# Tonkin + Taylor















## **Document Control**

Title: Mapua, Ruby Bay and Tasman					
Date	Version	Description	Prepared by:	Reviewed by:	Authorised by:
12/8/2020	0.1	Draft for client review	K Ng	M Pennington	J Rix
25/6/2021	1.0	Final	K Ng	M Pennington	J Rix

Distribution:

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#### 1 Introduction

#### 1.1 Scope

Tonkin & Taylor Ltd (T+T) were engaged by Tasman District Council (TDC) to develop the understanding of flood hazard for the Mapua, Ruby Bay, and Tasman townships based on "2D+" flood modelling. The scope of works included the following:

- Familiarisation with existing information and flood models.
- "2D+" stormwater model build and validation.
- Meeting to present and discuss model results with TDC staff.
- Flood assessments for catchment induced flooding and coastal overtopping.

This report summarises the model build, model validation, and design events model results.

All levels presented in this report are to the 2016 New Zealand Vertical Datum (NZVD2016) unless noted otherwise.

#### 1.2 Hydrological setting

The catchment area draining through the Mapua, Ruby Bay, and Tasman townships was delineated based on a 2015 DEM acquired from Land Information New Zealand (LINZ). The total catchment area is approximately 34 km², located between State Highway 60 (SH60) and the coastline.

#### 2 Modelling purpose and approach

#### 2.1 Purpose & objective

The purpose of TDC's flood modelling programme is to support their responsibilities to identify flood hazard across their district. It is also used to inform their flood risk management and flood risk mitigation approaches whilst also supporting economic growth. The information is used to help TDC fulfil their statutory responsibilities (e.g. in regards the Building Act, Resource Management Act, Local Government Act and Civil Defence and Emergency Management Act).

The objective of this project is to identify areas of potential flood hazard for the Mapua, Ruby Bay and Tasman townships using a 2D+ methodology (refer Section 2.2). This report identifies the model build process and presents the floodplains resulting from the work. The flood hazard information can be used to refine and prioritise areas for further investigation as required.

Due to the wide range of potential uses for flood modelling results, it is important that the limitations and assumptions of the project are well understood. Refer to Section 2.3 for more information.

#### 2.2 Approach

A 2D direct rainfall model was built, covering the Mapua, Ruby Bay and Tasman areas using TUFLOW HPC software. T+T selected the TUFLOW HPC engine to allow for the simulation of 1D elements within the 2D GPU domain, such as Seaton Valley Stream and road crossings. Run-times are faster compared to a classic CPU approach and a finer resolution can be achieved by enabling smaller grid sizes. A finer resolution means a more accurate representation of overland flow paths.

A direct rainfall approach has been applied to this model, which can highlight accuracy deficiencies in input data by showing small "puddles" in predicted flooding. It is usual with flood depth results from this kind of modelling approach that the results be "cleaned" by removing puddles before

publication. T+T has presented raw model results in this report, in anticipation of TDC undertaking "cleaning" of model results before publication and further use.

It is worth noting that the modelling undertaken as part of this study simulates the flood related effects of an extreme 100-year average recurrence interval (ARI) rainfall event and an extreme sea level rise scenario that are based on future climate predictions. The severity of such an event is above that which has been experienced in the Tasman District to date.

#### 2.3 Limitations and assumption

Due to the wide range of potential uses for flood modelling results, this section provides an overview of important project limitations and assumptions. The limitations and assumptions typically present opportunities for improvement in the future but acceptable in the short term to ensure that the project objectives are met using effort and investment that is proportionate to the outcomes sought. "Continual improvement" and "making best use of existing information" are key principles of a long term approach to flood risk management.

The adopted approach allows for high level risk assessment and identification of areas which may benefit from flood mitigation options. For areas where a higher levels of confidence in the design flood level are required, or where there is a significant consequence associated with flood level assessment, a site-specific assessment should be carried out. In such areas, the effect of uncertainty on the specific decision may be an important consideration (it may also not be). This is consistent with a risk based approach to consenting and design.

Listed below are a summary of limitations to the modelling work, described further in the following sections, that WBOPDC will need to consider when interpreting and using design flood level results.

- Digital Elevation Model (DEM) derived from remotely sensed LiDAR survey data (refer Section 3.2.2) The limitations of accuracy of LiDAR data are well understood, and these limitations will apply to the model results obtained. In particular, LiDAR survey data and the resulting DEM will have lower accuracy in areas such as incised waterways, heavily vegetated areas, places where above-ground features have been removed and water bodies. Furthermore, where ground levels have been changed since the LiDAR survey was captured, the DEM and hence the model will not recognise these changes (and will be out of date).
- Stormwater infrastructure (refer Section 3.2.3) For areas containing reticulated stormwater network, this model build uses a 2D+ methodology which consists of a simplified representation of many hydraulic elements within the model. A notable limitation of this approach is in the lack of ability for representation of detailed hydraulic performance. Although these areas of the model are limited in spatial extent due to the limited spatial extent of the stormwater infrastructure, and in most instances would only convey a small overall volume of water in a future climate 100-year ARI rainfall event (that which is the focus of this study), caution should be made when using flood level outputs upstream and downstream of such locations. On-the-ground verification is recommended in such areas.
- Stormwater infrastructure In addition to the 2D+ methodology, several 1D culverts were applied to the model at key locations where data on culvert size was made available. In many cases, asset data was not available, and this is particularly evident in rural areas where it is likely for numerous private culverts to exist or where asset data capture is less complete, along rail networks, and in areas of new roading infrastructure.
- Model validation for one historic event has been carried out (refer Section 4 for further information). No model calibration has been carried out. Additional model validation and/or calibration would provide increased confidence in model results.

