DESIGNING FOR A FLOOD THAT SHOULDN'T HAPPEN

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ABSTRACT

The MacKays to Peka Peka Expressway Project passes through Paraparaumu and Waikanae, north of Wellington. Resource consenting involved rigorous stormwater assessment and detailed modelling of flood risk in critical areas, including a crossing of the Waikanae River.

One unusual aspect was that the Expressway crosses an historical river overflow path (identified in the District Plan), running north from the Waikanae River, through parts of the Waikanae township, to the adjacent Waimeha Stream. The overflow is now protected by the Waikanae River stopbanks and therefore should not flow in the 1% AEP design event. However, a stopbank breach would flood residential areas, and the Expressway embankment could potentially block the passage of these overflows. The challenge was defining what constituted a "design" (or "super-design") event, how it could be modelled appropriately and the implications for the Expressway design. Further complications included identifying a suitable Waikanae River flow to use, the failure rate of the stop bank and the relative timing of the associated flood in both the river and the receiving Waimeha catchment.

As a consequence of this work, the Expressway includes a designed floodway to capture any breach flows, conveying them under the Waimeha Stream Expressway bridge via an additional 50 m land span, and returning them to the existing historical flow path downstream.

This paper outlines the methods applied in investigating and understanding the location of the breach, its failure mode, how its progressive development was modelled and the consequences of that modelling on the design and on the flood resilience of the community. The design was tested in MIKE FLOOD software in order to find an appropriate solution that maintained flood resilience.

KEYWORDS

Stop bank failure, secondary flow path, hydraulics, numerical modelling

PRESENTER PROFILE

Justin Kirkman - Senior Environmental Engineer with 8 years' experience in hydrology and hydraulics. Justin co-ordinated the stormwater design and compliance of the Waikanae River and Waimeha Stream catchments of the M2PP project.

Philip Wallace - Principal Engineer with DHI Water and Environment, specializing in river modelling, engineering and in floodplain management policy. He has over 25 years of experience in flood hazard modelling and investigations.

Graham Levy – Technical Director at Beca and M2PP technical design reviewer, Graham oversaw the M2PP project from resource consent through to detailed design.

1 INTRODUCTION

The Te Moana Floodway is an overbank flow path that forms when the Waikanae River swells beyond the capacity of the channel banks at the Chillingworth stopbank. This floodway is needed in case of a breach or overtopping of the stopbanks delivering flood flows through a part of the Waikanae beach residential zone and across the new MacKays to Peka Peka Expressway (the Project) corridor. This stopbank is designed to keep the Waikanae River flood waters within the channel, and so flooding is not expected during events up to and including the 100 year ARI. Figure 1 shows the relative location of the Te Moana Floodway to the new Expressway, Waikanae River and the Chillingworth stopbank.

