NL070295

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04/05/2009

Mr David Humphris
de Witt Consulting
7 Canberra Street
CHARLESTOWN NSW 2290

Dear David,

Re: Lot 9 and Lot 358 Morisset Park Road, Morisset Park – Response to Council and DOP Comments

We provide the following information to address Lake Macquarie City Council's (LMCC) concerns raised in their letter dated 30.01.09. The information provided has taken into account discussions and a subsequent meeting with Council Officers David Pavitt and Greg Fitzpatrick. The information below should be read in conjunction with the attachments and the *'Concept Stormwater Management Plan and Road Design'* report produced by Northrop Engineers in April 2008.

The number points shown in bold have been taken from Council's letter dated 30.01.09.

> ***7. There is additional catchment to the south of Morisset Park Road that has not been mentioned in the report. An existing drainage pipe/s directs runoff onto the site from the catchment at present and should be addressed in the report.***

From detailed survey (refer Attachment 1 for contours) Morisset Park Road falls to the east and west from a high point of approximately 25m AHD located at the proposed access point to the subdivision. Runoff from southern catchments is

currently collected in a formed table drains which run parallel to the road. The catchment of the table drain running to Chifley Road has been estimated as 14,600sq.m from topographic maps. In front of the proposed Lot 29 a pit currently collects runoff from roughly half of this catchment and diverts it to the northern side of the road via a headwall.

The *'Concept Stormwater Management Plan and Road Design'* report noted:

> "…Flow from Morisset Park Road and upstream catchments have not been included in the modeling as it is understood that those portions of Morisset Park Road and Chifley Road which front the site will be provided with kerb and gutter and an underground pit and pipe stormwater system. As such Morisset Park Road will collect stormwater from upstream catchments and divert it around the site…"

As such new kerb and gutter, and associated stormwater drainage, will be designed and constructed immediately in front of the site along the eastbound lane of Morisset Park Road to cater for runoff collected from southern catchments. While this stormwater system will be fully designed at Construction Certificate stage preliminary calculations (refer Attachment 2) estimate that the peak 1 in 10yr and 1 in 100yr flow collected by this section of Morisset Park Road will be 235L/s and 394L/s respectively.

The construction of kerb and gutter and associated stormwater drainage along the northern side of Morisset Park Road will divert all runoff from southern catchments away from the subdivision site. As such no allowance needs to be made for this runoff in the Concept Stormwater Plans for the internal subdivision.

8. A portion of the proposed subdivision in the south eastern corner of the site will not be directed to the basin and will therefore have no detention or water quality management.

Detention for this section of the site has been included in the DRAINS detention modeling. As noted in the *'Concept Stormwater Management Plan and Road Design'* report this section of the site, due to site levels, can not be directed to the detention basin and will not be detained. The drainage concept modeled has over-detained the remainder of the site such that the cumulative flow from the post developed site is equal to the pre-developed flow.

Further to discussions held with Council Officers it has been agreed that water quality for runoff from this section of the site shall be treated using stormwater "pit inserts" within the street drainage system. It is likely that only two stormwater pits will require inserts which will be equal to 'Watercyle Enviropits'.

9. The design of the detention basin shows that the basin will spill in the Q100 ARI event through private property. This is not desirable. Preferably the outflow pipe(s) from the basin would cater for the Q100 ARI flow out of the basin.

A covenant should be applied to the private land in question to advise that the overland flow path cannot be obstructed. This would also mean that boundary fencing between the basin lot and the private lot cannot obstruct flows.

As the application proposes works on Lot 2 DP 244002 (pipes and constructed overland flow path), it is considered that the application should include this land and have appropriate owners consent.

I note that the pipes that discharge from the basin are proposed to be 300mm in diameter. Council's Engineering Guidelines require that all public pipelines be a minimum of 375mm diameter.

The outlet from the basin has been redesigned to incorporate Council's comment where possible. At the weir level a large pit has now been designed to take high level flows previously directed to the weir. This pit will be located downstream of the low flow orifice (refer amended drawings in Attachment 1).

The outlet pipes from this pit have been sized to cater for the 1 in 100yr flow from the basin of 1.1m^3/s in the 1 in 100yr event. These pipes will be fully designed at Construction Certificate stage however preliminary sizing using Colebrook White Charts suggests 2 x 450mm diameter RCP will be sufficient.