## 3 Model inputs

## 3.1 General parameters

The general parameters used in the TUFLOW model are summarised in Table 3-1.

Table 3-1: TUFLOW parameters

Parameter	Value
Model Cell Size	2 m x 2 m (4 m <sup>2</sup> )
Timestep	The TUFLOW HPC model uses an adaptive timestep, based on a maximum Courant number of 1
Viscosity	The default approach for viscosity in TUFLOW is the Wu method.

## 3.2 Spatial data

#### 3.2.1 Model extent

The extent of the model was set to include the entire catchment of the Mapua, Ruby Bay, and Tasman townships and outlined in Figure 3-1.



Figure 3-1: Mapua, Ruby Bay, and Tasman TUFLOW model extent

#### 3.2.2 Model terrain

The base digital elevation model (DEM) for ground level used in the model was acquired from LINZ based on a 2015 LiDAR survey in New Zealand Vertical Datum (NZVD) 2016.

The DEM represents a 'bare earth' terrain with all buildings and above-ground features having been removed through an automated process. With this approach, it is sometimes possible that flooding is shown to occur through areas occupied by large buildings. This is because the model does not recognise building locations and works only off the bare earth terrain DEM. The intention is that the results from this model will be used to identify key areas in need of flood mitigation works. Further, more targeted analysis (which may include more modelling) may therefore be required for specific flood issue areas.

Manual corrections to the DEM were required, specifically around the Seaton Valley Stream immediately upstream of Stafford Drive (refer Figure 3-2). In this location the automated

classification of the LiDAR removed elevated bunds due to incorrectly classifying these as vegetation. These bunds were manually reinstated into the DEM based on the raw 2015 LiDAR information.



Figure 3-2: Seaton Valley Stream bunds

In addition, a development around the Morley Drain north of Iwa Street was included in the model based on Planscapes design for Mapua Coastal Village (provided to T+T by TDC on 20 April 2020), refer Figure 3-3.



Figure 3-3: Development around Morley Drain

#### 3.2.3 Channels and stormwater infrastructure

Channels and stormwater infrastructure are depicted in Figure 3-4.

A number of 1D elements were embedded in the model, including:

- Seaton Valley Stream.
- Morley Drain.
- Culverts connecting the streams past weirs.
- Pipes containing flap gates.
- Road crossings identified in preliminary simulation runs that otherwise result in ponding when the 2D+ methodology is applied.

The remaining stormwater pipes (termed "simulated pipes") were included in the 2D model domain by modifying the DEM. Stormwater pipes with diameters less than 600 mm were generally not represented, unless these appeared to be under road embankments and would likely cause a ponding issue if not included.

The parts of the DEM covering Seaton Valley Stream and Morley Drain were removed from the 2D domain of the model and included as 1D elements based on as-built and design cross section data. These are shown in cyan in Figure 3-4.

The simulated pipes are shown in blue in Figure 3-4. This approach has been demonstrated to produce reliable results in previous studies<sup>1</sup> and is considered appropriate for extreme event simulation given that smaller pipes carry a relatively small proportion of total flood volume in larger events. GIS files with pipe locations and sizes were provided by TDC. Note that in the simulation of these pipes, the DEM is modified so that the hydraulic effects (volume and conveyance) of such pipes is replicated, such that in the immediate location of these pipes flood depths in the model results will appear unrealistic.

A selection of pipes were represented as 1D pipes in the model, shown in green in Figure 3-4. These pipes were included as 1D pipes either because they were within the 1D domain, were flapgated pipes only allowing flow in one direction, or were identified in preliminary model simulations as causing ponding issues if they were not included.

<sup>&</sup>lt;sup>1</sup> Tonkin + Taylor (August 2014), Increased Flood Vulnerability: Overland Flow Model Build Report, published on the Canterbury Geotechnical Database.

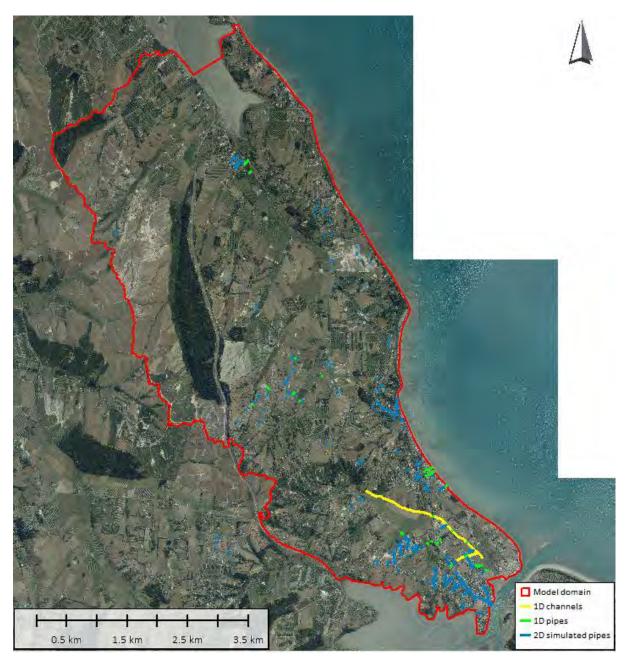


Figure 3-4: Channels and stormwater infrastructure

#### 3.2.4 Land Use

Roughness values adopted in the model were based on land use as categorised in Landcare Research's Land Cover Database version 4.1 (LCDB4). This database was released in July 2015 and last revised in November 2018. The database was the most current at the time of modelling. This data is freely available from the Land Research Information Systems portal. The land use for the model extent according to this information is shown in Table 3-2 and the assumed corresponding roughness values are summarised in Table 3-2.

