Elton reservoir

Dam break and flood inundation

MCR5780-RT002-R01-00

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Executive summary

HR Wallingford are undertaking a feasibility study, on behalf of the Canal & River Trust, the objective of which is to determine the impact of potential development on the reservoir and its flood plain. This report describes the dam breach modelling and the subsequent inundation modelling for the existing situation.

The breach modelling was undertaken using the EMBREA model for three flow conditions and three locations along the dam, a total of nine scenarios. The analysis showed that piping is the most likely failure mode for the Elton embankment dam and no overflowing occurred along the crest in all of the model runs.

The breach modelling results showed that the peak outflow ranges from approximately 15 m^3/s at section 1 with a 1 in 1,000 year flow event, to 97 m^3/sec at section 3 with the Probable Maximum Flood (PMF) inflow event. The hydrograph with the highest peak outflow for each flow event represents the worst case scenario, although the peak of the breach outflow hydrographs and the resulting flood depth and flow speed for a given breach location do not differ significantly by varying the flow event.

A 2D hydraulic model has been developed for the valley downstream of the dam to simulate the inundation due to breaching of the dam. The breach flow hydrograph outputs from the EMBREA modelling were adopted as input to the inundation model for each of the nine scenarios.

The flood inundation modelling shows that an urbanised area of North Radcliffe is at risk from inundation from a breach in the reservoir. The flood extent, flood depth, flow speed and hazard ratings are greatest for a breach at the location of the outlet pipes and smallest for a breach at the location of the spillway.

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1. Introduction

The Elton reservoir was built between 1804 and 1808 to supply water to the Manchester, Bolton and Bury (MBB) Canal. This canal is now no longer in use, so the reservoir is not now required to provide lockage water, although it does provide a sweetening flow. The location of the reservoir is shown in Figure 1.1.

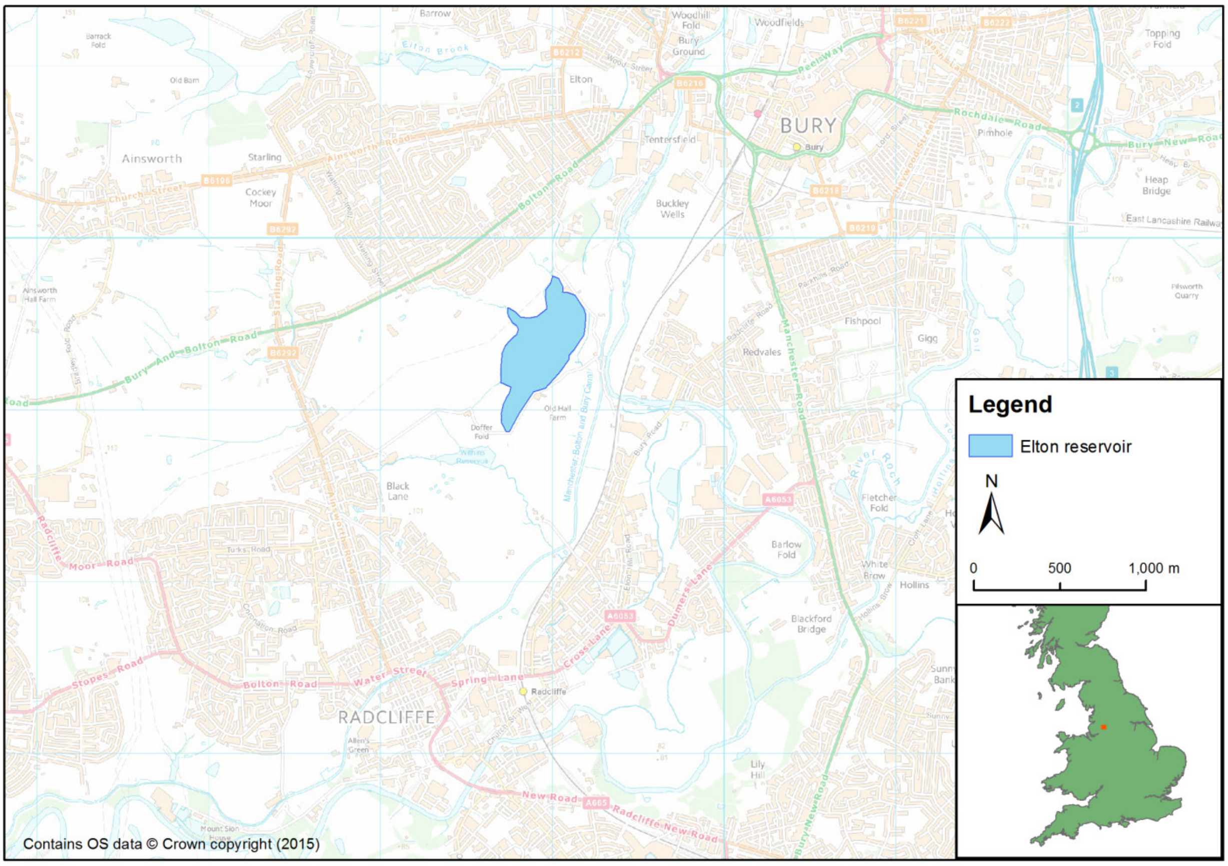

Figure 1.1: Elton reservoir

According to the Prescribed Form of Record, the reservoir, which is retained by a dam with a maximum height of 8.8 metres, has a capacity of 923,000 m^3 and a surface area of 22 hectares. Top Water Level in the reservoir is 87.70 m AOD.

The catchment draining to the reservoir is relatively small (given as 172 hectares in the Prescribed Form of Record). Approximately 42% of the catchment area is said to be urbanised in the outskirts of Bury.

The reservoir also receives water from the ‘Elton Feeder’, which discharges into the northern end of the reservoir. This feeder runs from an offtake on the River Irwell approximately 3 km north of the reservoir, which is controlled by sluice gates. The length of the feeder is longer than 3 km and it is culverted along much of its length.

1.1. Context to the flood risk study

Bury Council and the Greater Manchester Combined Authority (GMCA) have allocated the land around Elton Reservoir for housing development in the Draft Greater Manchester Spatial Framework (GMSF). Peel and the Canal & River Trust are principal landowners in the proposed allocation (alongside owners) and are supportive in principle of the allocation. The Canal & River Trust has raised the issues relating to Elton Reservoir in its response to the GMSF consultation. A masterplan to underpin the GMSF allocation needs to be prepared. It is therefore necessary to assess the impact of the reservoir on the proposals, identify where development may be located and what, if any, mitigation may be needed in to achieve that development. This may be example in relation to the Reservoir itself, the feeder channels and MBB Canal that the Reservoir outfalls to, or the engineering and drainage of adjacent land in order to achieve flood defences/levels.

The Canal & River Trust and Peel agreed that HR Wallingford undertake a feasibility study, on behalf of the Canal & River Trust to a brief to be agreed with Peel. The objective of the study is to determine the impact of proposed development on the reservoir and its flood plain.

1.2. Scope of this report

This report describes the dam breach modelling and the subsequent inundation modelling for the existing situation.

1.3. Review of the hydrological study

The hydrological study (MCR5780_RT001_R01-00) derived design inflow hydrographs for the following events required for the study:

- Probable Maximum Flood (PMF)
- 1 in 10,000 year flow
- 1 in 1,000 year flow.

Design inflow hydrographs were produced for the 1 in 1,000 year return period, the 1 in 10,000 year return period and the PMF using the best available catchment parameters, design rainfall and rainfall-runoff models. Peak inflows to the reservoir have been estimated as:

- 69.1 m^3/s for the PMF;
- 37.4 m^3/s for the 1 in 10,000 year return period;
- 15.3 m^3/s for the 1 in 1,000 year return period.

2. Dam breach modelling

The objective of the breach modelling work was to provide boundary conditions for the flood inundation model, which is being used to estimate the flood extents, depths, velocities and risk. To achieve this, the EMBREA model was used to model the breaching process for three embankment sections at the Elton dam.

2.1. An overview of the EMBREA model

In the development of the EMBREA (EMbankment BREAch) model, HR Wallingford undertook extensive research to identify the best approaches and tools to model the breaching of embankments and embankment dams. This included reviewing the existing methodologies used to model failure and breach (Mohamed 1998 and 2002), developing an improved model for failure and breach (Mohamed et al. 2002a) and testing the performance of existing tools against field and laboratory physical modelling (IMPACT 2005).

The early research into breach modelling approaches and models showed that a number of deficiencies existed. This led HR Wallingford to developing the EMBREA model to help meet industry needs for the prediction and management of dam breach formation due to overtopping or piping through flood defence embankments and embankment dams. EMBREA draws on research work undertaken around the world and at HR Wallingford, providing a state-of-the-art tool for predicting breach growth. The principal model output gives an estimate of the rate at which an embankment might fail under different hydraulic conditions and the associated outflow hydrograph.

Research within the EC Funded IMPACT Project (www.impact-project.net) has shown that the performance of the EMBREA[1] model is, on average, the best out of the models considered. This was based on comparison of outputs with the data of 5 prototype scale field tests and 22 laboratory tests, an extensive number of validation tests compared to existing tools. HR Wallingford has also continued to refine and extend the capabilities of this modelling tool through the Company's research programme and through the EC FLOODsite Project (www.floodsite.net).

Based upon this research and proven model performance, the EMBREA model has therefore been adopted in this study to model the piping failure of the Elton reservoir embankment.

Modelling piping in EMBREA is carried out assuming that a circular pipe has already been established along the embankment between the reservoir and the downstream face[2]. The model then simulates the following consecutive processes:

- Erosion of the material in the pipe;
- The collapse of the top part of the dam, above the pipe, either under its own weight or by the water pressure forces;
- Erosion of the dam body in a similar way to an overtopping failure.

Modelling overtopping in EMBREA is carried out assuming that an initial breach channel through the crest and downstream face is initiated. This 'initiation' channel constrains the initial breach flow and provides the focal point for breach simulation. In practical terms, this simulates a hole or dip in the embankment crest that might arise for a number of reasons, resulting in the focus of overtopping flow, leading to a breach. If the initial or subsequent water level is below the initiation channel invert level then no erosion takes place in the breach channel (i.e. no overtopping). If the water level exceeds the initiation channel invert and the embankment crest level, then overtopping flow for the breach channel and overflow over the crest are both calculated. In the latter case, the model simulates the following:

- Continuous erosion of material through the breach channel; and,

[1] Previously known as "HR Breach"

[2] The initiation of the overtopping and piping processes is not modelled in EMBREA.

- Slope instability of the breach channel sides. [It should be noted that this mechanism can cause jumps in the outflow hydrograph. When this happens, a slope instability occurs at the control section (i.e. the section at which the flow is calculated), which leads to a rapid widening of the breach and this consequently leads to a rapid increase in the outflow value. This process was encountered in this study and is noted in the results section for the breach modelling (see Section 2.4).]

These two processes, described above, continue until the flow conditions do not allow any further erosion.

2.2. Breach locations

Modelling runs were undertaken for three embankment sections along the Elton dam to establish the outflow hydrograph from the failure at those locations. These locations are:

- Next to the spillway (will be called location 1 subsequently in this document);
- At the section that slipped recently (will be called location 2 subsequently in this document);
- Next to the outlet (will be called location 3 subsequently in this document).

Figure 2.1 shows the locations of the sections that were modelled.

Figure 2.1: Breach locations

2.3. Model setup

This section provides a description of the model set up, including modelling boundary conditions, initial conditions and embankments' geometry and soil parameters.

- Upstream boundary condition: The following three flow events have been considered:
  - 1 in 1,000 year
  - 1 in 10,000 year
  - Probable Maximum Flood (PMF).

  The derivation of the inflow hydrographs to the reservoir for these events is described in the Hydrological Study (MCR5780-RT001-R02-00).

- Downstream boundary condition: It was assumed that the breach is not drowned (that the flow rate through the breach is not constrained by the downstream water level) in all model runs at the three sections;
- Embankment geometry: See Table 2.1.

Table 2.1: Embankment geometry

Parameter/Defence	Location		
	1	2	3
Crest level (m AOD)	88.78	88.4	88.77
Ground level (m AOD)	85.5	81.29	79.05
Crest width (m)	5	5	5
Downstream Slope (1:x)	2	2	2
Upstream slope (1:x)	3	3	3
Core crest level (m AOD)	87.55		
Core face slopes (1:x)	0.25*		

*Notes: * Assumed. Note that the narrower the core the more conservative the output will be. A slope of 1 in 0.25 is one of the steepest core slopes that we have ever come across.*

- Failure modes: Based on the upstream conditions and the embankment geometry (see Table 2.1), the failure mode for each section was defined. Overtopping was considered unlikely, since the reservoir levels do not exceed the crest levels for the above mentioned modelled events. Piping failure mode was considered as the likely failure mode in this case, given the reservoir levels and, hence, only piping failure was modelled.
- Initial conditions: As described in Section 2.1, the EMBREA model assumes that an initial circular pipe for piping failure mode has been already established, in order to model the failure. Therefore, a pipe with a 0.30 m diameter was assumed to be formed along the defence to initiate the piping failure. The level of this pipe was varied to locate the level that would produce the highest breach outflow for the three locations as shown in Table 2.2.

Table 2.2: Selection of the pipe level (where the bold entry depicts the largest outflow for that section of dam)

Location	Pipe level (m AOD)	Peak outflow (m^3/s)	Selected piping level (m AOD)
1	**86.00**	**17.83**	86.00
	86.50	16.33	
	87.00	14.12	
2	82.50	61.81	85.00
	83.00	60.94	
	83.50	61.58	
	84.00	62.30	
	84.50	64.52	

Location	Pipe level (m AOD)	Peak outflow (m^3/s)	Selected piping level (m AOD)
	85.00	**66.08**	
	85.50	23.98*	
3	79.50	90.84	80.00
	80.00	**96.74**	
	80.50	95.85	
	81.00	80.61	

*Notes: * Top of the pipe did not fail.*

- Soil properties: The embankment was modelled as a two layer embankment. Based on the soil investigation report (Norwest Holst Soil engineering Ltd.,1995) that was provided by the Client, the following soil properties were used for each layer for the three sections (see Table 2.3).

Table 2.3: Soil properties

Parameter	Outer layer	Inner layer (core)
Porosity	0.37	0.40
Dry unit weight (kN/m^3)	16.5	16.5
% of clay	7.5	15**
Friction Angle	36.5	35.0
Cohesion (kN/m^2)	0.3	10.0**
Plasticity index	13	13
Erodibility coefficient (cm/N.s)	1.5*	1.0*
Critical shear stress (N/m^2)	0.1**	0.5**

** Assumed based on typical soil values given by Terzaghi *et al* (1996) and previous experience.

* Estimated based upon the % of clay and dry unit weight using the following equation (Temple *et al.,* 1994):

$$K_d = \frac{10\gamma_w}{\gamma_d}\exp\left[-0.121(C\%)^{0.406}\left(\frac{\gamma_d}{\gamma_w}\right)^{3.10}\right]$$

Where:

- k_d : Erosion rate (cm^3/N-s)
- C% : Percent of clay
- y_d : Dry unit weight (ton/m^3)
- y_w : Unit weight of water in (ton/m^3).

Out of the soil parameters assumed or estimated above, model results are typically most sensitive to the value of k_d. Therefore, the impact of increasing k_d value by 25 and 50 percent for the outer and clay layers on the model results was investigated in Section 2.4.1.

2.4. Modelling results

Table 2.4 shows the results summary by location. Results show that the breach peak outflow increases as flow event increases (i.e. the event becomes severer), which is as expected. The highest breach peak outflow is estimated at section 3 under the PMF event. It can also be seen that at a section, the breach peak outflows do not differ significantly by varying the flow event. This is because the additional volume in the design events is relatively small compared to the total volume in the reservoir, with the water level at the top level of the spillway crest.

Table 2.4: Results summary by location

Location	Flow event	Breach Peak Outflow (m^3/s)
1	1000	14.8
	10000	16.2
	PMF	17.8
2	1000	58.9
	10000	64.0
	PMF	66.1
3	1000	87.6
	10000	92.5
	PMF	96.7

Figures 2.2, 2.3 and 2.4 show the outflow hydrographs of each flow event for three modelled breach locations. A number of jumps in the outflow hydrograph can be seen in those figures as described in Section 2.1.