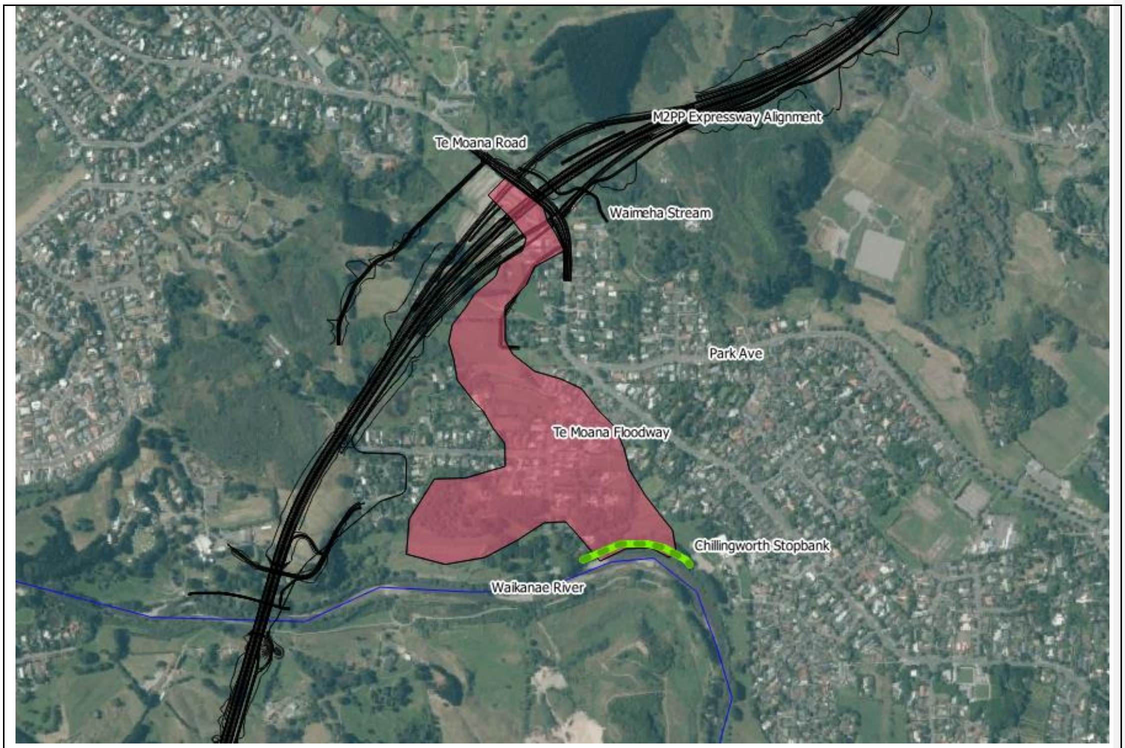

Figure 1: Location of the Te Moana Floodway relative to the Waikanae River and new Expressway

The assessment undertaken for the project and detailed in this paper is a very different situation to a 100 year flood in a large river. One must factor in not only the likelihood of a 100 year storm occurring but then on top of that the chance of the river stopbanks (which are designed specifically to stand up to and contain such a storm) also failing. This makes it much less likely to occur than a 100 year flood.

The Kāpiti Coast District Council's District Plan (KCDC), based on Greater Wellington Regional Council (GWRC) Hazard mapping, identifies a "residual overflow" path that crosses the Expressway alignment between Te Moana Road and the sand dune hills just south of Te Moana Road. This is the historic flood flow path from the Waikanae River that is now cut off and protected by GWRC's stopbanks at the river itself. However, if the stopbanks were to breach or overtop, flood flows would continue down the old flow path, hence the Council term “residual”. The flow path runs over the currently pastoral/market garden land and crosses Te Moana Road (near to the Golf Club entrance/carpark) on its way to the Waimeha Stream and the coast.

The duration over which the hydrograph exceeds the spill level (following the stopbank breach establishing) will affect the volume of spill discharged, and therefore potentially the peak flow rate through the residual overflow path downstream. This residual overflow path is shown in the KCDC District Plan, and will be blocked by (and therefore needs to be diverted around) the Expressway fill embankment.

In terms of the implications of the breach, and hence the potential effects of the proposed Expressway, there are several points to consider.

- The location and ultimate sill length and level of the breach;
- The duration of the upper portion of the river flood hydrograph, and the relationship between the time of the upper portion and the timing of the breach;
- Whether there is any attenuation effect downstream, meaning a longer duration and/or greater spill volume could result in greater peak flows (i.e. less attenuation) downstream.
- Effect of a coinciding flood peak in the smaller Waimeha Stream catchment.

In assessing these matters, it is important that care is taken to avoid combining worst case scenarios that would likely not occur together, resulting in a design basis that is for a worse case than the nominal design standard. These super-design scenarios should be assessed as sensitivity tests, rather than as a design case.

2 RISK TO THE COMMUNITY

2.1 FLOOD IMPACTS TO RESIDENTIAL PROPERTIES

Approximately 450 houses at Waikanae Beach are protected by stopbanks from Waikanae River flooding[1] and the Chillingworth stopbank protects around 50 of those. In extreme flood events, dwellings in the Te Moana floodway rely on the stopbank operation in the first instance and then on the floodway itself to prevent property damage and risk to safety. Concerns regarding the effect of the Expressway on the continued operation of this floodway were raised during the consenting phase of the project. Issues with consenting a permitted flood effect within a floodway that is no longer in operation presented challenges. To overcome the challenge, the Project Alliance undertook an investigation that explored the following:

Comment [PW1]: This para a little confused. I’m not sure if my suggestions make it clearer or not.