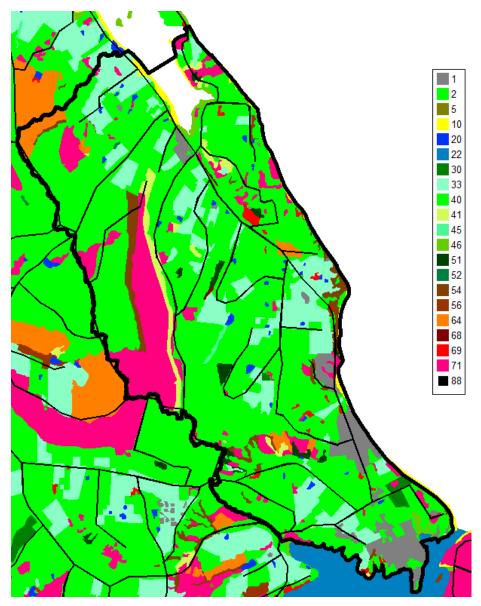


Figure 3-5: Land use in Mapua, Ruby Bay, and Tasman (LCDB4)

In addition to the above, all road centrelines were located in GIS and buffered to a width of 8 m. These areas were included on top of the land use layer and separate road Manning roughness values were adopted.

Table 3-2: LCDB4 Land types and corresponding Manning's n

Description	Code	Manning's n	Percentage Impervious
Built-up Area	1	Depth Varying	25%
Urban Parkland/ Open Space	2	0.033	0
Transport Infrastructure	5	0.016	0
Coastal Sand and Gravel	10	0.025	0
Lake and Pond	20	0.02	100%
Estuarine Open Water	22	0.022	100%
Short-rotation Cropland	30	0.1	0

Description	Code	Manning's n	Percentage Impervious
Orchard and Other Perennial	33	0.05	0
High Producing Exotic Grass	40	0.05	0
Low Producing Grassland	41	0.09	0
Herbaceous Freshwater Vegetation	45	0.1	0
Herbaceous Saline Vegetation	46	0.1	0
Gorse and Broom	51	0.125	0
Manuka and/or Kanuka	52	0.1	0
Broadleaved Indigenous Hard	54	0.1	0
Mixed Exotic Shrubland	56	0.08	0
Forest Harvested	64	0.16	0
Deciduous Hardwoods	68	0.125	0
Indigenous Forest	69	0.15	0
Exotic Forest	71	0.15	0
Road layer	88	0.02	100%

Depth varying Manning's n was used for 'Built-up Area'. This allows for a low roughness to be used at shallow depths to represent roofs and driveways. At higher depths, an increased roughness is applied to represent overland flow through urban areas where fences and buildings provide an impediment to flow, the depth varying roughness is outlined in Table 3-3.

Table 3-3: Depth varying manning's n coefficients for 'Built-up Area'

Depth	Manning's n
Less than 50 mm	0.015
50 mm – 100 mm	The value varies linearly from 0.015 to 0.05
Greater than 100 mm	0.05

#### 3.3 Boundary data

#### 3.3.1 Rainfall inputs

Rainfall in the model is simulated using a 'rain on grid' methodology. The same rainfall hyetograph is applied for over the entire model area, using an areal reduction factor assuming a 34 km<sup>2</sup> catchment area.

Rainfall depths were generated using NIWA's High Intensity Rainfall Design System (HIRDS) V4 for a range of event durations and exceedance probabilities. Both the present day rainfall and rainfall including the "representative concentration pathway" (RCP) 8.5 climate change horizon to 2090 were simulated.

The rainfall hyetographs for the 1% annual exceedance probability (AEP) rainfall event with climate change are shown in Figure 3-6. The total rainfall depths for other AEP events are summarised in Table 3-4.

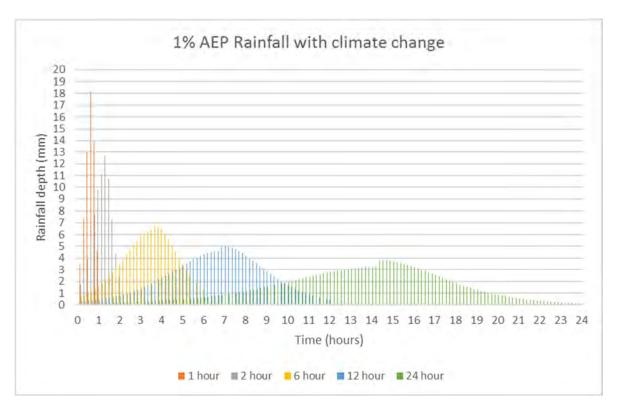


Figure 3-6: Example rainfall hyetographs for the 1% AEP storm with climate change

Table 3-4: Rainfall depths

AEP and climate change scenario	Duration (hours)	Total depth (mm)
1% AEP RCP8.5 to 2090	1	60.5
	2	80.0
	6	131.0
	12	165.1
	24	202.4
1% AEP Present Day	2	59.9
	6	96.3
50% AEP RCP8.5 to 2090	2	34.0
	6	54.8
50% AEP Present day	2	26.1
	6	41.9
10% AEP RCP8.5 to 2090	2	52.2
	6	84.9
10% AEP Present day	2	39.5
	6	63.5
2% AEP Present day	2	53.5
	6	86.1

In addition, a recorded rainfall event (19 July 2019) was used for the validation model (refer Section 4). The data was acquired from the TDC website at site location 'Mapua at Bowling Club' and is available for public access online.

