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ADVERTISED

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375 Swansea Road, Lilydale – Updated Flood Risk

1 OVERVIEW

Yarra Ranges Shire Council is considering a planning application for the proposed development at 375 Swansea Road, Lilydale. Melbourne Water, as the floodplain referral authority, has requested further information in its letter dated 7 July 2023 and through subsequent discussions, to provide a more detailed description to the flood risk profile at the site and the sensitivity of the development to potential floods greater than the 1% AEP Design Storm.

Following discussions with Melbourne Water, additional modelling was undertaken, to predict hydraulic conditions and define likely flood risk for floods much greater than the 1 in 100 (or 1%) AEP design standard. This letter summarises findings of the additional modelling.

2 HYDROLOGIC ANALYSIS

2.1 Overview

Olinda Creek, upstream of the subject site, has a catchment area of 47 km^2. The contributing catchment is shown in **Error! Reference source not found.Error! Reference source not found.**. A significant proportion of the upstream catchment is part of the Dandenong Ranges National Park and hence heavily forested.

It is noted that Silvan Reservoir is located within the upper reaches of the catchment. Silvan Reservoir is utilised for storage and has only a small contributing catchment area, hence does not fill and spill from rainfall and has no outflow capacity. The Silvan Reservoir catchment area, approximately 9.4 km^2, is not included in the Melbourne Water RORB model for Olinda Creek. This is appropriate for flood modelling purposes.

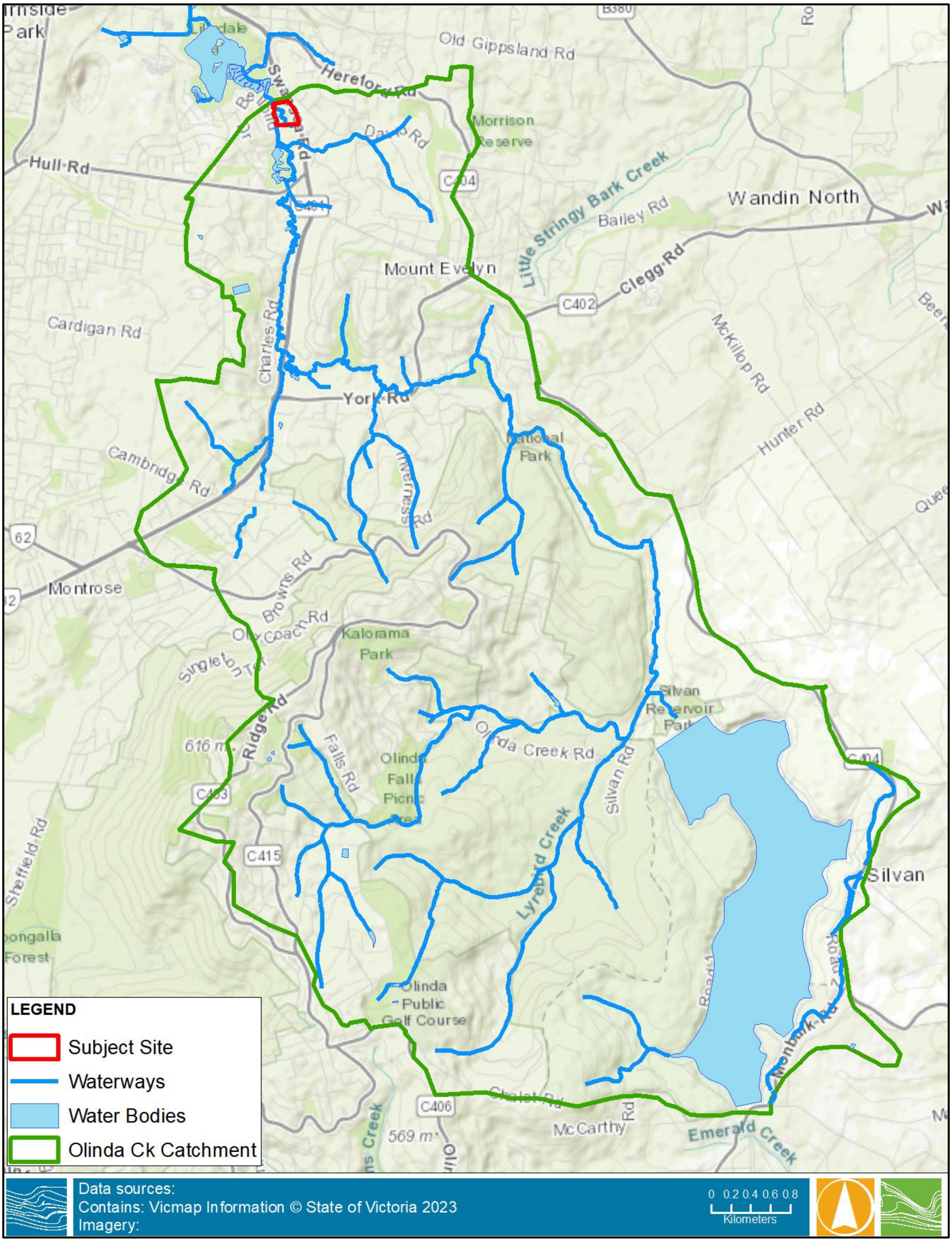


Figure 1 Olinda Creek Catchment upstream of 375 Swansea Road, Lilydale

2.2 Design Flows

The RORB model was reviewed to confirm the output location to extract design flows for hydraulic analysis.

Figure 2 shows a plan with the Subject Site and tributary catchments upstream and downstream. This highlights that:

- The Fuller Road Drain Catchment enters Olinda Creek approximately 200 m upstream of the Subject Site.
- The Hereford Road Drain enters Olinda Creek approximately 450 m downstream of the Subject Site near the inflow point to Lilydale Lake.
- Topographic data, the previous Melbourne Water flood levels and revised 2D flood levels all show there is significant elevation drop between the downstream extent of the Subject Site and Hereford Road Drain (Lilydale Lake), such that the levels at this location would have no impact on the Subject Site.

