



Engineering | Surveying | Planning

# MOLLERS LANE, LEOPOLD – REZONING AND SUBDIVISION

## DAM BREAK ASSESSMENT

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Reference: 15196-03  
January 2018



**Client:** Mollers Lane Developments Pty Ltd & Pamas Property Pty Ltd

**Project:** Mollers Lane, Leopold

**Document:** Dam Break Assessment Report

## Document Control

| Version | Document type                             | Prepared by | Reviewed by | Date Issued |
|---------|---|-------------|-------------|-------------|
| V00     | Dam Break Assessment Report               | LS          |             | 25/10/2017  |
| V01     | Dam Break Assessment Report (Amended POS) | LS          | MC          | 17/01/2018  |
|         |   |             |             |             |
|         |   |             |             |             |
|         |   |             |             |             |
|         |   |             |             |             |

## Project Details

|                               |  |
|-------------------------------|--|
| <b>Project Name</b>           | Mollers Lane, Leopold Rezoning and Subdivision             |
| <b>Client</b>                 | Mollers Land Developments Pty Ltd & Pamas Property Pty Ltd |
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| <b>Editor</b>                 | Mark Colegate  |
| <b>TGM Reference</b>          | 15196-03   |

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## Document Location

J:\15196 (Mollers Lane, Leopold)\03-CE (FIA and SSMP)\Report\15196-03 FIA - Mollers Lane - Dam Burst v01.docx

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## 1. BACKGROUND

TGM Group has been engaged by Mollers Lane Developments Pty Ltd and Pamas Property PTY LTD to prepare a combined Planning Scheme Amendment and Subdivision Application under Section 96A of the Planning and Environment Act 1987 for the rezoning and staged multi-lot subdivision of a rural land situated to the west of Mollers Lane in Leopold.

As part of the approval process, City of Greater Geelong has required the assessment of the potential flood risks associated to the failure of the earthen irrigation dam located between Ash Road and Mollers Lane, i.e. upstream of the southern development (Figure 2.1), can potentially pose to the new development.

The dam break assessment has been carried out in accordance with the "*Guidelines on the Consequence Categories for Dams*" by [ANCOLD-2012], which define three levels of assessment, called *Initial Assessment*, *Intermediate Assessment* and *Comprehensive Assessment*.

According to [ANCOLD-2012], the *Initial Assessment* is composed by three steps:

1. Estimate of the inundation area;
2. Estimate of total damage/loss;
3. Estimate of total Population At Risk (PAR).

In the present study, the estimate of the inundation area has been carried out by developing a 2D TUFLOW hydrodynamic model of the flood propagation resulting from the dam break.

The flood extent scenarios have been computed taking into consideration two different dam failure scenarios, defined as ***Sunny Day Failure Scenarios*** and ***Flood Failure Scenarios*** in [ANCOLD-2012]. In the ***Sunny Day Failure Scenarios***, the dam break is assumed to occur during sunny days with no reservoir inflows. In the ***Flood Failure Scenarios***, the dam break is assumed to occur during the probable maximum flood (PMF), which has been identified as the 1% AEP rainfall event.

In addition, two different potential locations of the breach have been considered to analyse the worst failure conditions and the associated flood extents.

The dam break assessment process can be considered complete at the *Initial* stage if no further analysis is required accordingly to [ANCOLD-2012].

### 1.1 Supporting Documentation

This report is supported by the following TGM Group documents:

1. TGM Group report [TGM-r2018] - Mollers Lane, Leopold – Rezoning, Site Stormwater Management Plan and Flood Impact Assessment – Version 2. 15196-03. January 2018.
2. ANCOLD [ANCOLD-2012] – Guidelines on the Consequence Categories for Dams. October 2012. ISBN 978-0-9808192-5-0.

## 2. STUDY AREA

### 2.1 Site Context

The dam is located within a significant rural holding of approximately 41 hectares (including the entire property at 92 Mollers Lane, Leopold) and is located to the south/east of the established urban development of Leopold.

The dam location and the developed site area potentially affected by the dam break are shown in Figure 2.1.



**Figure 2.1:** Mollers Lane Study Area and Dam Location

The dam site (red area, Figure 2.1) is located adjacent to the south-western corner of the proposed developed area. A detailed description of the land development plan and the new channel alignment is contained in **[TGM-r2018]**.

### 2.2 Dam Characteristics

The dam is completely contained within earthen levees, which enclose a surface area of 12,611.9 m<sup>2</sup> at a stage of 39 m AHD. At the end of the dam, an earthen weir regulates the maximum water level within the basin and discharges the water-excess into a narrow channel.

The channel conveys the flows to a proposed 1% AEP pipe immediately upstream of the design detention basin, which discharges to the main waterway located approximately 650 m downstream.

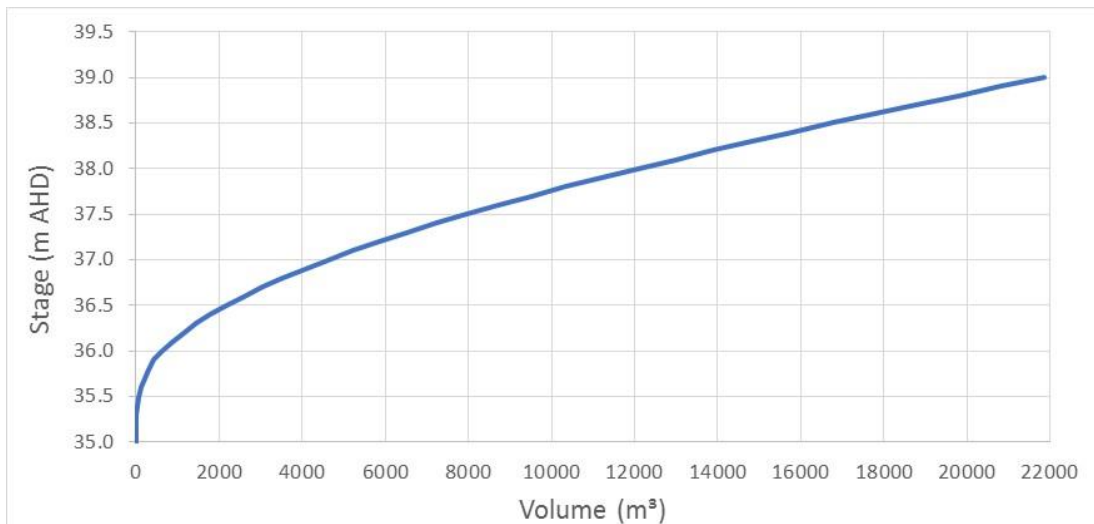
Please refer to **[TGM-r2018]** for a detailed description of the downstream designed detention basin characteristics and functioning.

A field survey campaign has been carried out by TGM to identify the bottom elevations of the basin and determine the capacity of the dam. The dam bottom elevations are shown in Figure 2.4.

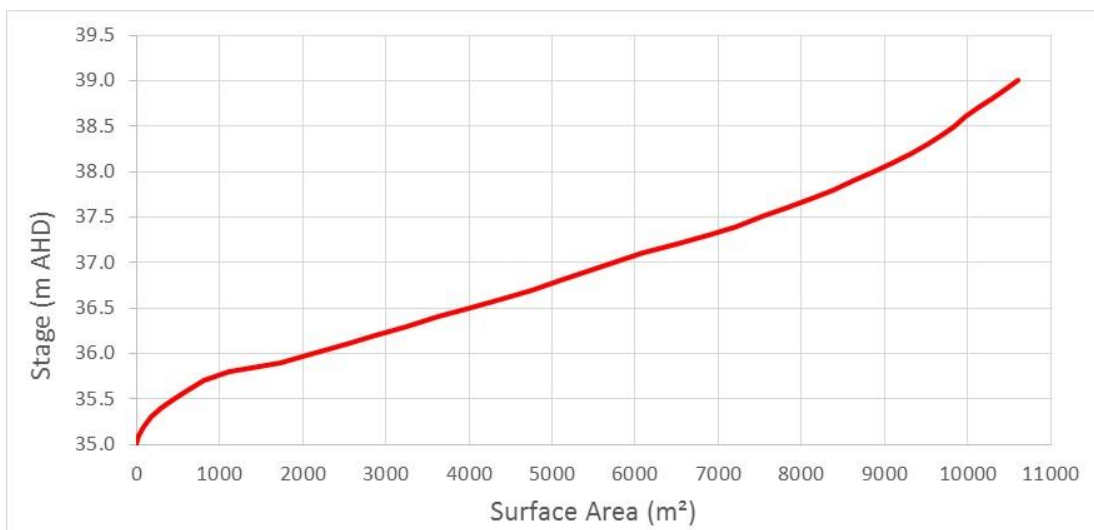
The characteristics of the dam are summarised in Table 2.1. The stage-storage volume and the stage-storage surface area relationships are shown respectively in Figure 2.2 and Figure 2.3.

**Table 2.1: Dam Characteristics**

| DAM CHARACTERISTICS            |                          |
|--------------------------------|--------------------------|
| Top Level of the Bank          | 39.0 m AHD               |
| Bottom Level of the Weir       | 38.5 m AHD               |
| Basin Length                   | 155 m                    |
| Dam Surface Area at 39.0 m AHD | 12,611.9 m <sup>2</sup>  |
| Dam Volume at 38.5 m AHD       | 16,765.42 m <sup>3</sup> |
| Dam Volume at 38.75 m AHD      | 19,441.57 m <sup>3</sup> |
| Dam Volume at 39.0 m AHD       | 21,871.92 m <sup>3</sup> |



**Figure 2.2: Stage-Storage Volume Relationship**



**Figure 2.3: Stage-Storage Surface Area Relationship**



Figure 2.4: Dam Bottom Elevation Survey

## 3. METHODOLOGY

### 3.1 Hydrodynamic Model

The mathematical simulations have been carried out using TUFLOW HPC.