#### 3.3.2 Infiltration losses

The Horton loss model was used to model the rainfall infiltration losses in the model. The Horton approach uses the equation:

$$f = f_c + (f_0 - f_c)e^{-kt}$$

Where  $f_0$  is the initial infiltration rate in mm/h,  $f_c$  is the final (indefinite) infiltration rate, t is time in hours and k is the Horton decay rate. For the TUFLOW implementation, the time (t) is the period of time that the cell is wet.

For the base case with starting parameters, the values adopted are summarised in Table 3-5. These parameters are best developed through calibration, which has not been carried out in detail as part of this model build. To ensure a conservative outcome the initial loss was set to zero.

The soil types are based on the Landcare Research fundamental soils layer, refer Figure 3-7. Table 3-5 identifies the infiltration parameters used by the model for different soil types. The land use percentage impervious (refer Table 3-2) reduces the infiltration applied.

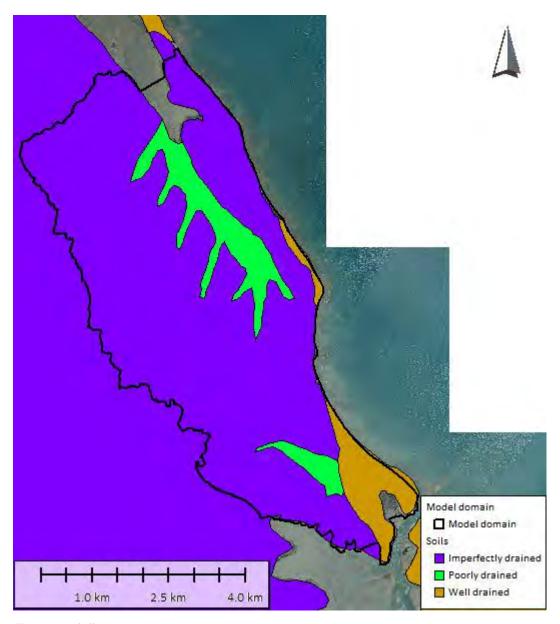


Figure 3-7: Soils

Table 3-5. Soil infiltration parameters using Horton Loss model

Soil type	Initial loss (mm)	Initial loss rate, f <sub>0</sub> (mm/hr)	Ultimate infiltration rate, fc (mm/hr)	Horton decay rate, k (1/hr)
Poorly drained	0	0.5	0.45	0.4176
Imperfectly drained	0	3.5	3.0	0.25632
Well drained	0	25	22.5	0.20844

### 3.3.3 Coastal boundary

The model includes a variety of coastal boundaries, dependent on the purpose of the simulations.

- Critical duration simulations were modelled with a static sea level set at Mean Sea Level (MSL,
   -0.23 m NZVD 2016), based on LINZ data.
- The design simulations include a dynamic tidal boundary with a peak of 1.73 m (Mean High Water Springs (MHWS)). The boundary is designed to peak concurrently with the arrival of the rainfall peak at the Tasman and Mapua estuaries, determined from the critical duration simulations.
- Coastal inundation simulations for extreme tide scenarios were modelled with dynamic levels coincident with overtopping flows.
- The coastal inundation scenarios included:
  - 1% AEP coastal inundation.
  - 1% AEP coastal inundation with 0.25 m sea level rise (SLR).
  - 1% AEP coastal inundation with 0.5 m SLR.
  - 1% AEP coastal inundation with 0.75 m SLR.
  - 1% AEP coastal inundation with 1.0 m SLR.
  - 1% AEP coastal inundation with 1.5 m SLR.
  - 1% AEP coastal inundation with 2.0 m SLR.
  - 2% AEP coastal inundation.
  - 0.5% AEP coastal inundation.

The tidal boundary associated with the coastal inundation scenarios was applied to the coastline inside the Mapua estuary. Coastal inundation due to overtopping is discussed in the following subsection.

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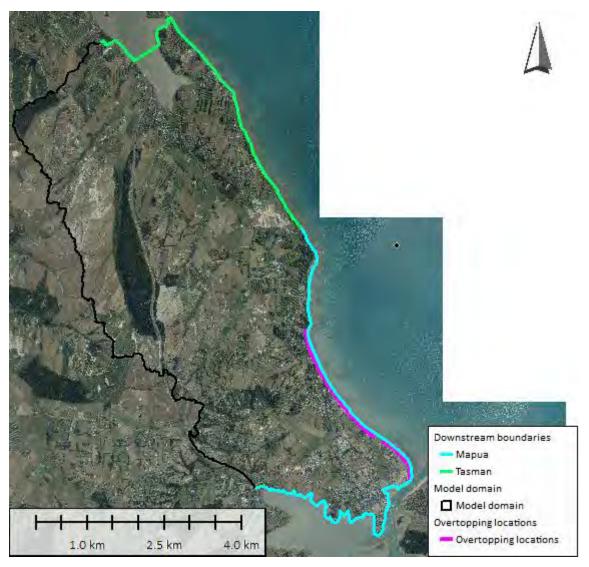


Figure 3-8: Downstream boundary and overtopping locations

#### 3.3.3.1 Coastal overtopping

Coastal inundation along the open coast is largely due to wave overtopping, which occurs when wave runup exceeds the height of the backshore slope and part of the wave moves onto the backing land. It is a function of water level (including storm surge and sea level rise), nearshore wave conditions and shoreline geology and geometry.