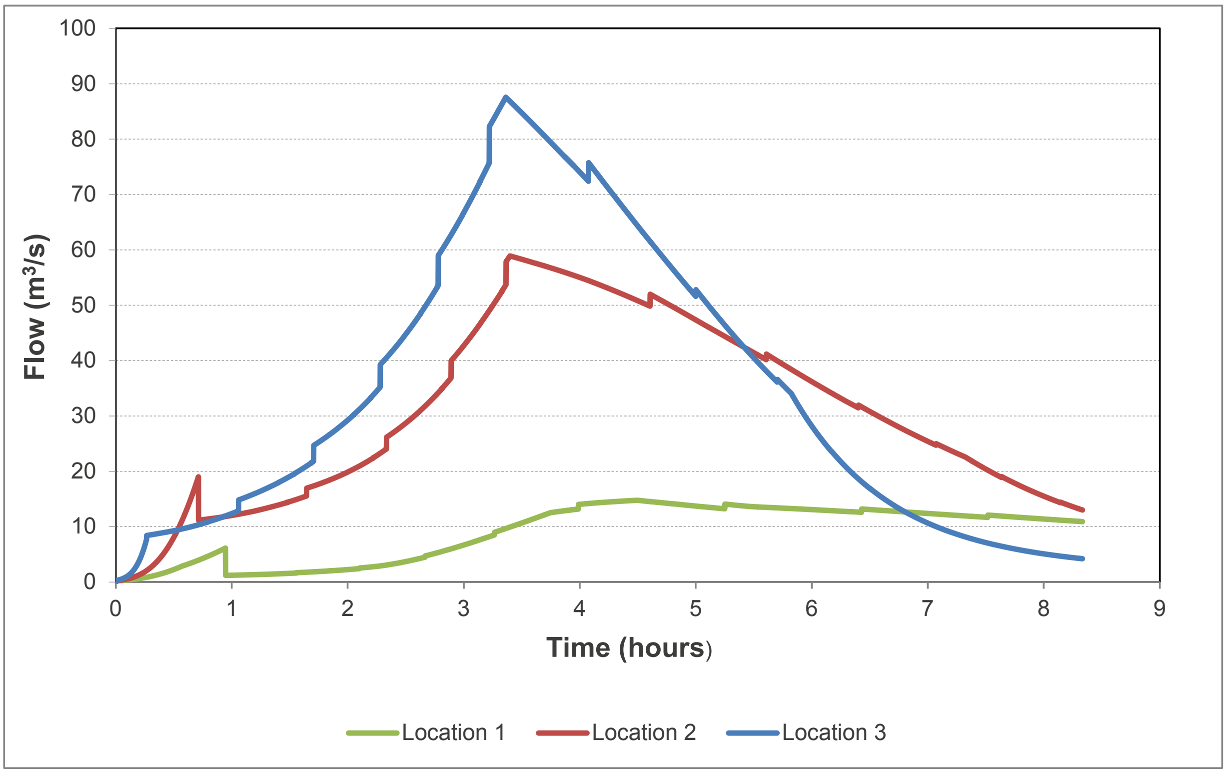

Figure 2.2: 1 in 1,000 year outflow hydrographs

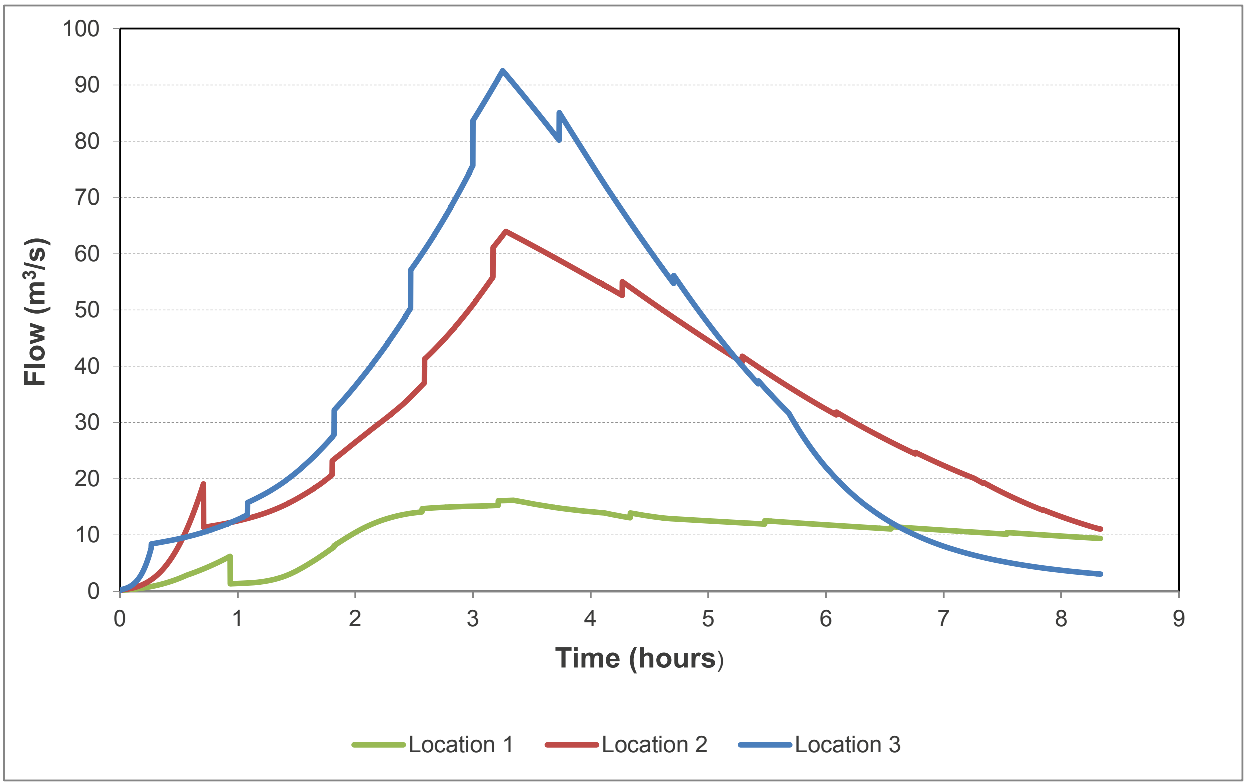

Figure 2.3: 1 in 10,000 year outflow hydrographs

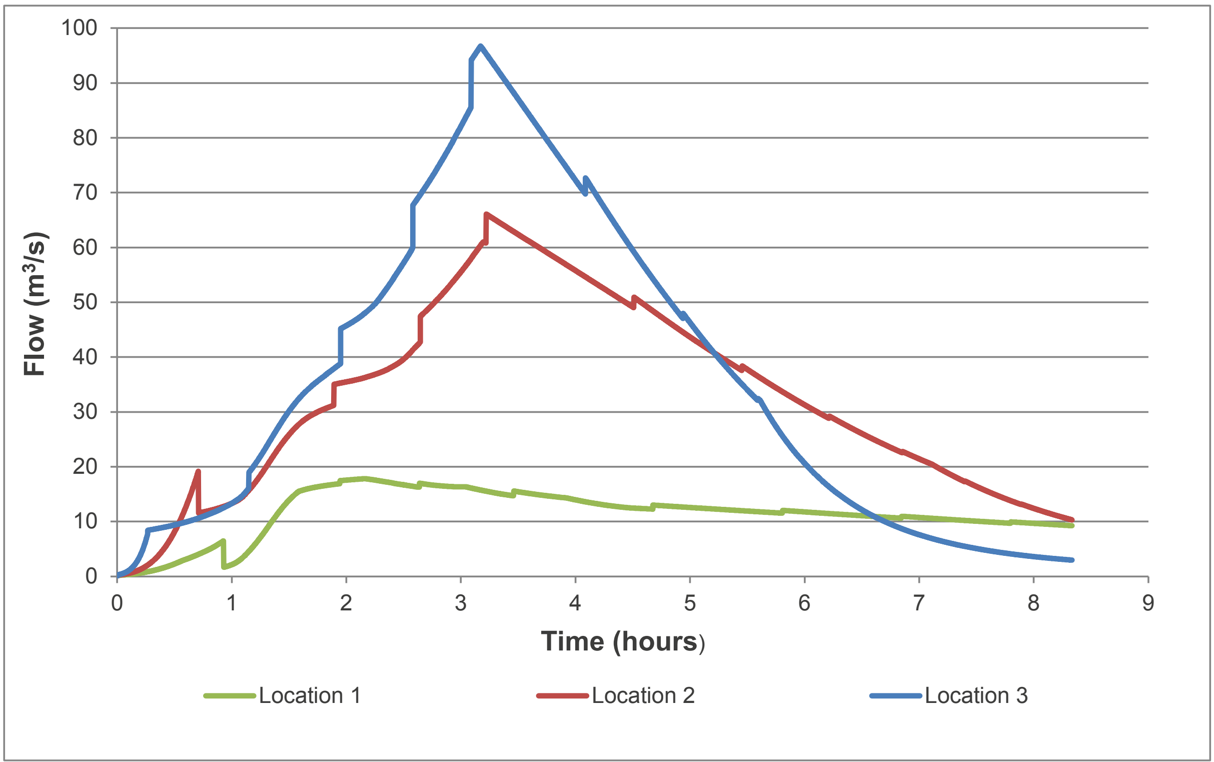

Figure 2.4: PMF outflow hydrographs

2.4.1. Sensitivity of model results to soil erodibility

As mentioned earlier, models results can be sensitive to the value of k_d. A number of sensitivity runs were undertaken to investigate the impact of increasing k_d values by 25 and 50 percent on the peak outflow value. Those runs were only undertaken for failure at location 3 with the PMF flow event. Table 2.5 shows the k_d values that have been used in those runs with the corresponding peak outflow values and percentage of change in those values compared to the base run. It can be seen that increasing k_d values by 25 and 50 percent resulted in an increase in the peak outflow by 11.5 and 18.5 percent respectively. The impact of this increase on the flood maximum depth, arrival time and extents is presented in Section 3 of the report.

Table 2.5: Sensitivity run inputs and results

Run	k_d (cm^3/N-s)		Peak outflow (m^3/s)	% Change compared to base run
	Outer layer	Inner layer (core)		
Sensitivity run 1	1.875	1.25	107.8	11.5
Sensitivity run 2	2.25	1.5	114.6	18.5

2.5. Observations and conclusions

The following points may be concluded from the results of breach modelling work undertaken:

- Breach modelling was successfully undertaken using the EMBREA model for three flow events and sections along the dam.
- Piping is the likely failure mode for the Elton embankment dam and no overflowing occurred along the crest in all of the model runs.
- The breach modelling results show that the peak outflow ranges from approximately 15 m^3/s at location 1 with a 1 in 1,000 year flow event, to 97 m^3/sec at location 3 with the PMF flow event. The hydrograph with the highest peak outflow for each flow event should be used in the inundation modelling to represent the worst case scenario.
- The peak of the breach outflow hydrographs per section do not differ significantly by varying the flow event.

3. Inundation modelling

3.1. Introduction

A 2D hydraulic model has been developed to simulate the inundation in the valley downstream of the dam due to breaching of the dam. Immediately downstream of the reservoir, the floodplain is currently agricultural fields, separated from the urbanised areas of South Bury and North Radcliffe by the canal and the railway line (see Figure 1.1).

3.2. Model description

The 2D numerical model has been constructed for the area using the Innovyze software Infoworks ICM. The 2D model in ICM solves the Shallow Water Equations (SWE), the depth averaged Navier-Stokes equations, using the first-order finite volume explicit scheme. The 2D algorithm is appropriate for representing rapidly varying flows (including shock capturing) as well as super-critical and trans-critical flows, which makes it particularly suitable for simulating dam break flows.

The software uses an irregular triangular mesh which benefits from:

- greater computational efficiency compared to a regular mesh approach, ensuring quicker model run times;
- the use of terrain sensitive meshing where the size of the mesh elements vary according to the complexity of the terrain. This means that smaller sized elements are used where there are sharp changes in topography. In theory, this means that features such as embankments or river channels can be modelled in a greater level of accuracy than with a fixed rectangular grid;
- better representation of linear features that are not directly aligned with the orientation of the regular mesh. This would create a stepped profile of the feature in the regular grid, which can affect the accuracy of the flow velocity.

3.2.1. Model mesh

The model mesh has been produced using terrain sensitive meshing where finer resolution mesh elements are used in areas of greatest topographic slope and coarser resolution mesh elements in areas with less variable topography. The maximum mesh size was set to 100 m^2 and the smallest to 4 m^2.

The mesh was manually refined to ensure that it represented the width of the canal, the bank elevation of the canal, the dimensions of the spillway channel and the stream draining from the valves, and the banks of the River Irwell.

The resulting mesh is shown in Figure 3.1.

Figure 3.1: Model computational mesh

Notes: *The mesh elements are shown in white. The dark colours indicate low elevation and the light colour high elevation*

The roughness in the 2D model varies spatially based on the land cover, where the roughness value in each mesh element has been set based on the predominant land cover at that location. The roughness values for each land cover category are given in Table 3.1 and shown spatially in Figure 3.2.

Table 3.1: Spatial roughness values

Land cover	Manning n value
Open Floodplain – Grass, pasture, marsh	0.060
Roads	0.025
River	0.030
Standing water	0.020
Trees and dense vegetation	0.100
Buildings	10.000

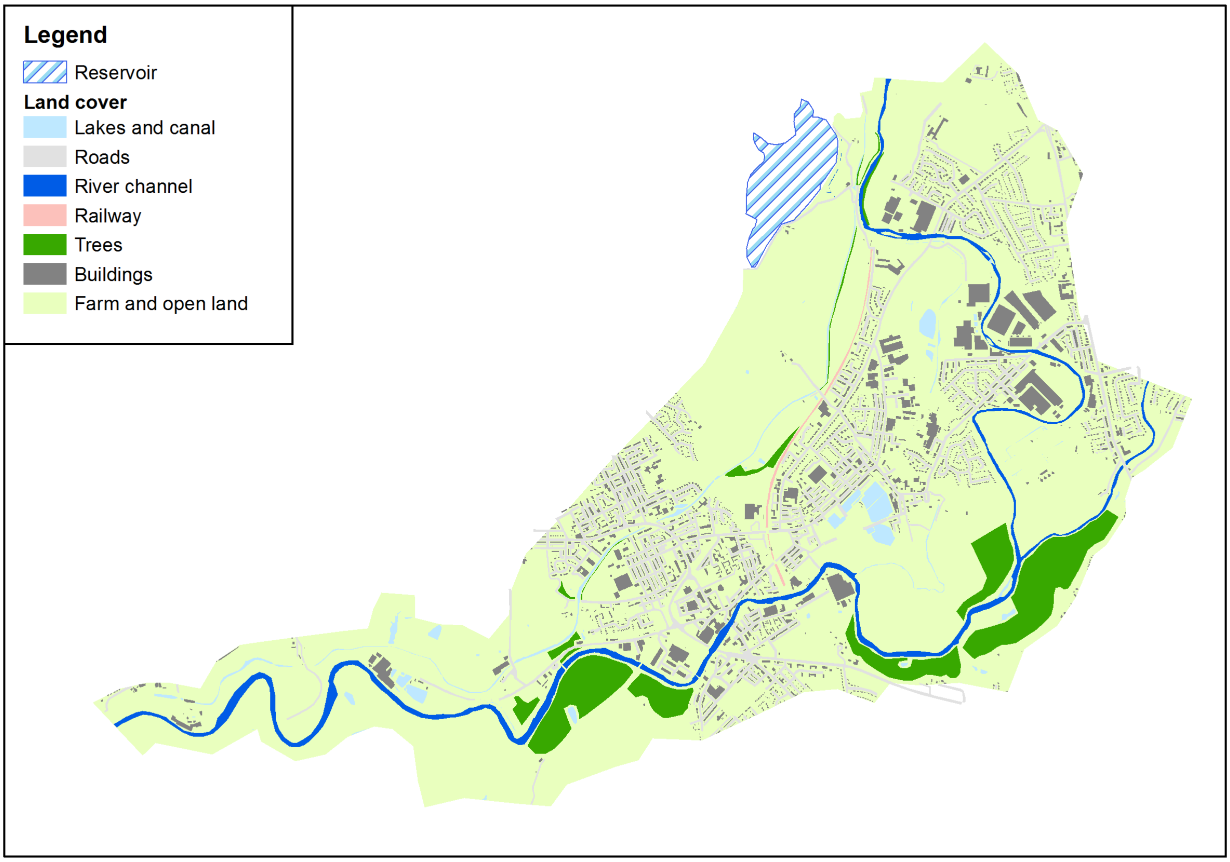

Figure 3.2: Land cover in the 2D model domain

3.2.2. Boundary conditions

The model contains inflow boundaries for the three different dam breach locations and the upstream extent on the River Irwell. The model contains a normal depth relationship between water level and flow at the downstream end of the model extent on the River Irwell. The location of the inflow boundaries are shown in Figure 3.3.

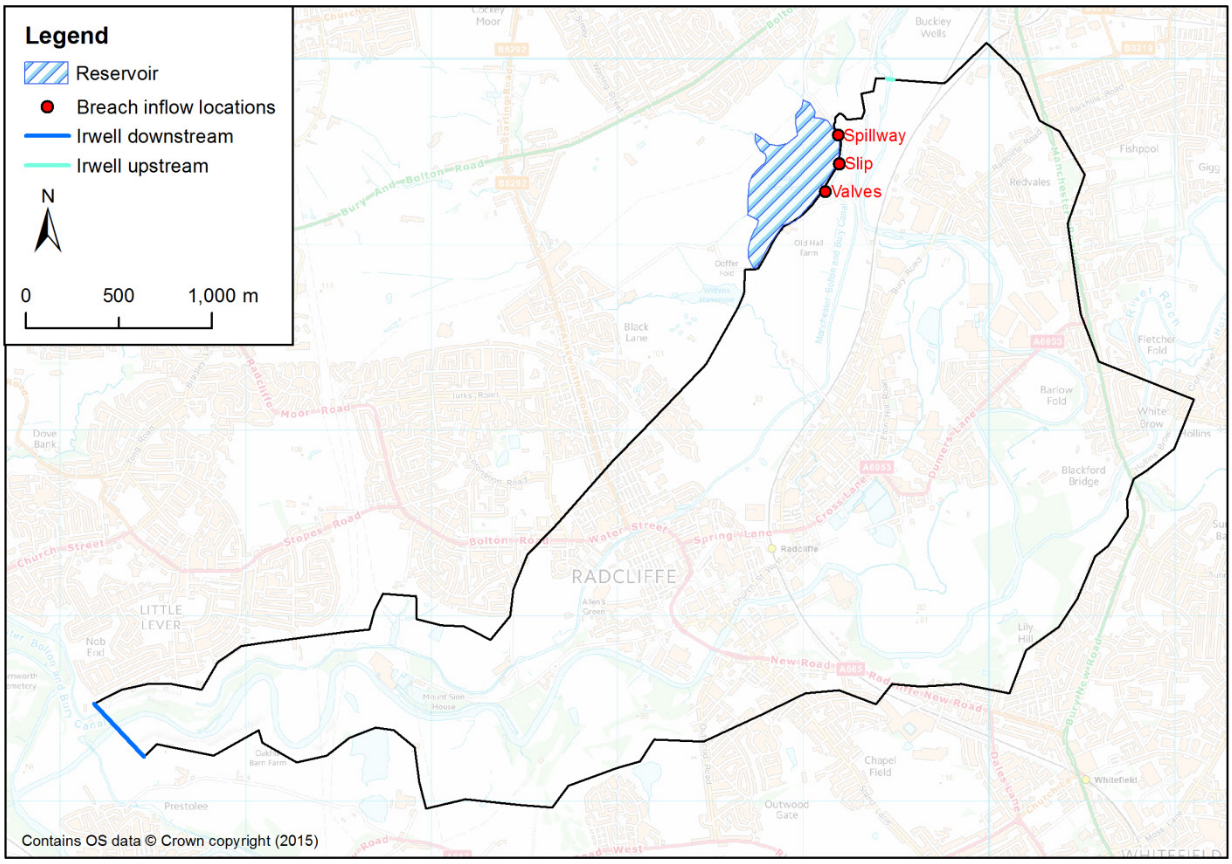

Figure 3.3: Flow boundary locations

The annual average flow on the River Irwell at Bury Ground (69044) is ~3.5 m^3/s and the Q50 (flow that is exceeded 50% of the time) from long term data is ~1.5 m^3/s. It is reasonable to assume that the LiDAR level represents the water surface for a flow condition in this range.

The Median Annual Flood (QMED) flow on the River Irwell at Bury Ground is 115 m^3/s. The model has been run with flow conditions on the Irwell of:

- 110 m^3/s;
- 70 m^3/s;
- 50 m^3/s;
- and 25 m^3/s.

In order to identify the flow condition that, on top of the elevations from LiDAR, would produce an initial condition for the Irwell with the water level around the bankfull level, taking into account that the model does not contain the true river bed elevation. These tests showed overbank flow occurred in a large number of locations for a flow of 110 m^3/s. A flow of 70 m^3/s showed less flooding and a flow of 50 m^3/s was predominantly in-bank except for the industrial area downstream of the railway bridge. It is possible that the LiDAR does not capture the bank level in this location and that the industrial area may be defended by a wall defence.

These runs indicate that adding a flow of 50 m^3/s to the River Irwell produce water levels that are roughly bankfull and suitable to be used in coincidence with the dam breach.

3.3. Scenarios

The following breach scenarios have been simulated in the 2D inundation model:

- Breach at location 3 with the PMF inflow to the reservoir;
- Breach at location 3 with the 1 in 10,000 year return period inflow to the reservoir;
- Breach at location 3 with the 1 in 1,000 year return period inflow to the reservoir;
- Breach at location 2 with the PMF inflow to the reservoir;
- Breach at location 2 with the 1 in 10,000 year return period inflow to the reservoir;
- Breach at location 2 with the 1 in 1,000 year return period inflow to the reservoir;
- Breach at location 1 with the PMF inflow to the reservoir;
- Breach at location 1 with the 1 in 10,000 year return period inflow to the reservoir;
- Breach at location 1 with the 1 in 1,000 year return period inflow to the reservoir.

Note that for all breach scenarios, an inflow of 50 m^3/s has been applied to the River Irwell that produces the bankfull water level condition prior to the breach.

3.4. Results

3.4.1. Breach at location 3 with the PMF inflow to the reservoir

The maximum water depth, flow speed and hazard rating are shown in Figure 3.4 to Figure 3.6. The flood hazard rating was prepared using the hazard rating equation produced as part of the Environment Agency's guidance on Flood Risk to People. The equation estimates the hazard posed to people exposed to flooding based on the flow depth, velocity and the probability of debris being carried by the floodwater.

The hazard rating takes all of these factors into account, with the equation given below:
$HR=d(v+0.5)+DF$

Where:

- HR is the hazard rating;
- d is the maximum flood depth (m);
- v is the maximum velocity of the floodwater (m/s);
- DF is a non-dimensional debris factor which is between 0 and 1.