While it is estimated that the 1in 100yr flow can be directed to Chifley Road using the underground pipes the basin will still be design with a weir to control overflow from storm events greater than the 1 in 100yr. As such the fencing proposed along the overflow route will be designed such that it does not obstruct flow below a 300mm in height. All fencing will be fully designed at Construction Certificate stage to ensure no obstructions to flow.

10. Council will not support the pipes from the basin being discharged directly to the table drain in Chifley Road. Currently the subdivision site drains, via sheet flow, over a large area along Chifley St and Frederick Street. The proposal is to mimic these site flow rates but to concentrate them all at one point. ie from a new headwall in Chifley Street. Therefore the flows at this discharge point are potentially much higher than those that currently exist. The existing drainage system in Chifley St consists of shallow table drains with piped driveway crossings. It is unlikely that this system would have adequate capacity to take the additional flows being directed to it. This should be investigated. It may be that a piped drainage system will be required to be installed in Chifley Street.

As part of the subdivision, we understand that kerb and gutter, including underground stormwater drainage, shall be provided along those lengths of Morisset Park Road and Chifley Road immediately fronting the site.

Preliminary calculations estimating the expected peak flow and required size of pipes are contained in Attachment 3. In accordance with LMCC guidelines the underground stormwater drainage should be designed such that excessive above ground flows are not experienced in the 1 in 10yr event. To be conservative the

calculations completed have allowed for the full 1 in 10yr flow to be conveyed below ground. The 1 in 100yr flow has also been calculated to show the requirement for the major overland flow path. In summary the following has been estimated:

- Estimated peak 1 in 10yr flow at Point A (intersection of Morisset Park Road and Chifley Road) is 235L/s, while the peak 1 in 100yr flow is approximately 394L/s. Based on Colebrook White Charts (k=0.15) a 375mm diameter pipe at 5% can be used to convey 450L/s.
- Estimated peak 1 in 10yr flow at Point B (Northern boundary line of Lot 19 along Chifley Road) is 447L/s, while the peak 1 in 100yr flow is approximately 803L/s. Based on Colebrook White Charts (k=0.15) a 450mm diameter pipe at 2.5% can be used to convey a flow of approximately 600L/s. Flow above 600L/s to be catered for in the existing road side swale (refer discussion below).
- Estimated peak 1 in 10yr flow at Point C (Point of connection from Detention basin to stormwater system in Chifley Road) is 952L/s, while the peak 1 in 100yr flow at point C is expected to be 1,712L/s.
- The capacity of the existing shallow road side drain along Chifley Road from Point C to Fredrick Street is approximately 873L/s (based on Mannings Calculations – refer Attachment 3).
- To cater for the peak flow in the 1 in 100yr storm event at Point C, an underground pipe system will be constructed from Point C down the western side of Chifley Road to the existing system at the intersection of Fredrick Street and Chifley Road. Based on Colebrook White Charts (k=0.15) a 525mm diameter pipe can be used to convey approximately 1,100L/s at 3.5%.
- Flow above 1,100L/s shall be conveyed by the existing roadside swale. Additional flow in the 1 in 100yr event approximately 612L/s. Capacity of swale approximately 873L/s.

Please note that a full hydraulic design of all stormwater drainage will need to be undertaken at Construction Certificate stage. The flows and pipe sizes provided above are estimates only.

11. The internal batters within the basin are proposed to be as steep as 1V:3H. Council's maintenance requirements are such that any grassed batters should be no steeper than 1V:4H.

The civil design plans and landscaping plans are to be reassessed to ensure that the proposal meets Council's design and maintenance requirements.

The internal batters on the basin have been revised to be a maximum of 1V:4H. The new stage storage relationship has been gained for the basin.

Stage (m AHD)	Storage (cu.m)
7.15	0
8.15	367
8.95	937
9.20	1149
9.40	1329

The DRAINS model was updated to include the following:

1. New Stage / Storage relationship shown above
2. A low flow outlet contained in a pit under the basin instead of a headwall (refer Comment 15)
3. High level pit with cover level at 8.95m AHD to take high level flows
4. 375mm outlet from low flow pit to high flow pit
5. 2 x 450mm pipe outlet from high flow pit to Chifley St

The results of the modeling are shown in the table below. In all events the peak discharge from the total developed site is equal to or less than the pre-developed flow.

