area (EngAust 1987).

The hydrographs derived from the WBNM were used as input to a hydrodynamic model of the Duchess Gully and its tributaries in the reach downstream of Ocean Drive extending to the ocean outlet. A hydrodynamic model rather than a steady state backwater model was considered preferable for the hydraulic modelling so as to be able to model explicitly changes in the low-lying area storage, and the creation of additional water bodies.

The MIKE-11 model was used for the hydraulic modelling. This model was used to estimate the peak flood levels, the peak velocities along Duchess Gully for the 1% AEP storm event, and the changes to these levels and velocities resulting from the proposed catchment development including the construction of an additional lake.

A number of lake configurations were modelled in order to find a combination which would result in no worsening of the peak discharges, levels, and velocities along Duchess Gully downstream of the existing lake.

4.3.5 Hydrology Modelling and Results

4.3.5.1 Probabilistic Rational Method

The probabilistic rational method (PRM) from Australian Rainfall and Runoff 1987 edition (AR&R) (EngAust 1987) was used to determine the peak discharge at the outlet of Duchess Gully for the 1% AEP storm event for the catchment in its natural (pre-development) state. The probabilistic rational method is the method recommended in Australian Rainfall and Runoff 1987 edition for estimating peak flows in small to medium size rural catchments in eastern New South Wales. The method is well-supported by recorded flow data from over three hundred streams in the eastern NSW region. AR&R states: "the only methods that can really be considered as approaching a satisfactory status are those based on observed flood data in the region of interest". The external catchments at the site do not have any unusual hydraulic characteristics and are typical of catchments from which calibration data was drawn for the PRM.

The following values in Table 35 were determined by applying this method.

Table 35 Catchment Runoff – Probabilistic Rational Method PRM)

Parameter	
Catchment Area	782 hectares at Duchess Gully ocean outlet
Time of Concentration	1.68 hours
C100	0.39
Peak Discharge for 1% AEP event	63.3 m3/sec (undeveloped catchment)

This estimate agrees with the probabilistic rational method estimate for the catchment as reported in the SMC December 1994 report (SMC 1994).

The probabilistic rational method was also used to estimate the 1% AEP peak discharge at the Ocean Drive southern culverts (Node 8 NODE8) in Figure 25. A peak discharge of 29.5 m3/sec was estimated at this point for the natural catchment.

The Cordery-Webb unit-hydrograph method had been applied to the catchment to determine design discharges for an earlier study (GMA Feb 1983). The peak discharge for the ocean outlet from the natural catchment had been estimated to be 62.3 m³/sec for the critical 6 hour storm duration. This estimate is consistent with the probabilistic rational method estimate, although it should be noted that design rainfall information would have changed since the time of application of the Cordery-Webb procedure.

4.3.6 Watershed Bounded Network Model (WBNM)

A watershed bounded network model was set up for the Duchess Gully catchment. The subdivision of the catchment into subareas was the same as the subdivision used for the SMC December 1994 study (SMC 1994). The subareas are shown in Figure 25.

Table 36 lists the subcatchment subareas.

Table 36 WBNM Catchment Model - Subareas

Subarea ID		Area (ha)
No	Name	
1	SUB1	73.36
2	SUB2	68.83
3	SUB3	21.17
4	SUB4	56.88
5	DNODE5	0
6	SUB6	19.69
7	SUB7	54.92
8	DNODE8	0
9	SUB9	50.16
10	SUB10	94.84
11	SUB11	63.13
12	SUB12	78.44
13	SUB13	53.28
14	SUB14	27.27
16	DNODE16	0
15	SUB15	45.16
17	LAKE17	0
18	SUB18	73.36
19	SUB19	25.63

4.3.6.1 Water Bodies

The existing lagoon was included in the WBNM model for the existing and future development scenarios. This lagoon is 6.3 hectares in area. The relationship between storage elevation, spillway discharge, and storage volume was obtained from the SMC December 1994 Report Appendix D.

4.3.6.2 WBNM Model Calibration

To “calibrate” the model to the 1% AEP probabilistic rational method estimate, it was necessary to adjust the lag coefficient “c” to a value of 1.95, and to specify linear routing for the full range of discharges. With the above values, the model predicted the discharges as shown in Table 37.

Table 37 Comparison of PRM and WBNM Model – Calculated Discharges

Location	Prob. Rational Method	WBNM	
	Peak Discharge (m ³ /sec)	Peak Discharge (m ³ /sec)	Critical Duration (hrs)
Ocean Drive (DNODE8)	29.5	31.1	2
Ocean Outlet (SUB19)	63.3	63.4	9

The “calibrated” WBNM was modified to account for both existing development and the possible future development of the site.

The existing urban and rural residential areas are shown on Figure 25. This figure also shows the areas which are planned to be developed as residential areas in the future.

The manner in which the lag parameter for overland flow, stream lag factor and losses were modified to account for urbanisation is as follows:

4.3.6.3 Lag Parameter for Overland Flow

The lag parameter for overland flow was reduced to account for urbanisation using the following relationship:

$$L = L_{nat} (1 + U)^{-2}$$

Where

- L = Lag parameter for developed catchment
- L_{nat} = Lag parameter for undeveloped catchment
- U = Fraction of catchment urbanised.

This relationship originates from the work of Aitken (1975), and is used in other runoff routing models such as URBS (BCC 1993) to account for the decrease in subcatchment runoff delay time caused by urbanisation. The relationship leads to the result that the lag in overland flow for a fully urbanised catchment is one-quarter of the lag of an equivalent urban catchment.

For determining the change in the lag parameter for the rural residential part of the catchment, it was assumed that the rural residential area was equivalent to 1/3 of a similar urban area in terms of its effect on the lag parameter for overland flow. This assumption was used in previous hydraulic studies for this catchment (SMC 1994) and is reasonable considering the lower density of rural residential subdivisions when compared with medium density residential subdivisions.

4.3.6.4 Stream Lag Factor

In the WBNM, changing the lag parameter for overland flow also changes the stream channel lag correspondingly. With the proposed development, it is intended to maintain a drainage corridor of approximately 60 metres width with a low flow channel similar to a natural channel. The stream channel lag would not be expected to change significantly with the proposed development. Therefore, to maintain the same stream channel lag before and after development, the stream lag factor was adjusted to compensate for any changes resulting from changes in the overland flow lag parameter.

The lag coefficients and the stream lag factors for the WBNM are shown in Table 38.

Table 38 WBNM Model Subarea Continuing Losses – 1% AEP Event

SUBAREA	Natural		Existing					Future Development Scenario		
	AREA	Lag Coeff c	AREA	Area Rural Res	Area Urban	Mod Lag Coeff (Note1)	Compens Adjust to Stream lag factor	Area to be urbanised	Modified Lag Coeff (Note 2)	Compens Adjust to Stream lag factor
	(ha)		(ha)	(ha)				(ha)		
SUB1	73.36	1.95	73.36	0.00	0.00	1.95	1.00	0.00	1.95	1.00
SUB2	68.83	1.95	68.83	0.00	0.00	1.95	1.00	0.00	1.95	1.00
SUB3	21.17	1.95	21.17	0.00	0.00	1.95	1.00	0.00	1.95	1.00
SUB4	56.88	1.95	56.88	0.00	0.00	1.95	1.00	0.00	1.95	1.00
DNODE5	0	1.95	0	0.00	0.00	1.95	1.00	0.00	1.95	1.00
SUB6	19.69	1.95	19.69	0.00	0.00	1.95	1.00	0.00	1.95	1.00
SUB7	54.92	1.95	54.92	21.17	0.00	1.53	1.27	0.00	1.53	1.27
DNODE8	0	1.95	0	0.00	0.00	1.95	1.00	0.00	1.95	1.00
SUB9	50.16	1.95	50.16	0.00	9.10	1.40	1.40	21.60	0.75	2.60
SUB10	94.84	1.95	94.84	7.97	0.00	1.85	1.06	0.00	1.85	1.06
SUB11	63.13	1.95	63.13	0.00	0.00	1.95	1.00	0.00	1.95	1.00
SUB12	78.44	1.95	78.44	2.03	0.00	1.92	1.02	0.00	1.92	1.02
SUB13	53.28	1.95	53.28	1.80	0.00	1.91	1.02	28.00	0.83	2.36
SUB14	27.27	1.95	27.27	0.00	0.00	1.95	1.00	25.60	0.52	3.76
DNODE16	0	1.95	0	0.00	0.00	1.95	1.00	0.00	1.95	1.00
SUB15	45.16	1.95	38.66	0.00	0.47	1.90	0.00	8.40	1.29	0.00
LAKE17	0	1.95	6.5	0.00	0.00	1.95	1.00	0.00	1.95	1.00
SUB18	73.36	1.95	73.36	0.00	0.00	1.95	1.00	36.20	0.87	2.23
SUB19	25.63	1.95	25.63	0.00	0.00	1.95	1.00	0.00	1.95	1.00
Totals	806.12			32.97	9.57			119.80		

4.3.6.5 Initial and Continuing Losses

The initial loss was assumed to be zero. The continuing loss for the natural catchment was assumed to be 2.5 mm/hour. For the developed catchment, the continuing loss for each subarea was decreased to take account of the zero losses which would occur on the impervious area within that subarea. The impervious area was assumed to be 45% of the urbanised part of the subarea, and 14.7% of the rural residential part of the subarea.

The adopted continuing losses are also shown in Table 38.

As peak discharges for ARI's less than 100-year were required, additional calibration of the WBNM model was necessary to ensure that the modelled discharges for these more frequent events were also compatible with peak discharges estimated using the probabilistic rational method. This additional calibration was carried out by adjustment of the pervious area initial and continuing losses for the design rainfall events that were used in WBNM to derive the peak discharges.

The rainfall losses were adjusted so that the peak discharges at Duchess Gully for the catchment area in its natural (no development) state predicted by WBNM were equivalent to the peak discharges predicted by the Probabilistic Rational Method. The peak discharges and the losses are summarised in Table 39.

Table 39 Calibration of WBNM Model - All Events

Design Event (ARI)	Design Rainfall Event Losses for pervious areas		Peak Discharge at Duchess Gully outlet (natural catchment) (m ³ /sec)	
	Initial Loss (mm)	Continuing Loss (mm/hr)	WBNM Estimate	Prob. Rational Method Estimate
5	25	6.5	25.3	25.0
10	20	6.0	31.7	31.1
20	20	5.5	39.7	38.9
100	0	2.5	62.8	62.9

Hydrographs from the calibrated WBNM model for the design events were used as input to the hydraulic model.

4.3.6.6 WBNM Model Calculated Peak Discharges

The peak discharges predicted by WBNM for the existing and future development scenarios are listed for selected points in Table 40. The peaks shown are for the critical storm duration at the ocean outlet of 9 hours.

Table 40 Peak Discharge – 1% AEP 9 hour Duration Storm

Development Condition	Peak Q at Ocean Drive – southern culverts (Node 8)	Peak local discharge from subarea:		
		9	14	18
Natural	30.5	6.26	3.8	8.42
Existing	31.1	6.99	3.8	8.42
Future Scenario	31.1	8.15	5.4	10.99

The model predicted a 1.2% increase in the 1% AEP peak outflow to the ocean for the future development scenario, when compared with the existing development, even though the peak discharges from some individual subareas had increased by up to 40%. A reason for this is the timing of the peak discharges. The area proposed for future development is in the downstream part of the Duchess Gully catchment, and is only around 15% of the total catchment area. Peak discharges from the downstream subareas occur earlier than the routed peak from the upstream subareas, and therefore do not have a significant effect on the peak outflow from the whole catchment.

4.3.7 Hydraulic Modelling and Results

4.3.7.1 Hydraulic Model Setup

The MIKE-11 hydrodynamic model was used to simulate the hydraulic behaviour for the 1% AEP design event and other lesser events. The model extends from Ocean Drive down to the ocean outlet, and includes the existing lagoon. A diagram showing the network layout of the model is included as Figure 26.

4.3.7.2 Network – Existing Conditions

Information on the model branches is given in Table 41.

Table 41 MIKE11 Network Model Description - Existing Conditions

Branch Name	Branch Length (m)	Description
Duch-N	1250	"Middle" reaches of Duchess Ck, joining with Duch-L branch, 154 metres upstream of junction of Duch-S and Duch-L branches
Duch-S	1700	"Upper" reaches of Duchess Gully extending from southern set of culverts under Ocean Drive, through existing lagoon to the junction with Duch-L branch
Duch-U	870	Upper Tributary of Duchess Gully, extending from the northern set of culverts under Ocean Drive to the junction with Duch-S, just upstream of the existing lagoon
Duch-L	1465	"Lower" reaches of Duchess Gully, extending from 154 metres upstream of the junction with the "Overflow", to the ocean outlet
Dlink-N-S	400	Flow link between the "upper" reaches and "middle" reaches of Duchess Gully
Dlink-U-S	140	Flow link between the "upper" reaches of Duchess Gully and the Upper Tributary

4.3.7.3 Cross Sections

The cross section information for the MIKE-11 model was sourced from the following information:

- Cross sections surveyed by GMA for its June 1986 study of Duchess Gully for a proposed resort development.
- Survey transects taken across the low-lying areas of Duchess Gully by Luke & Company, as described in the SMC December 1994 Investigation Report.

The chainages of the cross sections and the source of each cross section used in the MIKE-11 model are listed in Table 42.