The value of the debris factor used in the above equation is dependent on the probability that debris will lead to a significant hazard. Most guidance recommends the use of a depth-varying debris factor with a non-zero value at low depths. This approach has been adopted for this study. For depths of 0.00 to 0.25 m, a value of DF of 0.5 was used. Where flood depths were greater than 0.25 m, or velocities were greater than 2 m/s and depths were greater than 0.1 m, a value of DF of 1 was used.

The hazard rating values have been classified into bands relating them to the hazard the floodwater poses to people. The most recent guidance on these hazard classifications is shown in Table 3.2.

Table 3.2: Flood hazard classifications

Flood hazard rating	Hazard class	Danger posed to people by floodwater
0.00	No hazard	None
>0.00 to <0.75	Very low	Caution: Flood zone with shallow flowing water or deep standing water
0.75 to <1.25	Moderate	Danger for some: This includes children, the elderly and the infirm. It is a flood zone with deep or fast flowing
1.25 to 2.00	Significant	Danger for most: This includes the general public. It is a flood zone with deep fast flowing water.
>2.00	Extreme	Danger for all: This includes the emergency services. It is a flood zone with deep fast flowing water.

Source: Udale-Clarke et al, 2005

The results show highest flood depth, velocity and hazard immediately downstream of the breach. There is also high flood depth on the fields either side of the canal, which lead to significant and extreme hazard classification. At present these areas are un-populated.

There are, however, areas of significant flood hazard in the urban area north of Radcliffe, due to the combination of flood depths and flow velocity. The time to first inundation of this urban area is 2 hours 10 minutes from the initiation of the breach.

The industrial area between the River Irwell and the east side of the railway also has significant hazard rating, mainly due to the flood depth.

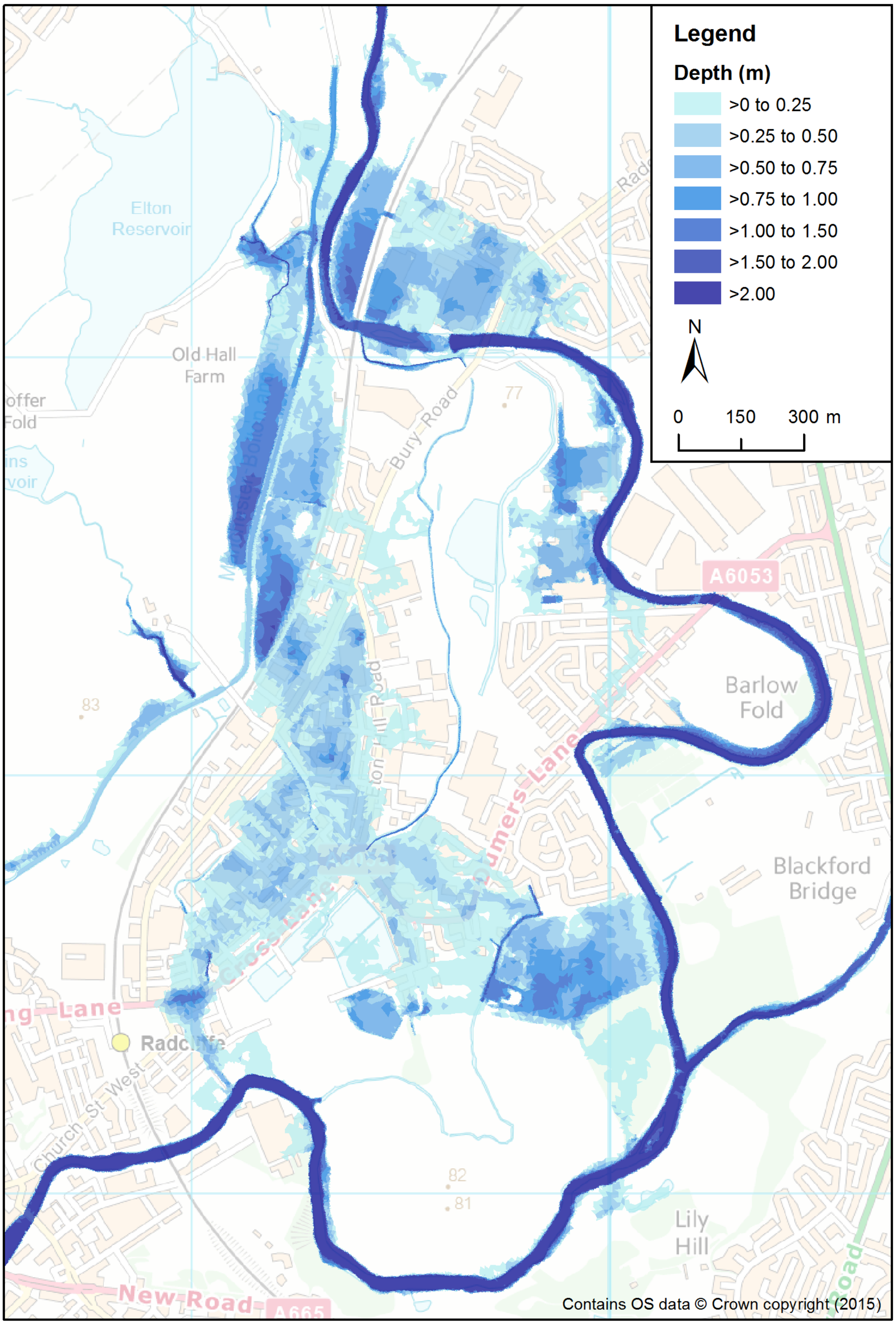

Figure 3.4: Maximum water depths with a breach at the outlet pipes in the PMF

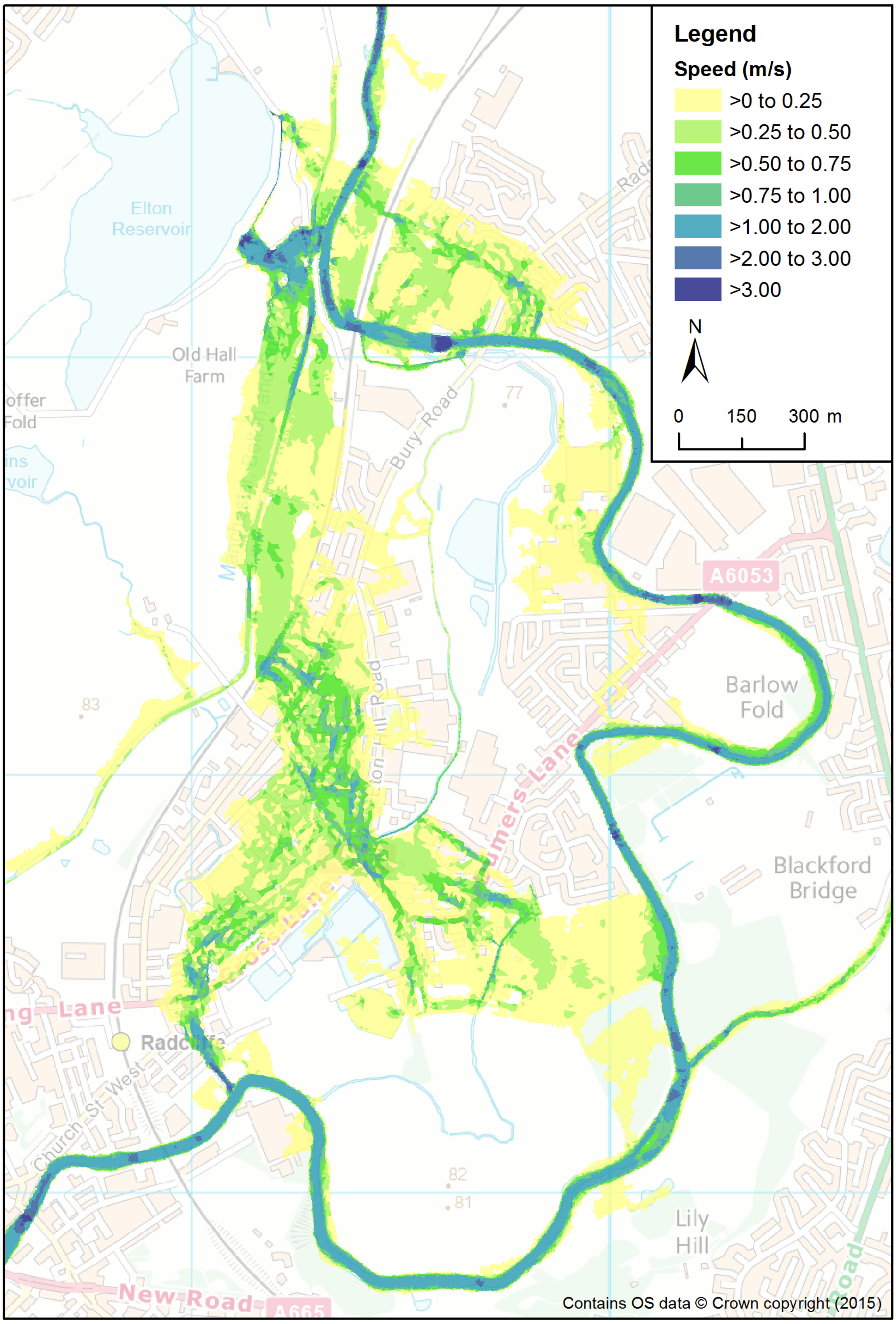

Figure 3.5: Maximum flow speed with a breach at the outlet pipes in the PMF

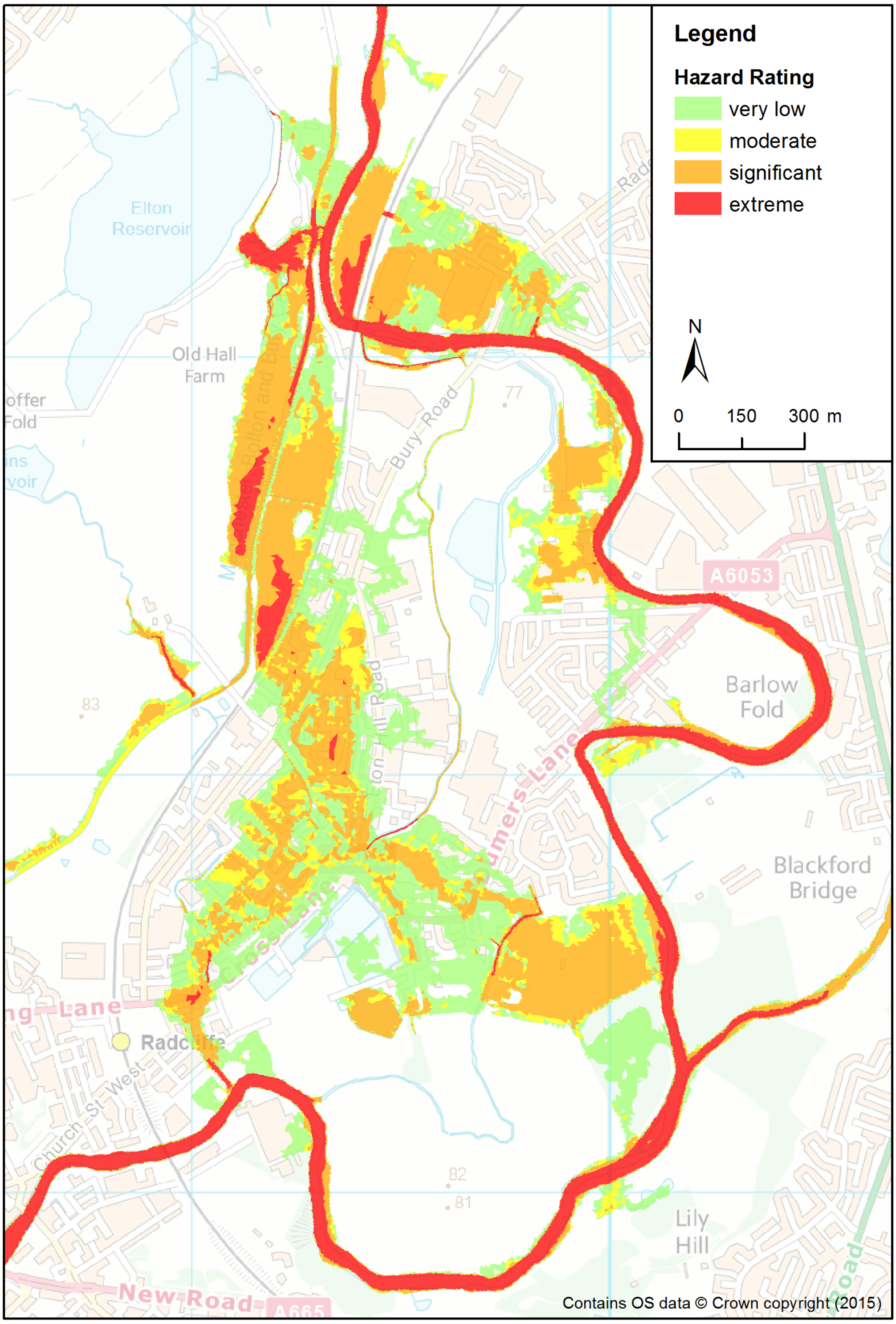

Figure 3.6: Maximum hazard rating with a breach at the outlet pipes in the PMF

Sensitivity to breach flows

The breach model was run with kd 25% and 50% higher than the best estimate. The outflows were run in the inundation model and the resulting hazard maps are shown in Figure 3.7 and Figure 3.8. These sensitivity runs show that there is very little impact on the areas of significant and extreme hazard classification if the breach model is run with values of the erodibility coefficient (kd) 25% and 50% higher than the best estimate. There is slightly greater extent of the low hazard across Bury Road at the northern most spill across the railway, and there is also greater extent from the River Irwell north of the A6053.

The time to first inundation of the urban area of North Radcliffe is 2 hours from the initiation of the breach with the erodibility coefficient 25% higher and 1 hour 50 minutes with the erodibility coefficient 25% higher than the best estimate of the erodibility coefficient. These are 10 and 20 minutes faster than with the best estimate.

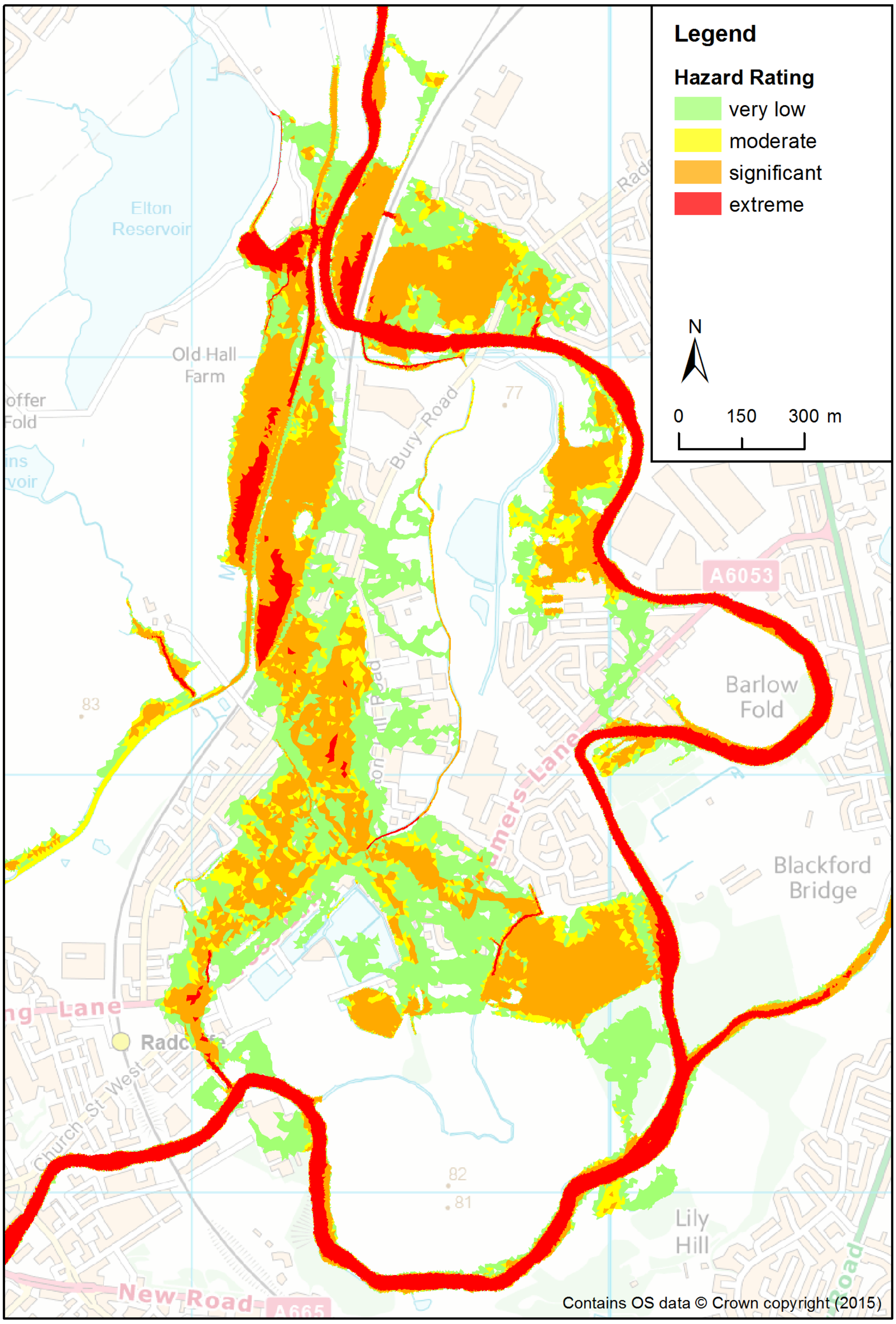

Figure 3.7: Maximum hazard rating with a breach at the outlet pipes in the PMF with kd increased by 25%

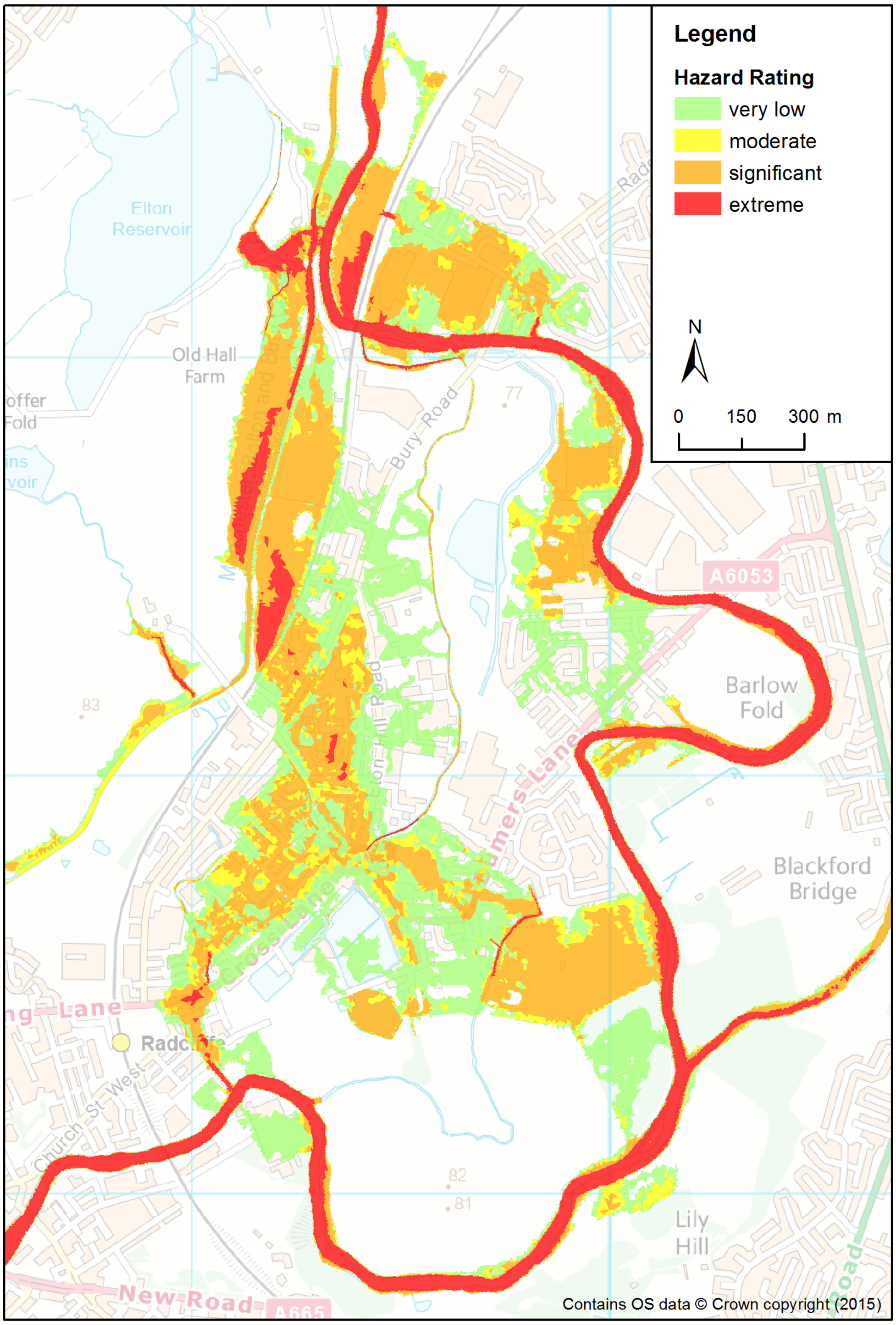

Figure 3.8: Maximum hazard rating with a breach at the outlet pipes in the PMF with kd increased by 50%

3.4.2. Breach at location 3 with the 1 in 10,000 year return period inflow to the reservoir

The maximum water depth, flow speed and hazard rating are shown in Figure 3.9 to Figure 3.11. These results show a similar pattern in the areas with highest flood hazard as with the PMF. The results show highest flood depth, velocity and hazard immediately downstream of the breach. There is also high flood depth on the fields either side of the canal, which lead to significant and extreme hazard classification. There are slightly smaller extents with extreme hazard rating than with the PMF inflow to the reservoir.