- The magnitude/frequency of storm event to which flood impacts are to be tested against;
- Mode of flooding risk, either by overdesign event or stopbank failure;
- Options for flood impact mitigation and assessment of flood effects from design.

[1] GWRC (2010), *“Waikanae Floodplain Management Plan – 10 year Review”*, Flood Protection Division, Wellington.

2.2 EXACERBATION OF FLOOD LEVELS BY THE EXPRESSWAY

The Expressway would block the passage of flows in the Te Moana floodway during a Chillingworth stopbank breach or overtopping event unless outfall was preserved. A key issue to designing a floodway outfall is how much conveyance is required and what is an acceptable effect. The consenting process did not stipulate an exact measure of success with regard to these design parameters. Therefore, the Expressway design could not be completed until this assessment was completed and the affects agreed to with GWRC.

2.3 EXISTING STOPBANK DESIGN

The stopbank at Chillingworth has been in operation during a number of floods in the Waikanae River including a 15 year and 28 year ARI floods in 1998 and a 1 in 80 year flood in 2005, and performed as intended by preventing overtopping or failure. However, a review of the flood models used to derive the height of the stopbank at Chillingworth showed the original design did not consider the effects of climate change on rainfall intensities. This resulted in an undersizing of the required stopbank height as reported in the Waikanae Floodplain Management Plan, 2010. Recent design work has been undertaken to increase the height of this stopbank to meet the needs of climate change projected river flood levels.

Comment [GJL2]: Do we have any specifics – event size?

Comment [JK123R2]: Yes sorry, these are stated in the Waikanae Floodplain Management Plan

Comment [PW4]: Part of the Jim Cooke Park design work I think. Physical works yet to begin?

Part of this assessment was in assessing the effects of climate change modified river flood levels on the existing stopbank height and determining the potential for design stopbank overtopping.

3 WAIKANAE RIVER HYDROLOGY

3.1 CATCHMENT PROPERTIES

The Waikanae River Catchment is the largest along the M2PP route, comprising 138 km² of densely vegetated hill slopes, flat farmland and incised gullies. The catchment response time is large and depending on antecedent catchment conditions, rainfall will generate a flood peak in the order of 3-6 hours at the M2PP project alignment. Figure 1 below shows the relative size of the Waikanae River catchment to the M2PP project alignment.

Comment [PW5]: Do you mean response time of 3-6 hrs, or a flood peak duration of 3-6hrs?