	20% AEP (l/s)	10% AEP (l/s)	5% AEP (l/s)	2% AEP (l/s)	1% AEP (l/s)	Peak Detention Storage* (m^3)
Pre-developed Site	516	665	871	1,100	1,330	-
Post developed Site	507	568	652	941	1,270	1,107

* Peak detention storage in the 1% AEP event

12. The provision of maintenance access to the basin should be considered.

The internal batters on the basin have now been revised to be a maximum of 1V:4H in line with Council recommendations for mowing and maintenance.

As noted by the *'Concept Stormwater Management Plan and Road Design'* report access to the basin for maintenance equipment and pedestrians is provided through the use of the concrete pad from the road system. From this pad the basin has been designed to have a 1V:6H slope to allow for easy access and egress from the basin invert.

13. The inclusion of safety devices (signs and fencing) around the basin should be discussed.

The use of a 1V:6H batter for access is recommended by the Lake Macquarie City Council's Draft '*Batter Slope Treatments and Fencing Guidelines for constructed Wetlands and Detention Basins'*. Further to the above; signage, planting and fencing along the top of the basin shall be used to discourage pedestrian access to that area containing the 1V:6H grades. These items will be fully designed at Construction Certificate submission.

14. The proposed stormwater management strategy has only provided a detention basin and gross pollutant trap. This strategy does not adequately manage the discharge of nutrients from the site. The stormwater management regime should be amended to provide for a formal wetland (or similar facility) to mitigate nutrient loads.

As noted by the 'Concept Stormwater Management Plan and Road Design' report the stormwater quality system described has been designed in accordance with LMCC's Stormwater Treatment Framework & Stormwater Quality Improvement Device Guidelines SQUID Guidelines (SQID Guidelines).

Table 3 from the 'Concept Stormwater Management Plan and Road Design' report has been reproduced below to include how the System Efficiency calculations were completed. Section 4 of the SQID Guidelines sets out the procedure for the calculation of pollutant removal at the end of a treatment train. The SQID Guidelines Targets are the required values, while the System Efficiency is that provided by the designed system.

	Removal efficiencies						
Pollutant Species	**Gross Pollutants**	**Coarse Sediments**	**Medium Sediments**	**Fine Sediments**	**Nutrients**	**Heavy Metals**	**Oils and Greases**
SQID Guidelines Targets	High-Very High 50-100%	High-Very High 50-100%	Moderate-High 30-80%	Moderate 30-50%	Moderate 30-50%	Moderate 30-50%	Moderate 30-50%
CDS Unit	Very High (90%)	Very High (90%)	Moderate (45%)	Low (20%)	Low (20%)	Low (20%)	Moderate (45%)
OSD	High (70%)	Moderate (45%)	Low (20%)	Low (20%)	Low (20%)	Low (20%)	Low (20%)
System Efficiency	**97.0% (90% + (10%x70%)**	**94.5% (90% + (10%x45%)**	**56% (45% + (20%x55%)**	**36% (20% + (20%x80%)**	**36% (20% + (20%x80%)**	**36% (20% + (20%x80%)**	**56% (45% + (20%x55%)**

Please note that in the table above the values for Medium Sediment and Oils and Greases have been corrected. All values for the Systems Efficiency fall within the SQID Guidelines Targets.

15. The current design of the detention basin has pipes discharging into the base of the facility. These flows then exit via the outflow headwall. This design means that the base of the basin would be often affected by trickle flows and can become muddy and unsightly and difficult to maintain. The aesthetics of the facility also suffer due to the presence of the headwalls. An alternate design that can be considered is to have the flows contained within pipes under the basin with storms surcharging up into the basin via surcharge grates.

As noted in comment 11 the design of the outlet has incorporated Council's suggestion.

16. Given the above comments it is likely that a larger footprint would be required for drainage facilities and as such may influence the proposed lot layout.

Based in the additional information provided we do not believe that a larger footprint is required to adequately manage stormwater on site and meet Council's guidelines.

I trust the above information provides sufficient information to address all of the concerns raised by Council in their letter dated 30.01.09. Should you require anything further at this stage, please do not hesitate to contact the undersigned.