An extract from the RORB catchment file is shown in Figure 3. With respect to this, it is noted that:

- The Fuller Road Drain tributary catchment and Olinda Creek including up to subarea BD are included at the reporting location "Olinda Ck at Akarana Rd".
- The catchment area for Olinda Creek up to Akarana Road, as calculated by GIS, matches the total area up to subarea BD in the RORB model (37.4 km^2).
- As mentioned above, this area calculation in RORB excludes the Silvan Reservoir catchment (approx. 9.4 km^2).

Figure 4 shows an extract from a RORB output file, highlighting that the **"Olinda Ck at Akarana Rd"** reporting point is **Location 15** in the output file. Figure 5 then highlights the 1% AEP peak design flow for the extraction location in the RORB output file, which is 87 m^3/s.

Peak design flood flows, up to and greater than 1 in 100 (1%) AEP (including 1 in 200 AEP and 1 in 500 AEP) were extracted from the Melbourne Water RORB hydrologic model at the same reporting location.

The PMF flow was estimated based on the prediction equation provided in Hydrological Recipes[1]. The 1 in 1,000 AEP and the 1 in 2,000 AEP design flood peaks were then estimated from an extrapolation of the 1 in 100, 1 in 200 and 1 in 500 AEP design peak flood estimates from RORB, in conjunction with the PMF estimate.

Peak design flow estimates for the RORB model and interpolated events up to the 1 in 2,000 AEP and the PMF are presented in Table 1. Figure 6 below shows a plot of the interpolated peak design flow values.

[1] Hydrological Recipes – Estimation Techniques in Australian Hydrology, (CRC for Catchment Hydrology, 1996)

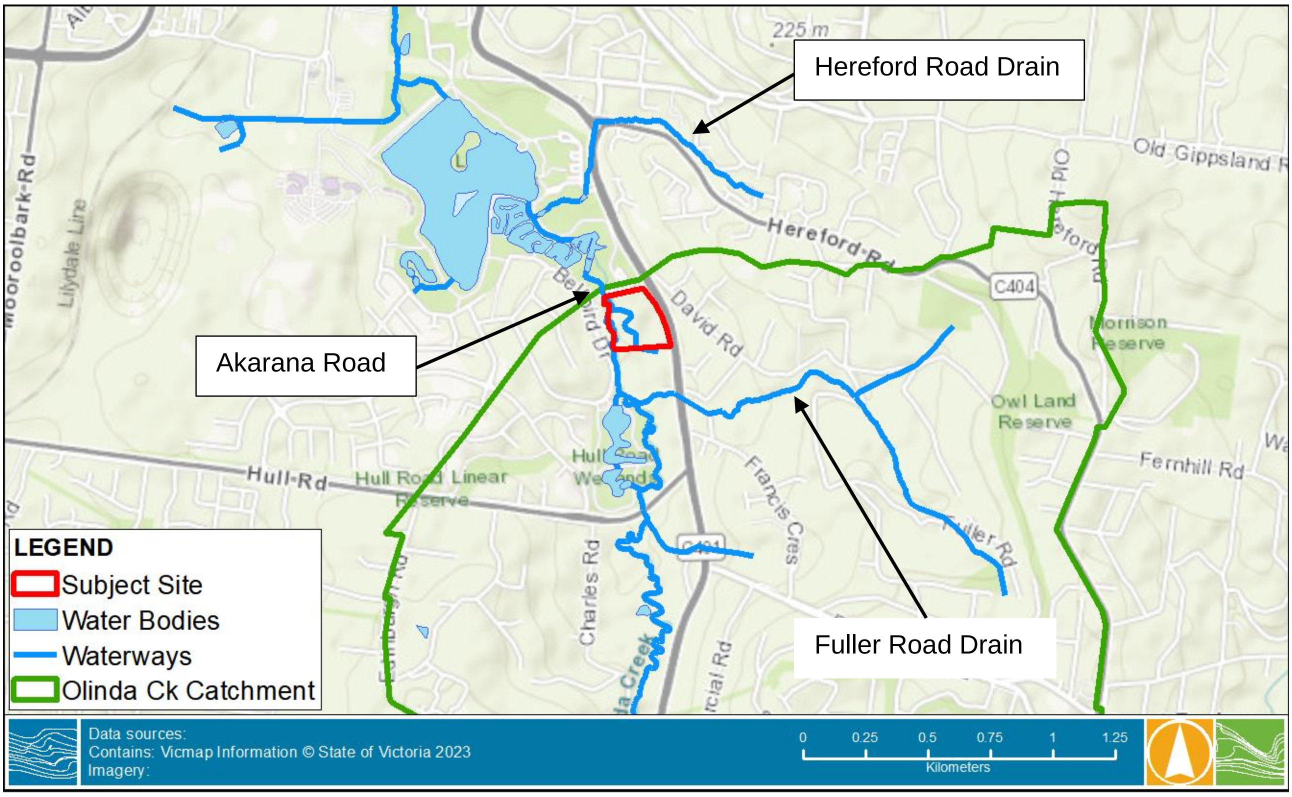


Figure 2 Location of tributary inflows in vicinity of Subject Site

```
1,1,0.65,-99,          subarea BA
4
5,2,0.35,3.33,-99
2,2,0.45,3.33,-99,     subarea BB
7
Fuller Rd Drn at Swansea Rd
5,2,0.20,3.33,-99
4
3
1,2,0.85,5.00,-99,     subarea BC
4
5,2,0.25,0.67,-99
2,2,0.20,0.67,-99,     subarea BD
7
Olinda Ck at Akarana Rd
5,2,0.30,0.67,-99
2,4,0.55,-99,          subarea BE
3
C  Start of Hereford Rd Drn Catch
1,2,0.60,5.00,-99,     subarea BF
2,2,0.55,4.00,-99,     subarea BG
2,2,0.55,2.50,-99,     subarea BH
2,4,0.40,-99,          subarea BI
7
Hereford Rd Drn us Lily Lake
4
3
1,2,0.50,4.00,-99,     subarea BJ
3
1,1,0.50,-99,          subarea BK
4
```