Table 42 MIKE11 Model - Cross Section Derivations

Existing (2000) Development			
Branch	Cross Section Chainage	Cross Section Data Source	Comment
Duch-N	20	Luke 1050	Part of transect only
Duch-N	185	Luke 1050	Part of transect only
Duch-N	315	Luke 1150	Part of transect only
Duch-N	445	Luke 1250	Part of transect only
Duch-N	565	Luke 1350	Part of transect only
Duch-N	675	Luke 1450	Part of transect only
Duch-N	835	GMA 35	Cross Section extended using contour plan
Duch-N	915	GMA 34	Cross Section extended using contour plan
Duch-N	1000	GMA 33	Cross Section extended using contour plan
Duch-N	1090	GMA 32	Cross Section extended using contour plan
Duch-N	1170	GMA 26	Cross Section extended using contour plan
Duch-N	1250	GMA 25	Cross Section extended using contour plan
Duch-S	0	Luke 70 Baseline 2	
Duch-S	70	Luke 0 Baseline 2	
Duch-S	160	Luke 150	Part of transect only, contours used also
Duch-S	280	Luke 250	Part of transect only, contours used also
Duch-S	540	Luke 350	Part of transect only
Duch-S	660	Luke 450	Part of transect only
Duch-S	780	Luke 550	Part of transect only
Duch-S	900	Luke 650	
Duch-S	1000	Luke 850	
Duch-S	1120	Luke 850	Part of transect only
Duch-S	1230	Luke 950	Part of transect only
Duch-S	1360	Luke 1100	Part of transect only
Duch-S	1440	Luke 1150 + 0/CS11	Part of transect only, plus 1983 study Cross section 11
Duch-S	1550	GMA 30	
Duch-S	1600	GMA 29	
Duch-S	1650	GMA 28	
Duch-S	1700	GMA 27	
Duch-U	0	Luke 60 Baseline 3	
Duch-U	80	Luke 0 Baseline 3	
Duch-U	150	Luke 50	Part of transect only, contour plan use also
Duch-U	300	Luke 150	Part of transect only, contour plan use also
Duch-U	440	Luke 250	Part of transect only, contour plan use also
Duch-U	550	Luke 350	Part of transect only, contour plan use also
Duch-U	650	Luke 450	Part of transect only, contour plan use also
Duch-U	780	Luke 550	Part of transect only
Duch-L	0	GMA 25	Cross Section extended using contours
Duch-L	70	GMA 24	Cross Section extended using contours
Duch-L	120	GMA 23	Cross Section extended using contours
Duch-L	160	GMA 22	Cross Section extended using contours
Duch-L	210	GMA 21	Cross Section extended using contours
Duch-L	320	GMA 20	Cross Section extended using contours
Duch-L	370	GMA 19	Cross Section extended using contours
Duch-L	420	GMA 18	Cross Section extended using contours
Duch-L	470	GMA 17	Cross Section extended using contours
Duch-L	530	GMA 16	Cross Section extended using contours
Duch-L	580	GMA 15	Cross Section extended using contours
Duch-L	625	GMA 14	Cross Section extended using contours
Duch-L	675	GMA 13	Cross Section extended using contours
Duch-L	725	GMA 12	Cross Section extended using contours
Duch-L	780	GMA 11	Cross Section extended using contours
Duch-L	830	GMA 10	Cross Section extended using contours
Duch-L	885	GMA 9	Cross Section extended using contours
Duch-L	935	GMA 8	Cross Section extended using contours
Duch-L	985	GMA 7	Cross Section extended using contours
Duch-L	1035	GMA 6	Cross Section extended using contours
Duch-L	1085	GMA 5	Cross Section extended using contours
Duch-L	1135	GMA 4	Cross Section extended using contours
Duch-L	1235	GMA 3	Cross Section extended using contours
Duch-L	1355	GMA 2	Cross Section extended using contours
Duch-L	1465	GMA 1	Cross Section extended using contours

4.3.7.4 MIKE11 Model Calibration

It was not possible to calibrate the hydrodynamic model as there was lack of recorded discharge and water level data for the drainage system. Flow roughness factors were determined from the vegetation and channel characteristics of the area. Aerial photography and photos of Duchess Gully (eg Figure 3) were available for this. The flow roughness factors (Mannings n values) chosen are listed in Table 43.

Table 43 MIKE11 Model - Mannings n Roughness - Existing Conditions

River Reach	Manning's n values			Comment
	Left Bank	Main Channel	Right Bank	
Duch-U 0 to 870	0.07	0.07	0.07	
Duch-S 0 to 900	0.07	0.07	0.07	
Duch-S 900-1360	0.07	0.035	0.07	Existing lagoon
Duch-S 1360- 1700	0.08	0.06	0.08	
Duch-N 0-700	0.07	0.07	0.07	
Duch-N 700-1250	0.08	0.07	0.08	
Duch-L 0-180	0.1	0.06	0.1	
Duch-L 180-800	0.1	0.05	0.1	
Duch-L 800-900	0.08 – 0.09	0.04	0.08 - 0.09	
Duch-L 900-1465	0.07 – 0.08	0.035	0.07 – 0.08	

4.3.7.5 Existing Lagoon Outlet

Water levels in the existing lagoon during major storm events are controlled by the level of an existing gravel track crossing of the creek downstream of the lagoon. The SMC December 1994 report indicates the level of this track is from RL 4.5 to 4.8 metres. The report also states that there are a number of small pipes under the track, with inverts ranging from RL 2.5 to 2.8 m AHD.

The track crossing has been modelled as a weir with the following width versus crest length relationship, as determined from the available survey information.

Table 44 MIKE11 Model - Existing Lake Outlet

Level (mAHD)	Width (m)
4.5	0.0
4.8	50
5.0	100
5.5	150

The pipes under the road have been ignored in the hydraulic analysis, as they have a very small capacity, and it is probable that the lake level would have risen to the road level prior to any significant design storm event.

4.3.7.6 Model Boundary Conditions

The inflow hydrographs for the 1% AEP design events were provided by the WBNM model. The source of the inflow hydrographs and the location of their input to the MIKE-11 model for the existing development scenario are listed in the Table 45.

Table 45 MIKE11 Model - Inflow Hydrographs – Existing Conditions

WBNM flow hydrograph	Location of inflow	Comment
Total runoff at node 8 outlet	Duch-S 0.00	Flow from area upstream of Ocean Drive southern culverts
Total runoff at Subarea 13 outlet	Duch-U 0.00	Flow from area upstream of Ocean Drive northern culverts
Local runoff from subarea 9	Duch-S 280	
Local runoff from subarea 14	Duch-U 550	
Local runoff from subarea 15	Duch-S 1120	
Local runoff from subarea 18	Duch-N 60	In cases with the new constructed wetland, runoff is assumed directly to new wetland
Local runoff from subarea 19	Duch-L 725	
Local runoff from subarea 17	Duch-S 950	Rain directly on existing lagoon

An ocean level at Duchess Gully outlet is required for the modelling. The mean high water springs level (MHWS) for the area is approximately RL 0.65 m AHD. A 1% AEP storm tide level of 2.6 m AHD is recommended for the area in recent planning guidelines (DECCW 2009b).

The probability of the 1% AEP peak discharge and the 1% AEP storm tide level occurring simultaneously is expected to be much less than 1% AEP, therefore it is reasonable to assume a lower ocean level than the 1% AEP storm tide level during the 1% AEP event for Duchess Gully. Planning guidelines (DECCW 2009b) recommend that the 5% AEP (20 year ARI) ocean level of RL 2.3 be adopted as likely to coincide with the 1% AEP storm runoff. A level of RL 2.3 m AHD has therefore been assumed as the tailwater level at the mouth of Duchess Gully for the modelling.

The higher level of RL 2.6 m AHD was modelled also as part of a sensitivity analysis.

4.3.8 Development Scenarios Modelled

The model was used to determine the peak levels, discharges and velocities for the 1% AEP 9 hour duration storm event. This duration produced the peak discharge at the ocean outlet, and the peak levels from the existing lagoon to the ocean outlet.

A number of development scenarios were modelled, and descriptions of the relevant scenarios follow:

4.3.8.1 Existing Development

This case (RB-Exist1) modelled the existing situation which included the existing 6.8 hectare lagoon and the outlet arrangement. The existing residential development includes a total of about 9 hectares of urban development in sub-area 9 and about 33 hectares of rural residential development in the catchment upstream of Ocean Drive. The Mike-11 model setup for this case has been described previously.

The peak outflow to the ocean was estimated at 63.3 m³/sec, and the peak level reached in the lake was estimated at 5.45 m AHD for the 9 hour 1% AEP event. Summary details of the results including peak discharges, peak velocities and peak water levels at each of the cross sections is included in later tables.

4.3.8.2 Future Urban Development with Constructed Wetland

For this case, (case RB-DEV24C) it was assumed that a new wetland would be constructed north and east of the existing lagoon. The new constructed wetland would also be a local source of fill material for the proposed residential areas. The new constructed wetland is 10.5 hectares in area with approximate dimensions of 266 metres width and 500 metres length (including adjoining stormwater treatment water bodies), and connected to both the existing lagoon and the “middle” reaches of Duchess Gully (at Duch-N 1090).

The proposed constructed wetland has been modelled as a branch “Newlake2” with 8 cross sections. A diagram showing the network layout of this model is given in Figure 27.

A description of the model branches for the proposed development is given in Table 46.

Table 46 MIKE11 Model Network - Proposed Development

Branch Name	Branch Length (m)	Description
Duch-N	431	“Middle” reaches of Duchess Ck, joining with Duch-L branch, 154 metres upstream of junction of Duch-S and Duch-L branches
Duch-S	1700	“Upper” reaches of Duchess Gully extending from southern set of culverts under Ocean Drive, through existing lagoon then continuing via the “Overflow” channel to the junction with Duch-L branch
Duch-U	870	Upper Tributary of Duchess Gully, extending from the northern set of culverts under Ocean Drive to the junction with Duch-S, just upstream of the existing lagoon
Duch-L	1465	“Lower” reach of Duchess Gully, extending from 154 metres upstream of the junction with the Overflow, to the ocean outlet
Dlink-U-S	140	Flow link between Duch-U and Duch-S branches
Newlake2	847	Proposed Constructed Wetland, connecting with Dutch-S branch
Lakelink2	20	Flow link between Newlake2 and Duch-N branches

4.3.8.3 Proposed Outlet Structures

The normal top water level in the new constructed wetland was assumed to be RL 3.0 m AHD. This level will be maintained by a constructed “base flow” channel at the northern end of the constructed wetland connecting to “middle” reach of Duchess Gully at the north-east of the site (see Figure 30). This low-capacity channel is only sufficient to carry base flows from the catchment and small runoff events.

A new weir structure with a crest level of RL 3.4 m AHD will be provided to connect the constructed wetland to the existing lagoon and divert base flows from the existing lagoon through the new constructed wetland. This connection is shown in Figure 33. These control structures will serve to re-establish the original flow path of Duchess Gully.

Minor event flows discharging from the constructed wetland up to and including the 5 Year ARI events will be controlled by a weir structure with a crest level of RL 3.7 m AHD and 25 metres crest length which connects to Duchess Gully further downstream in the “middle” reach. The location and layout for this weir are shown in Figure 28. The level of this weir is dictated by the level of services pipelines which run north-south along the western bank of Duchess Gully as shown in Figure 28. The outlet channel from this weir structure passes over the pipelines which will be protected with reinforced concrete surround at this section as shown in Figure 29.

Minor event flows (up to 5 Year ARI) from the existing lagoon will be divided equally between the constructed wetland and the existing “overflow” channel which connects directly to the “lower” reach of Duchess Gully by providing another new weir with a crest level of RL 3.5 to replace the existing pipe culvert under the existing gravel track. This structure is shown in Figure 31. The length of this weir and the “connection” weir are each 5 metres which provides sufficient capacity to pass 5 Year ARI flows.

During major rainfall events exceeding 5 Year ARI, flows from the existing lagoon and the constructed wetlands will overtop the banks around the water bodies along selected sections of the banks which will be regraded level to control and evenly distribute the flow. The lengths of the level regraded sections of the banks were adjusted so that maximum head on the banks for the 100 year ARI storm event is no greater than 300 mm. This limit ensures that the flow velocity across the banks is limited and does not exceed the scour velocity for grassed surfaces. The arrangement of these bank overflow sections are shown in Figure 28 and Figure 31.

The dimensions of these weir structures and bank overflow sections are summarised in Table 47.

Table 47 MIKE11 Model – Flow Control Structures – Development Conditions

Flow control description	ID	Dimensions			
		Low Flow Weir		Bank high flow overflow	
		Level RL mAHD	Length m	Level RL mAHD	Length M
Existing lagoon outlet to “Overflow”	S3	3.5	5	4.85	100
New constructed wetland outlet to “middle” reaches of Duchess Gully	S4, S5	3.7 (S4)	25 (S4)	4.1 (S5)	150 (S5)
Connection between existing lagoon and new constructed wetland	S2	3.4	5	4.85	100

This arrangement of weirs provides a “minor/major” system of controls which satisfies Port Macquarie Hastings Council's drainage guidelines (AUS-SPEC 2003) and the NSW Floodplain Development Manual (DIPNR 2005) by segregating and confining flows up to 5 Year ARI in defined structures and only allowing controlled overland flows for more severe major storm events.

4.3.8.4 Proposed Channel Improvements

In addition to the new constructed wetland, the “Development Conditions” case also incorporated changes to the cross sections of the drains in the Upper Tributary upstream of the existing lake. To counteract loss of conveyance from encroachment of the proposed sports fields, a channel diversion with a trapezoidal section of 1 metres bed width and bed level 0.5 metres below the existing bed level was assumed over a 170 metre length from Ch 80 to Ch 250. This diversion channel was sufficient to maintain “Development” flood levels at Ocean Drive at no greater than the “existing” flood levels.

4.3.8.5 Results for 100 Year ARI Rainfall Event – Developed Conditions

The peak outflow to the ocean for this case was estimated at 62.7 m³/sec (about 5% less than for the existing), and the peak lake level in the existing lagoon was estimated as RL 5.15 m AHD. The peak lake level in the new wetland was estimated as RL 4.56 m AHD. In the “lower” reach of Duchess Gully downstream of the “overflow” junction, peak levels were up to 70 mm lower and peak velocities slightly less than for the existing situation.

The 100 Year ARI rainfall event maximum water levels for “Developed Conditions” are tabulated in Table 48 and shown plotted in Figure 36.

The peak outflow from the existing lagoon to the “lower” reaches of Duchess Gully has been decreased by about 45%. The reason for this decrease is that flow also occurs from the existing lagoon through the new constructed wetland and into the “middle” reaches of Duchess Gully. The peak flow in the “middle” reaches of Duchess Gully between the new wetland outlet and the junction with the “overflow” has increased approximately three-fold (from 9 m³/sec to 30 m³/sec). The peak velocities in this reach are still significantly less than the peak velocities in the “lower” reaches of Duchess Gully downstream of the “overflow” junction.

The 100 Year ARI peak flows for “Developed Conditions” are tabulated in Table 49. Plots of the discharge hydrographs at the mouth of Duchess Gully for the existing (RB-exist1), and developed with 12.7ha lake (RB-dev24C) cases are shown in Figure 34.

Stormwater and stream water level gradients throughout the Open Space Corridor are shown plotted in Figure 36. Water levels are presented for three separate longitudinal profiles, namely:

- Profile ‘A’ – Tracking water levels along the edge of the proposed reclamation areas and through the constructed wetlands then along the lower reach of Duchess Gully;
- Profile ‘B’ – Water levels along the upper reach of Duchess Gully then following the “overflow” channel into Duchess Gully;
- Profile ‘C’ – Water levels in the lower reach of Duchess Gully.

The location and alignments of these three profiles are shown in Figure 35.

Stormwater and stream water levels are not worsened in the “upper” reaches of Duchess Gully adjacent to the existing residential areas. Maximum water levels are also not worsened at the boundary of the site at either of the culverts under Ocean Drive.

Water levels are increased by 110 mm at the boundary of the adjoining Milland property to the north-east of the site (Chainage 835 DUCH-N branch on the MIKE11 model) but this afflux reduces to zero only 85 metres further upstream within the adjoining Milland property. This impact is considered minor and unlikely to adversely affect any future use of that property. The steepness of the land in the area indicates that only small volumes of site fill would be required to offset the effects of the afflux. This impact is described in detail in Section 4.3.8.9

Figure 37 shows stream velocities and flood depths throughout the low-lying areas of Duchess Gully. The resulting hazard conditions are discussed further in Section 4.7.3.3.