Yours sincerely

Ben Clark

Civil Engineer

Attachment 1 – Revised Drawings

Attachment 2 – Calculations for Catchments South of Morisset Park Road

PROJECT:
MORISSET PARK RD.
SUBDIVISION

JOB No: NL070295
PAGE No:
DESIGNED: BAC
DATE: 15.04.09.

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CATCHMENT TO THE SOUTH OF MPR = 14,600 m²

2,000 m² IMPERVIOUS (13%).

→ FROM DRAINAGE HANDBOOK.

$C_{10} = 0.48$

$t_c = 0.76\ (0.014)^{0.38}$ → 0.15 hrs → 9 mins.

Adopt t_c = 10 mins. $I_{10,100yr}$ = 168.5 mm/hr. $I_{10,10}$ = 121

$Q_{100yr} = 0.000278 \times 14,600 \times 1.2 \times 0.48 \times 168.5 = \underline{393.9}$ L/s.

$Q_{10yr} = 0.000278 \times 14,600 \times 0.48 \times 121 = \underline{235}$ L/s.

Capacity of ϕ375 @ 1% = 200 L/s.

∴ ALLOWING FOR A SMALL AMOUNT OF SURFACE FLOW ϕ375 OR LARGER WILL BE REQ. TO BE FULLY DESIGNED AT CC.

Attachment 3 – Calculations for External Road Stormwater Drainage Systems.

PROJECT: Morisset Park Rd, Subdivision

JOB No: NL070295

PAGE No: 1

DESIGNED: BAC

DATE: 20/4/09.

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* Determining Peak flows and estimating stormwater req. along Chifley Road.

Catchment 1 - Morisset Park Rd + pervious catchment to the south.

A = 14,600 m² % imp = 13%.

Catchment 2 - Subdivision not directed to basin (subcatchment 2

A = 8,600 m² % imp = 50%.

Catchment 3 - Portion of Chifley Road upstream of Point B. (refer diagram).

A = 1,750 m² % imp = 50%

Catchment 4 - Subcatchment 1.

A = 5 4,100 m² % imp = 50%.

Catchment 5 - Chifley Road + existing lots between Point C + Point B.

A = 6,745 m² % imp = 45%

At Point (A) → Contributing catchment = 1.

Q_{10} = 235 L/s. Capacity of ϕ375 @ 5% = 450 L/s.

Q_{100} = 394 l/s ∴ OK.

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At Point (B) → Contributing = Catchment 1 + 2 + 3.

t_c = 12 mins → $I_{10,12}$ = 112.7 mm/hr

$I_{100,12}$ = 168.5 mm/hr.

A = 24,641 m² (% imp = 28% → C_{10} = 0.58)

Q_{10} = 0.000278 x 24,641 x 0.58 x 112.7 → 447.8 L/s.

Q_{100} = 0.000278 x 24.641 x 0.58 x 1.2 x 168.5 → 803.4 L/s.

Ø450 @ 2.5% = 600L/s ∴ Capacity OK.

At Point (C) → Contributing = All catchments.

A = 85,486 m²

Assume Catchment 2 & 4 0% imp due to basin.

∴ % imp = 6.8% → C_{10} = 0.45

t_c = 20 mins → $I_{10,20}$ = 89 mm/hr

$I_{100,20}$ = 133.4 mm/hr.

Q_{10} = 0.000278 x 85,486 x 0.45 x 89 = 951.8 L/s.

Q_{100} = 0.000278 x 85,486 x 0.45 x 1.2 x 133.4 = 1,711.9 L/s.

Ø525 @ 3.5% = 1,100 L/s. ∴ OK capacity for 1 in 10yrs.

PROJECT: Morisset Park Road.

JOB No: NL070295

PAGE No: 3

DESIGNED: BAC

DATE: 24.4.09.