TUFLOW HPC solves the two-dimensional (2D) Saint Venant equations, which describe the 2D motion of open channel flows in the hypothesis of hydrostatic distribution of pressures along the water depth, water velocities uniformly distributed along the water depth, and non-erodible bed.

The equations are solved using a finite volume numerical technique on a regular grid of square elements, which schematizes the study area.

The model also allows the use of one-dimensional (1D) elements, which are dynamically linked to the 2D cells to better schematize all the morphological features of the territory that can potentially affect the flood evolution. In TUFLOW HPC, the 1D Saint Venant equations are solved using an explicit Runge-Kutta numerical approach.

The 1D elements are usually adopted to represent small tributaries, urban channels and pipe network systems, where the use of 2D schemes can bring to problems related to the stability and accuracy of the numerical solution.

In addition, 1D elements are employed to represent all the hydraulic devices which control the flood dynamics within the study area, such as culverts, weirs, sluice gates, pumps, and so on.

### 3.2 Schematization of the Study Area

The extension and geometry of the hydraulic model boundary has been defined taking into account the distinctive morphological features of the territory that can have possible effects in the containment and control of floods, such as terrain slopes, road embankments, natural and artificial levees, thus avoiding any possible effects of the model boundary definition on the maximum flood extent.

In particular, the hydraulic model boundary has been defined taking into account the potentially affected areas and optimize the simulation runtimes. More in detail, the model boundary comprises the dam, the tributary which connects the dam to the main waterway, and the adjacent rural and developed areas (Figure 3.1).

A cell size of 3 m has been adopted to represent the active areas of the 2D domain in order to fulfill the requirements imposed by the mathematical derivation and numerical solution of the shallow water equations solved by TUFLOW.

A 1D scheme has been introduced to better represent the tributary of the main waterway. The channel is characterized by a small width if compared to the adopted 2D cell size, therefore a 2D schematization would not be suitable to correctly represent the channel cross-sections and, hence, its conveyance. Furthermore, the channel is characterized by prismatic cross-sections and not pronounced bends, therefore, the 1D element representation is a suitable schematization for this type of channel geometry.

The 1D channel has been defined by extracting 56 cross-sections from the Digital Elevation Map (DEM) of the terrain and by linearly interpolating the elevation between each section.

Finally, 1D spillway elements have been adopted in the TUFLOW model to better reproduce the functioning of both the high-flow spillway and the dam crest, and correctly calculate the flows discharged above both these spillways and conveyed in the downstream main waterway.



Figure 3.1: Hydrodynamic Model Boundary

### 3.3 Modelling Approach and Assumptions

Detailed geotechnical investigations about the properties and characteristics of the materials used to build the dam are necessary to determine the exact location of the potential breach and its evolution over the time. However, this information is often not available for earthen rural irrigation dams like in the case at hand, and a series of assumptions about breach width, location and timing are usually made to perform this kind of analysis.

In this report, a breach width of 40 m occurring in two different locations has been analysed to take into consideration the most dangerous dam break scenarios in terms of flood extent. The two breach locations investigated in this report are indicated as **Scenario A – Weir** and **Scenario B - Wall** as shown in Figure 3.2.

The breach locations along the southern wall of the dam, were selected where the water head and load are at their highest, and the likelihood of a bank failure is greater.

The **Scenario A – Weir** assumes the occurrence of the bank failure at the dam weir location, i.e. in line with the natural flow path followed by the dam outflows. Conversely, the **Scenario B – Wall** simulates the occurrence of the breach along the south-west side of the dam, i.e. in a location that is not in line with the gully, thus representing the most serious breach occurrence condition.

The dam break analysis has been carried out in accordance with the "Guidelines on the Consequence Categories for Dams" ([ANCOLD-2012]).

The guidelines by [ANCOLD-2012] propose a consequence assessment process composed by three main phases: (1) *Initial Assessment*; (2) *Intermediate Assessment*; (3) *Comprehensive Assessment*.

The first step of the *Initial Assessment* requires to estimate the inundation area, which can be performed through both the analysis of historical dam-break flood maps (where applicable) and mathematical simulations. Since there are no historical dam-break flood maps available for the area at hand, in the present report the dam break analysis has been performed entirely through the support of the mathematical modelling.

According to [ANCOLD-2012], the dam break analysis has been carried out by considering both a **Sunny Day Failure Scenario** and a **Flood Failure Scenario**, as detailed in Sections 3.4.1.1 and 3.5.1.1.

A summary of the analysed dam break scenarios is contained in Table 3.1.

**Table 3.1:** Summary of the Modelled Scenarios

| Scenario No. | Simulation I.D. (Internal use) | Scenario Description   |
|--------------|--------------------------------|--|
| 1A           | 225                            | Sunny Day Failure Scenario – Collapse occurring at weir location |
| 1B           | 235                            | Sunny Day Failure Scenario – Collapse occurring at wall location |
| 2A           | 226                            | Flood Failure Scenario – Collapse occurring at weir location     |
| 2B           | 236                            | Flood Failure Scenario – Collapse occurring at wall location     |

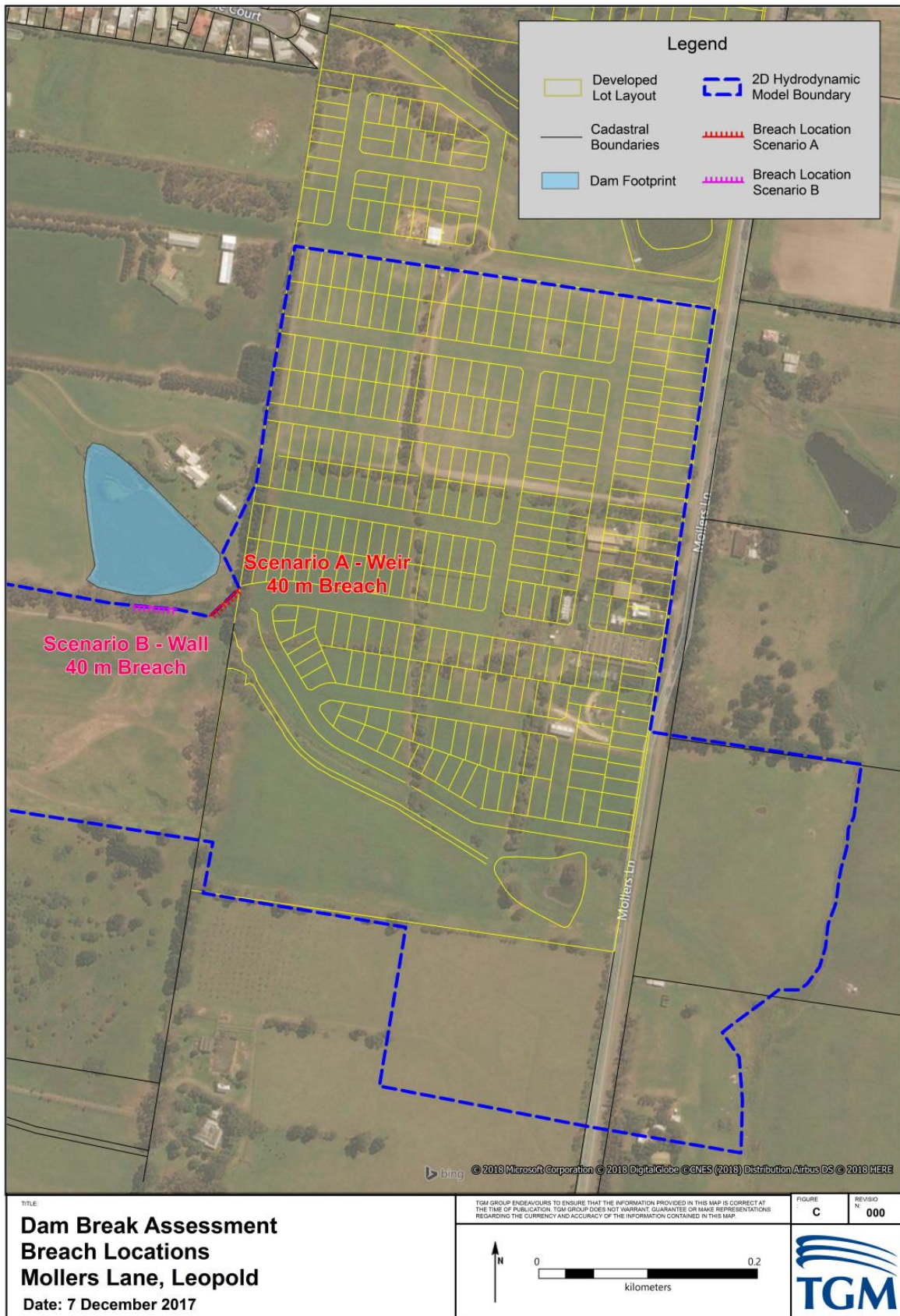


Figure 3.2: Breach Locations

### 3.4 Boundary Conditions

#### 3.4.1 Inflow Boundary Condition

The dam break analysis has been carried out by considering the worst dam burst occurrence, i.e. a sudden collapse of the dam bank and an instant formation of a breach characterised by a width of 40 m. It is worth pointing out that the severity of the hazards depends on failure time, with less serious scenarios occurring for longer failure times. Please note that the adopted breach formation assumptions represent the worst case, since actual dam failure would likely occur over a much longer duration period.