Table 48 MIKE11 Model Results - 100Yr ARI Event – Maximum Water Levels

<i>Catchment development</i>	<i>Existing</i>	<i>Site developed</i>
<i>U/S exist Lagoon</i>	<i>Existing</i>	<i>Excavated+Fill</i>
<i>Existing Lagoon level S3</i>	<i>RL 4.5 m</i>	<i>RL 3.5 - (outlet weir)</i>
<i>New Wetland outlet S5</i>	<i>Na</i>	<i>RL 3.0 - (outlet channel)</i>
<i>Connection S2</i>	<i>Na</i>	<i>RL 3.4 - (invert)</i>
<i>Tailwater Level</i>	<i>RL 2.3</i>	<i>RL 2.3</i>
<i>Storm ARI & Duration</i>	<i>100yr 9hr</i>	<i>100yr 9hr</i>

Branch & Chainage	Max WL RL m	Max WL RL m	Afflux Mm
DUCH-S 0.00	6.685	6.669	-16
DUCH-S 70.00	6.474	6.411	-63
DUCH-S 160.00	5.968	5.915	-53
DUCH-S 280.00	5.772	5.782	10
DUCH-S 540.00	5.540	5.497	-43
DUCH-S 540.00	5.540	5.497	-43
DUCH-S 660.00	5.492	5.382	-110
DUCH-S 780.00	5.461	5.175	-286
DUCH-S 800.00	5.458	5.145	-313
DUCH-S 800.00	5.458	5.145	-313
DUCH-S 900.00	5.444	5.128	-316
DUCH-S 1000.00	5.442	5.124	-318
DUCH-S 1120.00	5.442	5.124	-318
DUCH-S 1230.00	5.441	5.123	-318
DUCH-S 1360.00	5.441	5.123	-318
DUCH-S 1360.00	5.441	5.123	-318
DUCH-S 1440.00	5.394	4.474	-920
DUCH-S 1550.00	4.693	4.461	-232
DUCH-S 1600.00	4.507	4.388	-119
DUCH-S 1650.00	4.363	4.340	-23
DUCH-S 1700.00	4.139	4.275	136
DUCH-L 0.00	4.142	4.358	216
DUCH-L 70.00	4.140	4.311	171
DUCH-L 120.00	4.139	4.290	151
DUCH-L 154.87	4.139	4.275	136
DUCH-L 154.87	4.139	4.275	136

DUCH-L 160.00	4.130	4.266	136
DUCH-L 210.00	4.070	4.202	132
DUCH-L 320.00	3.875	3.996	121
DUCH-L 370.00	3.782	3.898	116
DUCH-L 420.00	3.763	3.880	117
DUCH-L 470.00	3.720	3.835	115
DUCH-L 530.00	3.649	3.763	114
DUCH-L 580.00	3.574	3.683	109
DUCH-L 625.00	3.526	3.633	107
DUCH-L 675.00	3.416	3.518	102
DUCH-L 725.00	3.316	3.413	97
DUCH-L 780.00	3.251	3.342	91
DUCH-L 830.00	3.192	3.281	89
DUCH-L 885.00	3.113	3.199	86
DUCH-L 935.00	3.052	3.135	83
DUCH-L 985.00	3.025	3.106	81
DUCH-L 1035.00	2.970	3.047	77
DUCH-L 1085.00	2.905	2.979	74
DUCH-L 1135.00	2.824	2.890	66
DUCH-L 1235.00	2.711	2.765	54
DUCH-L 1355.00	2.561	2.596	35
DUCH-L 1465.00	2.300	2.300	0
DUCH-U 0.00	6.958	6.909	-49
DUCH-U 80.00	6.642	6.501	-141
DUCH-U 150.00	6.283	6.144	-139
DUCH-U 300.00	5.627	5.587	-40
DUCH-U 440.00	5.542	5.490	-52
DUCH-U 550.00	5.533	5.460	-73
DUCH-U 550.00	5.533	5.460	-73
DUCH-U 650.00	5.516	5.406	-110
DUCH-U 750.00	5.494	5.333	-161
DUCH-U 870.00	5.458	5.145	-313
DLINK-U-S 0.00	5.533	5.460	-73
DLINK-U-S 140.00	5.540	5.497	-43
DUCH-N 750.00	4.46	4.47	10
DUCH-N 835.00	4.360	4.47	110
DUCH-N 915.00	4.298	4.474	176
DUCH-N 1000.00	4.204	4.474	270
DUCH-N 1090.00	4.146	4.474	328
DUCH-N 1090.00	4.146	4.474	328
DUCH-N 1170.00	4.143	4.474	331
DUCH-N 1250.00	4.142	4.474	332
NEWLAKE2 0.00		5.123	
NEWLAKE2 22.00		4.548	
NEWLAKE2 43.00		4.548	
NEWLAKE2 110.00		4.548	
NEWLAKE2 207.00		4.548	
NEWLAKE2 331.00		4.548	
NEWLAKE2 419.00		4.548	
NEWLAKE2 508.00		4.548	
NEWLAKE2 508.00		4.548	

NEWLAKE2 581.00		4.548	
NEWLAKE2 647.00		4.548	
LAKELINK2 0.00		4.548	
LAKELINK2 20.00		4.474	

Table 49 MIKE11 Model Results - 100Yr ARI Event - Peak Discharges

<i>Catchment development</i>	<i>Existing</i>	<i>Site developed</i>
<i>U/S Existing Lagoon</i>	<i>Existing</i>	<i>Excavated+Fill</i>
<i>Existing Lagoon level S3</i>	<i>RL 4.5 m</i>	<i>RL 3.5 - (outlet weir)</i>
<i>New Wetland S5</i>	<i>na</i>	<i>RL 3.0 - (outlet weir)</i>
<i>Connection S2</i>	<i>na</i>	<i>RL 3.4 - (invert)</i>
<i>Tailwater Level</i>	<i>RL 2.3</i>	<i>RL 2.3</i>
<i>Storm ARI & Duration</i>	<i>100yr 9hr</i>	<i>100yr 9hr</i>

Branch & Chainage	Peak Q (m3/sec)	Peak Q (m3/sec)	% Change
DUCH-S 35.00	31.0	31.1	0%
DUCH-S 115.00	31.1		
DUCH-S 220.00	30.9	30.9	0%
DUCH-S 410.00	36.6	37.3	2%
DUCH-S 600.00	19.6	26.0	33%
DUCH-S 720.00	19.1	26.0	36%
DUCH-S 790.00	18.8	25.9	38%
DUCH-S 850.00	55.5	63.0	13%
DUCH-S 950.00	54.8	62.8	15%
DUCH-S 1060.00	54.7	63.2	16%
DUCH-S 1175.00	57.2	67.2	17%
DUCH-S 1295.00	56.7	67.1	18%
DUCH-S 1430.00	55.9	32.9	-41%
DUCH-S 1495.00	55.9	32.5	-42%
DUCH-S 1575.00	55.9	32.4	-42%
DUCH-S 1625.00	55.9	32.4	-42%
DUCH-S 1675.00	55.9	32.4	-42%
DUCH-L 35.00	6.1	36.8	507%
DUCH-L 95.00	6.1	36.9	507%
DUCH-L 137.43	6.1	36.9	507%
DUCH-L 157.43	61.9	67.5	9%
DUCH-L 185.00	61.9	67.5	9%
DUCH-L 265.00	61.9	67.5	9%
DUCH-L 345.00	61.9	67.5	9%
DUCH-L 395.00	61.9	67.5	9%
DUCH-L 445.00	61.9	67.5	9%
DUCH-L 500.00	61.9	67.5	9%
DUCH-L 555.00	61.9	67.5	9%
DUCH-L 602.50	61.9	67.5	9%
DUCH-L 650.00	61.9	67.5	9%
DUCH-L 700.00	61.9	67.5	9%
DUCH-L 752.50	63.3	69.3	10%
DUCH-L 805.00	63.3	69.3	10%
DUCH-L 857.50	63.3	69.3	10%
DUCH-L 910.00	63.3	69.3	10%

DUCH-L 960.00	63.3	69.3	10%
DUCH-L 1010.00	63.3	69.3	10%
DUCH-L 1060.00	63.3	69.3	10%
DUCH-L 1110.00	63.3	69.3	10%
DUCH-L 1185.00	63.3	69.3	10%
DUCH-L 1295.00	63.3	69.3	10%
DUCH-L 1410.00	63.3	69.3	10%
DUCH-U 40.00	25.3	24.7	-2%
DUCH-U 115.00	25.3	24.7	-2%
DUCH-U 225.00	25.3	24.7	-3%
DUCH-U 370.00	25.0	24.7	-1%
DUCH-U 495.00	24.8	25.0	1%
DUCH-U 600.00	39.5	37.4	-5%
DUCH-U 700.00	38.5	37.3	-3%
DUCH-U 810.00	37.6	37.2	-1%
DLINK-U-S 70.00	0.1	0.0	
DUCH-N 827.00		3.3	
DUCH-N 875.00		3.3	
DUCH-N 957.50		3.0	
DUCH-N 1045.00		2.8	
DUCH-N 1130.00		36.6	
DUCH-N 1210.00		36.7	
NEWLAK2 11.00		34.2	
NEWLAK2 32.50		34.0	
NEWLAK2 76.50		33.5	
NEWLAK2 158.50		32.9	
NEWLAK2 269.00		32.4	
NEWLAK2 375.00		36.2	
NEWLAK2 463.50		35.4	
NEWLAK2 544.50		2.4	
NEWLAK2 614.00		0.7	
LAKELINK2 10.00		40.4	

4.3.8.6 Comparison of results – 5, 10, 20, and 100 Year ARI Events

The summary details for the 5, 10, 20 and 100 Year ARI events at selected locations are tabulated in Table 50 and Table 51. Peak heights and peak discharges at each cross section are shown in Table 52 and Table 53.

Results are shown for all events including 5-year, 10-year, 20-year, and 100-year ARI events. The peak levels and discharges for selected locations are also summarised in the tables.

Table 50 MIKE11 Model - Peak Levels at Selected Locations

Location		Peak Water level (RL m AHD) At location for various ARI of:			
Description	Branch & Chainage	5 year	10 year	20 year	100 year
Existing Lagoon	Duch-S 1360	4.62	4.94	5.00	5.12
Constructed Wetland	Newlake2 331	4.06	4.23	4.33	4.54
Near upstream boundary existing lagoon	Duch-S 540	5.29	5.33	5.39	5.50
Ocean Drive - Northern Culverts	Duch-U 0.00	6.71	6.75	6.80	6.91
Ocean Drive – Southern Culverts	Duch-S 0.00	6.51	6.54	6.59	6.69

Table 51 MIKE11 Model – Peak Discharges at Selected Locations

Location		Peak Discharge (m ³ /sec) At location for various ARI of:			
Description	Branch & Chainage	5 year	10 year	20 year	100 year
Duchess Ck Outlet	Duch-L 1410	14.7	31.7	44.4	69.3
Outlet from Existing Lagoon to “overflow” (S3)	Duch-S 1495	7.6	14.6	19.7	32.5
Outlet from constructed wetland (S5)	Lakelink2 10	7.5	17.2	24.6	40.2
Ocean Drive - Northern Culverts	Duch-U 40	10.6	13.1	16.2	24.7
Ocean Drive – Southern Culverts	Duch-S 35	6.2	17.1	21.4	36.8

Table 52 MIKE11 Model Results - 5, 10, 20Yr ARI Events - Max Levels

Catchment development	<i>Site developed</i>	<i>Site developed</i>	<i>Site developed</i>
U/S Existing Lake	<i>Excavated+Fill</i>	<i>Excavated+ Fill</i>	<i>Excavated+ Fill</i>
Existing Lagoon level S3	<i>RL 3.5 - (outlet weir)</i>	<i>RL 3.5 - (outlet weir)</i>	<i>RL 3.5 - (outlet weir)</i>
New Wetland S5	<i>RL 3.0 - (outlet weir)</i>	<i>RL 3.0 - (outlet weir)</i>	<i>RL 3.0 - (outlet weir)</i>
Connection S2	<i>RL 3.4 - (invert)</i>	<i>RL 3.4 - (invert)</i>	<i>RL 3.4 - (invert)</i>
Tailwater Level	<i>RL 2</i>	<i>RL 2</i>	<i>RL 2</i>
Storm ARI & Duration	<i>5yr 9hr</i>	<i>10yr 9hr</i>	<i>20yr 9hr</i>

Branch & Chainage	Max WL RL m	Max WL RL m	Max WL RL m
DUCH-S 0.00	6.34	6.52	6.57
DUCH-S 70.00	6.16	6.30	6.34
DUCH-S 160.00	5.66	5.79	5.83
DUCH-S 280.00	5.42	5.59	5.65
DUCH-S 540.00	5.20	5.32	5.38
DUCH-S 540.00	5.20	5.32	5.38
DUCH-S 660.00	5.11	5.22	5.28
DUCH-S 780.00	4.84	5.00	5.06
DUCH-S 800.00	4.62	4.95	5.02

DUCH-S 800.00	4.62	4.95	5.02
DUCH-S 900.00	4.62	4.94	5.01
DUCH-S 1000.00	4.62	4.94	5.01
DUCH-S 1120.00	4.62	4.94	5.01
DUCH-S 1230.00	4.62	4.94	5.00
DUCH-S 1360.00	4.62	4.94	5.00
DUCH-S 1360.00	4.62	4.94	5.00
DUCH-S 1440.00	2.87	3.52	3.88
DUCH-S 1550.00	2.87	3.52	3.87
DUCH-S 1600.00	2.79	3.45	3.81
DUCH-S 1650.00	2.73	3.39	3.76
DUCH-S 1700.00	2.63	3.33	3.70
DUCH-L 0.00	2.68	3.40	3.77
DUCH-L 70.00	2.65	3.35	3.73
DUCH-L 120.00	2.64	3.34	3.71
DUCH-L 154.87	2.63	3.33	3.70
DUCH-L 154.87	2.63	3.33	3.70
DUCH-L 160.00	2.63	3.32	3.69
DUCH-L 210.00	2.59	3.27	3.63
DUCH-L 320.00	2.51	3.14	3.47
DUCH-L 370.00	2.48	3.08	3.40
DUCH-L 420.00	2.46	3.05	3.37
DUCH-L 470.00	2.44	3.02	3.33
DUCH-L 530.00	2.40	2.96	3.27
DUCH-L 580.00	2.38	2.91	3.21
DUCH-L 625.00	2.36	2.88	3.17
DUCH-L 675.00	2.32	2.80	3.07
DUCH-L 725.00	2.27	2.73	2.98
DUCH-L 780.00	2.25	2.69	2.93
DUCH-L 830.00	2.31	2.64	2.87
DUCH-L 885.00	2.25	2.56	2.79
DUCH-L 935.00	2.22	2.51	2.74
DUCH-L 985.00	2.20	2.49	2.71
DUCH-L 1035.00	2.18	2.45	2.66
DUCH-L 1085.00	2.16	2.41	2.60
DUCH-L 1135.00	2.14	2.36	2.53
DUCH-L 1235.00	2.11	2.29	2.43
DUCH-L 1355.00	2.06	2.18	2.28
DUCH-L 1465.00	2.00	2.00	2.00
DUCH-U 0.00	6.71	6.75	6.80
DUCH-U 80.00	6.12	6.22	6.35
DUCH-U 150.00	5.81	5.88	6.06
DUCH-U 300.00	5.36	5.41	5.47
DUCH-U 440.00	5.26	5.31	5.36
DUCH-U 550.00	5.24	5.28	5.34
DUCH-U 550.00	5.24	5.28	5.34
DUCH-U 650.00	5.18	5.22	5.28
DUCH-U 750.00	5.11	5.15	5.22
DUCH-U 870.00	4.90	4.95	5.02
DLINK-U-S 0.00	5.24	5.28	5.34
DLINK-U-S 140.00	5.29	5.32	5.38