The time to first inundation of the urban area of North Radcliffe is 2 hours 15 minutes from the initiation of the breach. This 5 minutes slower than with the PMF inflow to the reservoir.

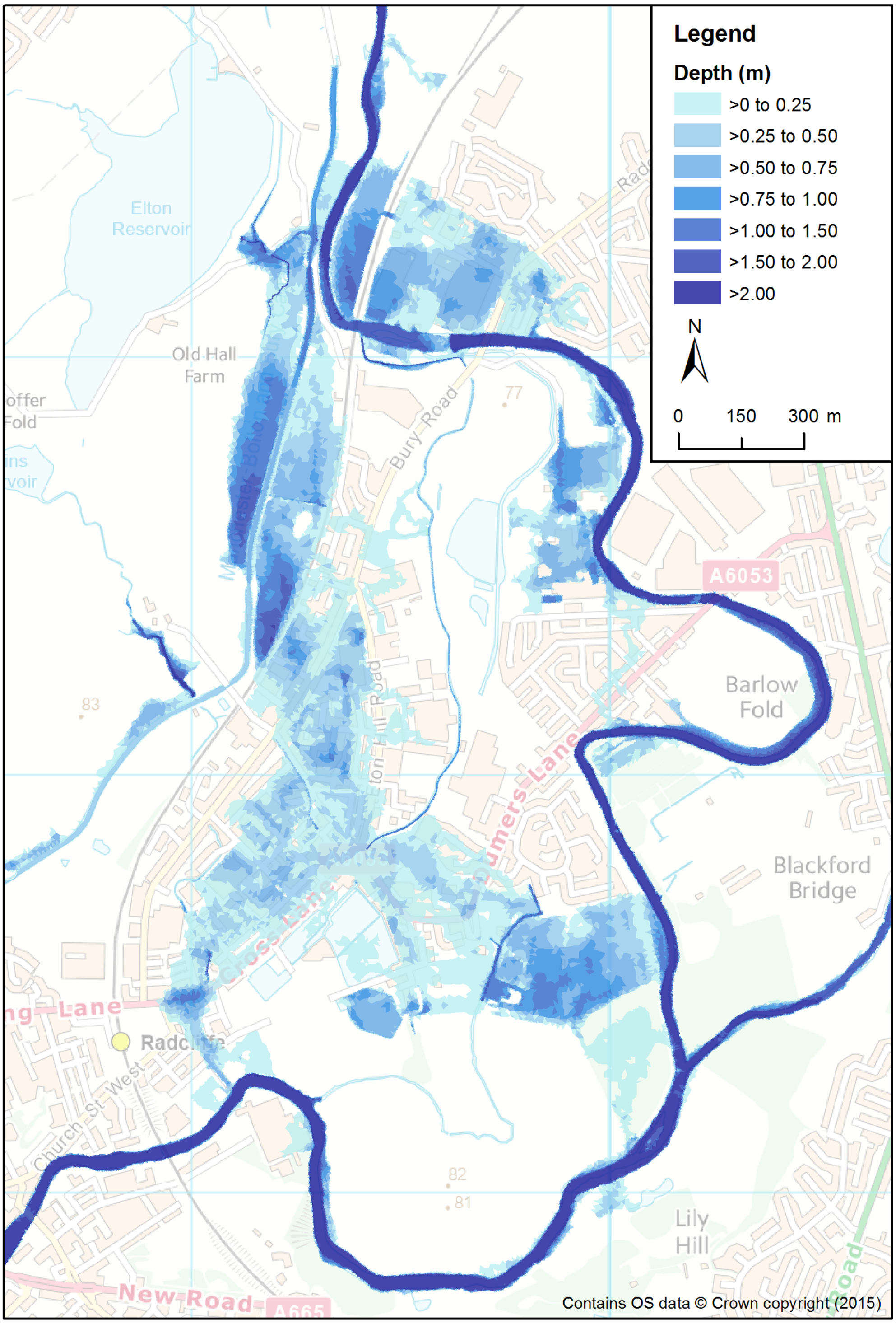

Figure 3.9: Maximum water depths with a breach at the outlet pipes in the 1 in 10,000 year inflow

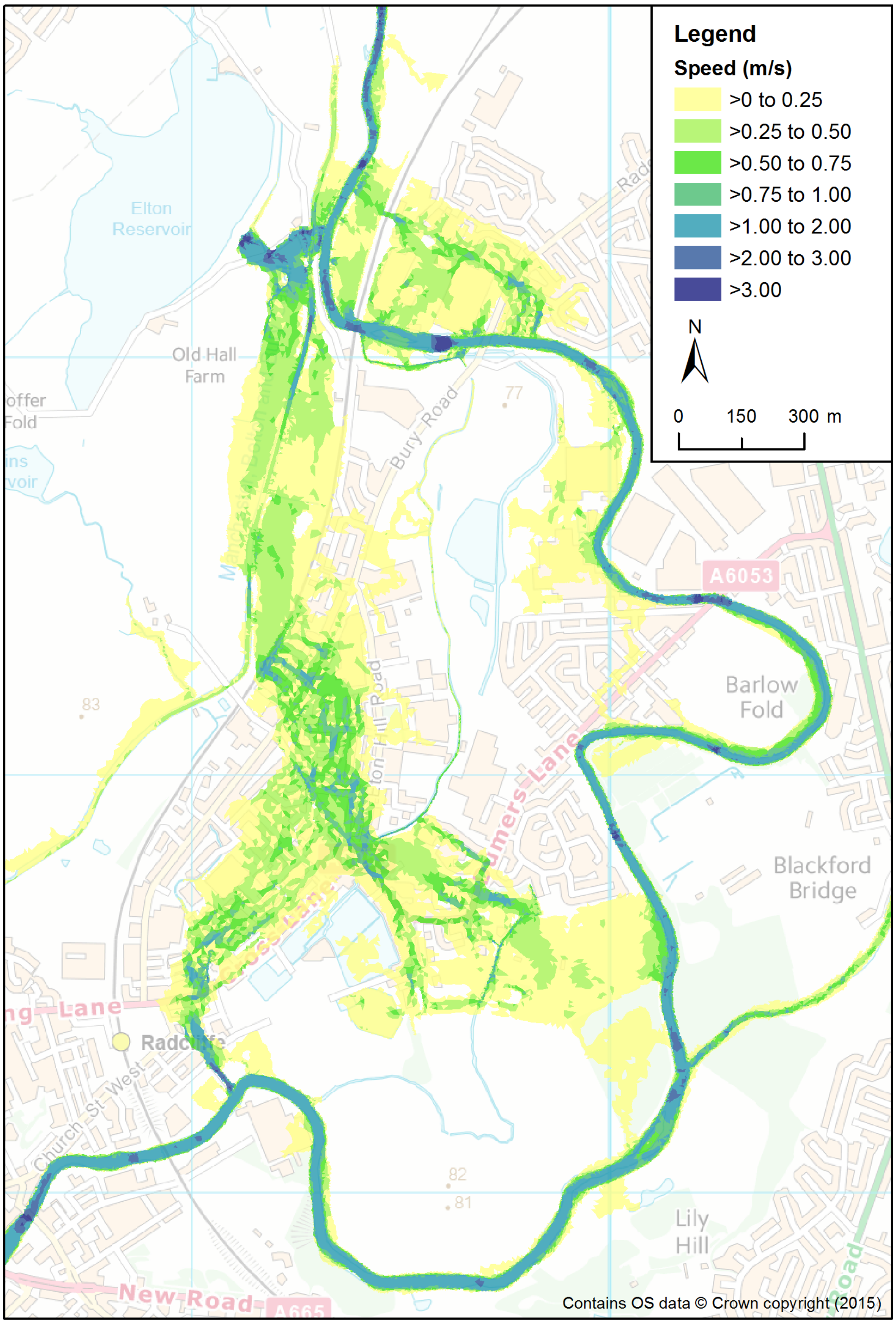

Figure 3.10: Maximum flow speed with a breach at the outlet pipes in the 1 in 10,000 year inflow

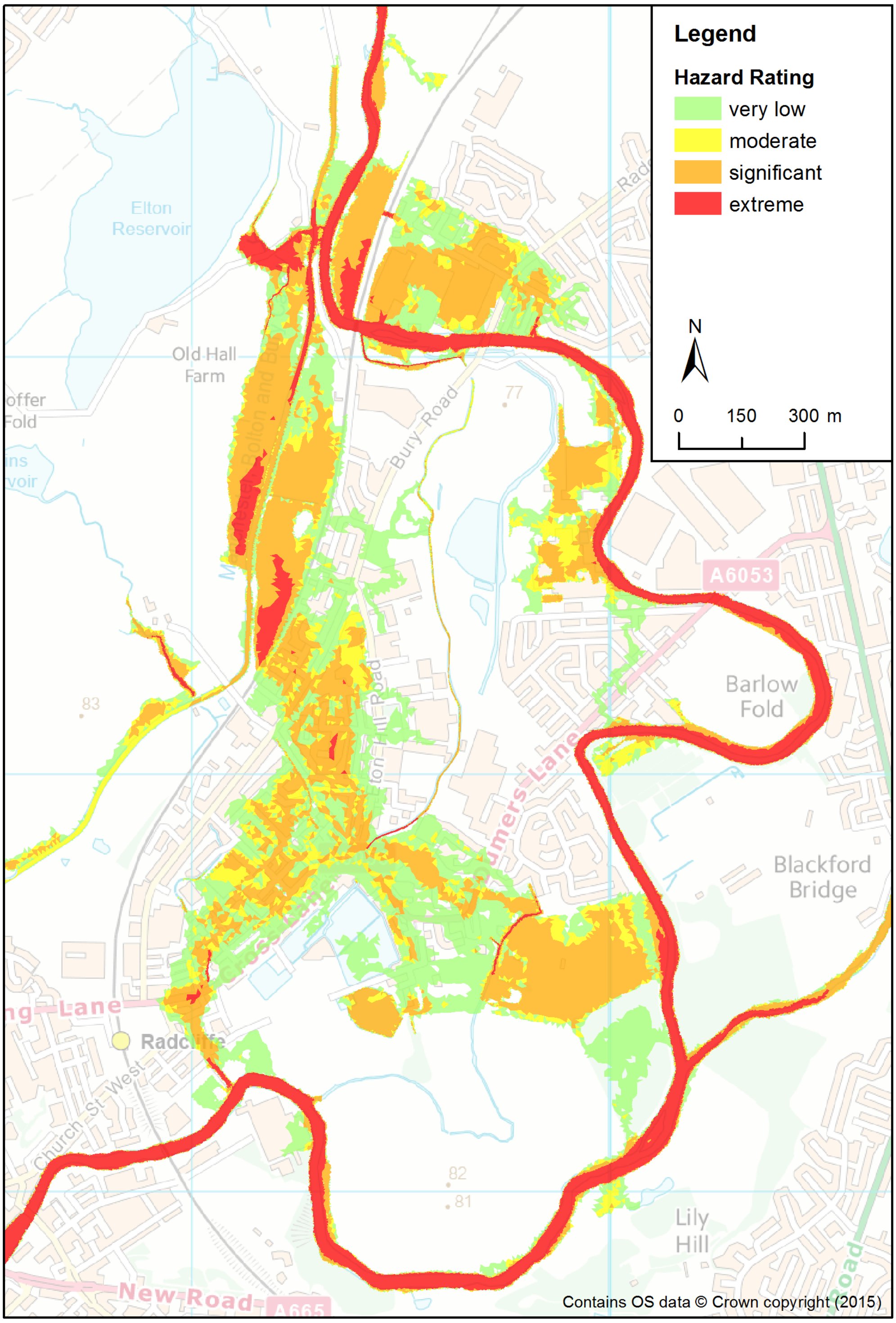

Figure 3.11: Maximum hazard rating with a breach at the outlet pipes in the 1 in 10,000 year inflow

3.4.3. Breach at location 3 with the 1 in 1,000 year return period inflow to the reservoir

The maximum water depth, flow speed and hazard rating are shown in Figure 3.12 to Figure 3.14. These results show a similar pattern in the areas with highest flood hazard as with the PMF or 1 in 10,000 year inflows. The flood extents and hazard rating are similar to with the 1 in 10,000 year inflow.

The time to first inundation of the urban area of North Radcliffe is 2 hours 18 minutes from the initiation of the breach. This 8 minutes slower than with the PMF inflow to the reservoir.

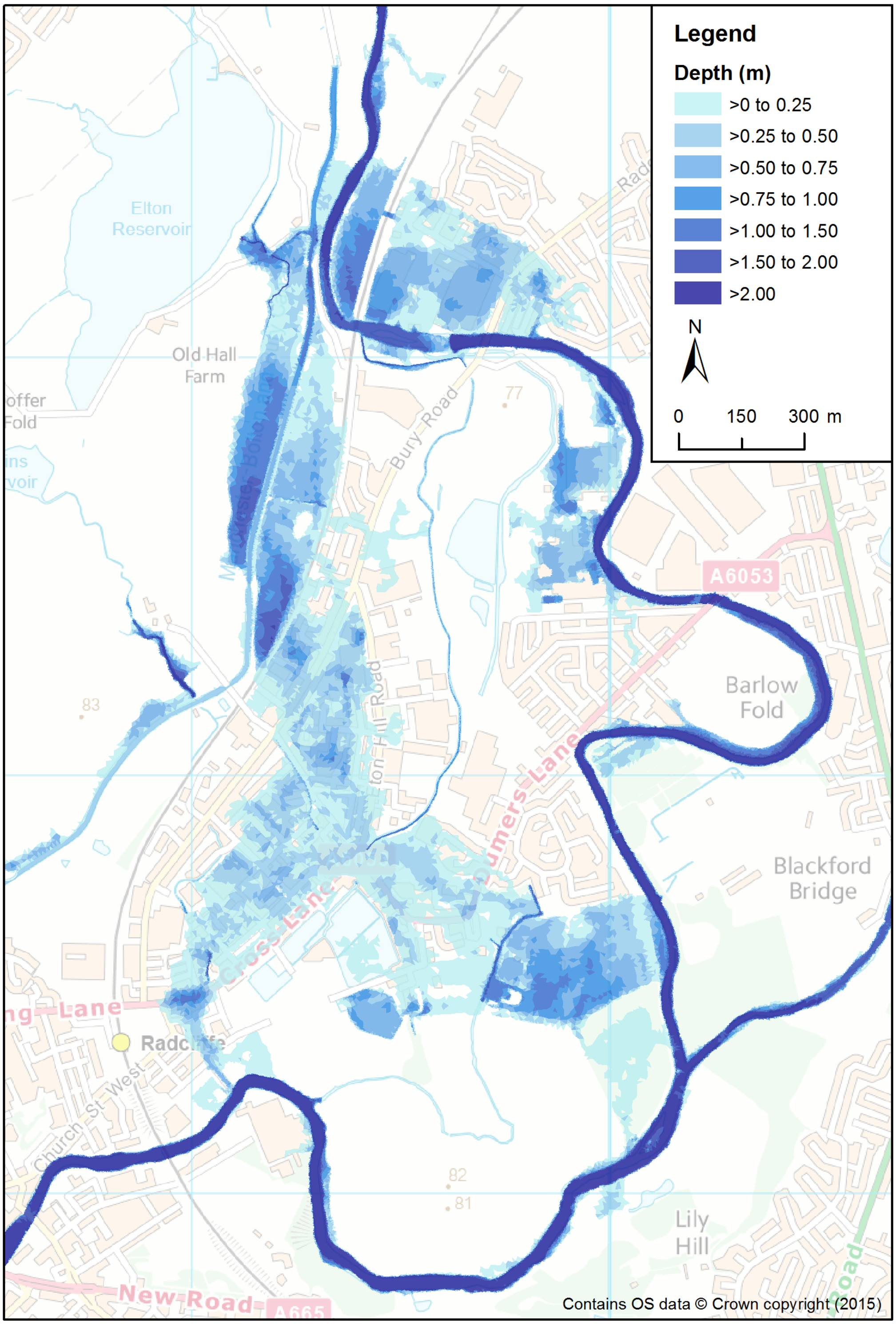

Figure 3.12: Maximum water depth with a breach at the outlet pipes in the 1 in 1,000 year inflow