Flow Extraction Location

Figure 3 Extract of RORB model CAT file showing location of flow print out

```
Parameters:  kc =    13.00    m = 0.80

Loss parameters     Initial loss (mm)     Runoff coeff.
                          20.00                0.60

Peak  Description
  01  Calculated hydrograph,   LyrebirdG Ck at Olinda Ck Rd
  02  Calculated hydrograph,   Olinda Ck at Falls Rd
  03  Calculated hydrograph,   OlindaCk us LyrebirdGCk junc
  04  Calculated hydrograph,   OlinCk+LyreGCk ds OlindaCkRd
  05  Calculated hydrograph,   Olinda Ck at Hazel St
  06  Calculated hydrograph,   Olinda Ck at York Rd
  07  Calculated hydrograph,   Olinda Ck at us Swansea Rd
  08  Calculated hydrograph,   HeathF Rd Dn us CamRdDn junc
  09  Calculated hydrograph,   Cambr Rd Dn us HF Rd Dn junc
  10  Calculated hydrograph,   HeathF Rd Drn + CamRdDn junc
  11  Calculated hydrograph,   HeathF Rd Drn us OlinCk junc
  12  Calculated hydrograph,   OlinCk+HeaF Dn at Swansea Rd
  13  Calculated hydrograph,   Olinda Ck at Hull Rd
  14  Calculated hydrograph,   Fuller Rd Drn at Swansea Rd
  15  Calculated hydrograph,   Olinda Ck at Akarana Rd
  16  Calculated hydrograph,   Hereford Rd Drn us Lily Lake
  17  Calculated hydrograph,   Olinda Ck at Lilydale Lake
  18  Special storage :    Existing Lilydale Lake RB - Outflow
  19  Special storage :    Existing Lilydale Lake RB - Inflow
  20  Calculated hydrograph,   Olinda Ck at Maroondah Hwy
  21  Calculated hydrograph,   Olinda Ck at Nelson Rd
  22  Calculated hydrograph,   Olinda Ck us Lilydale Dn Jnc
  23  Calculated hydrograph,   LilyDrn d/s MaroonHwy/ASEABB
  24  Calculated hydrograph,   Lilydale Drn at Beresford Rd
  25  Calculated hydrograph,   Lilydale Drn at Nelson Rd
  26  Calculated hydrograph,   Lilydale Dn us Olinda Ck Jnc
  27  Calculated hydrograph,   Olinda Ck+Lilydale Drn Junc
```

Figure 4 Extract of RORB output file showing location of flow printout

Run	Dur	ARI	Rain(mm)	ARF	Peak0001	Peak0002	Peak0013	Peak0014	Peak0015	Peak0016	Peak0017
1	10m	100y	22.48	0.83	0.0064	7.5073	6.4865	3.6452	9.1149	11.0205	37.6212
2	15m	100y	27.43	0.83	0.8297	13.7786	9.0305	6.5272	16.4899	11.0294	38.6600
3	20m	100y	31.23	0.83	2.1059	16.9754	13.3517	9.0756	23.6673	11.1733	37.2350
4	25m	100y	34.35	0.83	3.2990	19.8672	17.3221	10.3837	29.7182	12.0207	44.4827
5	30m	100y	36.96	0.83	4.3629	19.2842	20.3439	11.5901	32.1122	11.9881	49.1223
6	45m	100y	43.05	0.86	7.8275	22.4099	29.4907	13.0667	41.8894	12.8986	61.6944
7	1h	100y	47.61	0.90	10.9548	25.4018	38.0967	13.9384	50.0429	13.5859	64.9172
8	1.5h	100y	55.24	0.91	15.2780	28.7170	41.2698	14.4925	55.3560	15.7689	69.2824
9	2h	100y	61.15	0.93	18.3542	32.5051	43.8430	16.6577	57.2147	17.1420	72.4301
10	3h	100y	70.35	0.96	20.0953	22.1328	44.9552	12.4780	58.0413	10.6963	69.8044
11	4.5h	100y	80.84	0.96	20.1403	26.4908	49.2803	14.7864	56.3843	13.0809	72.0891
12	6h	100y	89.24	0.97	21.0757	20.4534	59.2319	11.8604	61.7983	9.3842	75.4651
13	9h	100y	102.67	0.97	25.0089	19.2467	67.0165	10.9498	75.4561	8.6611	89.8116
14	12h	100y	113.44	0.97	21.4650	17.4529	79.6759	9.6608	87.0026	8.5869	94.2588
15	18h	100y	133.90	0.98	16.6247	11.0830	70.2820	6.0776	77.2047	5.4981	82.6700
16	24h	100y	150.28	0.98	19.7079	11.7221	62.7413	6.5802	73.6962	5.5558	84.4967
17	30h	100y	163.94	0.98	15.3046	9.4399	62.5437	5.2140	70.3702	4.3922	77.1962
18	36h	100y	175.59	0.98	15.9686	8.6132	60.4440	4.7735	68.4901	4.0028	76.4249
19	48h	100y	194.48	0.98	15.0684	10.1735	58.6892	5.7534	65.0528	4.6198	74.5609
20	72h	100y	220.42	0.98	10.4760	6.6002	45.3111	3.6601	49.9974	3.1589	53.5755

Figure 5 Extract of RORB model output file showing 1% AEP Peak Flow

Table 1 Estimated Peak Design Flood Flows – Olinda Creek

Design Storm AEP (1 in X / %)	Peak Flow (m^3/s)	Comment
1 in 100 / 1%	87	RORB
1 in 200 / 0.5%	124	RORB
1 in 500 / 0.2%	145	RORB
1 in 1,000 / 0.1%	195	Interpolated between the Very Rare and PMF peak flow
1 in 2,000 / 0.05%	250	Interpolated between the Very Rare and PMF peak flow
PMF	1,400	Computation based on regression equations for PMF (from hydrological recipes)