DUCH-N 819.00	2.95	3.53	3.90
DUCH-N 835.00	2.93	3.53	3.90
DUCH-N 915.00	2.92	3.53	3.90
DUCH-N 1000.00	2.92	3.53	3.90
DUCH-N 1090.00	2.92	3.53	3.90
DUCH-N 1090.00	2.92	3.53	3.90
DUCH-N 1170.00	2.84	3.45	3.83
DUCH-N 1250.00	2.81	3.40	3.77
NEWLAKE2 0.00	4.86	4.94	5.00
NEWLAKE2 22.00	4.06	4.30	4.34
NEWLAKE2 43.00	4.06	4.30	4.34
NEWLAKE2 110.00	4.06	4.30	4.34
NEWLAKE2 207.00	4.06	4.30	4.34
NEWLAKE2 331.00	4.06	4.30	4.34
NEWLAKE2 419.00	4.06	4.30	4.34
NEWLAKE2 508.00	4.06	4.30	4.34
NEWLAKE2 508.00	4.06	4.30	4.34
NEWLAKE2 581.00	4.06	4.30	4.34
NEWLAKE2 647.00	4.06	4.30	4.34
LAKELINK2 0.00	4.06	4.30	4.34
LAKELINK2 20.00	2.92	3.53	3.90

Table 53 MIKE11 Model Results - 5, 10, 20Yr ARI Events - Peak Discharges

<i>Catchment development</i>	<i>Site developed</i>	<i>Site developed</i>	<i>Site developed</i>
<i>U/S Existing Lagoon</i>	<i>Excavated+ Fill</i>	<i>Excavated+ Fill</i>	<i>Excavated+ Fill</i>
<i>Existing Lagoon level S3</i>	<i>RL 3.5 - (outlet weir)</i>	<i>RL 3.5 - (outlet weir)</i>	<i>RL 3.5 - (outlet weir)</i>
<i>New Wetland S5</i>	<i>RL 3.0 - (outlet weir)</i>	<i>RL 3.0 - (outlet weir)</i>	<i>RL 3.0 - (outlet weir)</i>
<i>Connection S2</i>	<i>RL 3.4 - (invert)</i>	<i>RL 3.4 - (invert)</i>	<i>RL 3.4 - (invert)</i>
<i>Tailwater Level</i>	<i>RL 2</i>	<i>RL 2</i>	<i>RL 2</i>
<i>Storm ARI & Duration</i>	<i>5yr 9hr</i>	<i>10yr 9hr</i>	<i>20yr 9hr</i>
<i>Branch & Chainage</i>	<i>Peak Q (m3/sec)</i>	<i>Peak Q (m3/sec)</i>	<i>Peak Q (m3/sec)</i>
DUCH-S 35.00	6.2	17.1	21.4
DUCH-S 115.00			
DUCH-S 220.00	6.1	17.0	21.2
DUCH-S 410.00	10.0	20.6	25.7
DUCH-S 600.00	6.1	12.3	16.1
DUCH-S 720.00	6.0	12.2	16.0
DUCH-S 790.00	6.0	12.0	15.9
DUCH-S 850.00			
DUCH-S 950.00	18.6	31.8	40.6
DUCH-S 1060.00	18.0	31.7	40.8
DUCH-S 1175.00	18.4	32.9	43.0
DUCH-S 1295.00	17.5	32.6	42.8
DUCH-S 1430.00	8.0	15.7	20.8
DUCH-S 1495.00	7.6	14.6	19.7
DUCH-S 1575.00	7.6	14.6	19.7
DUCH-S 1625.00	7.6	14.5	19.7
DUCH-S 1675.00	7.6	14.5	19.7
DUCH-L 35.00	7.6	17.1	24.1

DUCH-L 95.00	7.7	17.1	24.1
DUCH-L 137.43	7.7	17.1	24.1
DUCH-L 157.43	14.5	31.4	43.7
DUCH-L 185.00	14.5	31.4	43.7
DUCH-L 265.00	14.5	31.4	43.7
DUCH-L 345.00	14.5	31.4	43.7
DUCH-L 395.00	14.5	31.4	43.6
DUCH-L 445.00	14.5	31.4	43.6
DUCH-L 500.00	14.5	31.4	43.6
DUCH-L 555.00	14.5	31.3	43.6
DUCH-L 602.50	14.5	31.3	43.6
DUCH-L 650.00	14.5	31.3	43.6
DUCH-L 700.00	14.5	31.3	43.6
DUCH-L 752.50	14.7	31.7	44.4
DUCH-L 805.00	14.7	31.7	44.4
DUCH-L 857.50	14.7	31.7	44.4
DUCH-L 910.00	14.7	31.7	44.4
DUCH-L 960.00	14.7	31.7	44.4
DUCH-L 1010.00	14.7	31.7	44.4
DUCH-L 1060.00	14.7	31.7	44.4
DUCH-L 1110.00	14.7	31.7	44.4
DUCH-L 1185.00	14.7	31.7	44.4
DUCH-L 1295.00	14.7	31.7	44.4
DUCH-L 1410.00	14.7	31.7	44.4
DUCH-U 40.00	10.6	13.1	16.2
DUCH-U 115.00	10.6	13.1	16.2
DUCH-U 225.00	10.6	13.1	16.2
DUCH-U 370.00	10.6	13.1	16.2
DUCH-U 495.00	10.8	13.2	16.4
DUCH-U 600.00	14.9	21.8	25.8
DUCH-U 700.00	14.7	21.1	25.4
DUCH-U 810.00	14.3	20.6	25.2
DLINK-U-S 70.00	0.2	0.0	0.0
DUCH-N 827.00	1.9	0.2	0.2
DUCH-N 875.00	1.9	0.2	0.2
DUCH-N 957.50	1.9	0.2	0.2
DUCH-N 1045.00	1.9	0.2	0.3
DUCH-N 1130.00	7.6	17.2	24.2
DUCH-N 1210.00	7.6	17.1	24.1
NEWLAK2 11.00	9.1	16.9	22.0
NEWLAK2 32.50	8.9	16.7	21.9
NEWLAK2 76.50	8.6	16.5	21.8
NEWLAK2 158.50	8.2	16.3	21.7
NEWLAK2 269.00	7.8	16.1	21.7
NEWLAK2 375.00	7.9	17.4	24.7
NEWLAK2 463.50	7.7	17.3	24.6
NEWLAK2 544.50	1.7	2.2	2.4
NEWLAK2 614.00	0.5	0.6	0.7
LAKELINK2 10.00	7.5	17.2	24.6

4.3.8.7 Flows at Hydraulic Control Structures

Peak flows at the flow control structures are shown in Table 54.

Table 54 MIKE11 Model Results – Flows at Hydraulic Control Structures

Location	Code		5 Yr ARI	10 Yr ARI	20 Yr ARI	100 Yr ARI
Existing Lagoon Connection to Wetlands	S2	Low Flow Weir	9.1	13.0	13.7	15.3
		High Flow Overflow	0.0	3.9	8.3	18.8
Existing Lagoon "Overflow"	S3	Low Flow Weir	8.0	11.7	12.5	14.0
		High Flow Overflow	0.0	3.9	8.2	18.8
Wetlands Outlet	S4	Low Flow Weir	7.5	13.5	15.4	16.8
	S5	High Flow Overflow	0.0	3.7	9.2	23.4

- For minor runoff events, most of the outflow occurs through the water level control "low flow" weir structures;
- Overflow across the embankments only occurs for major storm events.
- For both the existing lagoon and the constructed wetland, the recurrence interval (ARI) for embankment overflow is 5 years ARI. This is in accordance with Council's "major/minor" recommendations for drainage design.
- The flows through the "low flow" control structures increase only moderately as the storm severity increases.
- Flow conditions on the embankments during overflows in major storm events are generally within the non-scouring limit for grassed surfaces with velocities of less than 1.8 m/sec.
- The frequency of inundation of the frog habitat on the north bank of the existing lagoon is reduced. With development, this habitat is only inundated by events in excess of the 5 Year ARI storm event (which has a maximum water level of RL 4.62) whereas the habitat is inundated by lesser events under existing conditions.

4.3.8.8 Impacts on Frog Breeding Habitat

The reduction in frequency of inundation of the frog habitat is not expected to reduce the effectiveness of this area as a frog breeding site.

Anecdotal evidence from the site and recent photos show that the frog habitat depression remains waterlogged for extended periods even without major runoff in the lagoon and overflow into the depression. These waterlogged conditions can be maintained by rainfall directly onto the habitat area combined with the presence of an impermeable surface clay layer in this area (see the bore log for Borehole BH#3) which reduces infiltration losses.

On balance it appears that the existing frog breeding conditions are dependent on local rainfall rather than periodic overflows from the existing lagoon. It was also demonstrated in Section 2.11.8.3 that the water levels in the frog habitat are not reliant on groundwater inflows. In that case, the proposed development is not expected to change conditions in the habitat.

As a result the existing frog habitat site will continue to function as an opportunistic breeding site. In addition, the proposed constructed wetland and stormwater treatment wetlands will further increase frog breeding habitat.

4.3.8.9 Impacts on Adjoining Milland Property

As noted previously in Section 4.3.8.5, inundation levels for the design storm event are increased as a result of the proposed development by 110 mm at the boundary of the adjoining Milland property to the north-east of the site (Chainage 835 DUCH-N branch on the MIKE11 model) but this afflux reduces to zero only 85 metres further upstream within the adjoining Milland property.

The attached diagram Figure 38 indicates “Existing Inundation Limits”, “Developed Inundation Limits” and “Development Inundation Limits with +20% Rainfall Intensity due to Climate Change”. The relevant water levels at various chainages along the stream, including the boundaries of the Milland property, for the different scenarios are listed in the following Table 55.

Table 55 Inundation Water Levels within the Adjoining Milland Property

Model Configuration	Stream Chainage (Figure 38)			
	915 (SVF site)	835 (boundary)	750 (boundary)	675 (SVF site)
“Existing” (100 yr rain; Ocean RL 2.3)	4.3	4.36	4.46	4.55
“Developed” (100 yr rain; Ocean RL 2.3)	4.47	4.47	4.47	4.47
Afflux caused by development (mm)	+170	+110	+10	-100
“Developed with Climate Change” (100 yr rain + 20%; Ocean RL 3.2)	4.74	4.74	4.74	4.74

This impact is considered minor and unlikely to adversely affect any future use of that property. The steepness of the land in the area indicates that only small volumes of site fill would be required to offset the effects of the afflux.

The proposed constructed wetlands re-establishes the original natural flow path of Duchess Gully discharging into the northern end of the defined channel of the lower reach of Duchess Gully.

The hydrodynamic modelling to date has not simulated these original natural flow conditions. Nonetheless, flows in the northern end of the Duchess Gully channel (through the Milland property) under proposed "Developed Conditions" will be less than for the original natural conditions. This is because, under the current development proposals, the existing man-made "Overflow" channel (constructed in the early 1900's along the northern boundary of Council's STP) is being retained. This channel will continue to bypass some storm flow directly to the lower reach of Duchess Gully which would otherwise have flowed via the northern end of the Duchess Gully channel under original natural conditions.

4.3.9 Sensitivity Testing of Hydraulic Models

4.3.9.1 Storm Duration

The results reported above are for a 9 hour duration storm, which duration is critical for the whole catchment. Some sensitivity testing was carried out for shorter (6 hr) and longer (12hr) duration storm events. As expected, the attenuation achieved by the new lagoon was greater for the 6 hour storm event and less for 12 hour storm event. However as the peak discharges and levels were greatest for the 9 hour storm at all locations downstream of the water bodies, the results for the 9 hour duration storm event are relevant for the comparison of development cases.

4.3.9.2 Ocean Storm Surges – "Tailwater" Level in Duchess Gully

The tailwater level adopted for all the cases above was RL 2.0 mAHD. A case was run with a higher tailwater level of RL2.6 m AHD (equivalent to a 1% AEP storm surge level) for the proposed development with the constructed wetland. When compared with the 2 m AHD tailwater assumption, the constructed wetland water levels were found to be the same, and the peak level in Duchess Gully just downstream of the "overflow" (Duch-L 160) was found to be approximately 20 mm higher. This is an insignificant difference.

4.3.9.3 Low Tide Conditions

The mean high water spring tide for the ocean at this location is 0.65 m AHD. A case was run with this lower tailwater level. The wetland level was the same as for the high tailwater levels. The peak level in Duchess Gully downstream of the "overflow" junction (Duch-L 160) was found to be 14 mm lower, again an insignificant difference. It appears therefore that the tailwater level does not affect peak water levels upstream of the "overflow" junction of Duchess Gully.

4.3.9.4 Climate Change Impacts

It is a requirement of recent NSW state government policies (DECCW 2007, DECCW 2009d, DECCW 2009a, and DECCW 2009b) that planning and development assessments must include consideration of the potential impacts of recommended benchmark climate changes.

The relevant factors for the proposed development arising from climate change are:

- sea level rises
- catchment runoff flow rates
- catchment runoff volumes

Sea Level Rise

NSW State Government Sea Level Rise Policy (DECCW 2009d) requires that inundation risk assessment allow for a projected 0.9 metres rise in average sea level by 2100. This requirement is incorporated in other recent NSW guidelines including the Floodplain Risk Management Guideline Practical Consideration of Climate Change (DECCW 2007).

In accordance with this requirement, the MIKE11 storm event model was re-run with a boundary ocean level of RL 3.2, i.e. the adopted boundary level of RL 2.3 mAHD (see Section 4.3.7.6) plus a sea level rise of 0.9 metres.