The analysis was performed by applying a hydrograph representing the dam break dynamics, as the inflow boundary condition. The hydrograph is composed by three main phases, as schematised in Figure 3.3.

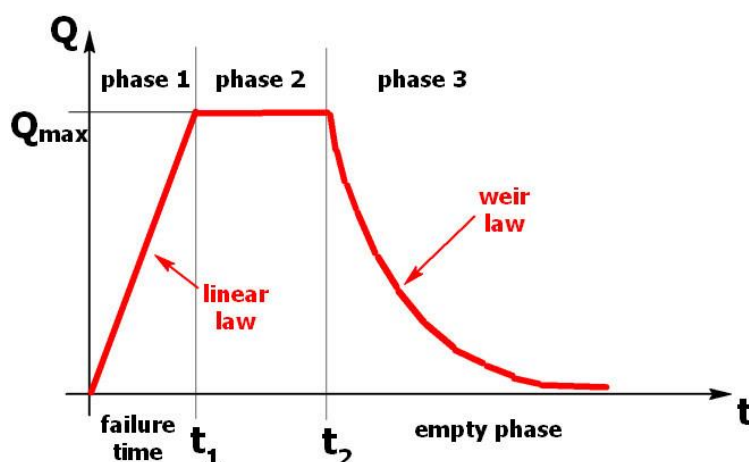


Figure 3.3: Sketch of the hydrograph assumed as boundary condition

In Phase 1, called “failure time”, the hydrograph suddenly reaches the maximum discharge  $Q_{max}$ . This phase is assumed to be as fast as possible to be considered as the worst-scenario condition.

However, an assumption about the failure time has been made to simulate the dam break wave propagation using the hydrodynamic model, thus avoiding numerical instabilities, and the failure time has been conveniently chosen as half of the Phase 2 duration. It is worth noting that this phase usually can vary from few minutes to hours depending on the dam size and characteristics, therefore maximum failure time assumptions of the order of 1 minute are still describing an almost instant collapse of the dam bank, i.e. the worst-case scenario.

The sudden collapse of the dam bank can be holistically treated as the sudden removal of a sluice gate. Indeed, in both the physical phenomena, a negative wave will be produced and the maximum discharge is calculated as:

$$Q_{max} = \frac{8}{27} \sqrt{g} y_0^{3/2} b$$

where  $g$  is the gravity acceleration,  $y_0$  is the water depth and  $b$  is the breach width.

In Phase 2, a negative wave is produced within the basin and the maximum discharge  $Q_{max}$  has been assumed constant between  $t_1$  and  $t_2$ . The time interval  $(t_1 - t_2)$  has been evaluated by considering the time taken by the

negative wave to reach the opposite side of the basin and come back. The negative wave celerity,  $c$ , is calculated as:

$$c = \sqrt{gy_0}$$

with  $g$  and  $y_0$  defined as above, and the time interval ( $t_1 - t_2$ ) is calculated as:

$$(t_1 - t_2) = \frac{L}{c}$$

where  $L$  is the length of the basin.

In Phase 3, called "the emptying phase", the dam gets empty following a "weir mathematical law" determined by the breach characteristics and calculated as:

$$Q_i = Cq\sqrt{2g}y_i^{3/2}b$$

where  $Q_i$  is the discharge calculated at each instant  $i$ ,  $Cq$  is the coefficient of discharge, and  $y_i$  is the water depth at each instant  $i$ . The coefficient of discharge  $Cq$  is usually derived as:

$$Cq = \frac{2}{3}Cc \left[ \left( 1 + \frac{v_0^2}{2gh_0} \right)^{3/2} - \left( \frac{v_0^2}{2gh_0} \right)^{3/2} \right]$$

where  $Cc$  is the contraction coefficient (i.e., a coefficient which takes into account the contraction of the cross-section of the stream at the weir bottom and sides),  $v_0$  is the velocity of approach and  $h_0$  is the water head above the weir.

In the most common applications, the velocity  $v_0$  is negligible and  $Cq$  usually ranges between 0.36 and 0.5 depending on the weir geometry. However, in this scenario the velocity of approach,  $v_0$ , is not negligible, and  $Cq = 0.7$  has been adopted to fulfil this condition.

The following paragraphs briefly summarise the calculations and assumptions made for all the investigated scenarios.

### 3.4.1.1 Sunny Day Failure Scenarios

The **Sunny Day Failure Scenarios** have been performed for both the weir and wall breach locations (respectively Scenario 1A and Scenario 1B).

According to [ANCOLD-2012], in these scenarios it is assumed that the dam break occurs during sunny days, with no stormwater inflow, and the water level within the basin is set at the full supply level.

The initial water level within the basin has been assumed coincident with the bottom weir level, and the initial water depth  $y_0$  has been calculated according to this assumption.

The dam break parameters calculated following the procedure described at Section 3.4 for the **Sunny Day Failure Scenarios** are summarised in Table 3.2.

**Table 3.2:** Boundary Conditions and Parameters – Sunny Day Failure Scenarios

| SUNNY DAY FAILURE - SCENARIO 1A (WEIR) [125] |           |                        |
|--|-----------|------------------------|
| Initial Water Depth – From the Weir Bottom   | $y_0$     | 1.2 m                  |
| Maximum Discharge                            | $Q_{max}$ | 48.8 m <sup>3</sup> /s |
| Celerity of the Negative Wave                | $c$       | 3.43 m/s               |
| Phase 1 – Time                               | $t_1$     | 50 sec                 |
| Phase 2 – Time                               | $t_2$     | 100 sec                |
| Breach Width                                 | $b$       | 40 m                   |
| SUNNY DAY FAILURE - SCENARIO 1B (WALL) [135] |           |                        |
| Initial Water Depth – From the Weir Bottom   | $y_0$     | 1.7 m                  |
| Maximum Discharge                            | $Q_{max}$ | 82.3 m <sup>3</sup> /s |
| Celerity of the Negative Wave                | $c$       | 4.08 m/s               |
| Phase 1 – Time                               | $t_1$     | 40 sec                 |
| Phase 2 – Time                               | $t_2$     | 80 sec                 |
| Breach Width                                 | $b$       | 40 m                   |

The hydrographs applied as inflow boundary conditions for the "**Sunny Day Failure Scenarios**" are shown in Figure 3.4 and Figure 3.5.

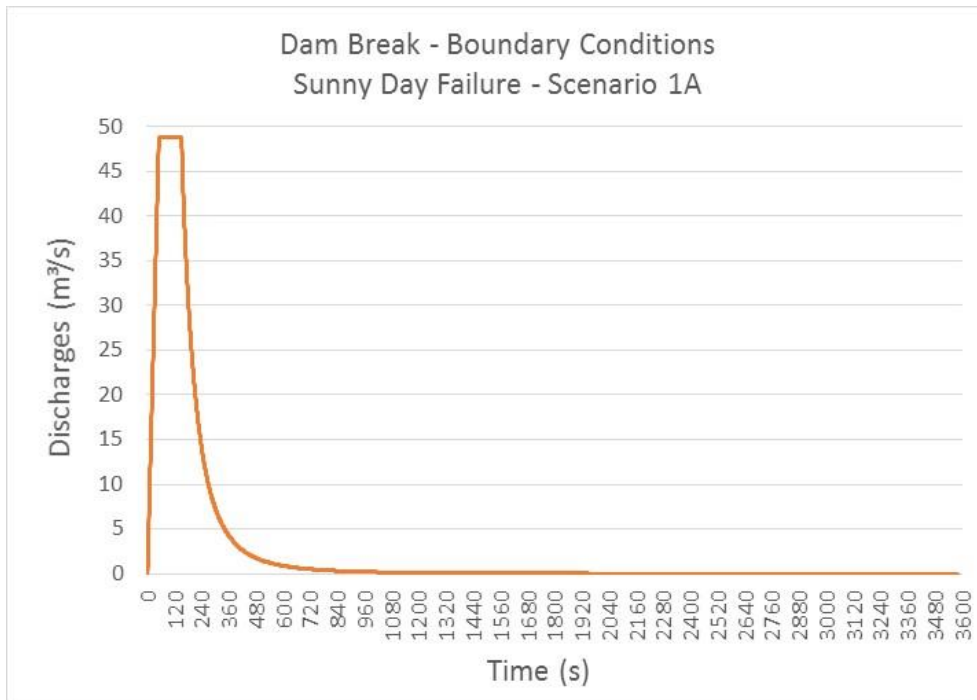


Figure 3.4: Inflow Boundary Condition - Sunny Day Failure - Scenario 1A (Weir)