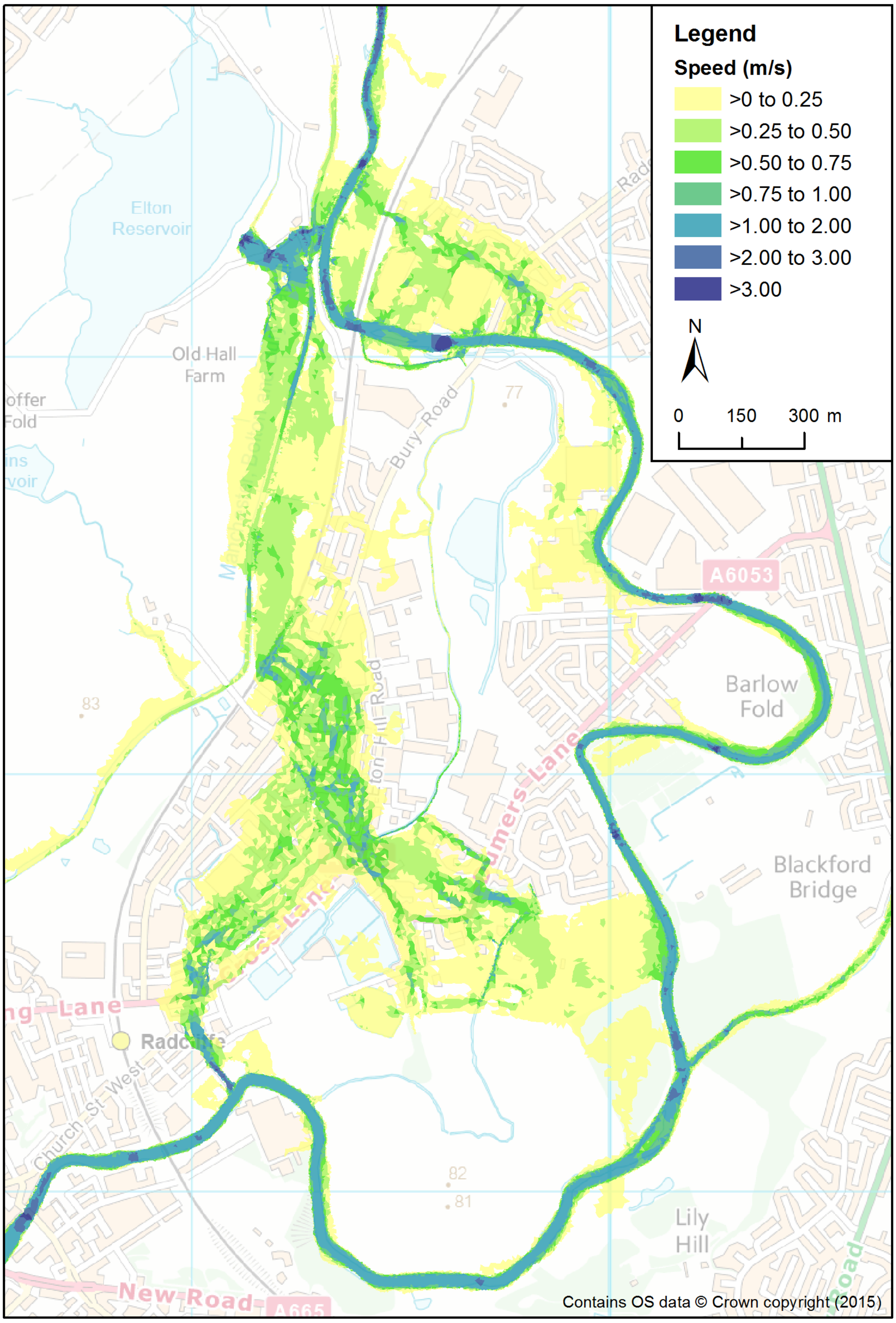

Figure 3.13: Maximum flow speed with a breach at the outlet pipes in the 1 in 1,000 year inflow

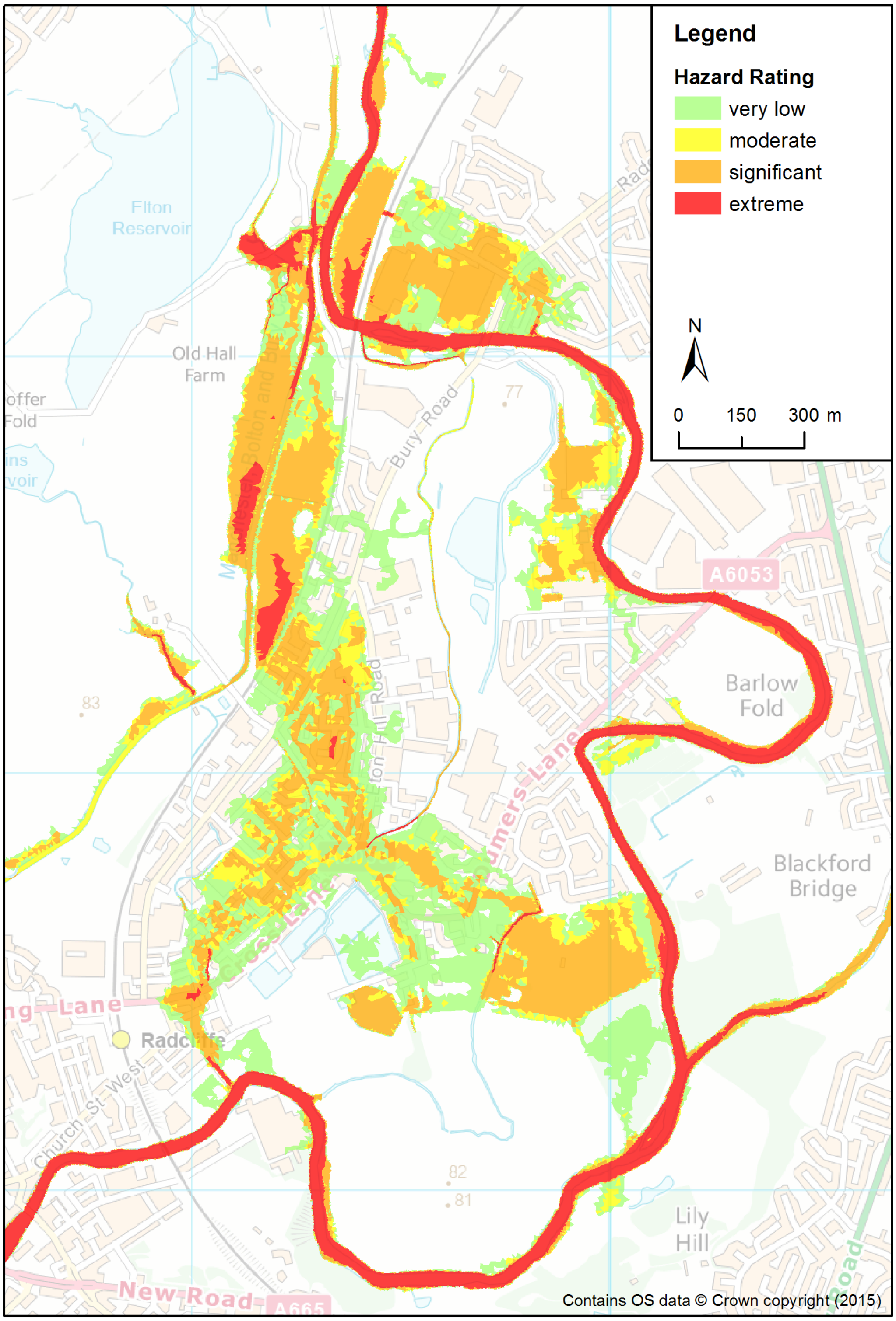

Figure 3.14: Maximum hazard rating with a breach at the outlet pipes in the 1 in 1,000 year inflow

3.4.4. Breach at location 2 with the PMF inflow to the reservoir

The maximum water depth, flow speed and hazard rating are shown in Figure 3.15 to Figure 3.17. The results shows that with the breach at the location of the historical slip there is a smaller flood extent than with the breach at the location of the outlet pipes. In the urban area north of Radcliffe the flood depths and flow velocity are also lower which means that the flood hazard rating is reduced from significant to very low. The time to first inundation of the urban area of North Radcliffe is 3 hours 20 minutes from the initiation of the breach at location 2 (the site of the previous slip). This is 1 hour later than for the breach at location 3.

The fields adjacent to the canal have significant hazard with smaller areas of extreme hazard, and the industrial area between the River Irwell and the east side of the railway also has significant hazard rating.

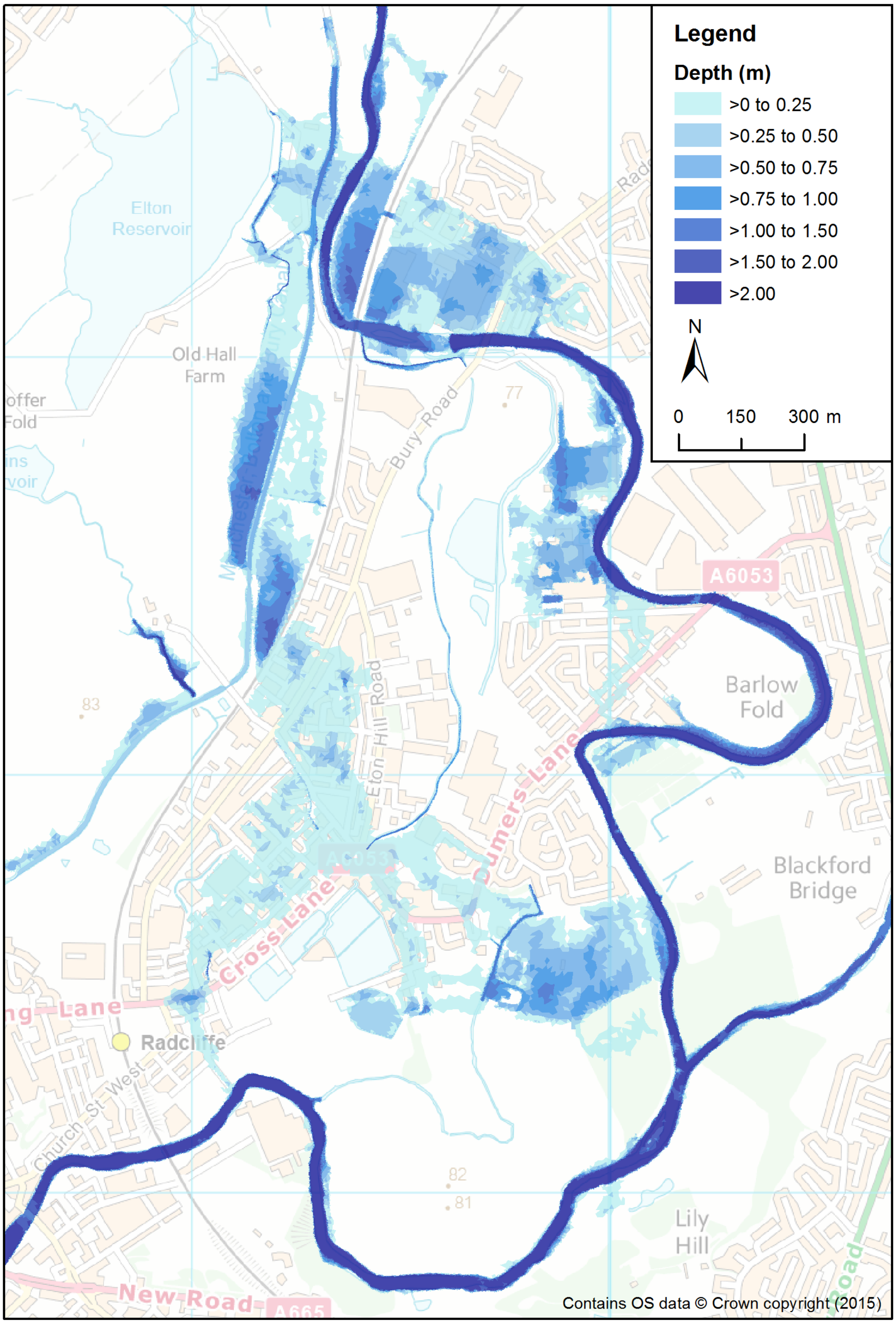

Figure 3.15: Maximum water depth with a breach at location 2 with the PMF inflow

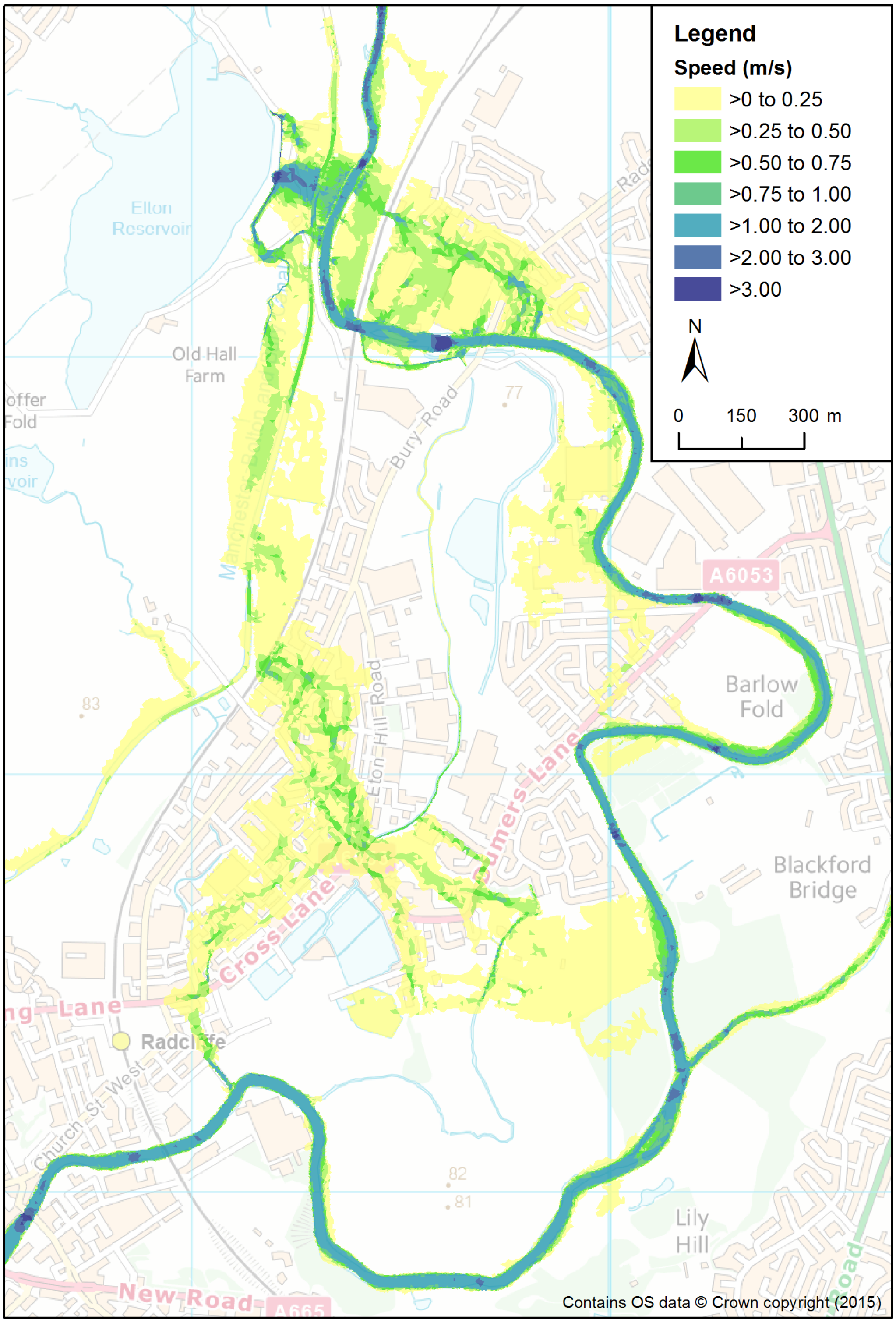

Figure 3.16: Maximum flow speed with a breach at location 2 with the PMF inflow

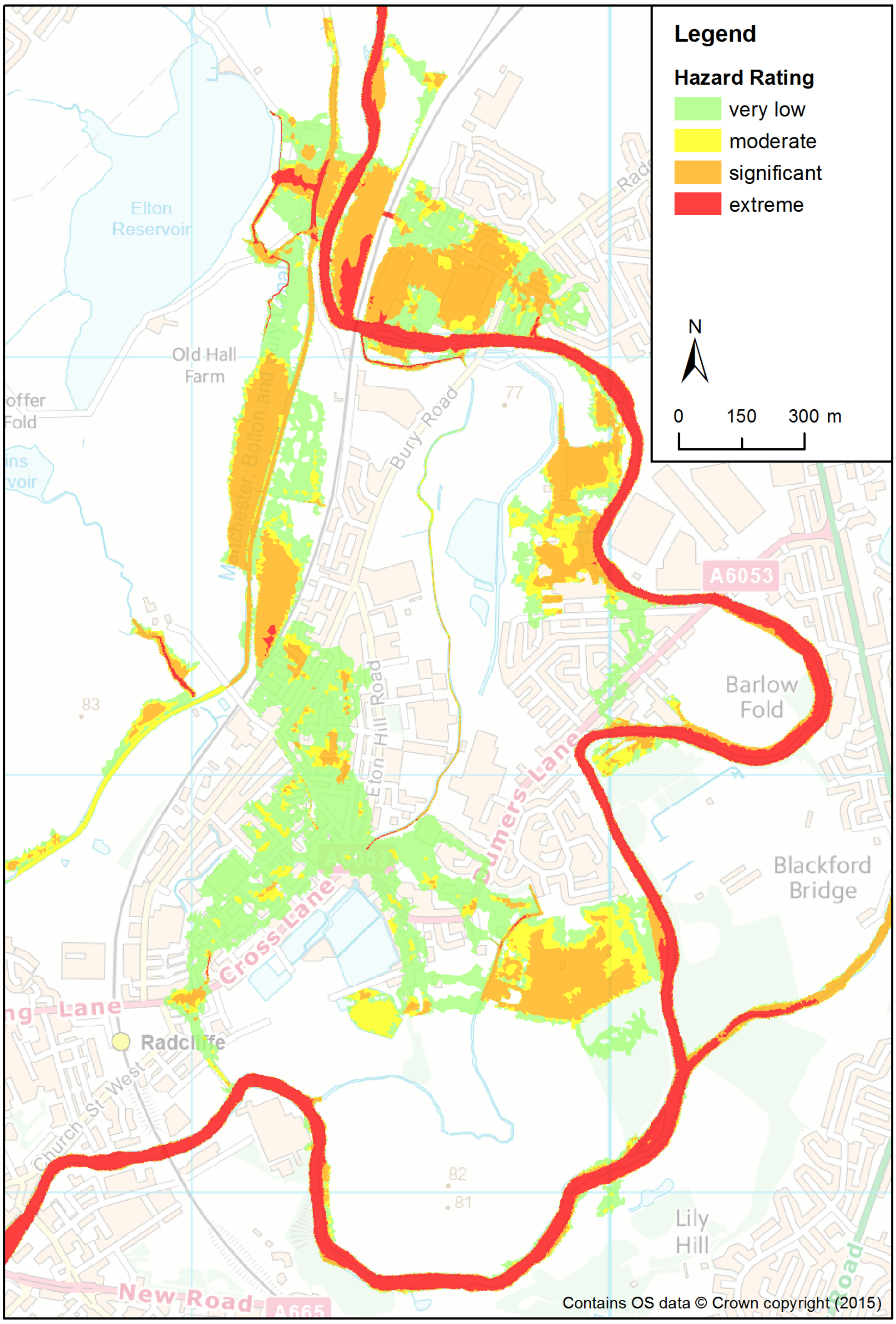

Figure 3.17: Maximum hazard rating with a breach at location 2 with the PMF inflow

3.4.5. Breach at location 2 with the 1 in 10,000 year return period inflow to the reservoir

The maximum water depth, flow speed and hazard rating are shown in Figure 3.18 to Figure 3.20. These results show a similar pattern in the areas with highest flood hazard as with the PMF. The time to first inundation of the urban area of North Radcliffe is 3 hours 25 minutes from the initiation of the breach. This 5 minutes slower than with the PMF inflow to the reservoir.