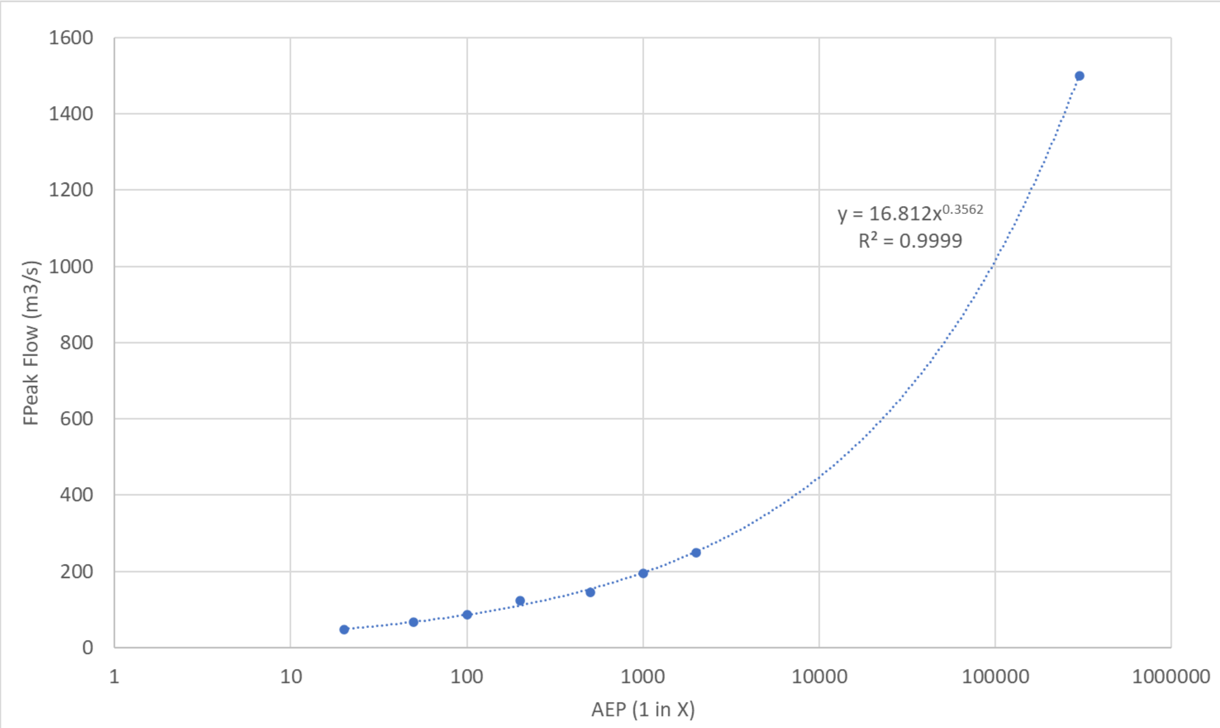


Figure 6 Interpolation of Peak Design Flood Flows

It is noted that in the range of flows above, the 1 in 200 AEP flow estimate is more than 30% greater than the 1 in 100 AEP and hence sensitivity to climate change could be considered in the results by assessing the 1 in 200 AEP design flood case.

2.3 Hydrology Check

The status of the Olinda Creek RORB model in terms of calibration is not known. As a check on the design flows, data for the catchment was extracted from the ARR Regional Flood Frequency Estimation Model (RFFE). A printout of the results is attached to this letter with a summary below in Table 2.

This provides a lower 1% AEP peak flood estimate of 41 m^3/s compared to RORB 87 m^3/s. The RORB estimate is within the 95% confidence limits of the RFFE which is considered reasonable. It is acknowledged that the RFFE provides a very approximate estimate of peak flow, however this result does provide some confidence that the adopted RORB flows are a reasonable estimates of design flows at the Subject Site.

Table 2 ARR RFFE Design Flow Estimates at Subject Site

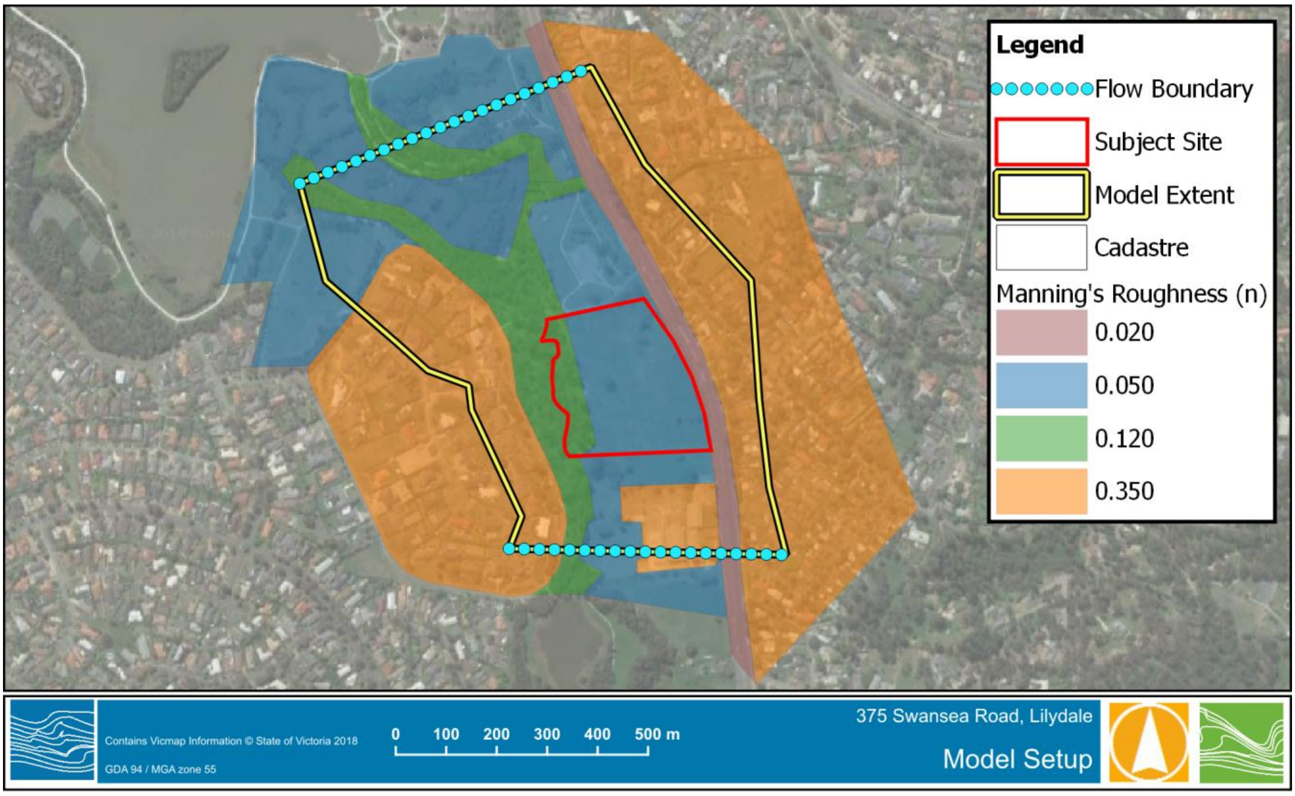
AEP % (1 in X)	RFFE Estimated Discharge (m^3/s)	RFFE Lower Confidence Limit (5%) (m^3/s)	RFFE Upper Confidence Limit (95%) (m^3/s)	RORB Peak Estimate (m^3/s)
50 (1 in 2)	7.9	3.3	19.1	18.3
20 (1 in 5)	14.2	6.2	32.8	29.2
10 (1 in 10)	19.4	8.4	45.1	35.1
5 (1 in 20)	25.1	10.7	59.8	47.4
2 (1 in 50)	33.8	13.8	83.3	67.5
1 (1 in 100)	41.2	16.4	105.0	87.0