A sinusoidal tidal boundary with a peak equivalent to MHWS was used. The NIWA coastal calculator was used to determine joint probability combinations of storm tide (i.e. including storm surge) and wave height for each Annual Exceedance Probability (AEP) modelled. The combination which resulted in the highest overtopping rate (I/s/m) was chosen for each scenario.

Storm surge and wave heights were modelled to vary over a 24-hour period to better match wave height distributions seen at the Port of Nelson during large storm events.

The model includes a variety of coastal inundation scenarios, dependent on the purpose of the simulations, inputs of which are outlined in Table 3-6.

Table 3-6: Coastal inundation scenario inputs

Joint probability AEP (%)	Wave height (m)	Max storm tide (m) <sup>1</sup>	Sea level rise addition (m)
0.5	2.29	2.27	0
1	2.23	2.23	0, 0.25, 0.75, 1.0, 1.5 and 2.0
2	2.26	2.20	0

<sup>1 –</sup> levels expressed in NZVD2016

Mean wave overtopping discharge rates were calculated following the methodology outlined in Equation of 6.5 of the EurOtop manual (2018<sup>2</sup>), shown below:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.09 \times \exp \left[-(1.5 \frac{R_c}{H_{m0} \times \gamma_f \times \gamma_\beta})^3\right]$$

Where:

q = average overtopping flow (m<sup>3</sup>/s/m)

 $H_{m0}$  = estimated significant wave height (m)

R<sub>c</sub> = crest freeboard of structure/ berm (m)

 $\gamma_{eta}$  = influence factor for oblique wave attack

 $\gamma_f$  = influence factor for the permeability and roughness of or on the slope

The shoreline was assumed to have an average crest height of 3.7 m (NZVD2016), based on available LiDAR information. For scenarios with extreme sea level rise values (1%AEP + 1.5 m SLR and +2.0 m SLR), the still water level exceeds the assume crest level. In this case negative freeboard occurs at peak tides, resulting in overflows of the floodplain as well as wave overtopping (Figure 3-9). Equation 5.20 outlined in the EurOtop manual is used to model overflows during periods of negative freeboard, where qoverflow is the mean overflow rate (m³/s/m):

$$q_{overflow} = 0.54 \sqrt{g \times \left| -R_c^3 \right|}$$

 $<sup>^{2}</sup>$  EurOtop, 2018. Manual on wave overtopping of sea defences and related structures. 2nd edition.

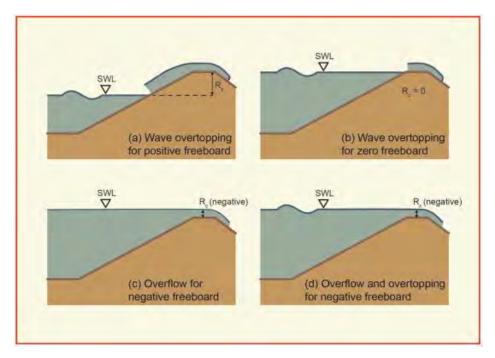


Figure 3-9: Wave overtopping for positive, negative and zero freeboard (EurOtop, 2018)

Overtopping rates (I/s/m) are shown in Figure 3-10 and the peaks are summarised in Table 3-7

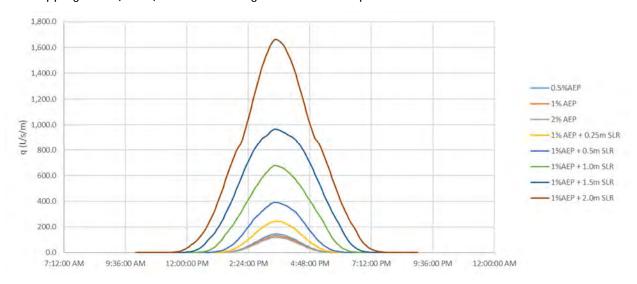


Figure 3-10: Mean overtopping rates for each scenario

Table 3-7: Peak overtopping values for each scenario

Coastal boundary scenarios	Peak overtopping rate (L/s/m)
0.5% AEP	144
1% AEP	130
2% AEP	118
1% AEP + 0.25 m SLR	245
1% AEP + 0. 5 m SLR	393
1% AEP + 0.75 m SLR	525

1% AEP + 1.0 m SLR	680
1% AEP + 1.5 m SLR	966*
1% AEP + 2.0 m SLR	1661*

<sup>\*</sup> Negative freeboard occurs so overflow occurs.

#### 4 Model validation

The data available to support a model validation comprises:

- 1 Rainfall data recorded at Mapua at Bowling Club (since April 2018).
- 2 Tide gauge records (NIWA).
- 3 Water level records for the Seaton Valley Stream.
- 4 Historic flood photographs from flood event on 29 June 2003.
- 5 Anecdotal record of flooding from TDC staff for a flood event on 19 July 2019.

The 19 July 2019 was simulated in the hydraulic model using recorded rainfall and tidal time series from records. The model results were compared with the water level record at Seaton Valley Stream and with the patterns of flooding from the (earlier) 2003 photos. The validity of the modelled results compared to the photos were confirmed based on recollections from TDC staff. The 2019 rainfall event was modelled as there was no rainfall record available for the 2003 event.

The Seaton Valley Stream water level gauge data was compared to the water level in the 19 July 2019 simulation (refer Figure 4-1). The results show that the modelled water level was comparable to the gauged water level, though there was a discrepancy with the timing of between 2-4 hours.

The datum of the Seaton Valley Stream water level gauge could not be confirmed although we suspect that it is Nelson Vertical Datum due to the age of the record. In order to compare the water level results to NZVD 2016, the water levels were adjusted by 0.34m. We recommend confirming the level of the datum to give additional confidence and prior to any additional calibration or validation exercises.