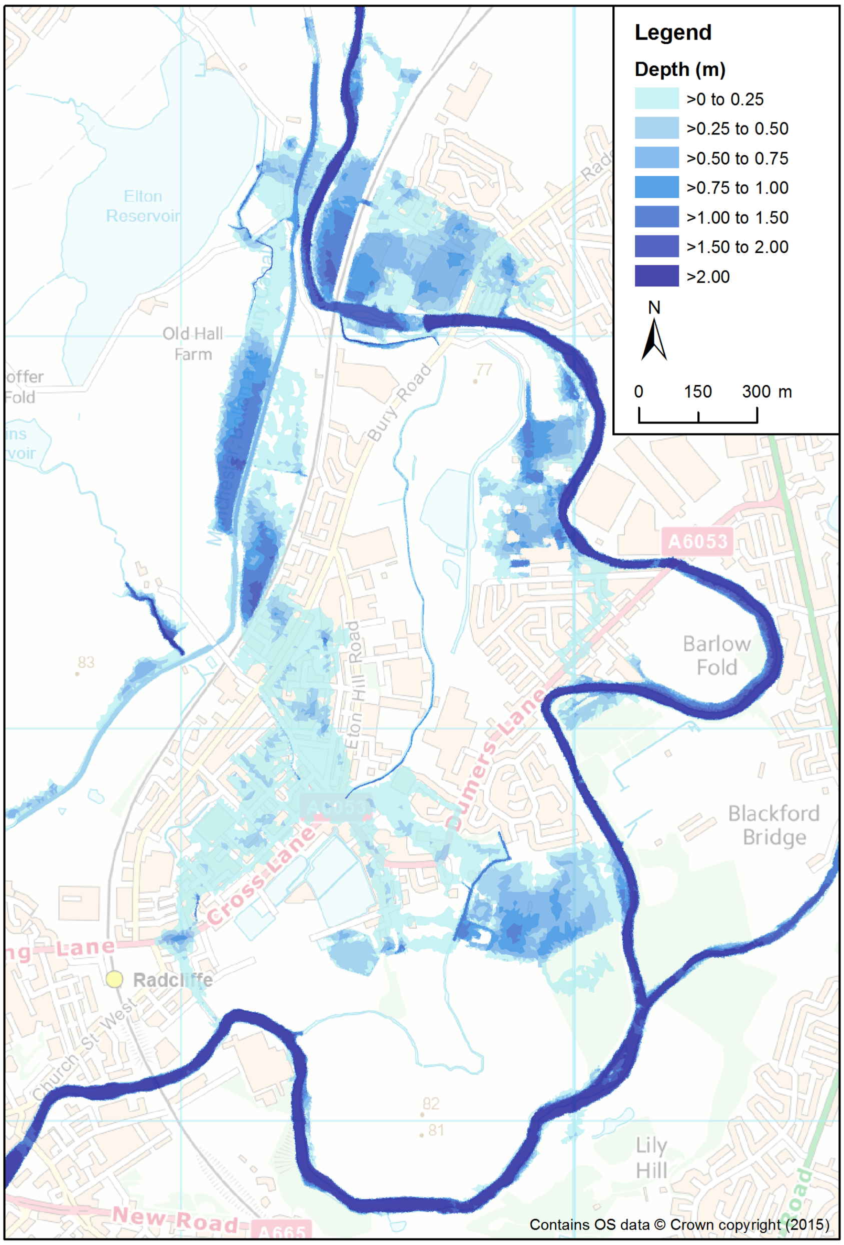

Figure 3.18: Maximum water depth with a breach at location 2 with the 1 in 10,000 year inflow

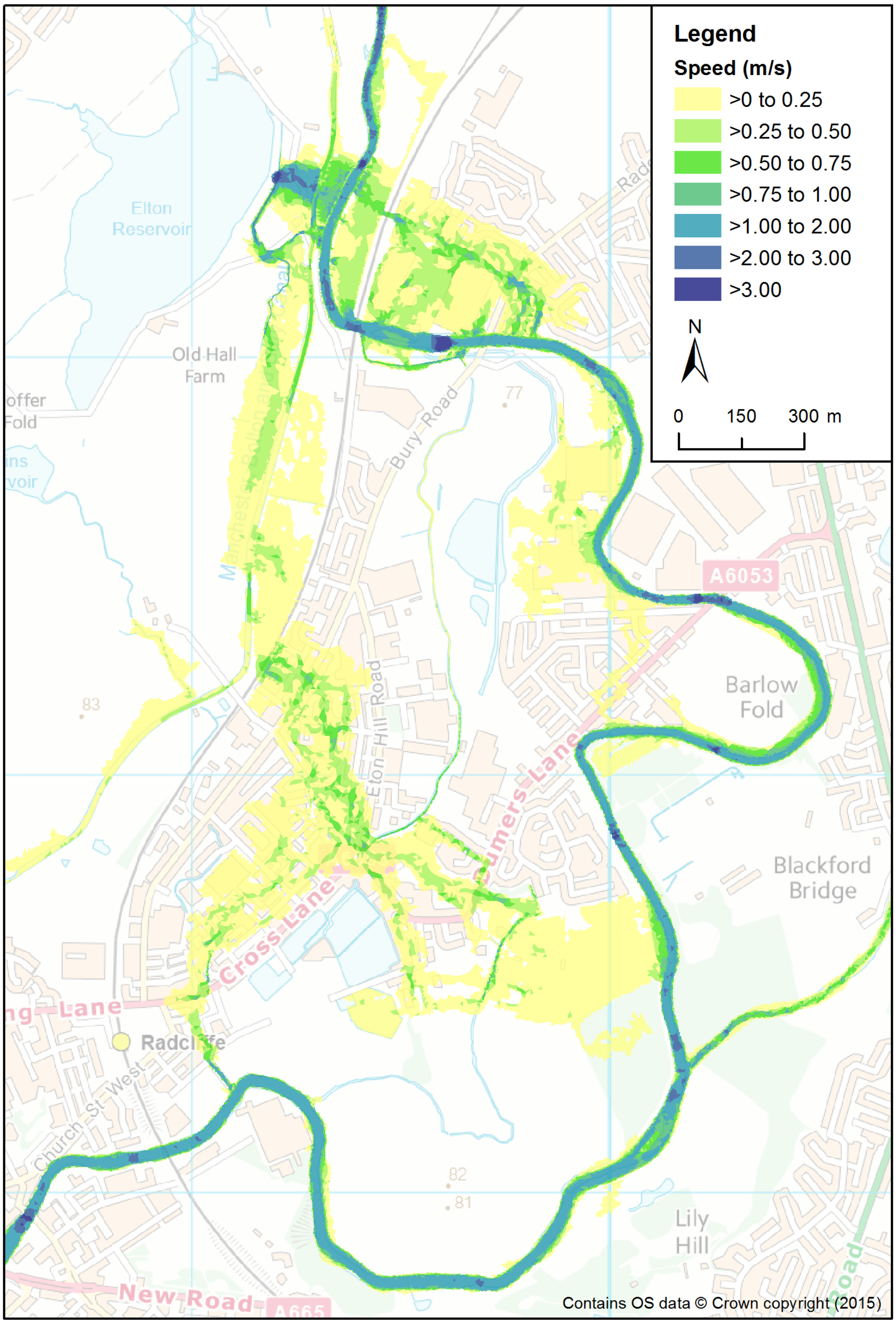

Figure 3.19: Maximum flow speed with a breach at location 2 with the 1 in 10,000 year inflow

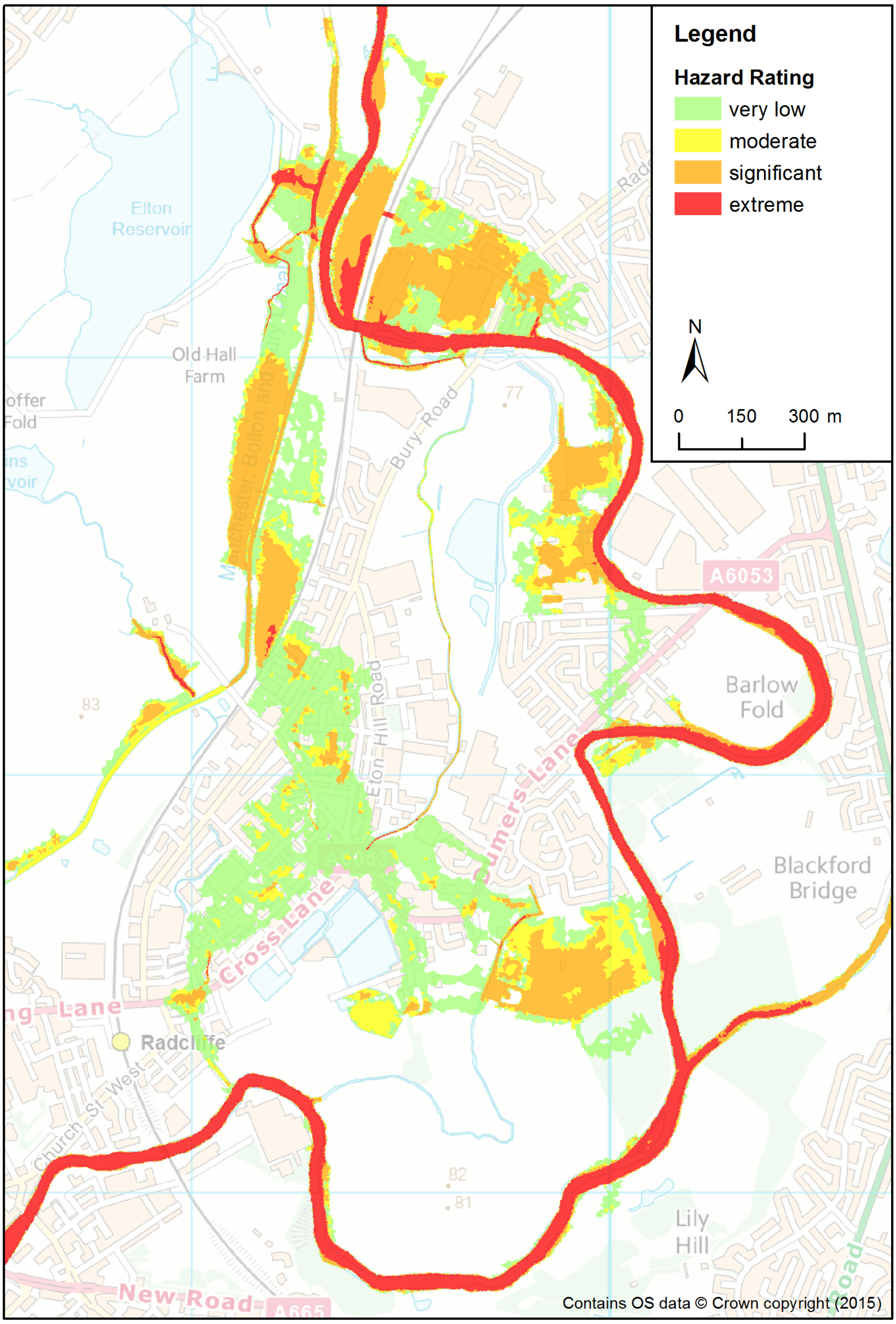

Figure 3.20: Maximum hazard rating with a breach at location 2 with the 1 in 10,000 year inflow

3.4.6. Breach at location 2 with the 1 in 1,000 year return period inflow to the reservoir

The maximum water depth, flow speed and hazard rating are shown in Figure 3.21 to Figure 3.23. These results show a similar pattern in the areas with highest flood hazard as with the PMF. The time to first inundation of the urban area of North Radcliffe is 3 hours 33 minutes from the initiation of the breach. This 13 minutes slower than with the PMF inflow to the reservoir.

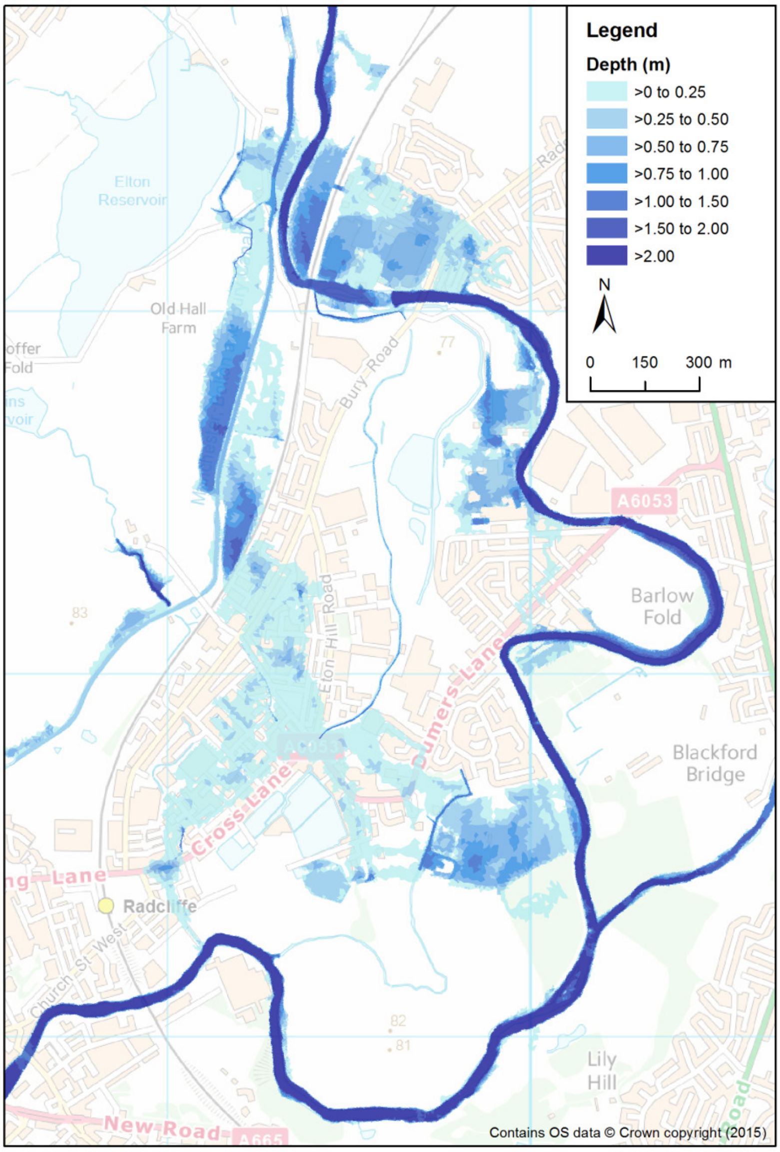

Figure 3.21: Maximum water depth with a breach at location 2 with the 1 in 1,000 year inflow

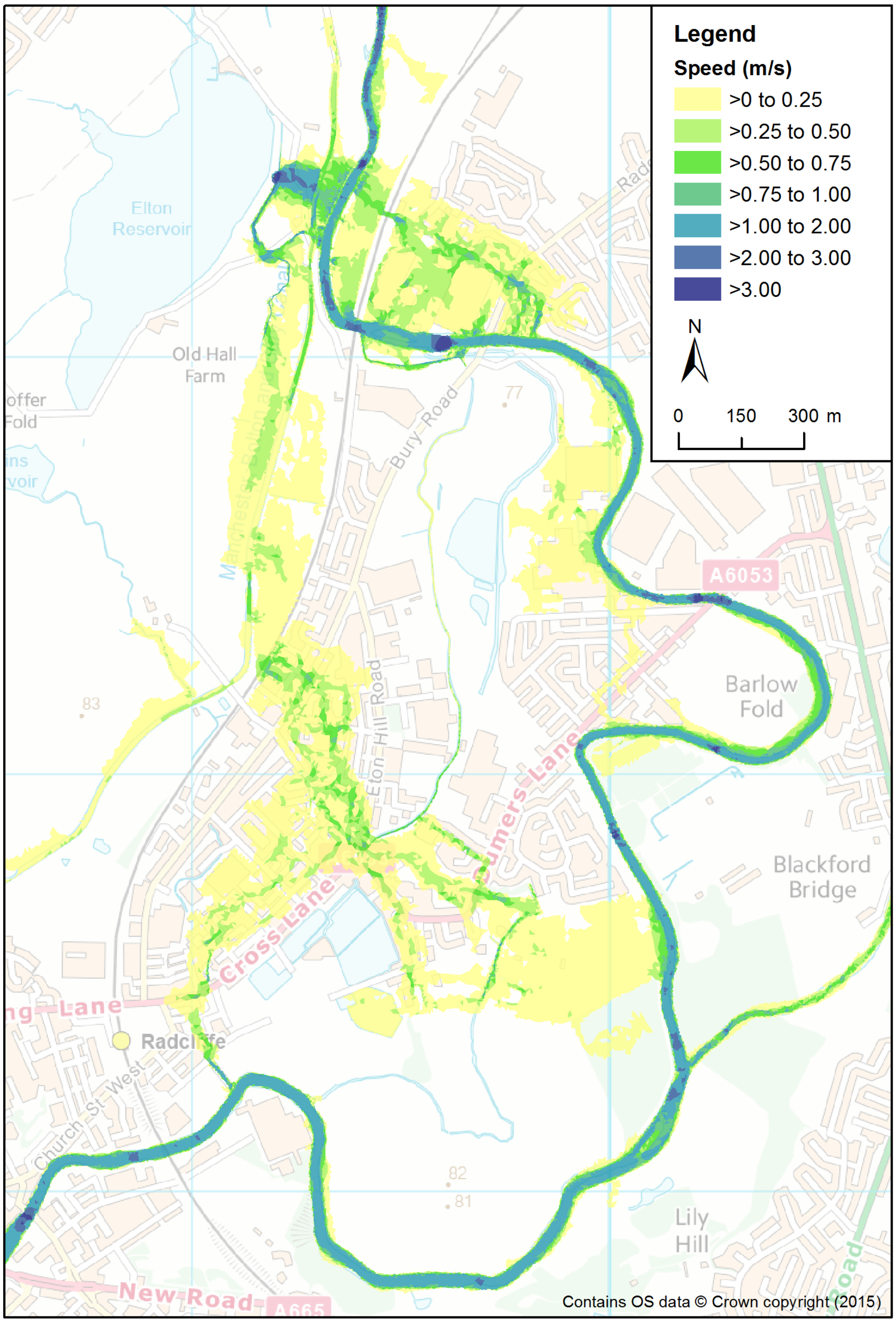

Figure 3.22: Maximum flow speed with a breach at location 2 with the 1 in 1,000 year inflow

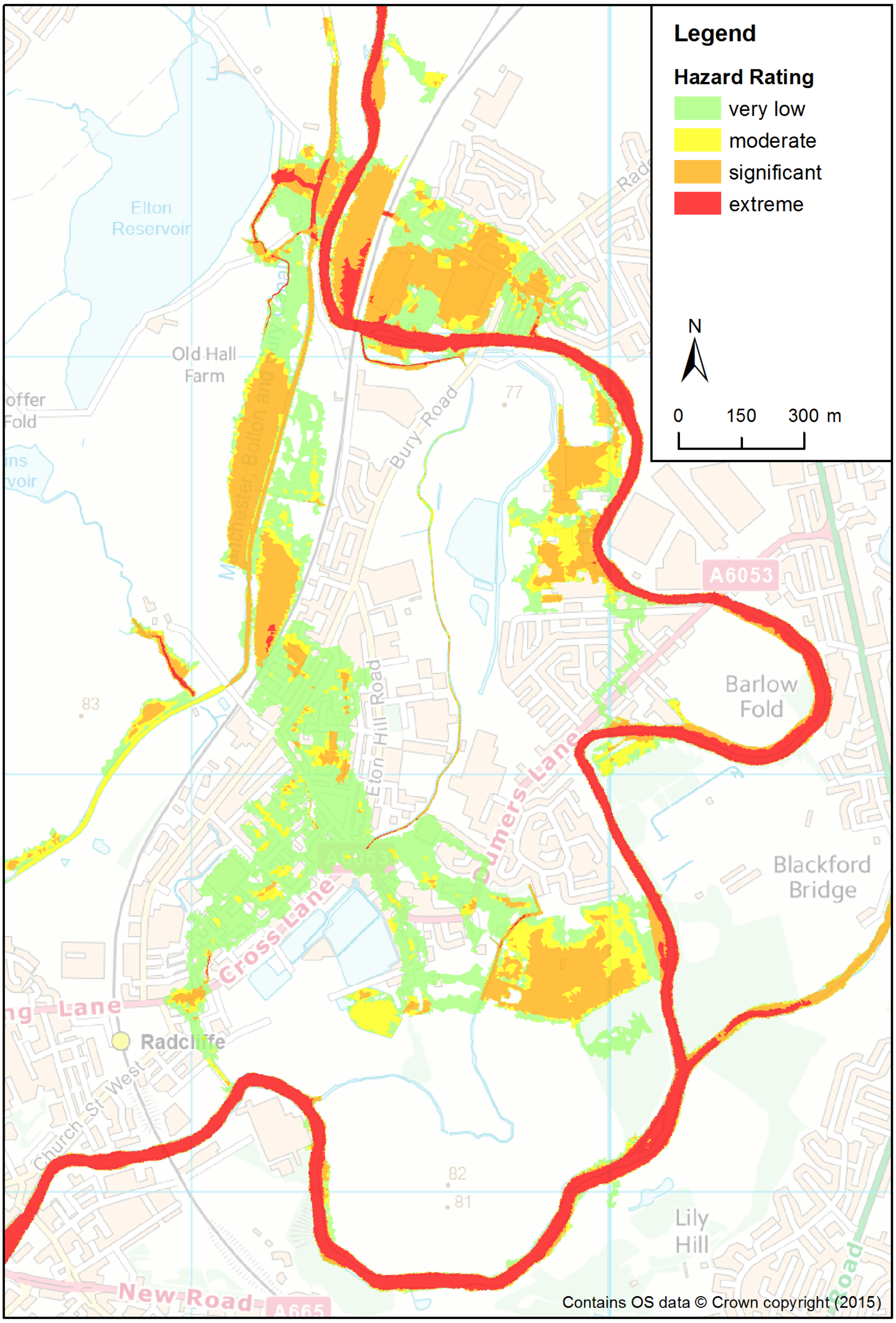

Figure 3.23: Maximum hazard rating with a breach at location 2 with the 1 in 1,000 year inflow

3.4.7. Breach at location 1 with the PMF inflow to the reservoir

The maximum water depth, flow speed and hazard rating are shown in Figure 3.24 to Figure 3.26. the results show that with the breach at the spillway the flow speeds are significantly reduced compared to with the other two breach locations (except immediately downstream of the dam). This leads to the majority of the urban areas at risk near North Radcliffe to be classified as very low hazard. The time to first inundation of the urban area of North Radcliffe is 4 hours 25 minutes from the initiation of the breach at location 1 (the spillway). This is 2 hours later than for the breach at location 3.