3 HYDRAULIC MODELLING

TUFLOW is widely used software that is suitable for the analysis of overland flows in urban areas. The TUFLOW model routes flows overland across the topographic surface (2D Domain) to create flood extents, depths and velocities. The previous hydraulic (TUFLOW) model was constructed to analyse overland flooding at the site. The Olinda Creek flood model was used to determine potential flood extents and levels under the estimated 1 in 1,000 and 1 in 2,000 AEP design floods.

Hydrographs for these design floods were created by scaling the 1 in 100 AEP 12 hr design flood hydrograph previously provided by Melbourne Water and the peak design flood flows presented in Section 2.

Further to this, the projects/developments original TUFLOW model's manning's roughness "n" values across the vegetated waterway/riparian areas (green area in the figure below) and across the open space (blue area in the figure below) parts of the floodplain were increased for this assessment. The "n" values were matched to the higher range of Melbourne Water's 'AM STA 6200 Flood Mapping Projects Specifications (Melbourne Water, 2021)' roughness coefficients for these land types for conservatism. The roughness for the vegetated waterway/riparian areas was increased from 0.06 to 0.12 and for the open space it was increased from 0.04 to 0.05.



3.1 1 in 100 AEP Design Flood

Figure 7 shows the flood extent under the modelled 1 in 100 AEP design flood (post-development). The modelled flood levels near the southern (upstream) boundary of the subject site are around 109.55 m AHD and 109.2 m AHD at the downstream property boundary. It is demonstrated in the model results that the peak flood surface is quite flat through the area adjacent to the subject site. This is because the floodplain downstream of the site forms a choke point resulting from what appears to be historic filling of the Bellbird Park area.

It is important to note that where levels are taken on the upstream boundary has a significant bearing as levels are not flat in this area, dynamically responding to the floodplain topography. The image to the left (extract from Figure 7) highlights the difference between the east and west side of the property boundary. The previous WT reports conservatively adopted the maximum level on the west side which is higher than the flood level that actually impacts the area of development (in this example 109.55 m vs 109.25 m). In the report we therefore quote both levels and highlight the freeboard that relates to each.

It is noted that the current flood levels in MW's system are 109.85 m AHD at the upstream property boundary and 108.5 m AHD at the downstream property boundary. These levels are from a 1D model that is many years old with unknown hydrology and it is assumed they are uncalibrated. Subsequently, it is considered that peak flood levels derived from the 2D model results produced for this investigation are likely to be of greater reliability than the previous 1D model results. The current modelling is using accurate LIDAR survey data (updated to used LiDAR captured in 2017/2018), the latest hydraulic modelling software and appropriate/conservative hydraulic roughness values to determine flood heights. This can be considered best practice modelling and more reliable than older data.

Based on the above, and for the purposes of sensitivity testing of the model results to larger flood flows, there is considered to be limited benefit in artificially modifying levels in the 2D hydraulic model to achieve the same peak flood levels as the old 1D model. This would most likely require unrealistic hydraulic model parameters to be used (either too high and/or too low).

For the purposes of floodplain sensitivity impacts (and setting appropriate design flood levels), the 2D model results presented in this memo are considered appropriate. Specifically:

- As the RFFE flow check suggests, the existing RORB peak design flows are likely, if anything, to be on the high side and hence can be considered conservative.
- The hydraulic modelling is based on accurate, recent LiDAR survey.
- The hydraulic roughness values applied are at the high end of the recommended range which is also considered conservative.

Melbourne Water is responsible for determining the appropriate NFPL for the site. The plans for the development could be conditioned to match the original MW advice regarding design flood levels. The sensitivity analysis and assessment of freeboard for larger storms is valid irrespective of the applied NFPL[2].

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[2] The Nominal Flood Protection Level (NFPL) is typically the minimum level designated for the protection of assets and people in developments where there is some level of existing or future flood risk. The NFPL is typically determined by the 1 in 100 AEP flood level plus a nominated freeboard. In riverine flooding contexts Melbourne Water typically applies a minimum 600 mm freeboard. For this development the NFPL has been applied to the fill pad. Actual dwelling floor levels will be higher than the NFPL.