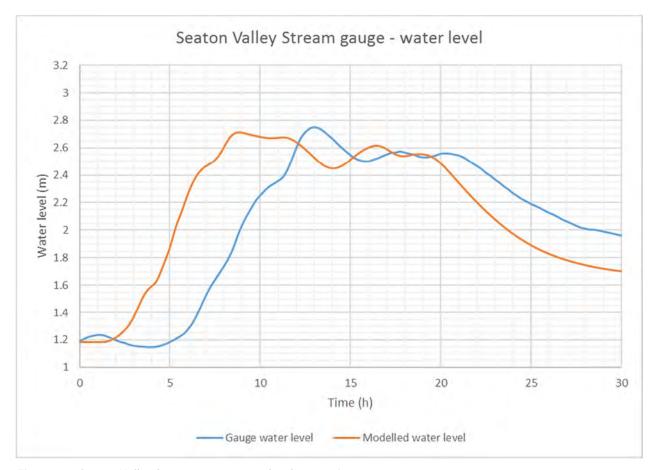


Figure 4-1: Seaton Valley Stream gauge water level comparison

A selection of the comparisons between the photographs from 2003 and modelled results are presented in Figure 4-2.

Maps of the validation model results are included in Appendix A.

Site	Photograph (2003 event)	Modelled (19 July, 2019 flood event)	Comment
Seaton Valley Road  – looking to Mapua Drive			Extent of flooding within roadside swale appears consistent between photo and model results. Some flooding on road in photo appears to be shallow and would be removed from model results by 100 mm minimum depth cut-off.
Corner of Seaton Valley Road and Mapua Drive	73 6 25		Photographs show similar shape of flooding to model results.
Seaton Valley Stream floodplain - from Seaton Valley Road			Model has slightly less flooding than the 2003 flood photos, however it is understood that the 2003 flood was a bigger event than the 2019 event.
Seaton Valley Road - Seaton Valley Stream floodplain on left, Wells land on right			Model shows flooding on either side of the road, ponding area on the right side of photo seems to match well.

Figure 4-2: Validation photos compared to model results

#### 5 Assessment scenarios and results

The following subsections present the model results from catchment induced flooding and from coastal overtopping.

#### 5.1 Catchment induced flooding

The catchment-induced flooding (from rainfall) was determined based on an assessment of critical duration storm, followed by an assessment of the floodplain for design storms of critical duration. Due to the lack of data to support a comprehensive model validation, a sensitivity assessment of the floodplain to catchment parameters was carried out.

#### 5.1.1 Critical duration

Design storm durations of 1 hour, 2 hours, 6 hours, 12 hours, and 24-hours was considered for the 1% AEP rainfall event to identify the critical duration in the areas of interest (i.e. the storm duration that gives the highest peak water level).

The assessment was carried out using a static mean sea level (-0.23 m) downstream boundary for consistency.

A map showing the critical duration throughout the model is shown in Figure 5-1, and included in higher resolution in Appendix B. the results show that the critical storm duration in the areas of interest to TDC were the 2 and 6-hour storms.

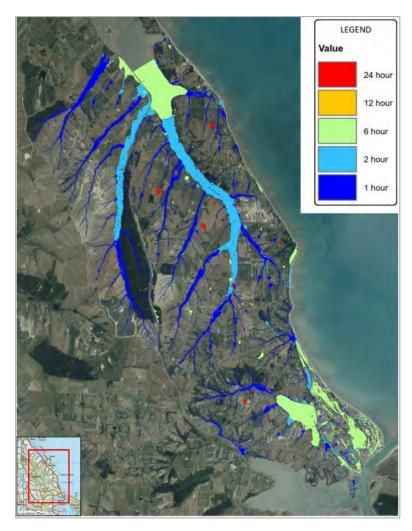


Figure 5-1: Critical duration

#### 5.1.2 Design storm events

The model was set up to complete the following twelve simulations.

- 1% AEP rainfall with dynamic MHWS boundary present day for 2 and 6-hour durations.
- 1% AEP rainfall with dynamic MHWS boundary with climate change RCP8.5 to 2090 for 2 and 6-hour durations.
- 10% AEP rainfall with dynamic MHWS boundary present day for 2 and 6-hour durations.
- 10% AEP rainfall with dynamic MHWS boundary with climate change RCP8.5 to 2090 for 2 and 6-hour durations.
- 50% AEP rainfall with dynamic MHWS boundary present day for 2 and 6-hour durations.
- 50% AEP rainfall with dynamic MHWS boundary with climate change RCP8.5 to 2090 for 2 and 6-hour durations.

The results of the different storm durations have been combined into maximum single flood depth, velocity and DxV (depth x velocity) figures. The results are presented for the 1%, 10% AEP and 50% AEP design storms in Appendices C, D and E respectively.

#### 5.1.3 Sensitivity

Due to the lack of information to support a comprehensive model validation exercise, a model sensitivity assessment was carried out. The sensitivity of the floodplain to roughness and infiltration parameters was considered for the 6 hour duration, 1% AEP rainfall with allowance for climate change.

The roughness and infiltration parameters were increased and decreased by 20%.

The sensitivity results are presented in Appendix H.

A "fuzzy map" was produced, which presents a summary of all the sensitivity and design results, allowing a quick understanding of the percentage of scenarios where a given grid cell is wet or dry. In cases where the cell is either always wet (where a cell is considered wet if it has 0.1m flood depth or more) or always dry in all scenarios, the model is considered less sensitive. Conversely, if a cell is only wet in 50% of scenarios, this indicates that the cell is more sensitive to the modified parameters. The fuzzy map showed that the model was not overly sensitive to modified parameters.