The hydraulic controls incorporated into the lagoon and wetland systems which control the water levels in the water bodies during major floods are located at elevations of RL 3.5 mAHD and higher. An increase in the adopted sea level used in storm modelling to RL 3.2 mAHD should not materially alter the behaviour of the hydraulic systems in the development site because the raised sea level at the outlet of Duchess Gully is still below the hydraulic control structures.

Catchment Runoff Flow Rates and Rainfall Intensities

Catchment runoff rates are directly affected by rainfall intensities. Runoff flow rates will change in proportion to any change in rain intensities. Water levels in the streams bordering the development are in turn affected by the runoff flow rates.

It is often suggested that rainfall intensities, storm intensities and storm frequency will be affected by climate changes resulting from global greenhouse warming. All these effects are reflected in the "IFD" curves (Intensity Frequency Duration curves) derived from rainfall data. The authoritative reference for IFD curves is the publication "Australian Rainfall and Runoff" which is revised from time to time to allow for improved estimation procedures.

At the present time there are no proposals to amend the current recommended IFD curves to allow for climate changes apparently because trends in rainfall intensity changes are not evident in the data and are uncertain in prediction models.

The NSW "Floodplain Risk Management Guideline Practical Consideration of Climate Change" (DECCW 2007) recommends that flood modelling should include sensitivity testing with rainfall intensities increased by up to +30%.

These guidelines also quote recent estimates from the CSIRO of indicative changes in future extreme rainfall rates in the region of the site as listed in Table 56.

Table 56 Indicative Changes in Extreme 1-day Rainfall Totals

Catchment Location	2030	2070
Hunter – Central Rivers	-10% to +12%	-7% to +10%
Northern Rivers	-10% to +5%	+5% to +10%

Catchment Runoff Volumes

Catchment runoff volumes could also be affected by climate changes. The Bureau of Meteorology "trend maps" of rainfall totals from 1970 to the present for New South Wales show a decrease in wet days and heavy rain days. Other published trends show declines in average rainfall totals in several regions of Australia including NSW.

Increasing runoff volumes do not pose an increased risk of inundation because the water bodies are provided with overflows which rapidly discharge any excess runoff. The standing water levels in the water bodies are strictly limited by these overflows.

Increasing runoff volumes could lead to some increased accumulation of erosion damage in the water courses and streams in the Open Space Corridor.

Results for Storm Modelling with Climate Change

The MIKE11 model of "Developed Conditions" was run with a sea level of RL 3.2 mAHD and catchment model runoff hydrographs incorporating +30% greater rainfall intensities in order to estimate the sensitivity of maximum inundation levels to rainfall changes.

The resulting water levels are shown plotted in Figure 39. Inundation levels are increased with 30% greater rain but are everywhere still below the proposed reclamation fill levels which will be 0.5 metres higher than the estimated 100 Year ARI water levels.

The location most sensitive to rainfall increases within the proposed residential areas is along the western edge of the constructed wetlands (See Figure 39a). At this location the inundation level rises 368 mm with a 30% increase in rainfall rates.

Inundation immunity for floor levels and reclamation fill at this location were interpolated from this result as shown in Table 57.

Table 57 Floor Level Inundation Immunity with Rainfall Increase

Rainfall Increase	Floor Level Immunity (above inundation water level)	Reclamation Fill Level Immunity (above inundation water level)
0%	800 mm	500 mm
10%	677 mm	377 mm
24.5%	500 mm	200 mm
30%	432 mm	132 mm

These results show that:

- Floor level inundation immunity will be at least 677 mm for the most likely actual rainfall increase of +10% under climate change conditions.
- Floor level inundation immunity of at least 500 mm can be maintained even with rainfall intensity increases of 24.5%.

With the increased rainfall rates, there are increases in flows and velocities in the lower reach of Duchess Gully (from the constructed wetlands outlet to the ocean) which may resulting in some increased erosion damage during major flood events.

4.3.10 Stream Erosion in Duchess Gully – Developed Conditions

4.3.10.1 “Upper” Reaches

For developed conditions, the flows and velocities in the “upper” reach of Duchess Gully (DUCH-S Chainages 600 to 790) are increased by up to 38% from 0.4 m/sec to 0.55 m/sec. This increased velocity is still very low and not likely to cause any increased erosion in the stream.

4.3.10.2 “Lower” Reaches

The velocities for the 100 Year ARI storm event plotted in Figure 36 and Figure 37 in the “lower” reaches of Duchess Gully (DUCH-L Chainages 157 to 1410) are of the order of 1.5 metres/sec. These velocities are only 10% greater than existing velocities in this area (see Table 49) and are below the non-scouring limit for well-vegetated surfaces. Since the vegetation in this area has apparently survived existing stream velocities it is expected that there will be no increase in erosion in this area after development.

The flows and velocities in “lower” reaches of Duchess Gully between the new outlet structure and the junction with the “overflow” channel (DUCH-L Chainages 35 to 137.43) are increased five-fold because of the diversion of flows through the new constructed wetland but the absolute velocities are still only about 0.7 m/sec which is well below the non-scouring limit for well-vegetated surfaces and will not cause erosion in this area.

4.3.11 Summary of Hydraulic Modelling Results

Proposed urban development within the site will increase the quantity of runoff, increase the rate of runoff response and increase peak discharges from the developed areas for major storm events.

However, modelling has shown that it is possible to develop the site without external impact, (that is, without increasing flood levels or discharges upstream or downstream of the site) by changing the existing lagoon outlet geometry, and incorporating an additional constructed wetland of 10.5 hectares in the development.

It can be concluded that the hydraulic impacts of proposed urban development can be satisfactorily ameliorated by construction of an additional water body (the constructed wetland). Consequently these issues do not limit the development potential of the site.

4.4 Hydraulic Control Structures

The changes to the existing waterway features within the Open Space Corridor incorporated in the development proposals are summarised as follows:

- Realignment and widening of the channel of the Upper Tributary of Duchess Creek over a channel length of 170 metres; the constructed channel will have a bed width of one metre (see Figure 7).
- Two new open channel weir structures at the eastern end of the existing lagoon; one weir to control flows from the existing lagoon into a new proposed constructed wetland (see Figure 33), and the other weir to control flows from the existing lagoon to the existing “overflow” channel connecting direct to the “lower” reach of Duchess Gully (see Figure 31).
- A regraded embankment around the eastern edge of the existing lagoon to control major storm event overflows (see Figure 31).

- A new 10.5 hectare constructed wetland in the “middle” reach of Duchess Gully (see Figure 8).
- A constructed “base flow” channel between the constructed wetland and Duchess Gully to maintain the standing water level of RL 3.0 in the constructed wetland (see Figure 30).
- An open channel weir structure on the eastern edge of the constructed wetland to control minor storm event flows from the constructed wetland to Duchess Gully (see Figure 28). This structure incorporates special protection for the existing underground services pipelines running alongside Duchess Gully.
- A regraded embankment along the eastern edge of the constructed wetland to control major flood overflows into Duchess Gully (see Figure 31). This embankment incorporates the minor event control weir above.

These structures will be designed in accordance with the latest engineering standards and regulations to withstand the impacts of the 100 Year ARI design event without damage and they will incorporate features designed to enhance the desirable ecological functions of Duchess Gully including vegetation rehabilitation and fauna movement in accordance with NSW guidelines (DPI 1999). Typical details are shown in the accompanying Figures.

The flow control structures have been designed in accordance with the requirements of the Port Macquarie Hastings Council's AUS-SPEC drainage guidelines and the NSW Floodplain Management Manual.

The impacts of these flow controls on stream flow hazard and risk conditions throughout the Open Space Corridor are comprehensively reviewed in Section 4.7.

4.5 Local Drainage - Design Tailwater Levels

The stormwater drainage systems within the proposed urban residential areas are designed for peak flow rates generated by short duration storms.

The water level in the downstream receiving water bodies (the existing lagoon and proposed constructed wetland) will have risen only marginally at the time of peak flow in the residential drains because the response time for the whole catchment is greater than four hours.

As a consequence, it is sufficient to adopt a tailwater level of RL 3.6 mAHD for the design of the residential drain systems. This level is 0.1 metres higher than the standing water level in the stormwater treatment wetlands into which the drains discharge.

4.6 Catchment Flow Regimes

Three flow regimes can be distinguished within the Open Space Corridor for the purpose of designing flow control structures and assessing storm hazard risk as follows.

4.6.1 Base flows

Base flows represent the persistent flow in Duchess Gully in the absence of rainfall runoff. These flows are important for ecological functioning such as fish migration.

The proposed development restores the base flow path in Duchess Gully to its original location through the “middle” reaches of the Gully as shown in Figure 5 and Figure 6.

4.6.2 Minor Event Flows

Under the Port Macquarie Hastings Council's drainage policy, based on the "minor/major" design method of stormwater controls, "minor" event flows include runoff flows up to and including the 5 Year ARI event. These flows should be contained within well-defined drainage structures with no overland flow.

The proposed flow control structures within the residential areas and the Open Space Corridor are designed to achieve this objective. Within residential areas, minor flows are carried in underground pipes. Minor flows are discharged from the existing lagoon and the proposed constructed wetland within the Open Space Corridor via well-defined weir structures with flows downstream from the structures confined in channels with erosion protection.

The locations of the proposed structures and the minor event flow paths are shown in Figure 5 and Figure 6.

4.6.3 Major Event Flows

Council's drainage policy also requires that overland flows occurring during runoff events exceeding the 5 Year ARI event must be conveyed via well-defined pathways such that flow conditions comply with the NSW Floodplain Management Manual (DIPNR 2005) including requirements that:

- Housing must be located outside of major flow paths;
- Surfaces are to be designed to withstand the 100 Year ARI flow conditions without erosion damage.

The proposed development satisfies these requirements. Major flows within residential areas are carried in grassed swales at non-scouring velocities. Overland velocities within the Open Space Corridor are less than the non-scouring velocity. The embankments around the existing lagoon and the constructed wetland are designed so that major storm event flows up to and including the 100 Year ARI event will overtop these embankments at heads of less than 0.5 metres and velocities not exceeding the non-scouring velocity for grassed surfaces of 1.8 metres/sec.

The major stormwater and flood flow paths are shown in Figure 36 and Figure 37.

Major event conditions and the requirements of the Floodplain Management Manual are discussed in more detail in the following Section 4.7.

4.7 Floodplain Management

4.7.1 Regulatory Requirements (Floodplain Management Manual)

DGR CP4.4 requires an assessment of any flood risk on site under any relevant provisions of the NSW Floodplain Management Manual (DIPNR 2005).

The site is not mapped as "flood-prone" (as defined in the Interim Flood Policy of April 2007) on the Section 149 Planning Certificate issued for the site.

4.7.2 Development Strategy

Although the development site is not classed as “flood prone”, the flat low-lying nature of parts of the site gives rise to occasional inundation from local severe storm events at some locations. As a result, the development has been conservatively designed so as to meet all of Council’s usual requirements and performance criteria for flood-prone land.

The resulting objectives of this section of the assessment are to:

- Demonstrate that the development proposals satisfy the requirements for flood-prone land.
- Demonstrate that the development satisfies the requirements of the Floodplain Management Manual.
- Demonstrate that the proposed development minimises inundation risk.

To meet these objectives it is necessary for this assessment to:

- Assess storm event and inundation risks within the development site area allowing for site-specific factors.
- Document features of the proposed development designed to mitigate risks and enhance the management of high rainfall risks.
- Document management options appropriate to the proposed development in a form suitable for incorporation in a Floodplain Risk Management Plan.
- Estimate likely costs of risk management including operational and maintenance costs.

Operational and maintenance costs are dealt with further in Section 5.8.

A Floodplain Risk Management Plan must address:

- Existing stormwater or inundation risk for existing development;
- Future risk for likely proposed development;
- Continuing risks after allowing for the effects of any structural stormwater or flood controls.

It is not within the scope of this assessment document or within the scope of development proposals to prepare a Floodplain Risk Management Plan. This assessment does however assist with all the necessary background data required for such a risk management plan including:

- Stream velocity and depth hazards;
- Inundation extents;
- Housing inundation immunity; and
- Evacuation routes.

4.7.3 Development Impacts on Floodplain Risk Management Issues

4.7.3.1 Existing Development Inundation Immunity

It is shown in Section 4.3 that inundation levels in existing residential areas along the southern boundary of the site on Duchess Gully are not worsened by the proposed development.

4.7.3.2 Development Reclamation (Fill) Levels

Inundation levels in Duchess Gully for the 100 Year ARI event (1% AEP) are calculated in Section 4.3 and are shown plotted in Figure 36 and Figure 37. Council's Interim Flood Policy requires that the minimum level of residential lots must be at least equal to these 100 Year ARI levels.

Reclamation fill levels in the proposed residential areas will be constructed to a freeboard of 0.5 metres above these 100 Year ARI event water levels or higher. This conservative approach minimises inundation risk beyond the normal required standard.

Residential lots will also be constructed to a freeboard of 0.5 metres above local 100 Year ARI water levels for drainage systems designed in accordance with Port Macquarie Hastings Council requirements under AUS-SPEC 2003.

Road crossings over flow paths within the proposed development will be designed to provide 100 Year ARI event immunity.

It is a requirement of Council's Interim Flood Policy that house floor levels be constructed with a freeboard of 0.8 metres above the 100 Year ARI event water levels. This requirement ensures that house floor levels will generally be higher than the surrounding allotment fill levels.

4.7.3.3 Evacuation Access Routes

The contours for the proposed development plotted in Figure 7 and Figure 8 show that for all significant rainfall events there are inundation-free evacuation routes via the internal road system to higher ground along Ocean Drive.

Local roads may be partly inundated during major storm events to the extent allowed under drainage design limits set out in AUS-SPEC 2003.

4.7.3.4 Public Safety- Flood Hazard for Development Conditions

Inundation hazard conditions expressed as depth and velocity for the 100 Year ARI as defined in the Floodplain Management Manual are shown plotted in Figure 36 and Figure 37. This figure also shows the 100 Year ARI water levels throughout the Open Space Corridor and the major stormwater flow paths.

The plotted data shows that the Open Space Corridor is a "hydraulic floodway" or flow path throughout its length but conditions of velocity and depth fall into the "low hazard" category everywhere in the low-lying areas except for the following areas categorised as "high hazard":

- the channel proper of Duchess Gully where depths may exceed two metres and velocities may exceed 1.0 metres/sec in the "lower" reaches;
- the water bodies of the existing lagoon and the constructed wetland where depths exceed two metres;
- at the control structures on the existing lagoon and the constructed wetland and immediately downstream of these structures;
- at the overflow sections of the embankments surrounding the existing lagoon and the constructed wetland.

These “high hazard” areas are shown shaded in Figure 36 and Figure 37. “High hazard” areas may need to be suitably signed to alert pedestrian and cycling traffic. Public risk issues within the Open Space Corridor are discussed further in Section 5.6.