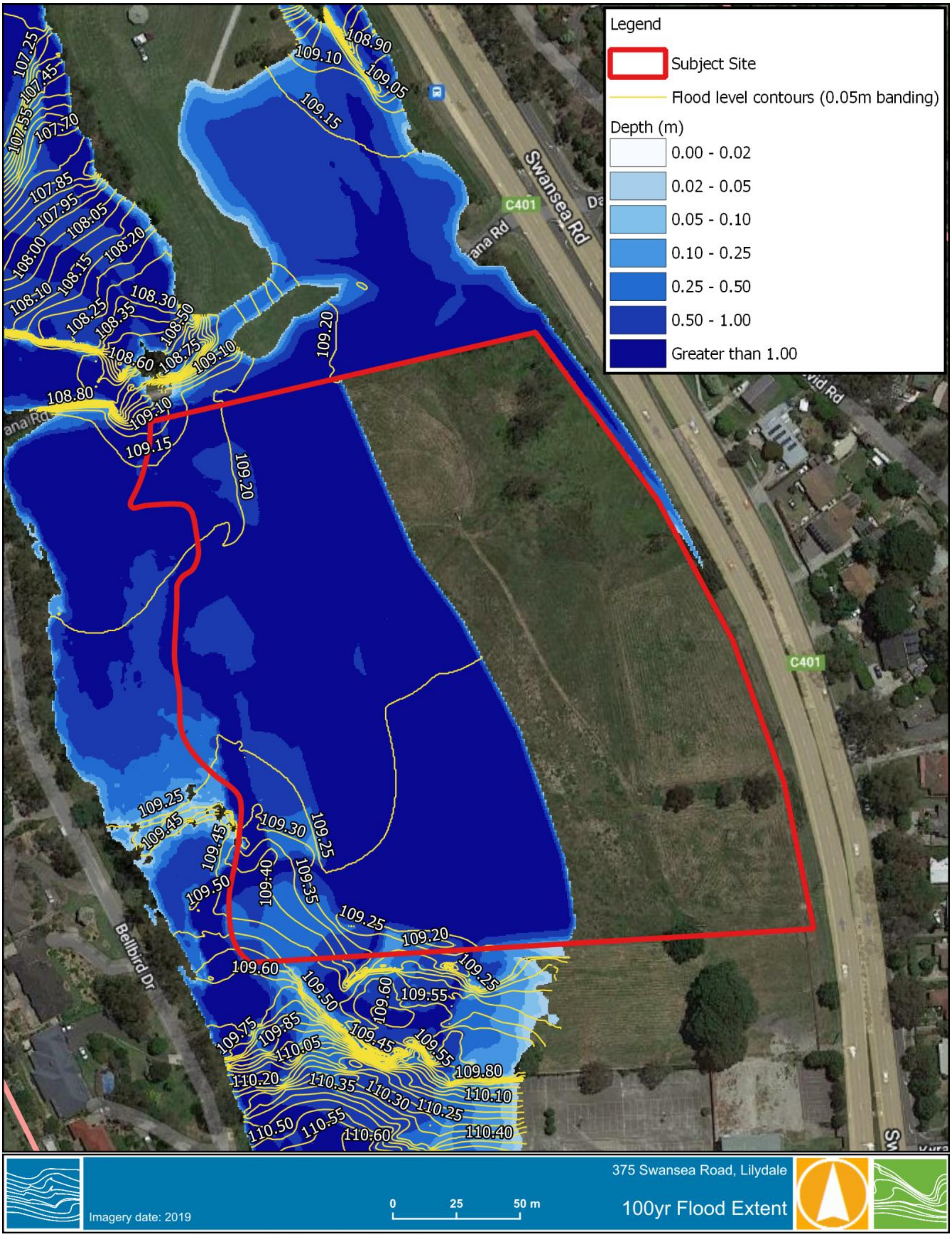
Legend
Subject Site
Flood level contours (0.05m banding)
Depth (m)
0.00 - 0.02
0.02 - 0.05
0.05 - 0.10
0.10 - 0.25
0.25 - 0.50
0.50 - 1.00
Greater than 1.00
Swansea Rd
C401
Bellbird Dr
Imagery date: 2019
0
25
50 m
375 Swansea Road, Lilydale
100yr Flood Extent

Figure 7 1 in 100 AEP Design Flood Extent

3.2 1 in 1,000 AEP Design Flood

The 1 in 1,000 AEP design flood peak is 2.24 times the 1 in 100 AEP flow (195 m^3/s vs 87 m^3/s, or 124% higher).

Figure 8 shows the flood extent under the modelled 1 in 1,000 AEP design flood. Flood levels in the modelled 1 in 1,000 AEP design flood are in the order of approximately 500 mm higher than the 1 in 100 AEP flood levels along the development.

Based on these results, it is considered that the Nominal Flood Protection Level (i.e., applicable 1 in 100 AEP flood level + 600 mm freeboard) will provide protection against a predicted 1 in 1,000 AEP magnitude flood.