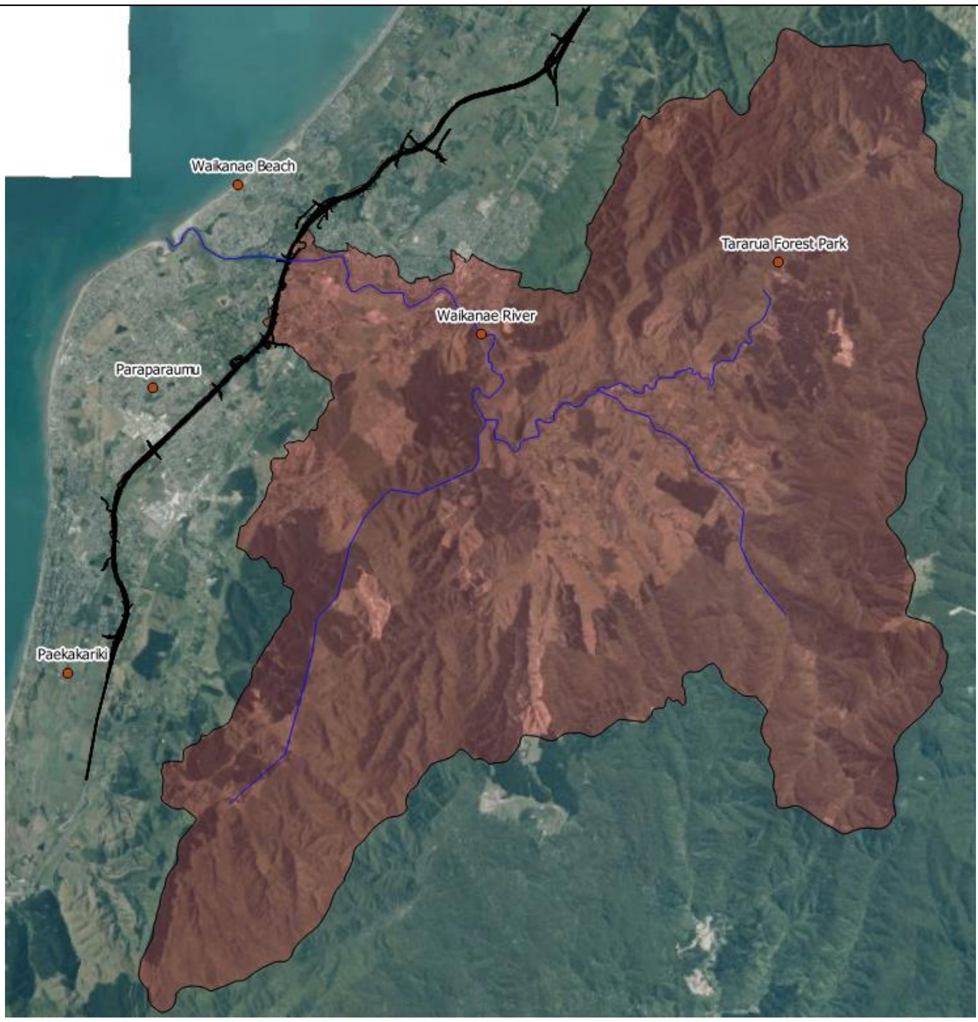

Figure 2: Waikanae River Catchment on the Kápiti Coast

3.2 RAINFALL

Climate change adjusted rainfall for the Waikanae Catchment was taken from the NIWA website and 2.0 degree Celsius climate change adjusted rainfall intensities extracted. Longer duration storms within the Waikanae catchment could be expected to have peaks representing a high intensity burst within a longer duration storm. This would result in a peak reflecting that shorter higher intensity burst, with shoulders on the hydrograph representing the ongoing storm at a lower average intensity but similar overall ARI. Considering the typical IDF curves for this catchment (taken from HIRDS V3 as shown in Figure 3), the following can be observed:

Comment [GJL6]: Try and be consistent – return period or AEP, not either/or

- The 2 hour intensity is about 70% greater than the 6 hour intensity;
- The 12 hour intensity is about 70% of the 6 hour intensity;
- The 24 hour intensity is about 50% of the 6 hour intensity;

- The 48 hour intensity is about 30% of the 6 hour intensity.

High Intensity Rainfall System V3											
Intensity-Duration-Frequency results (produced on Wednesday 7th of September 2011)											
Sitename: Waikanae											
Coordinate system: NZMG											
Easting: 2693544											
Northing: 6028659											
Projected temperature change: 2.0 degree Celsius											
Rainfall intensities (mm/h)											
					Duration						
ARI (y)	aep	10m	20m	30m	60m	2h	6h	12h	24h	48h	72h
1.58	0.633	46.2	35.1	29.6	22.1	16.3	10	7.4	5.4	3.4	2.6
2	0.5	50.4	37.8	31.8	23.9	17.6	10.7	7.8	5.7	3.7	2.8
5	0.2	63.6	47.7	40.4	30.6	22.2	13.4	9.8	7.1	4.5	3.5
10	0.1	74.4	56.1	47.6	35.9	26.1	15.6	11.3	8.2	5.2	4
20	0.05	86.4	65.4	55.4	42.1	30.4	18.1	13	9.4	6	4.6
30	0.033	94.2	71.4	60.8	46.2	33.2	19.7	14.2	10.2	6.5	5
40	0.025	100.2	75.9	64.8	49.1	35.2	20.8	14.9	10.7	6.9	5.3
50	0.02	105	79.8	67.8	51.4	36.9	21.7	15.5	11.1	7.1	5.5
60	0.017	109.2	82.8	70.6	53.5	38.2	22.4	16.1	11.5	7.4	5.7
80	0.012	116.4	87.9	75	56.8	40.5	23.7	16.9	12	7.7	6
100	0.01	121.8	92.1	78.4	59.5	42.4	24.7	17.6	12.5	8	6.2
						172%					
								71%			
									51%		
										32%	

Figure 3: Rainfall IDF for the Waikanae River Catchment from NIWA

These rainfall intensities used in conjunction with river gauge data can be used to better understand the Waikanae River hydrograph development.