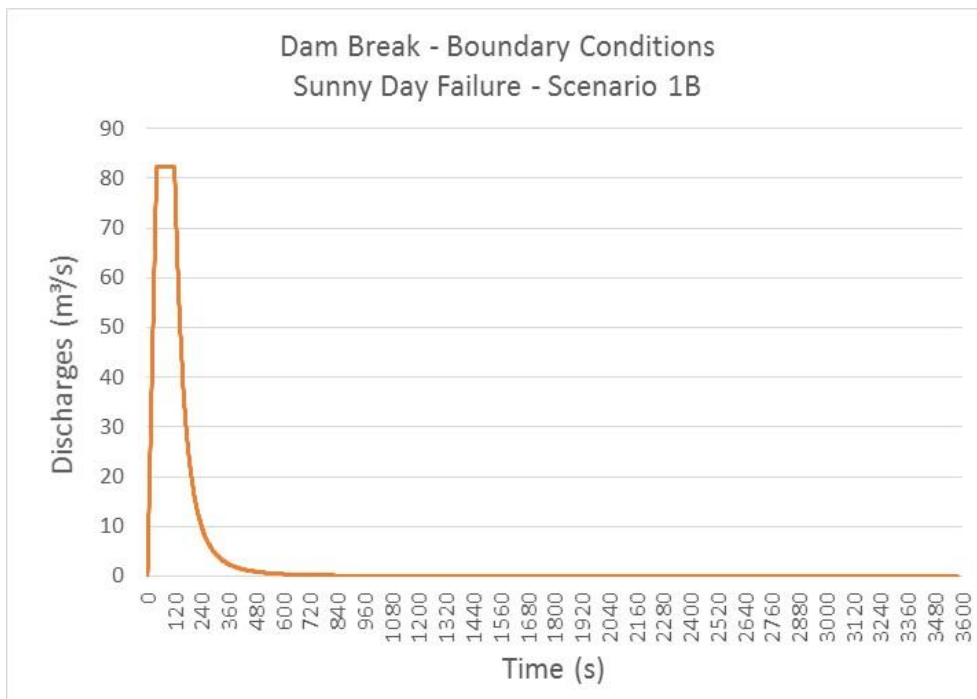


Figure 3.5: Inflow Boundary Condition - Sunny Day Failure - Scenario 1B (Wall)

### 3.5.1.1 Flood Failure Scenarios

In analogy with the previous scenarios, the **Flood Failure Scenarios** have been performed for both the weir and wall breach locations (respectively Scenario 2A and Scenario 2B).

According to [ANCOLD-2012], in the **Flood Failure Scenarios** it is assumed that the dam break occurs during the probable maximum flood, which has been identified as a consequence of the 1% AEP rainfall event.

The 1% AEP initial water level within the dam (38.78 m AHD) has been estimated on the basis of the numerical simulations carried out in [TGM-r2017]. The dam break parameters calculated for the **Flood Failure Scenarios** are summarised in Table 3.3.

**Table 3.3:** Boundary Conditions and Dam Break Parameters – Flood Failure Scenarios

| FLOOD FAILURE - SCENARIOS 2A (WEIR) [126]     |           |                         |
|---|-----------|-------------------------|
| Initial Water Depth (From 1% AEP Water Level) | $y_0$     | 1.48 m                  |
| Maximum Discharge                             | $Q_{max}$ | 66.8 m <sup>3</sup> /s  |
| Celerity of the Negative Wave                 | $c$       | 3.81 m/s                |
| Phase 1 – Time                                | $t_1$     | 40 sec                  |
| Phase 2 – Time                                | $t_2$     | 80 sec                  |
| Breach Width                                  | $b$       | 40 m                    |
| FLOOD FAILURE - SCENARIOS 2B (WALL) [136]     |           |                         |
| Initial Water Depth (From 1% AEP Water Level) | $y_0$     | 1.98 m                  |
| Maximum Discharge                             | $Q_{max}$ | 103.4 m <sup>3</sup> /s |
| Celerity of the Negative Wave                 | $c$       | 4.41 m/s                |
| Phase 1 – Time                                | $t_1$     | 40 sec                  |
| Phase 2 – Time                                | $t_2$     | 80 sec                  |
| Breach Width                                  | $b$       | 40 m                    |

In addition, an initial water level of 20.4 m AHD, which corresponds to the 1% AEP water level, has been assigned within the designed detention basin located at the south-east corner of the developed area.

The dam break analysis for the **Flood Failure Scenarios** has been carried out by overlapping the dam burst at the 1% AEP rainfall event simulated in [TGM-r2017]. Please refer to [TGM-r2017] for a detailed description of the hydrological model calibration and the 1% AEP hydrographs adopted as inflow boundary conditions to carry out the flood simulations.

In the present simulations a new hydraulic computational boundary has been chosen compare to [TGM-r2017], in order to represent the area of interest only and reduce the simulation runtimes. Consequently, some elaborations on the 1% AEP hydrographs derived in [TGM-r2017] have been made to correctly apply the boundary conditions to the new 2D hydrodynamic model boundary.

In particular, the hydrographs computed in different locations of the channel course and adjacent areas have been shifted to the appropriate locations along the model boundary by applying an (hyperbolic) kinematic wave modelling approach. The (hyperbolic) kinematic wave model simplifies the flow motion equations by neglecting the terms characterized by temporal and spatial derivatives, thus allowing to estimate the flood wave characteristics.

Following the kinematic approach, the celerity of the flood wave,  $c$ , is derived as:

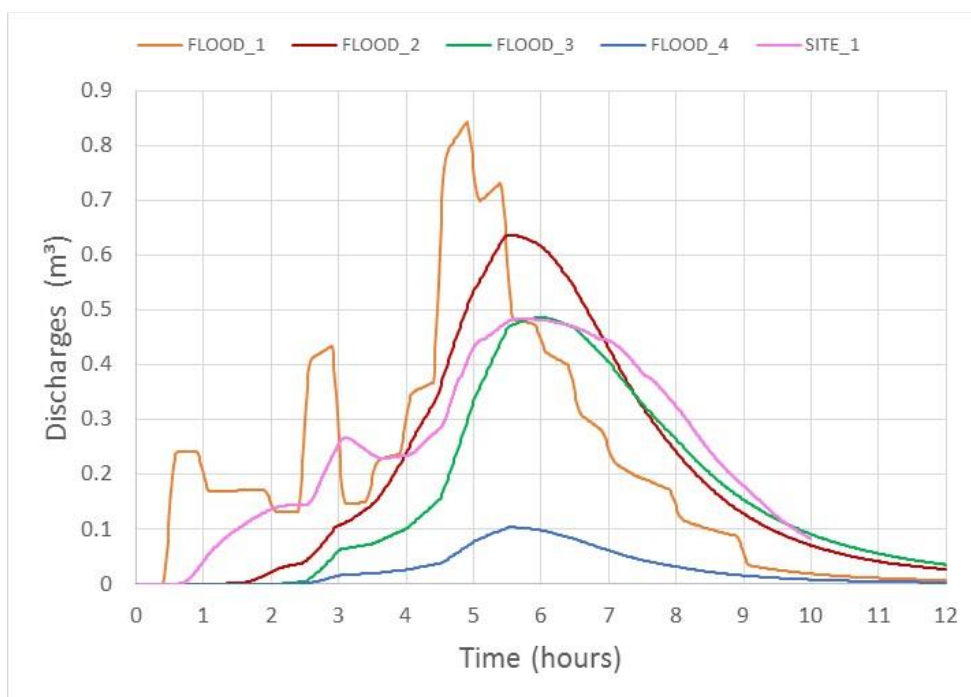
$$c = 1.5 U$$

where  $U$  is the water velocity within the channel. Therefore, the hydrographs have been shifted in time of the quantity  $\Delta t$  determined by:

$$\Delta t = \frac{x}{c}$$

where  $x$  is the distance between the previous and the present hydrograph locations.

The hydrographs obtained following the (hyperbolic) kinematic wave modelling approach are shown in Figure 3.6.



**Figure 3.6:** Flood Failure Boundary Conditions - 1% AEP Flood Hydrographs

The locations where the 1% AEP hydrographs have been assigned are shown in Figure 3.7.

The choice of the timing of the dam burst is crucial in order to investigate the worst-case scenario. Therefore, a dam break occurring at the peak flood event has been simulated, i.e. after 5 hours (18000 seconds) from the beginning of the flood event simulation.

The hydrographs applied as inflow boundary condition for the “**Flood Failure Scenarios**” are shown in Figure 3.8 and Figure 3.9.