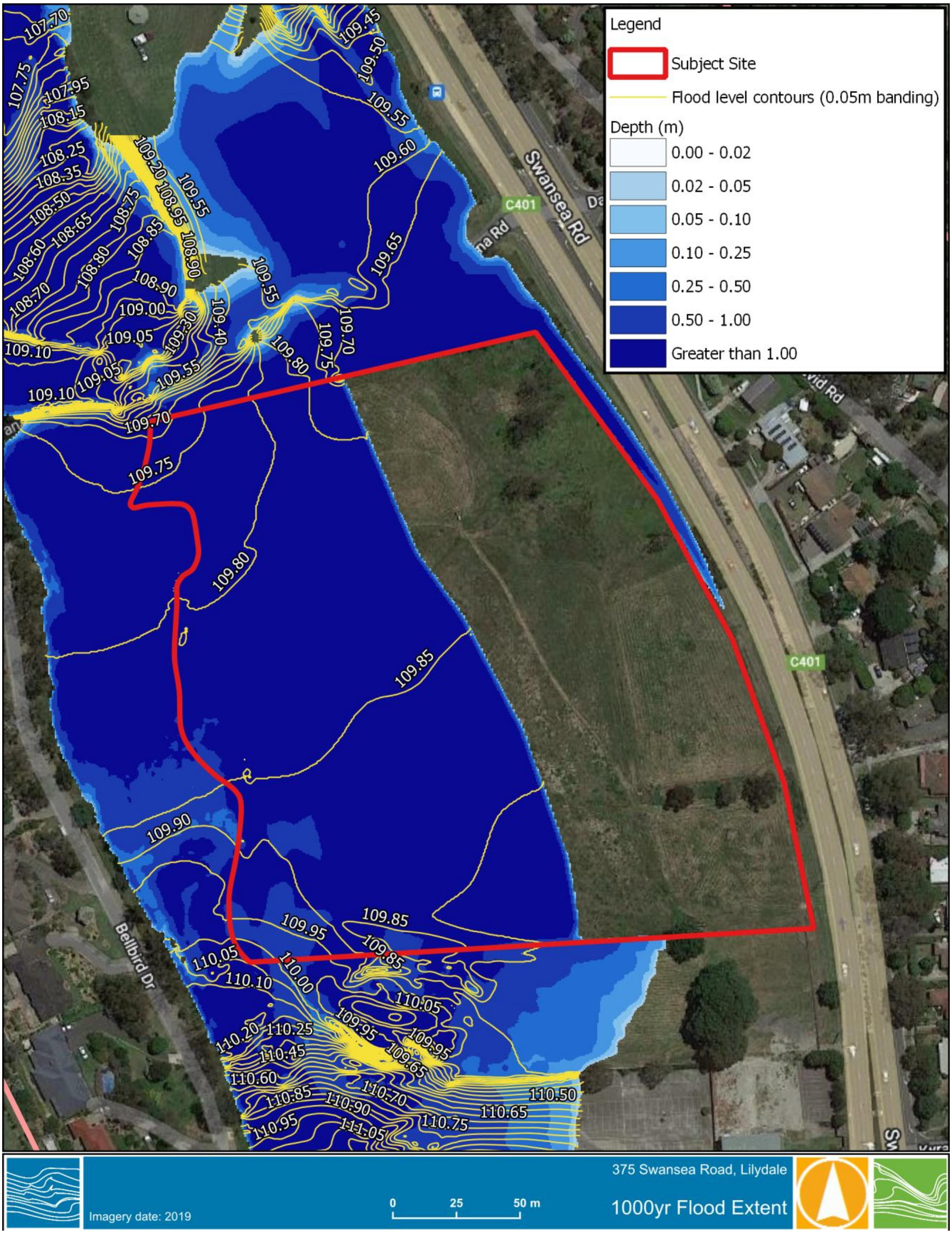

Figure 8 1 in 1,000 AEP Extent

3.3 1 in 2,000 AEP Design Flood

The 1 in 2,000 AEP design flood peak is 2.87 times the 1 in 100 AEP flow (250 m^3/s vs 87 m^3/s, or 187% higher).

Figure 9[3] shows the flood extent for the modelled 1 in 2,000 AEP design flood. The peak flood level for the 1 in 2,000 AEP design flood at the upstream end of the development is ~110.25 m AHD[4]. The 1 in 2,000 AEP flood levels fronting the development are in the order of approximately 700 mm above the 1 in 100 AEP flood levels (post-development). This would be a maximum of 150 mm above the NFPL for part the site. This is within the H1 hazard classification band (ARR 2019) as shown in Figure 10, which is considered generally safe for vehicles, people and buildings. This represents a very high level of flood protection and extremely low risk to residents or visitors to the site.

It is also noted that the actual maximum flood level at the development site would be around 110.1 m AHD which is equal to the fill pad level.

[3] Note that the fill pad level (NFPL) adopted in the modelling is indicative, therefore the flood results (depth, extent etc) shown across the development area/pad itself is indicative.
[4] The 1 in 2,000 AEP flood levels herein are slightly lower than the 1 in 2,000 AEP flood levels quoted in the version 1 of this letter (Water Technology, September 2023). This is because the previous (September 2023) 1 in 2,000 AEP flood modelling was erroneously undertaken using a hydrograph with a peak flow rate of 370 m^3/s, as opposed to the actual estimated 1 in 2,000 AEP design flow peak of 250 m^3/s.