3.3 RIVER HYDROGRAPH DEVELOPMENT

The hydrographs developed to represent the Waikanae River in flood have come from analysis by NIWA of historical floods. These have been normalized to compare shape as shown in Figure 4.

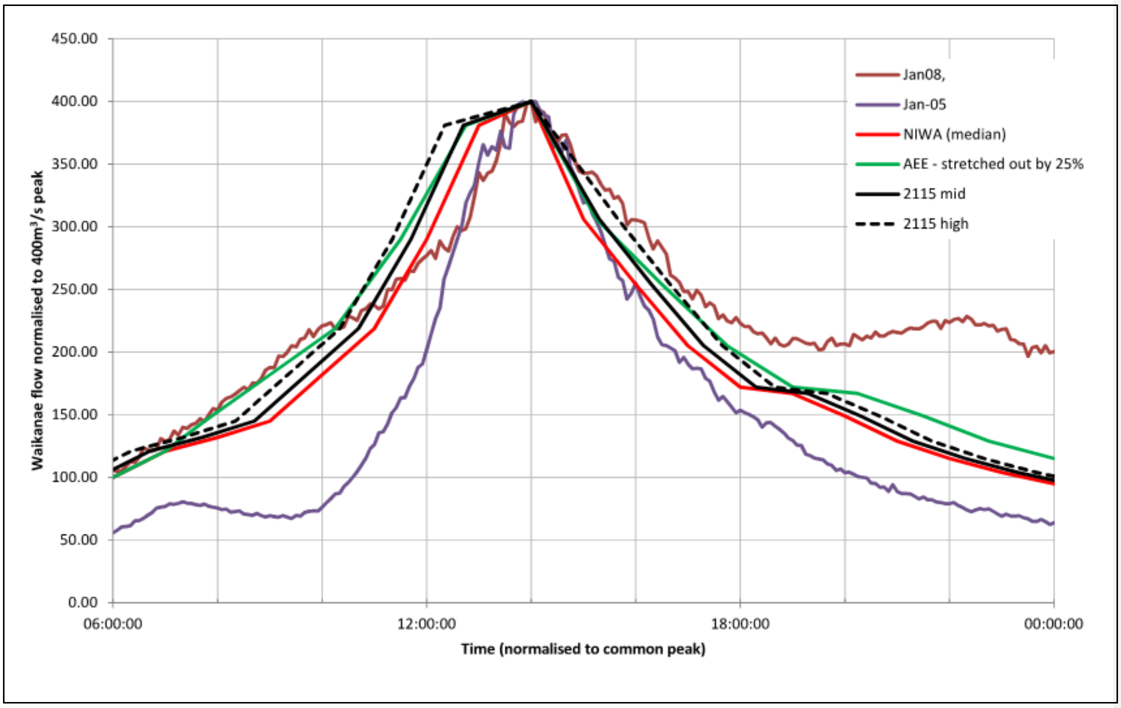

Figure 4: Historical and modelled Waikanae hydrographs

Figure 4 shows both stream gauge records and the modified inflow hydrograph used in the modelling. The original hydrograph derived using the Natural Resources Conservation Service (NRCS) temporal pattern[2] was stretched out at the peak to better match the observed data. The historical hydrographs show that there was typically a distinct peak of around 1 to 2 hours in duration, with (in some cases) shoulders at about 20% to 50% of that peak such as:

- For the January 2008 storm, there was a sustained tail and minor secondary peak at about 50% of the main peak;
- For the January 2005 storm, there was a lead in flowrate that peaked at 20% of the main peak;
- Both these events appear, from the length of the hydrograph, to reflect storms of the order of 24 hours duration.

Comment [PW7]: What does NRCS stand for?

Comment [GJL8]: Comment – took NIWA, then for sensitivity stretched peak duration, and applied climate change etc. to peak

Comment [JK129]: New topic sentence added to give a more accurate description of figure 4.

Comment [GJL10]: Meaning?