The areas upstream of the canal and between the canal and the railway are still classified as significant hazard because the flood depths are higher due to storage behind the embankments.

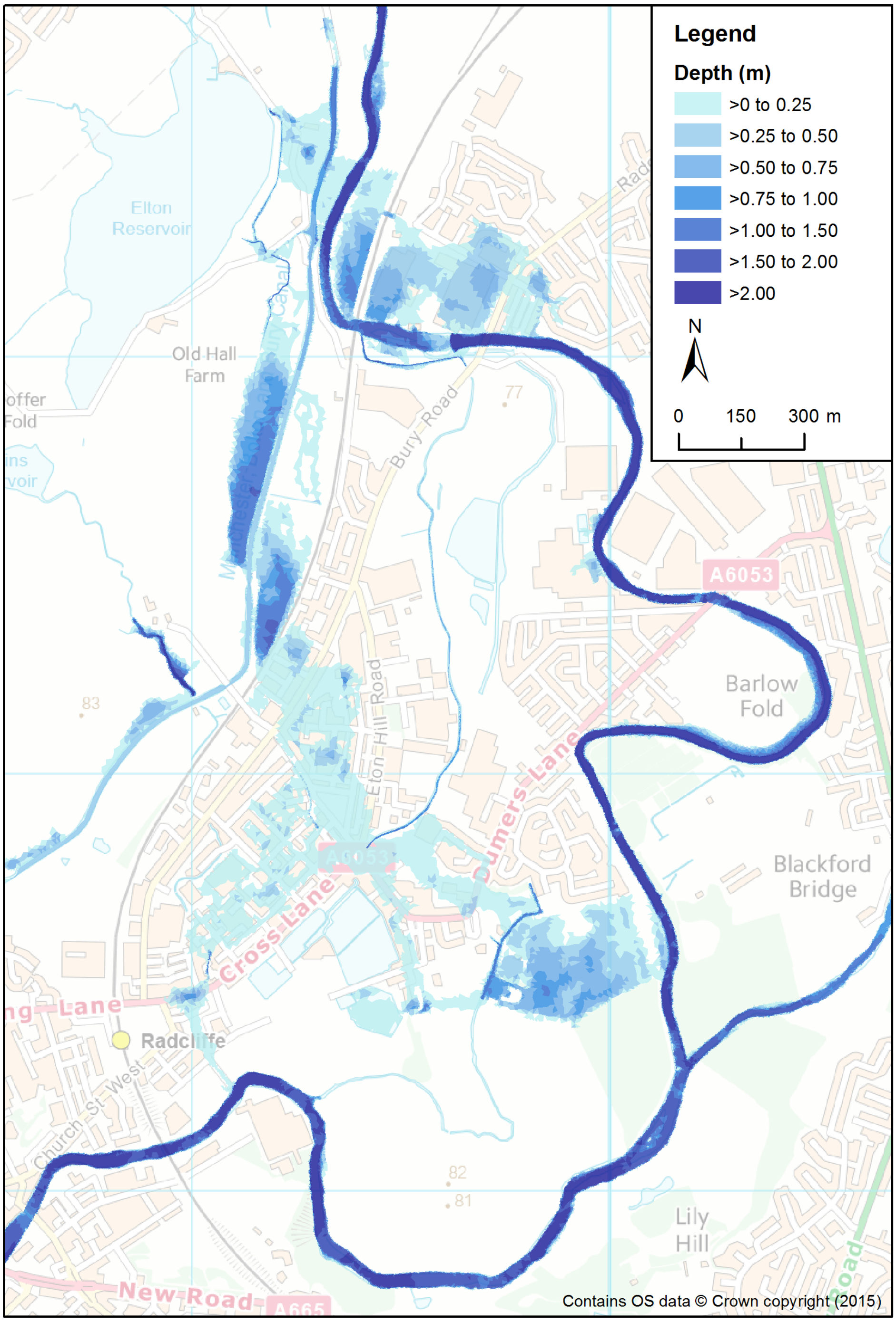

Figure 3.24: Maximum water depth with a breach at the spillway with the PMF inflow

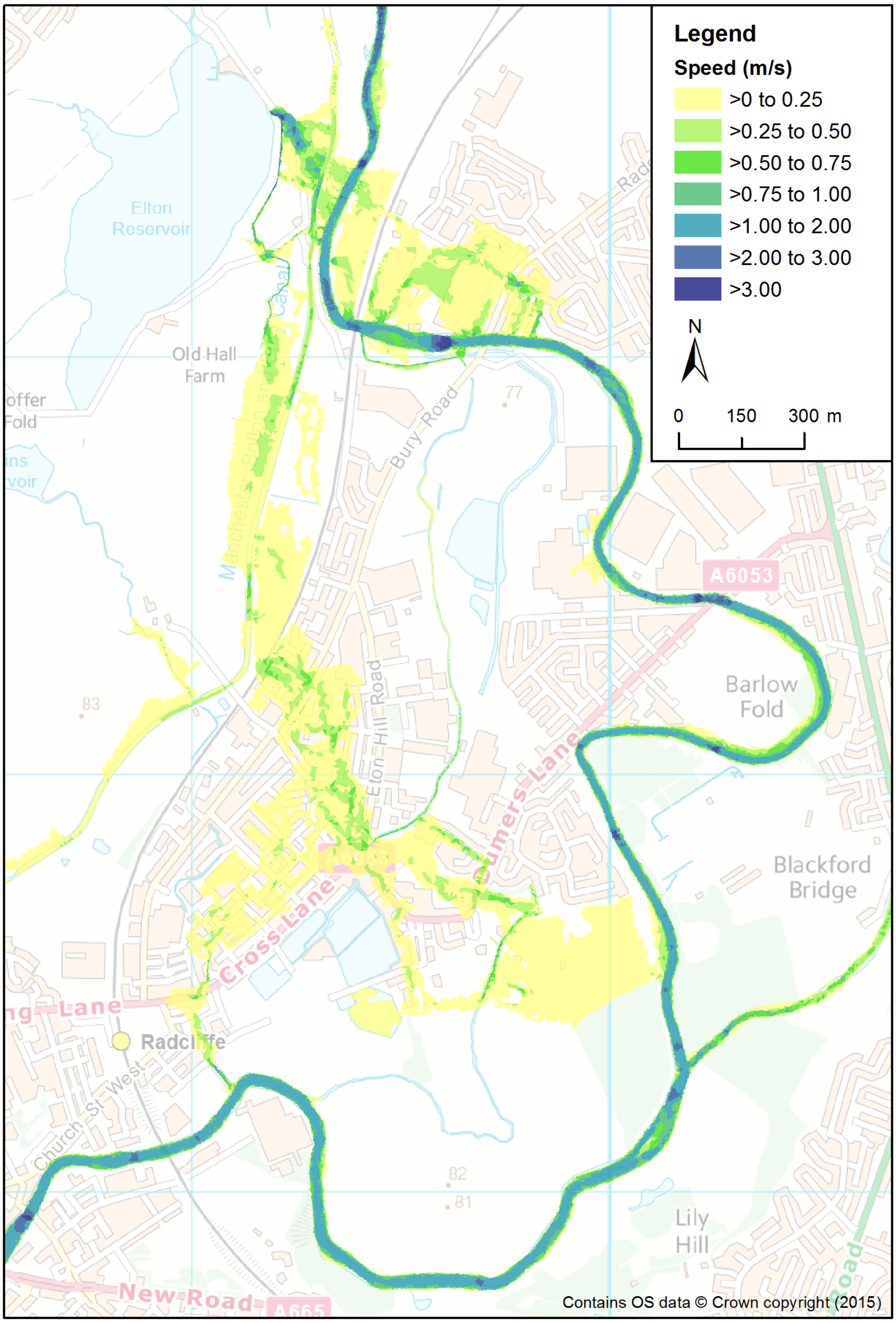

Figure 3.25: Maximum flow speed with a breach at the spillway with the PMF inflow

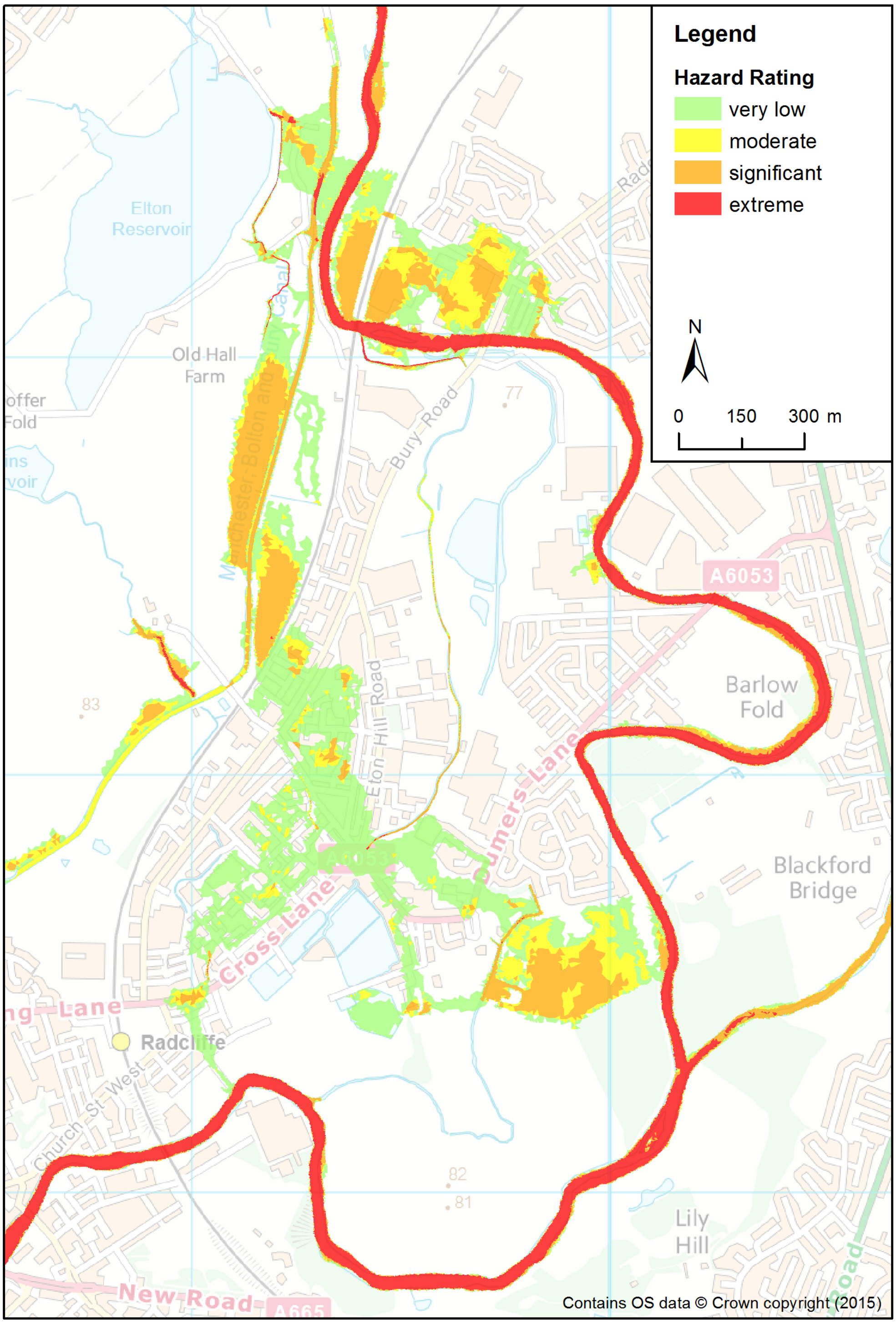

Figure 3.26: Maximum hazard rating with a breach at the spillway with the PMF inflow

3.4.8. Breach at location 1 with the 1 in 10,000 year return period inflow to the reservoir

The maximum water depth, flow speed and hazard rating are shown in Figure 3.27 to Figure 3.29. These results show a similar pattern in the areas with highest flood hazard as with the PMF. The time to first inundation of the urban area of North Radcliffe is 4 hours 50 minutes from the initiation of the breach. This 25 minutes slower than with the PMF inflow to the reservoir.

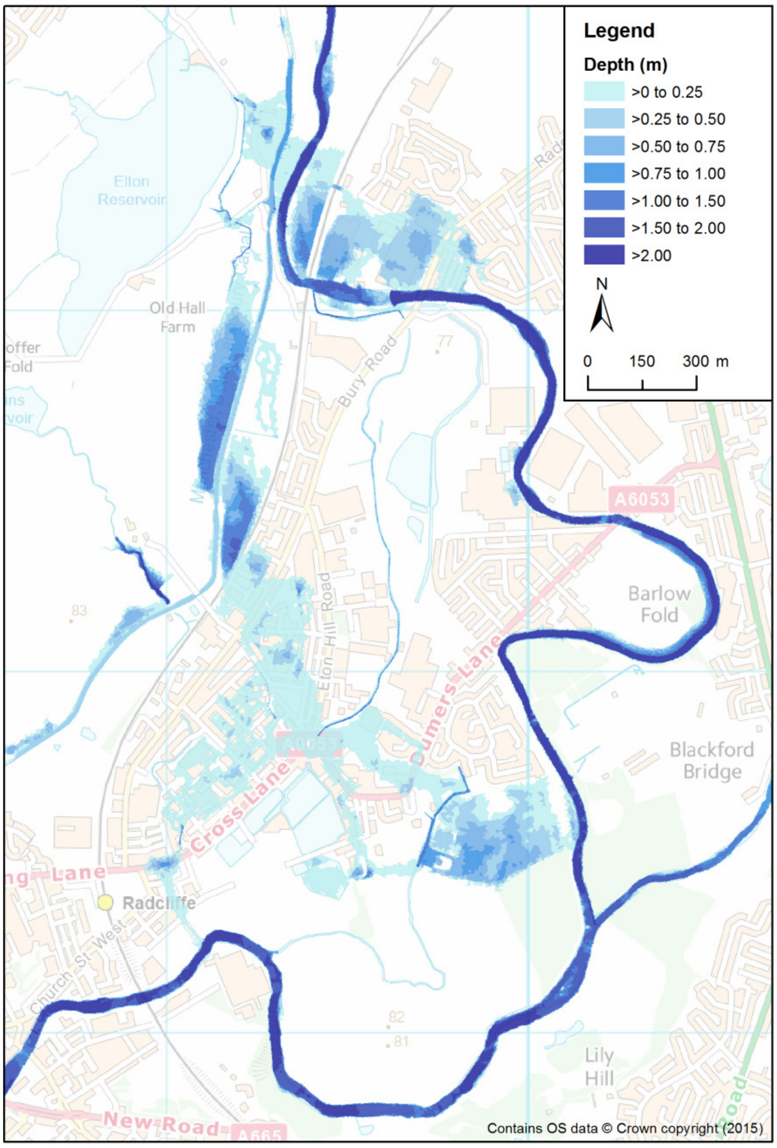

Figure 3.27: Maximum water depth with a breach at the spillway with the 1 in 10,000 year inflow