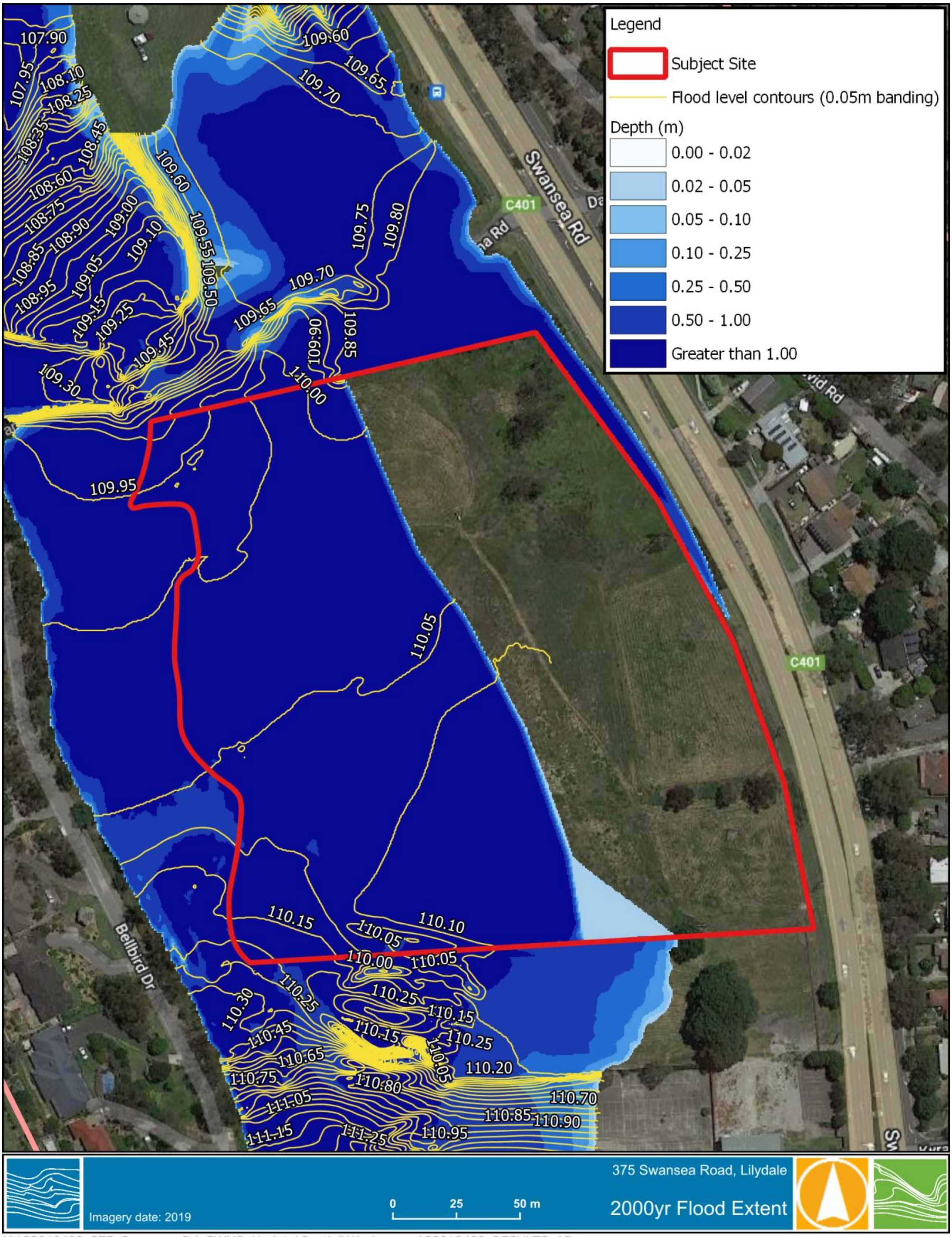

Figure 9 1 in 2,000 AEP Design Flood Extent

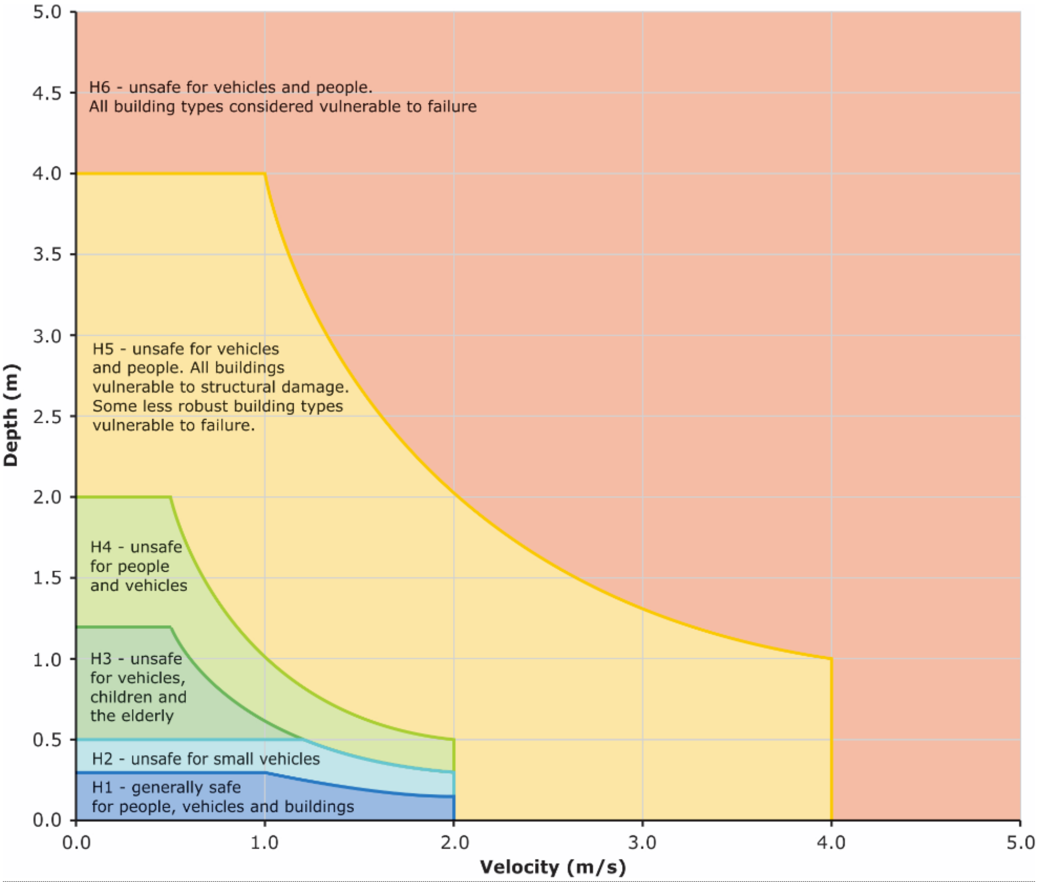

Figure 6.7.9. Combined Flood Hazard Curves (Smith et al., 2014)

Figure 10 ARR 2019 – Flood Hazard Curves

4 DISCUSSION

The levels in Table 3 show (for the upstream and downstream extents of the proposed development) the difference between the NFPL and peak flood levels for a number of extreme flood cases. This shows that the site is predicted to be flood free for the 1 in 500 and 1 in 1,000 and 1 in 2,000 AEP floods. The peak flood level for the 1 in 2000 AEP on the west side of the floodplain at the property boundary is 0.15 m higher than the proposed fill pad height however the modelling shows this level is not experienced at the development area. If this level was translated to the site, it meets the H1 hazard classification, which would not pose a threat to life in this extreme circumstance. It is also noted that dwelling floor levels are expected to be above the NFPL (which is the fill level) and hence an additional level of protection will be provided to the dwellings from flood damage.

Based on the sensitivity flood modelling that has been undertaken for design floods rarer than the 1 in 100 AEP, it is considered that the site's current Nominal Flood Protection Level (i.e., applicable 1 in 100 AEP flood level + 600 mm freeboard) provides a very high and appropriate level of protection against riverine flooding from very rare and extreme floods. It is noted that a very conservative assumption has already been made with respect to setting the development fill level as this was set at the maximum 1% AEP flood level over the whole property (109.5 m). It is evident that this flood level does not impact the fill area and provides > 600 mm freeboard across the development.

Any residual risk to property and life is extremely low and tolerable. Residual risk at the site could be further reduced through a Flood Response Management Plan for the site which could be readily implemented given the land ownership and collective management of the site in the future.

Table 3 Estimated Peak Design Flood Levels and Freeboard (to fill pad) at the site

Case		U/S end of development		D/S end of development	
	Fill Level (m)	Flood Level West/East (m)	Freeboard West/East (m)	Flood Level (m)	Freeboard (m)
WT Report (May 2022) 1:100 AEP	110.1	109.5 / 109.2	0.6 / 0.9	109.1	1.0
MW Levels 1:100 AEP	110.1	109.85	0.25	108.5	1.6
WT Revised model 1:100 AEP	110.1	109.55 / 109.25	0.55 / 0.85	109.2	0.9
WT Revised model 1 in 1,000 AEP	110.1	110.0 / 109.9	0.1 / 0.2	109.7	0.4
WT Revised model 1 in 2,000 AEP	110.1	110.25 / 110.1	-0.15 / 0.0	109.9	0.2

Please contact me if you have any questions or require further information.

Yours sincerely

Warwick Bishop
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WATER TECHNOLOGY PTY LTD