Comment [JK1211]: Passage adjusted to suit context.

[2] NRCS temporal pattern is a nested storm approach built from Intensity, Duration, Frequency (IFD) data. This approach is built by placing the rainfall depth for 10minute storm duration at the peak or middle of the temporal distribution and surrounding this peak using the rainfall depths for longer duration design storms up to a 24 hour rainfall depth. This approach estimates the flow for all storm durations in the IDF table, thereby removing the need to discover the critical duration storm for a catchment.

4 BREACH FORMATION

4.1 POSSIBLE MODES OF FAILURE

The Chillingworth stopbank breach scenario differs from a dam failure scenario, as the stopbank is relatively low in height compared to flood levels and general downstream ground levels, and is constrained between high points either side, particularly by a terrace upstream. Most empirical breach development formulae are based on a known volume of water contained in a dam and therefore likely to go through the breach, but the volume through the breach in this case is not easily defined – it depends on the size of the breach, rate of development, and the shape of the river flow hydrograph.

We had no site-specific geotechnical information for the natural ground or the stopbank. However, from investigations for the M2PP expressway, and from the groundwater modelling in the wider area, we expect that there will be river gravel interspersed with lenses of silt and sand deposited on the river berms and backwaters during flood events in the Waikanae River. An 1870s map of this area shows a branch of the Waikanae flowing to the Waimeha Stream from further upstream near the SH1 bridge. While it does not show any river channels in the vicinity of Chillingworth, the groundwater data suggests there may be old river channels north of the Waikanae River, and the possibility that there is subsurface connection cannot be ruled out.

The likely mode of stopbank failure has not been investigated from a geotechnical point of view. It is possible that piping could occur, although we would expect the GWRC stopbank foundations to have been built to a reasonable standard to manage this risk, so the probability of this is likely to be low. We also expect the stopbank will contain gravel material that will reduce the rate of erosion and breach formation, particularly as the breach gradually opens up laterally.

The mode of stopbank failure affects the flowrate through the breach, with one of the following identified as the likely processes:

1. Vertical degradation - The breach is a progressive reduction in overflow level, taking an hour to reach the final invert level with full width erosion throughout the failure, decaying in elevation only over time.

2. Slip failure with width degradation - The failure is a relative quick collapse down to final invert level at the point where the stopbank is highest and there is a flow path available downstream. The breach then widens progressively upstream through lateral erosion of the stopbank structure until high land is reached.

Comment [GJL12]: I think we actually used a quick drop to invert, then progressive widening of the gap as the stopbank eroded along its length.

The second mode is more likely and this was selected for modelling of flowrate through the breach. The 1% AEP flood level in the vicinity of the Chillingworth stopbank is about 7.8 mRL (2115 scenario, 1% AEP) and the modelling has assumed a breach cutting down to 6.5 mRL, which is the general lowest natural ground level in this area and for some distance downstream.

Comment [PW13]: Some background: Originally (2011) I assumed a progressive widening and a progressive deepening, both occurring over 60min. Subsequently (2014), I modelled the 2nd case on Graham's advice – as his comment above says.

4.2 STOPBANK BREACH DIMESIONS AND DEVELOPMENT RATE

An important concept in understanding the relationship between Waikanae River flooding and the flow through the breach is how fast the stopbank embankment material can be eroded and mobilized. This is a function of the flood velocity, failure initiation mode, plasticity of the stopbank material and presence of stopbank weaknesses. Given the expected presence of gravel in the construction, and the fact that there is a relatively level outflow path for some distance downstream at about 6.5 mRL, we have assumed it is unlikely that the breach would cut down much below this level. This gives a water depth at the breach of about 1.3 m, and generally less.

Comment [PW14]: FYI, see attached topo. There is one small outflow path at ~6.5m RL high point, otherwise 6.5m not reached until some distance away from breach.

Initial modeling showed velocity through the breach site was less than 1 m/s (reflecting relatively flat hydraulic grade downstream), with the maximum velocity on the flood plain immediately upstream being up to 1.5 m/s. These would indicate a slow lateral erosion of the stopbank once initial localized failure had taken place.