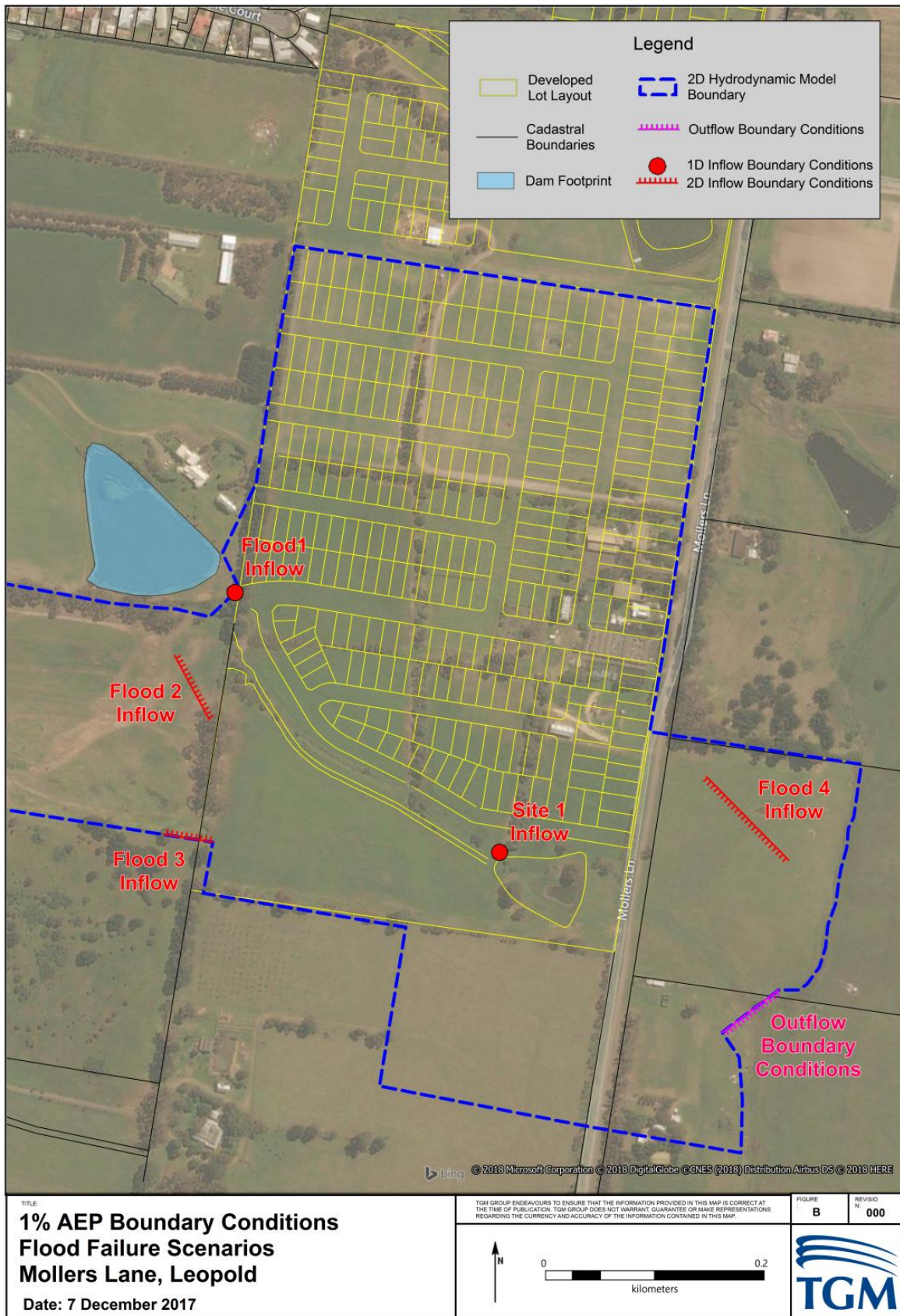


Figure 3.7: Flood Failure - Boundary Condition Locations

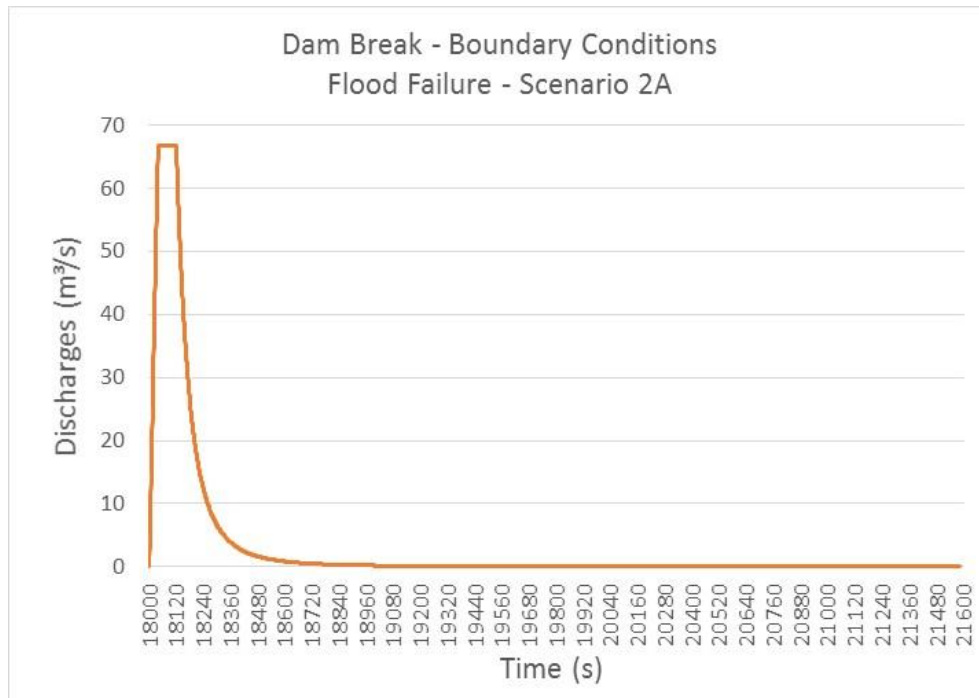


Figure 3.8: Inflow Boundary Condition - Flood Failure - Scenario 2A (Weir)

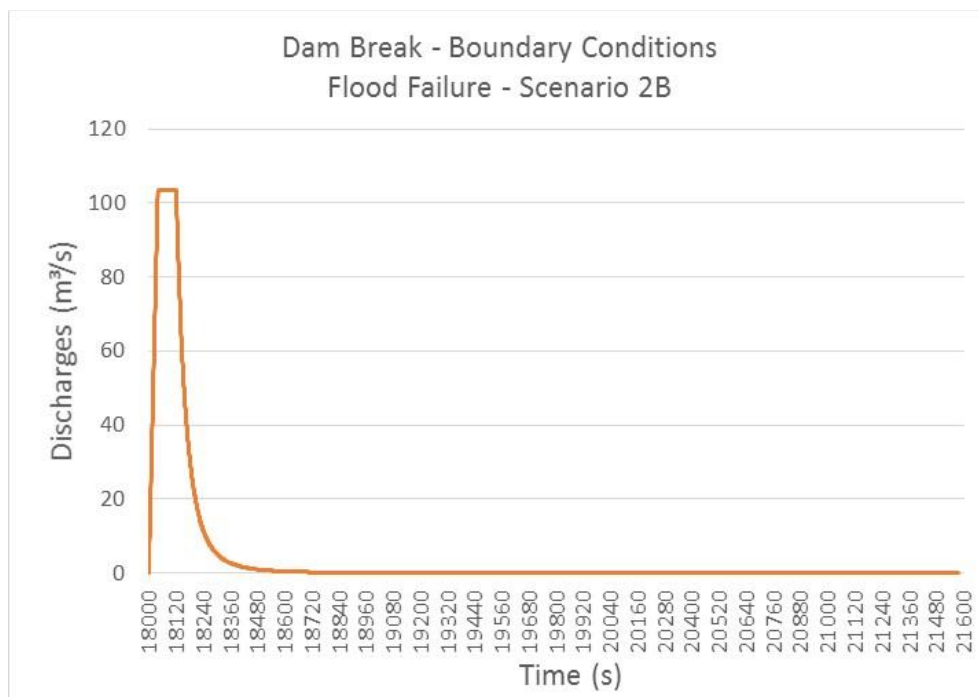


Figure 3.9: Inflow Boundary Condition - Flood Failure - Scenario 2B (Wall)

### 3.5.2 Outflow Boundary Conditions

A stage-discharge relationship in the outflow cross-section of the model has been adopted as outflow boundary condition.

The location of the outflow boundary condition is shown in Figure 3.7, above.

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### 3.6 Initial Conditions

The downstream design detention basin and wetland facility adopted an initial water level condition reflecting the expected capacity of the basin and or wetland facility before a potential dam failure event.

An initial water level equal to the designed Normal Top Water Level (NTWL) has been adopted in the **Sunny Day Failure** simulations. The designed NTWL of the downstream wetland is 19.6 m AHD (Figure 3.10).

An initial water level equal to the designed 1 in 100 years Top Water Level (100YR TWL) has been adopted in the **Flood Failure** simulations. The designed 100YR TWL of the downstream detention dam is 20.4 m AHD (Figure 3.11).



**Figure 3.10: Initial Conditions - Sunny Day Failure Scenarios**



**Figure 3.11: Initial Conditions - Flood Failure Scenarios**

## 4. MODELLING RESULTS

### 4.1 Sunny Day Failure Scenarios

#### 4.1.1 Flood Depths and Extent

The maximum water depths and flood extents and flood hazards resulting from the **Sunny Day Failure Scenarios** simulations are shown in Figure 4.1 for the “weir” location (Scenario 1A), and in Figure 4.2 for the “wall” location (Scenario 1B).

The numerical simulation results for both 1A and 1B Scenarios show negligible impacts on the developed site.

In both breach location scenarios, the flows generated by the dam break are almost completely contained or collected within the gully and discharged within the designed detention basin.

In the Scenario 1A, where the breach is located in line with the channel, the dam break outflows ( $Q_{max} = 48.8 \text{ m}^3/\text{s}$ ) are completely contained within the channel and produce no impacts on the developed site area.

In the Scenario 1B, where the breach is not located in line with the natural flow path and the maximum discharge resulting from the dam break is far higher ( $Q_{max} = 82.3 \text{ m}^3/\text{s}$ ), the breach outflows follow the maximum terrain gradients, thus promoting once again the capture of the flows within the channel and the conveyance of flows to the main waterway.

On the whole, in the **Sunny Day Failure Scenarios** water depths ranging between 50 and 500 mm occur along the developed site roads adjacent to the channel, with no impacts on the properties.

#### 4.1.2 Hazard Maps

The flood hazard envelope resulting from the **Sunny Day Failure Scenarios** simulations are shown in Figure 4.3 for the “weir” location (Scenario 1A), and in Figure 4.4 for the “wall” location (Scenario 1B).