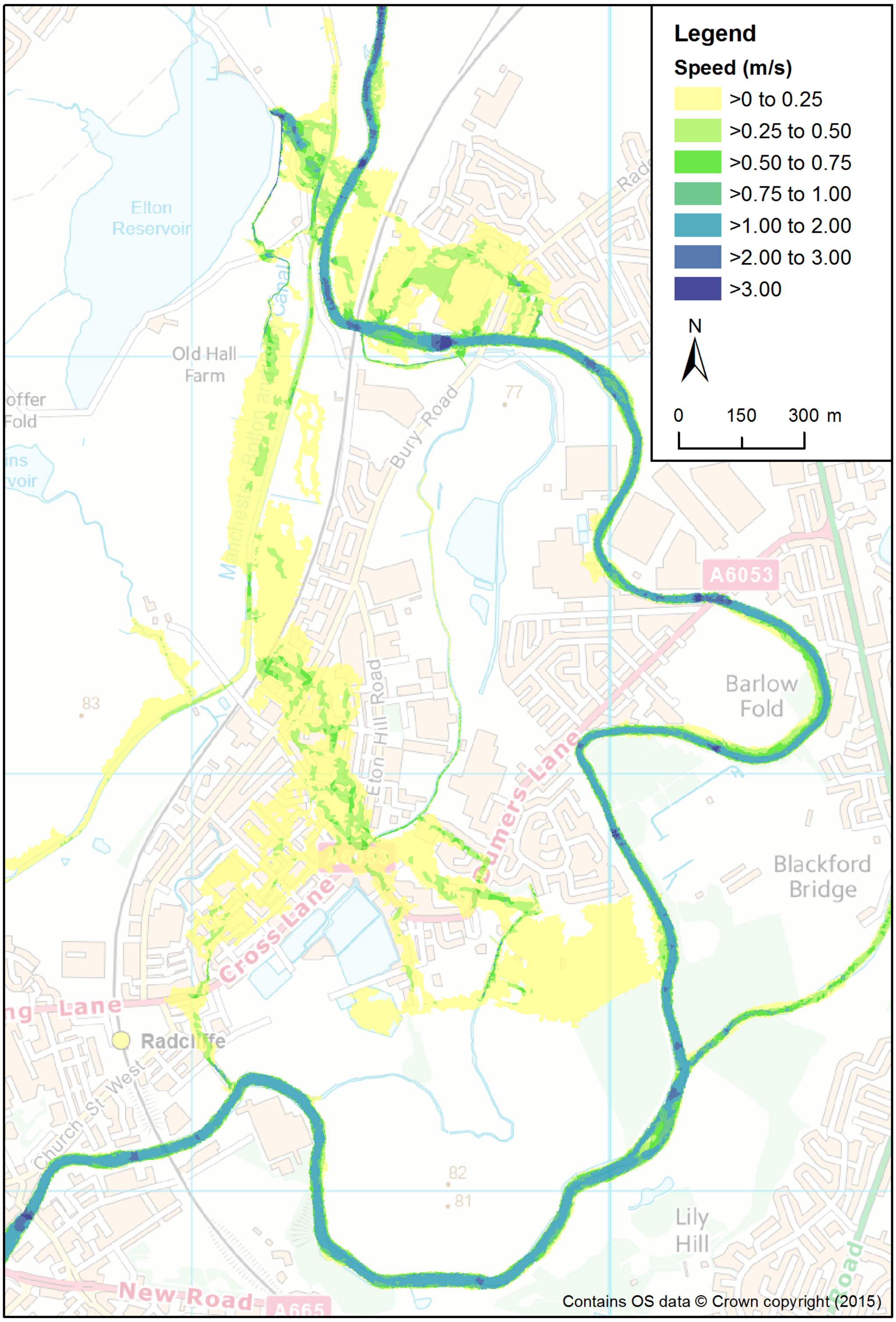

Figure 3.28: Maximum flow speed with a breach at the spillway with the 1 in 10,000 year inflow

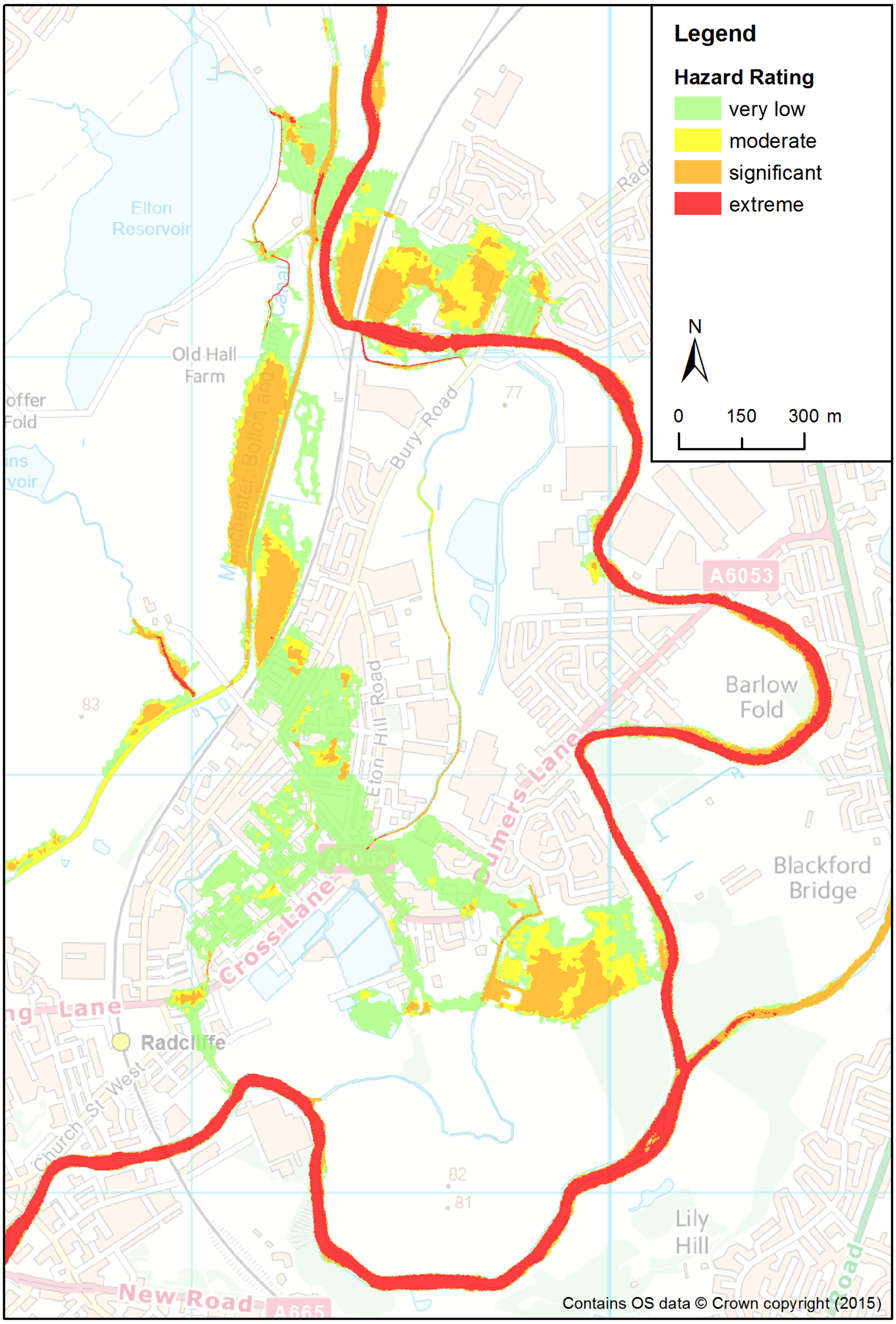

Figure 3.29: Maximum hazard rating with a breach at the spillway with the 1 in 10,000 year inflow

3.4.9. Breach at location 1 with the 1 in 1,000 year return period inflow to the reservoir

The maximum water depth, flow speed and hazard rating are shown in Figure 3.30 to Figure 3.32. These results show a similar pattern in the areas with highest flood hazard as with the PMF. The time to first inundation of the urban area of North Radcliffe is 5 hours 35 minutes from the initiation of the breach. This 1 hour and 10 minutes slower than with the PMF inflow to the reservoir.

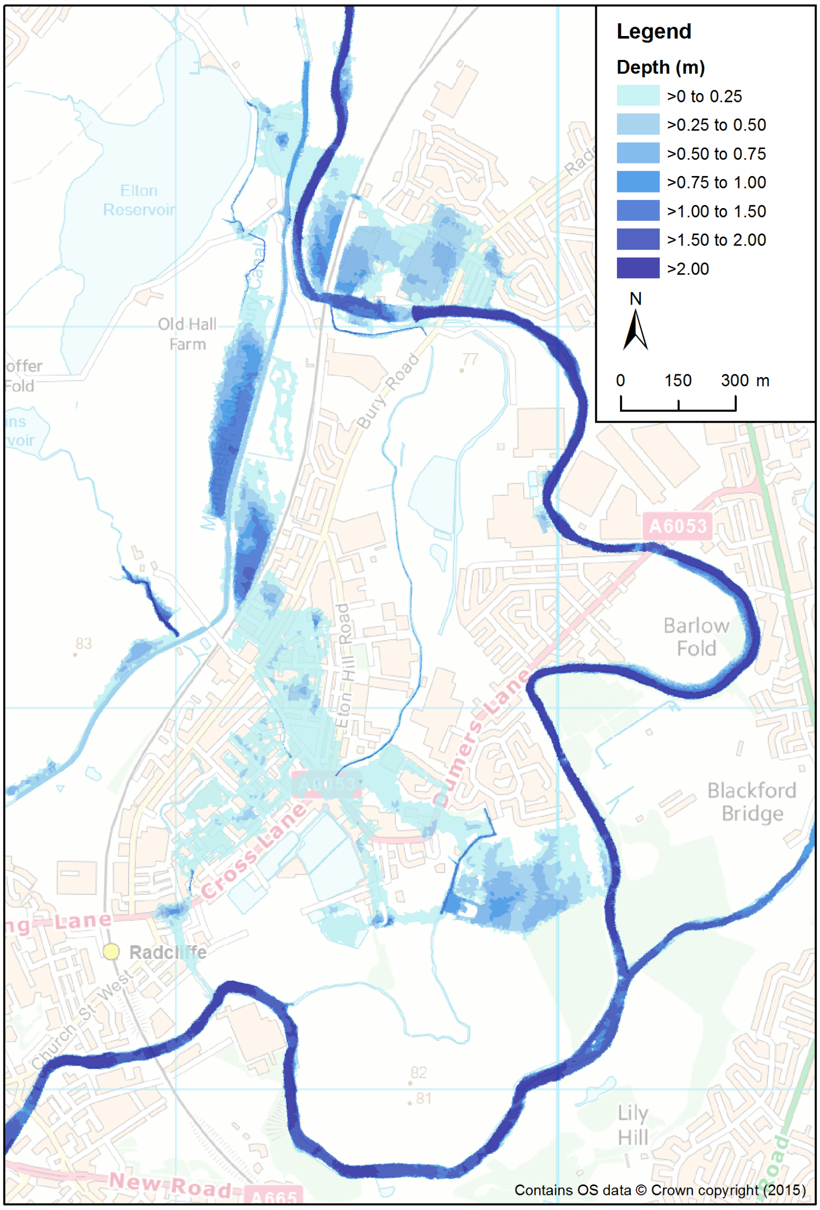

Figure 3.30: Maximum water depth with a breach at the spillway with the 1 in 1,000 year inflow

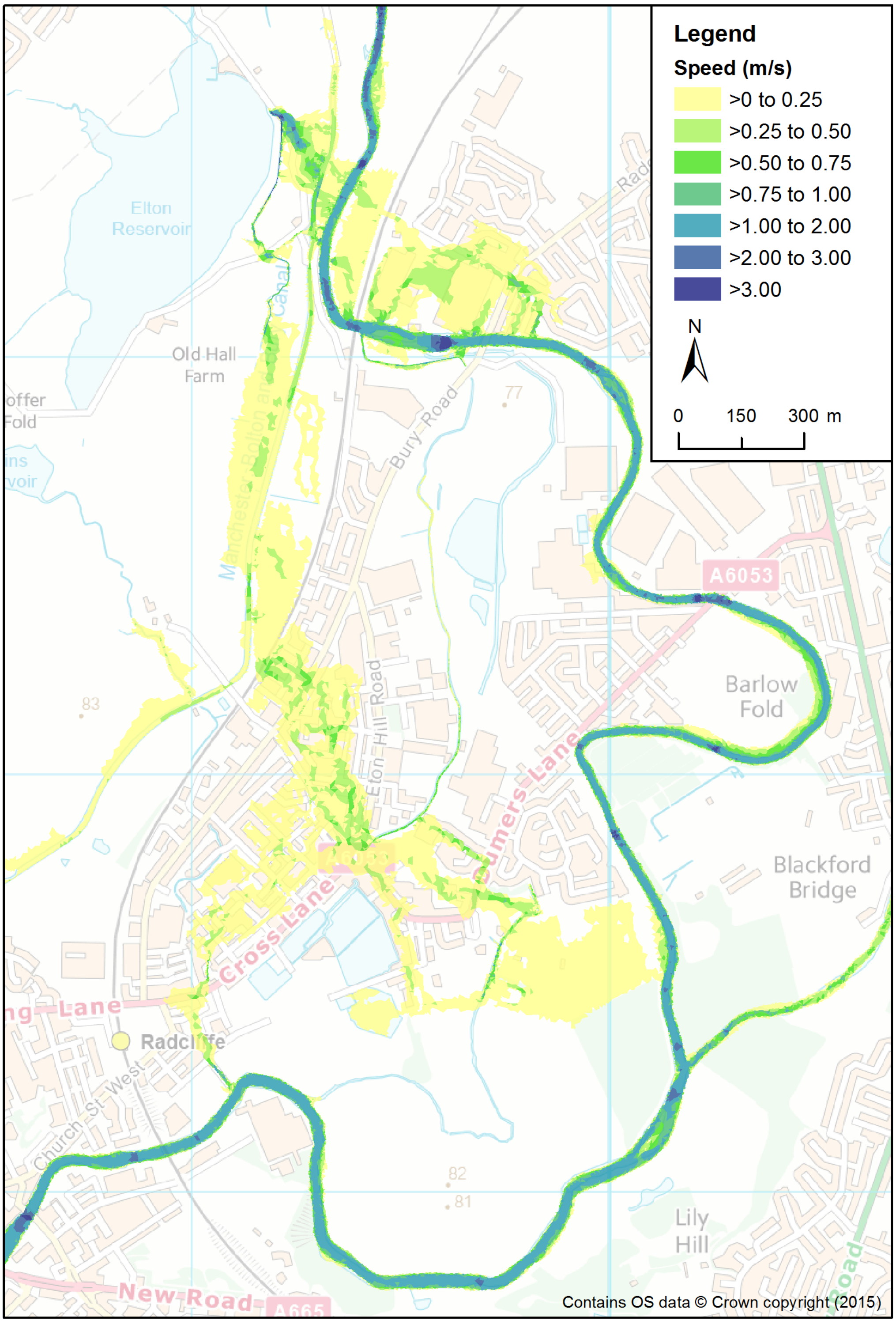

Figure 3.31: Maximum flow speed with a breach at the spillway with the 1 in 1,000 year inflow

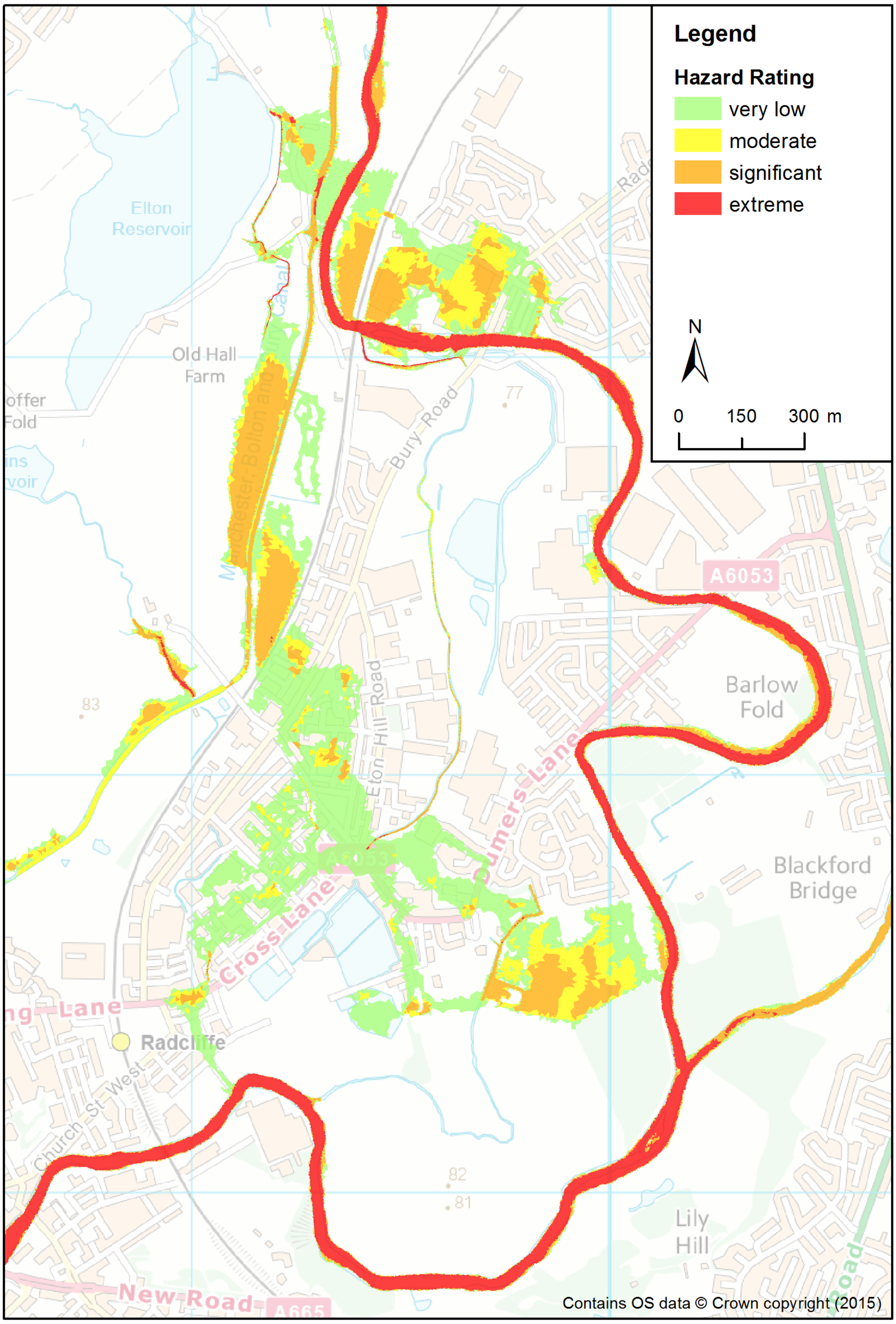

Figure 3.32: Maximum hazard rating with a breach at the spillway with the 1 in 10,000 year inflow

4. Summary

The following points may be concluded from the results of the breach and inundation modelling work undertaken:

- Piping is the likely failure mode for the Elton embankment dam and no overflowing occurred along the crest in all of the model runs.
- Breach modelling results show that the peak outflow ranges from approximately 15 m^3/s at section 1 with a 1 in 1,000 year flow event to 97 m^3/sec at section 3 with the PMF flow event.
- The peak of the breach outflow hydrographs and the resulting flood depth and flow speed for a given breach location do not differ significantly by varying the flow event.
- The flood inundation modelling shows that an area of North Radcliffe is at risk from inundation from a breach in the reservoir. This inundated area is classified as significant hazard for a breach at the outlet pipes.
- The flood inundation modelling shows that the area of greatest hazard due to a breach in the reservoir is within 1.5 km of the dam.
- The flood extent, flood depth, flow speed and hazard ratings are greatest for the breach at the outlet pipes and smallest for the breach at the spillway.

5. References

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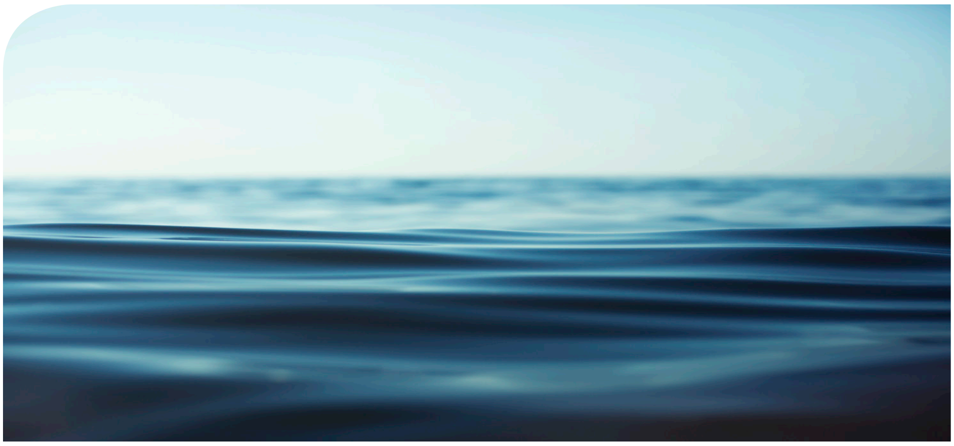

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