Using empirical dam break formulae for breach formation based solely on water height (since upstream storage volume is not applicable here), and using formulae from USBR 1988 (Froelich and others) as referenced in Wahl (2004) we determined an initial failure down to 6.5 mRL at a width of 4.5 to 6 m. Considering all the data from Froelich (1990) as referenced in Wahl (2004), for sites where the water depth was small, the maximum likely average breach width would be 14 m. Our modelling has conservatively assumed a width of 60 m, reflecting the effective length of the stopbank between upstream and downstream higher ground.

Based on the empirical formulae in USBR 1988 (Froelich and others) and Von Thun and Gillette (1990) as referenced in Wahl (2004) it is likely that the initial failure would occur in a matter of minutes. The modeling has therefore assumed 5 minutes for the breach to reach 6.5 mRL. The further development out to full width has been assumed to take 60 minutes from initiation of the breach as indicated in Figure 5.

Comment [PW15]: Use the plot in the attached spreadsheet to illustrate.

Comment [GJL16]: Phil to confirm what was modelled

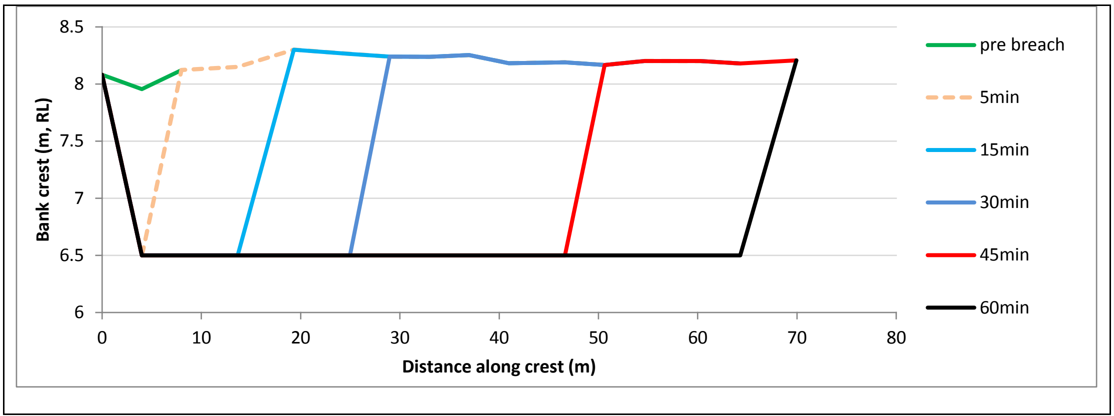

Figure 5: Breach development assumed

4.3 RELATIVE BREACH TIMING

The timing of the breach in the modelling to date is for it to commence in advance of the hydrograph peak, and to be fully developed before the peak flow in the river occurs (Figure 6).

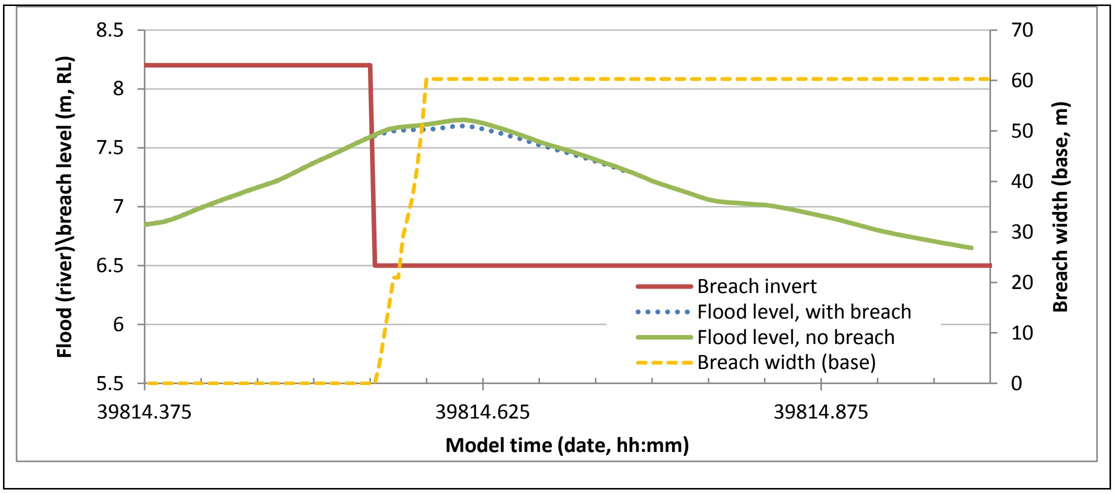

Figure 6: Timing of Breach Relative to River Level

The result of these events occurring in the timeline presented in Figure 6 is a conservative yet possible scenario of the stopbank failure.