The data plotted in Figure 36 and Figure 37 also show that flow paths within the residential areas are “low hazard hydraulic floodways”.

There are no areas of “flood storage” or “flood fringe” areas in the proposed development site.

There are no levees in the proposed development since all residential areas are reclaimed to heights above the designated inundation levels.

There are no island areas within the proposed residential development areas.

There is an island within the Open Space Corridor being the pronounced hill at the south-east corner of the site which will become isolated during storm events exceeding the 5 Year ARI event which overtop the water body embankments. No residential areas are proposed for that area but it is accessible from pedestrian footpaths proposed for the Corridor and could result in people becoming isolated here. The resulting risk is low because this hill does not become submerged even during extreme rainfall events (see Section 4.7.3.7).

4.7.3.5 Future Development Inundation Risk

Inundation risk within the development site is not likely to be increased with further future development in external catchment areas west of Ocean Drive because it is feasible to apply a development condition to these areas requiring that peak storm runoff rates are not increased by development. This condition is incorporated in Council’s flood policies.

4.7.3.6 Continuing Inundation Risk

There is no continuing flood/inundation risk under the proposed development because all residential areas are reclaimed to heights above the designated levels.

4.7.3.7 Probable Maximum Flood (PMF) Conditions

PMF conditions were calculated in accordance with procedures given in Bureau of Meteorology publications BOM (2003a) and BOM (2003b).

The estimated 9 hour duration PMP rainfall intensity was 100 mm/hour and the estimated peak flow rate for the PMF at the outlet of Duchess Gully was 208 m³/sec.

At this flow rate water levels are governed by the hydraulic capacity of the embankments surrounding the existing lagoon and the constructed wetland. The resulting maximum inundation levels are shown in Table 58.

Table 58 PMF Maximum Water Levels

Location	Peak PMF Flood Level mAHD
Existing Lagoon	5.5
Constructed Wetland	4.8

Inundation levels in the “upper” reach of Duchess Gully and the Upper Tributary have not been calculated in detail but they are likely to be 0.6 to 1.0 metres higher than the levels shown in Figure 36 and Figure 37. At these flow rates and water levels, the average water depth in the low-lying areas increases by 0.6 metres to about 1.0 metre and the velocities increase by 0.5 m/sec to about 0.8 m/sec. The hazard (depth x velocity) under these conditions generally falls within the “high hazard” category.

Much of the proposed development will still be free of inundation at these water levels and access paths to higher ground will still be available.

4.7.3.8 Site Maintenance

The control structures within the Open Space Corridor will be designed to withstand the 100 Year ARI event conditions without damage. General details are shown in Figure 28 to Figure 33.

Flow velocities for the 100 Year ARI event are generally well below the non-scouring limit for grassed surfaces and will sustain no significant damage for storm events up to and including the 100 Year event.

Some damage may result from storm events exceeding the 100 Year ARI event.

Maintenance requirements to reinstate damage following these events are discussed in more detail in Section 5.8.

4.8 Acid Sulfate Soils

4.8.1 Council Section 149 Planning Certificate and DGR Provisions

The Section 149 Planning Certificate issued for the site indicates that management of acid sulfate soils (ASS) may be a potential problem.

DGR PA6.3 requires that the identification and management of acid sulfate soils be assessed.

4.8.2 Site Investigations for Acid Sulfate Soils (ASS)

A number of site investigations have been undertaken by Holmes & Holmes Pty Ltd as follows –

Table 59 ASS Site Investigations

Date	Bore ID	Reference	Type	ASS Test Type
25-11-93	BH1 – BH26	Holmes & Holmes (1993)	Backhoe Pits	Field Tests
“	BH27 – BH40	Holmes & Holmes (1993)	Auger	Field Tests
15-12-98	101 – 106	Holmes & Holmes (1998)	Auger	Laboratory Tests
26-11-02	W1 – W13	Holmes & Holmes (2002)	Auger	Bore log soil description
21-3-03	107 – 111	Holmes & Holmes (2003b)	Auger	Laboratory Tests

Date	Bore ID	Reference	Type	ASS Test Type
3-4-03	29448 – 29498	Holmes & Holmes (2003c)	Auger	Bore log soil description
14-4-04	1 – 4	Chandler (2004)	Auger	Laboratory Tests

The borehole locations are shown in Figure 14.

The results of these investigations indicate –

- ASS and PASS soils occur throughout the area of the proposed constructed wetlands.
- There is a clear demarcation between surface soils with “low” potential acidity (PASS) requiring little or no treatment and soils at deeper levels with “high” potential acidity requiring management and significant treatment.
- This boundary corresponds consistently with the occurrence of distinctive deeper grey clays of estuarine origin.
- The maximum calculated required liming treatment rate (the ANC or Acid Neutralising Capacity) for the “low PASS” materials was 2.0 kgs/m³. This treatment rate includes liming requirements for actual acidity (ASS) but did not include any multiplying “safety margin” factor (usually 1.5).

4.8.3 Implications for Development Proposals

The occurrence of “high PASS” soils only at lower subsurface levels with a well-defined boundary suggests the following management strategies for the proposed constructed wetlands –

- Excavation for the wetlands will avoid the “high PASS” material which is best left undisturbed.
- Excavation can be carried out “in the dry” with appropriate dewatering of each excavation stage since dewatered areas will all lie within the upper strata of “low PASS” soil zones resulting in minimal acid generation during construction.
- Drawdown of the water table during excavation operations should be minimised in extent and duration. Water table levels during excavation should not be reduced below the “high PASS” boundary. Provided this condition is met, acid generation within zones of dewatered soil will be minimal because of the low levels of potential acidity in these zones.
- Excavated material must be handled and placed in accordance with statutory regulations for management of acid sulfate soils (see Section 4.8.6 and Appendix A).
- Permanent water table levels should be maintained as high as practicable and at least as high as the “high PASS” soil boundary. Water levels within the wetlands should be based on this criterion. It is expected that the permanent wetlands water level will be RL 3.0 mAHD, which satisfies this criterion. Even during drought conditions the water level is not expected to fall below RL 2.1 (0.9 metres fall) at which the water table is still above the high-PASS material.

4.8.4 Defining Constructed Wetlands Excavation Based on “Low” PASS Soils

The boundary between “low PASS” soils (requiring minimal or no treatment) and “high PASS” soils (requiring significant treatment) was calculated for all relevant boreholes in the eastern part of the site (where the constructed wetlands are located) as reduced levels on AHD and represented as a continuous “surface”.

The resulting boundary between soils of “low PASS” requiring minimal treatment and those of “high PASS” requiring significant treatment varies in level between RL 0.1 mAHD and RL 2.0 mAHD within the area of the proposed constructed wetlands but is generally of the order of RL 1.25 mAHD.

This subsurface boundary was then used to define the bed of the constructed wetlands as shown in Figure 15. Limiting the bed of the wetland to be no deeper than this boundary means that “high” PASS soils are avoided entirely and disturbance of acid sulfate soils is minimised. Excavated material from the constructed wetlands is expected to show very low acid potential and require no or minimal treatment.

A constructed wetland configured as shown in Figure 15 provides sufficient material for filling the reclaimed areas in residential zones of the proposed development. Table 60 lists the volumes required.

Table 60 Earthworks Volumes

Location	Volume m ³
FILL REQUIREMENT	
Northern School Site – Site 1	30,640
North West Fill Area – Site 2	172,200
North East Fill Area – Site 3	182,700
District Sporting Fields – Site 4	20,400
Perimeter roads – outer batter	11,250
TOTAL VOLUME FILL REQUIRED Measured dimensions	417,200
EXCAVATION	
Constructed Wetlands Zone plus topsoil stripped from fill areas – Measured dimensions	518,480
Net consolidated material available to be utilised for fill, after allowances for compaction losses of 20% for structural fill and 15% for non-structural fill respectively	418,800

4.8.5 PASS Material within Groundwater Drawdown Zone

It was shown in Section 2.11.9.2 that permanent drawdown of the water table on the north-west side of the constructed wetlands exposes only very low PASS concentration soils to atmospheric oxidation. These soils are not expected to require any treatment under the management guidelines and exposure to the air will not generate acidity in seepage flows likely to adversely affect the water quality in the water bodies.

4.8.6 ASS Management Procedures

Because of the presence of acid sulfate conditions within the site, it is proposed that earthworks operations be carried out in strict accordance with appropriate management procedures complying with statutory requirements in New South Wales.

A suitable Acid Sulfate Soil Management Plan (ASSMP) is included as Appendix A.

The soil assessments carried out to date are adequate to satisfy the requirement of the Hastings Local Environment Plan 2001 that consent to the proposals must be informed by suitable preliminary soil assessments.

4.8.7 Soil Treatment Rates

Acid sulfate soil encountered during excavation will be treated using lime admixture during placement of fill. This method is appropriate and economic for the dry excavation method proposed and the levels of soil treatment indicated.

The actual liming rates will be dictated by testing procedures given in the ASSMP (Appendix A).

It is recommended that lime treatment be applied immediately to any excavated material because this material is likely to be sandy in nature allowing acid conditions to develop readily.

Initial liming rates to be used until test results from the site become available (see the ASSMP procedures) can be based on the site investigations to date summarised in Section 4.8.1. These indicate that a liming rate (ANC) of 2 kg/m^3 is sufficient for the worst of any "low PASS" soil likely to be encountered.

It is further recommended that a "safety margin" factor of 2.0 should be applied in this case because the site investigation results are based on superseded analysis procedures relative to current procedures. The initial actual applied liming rate should therefore be 4 kg/m^3 .

It is expected that the bulk of excavated material may require little or no treatment. A liming rate of 4 kg/m^3 must be applied unless actual soil testing results during the construction process indicate that a lesser rate is sufficient.

4.8.8 Construction Phase ASS Management Plan

4.8.9 pH Monitoring of Construction

Excavation dewatering will be limited to the upper soil strata and "high PASS" materials will remain submerged at all times. As a result, acid generation within zones of dewatered soil will be minimal because of the very low levels of potential acidity in these zones.

It is recommended that groundwater in the dewatered zone adjacent to excavations be monitored for acidity. There are provisions in the ASSMP (Appendix A) for this.

4.9 DGR Item Responses

RAINBOW BEACH CONCEPT PLAN (CP)

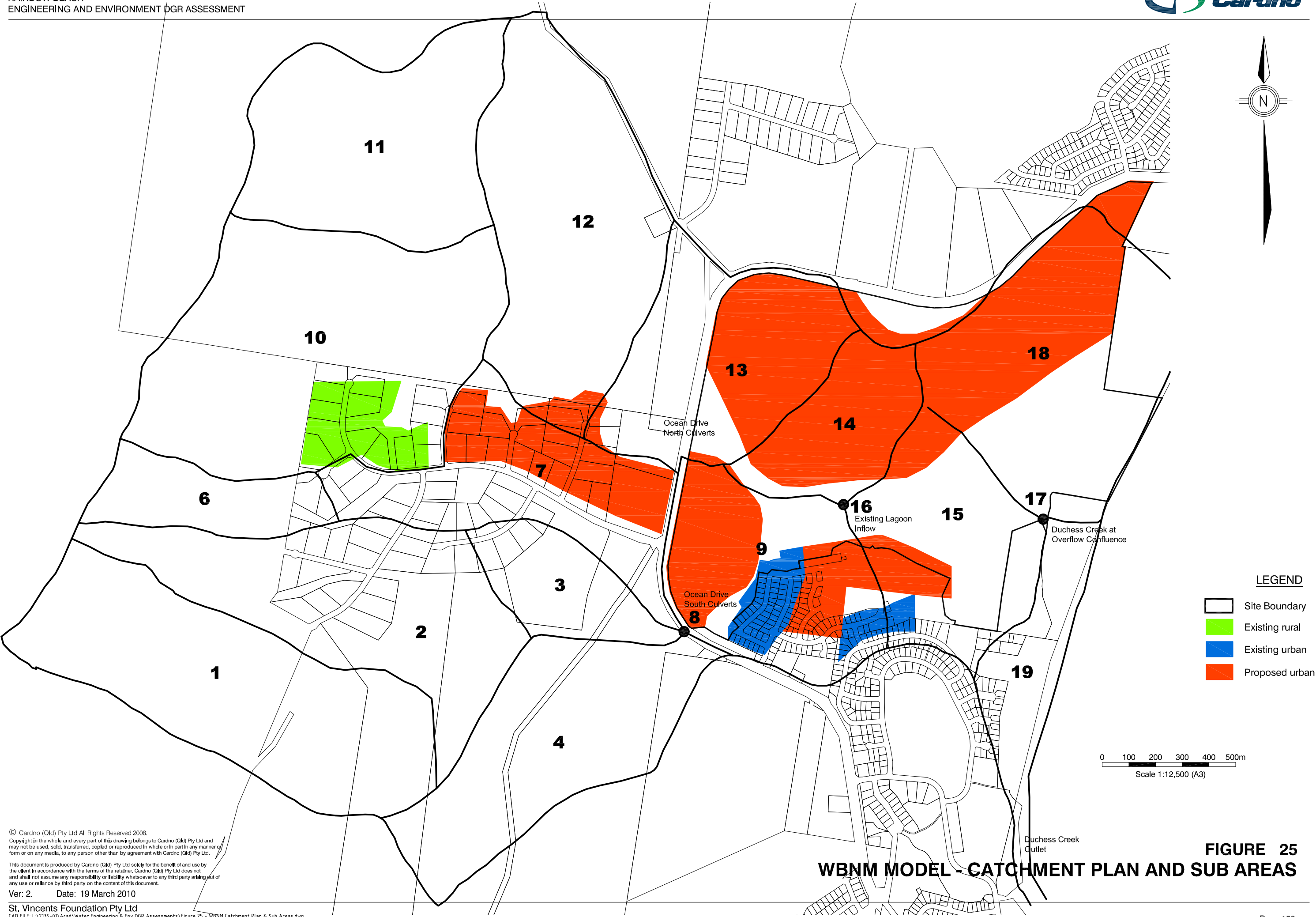
Item	Topic
CP4.3	Identify the presence and extent of acid sulphate soils on the site and if necessary, appropriate mitigation measures.
Response: Potential acid sulfate soils on the site have been identified by geotechnical survey. ASS and PASS soils occur throughout the area of the proposed constructed wetlands. There is a clear demarcation between surface soils with "low" potential acidity (PASS) requiring little or no treatment and soils at deeper levels with "high" potential acidity requiring management and significant treatment (Section 4.8.2). Appropriate management and mitigation measures are described (Section 4.8.3) including: - Excavation for the wetlands will avoid the "high PASS" material which will be left undisturbed. - Excavation "in the dry" with appropriate dewatering of each excavation stage resulting in minimal acid generation during construction. - Excavated material will be handled and placed in accordance with an ASS Management Plan complying with statutory regulations for management of acid sulfate soils (see Section 4.8.6 and Appendix A). Lime treatment will be used where required - Permanent water table levels will be maintained as high as practicable and at least as high as the "high PASS" soil boundary. Water levels within the constructed wetlands will be RL 3.0 mAHD which satisfies this criterion.	
CP4.4	Provide an assessment of any flood risk on site in consideration of any relevant provisions of the NSW Floodplain Management Manual. Consider the potential impacts of any filling on the flood regime of the site and adjacent land.
Response: Flood risk in terms of the NSW Floodplain Management Manual 2005 was assessed using a MIKE11 hydraulic model to simulate storm events for both existing and development conditions. The effect of proposed fill was incorporated in the model. (Section 4.3) Water levels are not worsened by the proposed development for the 100 year ARI Designated Event except for an increase of 110 mm at the boundary of the adjoining Milland land to the north-east of the site (Chainage 835 DUCH-N). This increase reduces to zero within a short distance further upstream from the property boundary (Section 4.3.8.5 and 4.3.8.9). A number of flow control structures and channel improvements are included in the development proposals. It is demonstrated that these controls comply with safety requirements of the Floodplain Management Manual (Sections 4.3.8 and 4.4) Proposed hydraulic structures have been designed to minimize storm damage and maintenance costs and enhance ecological functioning including fish migration (Section 4.4). The possible effects of climate change have been assessed and it is demonstrated that climate change does not adversely affect the development (Section 4.3.9.4) Floodplain risk management factors have been assessed and it is shown that hazard conditions are low (Section 4.7)	