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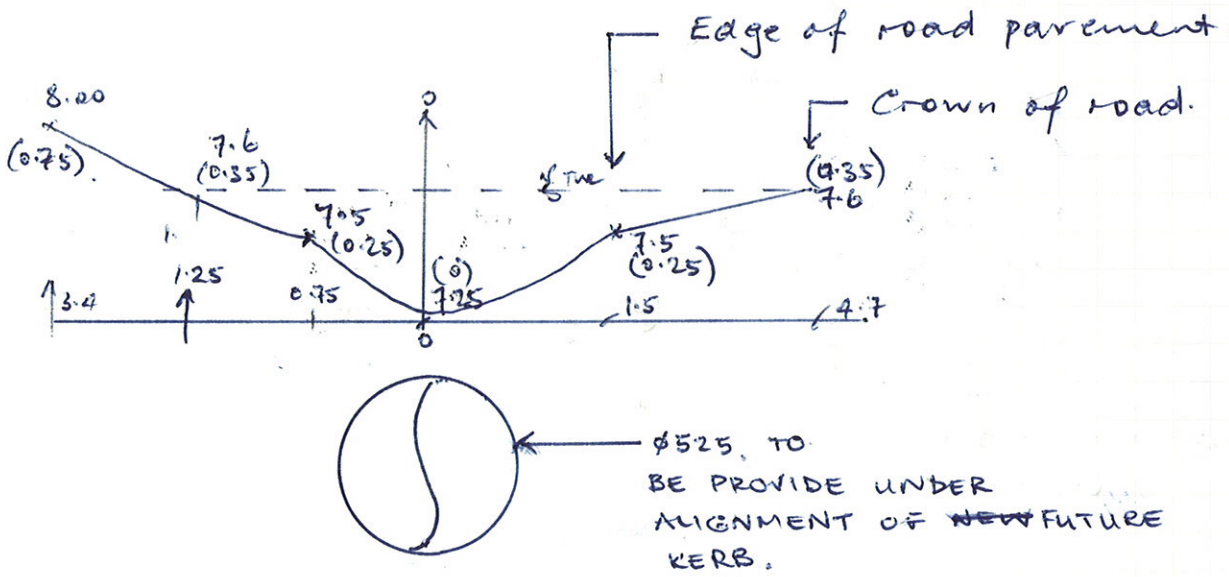

TYPICAL ROADSIDE SWALE IN CHIFLEY ROAD.

From Mannings Spreadsheet:

Capacity of roadside swale in Chifley road ≈ 873 L/s (refer spreadsheet)

Longitudinal grade = 3.5%.

Mannings 'n' = 0.035.

CAPACITY OF OPEN CHANNEL USING MANNINGS EQUATION

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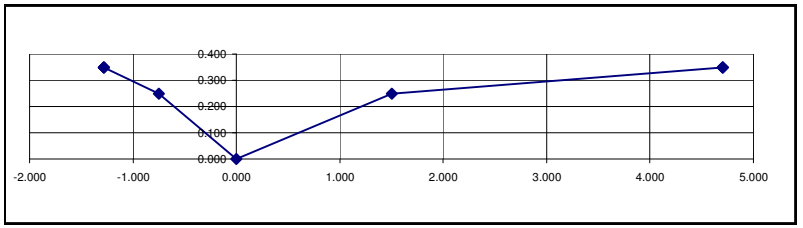
Job No. NL070295
Job Name Morisset Park Road

Calcs By BAC
Date 24.04.09

Channel Name = **Chifley Road Swale**

AREA sq.m	=	0.69
WETTED PERIMETER m	=	6.05
VELOCITY m/s	=	1.26
VELOCITY HEAD m	=	0.08
VxD	=	0.44
MANNINGS "n"	=	0.035
GRADE %	=	3.5
CAPACITY cu.m/s	=	**0.873**



CHANNEL PROFILE

	Left offsets (-m)				Centre				Right offsets (+m)
Offsets	-1.280	-1.280	-1.280	-0.750	0.000	1.500	4.700	4.700	4.700
Depth (-m)	0.350	0.350	0.350	0.250	0.000	0.250	0.350	0.350	0.350

Note: User to enter information only in cells coloured yellow.

CATCHMENT 3

CATCHMENT 5.

CATCHMENT 2

CATCHMENT 4

CATCHMENT 1.

CATCHMENT PLAN NL070295

LEGEND

CHIFLEY ROAD

MACQUARIE ROAD

MACQUARIE PARK ROAD

FREDERICK STREET

SITE BOUNDARY

ROAD 1

ROAD 2

ROAD 3

VACANT PUBLIC LAND

SCRUB

WIRE DWELLING

FIBRO DWELLING

SHED

W/C

KENNELS

EDGE OF BITUMEN

DRAWN: S.STARTIN DESIGNED: B.CLARK JOB MANAGER: B.CLARK VERIFIER: B.CLARK