Comment [PW17]: This from earlier work – where full depth took longer to develop.

5 MIKE FLOOD MODELLING

5.1 MODELLING METHODOLOGY

Modelling for the Project was carried out using the existing Waikanae flood model built in MIKE FLOOD software. MIKE FLOOD incorporates MIKE 21 (i.e. 2-D flow equations) and MIKE 11 (1-D flow equations), allowing them to be dynamically linked during a simulation. In the case of the Waikanae model, the 1-D components included the Waikanae River main channel, various drains in the area and culverts under the proposed expressway. The 2-D component covered the river berms and floodplain, and was represented by a grid at a resolution of 4 m. The 2-D model topography was derived from LiDAR data with the expressway design superimposed.

5.1.1 STOPBANK REPRESENTATION

The stopbank was represented in the 2-D model topography, and in design flow simulations the 2-D cells along the stopbank crest act as a barrier to flood passage through the Te Moana floodway.

Stopbank failure was modelled by a time-varying bathymetry in the 2-D component of the model. A pre-defined start time of the breach was provided, just over an hour before the river flood levels peaked adjacent to the breach site in the "no-breach" case. The model topography was varied over the next hour (as defined in Figure 5 in the case of the Chillingworth stopbank breach.)

5.2 BREACH SCENARIOS

Comment [PW18]: The scenarios we looked at were in fact breaches at different locations, in 2011. The Chillingworth breach was the one that had the most impact on the floodway design (ie the largest outflow).

Initially, breaches were modelled in three locations along the stopbank where breach flows could potentially flow into the Te Moana floodway: in addition to the Chillingworth stopbank, breaches in the Kauri-Puriri stopbank downstream and the Jim Cooke Memorial Park stopbank upstream were modelled. Of the three breaches, the Chillingworth breach led to the greatest floodway flows.

For the 1% AEP flood event, the predicted peak outflow from the Chillingworth breach was $20m^3/s$. Some minor attenuation was predicted as the outflow travelled downstream: the peak outflow about 600 m downstream of the breach, a little upstream of the expressway, was $18m^3/s$.

6 CONCLUSIONS

The 50 residential dwellings that lie in the Te Moana residual flow path have been protected by the Chillingworth stopbank from flood events as large as the 80 year flood in recent years. Failure of this stopbank could lead to $20m^3/s$ of flood waters flowing through the Te Moana floodway with a high risk of property damage and potentially, loss of life. The Expressway design developed has considered this event occurring and design has responded accordingly with a 50m wide floodway allowed for alongside Te Moana Road. This was provided by designing in additional spans to the SH1 overpass. The floodway may not protect the community from flooding in the event of stopbank failure. However, it will maintain a relief flow path in the event of the high volume floodwaters spilling into the floodway and preserve the level of flood resilience that the residents of the Te Moana floodway have enjoyed prior to the Expressways construction.

ACKNOWLEDGEMENTS

The following have provided valuable input in developing assessing and developing this project and paper:

- Kyle Christensen, PDP, Stormwater peer reviewer on the M2PP project. Reviewed the assessment and results of the modelling on behalf of Greater Wellington Regional Council.

- Craig Pitchford, NZTA owner's representative, for providing permission to submit this paper to the Stormwater Association.

- Iain Smith, Beca stormwater lead, for overseeing this assessment and leading the results of this assessment through detailed design of the Expressway.

REFERENCES

[1] GWRC (2010), *"Waikanae Floodplain Management Plan – 10 year Review"*, Flood Protection Division, Wellington.

[2] Wahl (2004), "*Uncertainty of Predictions of Embankment Dam Breach Parameters*", Journal of Hydraulic Engineering, Vol. 130, No. 5, May 1, 2004.