The hazard maps have been created in accordance with the Safety and Hazard Criteria defined by ARR Project 10, which state that flow velocity, depths and the product of velocity and depth must not exceed safety limits for people and vehicle access (egress) to (from) the site. The criteria are as follows:

1. Site Safety (People) [ARR 2010]
  - Depth must be no greater than or equal to 0.5 metres;
  - Velocity must be no greater than or equal to 3.0 m/s; and
  - The product of depth multiplied by velocity must be no greater than or equal to 0.4 m<sup>2</sup>/s.
2. Access Safety (Vehicles) [ARR 2010]
  - Depth must be no greater than or equal to 0.3 metres;
  - Velocity must be no greater than or equal to 3.0 m/s; and
  - The product of depth multiplied by velocity must be no greater than or equal to 0.3 m<sup>2</sup>/s.

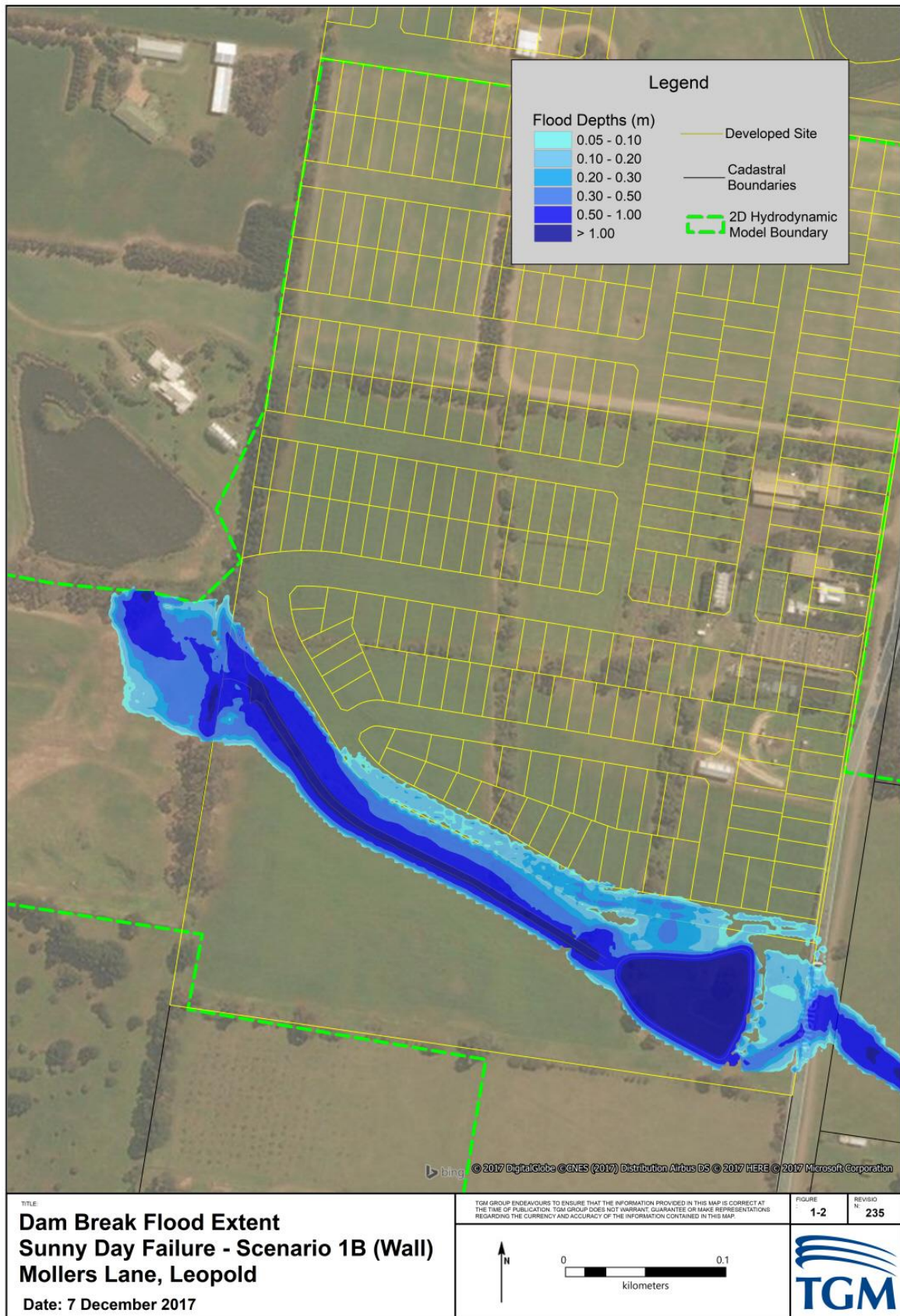
The numerical simulation results show some differences in the flood extent, maximum water depths and hazards when considering different breach locations.

In Scenario 1A, the flood hazard along the southern road of the new development is classified as safe for both people and vehicles.

In Scenario 1B, some significant flood hazards are localised in a small area along the southern road of the developed site.



Figure 4.1: Sunny Day Failure Scenario 1A (Weir Location) - Maximum Water Depths



**Figure 4.2: Sunny Day Failure Scenario 1B (Wall Location) - Maximum Water Depths**



Figure 4.3: Sunny Day Failure Scenario 1A (Weir Location) - Hazard Envelope

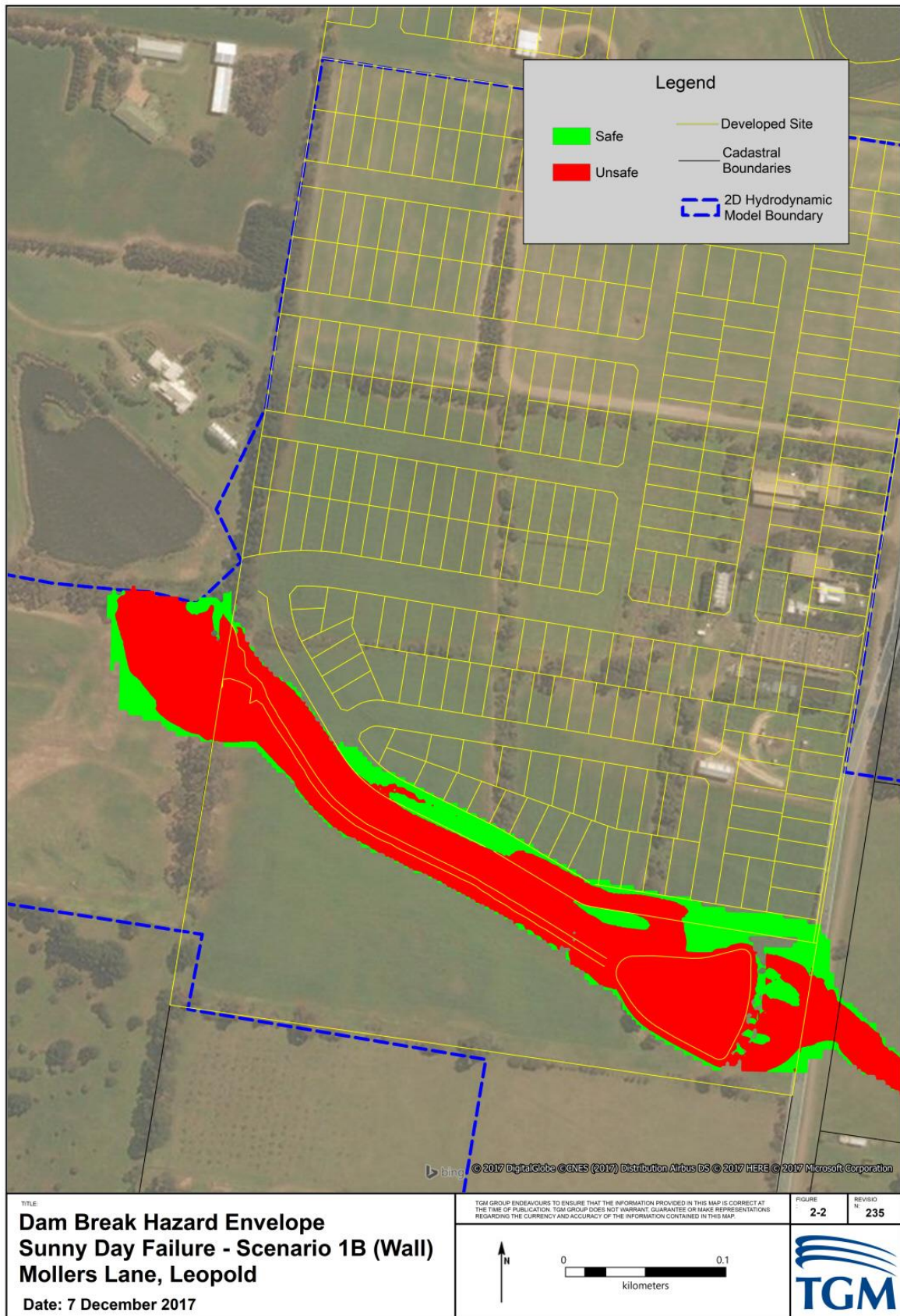


Figure 4.4: Sunny Day Failure Scenario 1B (Wall Location) - Hazard Envelope

## 4.2 Flood Failure Scenarios

### 4.2.1 Flood Depths and Extent

The maximum flood depths and extent resulting from the **Flood Failure Scenarios** simulations are shown in Figure 4.5 for the “weir” breach location (Scenario 2A), and in Figure 4.6 for the “wall” breach location (Scenario 2B).

Please note that the flood maps in Figure 4.5 and Figure 4.6 focus on the water depths greater of 50 mm, thus neglecting the sheet flows characterised by water depths smaller than 50 mm.