Furthermore, differences between the baseline and the sensitivity simulations were checked to determine the actual effect of each independent parameter.

The results show that at a catchment scale:

- Infiltration did not have a significant effect on the overall extent of flooding, with the most pronounced effect in ponding areas in the Mapua catchment, with flood depth differences of less than 0.05 m.
- Modifying the roughness had a moderate effect on the flooding, primarily in the fast moving main channels in the Tasman catchment. The differences were generally less than 0.1 m, except in a small area near the intersection of Aporo Road and Permin Road, where differences were in the 0.1 0.2 m range. This area is heavily planted.

#### 5.2 Coastal overtopping

Coastal inundation at Ruby Bay was modelled for the following combinations of rainfall and overtopping (Section 3.3.3.1). The Annual Exceedance Probabilities (AEPs) are the combined joint probability for storm tide and wave height.

Table 5-1: Modelled coastal inundation scenarios

	Rainfall				
Coastal boundary scenarios	No rainfall	1% AEP rainfall (present day)	2% AEP rainfall (present day)	10% AEP rainfall (present day)	50% AEP rainfall (present day)
0.5% AEP	Р				
2% AEP		Р	Р	Р	Р
1% AEP		Р	Р	Р	Р
1% AEP + 0.25 m SLR		Р	Р	Р	Р
1% AEP + 0.5 m SLR		Р	Р	Р	Р
1% AEP + 0.75 m SLR		Р	Р	Р	Р
1% AEP + 1.0 m SLR		Р	Р	Р	Р
1% AEP + 1.5 m SLR		Р	Р	Р	Р
1% AEP + 2.0 m SLR		Р	Р	Р	Р

The flood depth, velocity and  $D^*V$  (depth x velocity) is presented for the scenarios identified in Table 3-7 in Appendices G, H and I respectively.

#### 6 Conclusion

Tonkin & Taylor Ltd (T+T) were engaged by Tasman District Council (TDC) to identify areas of potential flood hazard for the Mapua, Ruby Bay, and Tasman townships based on a "2D+" flood modelling approach.

The adopted modelling approach allows for high level risk assessment and identification of areas which may benefit from flood mitigation options. Flood maps showing indicative maximum flood depths, flood velocities and D\*V (depth x velocity) were developed for all the modelled scenarios. These flood maps are suitable for identifying areas of flood hazard and can be used to refine and prioritise areas for further investigation. For areas where higher levels of confidence are required, or where there are significant consequences associated with flood level assessment, a site-specific assessment should be carried out.

Model validation for one historic event has been carried out based on the availability of historic flood records. If additional information can be gathered, then there are opportunities to reduce model uncertainty by carrying out additional model validation and/or calibration. In the absence of additional validation and/or calibration, a sensitivity assessment was carried out to assess the effect of roughness and infiltration parameters on the floodplain. The results show that the catchment-wide floodplain is largely insensitive to these parameters. This conclusion may not apply at site specific locations where small scale sensitivity to model parameters is not well represented when analysing catchment scale results.

The results of the study, in particular the floodmaps, can be used to inform a spatial flood risk assessment (based on consequence) and to inform land use decision making and prioritise areas for flood mitigation.

### 7 Applicability

This report has been prepared for the exclusive use of our client Tasman District Council, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

Tonkin & Taylor Ltd

Report prepared by: Authorised for Tonkin & Taylor Ltd by:

Kevin Ng Jon Rix

Water Resources Engineer Project Director

Report reviewed by:

Mark Pennington, Senior Water Resources Engineer

KKSN

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## Appendix A: Validation data

## Appendix B: Critical duration

Appendix C: Catchment flooding - 1% AEP flood

results

Appendix D: Catchment flooding- 10% AEP flood

results

Appendix E: Catchment flooding - 50% AEP flood results

## Appendix F: Sensitivity



1004543.3100

## Appendix G: Coastal overtopping flooding - Depth

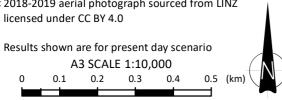
## Appendix H: Coastal overtopping flooding - Velocity

Results shown are for present day scenario A3 SCALE 1:10,000 0.5 (km)



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PEAK FLOOD VELOCITY 50% AEP RAINFALL WITH 1% AEP OVERTOPPING + 0.25m SLR Figure H3



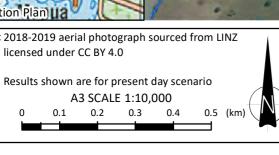


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MAPUA, RUBY BAY AND TASMAN FLOOD RISK
PEAK FLOOD VELOCITY

50% AEP RAINFALL WITH 1% AEP OVERTOPPING + 0.50m SLR

FIGURE No. Figure H4





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MAPUA, RUBY BAY AND TASMAN FLOOD RISK
PEAK FLOOD VELOCITY
50% AEP RAINFALL WITH 1% AEP OVERTOPPING + 0.75m SLR

FIGURE No. Figure H5

Results shown are for present day scenario

A3 SCALE 1:10,000

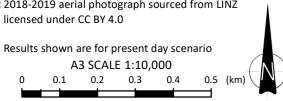
0 0.1 0.2 0.3 0.4 0.5 (km)



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SCALE (AT A3 SIZE)			
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MAPUA, RUBY BAY AND TASMAN FLOOD RISK
PEAK FLOOD VELOCITY
50% AEP RAINFALL WITH 1% AEP OVERTOPPING +1.00m SLR