PROJECT APPLICATION – OPEN SPACE CORRIDOR & CONSTRUCTED WETLAND (PA)

Item	Topic
PA5.3	Provide an assessment of any flood risk on site in consideration of any relevant provisions of the NSW Floodplain Development Manual (2005). Consider the potential impacts of any filling on the flood regime of the site and adjacent lands. Response: Flood risk in terms of the NSW Floodplain Management Manual 2005 was assessed using a MIKE11 hydraulic model to simulate storm events for both existing and development conditions. The effect of proposed fill was incorporated in the model. (Section 4.3) Water levels are not worsened by the proposed development for the 100 year ARI Designated Event except for an increase of 110 mm at the boundary of the adjoining Milland land to the north-east of the site (Chainage 835 DUCH-N). This increase reduces to zero within a short distance further upstream from the property boundary (Section 4.3.8.5 and 4.3.8.9). A number of flow control structures and channel improvements are included in the development proposals. It is demonstrated that these controls comply with safety requirements of the Floodplain Management Manual (Sections 4.3.8 and 4.4) Proposed hydraulic structures have been designed to minimize storm damage and maintenance costs and enhance ecological functioning including fish migration (Section 4.4). The possible effects of climate change have been assessed and it is demonstrated that climate change does not adversely affect the development (Section 4.3.9.4) Floodplain risk management factors have been assessed and it is shown that hazard conditions are low (Section 4.7)
PA6.3	Identify the presence and extent of acid sulphate soils on the site and if necessary, appropriate mitigation measures. Response: Potential acid sulfate soils on the site have been identified by geotechnical survey. ASS and PASS soils occur throughout the area of the proposed constructed wetlands. There is a clear demarcation between surface soils with "low" potential acidity (PASS) requiring little or no treatment and soils at deeper levels with "high" potential acidity requiring management and significant treatment (Section 4.8.2). Appropriate management and mitigation measures are described (Section 4.8.3) including: • Excavation for the wetlands will avoid the "high PASS" material which will be left undisturbed. • Excavation "in the dry" with appropriate dewatering of each excavation stage resulting in minimal acid generation during construction. • Excavated material will be handled and placed in accordance with an ASS Management Plan complying with statutory regulations for management of acid sulfate soils (see Section 4.8.6 and Appendix A). Lime treatment will be used where required • Permanent water table levels will be maintained as high as practicable and at least as high as the "high PASS" soil boundary. Water levels within the constructed wetlands will be RL 3.0 mAHD which satisfies this criterion.

FIGURES SECTION 4

- Figure 25 WBNM Model Catchment Plan and Subareas
- Figure 26 MIKE11 Model Network Layout Plan – Existing Conditions
- Figure 27 MIKE11 Model Network Layout Plan – Proposed Development Conditions
- Figure 28 Constructed Wetlands – Outlet Control Structure S5 Plan
- Figure 29 Constructed Wetlands – Outlet Control Structure S5 Sections
- Figure 30 Constructed Wetlands – Base Flow Control Channel S4 Plan and Details
- Figure 31 Existing Lagoon – “Overflow” Outlet Control Structure S3 Plan
- Figure 32 Existing Lagoon – “Overflow” Outlet Control Structure S3 Details
- Figure 33 Existing Lagoon – “Connection” Control Structure S2 Plan and Details
- Figure 34 Storm Event – Discharge Hydrographs
- Figure 35 Storm Event – Water Level Profile Alignments (Parts A, B and C)
- Figure 36 Storm Event – Water Levels – 100 Year ARI Conditions (Parts A, B and C)
- Figure 37 Storm Event – Hazard Conditions Plan – 100 Year ARI Conditions
- Figure 38 Storm Event – Water Levels on Adjoining Milland Property
- Figure 39 Storm Event – Water Levels – Climate Change Sensitivity (Parts A, B and C)



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CAD FILE: L:\7135-01\Acad\Water Engineering & Env DGR Assessments\Figure 25 - WBMM Catchment Plan & Sub Areas.dwg
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FIGURE 26

MIKE 11 MODEL NETWORK LAYOUT PLAN - EXISTING CASE

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CAD FILE: L:\1155-01\Acad\Water Engineering & Env DGR Assessments\Figure 26 - MIKE Model Existing.dwg

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FIGURE 27
MIKE 11 MODEL NETWORK LAYOUT PLAN - DEVELOPED CASE

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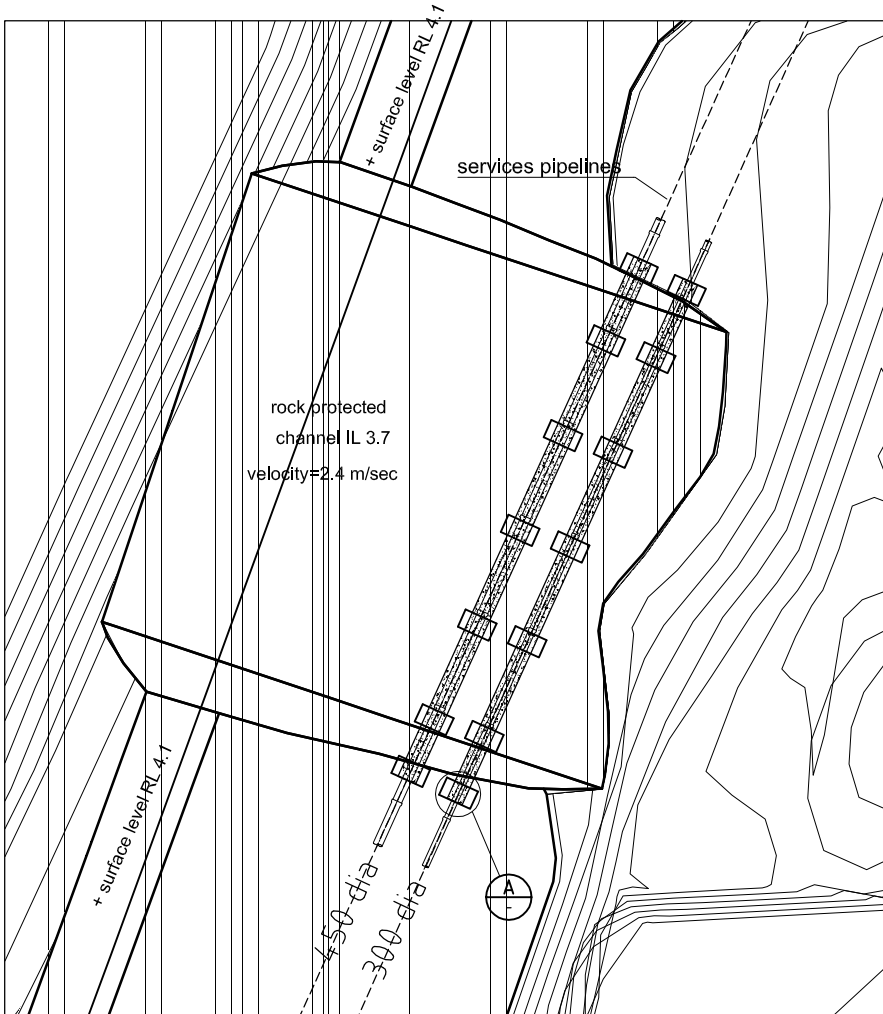
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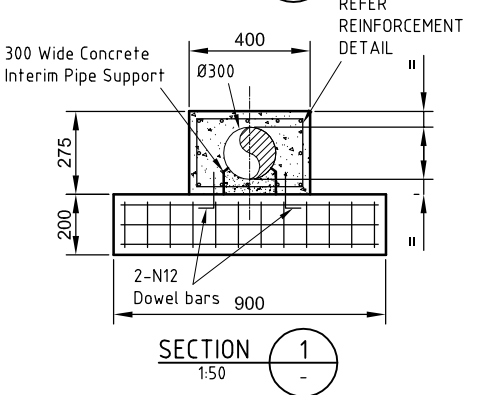
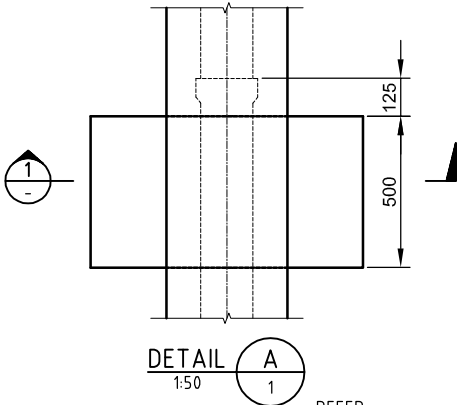


Refer to Figure 29 for
Detail of cross sections

PLAN - CONSTRUCTED WETLAND OUTLET
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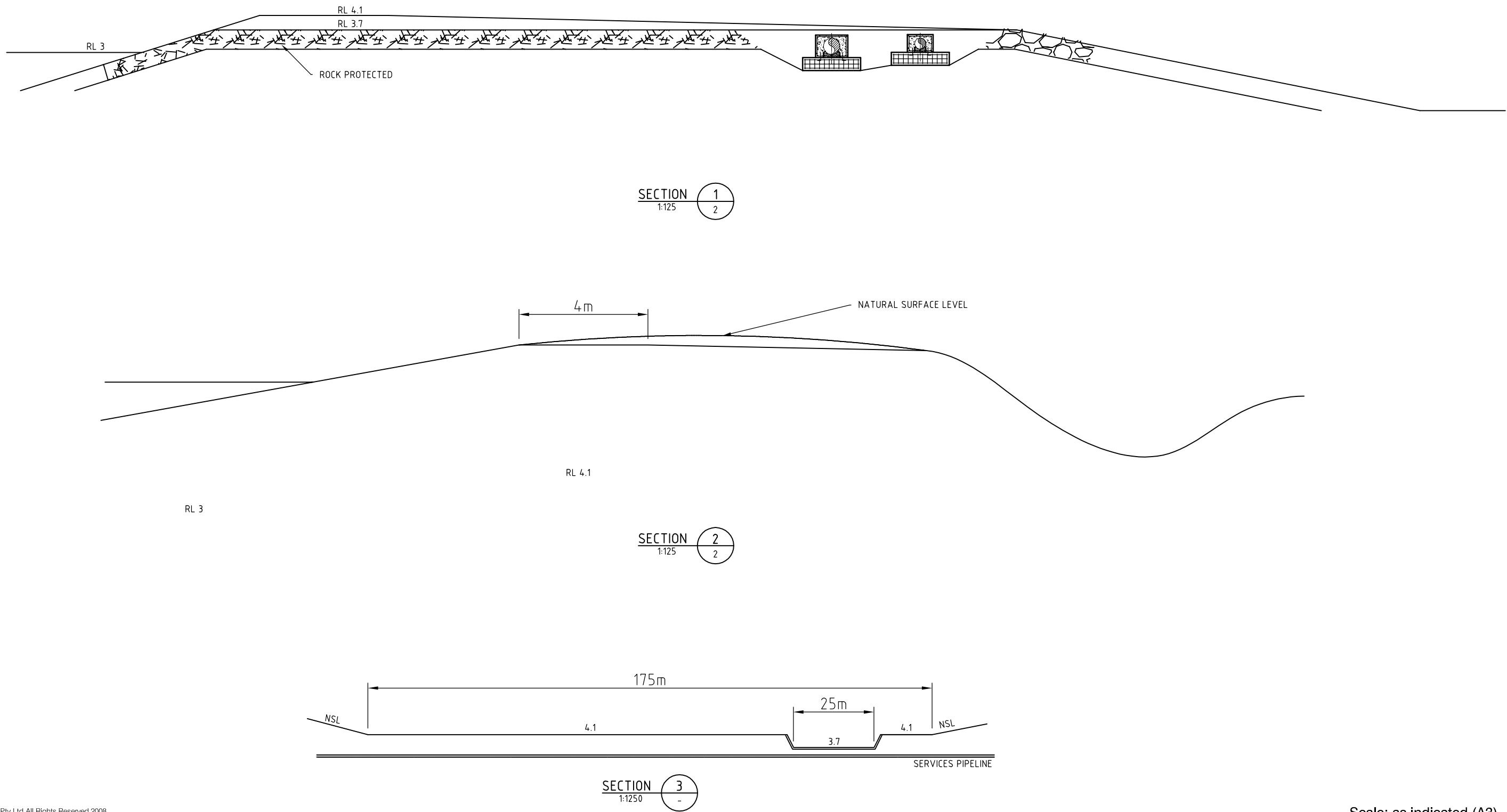


PLAN - SERVICES PIPELINE CROSSING
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FIGURE 28
CONSTRUCTED WETLANDS OUTLET CONTROL STRUCTURE S5 PLAN