The **Flood Failure Scenarios** take into account the combined effect of the 1% AEP rainfall event and the dam burst occurring at the maximum peak flood, i.e. after 5 hours from the beginning of the rainfall event.

In these scenarios, the flooding affects, as expected, a larger area than in the **Sunny Day Failure Scenarios**. However, the impacts of floods are not significant in the developed site and small differences in the flood extent can be detected for the two breach location scenarios.

In both breach scenarios, the dam break outflows affect the southern roads of the developed area only, where the water depths range between 50 and 500 mm, with no impacts on the properties.

### 4.2.2 Hazard Maps

The hazard envelopes resulting from the **Flood Failure Scenarios** simulations are shown in Figure 4.7 for the “weir” location (Scenario 2A), and in Figure 4.8 for the “wall” location (Scenario 2B).

The hazard maps have been created accordingly to the procedure detailed in Section 4.1.2.

Please note that the hazard envelopes have been produced by considering the sheet flows, which are defined by water depths lower than 50 mm. For this reason, the hazard maps show a greater flood area when compared to the flood extent maps.

In both Scenarios 2A and 2B hazard impacts are detected along the southern road of the new development, which is classified as unsafe according to ARR Project 10.

In the Scenario 2B, the properties of the new development are skimmed by the floods only and no hazards are detected within the properties. Please note that the interactions between the properties and the floods take into consideration the sheet flows, therefore, no flood impacts actually affect the properties.