FIGURE No. Figure H6





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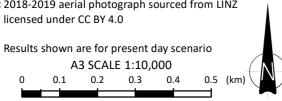
IASMAN DISTRICT COUNCIL

MAPUA, RUBY BAY AND TASMAN FLOOD RISK

PEAK FLOOD VELOCITY

50% AEP RAINFALL WITH 1% AEP OVERTOPPING + 1.50m SLR

FIGURE No. Figure H7





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MAPUA, RUBY BAY AND TASMAN FLOOD RISK
PEAK FLOOD VELOCITY

50% AEP RAINFALL WITH 1% AEP OVERTOPPING + 2.00m SLR

FIGURE No. Figure H8

SCALE (AT A3 SIZE)

51 Halifax Street, Nelson www.tonkintaylor.co.nz 1:10,000 PROJECT No. 1004543.3103

Figure H11

10% AEP RAINFALL WITH 1% AEP OVERTOPPING + 0.25m SLR

A3 SCALE 1:10,000

0.5 (km)

Results shown are for present day scenario

A3 SCALE 1:10,000

0 0.1 0.2 0.3 0.4 0.5 (km)



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MAPUA, RUBY BAY AND TASMAN FLOOD RISK
PEAK FLOOD VELOCITY

10% AEP RAINFALL WITH 1% AEP OVERTOPPING + 0.50m SLR

FIGURE No. Figure H12

SCALE (AT A3 SIZE)

51 Halifax Street, Nelson www.tonkintaylor.co.nz 1:10,000 PROJECT No. 1004543.3103

Figure H13

10% AEP RAINFALL WITH 1% AEP OVERTOPPING + 0.75m SLR

A3 SCALE 1:10,000

0.5 (km)

Results shown are for present day scenario

A3 SCALE 1:10,000

0 0.1 0.2 0.3 0.4 0.5 (km)



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MAPUA, RUBY BAY AND TASMAN FLOOD RISK
PEAK FLOOD VELOCITY

10% AEP RAINFALL WITH 1% AEP OVERTOPPING +1.00m SLR

1:10,000 PROJECT No. 1004543.3103

Figure H15

51 Halifax Street, Nelson www.tonkintaylor.co.nz

0.5 (km)

Results shown are for present day scenario

A3 SCALE 1:10,000

0 0.1 0.2 0.3 0.4 0.5 (km)



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MAPUA, RUBY BAY AND TASMAN FLOOD RISK
PEAK FLOOD VELOCITY

10% AEP RAINFALL WITH 1% AEP OVERTOPPING + 2.00m SLR

FIGURE No. Figure H16

1:10,000 PROJECT No. 1004543.3103

Figure H19

51 Halifax Street, Nelson www.tonkintaylor.co.nz

0.5 (km)

2% AEP RAINFALL WITH 1% AEP OVERTOPPING + 0.25m SLR



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PEAK FLOOD VELOCITY 2% AEP RAINFALL WITH 1% AEP OVERTOPPING + 0.50m SLR



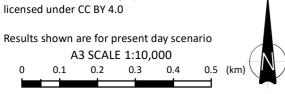
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PEAK FLOOD VELOCITY 2% AEP RAINFALL WITH 1% AEP OVERTOPPING + 0.75m SLR



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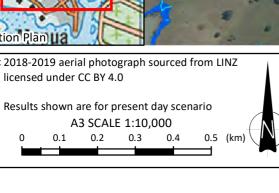
PEAK FLOOD VELOCITY 2% AEP RAINFALL WITH 1% AEP OVERTOPPING +1.00m SLR





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PEAK FLOOD VELOCITY 2% AEP RAINFALL WITH 1% AEP OVERTOPPING + 1.50m SLR





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PEAK FLOOD VELOCITY 2% AEP RAINFALL WITH 1% AEP OVERTOPPING + 2.00m SLR

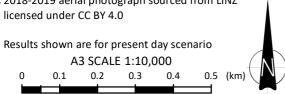
Figure H24

1:10,000 PROJECT No. 1004543.3103

Figure H27

51 Halifax Street, Nelson www.tonkintaylor.co.nz

0.5 (km)





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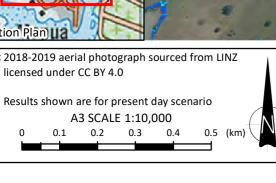
PEAK FLOOD VELOCITY 1% AEP RAINFALL WITH 1% AEP OVERTOPPING + 0.50m SLR



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PEAK FLOOD VELOCITY 1% AEP RAINFALL WITH 1% AEP OVERTOPPING + 0.75m SLR

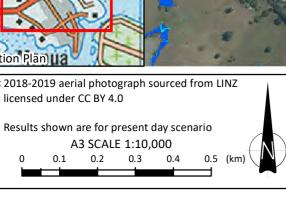
Figure H29





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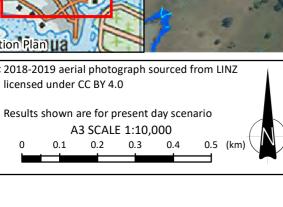
PEAK FLOOD VELOCITY 1% AEP RAINFALL WITH 1% AEP OVERTOPPING +1.00m SLR





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PEAK FLOOD VELOCITY 1% AEP RAINFALL WITH 1% AEP OVERTOPPING + 1.50m SLR





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ARCFILE Figure H32.mxd			
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PEAK FLOOD VELOCITY 1% AEP RAINFALL WITH 1% AEP OVERTOPPING + 2.00m SLR FIGURE No. Figure H32

Appendix I: Coastal overtopping flooding – D\*V

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