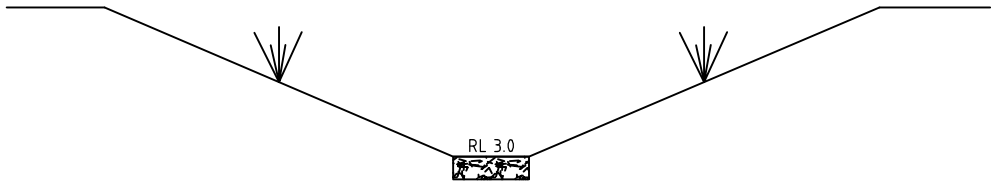
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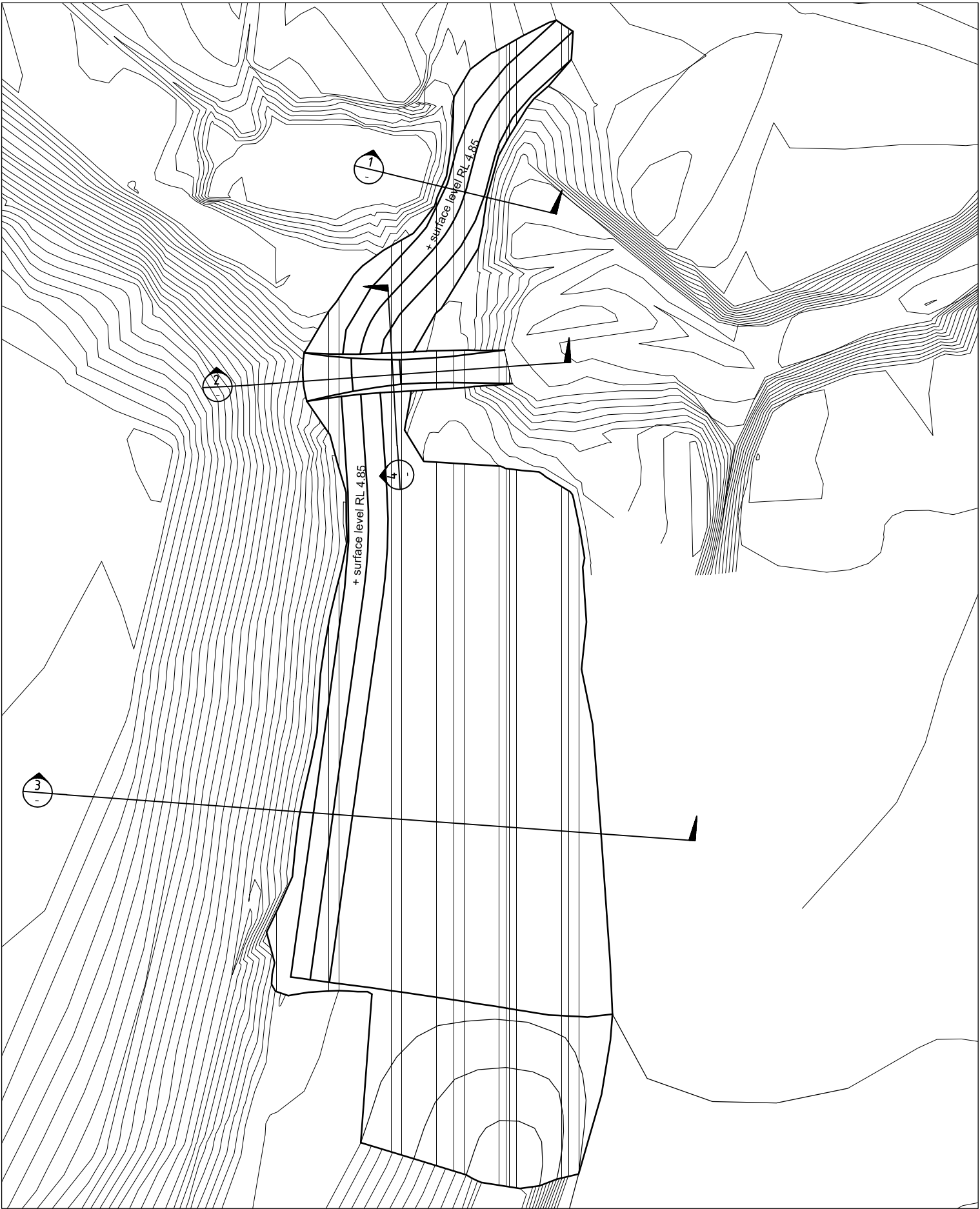
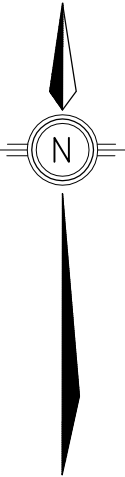
FIGURE 29
CONSTRUCTED WETLANDS OUTLET CONTROL STRUCTURE S5 SECTIONS



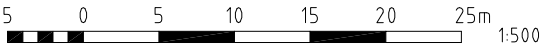
SECTION 1
1:50

PLAN - CONSTRUCTED WETLAND BASE FLOW OUTLET S4 CHANNEL
1:500

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FIGURE 30
CONSTRUCTED WETLANDS BASE FLOW OUTLET STRUCTURE S4 CHANNEL



Refer to Figure 32 for
Detail of cross sections



Scale 1:500 (A3)

FIGURE 31

EXISTING LAGOON "OVERFLOW" OUTLET CONTROL STRUCTURE S3 PLAN

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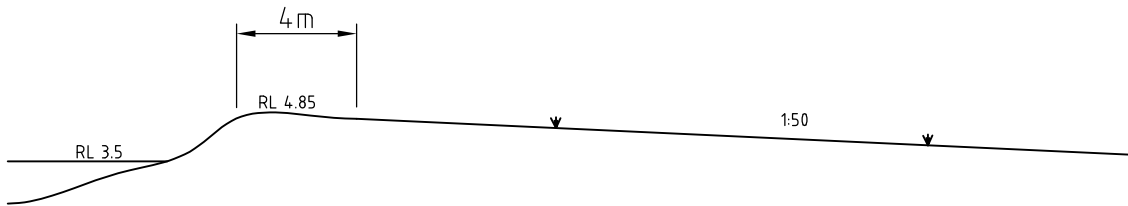
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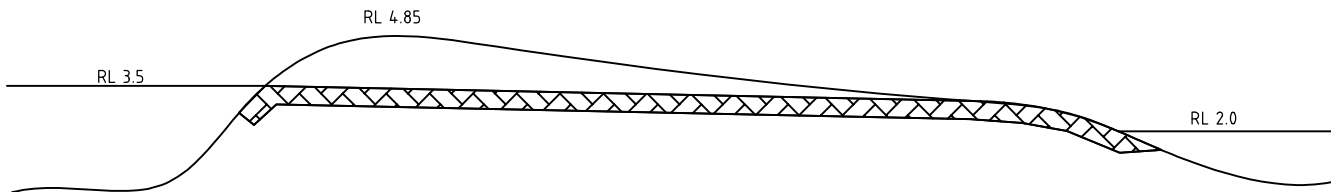
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SECTION 1
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SECTION 3
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SECTION 3
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SECTION 2
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SECTION 2
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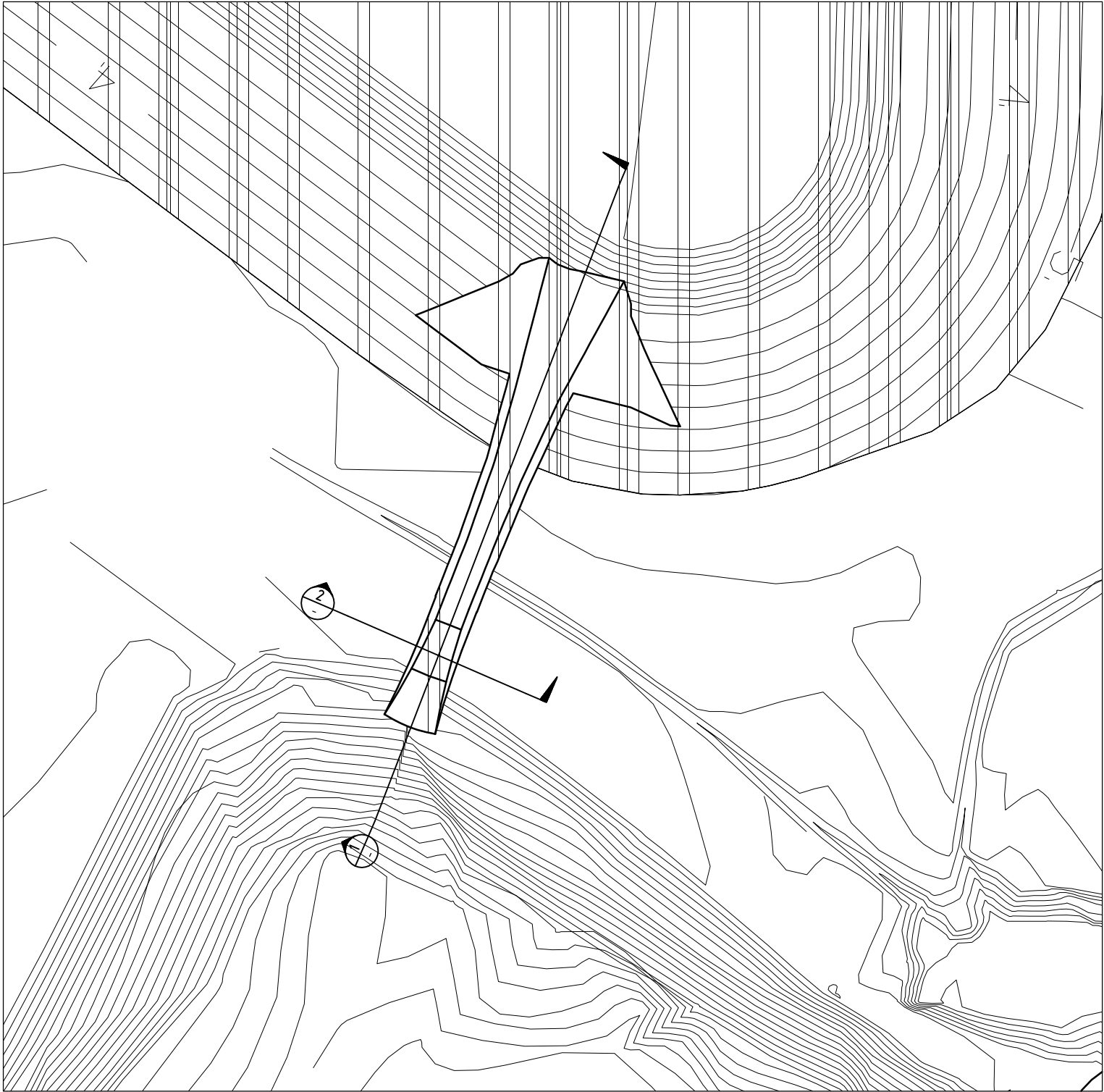


SECTION 4
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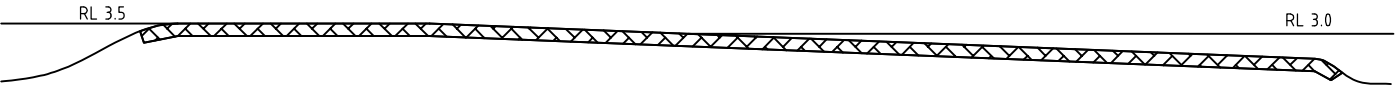
SECTION 4
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FIGURE 32
EXISTING LAGOON "OVERFLOW" OUTLET CONTROL STRUCTURE S3 - SECTIONS

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PLAN - "CONNECTION" CONTROL STRUCTURE S2
1:500



SECTION 1
1:300



SECTION 2
1:250

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FIGURE 33
"CONNECTION" CONTROL STRUCTURE S2 PLAN AND DETAILS

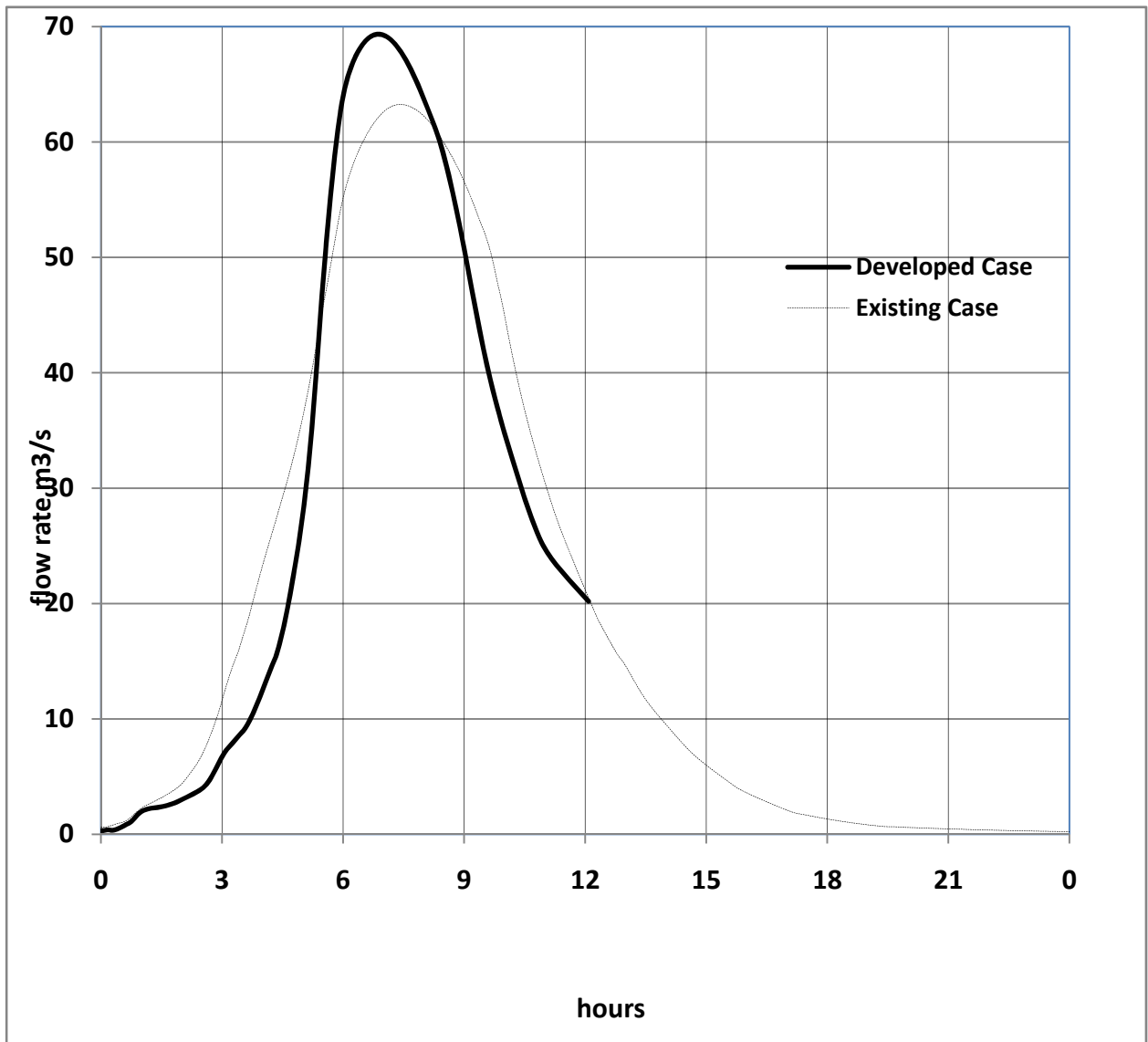


Figure 34 Storm Event – Discharge Hydrographs

FIGURE 34