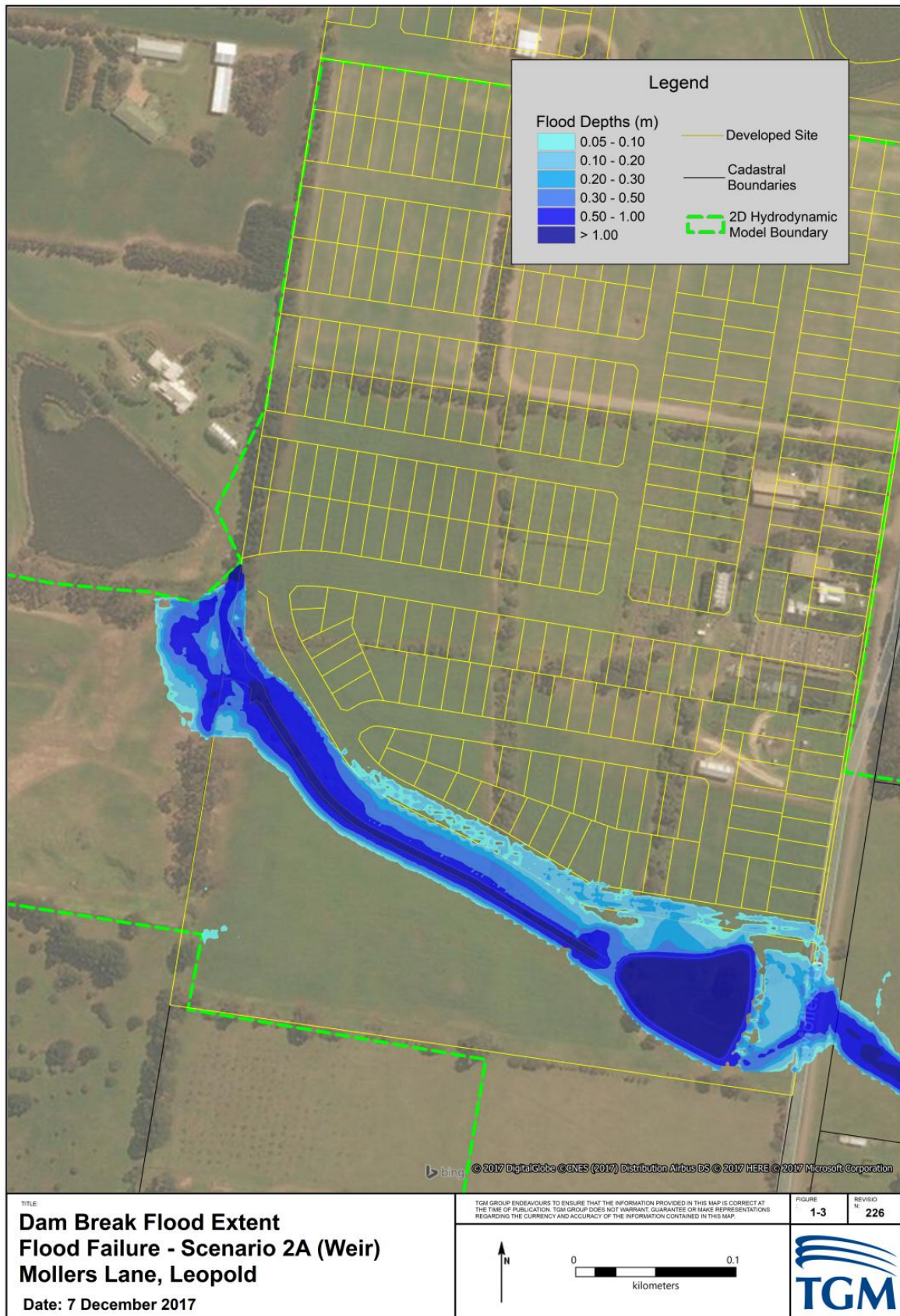


Figure 4.5: Flood Failure Scenario 2A (Weir Location) - Maximum Water Depths

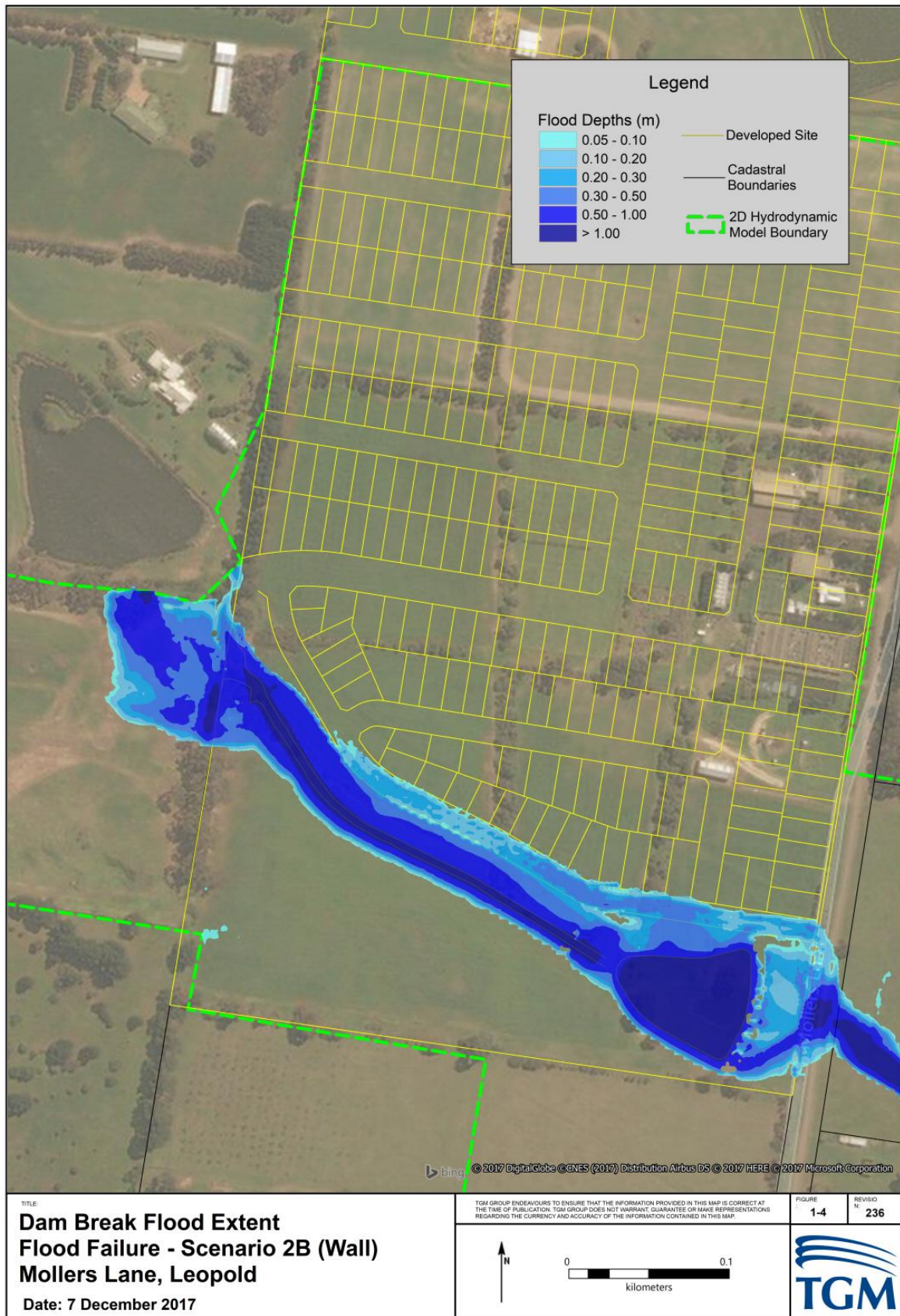


Figure 4.6: Flood Failure Scenario 2B (Wall Location) - Maximum Water Depths



Figure 4.7: Flood Failure Scenario 2A (Weir Location) - Hazard Envelope

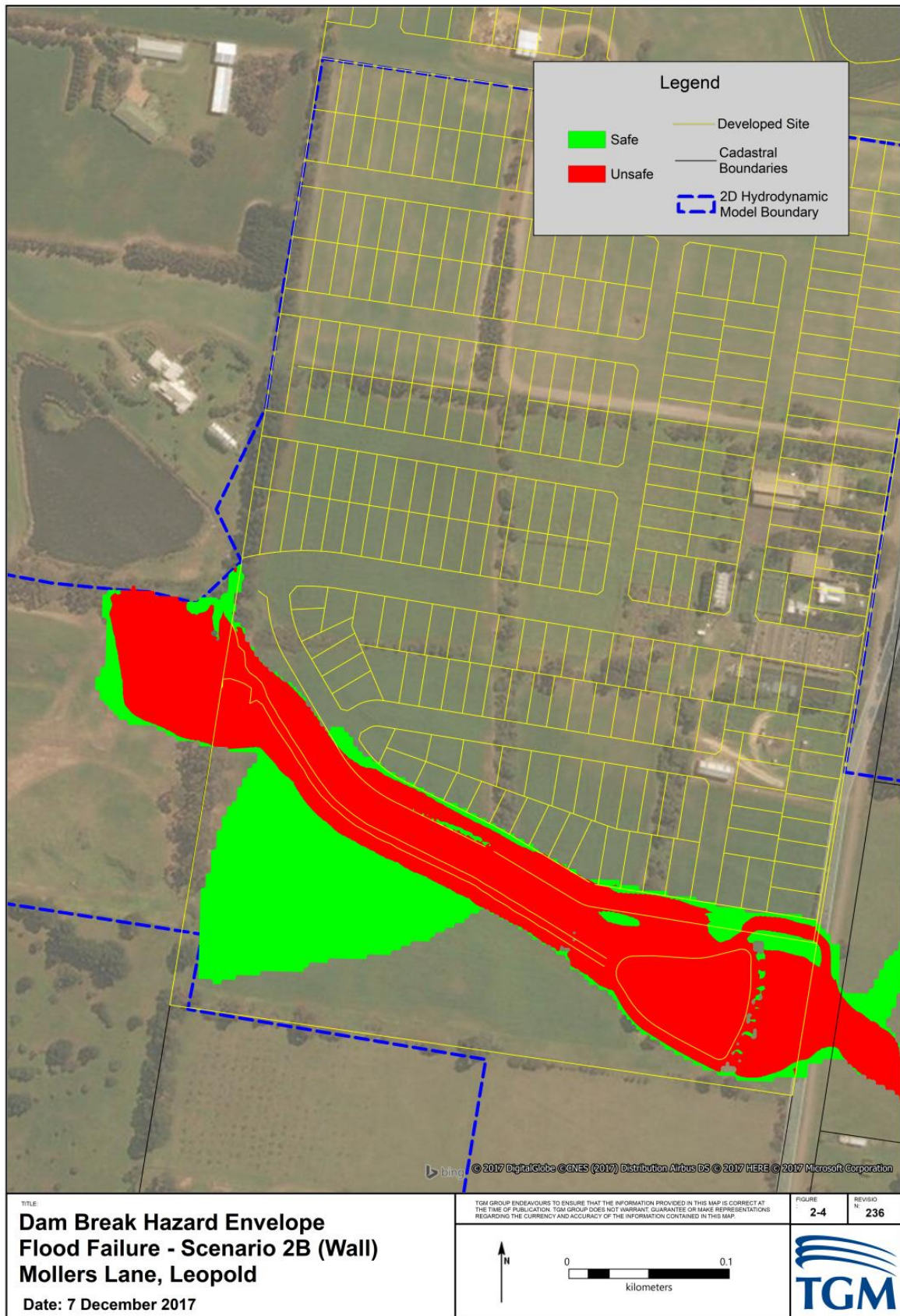


Figure 4.8: Flood Failure Scenario 2B (Wall Location) - Hazard Envelope

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## 5. CONSEQUENCE ASSESSMENT

### 5.1 Estimate of the total loss and damage

According to **[ANCOLD-2012]**, the *Initial Assessment* stage requires to estimate the potential direct and indirect loss/damage resulting from the dam failure.

The potential loss/damage is evaluated with respect to the following four categories:

1. Total infrastructure costs;
2. Impact on dam owner's business;
3. Health and social impacts;
4. Environmental impacts.

The severity of each of these potential damages and losses have been classified according to **[ANCOLD-2012]** and summarised in Table 5.1.

The mathematical simulations of the dam break flood propagation show that the most catastrophic flood event occurs within ten minutes, with maximum depths of 500 mm. The inundated area largely consists of a sealed asphalt road, which is characterised by a low degree of potential scouring.

In these circumstances, no damage is expected to affect the community infrastructure along the southern road of the developed site.

The highest level of severity of damage and loss is classified as Minor.

**Table 5.1:** Estimate of Potential Direct and Indirect Loss/Damage

| Damage and Loss                                     | Severity Level | Description of the Damage/Loss                             |
|---|----------------|--|
| <b>Total Infrastructure Costs</b>                   |                |  |
| Residential   | Minor          | No houses affected   |
| Commercial  | Minor          | No commercial business affected                            |
| Community Infrastructure                            | Minor          | No damage to the new development                           |
| Dam replacement or repair cost                      | Minor          | The dam repair is expected not to be significant           |
| <b>Impact on dam owner's business</b>               |                |  |
| Importance to the business                          | Minor          | Restrictions needed for a limited time period              |
| Effect on services provided by the owner            | Minor          | No services are provided to the community                  |
| Effect on continuing credibility                    | Minor          | No effects on the business credibility                     |
| Community reaction and political implications       | Minor          | No community reaction and political implications           |
| Impact on financial viability                       | Minor          | Able to absorb in less than one financial year             |
| Value of water in the storage                       | Minor          | Can be absorbed in less than a financial year              |
| <b>Health and social impacts</b>                    |                |  |
| Public Health                                       | Minor          | Less than 100 people potentially affected                  |
| Loss of Service to the community                    | Minor          | Less than 100 people potentially affected                  |
| Cost of emergency management                        | Minor          | Less than 1000 person days                                 |
| Dislocation of people                               | Minor          | No dislocation is necessary                                |
| Dislocation of businesses                           | Minor          | No dislocation is necessary                                |
| Employment affected                                 | Minor          | No employment affected                                     |
| Loss of heritage                                    | Minor          | No loss of heritage  |
| Loss of recreational facility                       | Minor          | No loss of recreational facilities                         |
| <b>Environmental impacts</b>                        |                |  |
| Area of impact                                      | Minor          | Less than 1 km <sup>2</sup>                                |
| Duration of impact                                  | Minor          | Less than 1 day  |
| Stock and fauna                                     | Minor          | No contamination of water supplies used by stock and fauna |
| Ecosystems  | Minor          | No impact on ecosystems                                    |
| Rare and endangered species                         | Minor          | No impact on rare and endangered species                   |
| <b>Highest level of severity of damage and loss</b> | <b>Minor</b>   |  |

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## 5.2 Estimate of Total Population At Risk (PAR)

The estimate of Total PAR takes into consideration all people who would be directly exposed to flood waters assuming they took no action to evacuate.

The estimated inundation area affects the southern road of the developed site. Therefore, the road users can be considered the only PAR.

The risks posed to road users is determined by considering the number of people present in or driving into the flooded areas during or after the flood wave passes through.

The affected road is located in a residential area and it is expected to be categorized by a very low traffic density.

The length of the road affected by the flood is 308 m. The speed limit in residential areas is 50 km/h, therefore, the expected average travel time is less than 30 seconds.

According to the mathematical simulations, the comprehensive effect of the flood wave induced by the dam break lasts ten minutes.

On the basis of all the above considerations, the probability of a vehicle to be in the inundation zone during the dam break flood event is negligible.

## 5.3 Consequence Category

The Consequence Category of the dam break is assigned on the basis of the Total PAR and severity of damage and loss according to **[ANCOLD-2012]**.

With a total PAR lower than one and a Minor severity of damage and loss, the Consequence Category is classified as Very Low.

Since the analysis has highlighted a Very Low Consequence Category, no further detailed analysis is required to refine the assessment and the assessment process can be considered concluded at the *Initial* stage.

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## 6. CONCLUSIONS

According to [ANCOLD\_2012], the *Initial Assessment* of the consequences of the dam break has been carried out by considering both **Sunny Day Failure Scenarios** and **Flood Failure Scenarios**, as well as two different locations of the breach on the south wall of the dam, in order to analyse the worst consequences in terms of floods.

The mathematical modelling investigations on the **Sunny Day Failure Scenarios** show no impact to the properties of the proposed development. Indeed, for the both breach location scenarios the water flows resulting from the dam failure are collected within the channel and conveyed into the main waterway.

The mathematical simulations of the **Flood Failure Scenarios**, where the dam break has been assumed occurring during the peak flow of the 1% AEP rainfall event show no adverse impacts on adjacent properties. Although the consequences in terms of flood extent and depths are higher compared to the **Sunny Day Failure Scenarios**, the floods only impact the southern roads of the proposed development, with no potential damage to the properties.

The dam break consequence assessment has highlighted minor severity of damage and loss, and no population at risk. In these circumstances, the Consequence Category of the dam break is classified as Very Low.

Since the estimation of the potentially inundated areas have highlighted no impact on the properties and Very Low Consequence Category, the analysis of the dam break consequences is to be considered concluded at the *Initial Assessment* step according to the assessment procedure suggested by [ANCOLD-2012].